# EAST MARICOPA FLOODWAY RITTENHOUSE BASIN DESIGN PREDESIGN STUDY

Flood Control District of Maricopa County
"The Quality of Life People"


The EMF is a broad flat channel designed to intercept drainage along its 27-mile path and convey it safely to the Gila River.

**FINAL REPORT**
**DECEMBER 2001**

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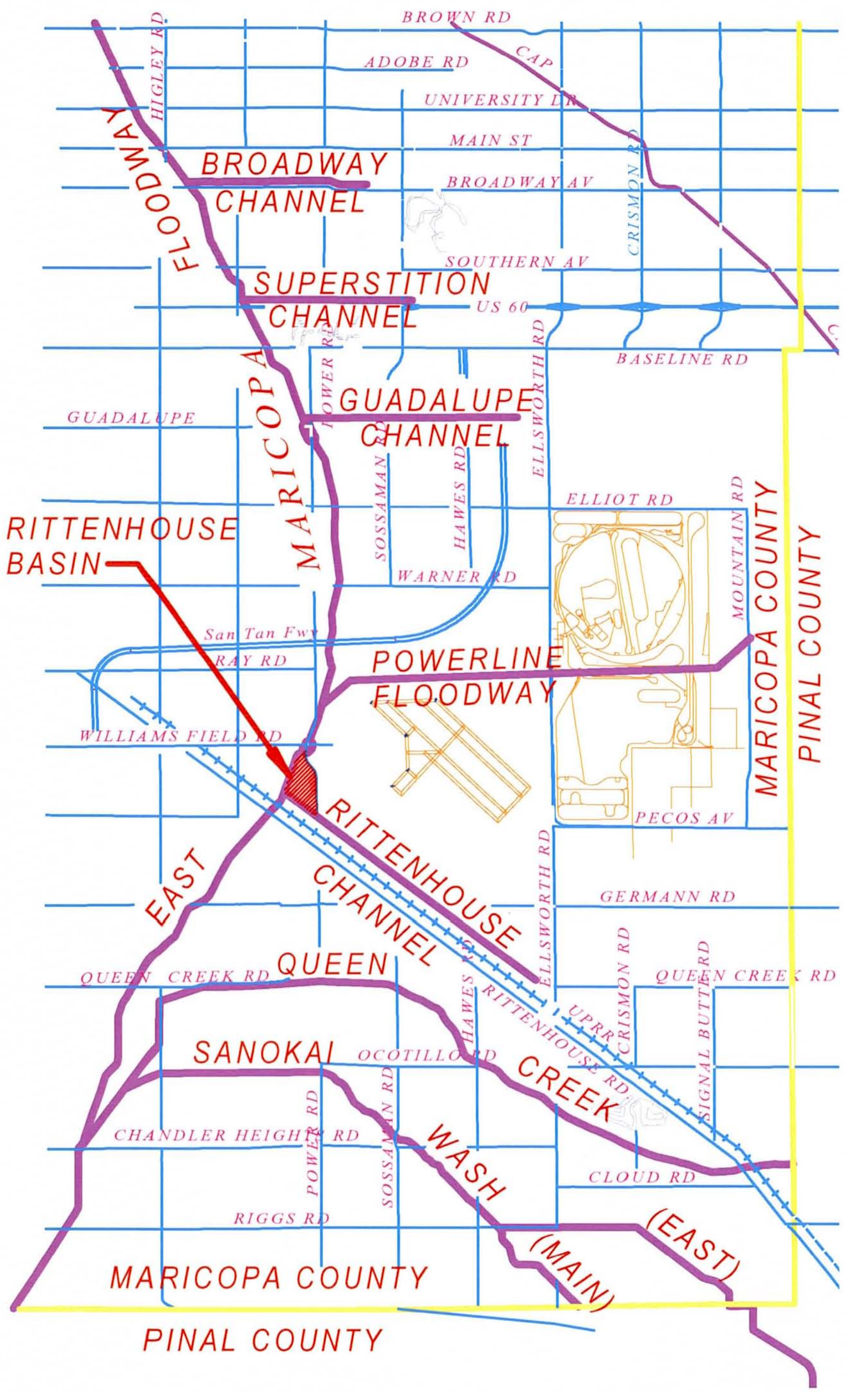



*Figure 1. Map of the Rittenhouse Basin area.*

## SUMMARY

The East Maricopa Floodway (EMF) is a regional flood control channel located in eastern Maricopa County and northern Pinal County. It serves as a primary outfall and flood conveyance for the City of Mesa, Town of Gilbert, Town of Queen Creek, Gila Indian Community, and for unincorporated areas of Maricopa and Pinal Counties, as shown in the map in Figure 1. The Flood Control District of Maricopa County (FCDMC) has determined the need for two large detention basins along the EMF to attenuate peak flood flows; one located north of Chandler Heights Road and the other located north of Rittenhouse Road.

In June 2001, the FCDMC contracted with Kirkham Michael & Associates, Inc. (KM) to initiate the preliminary design of the Chandler Heights and the Rittenhouse Road Detention Basins and ultimately develop final construction plans. This report presents proposals for the Rittenhouse Basin, to be located north of Rittenhouse Road. A companion report presents similar information for the Chandler Heights Basin. Further, this report presents the hydrologic and hydraulic analyses that were used to develop alternatives and evaluate them. All alternatives have as a goal meeting specific target flow limits in the EMF as established by FCDMC.

A proposed project for the Rittenhouse Basin is presented here. The project presented here provides a hydraulic solution to meet the project goals, but does not solve all of the design issues. Those will be handled during project design. Expected costs and benefits associated with the proposed plan are presented. This report also presents a discussion and evaluation of various trade-offs considered during analysis.

## PROJECT LOCATION

The Rittenhouse Detention Basin is located in eastern Maricopa County in the Town of Gilbert. The site is bounded by Power Road to the east, the Rittenhouse Channel to the southwest and the EMF to the west. Williams-Gateway Airport and the ASU East campus are located to the east of Power Road in the previous Williams Field Air Force Base. See Figure 1.

## PROJECT GOALS

Based on previous studies, FCDMC has identified a need to mitigate capacity deficiencies in the EMF and subsequently acquired land for the Rittenhouse Basin and the Chandler Heights Basin upon which to construct detention storage basins and associated works. The concept is to temporarily store a portion of the flood volume and release it after peak flows in the EMF have subsided. Diversion of a portion of the flow into storage will reduce the flow downstream, so that capacity limitations are eased. Stored flows should be released within 36 hours after the end of the storm event so that the storage basins do not become semi-permanent aquatic environments.

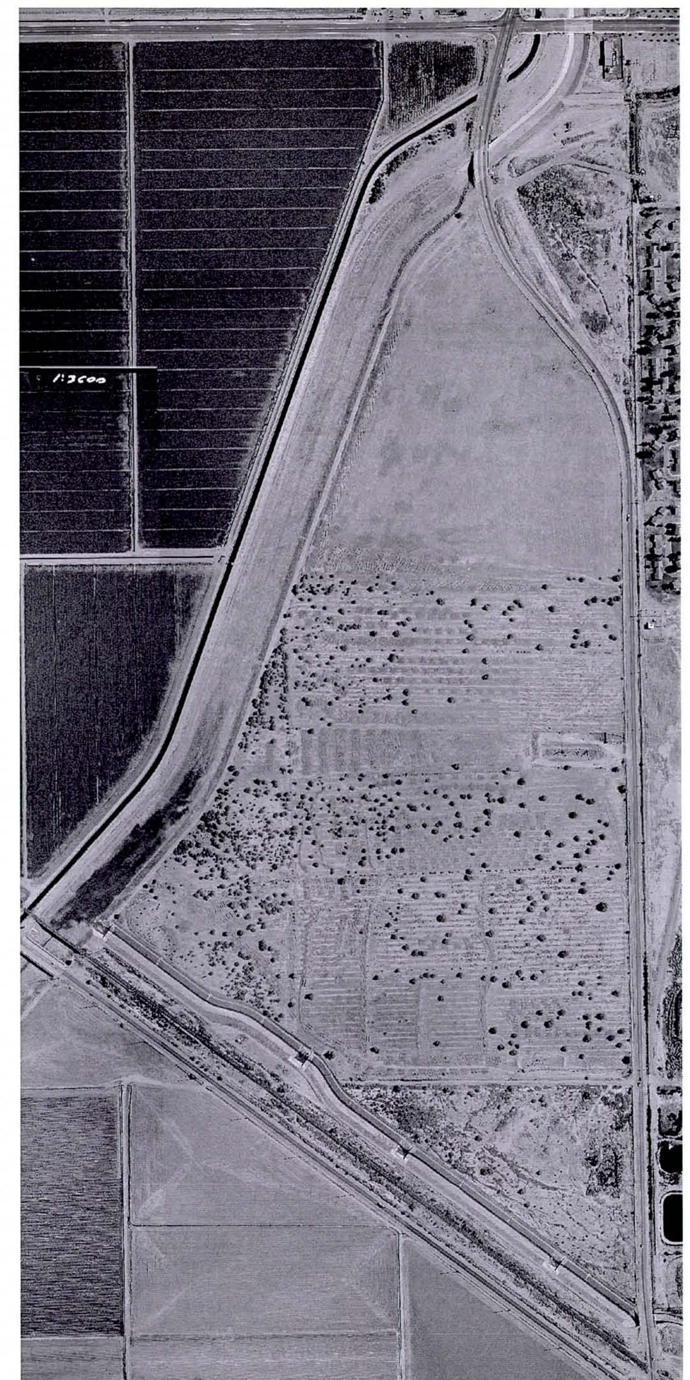


*Figure 2. This aerial photo of the Rittenhouse Basin area was taken in June 2001. It shows the area where the Basin will be located as well as surrounding features.*

### *Project design conditions*

This project is designed for the future (year 2020) development conditions. Peak flows for the future conditions are expected to be less than the existing conditions due to the construction of on-site detention and retention facilities required for future land development. In addition, the future conditions include planned capital improvement projects for flood control that should also reduce peak flow rates. Consequently, the proposed project facilities may experience higher flow rates than the design flow rates and may not attenuate flow to the design criteria until planned development and capital improvement projects are constructed.

### *Multi-use opportunities*

The site for the Rittenhouse Basin comprises approximately 150 acres—a sizable area for flood de-

*Figure 3. The East Maricopa Floodway is a 200-ft wide flood channel running through eastern Maricopa County. It's function is to intercept runoff and drainage from tributary systems and convey it to the Gila River in Pinal County. This view shows the EMF looking downstream from the Power Road bridge. The Rittenhouse Basin will be constructed to the left of the left embankment shown here.*

tention. By comparison, common detention basins associated with specific developments are usually on the order of 1-20 acres.

Because of the size of the detention basin, and the fact that the basin will only temporarily detain water after a significant rainfall event, the basin area is attractive for other compatible uses, such as outdoor recreational facilities. However, while the FCDMC favors multi-use and the design of the basin can provide for and take into consideration multi-use opportunities, the FCDMC will provide no funds for the design and construction of multi-use facilities. The principal focus of the FCDMC and this design effort is a functioning flood control facility. Multi-use facilities compatible with the flood control facility may be designed and constructed by others, so long as the FCDMC is in agreement and the multi-use facilities do not compromise the operation of the basin for flood control. In addition, with the proximity of Williams-Gateway Airport, any proposed multi-use facilities should limit permanent water features that would attract birds and possibly interfere with aircraft operations.

The Town of Gilbert has expressed an interest in the recreational development of the basin as a park, a trail system and/or golf course. The Town is currently undertaking preliminary studies to determine if the site is suitable for an 18-hole golf course at the Rittenhouse Basin site. This project's proposed basin design makes accommodations for a golf course at the basin site by leaving a raised area along Power Road for clubhouse facilities and parking. However, the basin is not contoured for a golf course and any proposed changes to the basin shape and configuration to accommodate a golf course will only be agreeable if it can be shown that the changes will not adversely impact the operation of the basin for flood control.

The basin site could also be utilized as part of the San Tan Regional Trail. The San Tan Regional Trail is a proposed recreational trail system running along the EMF from north Mesa to the San Tan Mountains. The proposed basin design could easily accommodate the proposed trail.

## PREVIOUS STUDIES

A number of previous studies have been conducted in the study area and serve as a basis, either directly or indirectly, for the development of hydrology and hydraulic models utilized in this study. In addition, several studies provide background information and show the progression of development of alternatives that have led to the concept of the Chandler Heights and Rittenhouse Detention Basins.

**East Maricopa Floodway Capacity Mitigation and Multi-Use Corridor Study (Collins-Pina, 2000).** This concept study evaluated alternatives and recommended a preferred alternative to improve the flood control capabilities of the EMF and identified multi-use opportunities that would be consistent with the proposed flood control recommendations. The study criteria and proposed alternatives were based upon existing conditions and future conditions/development of the watershed both of which included selected planned flood control facilities as identified by FCDMC. The study recommended construction of two detention basins along the EMF and one along the Powerline Floodway. The two basins along the EMF; one in the vicinity of Rittenhouse Road and the other in the vicinity of Chandler Heights Road are the focus of this design effort and the hydrologic/hydraulic models in this report provided the basis for the preliminary design of Rittenhouse Basin in this project.

*Figure 4. Upstream from the Power Road bridge the EMF has a segment with concrete bottom and sides.*

**East Maricopa Floodway Capacity Mitigation Study Report, (Huitt-Zollars, 2000).** This concept study developed and evaluated alternatives to resolve EMF conveyance deficiencies and developed concept plans for a preferred alternative. The study recommended a series of five EMF detention basins along with isolated channel improvements to the EMF to resolve EMF deficiencies. The study criteria and preferred alternative were based upon existing watershed conditions. This study also compiled and developed hydrology models for the EMF watershed (to Hunt Highway) for the future watershed conditions from previous hydrologic study models and updated hydrology developed during the study. The future conditions models served as the basis for the EMF Capacity Mitigation and Multi-Use Corridor Study by Collins-Pina.

**Queen Creek/Sanokai Wash Hydraulic Master Plan (HMP) (Huitt-Zollars, 2000).** This concept study recommended and developed conceptual plans for flood control improvements along Queen Creek and Sanokai Wash that would serve as guidelines for use by local municipalities in planning for future development. The study recommended channel improvements along Queen Creek Wash to incise the channel and minimize sediment transport and lateral migration. The study also recommended channel improvements to Main Branch and East Branch of Sanokai Wash to better channel overland flow. In addition, three detention basins were proposed along Sanokai Wash to attenuate peak flood flows. Among the alternatives presented were options of splitting Queen Creek and Sanokai Wash and rerouting them to the EMF. The proposed alternatives were based upon the future conditions development of the watershed as defined by municipal General Plans and existing zoning boundaries. Future conditions hydrology from this study was incorporated into the EMF Capacity Mitigation Study by Huitt-Zollars.

*Figure 5. Downstream from the transition below the Power Road bridge shown here, the EMF has a earthen bottom and slopes.*

**Sanokai Wash Floodplain Delineation Study (FDS) (Entellus, 1999).** This study developed the existing condition hydrology for the Sanokai Wash watershed and delineated the Sanokai Wash floodplain between Higley Road and Riggs Road. The hydrology and hydraulic models from this study were used as a basis to develop a future conditions model for the Sanokai Wash watershed in the Queen Creek/Sanokai Wash HMP.

**East Maricopa Floodway Capacity Assessment Study (HNTB, 1999).** This study assessed the conveyance capacity of the entire EMF for the existing conditions 100-yr discharge, the future conditions 100-yr discharge, and the Soil Conservation Service (SCS) EMF design discharge. The study also delineated the extent of flooding adjacent to the EMF for all three conditions. The study also determined the conveyance capacity of the EMF under bank-full conditions.

**East Mesa Area Drainage Master Plan (ADMP) (FCDMC and Dibble and Associates, 1998).** This study determined the existing and future conditions hydrology for the East Mesa area for planning purposes. It identified drainage problems and proposed improvements to provide flood protection in the East Mesa Area.

**Queen Creek Area Drainage Master Study (Wood and Associates, 1991).** This study identified stormwater problems in the Queen Creek and Sanokai Wash and provided a master drainage plan for resolution of them. The existing conditions hydrologic model from this study was updated and utilized as part of the EMF Capacity Mitigation Study and the Queen Creek/Sanokai Wash HMP.

# HYDROLOGY AND HYDRAULICS

## INTRODUCTION

FCDMC provided the hydrology models that serve as the base hydrology for the design of the study basins. The hydrology is developed in a series of five HEC-1 models that represent the contributing watershed to the EMF from the Princess Basin to the Maricopa/Pinal County Line along the Hunt Highway alignment. These models were the product of several previous studies and have been frequently modified and revised during the course of these studies. The hydrology is for future conditions, therefore it includes flood control, retention, and drainage features that FCMDC envisions being constructed upon the full development of the EMF watershed.

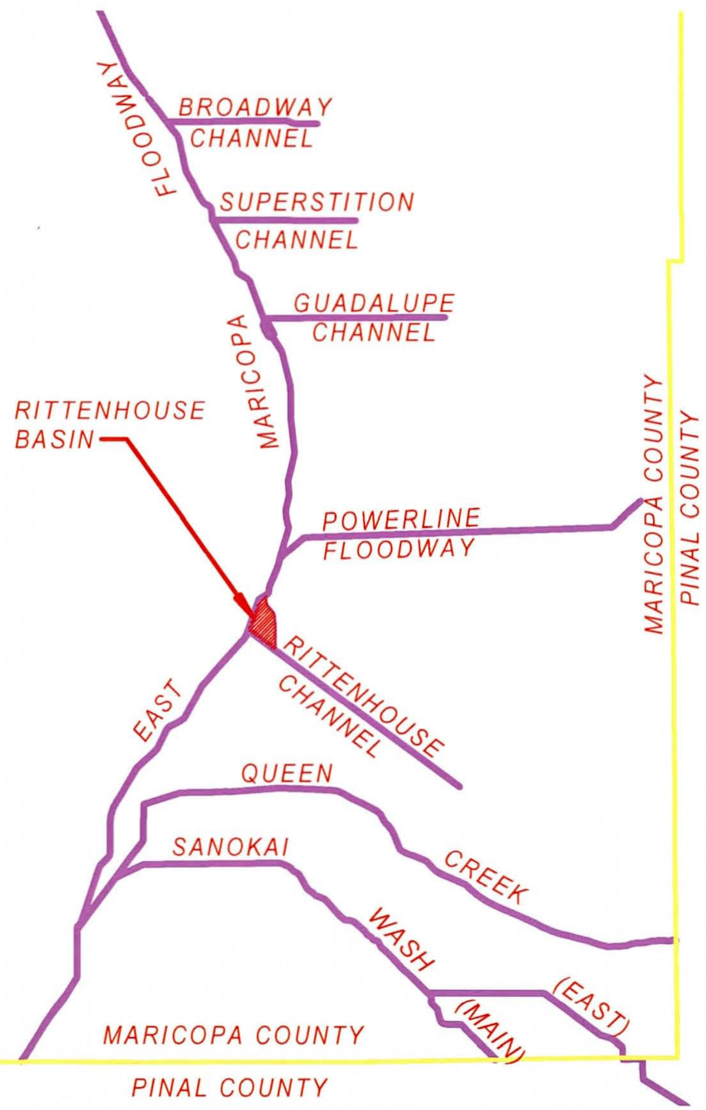



*Figure 6. Map showing tributaries to the EMF in the Rittenhouse Basin area.*

## PROJECT WATERSHED

The EMF serves as a primary outfall and flood conveyance for the City of Mesa, City of Chandler, City of Apache Junction, Town of Gilbert, Town of Queen Creek, Gila Indian Community and for unincorporated areas of Maricopa and Pinal County. It intercepts runoff from three watersheds: Buckhorn-Mesa, Apache Junction-Gilbert, and Williams-Chandler.

## SIGNIFICANT STRUCTURES AND FEATURES

The EMF channel begins at the Princess Basin (north of Brown Rd) and flows southerly approximately 27 miles before discharging into the Gila River. The floodway is mostly constructed as a compacted earthen trapezoidal channel, ranging from 150 to 300 feet in width and 8 to 12 feet in depth. A stretch of approximately one mile in length located along Williams-Gateway Airport is concrete lined, as is another approximately half-mile section of the floodway located in Pinal County.

Six major drainage channels discharge into the EMF: the Broadway Channel, the Superstition Freeway Channel, the Guadalupe Channel, the Powerline Floodway, the Rittenhouse Channel, and Queen Creek/Sanokai Wash (Figure 6).

**Broadway Channel.** The Broadway Channel collects and conveys drainage from Mesa and unincorporated Maricopa County. It discharges into the EMF south of Broadway Road.

**Superstition Freeway Channel.** The Superstition Freeway Channel collects and conveys drainage from Mesa and unincorporated Maricopa County. It discharges into the EMF north of the Superstition Freeway (US 60).

**Guadalupe Channel.** The Guadalupe Channel collects and conveys drainage from Mesa and unincorporated Maricopa County. It discharges into the EMF south of Guadalupe Road.

**Powerline Floodway.** The Powerline Floodway collects and conveys drainage from Mesa, Williams Gateway Airport, and unincorporated Maricopa County and Pinal County. It discharges into the EMF near Ray Road.

**Rittenhouse Road Channel.** The Rittenhouse Road Channel runs northwesterly along the north side of the SPRR and Rittenhouse Road. It collects and conveys drainage from Mesa, Gilbert, Queen Creek, Williams Gateway Airport, and unincorporated Maricopa and Pinal County. It discharges into the EMF north of Rittenhouse Road.

**Queen Creek/Sanokai Wash.**
Queen Creek and Sanokai Wash are ephemeral streams and are dry except after significant rainfall events. They are major conveyances for a large watershed that includes the Town of Queen Creek and extend into northern Pinal County.

Queen Creek is a well-defined natural channel that originates in the Superstition Mountains in northern Pinal County and flows southwesterly, passing through the Whitlow Reservoir and the Sanoqui Detention Dike before continuing westerly through Maricopa County and discharging into the EMF just north of Chandler Heights Road.

Sanokai Wash consists of two branches, the Main Branch and the East Branch. Both branches originate in the Santan Mountains in Northern Pinal County and flow northwesterly before joining in the proximity of Riggs Road and Hawes Road. The combined Sanokai Wash continues northwesterly through the Town of Queen Creek before draining towards Queen Creek Wash in a poorly defined manner, approximately along the Ocotillo Road alignment. After their confluence, Queen Creek/Sanokai Wash pass through a sedimentation basin prior to discharging into the EMF.

Other significant structures that affect functions in the EMF include:

**Central Arizona Project (CAP) Aqueduct.** A CAP aqueduct component (the Salt-Gila Aqueduct) interrupts the natural westerly drainage of the watershed. Pipe overchutes carry drainage

*Figure 7. The Rittenhouse Channel conveys drainage from the area around the Williams-Gateway Airport into the EMF just north of Rittenhouse Road. This view looks upstream from near the confluence with the EMF. The SPRR line runs parallel and to the right, and Rittenhouse Road is to the right of the railroad.*

past the aqueduct at select locations. Discharge from a large CAP overchute (4-72" pipes) conveys flow released from the Sanoqui Detention Dike across the CAP and continues as Queen Creek Wash.

**Bureau of Reclamation Sanoqui Detention Dike.** The Sanoqui Detention Dike is a flood retarding structure located in Pinal County, east of the CAP aqueduct. The structure collects, detains and releases flow into Queen Creek from a watershed upstream of the CAP aqueduct.

**San Tan Freeway.** Within the watershed, the alignment of the proposed San Tan Freeway runs easterly along the Knox Road alignment before turning north approximately between Hawes Road and Ellsworth Road. The proposed freeway will interrupt the natural westerly drainage of the watershed and include a number of bridges, culverts, drainage channels and detention basins. The freeway is still in the planning and design stage and drainage facilities have not been completely defined or finalized.

**Southern Pacific Railroad Line (SPRR).** A SPRR line runs northwesterly across eastern Maricopa County along the north side of Rittenhouse Road. The SPRR line is on a raised embankment that interrupts the natural westerly flow of the watershed. A few culverts convey flow across the SPRR in select locations.

**Open Aggregate Mining Pits.** Active and abandoned aggregate mining pits located adjacent to or within Queen Creek Wash, continue to alter flow paths, detain flow and contribute to the sediment load of Queen Creek Wash. The pits are located primarily in Pinal County, however, one pit is lies within Maricopa County, just west of the county line.

## BASE HYDROLOGY

FCDMC provided the 100-yr, 24-hr future conditions hydrology models that serves as the base hydrology for this project. Rittenhouse and Chandler Heights Basins and their associated components were not included in the base hydrology. The base hydrology reflects the watershed conditions prior to the development of the Rittenhouse and Chandler Heights Basins.

All the models were constructed according to the methodologies presented in the "Drainage Design Manual for Maricopa County, Volume I, Hydrology".

### *Changes to the base hydrology*

After receiving the base hydrology models from FCDMC, several revisions to the models were made to include more accurate and additional hydrologic information. These are described in the sections that follow.

#### Central Arizona Project (CAP) Aqueduct Overchutes

The Salt-Gila CAP Aqueduct interrupts natural westerly drainage of the EMF watershed. At several locations, pipe overchutes carry runoff that collects behind protective dikes across the aqueduct and contributes to flow in the EMF. In the hydrology models, these overchutes are modeled using hard-coded hydrographs that are generally based upon rough estimates of the overchute pipe capacities. Two overchute locations (CAP1A and CAP1B) were reevaluated by FCDMC and subsequently revised. For a larger overchute on Queen Creek Wash, and one that has a significant impact on the design of the Chandler Heights Basin (HY337), FCDMC initiated a study to better determine the resulting hydrology. The hydrology from this study was then incorporated into the base hydrology. (Figure 8)

*Figure 9. The Rittenhouse Channel, coming from the left, at the point of confluence with the EMF. The EMF is crossed by bridges for the SPRR and Rittenhouse Road. The concrete sill helps to protect the transition between Rittenhouse Channel and the EMF from erosion due to channel slope differences.*

## HYDRAULICS

### *Introduction*

FCDMC provided the HEC-RAS hydraulic models for the EMF, Rittenhouse Road Channel, Queen Creek, and Sanokai Wash. These models were developed in previous studies and include selected features that are being proposed for future construction but do not currently exist. These features include low flow channels, channel widening/ incising, changes in channel roughness (for aesthetic landscaping), and other items FCDMC has decided will be realized in future capital improvement projects.

### *FEQ vs. HEC-RAS 3.0 unsteady state model*

Rather than using peak flows only, all hydraulic modeling considered time changes in flows for the system elements. This allowed accounting for the diversion of flow into the storage basin and the return of the stored volume back into the EMF. This required using an unsteady state model, such as HEC-RAS 3.0 or FEQ.

The HEC-RAS model was developed by the US Army Corps of Engineers. It replaces the earlier HEC-2 model. HEC-RAS can provide either steady or unsteady modeling.

FEQ (Full Equations Model) was developed by the US Geological Survey and is an unsteady state model. FEQ runs in a DOS environment and has similar capabilities to HEC-RAS for unsteady flow.

We performed pilot studies to compare how the two models functioned, how easy it was to prepare data for and run the models, and to compare the results of each. As reported in Appendix A - Hydrology and Hydraulics, we determined that the two modeling systems gave similar results. Together with FCDMC we then decided to do the work in this project using the HEC-RAS 3.0 model because HEC-RAS has easier data preparation steps and requires much less time in data handling. Both models were able to handle sideweirs and storage, though each used a different approach. For a more detailed discussion of the two models refer to Appendix A .

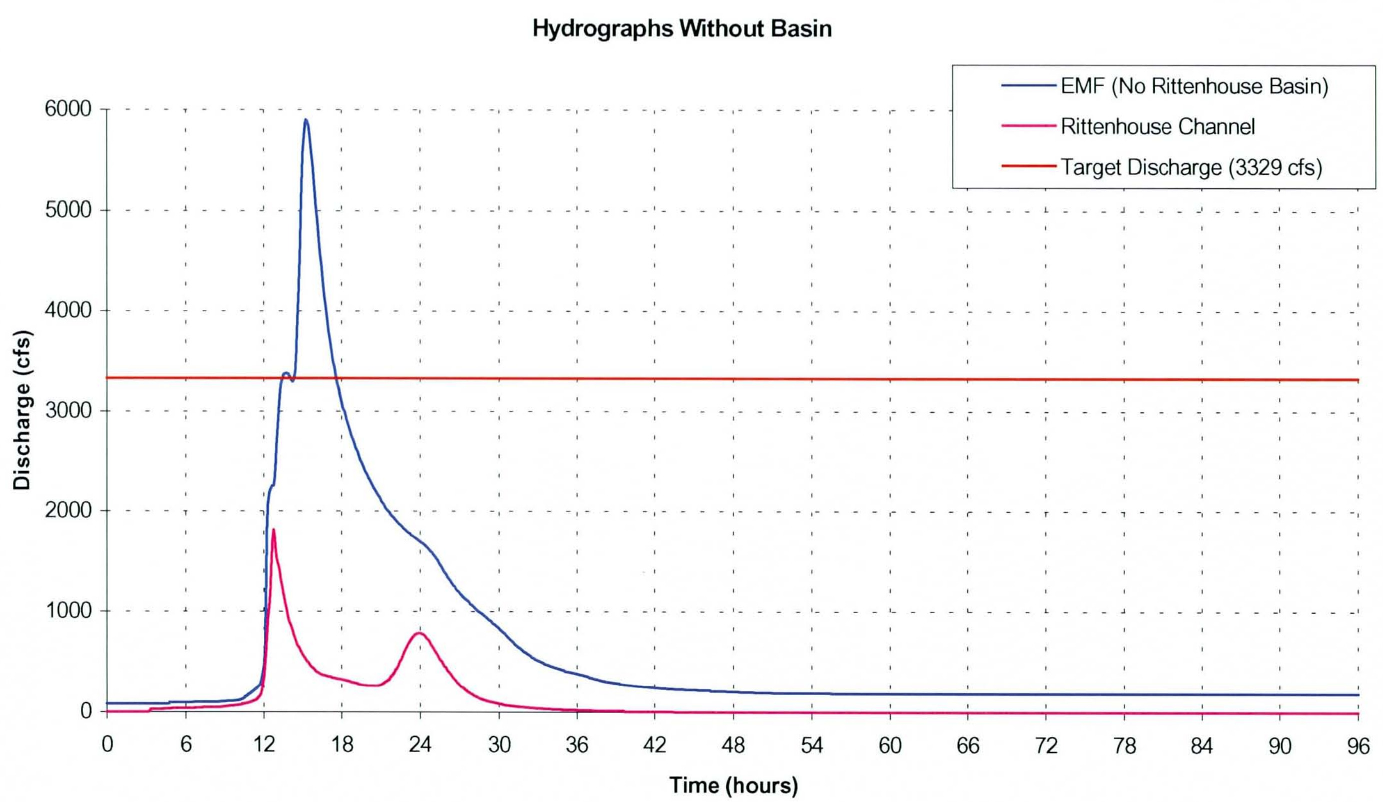



*Figure 8. These hydrographs show conditions without the Rittenhouse Basin in place. Notice that the peak flow in the EMF exceeds the target value of 3329 cfs by a considerable amount. Also notice that the first peak for the Rittenhouse Channel has subsided before the peak in the EMF.*

*Figure 10. The Rittenhouse Channel has a series of drop structures in the segment upstream from the confluence with the EMF. These allow the channel to have a mild slope so that bed degradation is minimized.*

# DESIGN CRITERIA

## DESIGN OBJECTIVES

FCDMC set forth several design objectives that we are to meet:

- Minimize the volume of the basins
- Optimize the confluence of Queen Creek, Sanokai Wash, and Rittenhouse Channel to minimize the volume of the basins
- Provide for multi-use opportunities for the basins to include recharge, recreation, and mitigation
- Maximize the basin configuration to use a gravity outlet
- Balance basin volume versus channel capacity (i.e.: inflows to the EMF) to minimize basin size
- Minimize Operations and Maintenance (O&M) requirements for sediment removal.

### *Optimizing basin volumes*

Optimizing the Basin volume is a balance between providing sufficient detention storage and flow attenuation to meet the design objectives while also minimizing project costs, land requirements and earthwork. Therefore, the basin storage volume will be sized to provide only the detention storage necessary to meet the target flow downstream of the basin and will not be "oversized" to provide additional storage or attenuation.

### *Provide for multiuse potential*

In the design objective to provide for multiuse, FCDMC directed us to coordinate with the Town of Gilbert for the possible future use of the Basin area for recreational purposes. Our design must allow, to the extent possible, for the Town to follow up with further development of the site. Such added uses cannot be inconsistent with the primary purpose of designing a flood control facility, however.

Details of coordination with the Town of Gilbert are contained in Appendix F—Multiuse Planning, and the results are shown on the plans and described in the Alternatives Development section of this report.

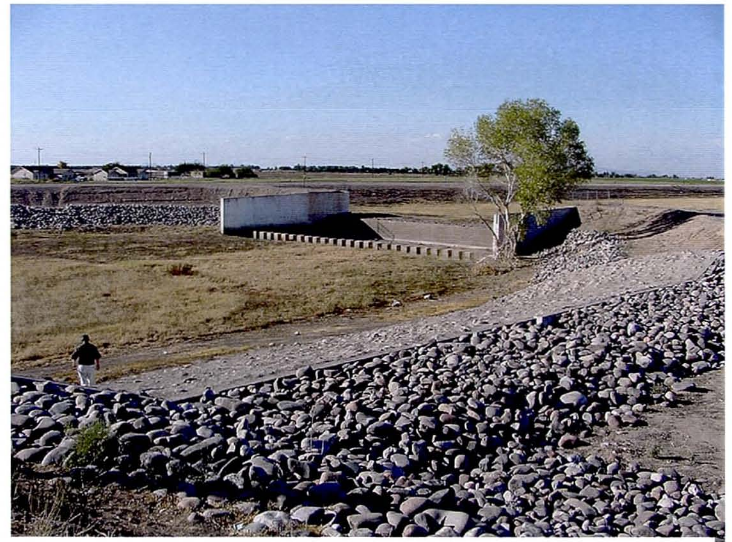


*Figure 11. A drop structure in the EMF. This one is located just upstream from the Chandler Heights Road bridge. The EMF flows from the upper right to the lower left, and the riprapped zone in the lower right is used to convey local runoff into the EMF.*

## DESIGN CRITERIA

Several categories of design criteria were used in the alternatives development and analysis and are explained below.

### *Event size and frequency*

For the design of the Chandler Heights and Rittenhouse Basins, FCDMC established the 100-yr, 24-hr future watershed conditions as the design hydrology. This hydrologic model includes selected capital improvement projects for flood control, retention and drainage that FCDMC envisions being constructed upon the full development of the watershed but may not currently exist.

### *Freeboard and water levels in detention basins*

Freeboard is the distance above the water surface level in an impoundment to the top of the containing embankment or structure. Freeboard is provided as a factor of safety against overtopping that may reduce the structure or stability of the embankment. Situations for which freeboard compensates include:

- Waves being generated to a height greater than the still water surface
- Settlement of the embankment more than expected
- Flood levels higher than design
- Malfunction of outlet works

FCDMC does not have specific criteria for freeboard in detention basins, but instead refers the designer to the standards in the community in which the basin is to be built. The Town of Gilbert's freeboard criteria call for one foot of freeboard. That criterion is fine for the typical smaller detention basins associated with residential and commercial developments. However, we do not feel that this is sufficient freeboard for a detention basin of this size, and sought other sources.

We decided to use the Arizona Department of Transportation (ADOT) unofficial policy. ADOT uses the following rule-of-thumb for the design of detention basins:

- Prefer that water level remain at least one foot below surrounding grades
- If water level must be higher than one foot below surrounding grades, then provide three feet of freeboard

### *Provisions for wave action*

The freeboard criteria above typically apply for protection of the embankment against overtopping by providing a factor of safety above the highest expected water levels. Another concern with larger basins is freeboard for protection against erosion and overtopping due to wave action. However, the two freeboard components do not need to be additive, and the more restrictive can apply for both.

One commonly-used reference, "Design of Small Dams" (DOSD) (US Dept of the Interior, Bureau of Reclamation, 1973) provides some guidance on the freeboard required for wave action. Since the height of waves generated by winds depends on factors such as wind velocity, wind duration, the geometry of the basin shape, and the distance across the water surface over which the wind can act. This last factor is called fetch.

Not just the height of waves is important. When waves approach a sloped face the water will run up the face to a height greater than the wave height. This is called uprush. Uprush is influenced by factors such as the depth of the water below the surface, the slope of the face being approached, and the roughness of the material on the face. DOSD reports that an uprush factor of 1.5 times the wave height can be experienced with dumped rock riprap, while uprush against smooth concrete can be considerably greater.

An American Society of Civil Engineers report gave a table of wave height as a function of fetch and wind velocity. The fetch for the Rittenhouse Basin is less than one mile, and with a wind velocity of 50 mph the anticipated wave height is 2.7 feet. With a wind velocity of 75 mph the wave height is expected to increase to 3.0 feet.

DOSD recommends "that the freeboard be sufficient to prevent overtopping of the dam due to wave rideup equal to 1.5 times the height of the wave".

For the Rittenhouse Basin we have two situations for which freeboard must be provided:

- Where the embankment rises to natural ground on the other side
- Where the embankment separates the basin from an adjacent channel

These two are illustrated in Figures 12 and 13.

In the first case we are not concerned with an embankment being weakened by overtopping, since there would be no breakout on the other side. In the second case we are concerned about overtopping but to a minimal extent since a breakout would result in the impounded water being delivered to the

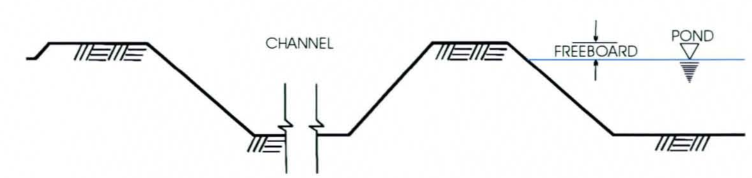



*Figure 12. Freeboard can also be in the lower end of the range where a flood control channel is on the other side of a pond embankment since overtopping is less likely to affect surrounding lands.*

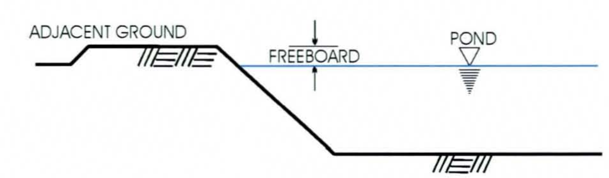



*Figure 13. With adjacent ground levels at the same elevation as the pond embankment top, there is little concern over the effects of overtopping, so freeboard can be less.*

adjacent flood control channel. Therefore, we can use minimal factors of safety for freeboard in the Basin. Again referring to DOSD, for a fetch of less than one mile they recommend a "Normal Freeboard" of 4 feet and a "Minimum Freeboard" of 3 feet. Those freeboard heights include both the effect of protection against overtopping as well as wave action, and "Normal Freeboard" applies for wind speeds of 100 mph and "Minimum Freeboard" for speeds of 50 mph. However, since we are not concerned as much with overtopping we propose to use a freeboard criteria of 3 feet in Rittenhouse Basin to compensate for all factors. The actual freeboard used in various parts of the basin are shown on the concept plan and typical sections in the Proposed Plan section of this report.

### Freeboard in channels

FCDMC has set as a goal to attenuate flow sufficiently that the EMF will meet NRCS freeboard design standards from Rittenhouse Road to the Maricopa County Line. These standards require channel freeboard to be:

- Minimum of 1-foot of clearance between the low chords of a bridge or top of culvert and the water surface elevation
- Freeboard $\geq 0.20\ (Y + V^2/2g)$ for subcritical flow
- Freeboard $\geq 0.25d$ for supercritical flow

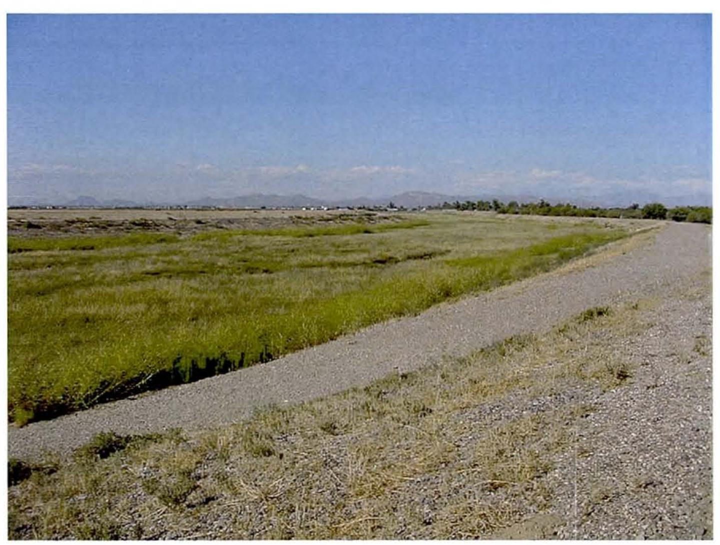


*Figure 14. This view looks upstream in the EMF from just below the bend next to the Rittenhouse Basin. The EMF is a broad, flat, channel and the bottom is grassed for most of its length. A maintenance access road is shown entering the channel bottom area from the east embankment.*

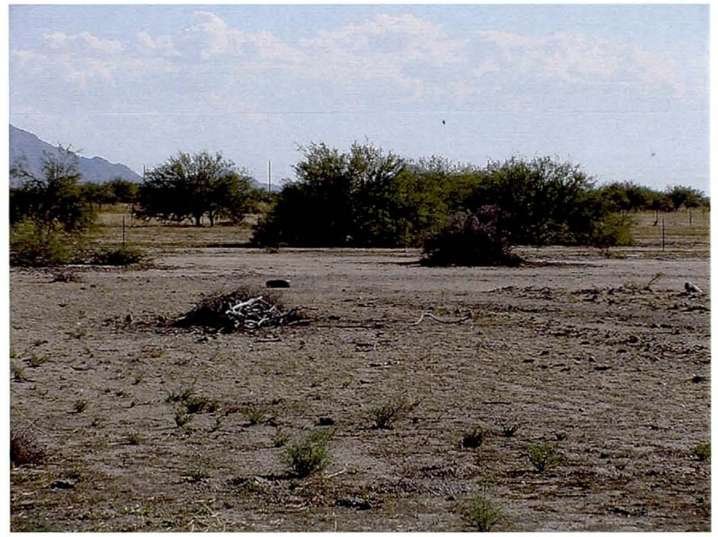


*Figure 15. The existing ground surface in the area where the Rittenhouse Basin will be constructed has been used for cattle grazing in the past. The ground surface is relatively flat. Many mesquite trees grew naturally in the area and are being sold by the District as a resource for removal prior to construction.*

Where: Y = channel flow depth (ft)
V = channel velocity (ft/s)
g = acceleration due to gravity (ft/s$^2$)
d = flow depth (ft)

### Target peak flow rates

FCDMC determined that the freeboard criterion for the EMF can be met by meeting limits on instantaneous peak flow rates at three locations along the EMF:

- 3329 cfs downstream of the EMF-Rittenhouse Channel confluence
- 5667 cfs downstream of the EMF-Queen Creek/Sanokai Wash confluence and north of the Chandler Heights Road bridge
- 8100 cfs just south of the Maricopa County Line.

Our work is directed toward meeting those target flow rates.

### Sediment capture

Because the Rittenhouse Basin is an off-line basin to the EMF, we did not see a need for sediment capture. We are only dealing with flows being carried by the EMF, and there is or will be a large number of smaller detention basins distributed throughout the drainage basin.

### Basin emptying

FCDMC directed that the Rittenhouse Basin be drained within 36 hours from the end of the storm event, if possible. This criterion is intended to avoid standing water problems that might contribute to breeding insects and that would attract birds to nest. Birds are a particular concern in this area due to the proximity of Williams-Gateway Airport.

### Landscaping and erosion control

FCDMC wants the Basin area to have perimeter landscaping both to soften the impact of the Basin and its facilities on the visual environment and to help provide erosion control of slopes. The details of planting criteria are described in Appendix F—Landscaping and Erosion Control, and are shown on the plans.

FCDMC did not want the flood control project to provide elements of landscaping directly required for any future multiuse of the facility, though any landscaping included in the plan should be designed with other future uses in mind so that they would not likely have to be changed for the future uses.

### Basin side slopes

Basin side slopes should be kept to 4:1 (H:V) or flatter, according to the information in Appendix B—Geotechnical Engineering Report. This slope ratio will maintain stable slopes that won't likely be affected by changing water depths in the basin. Flatter slopes may be desirable for landscaping or aesthetic purposes, and FCDMC prefers 5:1 slopes if they are to be grassed or vegetated.

Several maintenance access roads will be constructed into the basin from the upper ground surface. These roads will have flatter slopes and will serve vehicular traffic. Slopes at 5:1 or flatter are considered adequate for mowing. Refer to Appendix F—Landscaping and Erosion Control for more details about desired slopes and their uses.

*Figure 16. This view looks south along Power Road. The Rittenhouse Basin will be constructed to the right in this picture and the bottom will be approximately 15 feet below existing ground. The Town of Gilbert is considering constructing a golf course in the new basin bottom, and the club house and facilities are proposed to be located just off Power Road to the right in this picture.*

# PROPOSED PROJECT

## INTRODUCTION

While FCMDC had decided on a conceptual alternative that would attenuate flow in the EMF using detention basins near Chandler Heights and Rittenhouse Road, no design details had been explored. This report provides details about the more detailed study and analysis of the necessary hydraulic structures, storage volume requirements and availability and the hydraulic/hydrologic impact on the EMF.

## ALTERNATIVES DEVELOPMENT

In developing approaches to meet the target peak flows referred to under Design Criteria, we used both the hydrologic and hydraulic models in the alternatives development process.

### *Initial investigations*

Prior to developing alternatives for the Rittenhouse Basin, we reviewed EMF hydrology to identify critical hydrologic components, to better understand their impact on the EMF. We identified four significant hydrologic components that could be altered through the design and construction of the proposed Chandler Heights and Rittenhouse Basins in an effort to attenuate flow in the EMF:

- the EMF itself
- Rittenhouse Channel
- Queen Creek
- Sanokai Wash.

We considered various combinations of storage between the two basin sites, considering the largest potential storage volume that could be developed at each site. Because the tributary areas are fairly large, and because of the travel time between the two basins, we also considered the relative timing of hydrograph peaks for the four inflows.

We used the hydrology models to determine the effectiveness and efficiency of different scenarios. Alternatives included different basin designs (inline and offline), various basin combinations (Rittenhouse Basin and/or Chandler Heights Basin), and different combinations of detention scenarios (detaining flow from the EMF and/or Rittenhouse Channel, detaining flow from the EMF and/or Queen Creek and/or Sanokai Wash)

From these initial analyses, we made the following conclusions relative to Rittenhouse Basin:

- Neither a Rittenhouse Basin nor a Chandler Heights Basin alone can meet the target flows in the EMF. Both basins, therefore, must be utilized.
- Storing flows from the Rittenhouse Channel is relatively ineffective in reducing peak flow in the EMF because the peak occurs earlier than the inflow from above the confluence in the EMF itself. The Rittenhouse Basin, therefore, should be dedicated to detaining flow directly from the EMF. Hydrographs showing those relationships were presented in the chapter on Hydrology and Hydraulics.

We also found that using Rittenhouse Basin to meet target flows in the EMF reduced the flow in the EMF downstream at Chandler Heights Basin enough so that some of the flows from Queen Creek and Sanokai Wash could be by-passed directly into the EMF without having to use detention storage. The details for Chandler Heights Basin are given in a separate report under this project.

### *Reaching target flow goals*

We analyzed many different approaches to reducing peak flows in the EMF by using the hydrology models. To do this, we tried various combinations of diversion of flows from the EMF and its tributaries Rittenhouse Channel, Queen Creek Wash, and Sanokai Wash to determine what might be effective in meeting the target flows in the EMF. We then applied those results using hydraulic modeling to determine the design parameters for diversion and storage.

### *Setting downstream design conditions*

Once the selected scheme was refined using the hydraulic analyses described below, we had HEC-RAS output hydrographs for the resulting conditions downstream of the confluence of the Rittenhouse Channel. We then routed this hydrograph downstream to the Chandler Heights Basin area to provide boundary conditions for the design of that basin. See the separate report for the Chandler Heights Basin for those details.

### *Results of hydrologic modeling*

Our hydrologic modeling produced several conclusions important to our design:

- Diverting flows from Rittenhouse Channel would not be effective in meeting target flows because the peak in Rittenhouse Channel arrives at the confluence with the EMF earlier than the peak from upstream in the EMF itself.
- Target flows can be met by diverting flows from the EMF at Rittenhouse Basin.
- Diverting flows from Queen Creek Wash and Sanokai Wash will be more effective in meeting the target flows in the EMF at Chandler Heights than would be diverting flows from the EMF itself.

The results of hydrologic modeling for the first two conclusions are demonstrated in the flow vs. time hydrographs shown in Figure 17. The latter two conclusions are covered in the companion report on Chandler Heights Basin.

### *Development of design hydraulic models*

For the unsteady flow analysis done to evaluate and size basin components, we used a limited set of data rather than running the model for the whole basin. The boundary conditions and components included:

- A segment of the EMF channel from downstream of the Power Road bridge to downstream from the Rittenhouse Road bridge
- A detention basin along the EMF
- A sideweir along the east bank of the EMF that is hydraulically connected with the detention basin
- A short reach of Rittenhouse Channel upstream from the confluence with the EMF

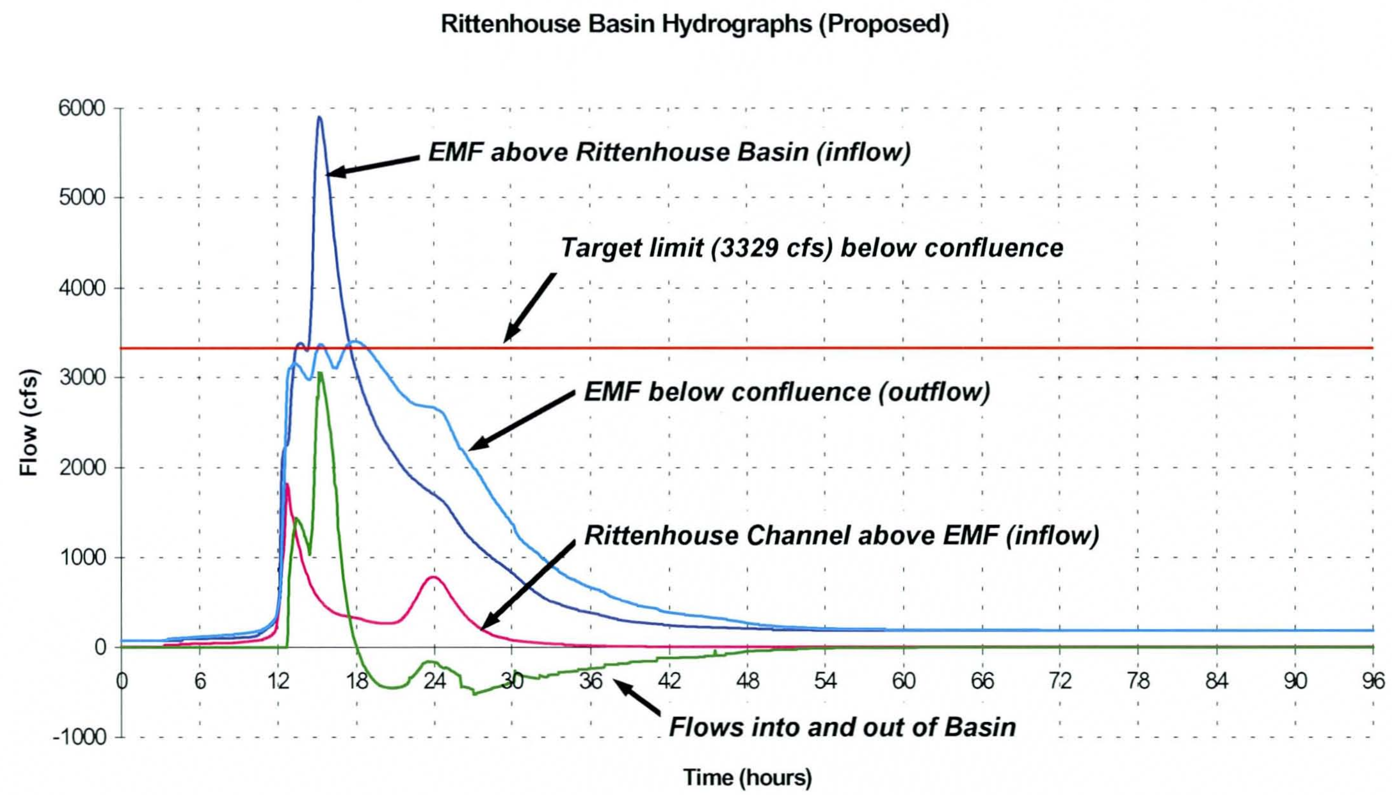



*Figure 17. Flow hydrographs at Rittenhouse Basin . The hydrographs for the EMF above Rittenhouse Basin and for Rittenhouse Channel above the EMF represent inflows to the system. The Target Discharge value is the design criterion for the EMF that applies to the Rittenhouse Basin system. The hydrograph for the EMF below Rittenhouse Channel includes the effect of the Rittenhouse Basin, and the hydrograph for Rittenhouse Basin Weir Flows shows the movement of water into and out of the proposed Basin.*

- We used normal depth to define the boundary condition at the downstream end of the modeled segment in the EMF.
- We used a friction slope for the EMF of 0.00031 feet/foot, taken from the steady flow modeling of the EMF in Reach 4 for downstream initial conditions.

We determined the stage-volume relationship for the proposed Rittenhouse Basin by assuming 4:1 (H:V) side slopes would be cut to form the basin and by reserving a portion of the available area for possible future golf course facilities. The latter is discussed in more detail in the section on Alternatives. This represented the maximum stage-volume potential, from which we could determine if enough storage volume would be available to meet the needs.

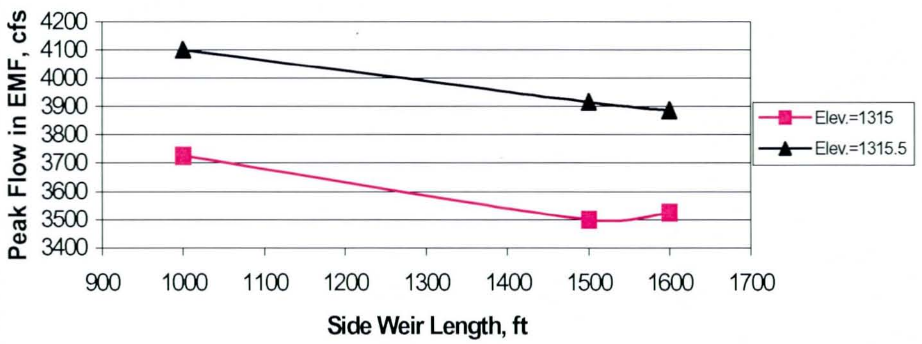



*Figure 18. The results of parameter testing on sideweir length showed that a weir length shorter than 1500 ft would reduce the volume that could be stored because they could not deliver enough water into the basin.*

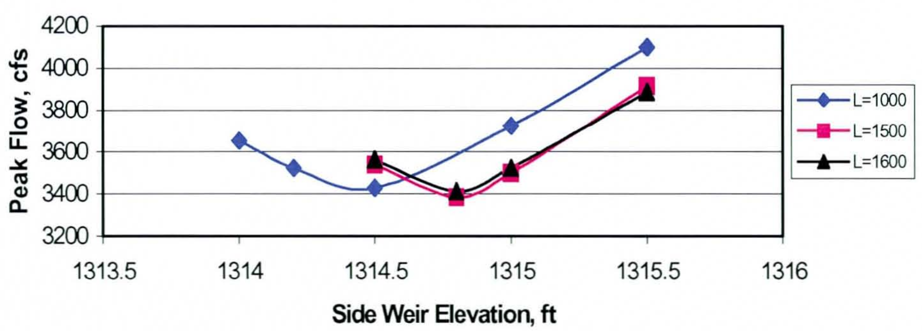



*Figure 19. The results of parameter testing on sideweir elevation showed that an elevation of 1315.0 would provide more reduction in peak flow in the EMF.*

We also estimated the depth of flow in the EMF under a 10-year size event, and set the initial sideweir elevation at or above that level. If at all possible, we wanted to limit the frequency under which the basin would become flooded because this would make multiuse functions less desirable. We estimated also that the 10-year event could be handled by the EMF without exceeding the target flows set forth by FCDMC.

### Initial conditions

We used as the initial flow data the flow rate in the first time interval for the hydrographs for the EMF and Rittenhouse Channel. We also assumed that the Rittenhouse Basin was empty at the start.

### Sideweir sizing

We performed a number of analyses and determined, for the basin volume available, that the sideweir would be optimized with a length of 1500 feet and an elevation of 1315.0. This elevation is approximately five feet above the EMF channel bottom and is also above the expected flow depth in the EMF for the 10-year event. The analyses that led to this conclusion are described in Appendix A - Hydrology and Hydraulics, and are illustrated in Figures 18 and 19.

We also evaluated using a control structure located in the bottom of the EMF itself to raise water levels in the EMF. This approach did not improve weir or basin efficiency or result in less basin volume required. Turn to Appendix A for more details.

The flow vs. time hydrograph shown in Figure 17 on the previous page gives the results of the sideweir selection used. This arrangement essentially meets the target flow criterion of 3329 cfs downstream from Rittenhouse Road.

### Draining the Basin

FCDMC expressed a desire to avoid pumping for draining the basin if at all possible. While lowering the basin floor would increase the amount of storage volume available below any water surface elevation, a floor below the EMF channel bottom could not be drained by gravity and would have to be pumped. Pumping would not only increase initial construction cost, but would also increase maintenance and operations cost and complexity.

Because of the added cost of using pumps to drain the basin, FCDMC also directed that the basin be drained by gravity to the extent possible. This criterion dictates that the Basin bottom must be above the elevation of the EMF channel adjacent to the outlet structure location. This criterion also limits the potential storage volume, since excavation cannot be done below the channel elevation unless some other suitable means of removing ponded water is provided.

Some water can be removed from storage by percolation into the ground at the basin bottom, and geotechnical studies were directed to estimate the capability of existing soils at that elevation to provide percolation capability. More information about percolation rates and other geotechnical topics can be found in Appendix B—Geotechnical Engineering Report.

We will design the outlet so that the majority of stored volumes will be released back into the EMF. However, because the Basin bottom must be relatively flat we are planning to use direct percolation into the Basin bottom to remove the last foot or so of stored water. The flat bottom slope will not allow much collection into channels in the bottom, and if a golf course or other recreation facilities are built in the Basin any uniform drainage pattern would likely be disrupted in any case.

We decided on a gravity drain system that would be made up of multiple box culvert sections. Flap gates on the EMF channel end of each box would restrict flow to only move from the basin to the channel, and would stay closed when the water elevation in the channel is higher than in the basin, thus avoiding flooding the basin except by over the sideweir.

Our analysis showed that a system of seven 4'x4' box culverts would provide enough capacity to drain the basin within the desired 36 hours following the termination of the 24-hour storm event. At the end of this time the basin should be sufficiently drained that only nuisance water should be remaining in the basin that will readily dissipate through ground infiltration. The drainage of the basin is illustrated in Figure 20.

Appendix A contains much more detailed descriptions of how the basin outlets were modeled.

### Function of a sideweir

A sideweir is constructed along the embankment of a channel, with its crest lower than the embankment top. Once water levels in the channel reach the sideweir crest, water is diverted into the storage basin on the other side of the sideweir. The higher the water level is in the channel, the greater the diverted flow.

Once storage is filled the water level rises at roughly the same rate in the channel and storage. And, once water levels in the channel recede, water from storage that is above the sideweir crest will flow back into the channel. A separate outlet is needed, however, to drain water stored below the weir crest elevation.

Several typical scenarios for the relative water levels between channel and storage are shown in Figures 21 through 24 on the next page.

### Embankment seepage protection

Seepage through an embankment is of concern because the movement of water through the soil mass could cause fine particles to migrate and the soil structure would degrade as a result. Our geotechnical engineering consultant has recommended that embankments that border on channels be protected against seepage effects by constructing a cutoff wall down the center of the embankment. This cutoff wall would extend below the channel bottom a distance equal to the height of the embankment.

### *Sideweir seepage protection*

In the area of the sideweir, we propose cutoff walls as an integral part of the weir structure, and these walls would be used instead of the core cutoff wall in the embankment under the weir. These cutoff walls serve two functions: to help reduce seepage effects, and to help protect against scour where the edge of the sideweir meets the channel or the basin. In the channel, there will be flows running parallel with the sideweir and though velocities are not generally high, the cutoff wall will help guard against local scour affecting the structure. And, on both sides of the sideweir, flows over the weir will terminate at the bottom and can cause erosion perpendicular to the sideweir. Though we provide erosion protection at the foot of the weir itself, the cutoff wall helps to preserve the integrity of the weir structure.

### *Basin drainage*

There are no areas immediately outside the proposed basin that receive drainage from areas beyond, so no interception system is needed. Rainfall on those areas at the top and sides of the embankments will travel by sheet flow down the embankment faces, and the surface treatment of the embankments will be designed to control erosion from these minor flows.

Rain that falls directly into the Basin area will add to the volume stored in those instances where flows have been diverted into the Rittenhouse Basin from the EMF and will simply add to the volume stored. No special provisions need to be made for this added volume, since it will simply be handled by the Basin itself. In the instances where flows are not diverted into the Basin from the EMF, the local rainfall will simply collect in the Basin and be removed by the same means as stored flows. If water levels are high enough the outlet system will release the captured flows into the EMF. For lower levels, any collected amounts will percolate directly into the Basin bottom. Design of any golf course or

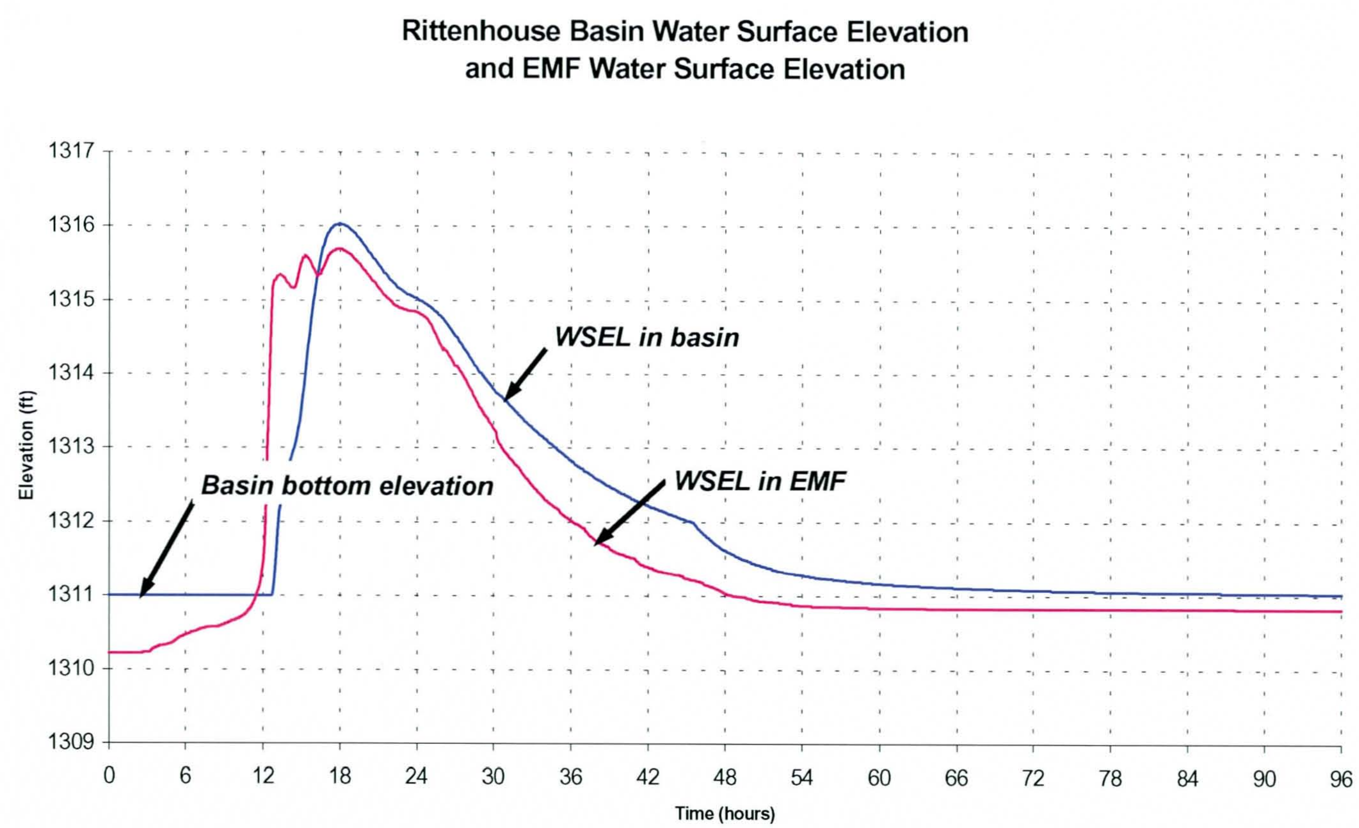



*Figure 20. This graph shows that the Basin can be drained in the desired time, as the water surface elevation (WSEL) in the Basin approaches the Basin bottom elevation.*

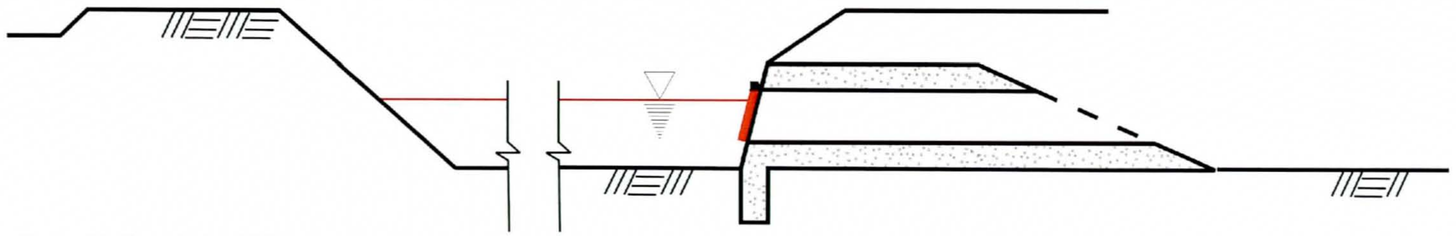


*Figure 21. Flows in the EMF below the sideweir crest level continue on downstream.*

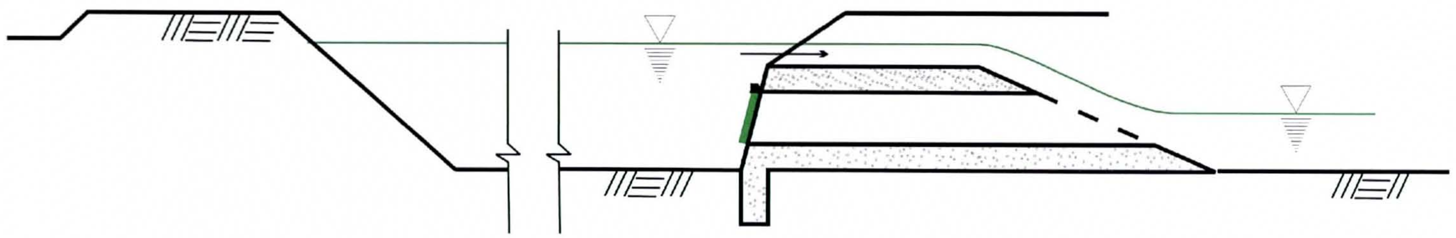


*Figure 22. Once flows in the EMF pass the sideweir crest the excess is diverted into the storage basin. The higher water level in the EMF keeps the outlet flap gates closed, and the basin fills.*

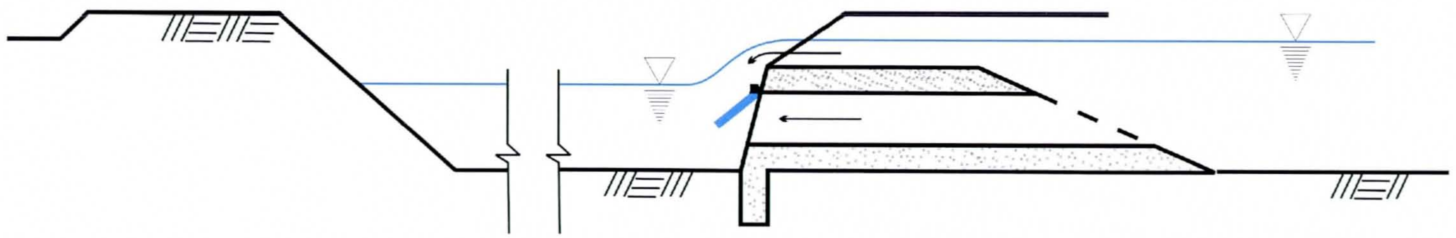


*Figure 23. As flows in the EMF recede, stored water in the basin will pass back over the weir into the EMF. The differential water levels between the EMF and the storage basin also open the flap gates and begin draining the basin. However, the sideweir will continue to be the primary means of draining the basin until basin water levels drop below the sideweir crest.*

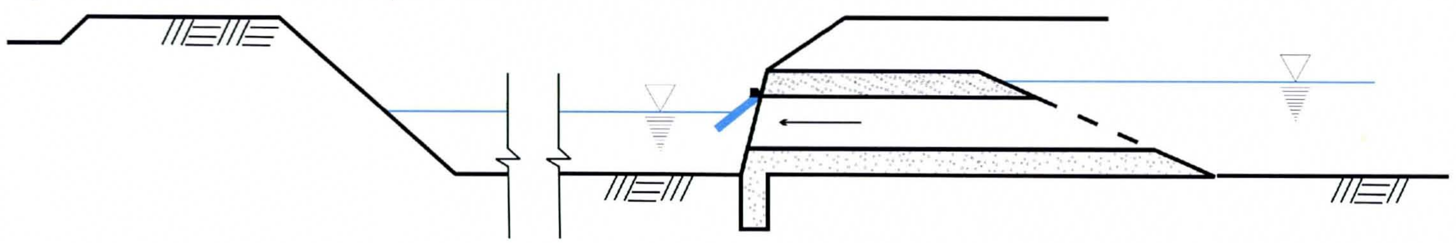


*Figure 24. With lower levels in the EMF, pressure from behind opens the outlet flap gates and the basin continues to drain through the outlet.*

multi-use facilities should take this into account in planning for local drainage collection and disposal.

## EVALUATING ALTERNATIVES

We considered two means of diverting peak flows from the EMF into a basin:

- A sideweir built into the EMF embankment so that the portion of the flow above the elevation of the weir would be diverted into the basin.
- A diversion structure built along the EMF which would divert the portion of the flow above the diversion crest into the basin.

Initial estimates of hydraulics showed that the diversion structure would probably not be feasible, so this alternative was held aside to consider only if a satisfactory sideweir solution could not be found.

### *Hydraulic results*

The flow vs. time hydrograph in Figure 17 showed the relationships among the various hydraulic components and demonstrates that the target flow condition downstream from Rittenhouse Road will be met with the arrangement proposed.

The stage vs. time hydrograph in Figure 20 shows what water levels will occur in the EMF at times during the event.

## *Proposed Project*

The basic proposed arrangement is shown in Figures 25 through 31, on the next seven pages. This has a sideweir at elevation 1315.0, which is approximately five feet above the EMF channel bottom. The weir length is 1500 feet. The Basin will contain approximately 649 acre-feet of water stored below the peak water surface elevation (1316 feet for the 100-year event, as determined in the model).

### Embankment construction

The storage basin will be constructed by excavating below existing grade throughout. Embankments will therefore be created by excavation, including those adjacent to the EMF and the Rittenhouse Channel.

### Embankment slopes

Basin side slopes are set at 4:1 (H:V) to meet geotechnical criteria as reported in the section on Design Criteria. Maintenance access roads are placed at strategic locations around the basin. These roads lead from the upper ground surface to the basin bottom and are used for foot, horse, and vehicle movements to and from the basin area.

### Embankment seepage

A concern is with the possibility of piping action through the embankments that are adjacent to channels. Piping is the result of water moving through the soil and creating a small channel by shifting the fine particles within the soil mass or by creating localized changes in the soil's compaction. Once started, piping usually progresses and can cause localized failures of the embankment. The edges of structures are especially susceptible to piping action because of the change in material along the face. We provide anti-piping measures at the ends of the sideweir structure that create a longer, less direct flow path.

Another concern is along the embankments themselves. The existing material that will be excavated is heterogeneous in nature and is different in various locations. This material does not have the inherent characteristics that would discourage seepage. There is also the potential for burrowing animals to create paths for water to enter the embankment more quickly than by normal seepage from the surface.

Our geotechnical engineers have considered various measures to protect against piping and seepage. Some measures that would provide this protection are quite costly, so we plan to do a risk analysis to help in selecting appropriate measures. That analysis will be reported at a later date. Our cost estimate in this report includes an item for an embankment core cutoff wall, but we will revise this as needed later. These matters are described in more detail in Appendix G - Geotechnical Engineering Report.

### Basin bottom

The basin bottom is set at approximately 1311 ft and is to be left with a fairly uniformly sloped surface draining toward the basin outlet. The slope is quite flat, however, in order to maximize the basin volume. A slope of 6 inches in 1000 feet is proposed. While this will not drain the basin completely, we expect that water at the lower elevations will be absorbed into the ground by percolation.

### Sideweir cross-section and materials

The sideweir shown in the concept plan is a linear concrete structure with a flat top, several steps leading into the basin, and a sloped face leading into the EMF. The uppermost level is a flat concrete slab 8 ft wide, which serves as the hydraulic weir crest. One foot below that on the basin side is a 20 foot wide flat slab. The wider area is intended for driving or walking on, and the step up to the 8 foot section helps to protect a vehicle from going over the step side into the EMF.

On the EMF side, a face sloped at 3:1 is provided to create a sloped drop into the EMF channel. Since the total height of the sideweir above the channel bottom is 60 inches, having this sloped at 3:1 or flatter meets ADA guidelines to avoid placing a handrail along the top of the weir. The limit on drop would be 42 inches if a steeper slope were used, and this would require an intermediate step on the EMF side.

On the basin side, two alternative arrangements are provided. The drop can be stepped or sloped. Four steps that drop one foot each will help to dissipate the energy of the flow over the weir as well as to provide steps that can be traversed on foot. While a sloped face (at 4:1) does not dissipate energy directly, a rip rapped area at the bottom helps to dissipate it. The sloped face can also be traversed on foot.

While the sideweir is 1500 feet long, it is only 4 feet high when viewed from the east (5 feet high when viewed from the EMF side). From the basin bottom it will appear as a long, low structure which would probably be mostly screened with plantings associated with the golf course. Viewed from normal ground elevations, such as along Power Road, the structure will be almost a half mile away and would be hardly visible.

We feel the face along the EMF and the sideweir crest and roadway areas should be poured concrete. The face along the EMF is subject to flow along the face as well as over it, and concrete will be more durable under those conditions.

The crest and roadway need the strength of poured concrete to remain dimensionally stable over time. We don't want the crest to settle in local spots because it would affect the hydraulic operation of the sideweir.

Optional materials of construction for the basin face include poured concrete and rock boxes for a stepped face. For a sloped face poured concrete, gunite, rock mattresses, and grouted riprap can be used. We expect velocities might be in the 5-10 fps range, so erosion protection is required.

The two basic alternatives of stepped and sloped face are shown in the typical details drawing in Figure 31. Two optional treatments for a stepped face are also shown in Figures 32 and 33.

### Sideweir cutoff walls

Along the EMF side and basin side edges of the sideweir structure we provide concrete cutoff walls extending at least 6 feet into the ground. These walls provide a stable edge for the sideweir faces and help prevent lateral shifting. They also provide protection against local scour at the points of transition between the hardened material of the sideweir and the adjacent earthen channel or basin.

### Plunge pool

We plan to have a plunge pool or receiving basin at the bottom of the basin side at the sideweir. This provides the final dissipation of energy from either the stepped or sloped sideweir face and helps to avoid erosion and scour at the point of transition between weir and basin floor. The width can be less for a stepped face because the velocities at the transition will be less than for the sloped face. The plunge pool should be formed using ungrouted riprap to avoid having an actual pool of standing water that might attract birds. With riprap, the water would be down between the rocks. We expect the last amounts of water in the plunge pool will percolate into the ground below.

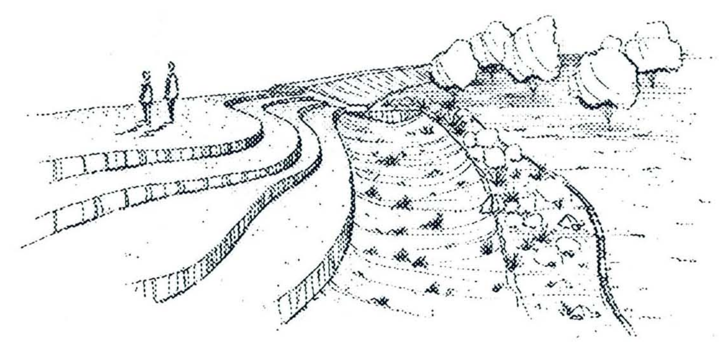


*Figure 32. This sketch shows an approach using curved lines for the steps instead of straight edges.*

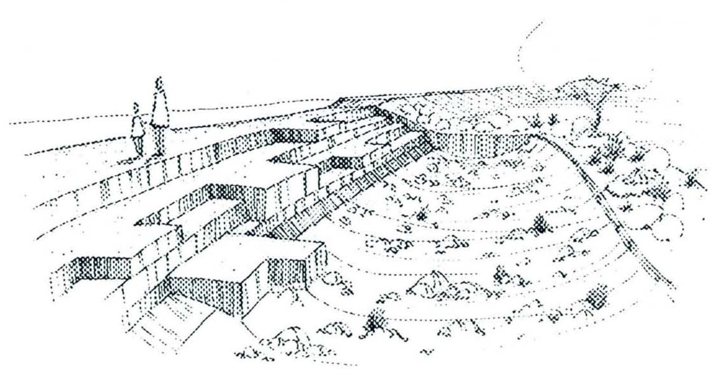


*Figure 33. Steps can also be done in an irregular fashion instead of having continuous edges.*

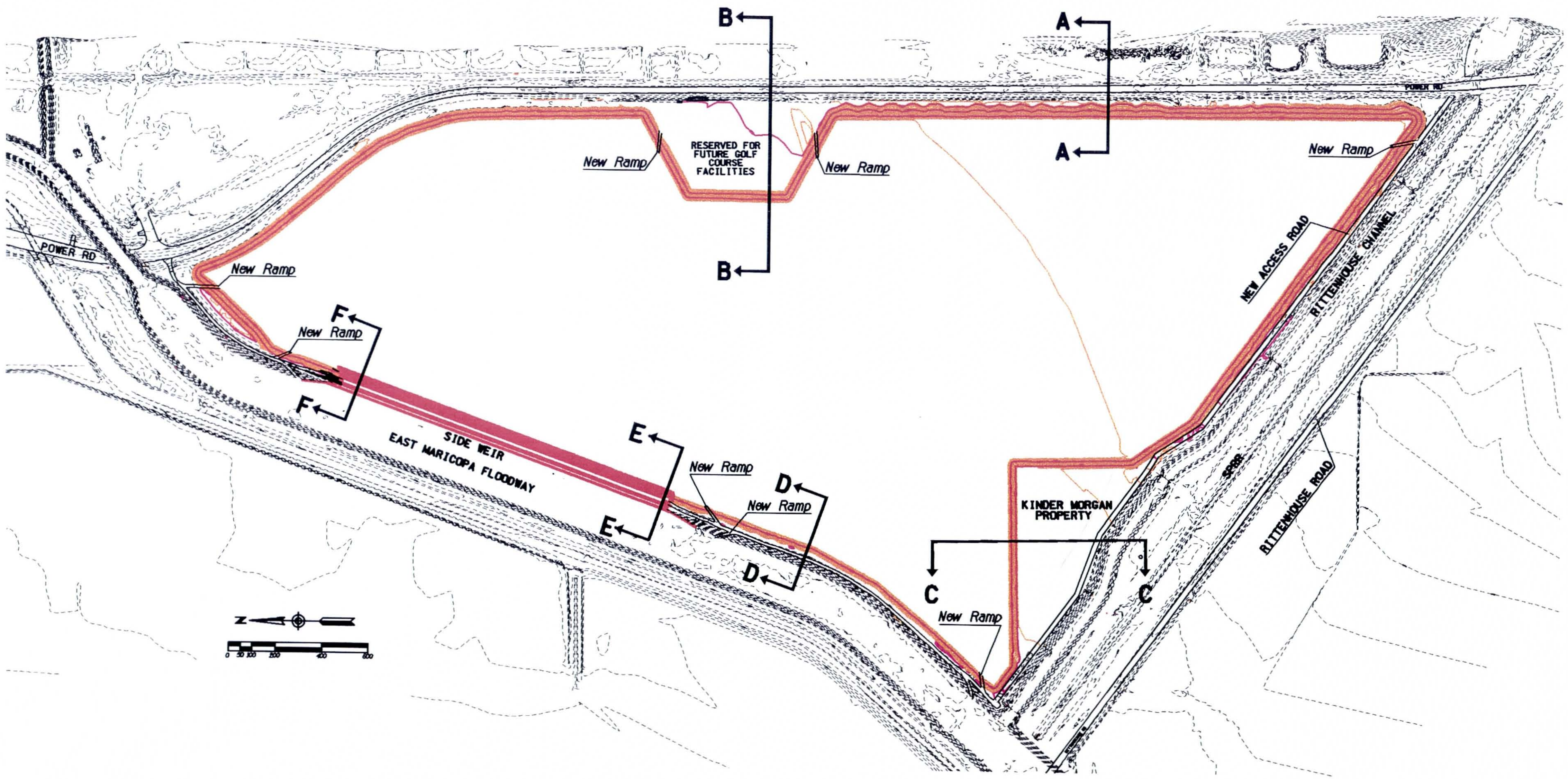
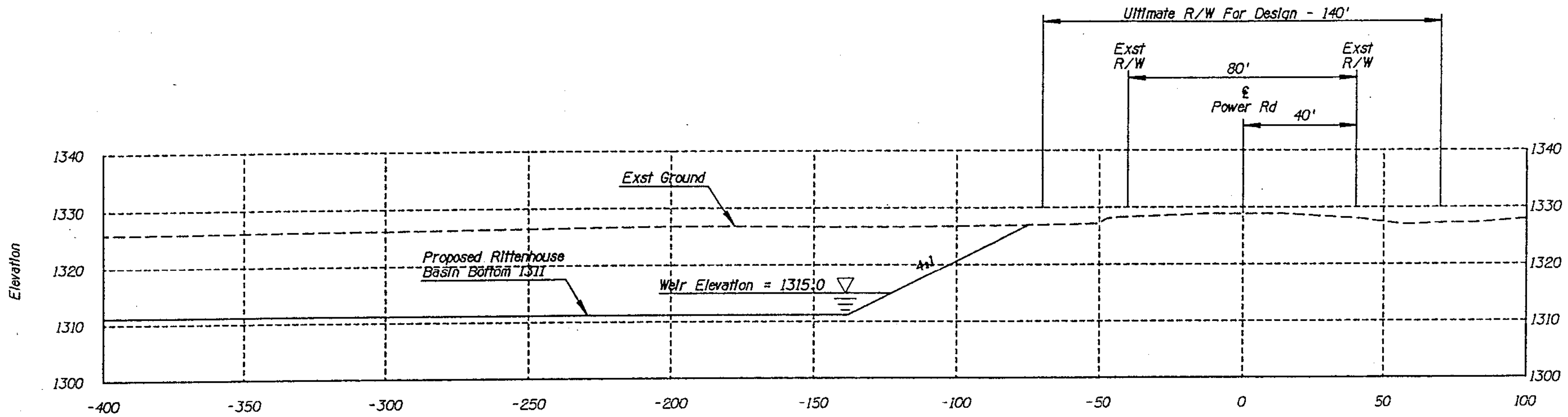
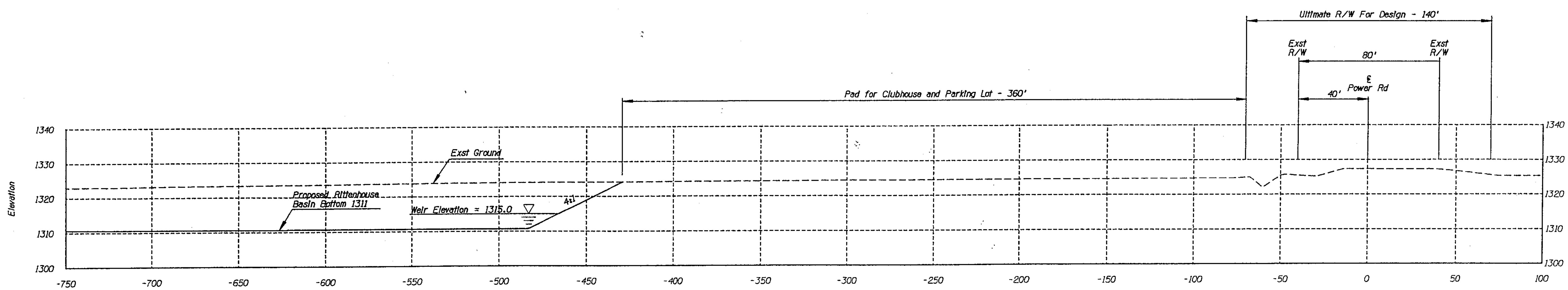



*Figure 25. This sketch shows the planned improvements to create the Rittenhouse Basin. The typical sections shown cut in this sketch are shown in more detail in Figures 26 through 30.*




SECTION A-A
LOOKING NORTH




SECTION B-B
LOOKING NORTH

*Figure 26. These typical sections and details show how the Basin would be designed.*

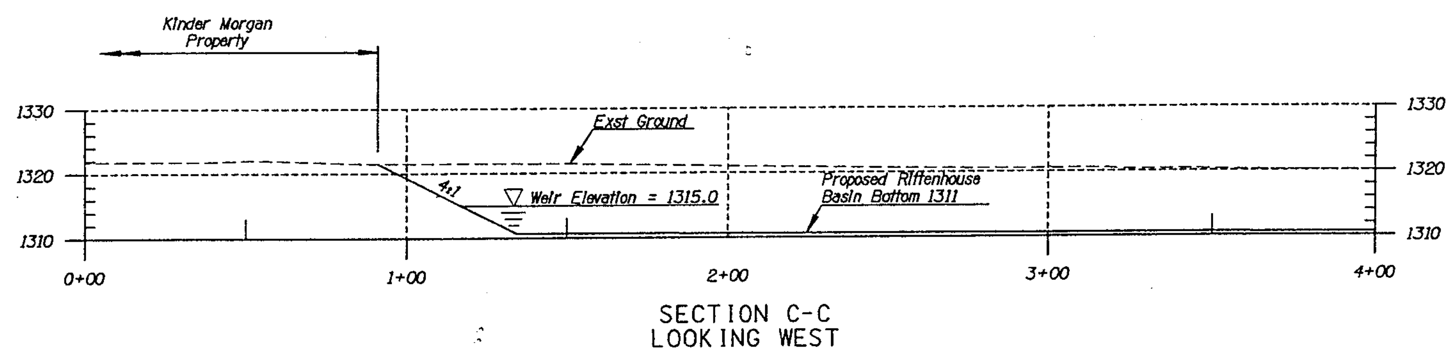
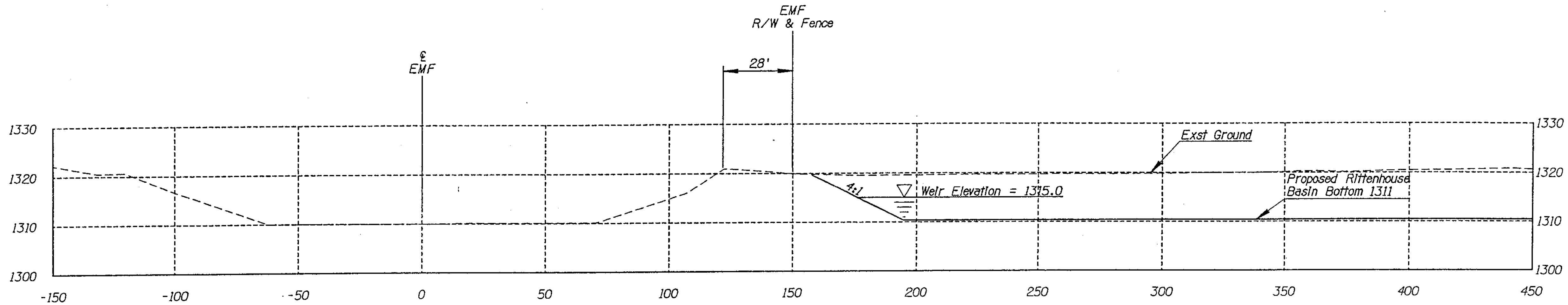
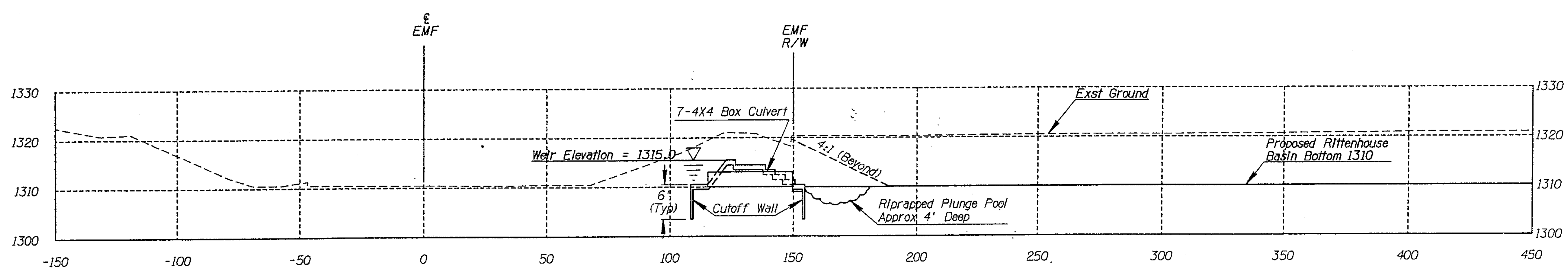



Figure 27. These typical sections and details show how the Basin would be designed.




SECTION D-D
LOOKING NORTH




SECTION E-E
LOOKING NORTH

*Figure 28. These typical sections and details show how the Basin would be designed.*

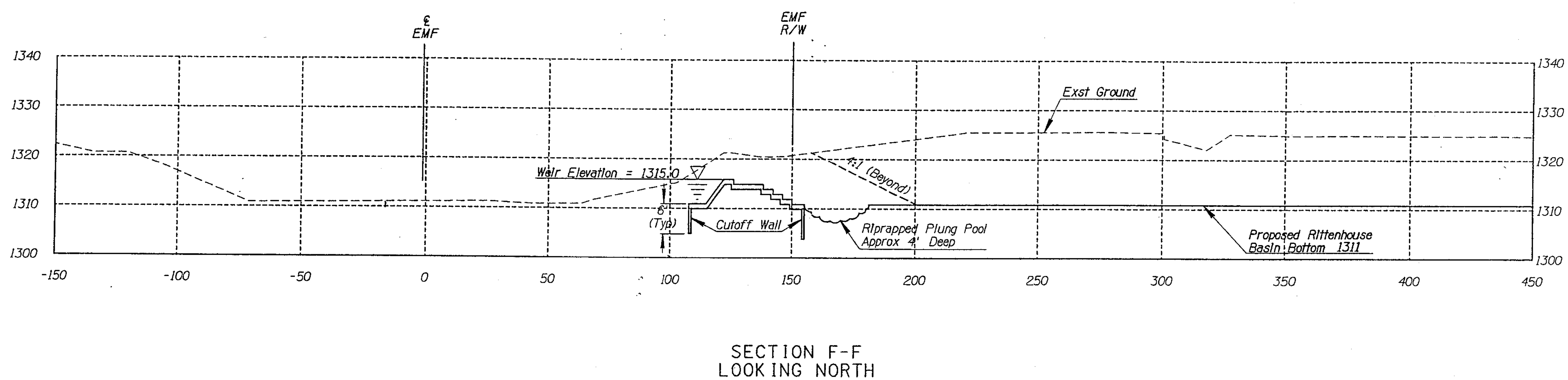



Figure 29. These typical sections and details show how the Basin would be designed.

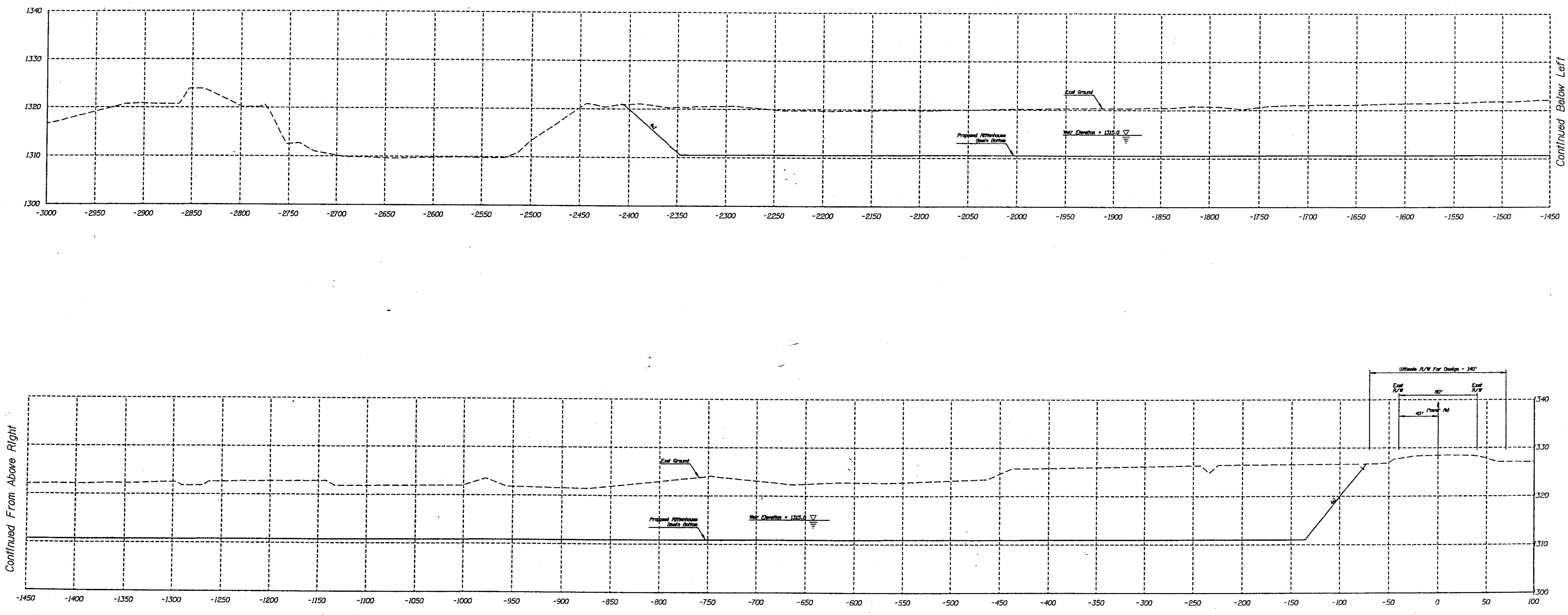



Figure 30. These typical sections and details show how the Basin would be designed.

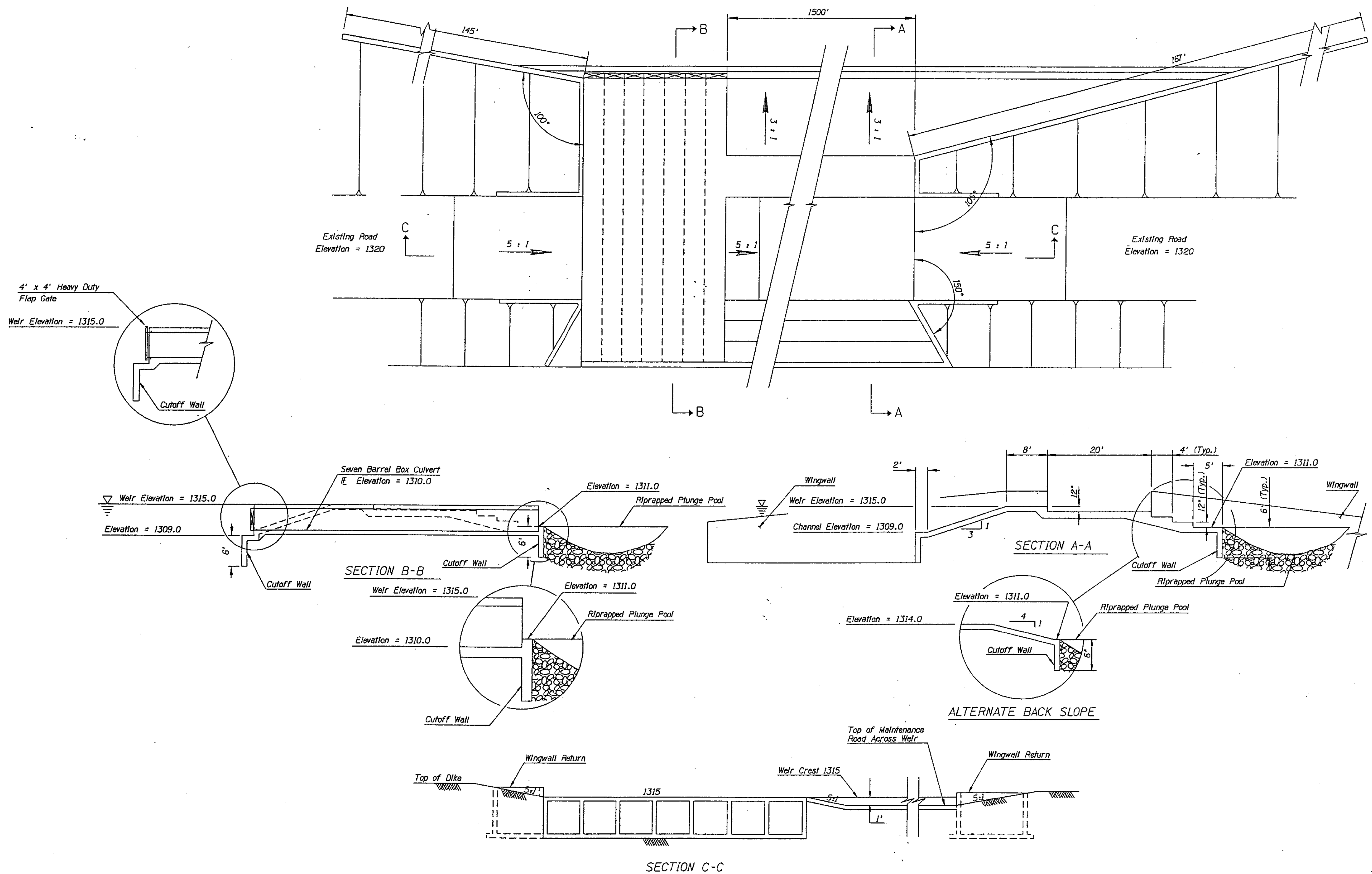



Figure 31. These typical sections and details show how the sideweir and outlet of the Basin would be designed.

### Access roads

All access roads, whether along an embankment or down the side of one, will have wearing surfaces composed of decomposed granite.

## Coordination for Multi-Use

The Basin site could be utilized for a number of different multi-use activities including trails, recharge, parks and other forms of recreational facilities.

Currently, the Town of Gilbert is evaluating development of a golf course that would coexist with the flood control facilities at Rittenhouse Basin. While our plans do not show any of this development, the possibility has been taken into account in several ways.

- An area along Power Road has been left at existing grades for siting clubhouse and other facilities that should not get flooded. The size is tentative, but is based on estimates provided by the golf course consultant. Since construction of the flood control facility will precede golf course construction, we propose that golf course construction plans would reshape the area to suit those needs.
- We expect that the golf course designers may want to change some of the embankments forming the basin boundaries. Our plans show limits on embankment slopes that must be met. We also set forth the design basin volume that we used for flood control purposes. While changes could be made for other uses, the amount of storage volume is critical and should not be reduced.
- We also expect that golf course designers will want to reshape the bottom to provide the typical relief of a golf course. This will be acceptable from a flood control standpoint as long as some criteria are met. The basin volume cannot be reduced, all runoff should be directed to the flood control outlet facilities or disposed of otherwise, and separate provisions must be made to discharge any drainage that collects below the outlet elevation. The hydraulic relationships are very critical in this basin, and the outlet as provide for in these plans is at the lowest elevation it can be and still drain the basin without the use of pumps.
- The golf course designers will also have to avoid water hazards in the golf course area. The concern is that standing water will attract birds, which may interfere with flight operations at Williams-Gateway Airport.

## Expected operation

The hydraulic operation of the proposed Rittenhouse Basin will accomplish the target peak flow rate in the EMF below the confluence of the Rittenhouse Channel and the EMF, as illustrated in the hydrograph shown earlier and repeated below in Figure 34. This hydrograph displays the composite effects of flow from the EMF, inflow from Rittenhouse Channel, and both withdrawal into and release from the new Rittenhouse Basin. Key events or situations are noted in the figure. The point of reference for this hydrograph is just downstream from the confluence of Rittenhouse Channel with the EMF.

The hydrograph in Figure 20 showed the water levels in the EMF and in the Rittenhouse Basin over time. The notations call out when water starts spilling into the Basin and various points at which the regime changes as the event progresses. The hydrograph shows that the Basin can be emptied within the desired 36-hour maximum time period from the end of the storm event.

## Expected costs

We have estimated likely construction costs, as shown in the table on the next page. Our estimates are very preliminary, however, because many decisions have yet to be made. As our design process moves forward and better definition is available for all project components, our estimates will become more precise.

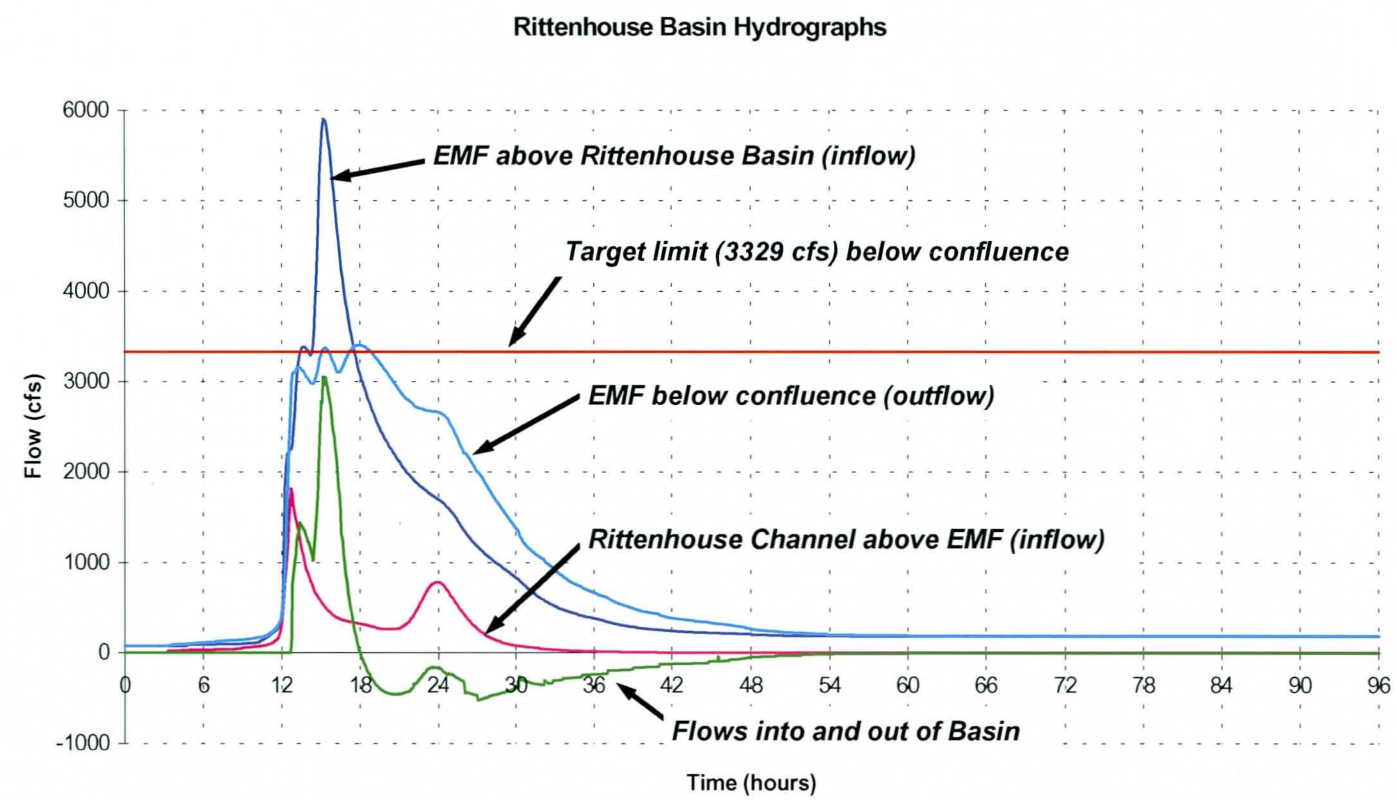



*Figure 34. Flow hydrographs at Rittenhouse Basin . The hydrographs for the EMF above Rittenhouse Basin and for Rittenhouse Channel above the EMF represent inflows to the system. The Target Discharge value is the design criterion for the EMF that applies to the Rittenhouse Basin system. The hydrograph for the EMF below Rittenhouse Channel includes the effect of the Rittenhouse Basin, and the hydrograph for Rittenhouse Basin Weir Flows shows the movement of water into and out of the proposed Basin.*

## PREDESIGN COST ESTIMATE
## RITTENHOUSE BASIN

| DESCRIPTION | UNIT | QUANTITY | UNIT PRICE | AMOUNT |
|---|---|---|---|---|
| **EARTHWORK** | | | | |
| Basin excavation (incl. haul and disposal) | CY | 3,018,100 | $3.75 | $11,317,875 |
| Structural excavation (incl. haul and disposal) | CY | 25,800 | $8.00 | $206,400 |
| Structure backfill (Placed and Compacted) | CY | 6,500 | $15.00 | $97,500 |
| **EMBANKMENT** | | | | |
| Core walls | LF | 4,800 | $265.00 | $1,272,000 |
| | | | | |
| **STRUCTURES** | | | | |
| **Weirs/spillways** | | | | |
| Sideweir (EMF to Rittenhouse Basin) | LF | 1,500 | $600 | $900,000 |
| Cut-off walls | CY | 50 | $300 | $15,000 |
| **Outlets** | | | | |
| CBC, 7-4'x4' | LF | 55 | $900 | $49,500 |
| Wingwalls | CY | 85 | $300 | $25,500 |
| Gates, trash racks | EA | 7 | $9,000 | $63,000 |
| **Erosion protection** | | | | |
| Rip rap (12") | CY | 5,600 | $35.00 | $196,000 |
| **Miscellaneous** | | | | |
| Decomposed granite maintenance road | SY | 8,300 | $23.85 | $197,955 |
| **RELOCATION AND REMOVALS** | | | | |
| Clearing and grubbing | AC | 160 | $150 | $24,000 |
| Utility relocation | LF | 1,500 | $30 | $45,000 |
| Miscellaneous removals | LS | 1 | $25,000 | $25,000 |
| **LANDSCAPING** | | | | |
| Turf seeding | SF | 340,000 | $0.20 | $68,000 |
| Hydroseeding | AC | 160 | $2,000 | $320,000 |
| Fencing (split rail) | LF | 9,000 | $30 | $270,000 |
| Irrigation system | LS | 1 | $215,000 | $215,000 |
| Trees (15 gal) | EA | 680 | $100 | $68,000 |
| Rip rap | SF | 45,000 | $3.15 | $141,750 |
| Decomposed granite paths | SF | 68,000 | $1.75 | $119,000 |
| **MISCELLANEOUS CONSTRUCTION ITEMS** | | | | |
| Mobilization/demobilization | LS | 1 | $200,000 | $200,000 |
| Survey and construction staking | LS | 1 | $200,000 | $200,000 |
| Construction, materials and quality control testing | LS | 1 | $234,500 | $234,500 |
| | | | | |
| **SUB-TOTAL CONSTRUCTION COSTS** | | | | **$16,270,980** |
| | | | | |
| **ENGINEERING (5%)** | | | | $813,549 |
| **CONTINGENCY (30%)** | | | | $4,881,294 |
| | | | | |
| **TOTAL ESTIMATED COST** | | | | **$21,965,823** |

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**APPENDIX A: HYDROLOGY AND HYDRAULICS REPORT**

**APPENDIX B: GEOTECHNICAL ENGINEERING REPORT**

**APPENDIX C: SUBSIDENCE AND FISSURES EVALUATION**

**APPENDIX D: SURVEY AND CONTROL**

**APPENDIX E: MAPPING**

**APPENDIX F: LANDSCAPING AND EROSION CONTROL, MULTIUSE PLANNING**

# HYDROLOGY AND HYDRAULICS REPORT

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# FLOOD CONTROL DISTRICT
# OF
# MARICOPA COUNTY

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## *EAST MARICOPA FLOODWAY*
## *CHANDLER HEIGHTS AND RITTENHOUSE BASIN DESIGN*

# *Hydraulic Report*

# *Rittenhouse Basin*

Revised: December 2001


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*Table of Contents*

# 1. Rittenhouse Basin Modeling Approach and Design Parameters

## 1.1. Design Goals and Parameters

The objective of the basin hydraulic design is to perform hydraulic analyses, evaluate a limited number of alternatives, and provide hydraulic design parameters as the engineering basis for the basin structural design. The parameters provided include detention basin size and key elevations, side weir size and top elevation, outlet structure size and invert elevation. The hydraulic design goal is to minimize flows in the EMF in a cost-effective manner. Specifically, it is to minimize the basin volume, while meeting the downstream EMF freeboard requirement; optimize the basin configuration to use a gravity outlet minimize basin O&M requirements; and provide multi-use opportunities for the basin.

The following Design Parameters or Criteria has been used for the hydrology and hydraulics design:

| | |
|---|---|
| Design Storm: | 100 year 24 hour storm |
| Minimum level of side weir: | 10 year storm elevation |
| Minimum level of basin: | EMF channel bottom |
| Side Weir Location: | Single weir on the EMF, at the upper end of Rittenhouse Basin |
| Outlet Location: | At the downstream end of the side weir |

## 1.2. Unsteady Flow Models

Due to the complex hydraulic interactions among the elements of the detention basin system, unsteady flow modeling was performed for the basin hydraulic design. Different from a steady flow analysis, the input for the unsteady flow modeling is a flow hydrograph instead of a peak flow. Two unsteady flow programs, HEC-RAS Version 3.0 and Full Equations Model (FEQ) Version 8.92, 1999, were tested in a pilot study. The pilot study was undertaken to compare the capabilities, ease of use, and results of each computer program (HEC-RAS and FEQ). The results of the pilot study were used to select which program would be used for further in-depth modeling.

### *1.2.1. HEC-RAS 3.0*

HEC-RAS Version 3.0 is a computer program for calculating water surface profiles for both steady or unsteady flow. It has been developed by the Corps of Engineers in a Windows environment. The unsteady flow equation solver was adapted from Dr. Robert L. Barkau's UNET model. The unsteady flow simulation can be used to model one-dimensional unsteady flow through complicated open channel systems with tributaries, split flows, bridges, culverts, in-line weirs, side weirs and other hydraulic structures. UNET is approved by FEMA for flood insurance studies with some exceptions (no floodway modeling, there are problems in bridge/culvert modeling).

### *1.2.2. FEQ Version 8.92*

FEQ Version 8.92 adopted by USGS is also an unsteady flow model, which has similar functions as the unsteady flow components in HEC-RAS 3.0. It operates in a DOS environment. The FEQ computer program was used on a 1995 Flood Control District of Maricopa County (District) project. FEQ model is approved by FEMA for flood insurance



studies with some exceptions (no floodway modeling, type 5 culvert flow needs verification with other FEMA models).

### *1.2.3. Model Selection*

A simplified basin and channel system model for Rittenhouse Basin was used during the pilot study. This system consists of a detention basin which connects to a segment of the EMF channel by a side weir. According to the test case study, both of the unsteady flow models reached similar results for the output flow hydrograph and stage hydrograph in the basin and EMF. However, there are significant differences on the modeling efforts required for each program. Below is a summary of some of the major issues encountered duringthe pilot study.

Model Setup

HEC-RAS 3.0: Similar to the previous version of HEC-RAS steady flow program, the window version of HEC-RAS 3.0 is possessed with user friendly interface. It is very easy to setup and modify a hydraulic system and the individual components. For example, to extend the side weir length, the only thing one needs to do is to change the length in the side weir input window.

FEQ 8.92: The DOS version FEQ program requires much more time and efforts to setup and revise the components of a system. It requires expertise even to modify a small section of the model. For example, to extend the side weir length requires extra effort to understand, revise and add data in a Network Control Matrix Form, which describes how the model elements are connected together.

Cross-section Requirement:

FEQ program has more strict cross-section requirement than HEC-RAS, manual smoothing of the digital CADD cross-sections had to be performed to make the cross-section more simple (with only one valley) in the pilot study.

Side Weir Modeling

HEC-RAS 3.0: The side weir equation used is the same as a normal in-line weir equation, no special adjustment is incorporated in the program to model the lateral flow over a side weir. Additionally, only one discharge coefficient can be input for each weir.

FEQ 8.92: FEQ program use a correction function in the weir equation to solve the side weir problem. Unfortunately, this correction function is based on an assumption that the weir height is zero, which is not the case forthis project.

Computation Instability

HEC-RAS 3.0: For certain channel slope configuration, especially for channel with drop structures, HEC-RAS 3.0 becomes computationally unstable, it will not converge on a value. This problem has been solved by using engineer's judgments to limit and modify the model to achieve computational stability, including dividing the model into submodels, removing reaches not relevant to the analysis, and other modeling techniques such as block the low flow channel in the cross-section at the top of a drop in Chandler Heights Basin model as discussed in the Hydraulic Report - Chandler Heights Basin prepared for this project. Analyses including steady flow modeling were conducted to verify that these modifications have no significant impacts on the system hydraulic performance. It is an area where extra efforts were exerted for the HEC-RAS unsteady flow modeling on this project.

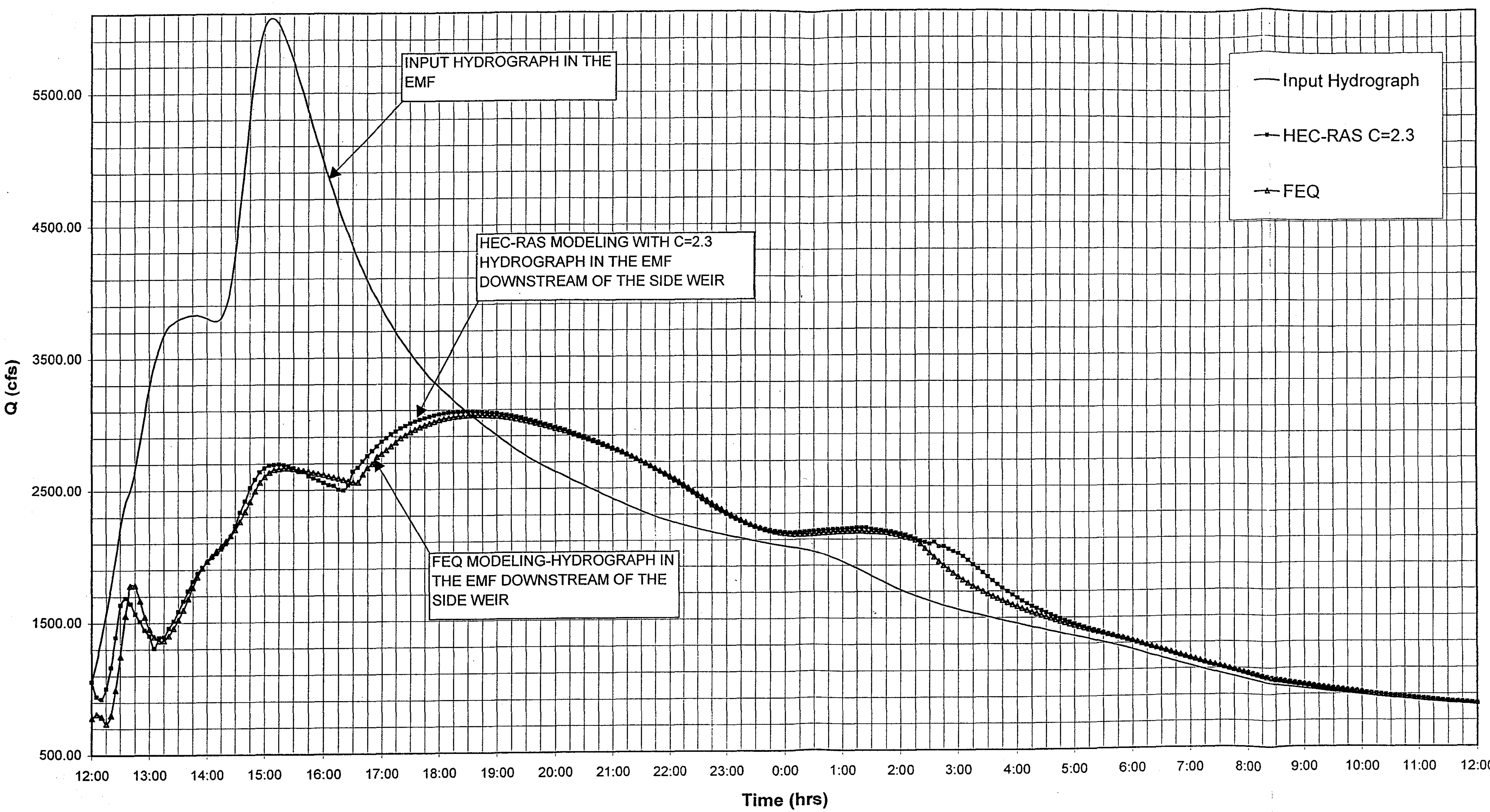
FEQ 8.92: Computational instability also occurred with the FEQ modeling. However, no conclusion can be reached regarding the frequency of the instability problem, since only limited modeling was conducted using this program.

Results Comparison

Figure 1 presents a comparison of the flow hydrographs for the pilot study obtained with the two unsteady flow programs.

Based on the modeling experience with the pilot study, HEC-RAS 3.0 was selected by District for the further unsteady flow modeling for this project.

FIGURE 1. COMPARISON OF THE RESULTANT FLOW HYDROGRAPHS BETWEEN HEC-RAS AND FEQ MODELING



## 2. Hydrology Model

### 2.1. Base Model

A future conditions HEC-1 model for a 100-year, 24-hour storm event was provided by District as base model for this project. This model consists of five sub-models and was updated by District based on models from East Mesa Area Drainage Master Plan Study prepared by Dibble & Associates and Queen Creek/Sanokai Wash Hydraulic Master Plan prepared by Huitt-Zollars & East Maricopa Floodway Capacity Mitigation Study prepared by Collins/Pina.

In addition to the base model, three target peak flows were provided by District for the EMF at the following locations, which will meet the channel capacity requirements.

| | |
|---|---|
| Peak Flow in EMF at Rittenhouse Road: | 3329 cfs |
| Peak Flow in EMF at Chandler Heights Road: | 5667 cfs |
| Peak Flow in EMF at Hunt Highway: | 8100 cfs |

### 2.2. Model Revisions

Revisions to the 100-year HEC-1 model provided by District or Kirkham-Michael during this study are listed below:

- Revision on routing two basins: 81A to Rittenhouse Channel, and 81B to EMF upstream Rittenhouse Basin.
- Revision on the flows from CAP overchutes CAP1a and CAP1b.

### 2.3. Input Hydrographs for the Rittenhouse Basin Hydraulic Design

Two hydrographs obtained from the HEC-1 modeling were required for Rittenhouse Basin unsteady flow modeling. One is the combined hydrograph in EMF at Williams Field Road (HEC-1 Station ID: EMFWFD), the second is the hydrograph in Rittenhouse Channel upstream of Rittenhouse Basin (HEC-1 Station ID: CP91).

### 2.4. Resultant Hydrograph from the HEC-RAS Modeling

Resultant hydrograph from the HEC-RAS modeling was coded into HEC-1 using QI Card. This hydrograph is input in the HEC-1 model (HEC-1 Station ID: RITBAS) to replace the hydrograph generated in the base HEC-1 model in the EMF at the same location. The revised HEC-1 model with the resultant hydrograph from the Rittenhouse Basin HEC-RAS modeling was used to generate hydrographs used in Chandler Heights Basin hydraulic design.



## 3. Rittenhouse Basin Hydraulic Design

### 3.1. HEC-RAS Model

The base Rittenhouse Basin system modeled in the unsteady flow analysis has the following configuration:

- A segment of EMF channel from downstream of Power Road Bridge to downstream of Rittenhouse Road Bridge (from Stations 17.082 to 16.0);
- A detention basin at east side of the EMF;
- A side weir at the east bank of EMF channel and hydraulically connect to the detention basin;
- A Short reach of Rittenhouse Channel, which joins the EMF at a location upstream of Rittenhouse Rail Road Bridge.

HEC-RAS models were developed for numerous basin and side weir alternatives. Additionally, inline weir structures at a location downstream of the side weir were modeled in the study to evaluate the impacts on the basin system hydraulics.

#### *3.1.1. Channel Hydraulics*

The base HEC-RAS hydraulic models for this study is based upon several hydraulic models developed in previous studies. In 1999, HNTB conducted the EMF Capacity Assessment Study and developed six separate EMF HEC-RAS models for the existing EMF configuration from it's confluence at the Gila River in Pinal County to the beginning at the Princess Basin in Mesa.

In 2000, the EMF models were modified by Collins-Pina as part of the EMF Capacity Mitigation and Multi-Use Corridor Study. The models were edited to include drainage, landscaping and multi-use features recommended as part of the study. Changes to the models included cross sectional modifications to include low-flow channels and increased n-values to account for an increase in channel vegetation as part of proposed landscaping features.

The Rittenhouse Channel hydraulic model for this study is based on the hydraulic model from Flood Insurance Study Southern Pacific Railroad Queen Creek Area, Maricopa County, prepared by Wood and Associates in 1990.

#### *3.1.2. Detention Basin Volume*

Several elevation and storage volume curves developed by Kirkham Michael for Rittenhouse Basin were evaluated in this study. Side weir was sizing is affected by thebasin volume curves. The final basin shape and the bottom elevation were selected to allow gravity drain of the basin back into EMF. The side slopes of the basin is 4:1. The bottom of the basin is about one foot higher than the EMF main channel bottom.

Below is the final elevation and storage volume for Rittenhouse Basin.

**Detention Storage Volume - Rittenhouse Basin (Bottom at 1311 ft)**

| Bain Elevation | Incremental Volume | | Cumulative Volume | |
|---|---|---|---|---|
| (ft) | ($ft^3$) | (acre-ft) | ($ft^3$) | (acre-ft) |
| 1311 | 0 | 0 | 0 | 0 |
| 1312 | 1,692,119 | 39 | 1,692,119 | 39 |
| 1313 | 6,178,262 | 142 | 7,870,381 | 181 |
| 1314 | 6,747,834 | 155 | 14,618,215 | 336 |
| 1315 | 6,803,809 | 156 | 21,422,024 | 492 |
| 1316 | 6,856,609 | 157 | 28,278,633 | 649 |
| 1317 | 6,909,975 | 159 | 35,188,608 | 808 |
| 1318 | 6,963,260 | 160 | 42,151,868 | 968 |

*3.1.3. Unsteady Flow Data*

As described in the hydrology section, hydrographs for EMF and Rittenhouse Channel were obtained from HEC-1 modeling for the future conditions.

*3.1.4. Boundary Conditions*

Normal Depth method was used to define the boundary condition at the most downstream reach of EMF, a friction slope of 0.00031 was used. This friction slope was obtained from a unsteady flow modeling of the EMF model, Reach 4.

*3.1.5. Initial Conditions*

Two types of initial condition are required in the modeling:

a) Initial Flow

Initial flows at the most upstream cross-sections of EMF and Rittenhouse Channel and at the cross-section just downstream of the Junction of the EMF and Rittenhouse Channel are required by HEC-RAS computer program. The initial flow data at the most upstream cross-sections of the channels should be equal to the flow rate at the first time interval in the hydrograph, while the initial flow data at the immediately downstream cross-section of the junction should be equal to the sum of the two initial flow used at the most upstream cross-sections.

b) Initial elevation of storage cells

For the Rittenhouse Basin analysis, the initial elevation of the basin is set to be equal to the basin bottom elevation.



## 3.2. Side Weir Coefficient

*3.2.1. Side Weir Coefficient Equation*

For the board crested side weir modeling with HEC-RAS, the discharge coefficient needs to be input manually, and will not change with the flow condition in the channel. The equation used to calculate side weir discharge coefficient is based on Willi H. Hager's study published on the Journal of Hydraulic Engineering (Hager, 1987). According to Equation (17) from Hager's paper, for the case of a nearly horizontal, prismatic side weir, the lateral outflow intensity over a side weir can be expressed as:

$$q = \frac{3}{5} c \sqrt{qH^3} (y - W)^{3/5} \left[\frac{1-W}{3-2y-W}\right]^{1/2} \tag{1}$$

Where, c = weir shape correct factor. For a vertical broad-crested weir with length B,

$$c = 1 - \frac{2}{9(1+\varsigma_b^4)}; \qquad \varsigma_b = \frac{H-w}{B};$$

H = upstream energy head;

w = height of side weir;

h = water depth upstream side weir;

$y = \frac{h}{H}$; and $W = \frac{w}{H}$.

Reorganize the Eqn. (1) we obtained:

$$q = 0.42\sqrt{2g}\, c \left[\frac{1-W}{3-2y-W}\right]^{1/2} \sqrt{(h-w)^3} \tag{2}$$

The weir equation used in HEC-RAS 3.0 for side weir discharge is:

$$Q = CLH^{3/2} \tag{3}$$

Where, H = the upstream water depth above the crest (In the side weir modeling, water surface option was selected for the weir flow calculation).

L = the length of the weir.

By comparison of Equations (2) and (3), the discharge coefficient C can be calculated using the following equation:

$$C = 0.42\sqrt{2g}\, c \left[\frac{1-W}{3-2y-W}\right]^{1/2} \tag{4}$$

*3.2.2. Side Weir Coefficient Determination*

The side weir discharge coefficients were manually calculated with Equation (4) using the average hydraulic data from HEC-RAS unsteady flow modeling at two cross-sections within the side weir limits. It is an iterative process. With an initial side weir coefficient, we obtained flow hydraulic properties at the two cross-sections within the side weir limits from HEC-RAS modeling for each time period. A new averaged C value then was calculated based on the hydraulic properties and re-input into HEC-RAS. According to our experience, convergence could be reached in only 2 or 3 iterations.

The side weir discharge coefficient determined for Rittenhouse Basin is 2.3. A detailed calculation table is included in the appendix.

*3.2.3. Sensitivity Analysis of the Side Weir Coefficient*

To evaluate the impact of the side weir discharge coefficient (C) on the hydrograph at the cross section immediately downstream of the side weir (Sta. 16.468), a sensitivity analysis was performed for one of the Rittenhouse Basin test case. The HEC-RAS model was run with different C values ranged from 2.0 to 3.0. Two relationship curves of the discharge coefficient, one with the hydrographs, another one with the water surface elevations at a cross section downstream of the side weir (Station 16.468) were generated as shown in the Figures 2 and 3.

The results show that the differences between the hydrographs are small. The difference between the peak flows at Station 16.468 for C=2.0 and C=3.0 is only 147 cfs, which is 5% of the peak flow for C=3.0. A larger C will result in more flow getting into the basin during the basin filling phase for a given stage in the EMF. This results in higher water surface elevations in the basin than for a smaller C. During this phase, less flow will remain in the EMF. However, at the basin draining phase, more reversed flow will get out of the basin, which will increase the total flow in the EMF.

The C value has slightly impacts on the water surface elevation in EMF at the cross-section downstream of the side weir.

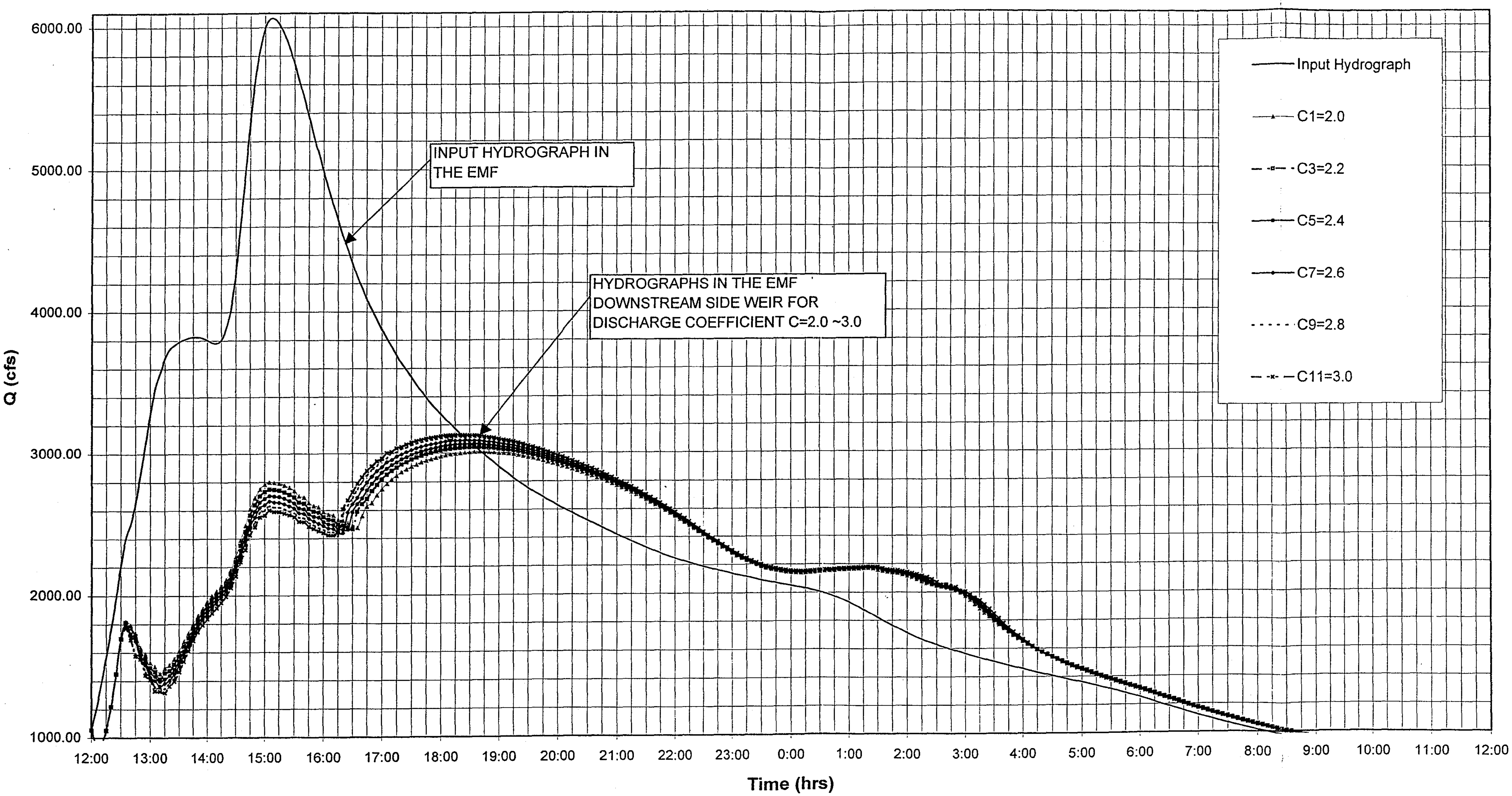
## 3.3. Side Weir Locating and Sizing

The location of the side weir was selected along the EMF at an upstream portion of the basin and downstream of the channel bend. This location was chosen in order to maximize the utilization of the basin volume, to avoid flow turbulence and negative super-elevation caused by bridge structure and the bend, and to reduce construction difficulty.

The EMF and Rittenhouse input hydrographs from HEC-1 modeling and the detention basin elevation and volume curve are two major factors in this project which affect sizing of the side weir. The side weir was sized to minimize the basin volume requirement, reduce the weir construction cost, and to meet the downstream target peak flow requirement.

Numerous modeling efforts were made for each alternative basin volume curves. Relationships between the side weir size (length and height) and the downstream maximum flow were generated and analyzed during the study. There are three peaks in the downstream flow hydrograph. For a fixed side weir height, with increase in the weir length, the maximum flow will shift between the peaks according to the amount of inflow and reverse flow over the weir, and the maximum discharge will reduce to some point and then begin to increase. Similarly, for a fixed side weir length, with increase in the weir height, the maximum discharge will also reduce to some point and then begin to increase, as shown in Figures 4 and 5.

# FIGURE 2. HYDROGRAPHS FOR DIFFERENT DISCHARGE COEFFICIENTS



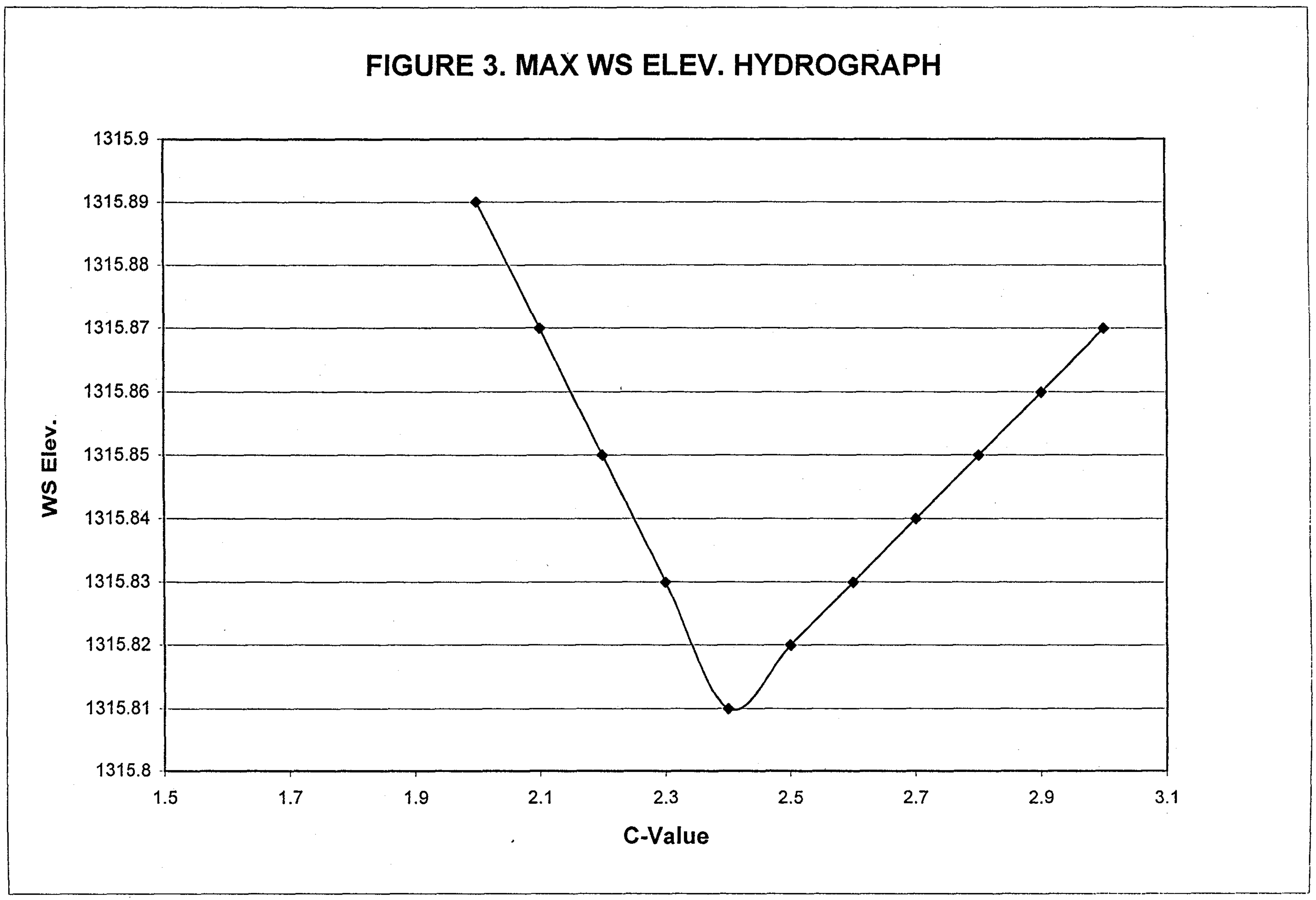


FIGURE 3. MAX WS ELEV. HYDROGRAPH
WS Elev.
C-Value
1315.9
1315.89
1315.88
1315.87
1315.86
1315.85
1315.84
1315.83
1315.82
1315.81
1315.8
1.5
1.7
1.9
2.1
2.3
2.5
2.7
2.9
3.1

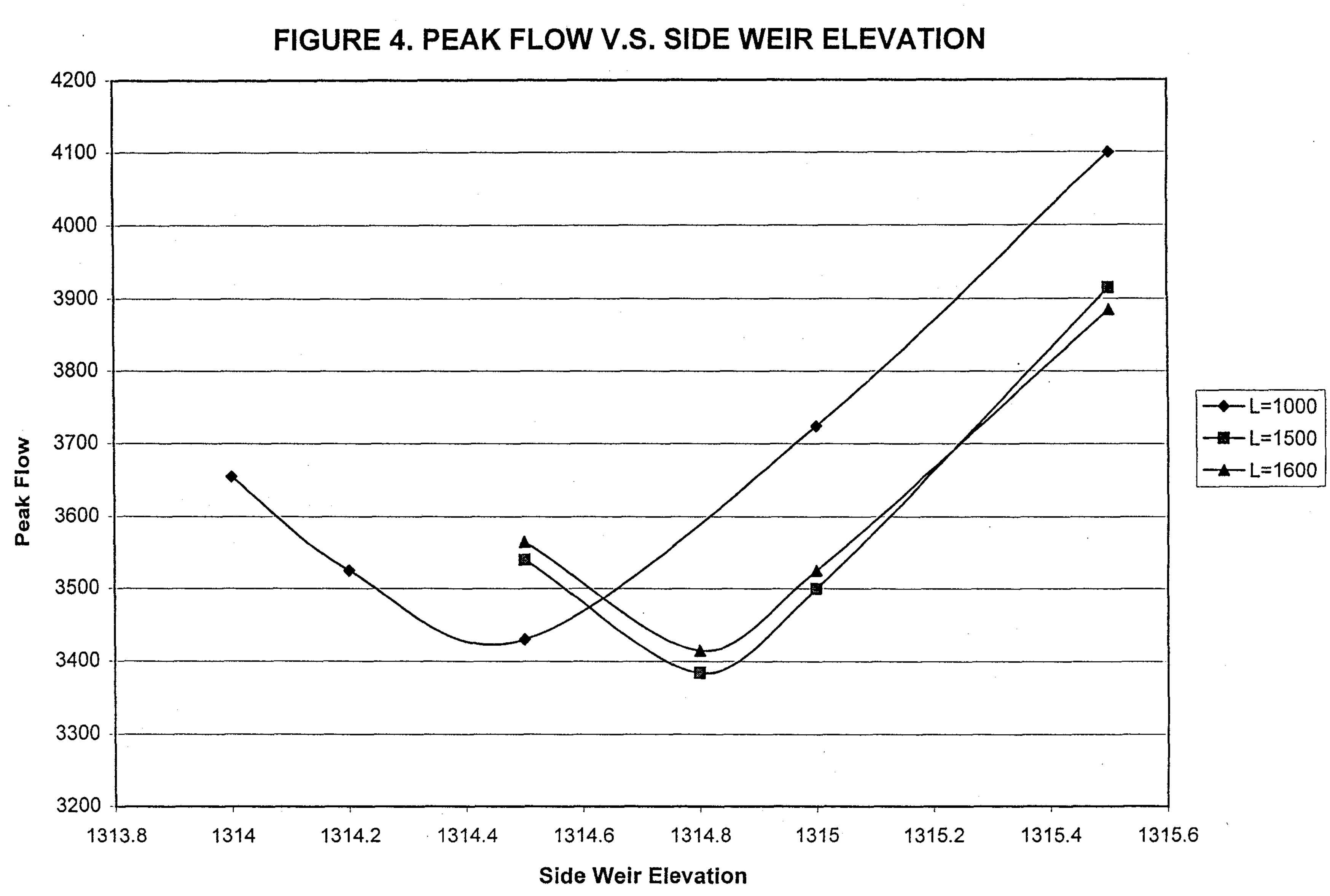
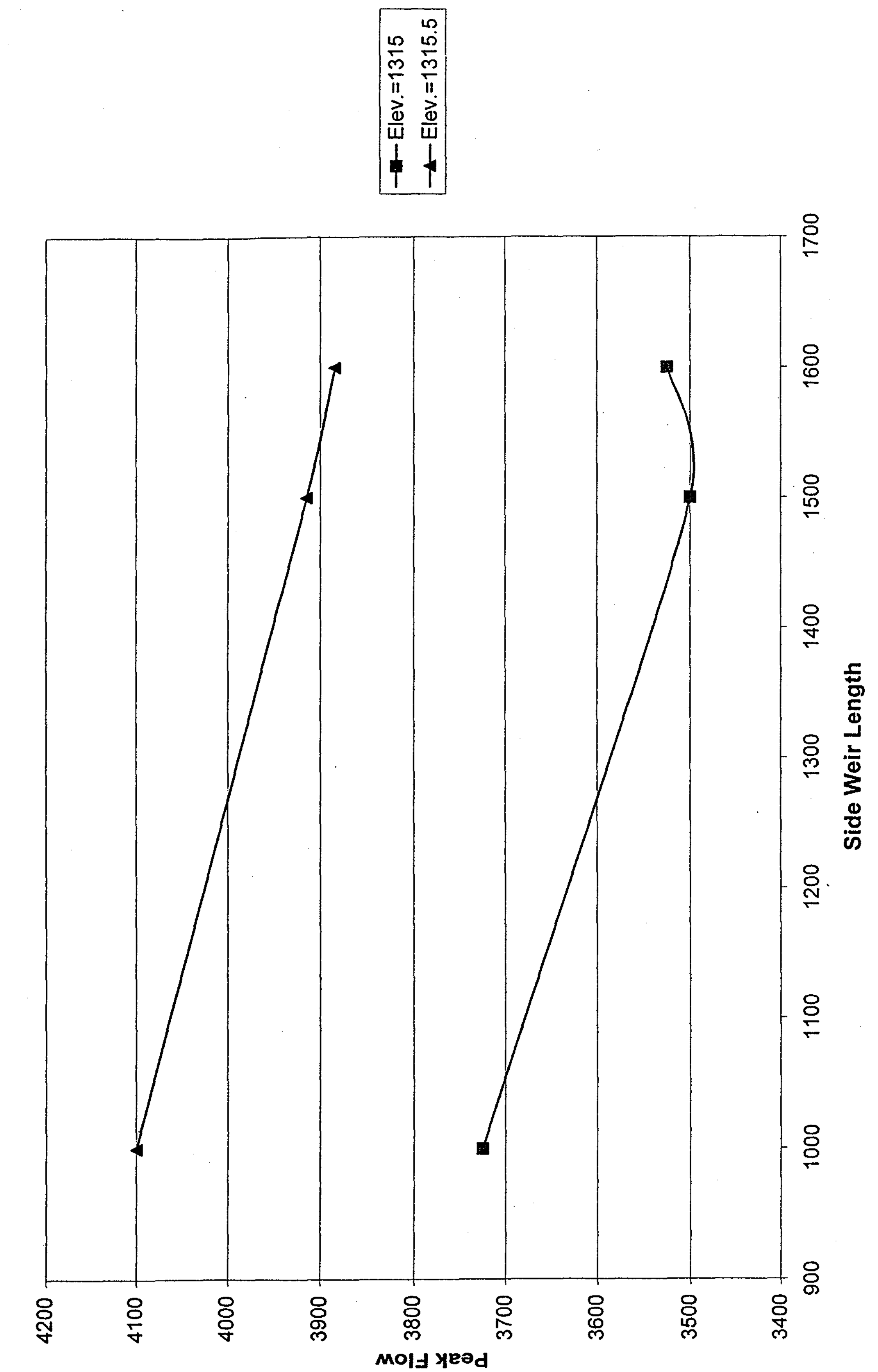


FIGURE 4. PEAK FLOW V.S. SIDE WEIR ELEVATION
Peak Flow
Side Weir Elevation
4200
4100
4000
3900
3800
3700
3600
3500
3400
3300
3200
1313.8
1314
1314.2
1314.4
1314.6
1314.8
1315
1315.2
1315.4
1315.6
L=1000
L=1500
L=1600

FIGURE 5. PEAK FLOW V.S. SIDE WEIR LENGTH




Based on the hydraulic analysis, a side weir of 1500 feet in length with a top elevation of 1315.0 is recommended for Rittenhouse Basin.

### 3.4. Impact of Inline Control Structure

Inline control structures in EMF were evaluated in the effort to raise the water surface elevation in EMF so as to reduce basin depth. The inline control structures evaluated in this study are inline weirs with an opening at the low flow channel location to allow low flow passing. According to the hydraulic modeling, when the inline structures are below 4 feet, there is no significant impact on water surface elevation and the side weir operation. When the inline structures are higher than 5 feet, which is about the same elevation as the sideweir, the weir will start having an effect on the basin system.An inline weir of this height will increase the construction costs and construction complications, therefore, is not recommended for the Rittenhouse Basin detention system.

### 3.5. Outlet Hydraulic Design

#### *3.5.1. Draining of Basin*

Basically, there are two ways to drain the basin: by gravity and by pump station. Draining by gravity usually is the most economical way, and therefore is preferred. The goal is to drain the basin in 36 hours starting from the end of the 24-hour storm event.

Shallow water that cannot be drained by gravity may be able to infiltrate into the groundwater.

#### *3.5.2. Gravity Drain*

The gravity drain system is designed to be self-regulating with flap gates at the outlet ends of box culverts. The flap gates allow discharge whenever the water surface in the basin is higher than in the EMF.

#### *3.5.3. Residual Flow*

The hydrograph in EMF at Williams Field Road from the original HEC-1 model for this study has a residual flow of 616 cfs and lasts indefinitely. This residual flow has significant impact on the basin draining system, since the water surface in the EMF would remain at a level that affects gravity draining of the basin.

As stated in hydrology section, the HEC-1 model was revised to better represent the real hydrology situation at the CAP overchutes, CAP1a and CAP1b, upstream of the basin. The residual flow in EMF at Williams Field Road was reduced to 186 cfs. This results in a water surface elevation of about 1310.8 in the EMF at the basin outlet.

#### *3.5.4. Impact of Basin Storage Volume Curve*

Through the outlet hydraulic analysis, it was discovered that the upper stages of basin can be drained very fast, but the last foot of the water in the basin will take a significant portion of time to drain. To enhance the basin drain efficiency and reduce the total drain time, using a smaller volume for the lowest foot of depth was recommended during the study. The basin bottom elevation was raised from 1310 to 1311.

### *3.5.5. Outlet Modeling*

Similar to other available unsteady flow modeling program, there is no option in HEC-RAS 3.0 to model a culvert with a flap gate. Additionally, we have experienced program instability problems when modeling of a culvert in the system. The option of a sluice gate was used in the study to model the outlet culvert with flap gate.

A sluice gate is modeled in HEC-RAS with two variables, a discharge coefficient and the gate opening. The height of opening can be varied as a function of time. However, only one discharge coefficient can be defined for each gate. Gate opening can be defined for each time interval. The key issue in outlet modeling is how to define the gate opening for each time interval to let the discharge flow from the sluice gate be equal to that from a culvert with a flap gate under the same hydraulic conditions.

Two sets of equations were developed to model the culvert with flap gate under the conditions of inlet control and outlet control.

<u>Inlet Control Equation</u>

For inlet control conditions, the capacity of the culvert is limited by the capacity of the culvert opening, rather than by conditions farther downstream. FHWA manual "Hydraulic Design of Highway Culverts" (FHWA, 1985), HDS-5, Appendix A – Design Methods and Equations, Table 9, Chart No. 10, Scale 3, provided the following two equations for unsubmerged and submerged culvert design:

Unsubmerged: $$H_w / D = K\left[\frac{Q}{AD^{0.5}}\right]^M \quad \text{for } Q/AD^{0.5} \le 3.5 \qquad (4)$$

Submerged: $$H_w / D = c\left[\frac{Q}{AD^{0.5}}\right]^M + Y - 0.5S \quad \text{for } Q/AD^{0.5} \ge 4.0 \qquad (5)$$

Where, $H_w$ = headwater energy depth above the invert of the culvert inlet

D = interior height of the culvert barrel

Q = discharge through the culvert

A = full cross sectional area of the culvert barrel

S = culvert barrel slope

K, M, c, Y = Equation constants, which very depending on culvert shape and entrance condition. For this study, using K=0.486, M=0.667, c=0.0252, and Y=0.805.

Using a 4'X4' box culvert as a unit outlet structure, $AD^{0.5} = BD^{1.5} = (4)(4)^{1.5} = 32$, and the unsubmerged condition is for $Q \le (32)(3.5) = 112$ cfs, which is the case experienced in Rittenhouse Basin outlet hydraulics for the inlet control flow condition.

Re-write Eqn. (4) with B=1' (unit width), we obtained:

$$Q = 2.95\,H_w^{1.5} \qquad (6)$$



<u>Outlet Control Equations</u>

For outlet control flow, the calculation is energy based. The total head loss through the culvert is calculated using the following formula:

$$H_L = h_{en} + h_f + h_{ex} + h_G \qquad (7)$$

Where, $h_{en}$ = entrance loss

$h_f$ = friction loss

$h_{ex}$ = exit loss

$h_G$ = flap gate loss

a) Friction Loss

The friction loss in the culvert is calculated using Manning's equation, which is expressed as follows:

$$h_f = L\left(\frac{Qn}{1.486AR^{2/3}}\right)^2 \qquad (8)$$

Where, $h_f$ = friction loss

L = culvert length

Q = flow rate in the culvert

n = Manning's roughness coefficient

A = area of flow

R = hydraulic radius

b) Entrance Loss

Entrance loss is calculated as a function of the velocity head inside the culvert at the upstream end as follows:

$$h_{en} = K_{en}\frac{V_{en}^2}{2g} \qquad (9)$$

Where, $h_{en}$ = energy loss due to the entrance

$K_{en}$ = entrance loss coefficient, 0.2 was used in this study

$V_{en}$ = velocity inside of culvert at entrance

g = acceleration due to gravity, g=32.2

c) Exit Loss

The exit loss is calculated with a simplified function, which is expressed as a function of the velocity head inside the culvert at the exit as follows:

$$h_{ex} = K_{ex}\left(\frac{\alpha_{ex}V_{ex}^2}{2g}\right) \qquad (10)$$

Where, $h_{en}$ = energy loss due to the exit

$K_{en}$ = exit loss coefficient, 0.65 was used in this study

$V_{en}$ = velocity inside of culvert at exit

$\alpha_{ex}$ = coefficient velocity weighting coefficient, using $\alpha_{ex}=1$

d) Flap Gate Head Loss

A loss equation was created for a 4'X4' flap gate based on the loss curve for a 48" steel drainage gate (flap gate) from Waterman Industries, Inc. Catalog Drawing No. 0049.

Through curve fitting, we developed an equation to represent the curve. The equation is a function of flow through the culvert and has the following polynomial form:

$$h_G = 0.000001Q^3 - 0.00023Q^2 + 0.0145Q - 0.072 \quad (11)$$

<u>HEC-RAS Sluice Gate Equation</u>

The equation used in HEC-RAS for a free flowing sluice gate is as follows:

$$Q = CWB\sqrt{2gH} \quad (12)$$

Where, W = width of the gate opening in feet

B = height of the gate opening in feet

H = upstream energy head above the spillway crest

C = coefficient of discharge

When the downstream tailwater increases to form a submergence condition, that is when the tailwater depth above the spillway divided by the energy headwater above the spillway is greater than 0.67, the following form of equation is used in HEC-RAS:

$$Q = CWB\sqrt{2g3H} \quad (13)$$

Where, H = difference between upstream energy head and downstream water surface

When the submergence reach 0.80, the HEC-RAS program will change to the fully submerged Orifice equation:

$$Q = CA\sqrt{2gH} \quad (14)$$

Where, A = area of the gate opening.

H = difference between upstream energy head and downstream water surface

<u>Outlet Modeling Procedure</u>

As discussed previously, the methodology of the outlet modeling in this study is to define the gate opening for each time interval to let the discharge flow calculated with the HEC-RAS sluice gate equations be equal to that from the inlet or outlet control equation. It is an



iterative process between manual calculations for gate openings and the HEC-RAS modeling.

The following procedure expresses the general steps of the calculation and HEC-RAS modeling:

Step 1. Run HEC-RAS model of the detention basin system with outlet gate fully open.

Step 2. Obtain the resultant stage hydrographs at EMF and in the basin starting from the time when water surface elevation in the basin is higher than the water surface elevation in EMF. Divide the hydrograph into sections.

Step 3. For each section of the hydrograph, calculate average discharge through the outlet system as follows:

a) Determine average water surface elevation in the Basin;

b) Determine the difference of the water surface elevation in the Basin and in EMF;

c) Calculate discharge using both inlet and outlet control equations;

d) Select the smaller one as calculated average discharge, $Q_c$, through the outlet system for this section of the hydrograph.

Step 4. Select a constant gate discharge coefficient C.

Step 5. For each section of the hydrograph, determine gate opening using the gate equation (12) or (13) according to the submergence condition, to let the calculated resultant $Q_g$ be equal to the $Q_c$ obtained at Step 3.

Step 6. Assign gate-opening values determined in Steps 4 and 5 for each time interval that is corresponding to each section of the hydrograph.

Step 7. Run HEC-RAS with the discharge coefficient and the gate opening for each time interval.

Step 8. Review the Stage Hydrographs to check whether or not the Stage Hydrographs are close enough to the previous ones.

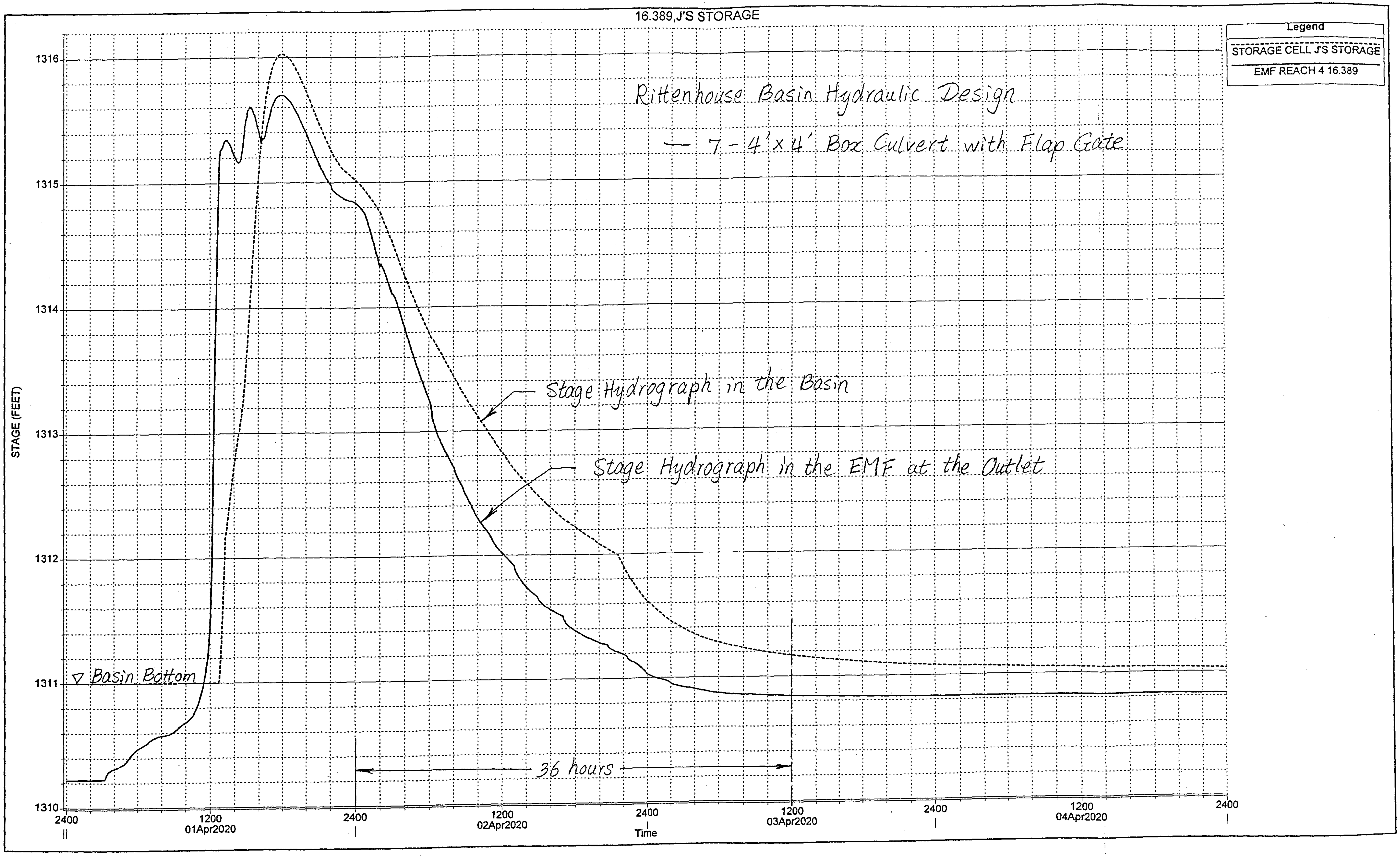
If NO, go back to Step 3.

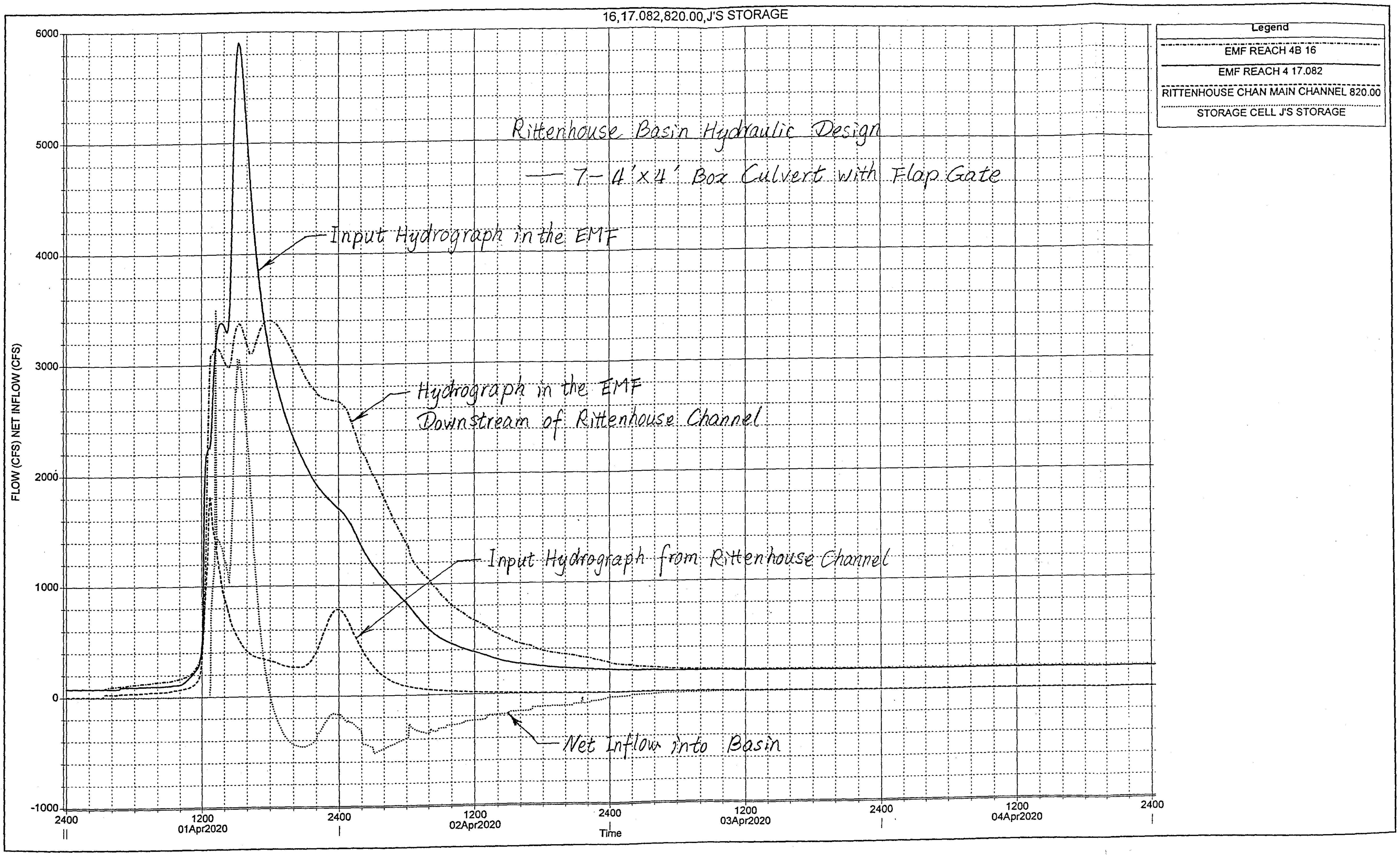
If YES, the outlet modeling is completed.

*3.5.6. Outlet Sizing Results*

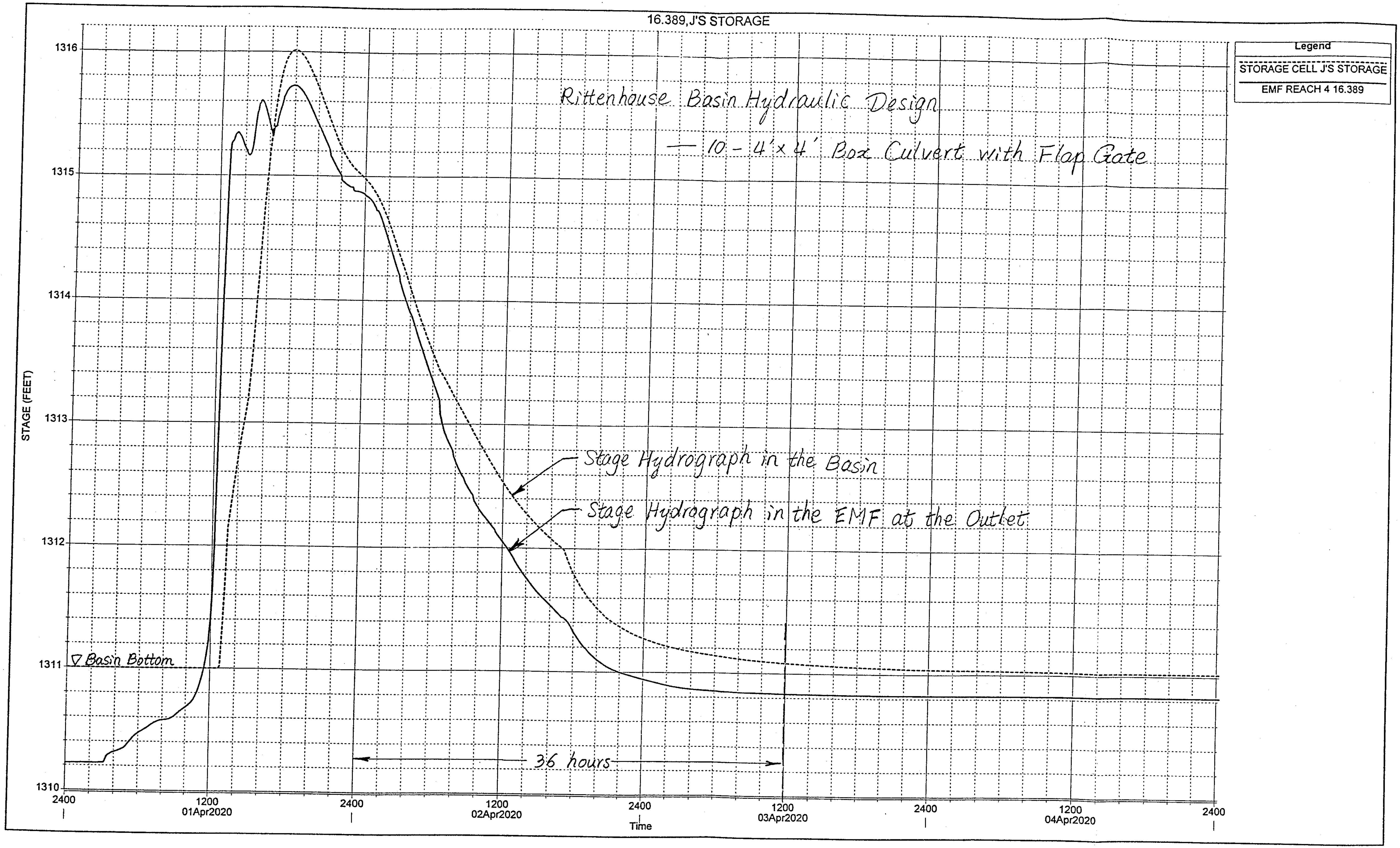
Outlet culvert was sized to drain the basin within 36 hours, which starts from the end of the 24 hour storm event. Based on the outlet modeling for the basin volume curve presented in the previous section, using a 7-barrel 4'X4' box culvert, the basin can be drained to a depth with 2 inch water remaining in the basin at the time of the end of 36 hours. Using a 10barrel 4'X4' box culvert, there will be about one inch water remaining in the basin at the time of the end of 36 hours.

It is recommended that a 7-barrel 4'X4' box culvert be used for Rittenhouse Basin.


16.389,J'S STORAGE
Legend
STORAGE CELL J'S STORAGE
EMF REACH 4 16.389
Rittenhouse Basin Hydraulic Design
— 7 - 4' x 4' Box Culvert with Flap Gate
Stage Hydrograph in the Basin
Stage Hydrograph in the EMF at the Outlet
▽ Basin Bottom
36 hours
STAGE (FEET)
1316
1315
1314
1313
1312
1311
1310
2400
1200
01Apr2020
2400
1200
02Apr2020
2400
1200
03Apr2020
2400
1200
04Apr2020
2400
Time



16,17.082,820.00,J'S STORAGE
Legend
EMF REACH 4B 16
EMF REACH 4 17.082
RITTENHOUSE CHAN MAIN CHANNEL 820.00
STORAGE CELL J'S STORAGE
Rittenhouse Basin Hydraulic Design
— 7- 4'x4' Box Culvert with Flap Gate
Input Hydrograph in the EMF
Hydrograph in the EMF
Downstream of Rittenhouse Channel
Input Hydrograph from Rittenhouse Channel
Net Inflow into Basin
FLOW (CFS) NET INFLOW (CFS)
6000
5000
4000
3000
2000
1000
0
-1000
2400
1200
01Apr2020
2400
1200
02Apr2020
2400
1200
03Apr2020
2400
1200
04Apr2020
2400
Time


16.389,J'S STORAGE
Rittenhouse Basin Hydraulic Design
— 10 - 4'x4' Box Culvert with Flap Gate
Legend
STORAGE CELL J'S STORAGE
EMF REACH 4 16.389
Stage Hydrograph in the Basin
Stage Hydrograph in the EMF at the Outlet
▽ Basin Bottom
36 hours
STAGE (FEET)
1316
1315
1314
1313
1312
1311
1310
2400
1200
01Apr2020
2400
1200
02Apr2020
2400
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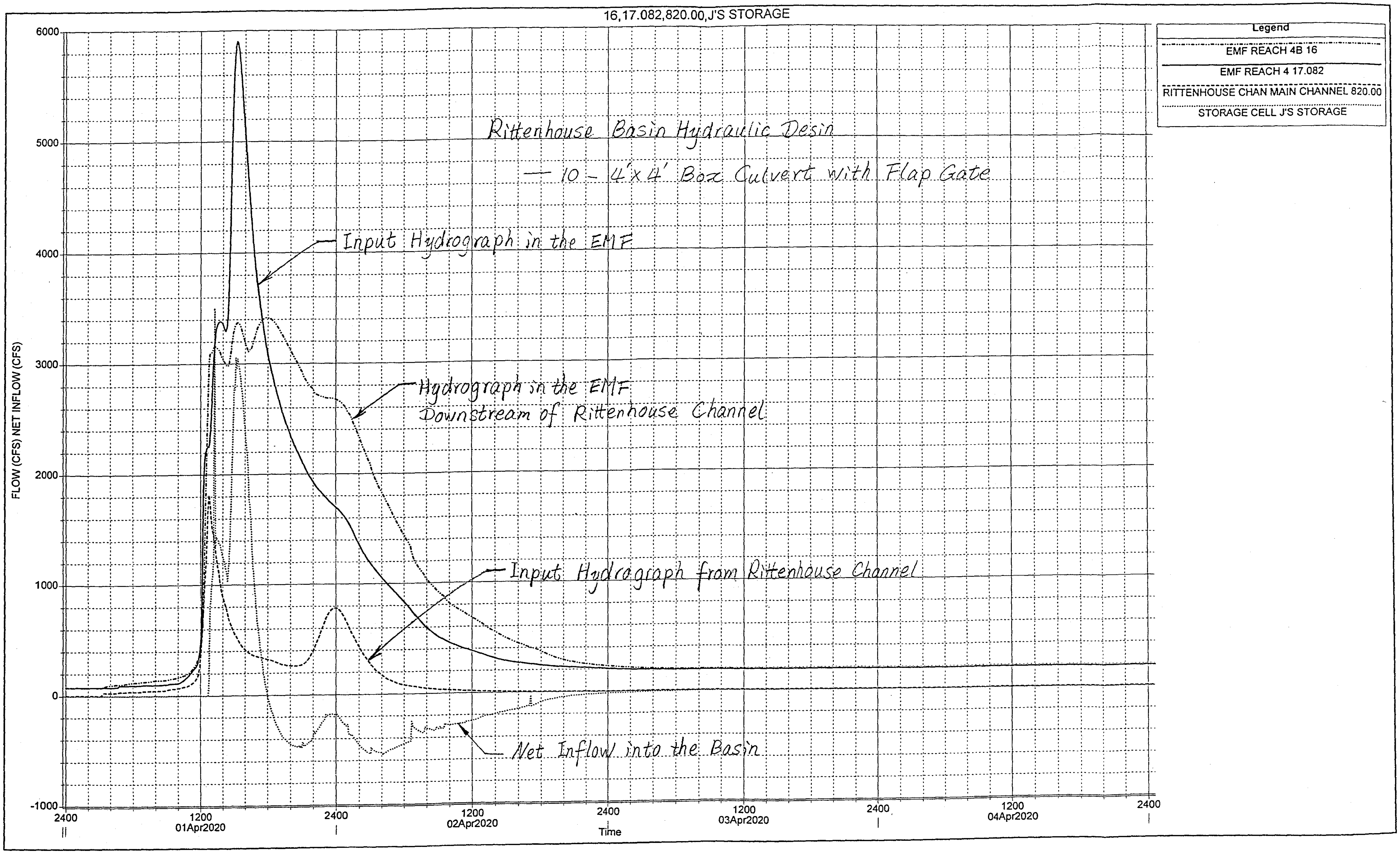
03Apr2020
2400
1200
04Apr2020
2400
Time



16,17.082,820.00,J'S STORAGE
Legend
EMF REACH 4B 16
EMF REACH 4 17.082
RITTENHOUSE CHAN MAIN CHANNEL 820.00
STORAGE CELL J'S STORAGE
Rittenhouse Basin Hydraulic Desin
— 10 - 4'x4' Box Culvert with Flap Gate
Input Hydrograph in the EMF
Hydrograph in the EMF
Downstream of Rittenhouse Channel
Input Hydrograph from Rittenhouse Channel
Net Inflow into the Basin
FLOW (CFS) NET INFLOW (CFS)
6000
5000
4000
3000
2000
1000
0
-1000
2400
1200
01Apr2020
2400
1200
02Apr2020
2400
1200
03Apr2020
2400
1200
04Apr2020
2400
Time

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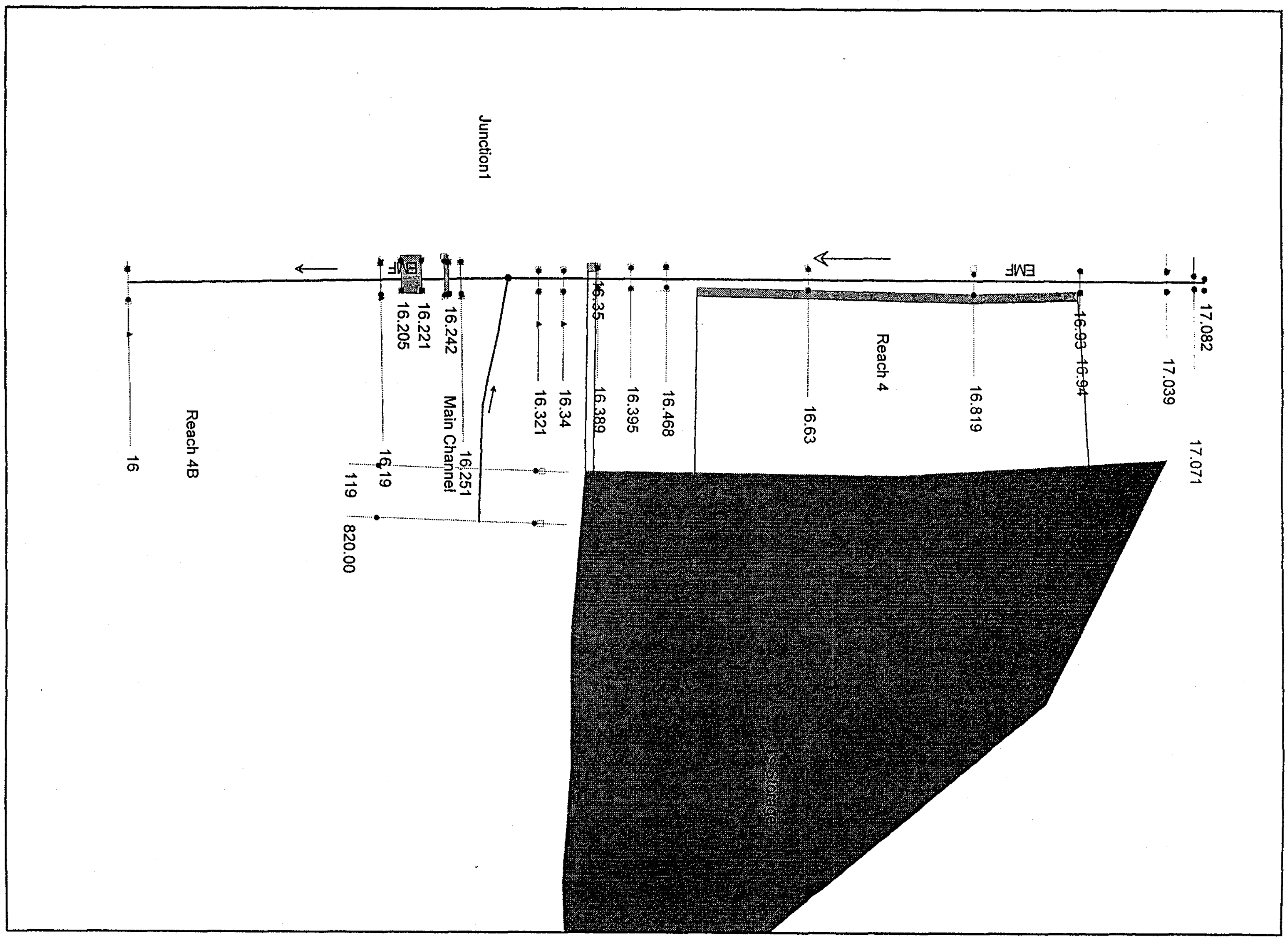



# Calculation of Side Weir Discharge Coefficient

C Calculations9-25 Design: Primatech, LLC

Equation: $q=C_d'(h-w)^{1.5}$

h -- water depth upstream side weir
w -- height of side weir

$C_d'=C_{d0}' * C_s'$

$C_{d0}'=C_{d0} * (2g)^{0.5} = 0.42 * (2g)^{0.5} = 3.37$

$C_s' = c * C_s$

$c = 1-2/(9*(1+\zeta_b^4))$

$\zeta_b = (H-w)/B$

$C_s = ((1-W)/(3-2*y-W))^{0.5}$

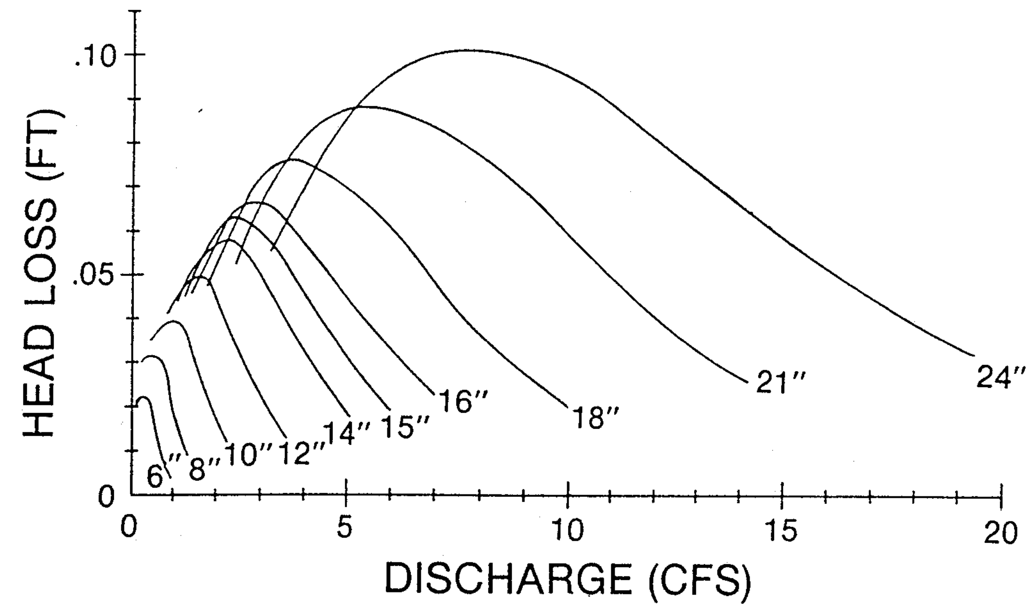
c -- weir shape correction factor (this formular is for for vertical broad-cresed weirs)
H -- Energy Head
B -- Width (along flow direction)
y = h/H
W = w/H

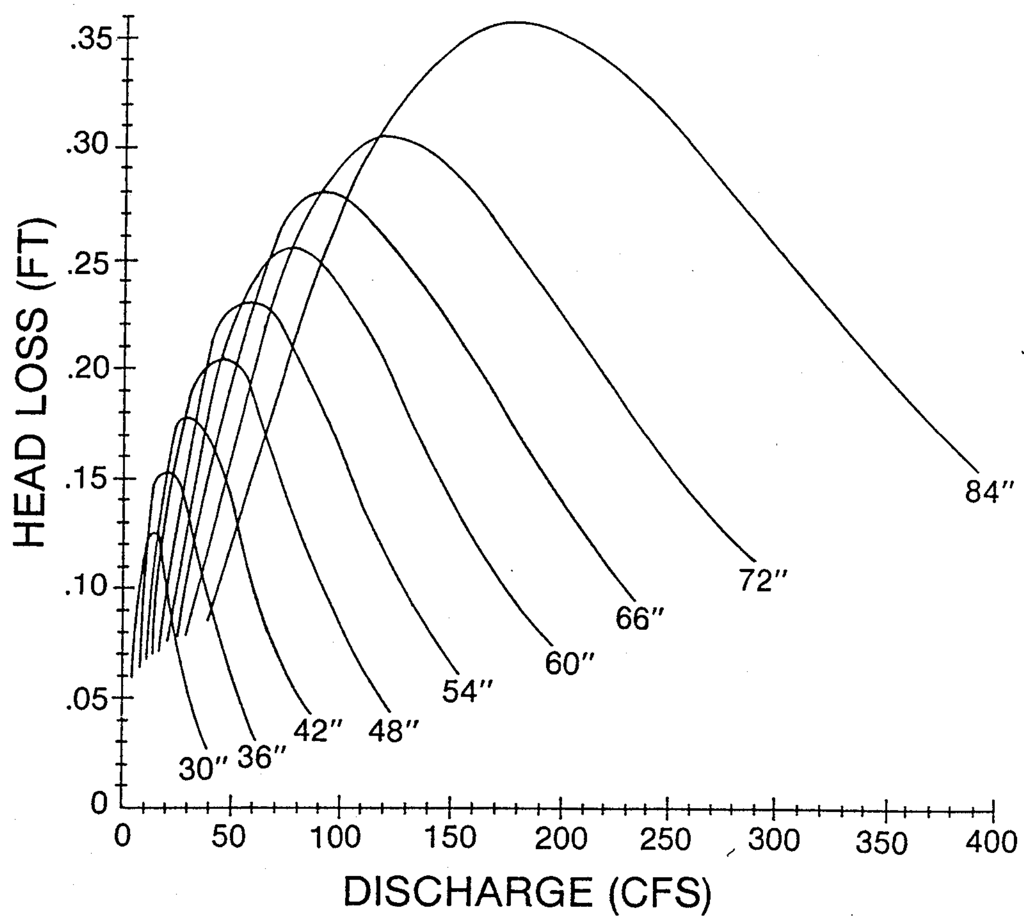
Friction Slope=0.00031 Weir Length=1500 Weir Elev=1315 Initial: Cd=2.3 Basin Elev = 1311 9/25/01

| Station | Appro. Flow | Q | B | w | h | H | y | W | $\zeta_b$ | c | $C_s$ | $C_s'$ | $C_d'$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | (cfs) | (ft) | (ft) | (ft) | (ft) | | | | | | | |
| 16.63 | At Peak | 3200.27 | 10 | 5.19 | 6.07 | 6.19 | 0.98 | 0.84 | 0.10 | 0.78 | 0.90 | 0.70 | 2.4 |
| -1309.81 | 80% Pre-Peak | 2543.48 | 10 | 5.19 | 5.68 | 5.77 | 0.98 | 0.90 | 0.06 | 0.78 | 0.87 | 0.68 | 2.3 |
| | 80% Post-Peak | 2563.45 | 10 | 5.19 | 5.97 | 6.05 | 0.99 | 0.86 | 0.09 | 0.78 | 0.92 | 0.71 | 2.4 |
| | 50% Pre-Peak | 1601.45 | 10 | 5.19 | 5.56 | 5.6 | 0.99 | 0.93 | 0.04 | 0.78 | 0.91 | 0.71 | 2.4 |
| | 50% Post-Peak | 1606.07 | 10 | 5.19 | 4.73 | 4.78 | 0.99 | 1.09 | -0.04 | | | | |
| 16.819 | At Peak | 5163.98 | 10 | 5.24 | 6.14 | 6.47 | 0.95 | 0.81 | 0.12 | 0.78 | 0.81 | 0.63 | 2.1 |
| -1309.76 | 80% Pre-Peak | 4003.53 | 10 | 5.24 | 6.01 | 6.22 | 0.97 | 0.84 | 0.10 | 0.78 | 0.84 | 0.65 | 2.2 |
| | 80% Post-Peak | 4105.48 | 10 | 5.24 | 5.91 | 6.14 | 0.96 | 0.85 | 0.09 | 0.78 | 0.81 | 0.63 | 2.1 |
| | 50% Pre-Peak | 2561.37 | 10 | 5.24 | 5.75 | 5.84 | 0.98 | 0.90 | 0.06 | 0.78 | 0.88 | 0.68 | 2.3 |
| | 50% Post-Peak | 2618.95 | 10 | 5.24 | 6.34 | 6.42 | 0.99 | 0.82 | 0.12 | 0.78 | 0.94 | 0.73 | 2.5 |

*The depths are refered to the main channel bottom elevations **Average: 2.3**

PERFORMANCE CHART APPLICABLE FOR WATERMAN F-10 & F-50 GATES. HEAD LOSS THRU HEAVIER F-25 & F-55 SERIES GATES MAY BE SLIGHTLY GREATER.




| MODEL | AUTOMATIC DRAINAGE (FLAP) GATE PERFORMANCE CHART | |
|---|---|---|
| SCALE | CATALOG DWG. NO. 0049 | REVISION NO. |

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DO NOT USE FOR INSTALLATION
UNLESS PART OF CERTIFIED & APPROVED SUBMITTAL

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**PRIMATECH**
ENGINEERS & CONSULTANTS

Job No. ______ Project EMF Sheet ____ of ____
Subject Box Culvert Discharge
By GLC Date 9/14/01 Checked By ______ Date ______

Rittenhouse Outlet.

From FHWA Manual "Hydraulic Design of Highway Culverts", HDS-5
Appendix A – Design Methods & Equations

From Table 9, pg 147 for Chart #10 - Scale 3
Coefficients –
Unsubmerged: Eqn Form = 2, K = 0.486, M = 0.667
Submerged: c = 0.0252, Y = 0.865

Unsubmerged: $H_w/D = K\left[Q/AD^{.5}\right]^M$ (27)

Submerged: $H_w/D = c\left[Q/AD^{.5}\right]^2 + Y - 0.5S$ (28)

Unsubmerged is for $Q/AD^{.5} \le 3.5$

Submerged is for $Q/AD^{.5} \ge 4.0$

Using a 4x4 box as a unit of design; $AD^{.5} = BD^{1.5} = 4(4)^{1.5} = 32$

∴ Unsubmerged condition is for $Q \le 3.5(32) = 112$ cfs

We can re-write Eqn 27 from manual to solve for Q as function of Hw

$$H_w/D = K\left[\frac{Q}{AD^{.5}}\right]^M$$

for B = 1', D = 4',

$$\frac{H_w}{D} = K\left[\frac{Q}{D^{1.5}}\right]^M$$

$$\left[\frac{Q}{D^{1.5}}\right] = \left[\frac{H_w}{DK}\right]^{1/M}$$

$$Q = D^{1.5}\left[\frac{H_w}{0.486\,D}\right]^{1/0.67} = \frac{D^{1.5}\,H_w^{1.5}}{(0.339)(D^{1.5})} \ldots$$

$$Q = 2.95\,H_w^{1.5}$$

# HYDRAULIC DESIGN OF HIGHWAY CULVERTS

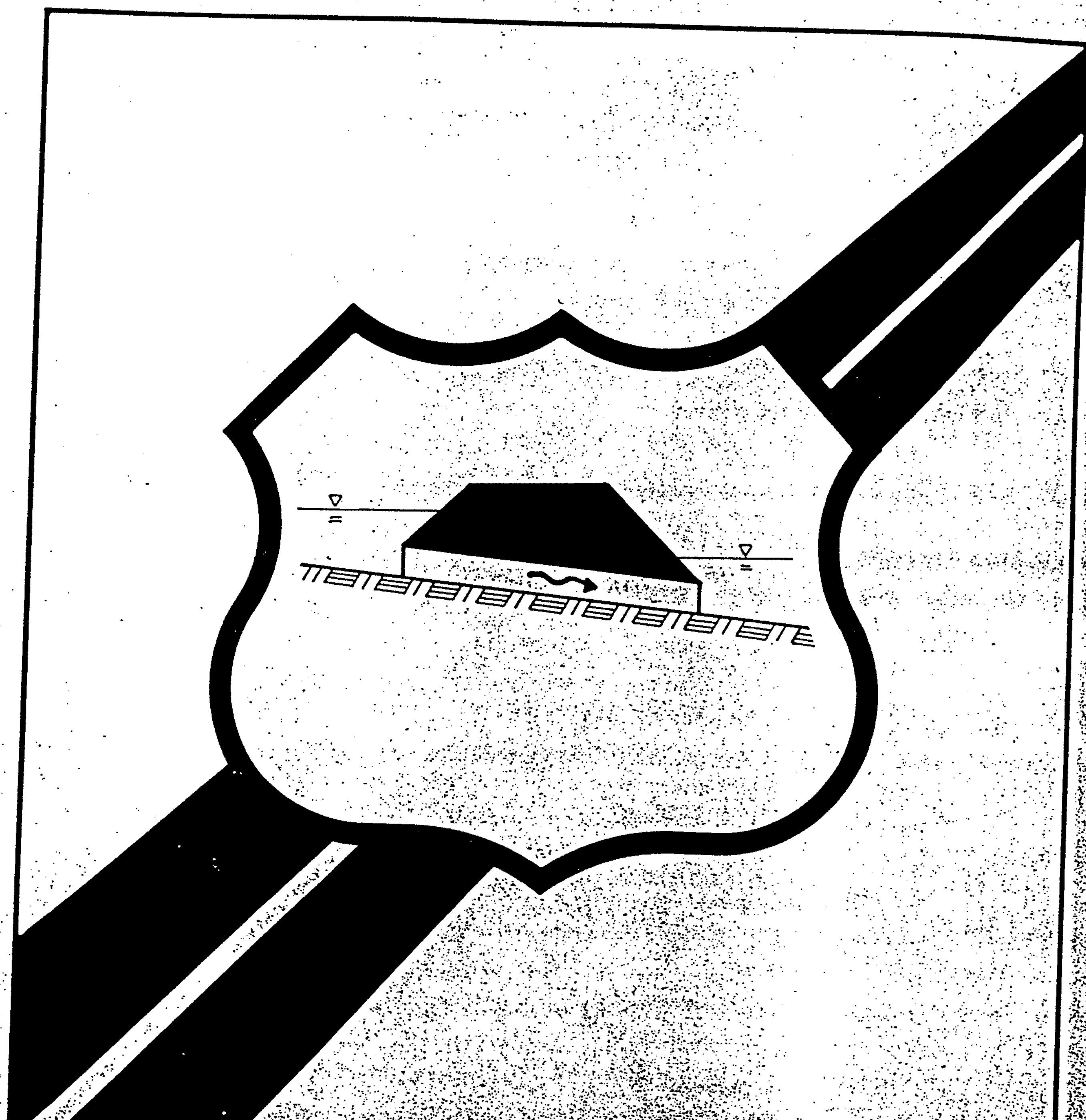
U.S. Department of Transportation
**Federal Highway Administration**

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101

Hydraulic Design Series No. 5

Report No. FHWA-IP-85-15

September 1985



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## APPENDIX A
## DESIGN METHODS AND EQUATIONS

A. Introduction.

This appendix contains explanations of the equations and methods used to develop the design charts of this publication, where those equations and methods are not fully described in the main text. The following topics are discussed: the design equations for the unsubmerged and submerged inlet control nomographs, the dimensionless design curves for culvert shapes and sizes without nomographs, and the dimensionless critical depth charts for long span culverts and corrugated metal box culverts.

B. Inlet Control Nomograph Equations.

The design equations used to develop the inlet control nomographs are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration). Seven progress reports were produced as a result of this research. Of these, the first and fourth through seventh reports dealt with the hydraulics of pipe and box culvert entrances, with and without tapered inlets. (4,7 to 10) These reports were one source of the equation coefficients and exponents, along with other references and unpublished FHWA notes on the development of the nomographs. (56,57)

The two basic conditions of inlet control depend upon whether the inlet end of the culvert is or is not submerged by the upstream headwater. If the inlet is not submerged, the inlet performs as a weir. If the inlet is submerged, the inlet performs as an orifice. Equations are available for each of the above conditions.

Between the unsubmerged and the submerged conditions, there is a transition zone for which the NBS research provided only limited information. The transition zone is defined empirically by drawing a curve between and tangent to the curves defined by the unsubmerged and submerged equations. In most cases, the transition zone is short and the curve is easily constructed.

Table 8 contains the unsubmerged and submerged inlet control design equations. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with two correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but form (2) is easier to apply and is the only documented form of equation for some of the inlet control nomographs. Either form of unsubmerged inlet control equation will produce adequate results.

The constants for the equations in table 8 are given in table 9. Table 9 is arranged in the same order as the design nomographs in appendix D, and provides the unsubmerged and submerged equation coefficients for each shape, material, and edge configuration. For the unsubmerged equations, the form of the equation is also noted.

**Table 8**
**Inlet control design equations.**

UNSUBMERGED [1]

Form (1) $$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^M - 0.5S\ ^{2} \qquad (26)$$

Form (2) $$\frac{HW_i}{D} = K\left[\frac{Q}{AD^{0.5}}\right]^M \qquad (27)$$

SUBMERGED [3]

$$\frac{HW_i}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^2 + Y - 0.5S\ ^{2} \qquad (28)$$

Definitions

| | |
|---|---|
| $HW_i$ | Headwater depth above inlet control section invert, ft |
| D | Interior height of culvert barrel, ft |
| $H_c$ | Specific head at critical depth ($d_c + V_c^2/2g$), ft |
| Q | Discharge, $ft^3/s$ |
| A | Full cross sectional area of culvert barrel, $ft^2$ |
| S | Culvert barrel slope, ft/ft |
| K,M,c,Y | Constants from table 9 |

NOTES:
1. Equations (26) and (27) (unsubmerged) apply up to about $Q/AD^{0.5}$ 3.5.
2. For mitered inlets use +0.7S instead of -0.5S as the slope corre tion factor.
3. Equation (28) (submerged) applies above about $Q/AD^{0.5} = 4.0$.

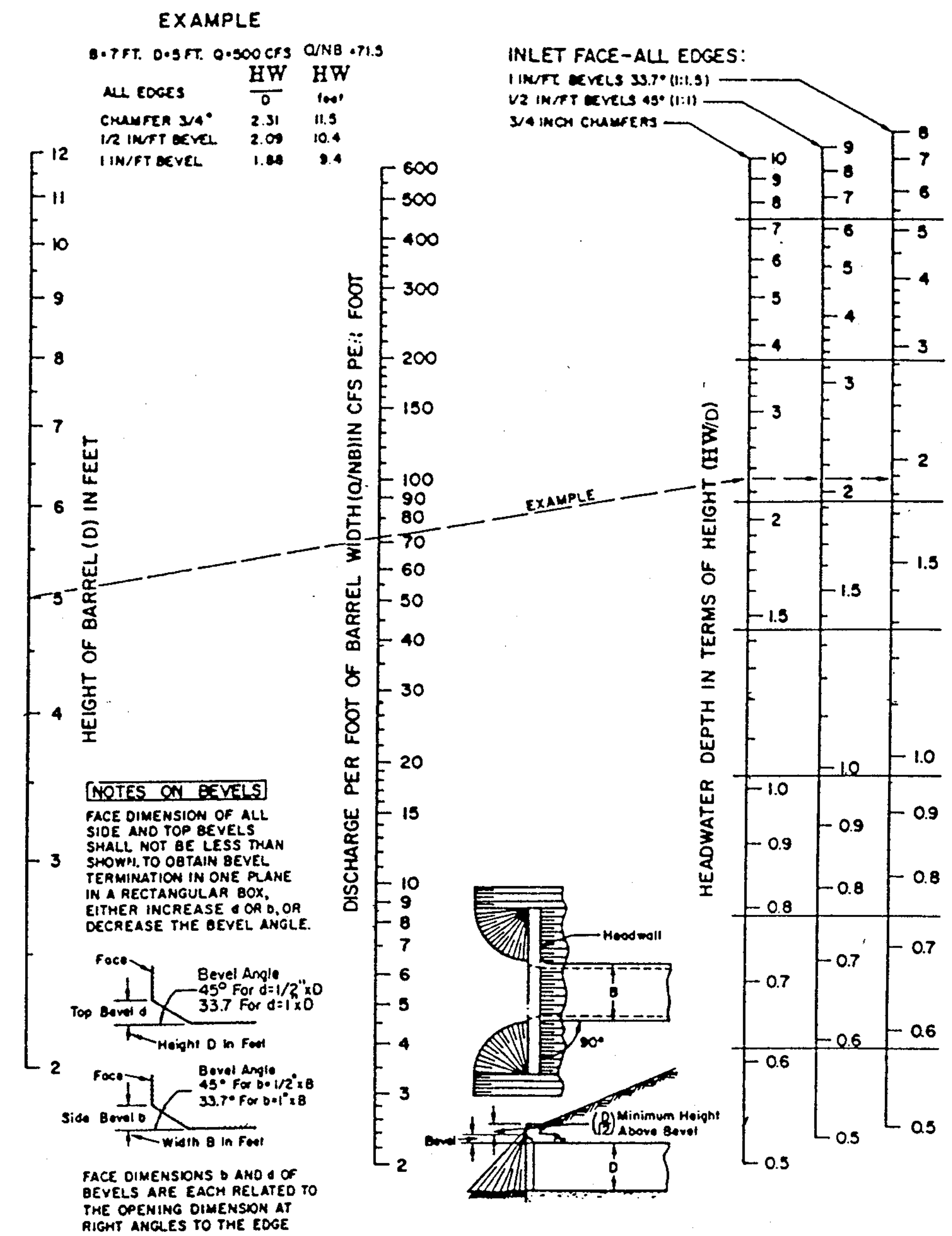


Table 9

Constants for inlet control design equations.

| CHART NO. | SHAPE AND MATERIAL | NOMOGRAPH SCALE | INLET EDGE DES[illegible]TION | UNSUBMERGED EQUATION FORM | UNSUBMERGED K | UNSUBMERGED M | SUBMERGED c | SUBMERGED Y | References |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Circular Concrete | 1 | Square edge w/headwall | 1 | 0.0098 | 2.0 | 0.0398 | 0.67 | (56) (57) |
| | | 2 | Groove end w/headwall | | .0078 | 2.0 | .0292 | .74 | (56) (57) |
| | | 3 | Groove end projecting | | .0045 | 2.0 | .0317 | .69 | (56) (57) |
| 2 | Circular CMP | 1 | Headwall | 1 | .0078 | 2.0 | .0379 | .69 | (56) (57) |
| | | 2 | Mitered to slope | | .0210 | 1.33 | .0463 | .75 | (57) |
| | | 3 | Projecting | | .0340 | 1.50 | .0553 | .54 | (57) |
| 3 | Circular | A | Beveled ring, 45° bevels | 1 | .0018 | 2.50 | .0300 | .74 | (57) |
| | | B | Beveled ring, 33.7° bevels* | | .0018 | 2.50 | .0243 | .83 | (57) |
| 8 | Rectangular Box | 1 | 30° to 75° wingwall flares | · | .026 | 1.0 | .0385 | .81 | (56) |
| | | 2 | 90° and 15° wingwall flares | 1 | .061 | 0.75 | .0400 | .80 | (56) |
| | | 3 | 0° wingwall flares | | .061 | 0.75 | .0423 | .82 | (8) |
| 9 | Rectangular Box | 1 | 45° wingwall flare d=.0430 | 2 | .510 | .667 | .0309 | .80 | (8) |
| | | 2 | 18° to 33.7° wingwall flare d=.0830 | | .486 | .667 | .0249 | .83 | (8) |
| 10 | Rectangular Box | 1 | 90° headwall w/3/4" chamfers | 2 | .515 | .667 | .0375 | .79 | (8) |
| | | 2 | 90° headwall w/45° bevels | | .495 | .667 | .0314 | .82 | (8) |
| | | 3 | 90° headwall w/33.7° bevels | | .486 | .667 | .0252 | .865 | (8) |
| 11 | Rectangular Box | 1 | 3/4" chamfers; 45° skewed headwall | 2 | .522 | .667 | .0402 | .73 | (8) |
| | | 2 | 3/4" chamfers; 30° skewed headwall | | .533 | .667 | .0425 | .705 | (8) |
| | | 3 | 3/4" chamfers; 15° skewed headwall | | .545 | .667 | .04505 | .68 | (8) |
| | | 4 | 45° bevels; 10°-45° skewed headwall | | .498 | .667 | .0327 | .75 | (8) |
| 12 | Rectangular Box 3/4" chamfers | 1 | 45° non-offset wingwall flares | 2 | .497 | .667 | .0339 | .803 | (8) |
| | | 2 | 18.4° non-offset wingwall flares | | .493 | .667 | .0361 | .806 | (8) |
| | | 3 | 18.4° non-offset wingwall flares 30° skewed barrel | | .495 | .667 | .0386 | .71 | (8) |
| 13 | Rectangular Box Top Bevels | 1 | 45° wingwall flares - offset | 2 | .497 | .667 | .0302 | .835 | (8) |
| | | 2 | 33.7° wingwall flares - offset | | .495 | .667 | .0252 | .881 | (8) |
| | | 3 | 18.4° wingwall flares - offset | | .493 | .667 | .0227 | .887 | (8) |
| 16-19 | C M Boxes | 1 | 90° headwall | 1 | .0083 | 2.0 | .0379 | .69 | (57) |
| | | 2 | Thick wall projecting | | .0145 | 1.75 | .0419 | .64 | (57) |
| | | 3 | Thin wall projecting | | .0340 | 1.5 | .0496 | .57 | (57) |

# CHART 10




HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

# GEOTECHNICAL ENGINEERING REPORT

NINYO & MOORE
GEOTECHNICAL AND ENVIRONMENTAL SCIENCES CONSULTANTS
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PHOENIX AZ 85040
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**INITIAL GEOTECHNICAL EVALUATION**
**EAST MARICOPA FLOODWAY**
**RITTENHOUSE DETENTION BASIN**
**MARICOPA COUNTY, ARIZONA**

**PREPARED FOR:**
Kirkham Michael Consulting Engineers
9210 North 25th Avenue, Suite 195
Phoenix, Arizona 85021
&
Flood Control District of Maricopa County
2801 West Durango Street
Phoenix, Arizona 85009-6399

**PREPARED BY:**
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December 27, 2001
Project No. 600198001



December 27, 2001
Project No. 600198001

Mr. Barry Ling, P.E.
Kirkham Michael Consulting Engineers
9210 North 25th Avenue, Suite 195
Phoenix, Arizona 85021

Subject: Initial Geotechnical Evaluation
East Maricopa Floodway
Rittenhouse Detention Basin
Maricopa County, Arizona

Dear Mr. Ling:

In accordance with our proposal dated May 7, 2001 and your authorization to proceed dated June 7, 2001, Ninyo & Moore has performed an Initial Geotechnical Evaluation for the above-referenced site. The attached report represents our methodology, findings, conclusions, and recommendations regarding the geotechnical conditions at the project site.

We appreciate the opportunity to be of service to you during this phase of the project. If you have any questions or comments regarding this report, please call at your convenience.

Sincerely,
**NINYO & MOORE**

Steven D. Nowaczyk

Steven D. Nowaczyk, P.E.
Senior Project Engineer

Robert W. McMichael, P.E.
Manager/Chief Engineer

SDN/RM/LLG/avv

Distribution: (2) Addressee

Registered Professional Engineer (Civil)
CERTIFICATE NO.
34866
STEVEN D.
NOWACZYK
Date Signed 12-27-01
ARIZONA, U.S.A.

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## 1. INTRODUCTION

In accordance with our proposal dated May 7, 2001 and your authorization to proceed dated June 7, 2001, we have performed a geotechnical evaluation for the Rittenhouse Detention Basin project located in eastern Maricopa County, Arizona. The purpose of our evaluation was to assess the subsurface conditions at the project site in order to formulate geotechnical recommendations for design and construction of the new basin. This report presents the results of our evaluation and our geotechnical conclusions and recommendations regarding the proposed construction.

## 2. SCOPE OF SERVICES

The scope of our services for the project generally included the following:

- Reviewing readily available aerial photographs and published geologic literature, including maps and reports pertaining to the project site and vicinity.
- Marking-out the boring locations and notifying Arizona Blue Stake of the boring locations prior to drilling.
- Drilling, logging, and sampling 17 small-diameter exploratory borings to depths of about 16 to 26 feet below ground surface (bgs). The boring logs are presented in Appendix A.
- Excavating, logging, and sampling three test pit explorations to depths of about 8.5 to 12 feet bgs. The test pit logs are also presented in Appendix A.
- Performing four field infiltration tests at the anticipated bottom-of-basin level, in general accordance with the City of Chandler method. The results are presented in Appendix C.
- Installing three piezometers in boreholes that were drilled along the East Maricopa Floodway (EMF).
- Performing laboratory tests on selected samples obtained from the borings to evaluate in-situ moisture content and dry density, grain size analysis, Atterberg limits, hydro-consolidation (swell/collapse) tests, maximum density/optimum moisture relationship, expansion index, agronomic testing (growability), permeability tests, unconsolidated undrained Triaxial Compression tests and corrosivity characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the laboratory testing are presented on the logs in Appendix A and/or the laboratory sheets present in Appendix B. The results from the agronomic testing are presented in Appendix D.





- Preparing this initial report that presents our findings, conclusions, and recommendations regarding the design and construction of the new basin.

## 3. SITE DESCRIPTION

Much of the project site is located in the southeast quarter of Section 36, Township 1 South, Range 6 East; however, a small portion of the site is located in the northeast quarter of Section 1, Township 2 South, Range 6 East. The project area covers about 160 acres of land and is situated in the town of Gilbert, Arizona. The project area is bounded by Power Road to the east, Rittenhouse Channel to the southwest, and the EMF to the northwest, and is depicted on the Site Location Map (Figure 1).

At the time of our evaluation, the project site was vacant. Farming apparently occurred on the site in the past, particularly in the central and northern portions. Scattered trees, small brush, and weeds were observed during our site visits. In addition, several unpaved roads crossed the site indiscriminately, except for an unpaved road that appeared to coincide with the alignment of Pecos Road in the southern portion of the project site. Some scattered piles of soil were also observed. We understand that some spoils from the original construction of the EMF were spread-out over the northern portion of this site.

According to the *Higley, Arizona 7.5-Minute USGS Topographic Quadrangle Map (1981),* the project area lies at an average elevation of roughly 1,325 feet relative to mean sea level (MSL). Based on the information from these quadrangle maps and the topographic information we obtained from your office, it appears the project area slopes very gently from the southeast to the northwest, toward the EMF, with a vertical relief of about 13 feet.

Two aerial photographs were reviewed for this project. A 1967 photograph from the *USDA Soil Survey of Eastern Maricopa and Northern Pinal Counties, Arizona* shows row crops planted near the central portion of the site. In addition, some unidentifiable activity was observed near the southern tip of the project area. A series of 1999 aerial photographs from *Landiscor's Phoenix Real Estate Photo Book* show the project area similar to its current condition. Our evaluation of the aerial photographs and visual reconnaissance did not indicate any large disturbed areas that might be indicative of past development or filling.

## 4. PROPOSED CONSTRUCTION

The project generally includes the construction of a new detention basin along the southeast side of the EMF, from Power Road to Rittenhouse Channel. The basin will collect stormwater during large storm events, retain the water for up to 36 hours, and then discharge it back into the EMF. The depth of the basin will roughly match the depth of the EMF, which is situated at about elevation 1,310 feet above MSL. Consequently, the excavation needed to create the basin area will extend about 10 to 20 feet bgs.

A 1,500-foot long, concrete side weir will be constructed near the northwest corner of the basin. This weir will enable stormwater to enter the basin from the EMF. The weir crest elevation is tentatively planned to be at about 1,315 feet above MSL. To allow the water to transfer back into the EMF, an outfall is planned beneath the southern-most portion of the side weir, about 2,100 feet southwest of the Power Road intersection with the EMF. This outfall is proposed to consist of multiple box culverts that will be incorporated structurally into the side weir. Based on our conversations with you and the Flood Control District of Maricopa County, we understand that the basin is not considered to be a jurisdictional dam (as defined by the Arizona Department of Water Resources) because the water that is retained will be situated below to existing ground surface.

The side slopes around the perimeter of the basin are proposed to be construction with a 4 vertical to 1 horizontal slope. The land use within the new basin is tentatively planned to be a golf course, with other recreational amenities. A small portion of the site located on the west side of Power Road, about 2,600 feet south of the Power Road intersection with the EMF, will not be excavated. This area is reserved for future golf course operations.





## 5. FIELD EXPLORATION

### 5.1. Soil Borings

Ninyo & Moore conducted a subsurface evaluation consisting of soil boring excavations from July 5 through 16, 2001 in order to evaluate the existing subsurface conditions and to collect soil samples for laboratory testing. Specifically, our evaluation consisted of the excavating, logging, and sampling of 17 small-diameter borings. The borings were drilled using a CME-75 truck-mounted drill rig. Of these borings, five were drilled along the EMF perimeter (denoted as RH-1 through RH-5), one was drilled adjacent to the Kinder Morgan property (denoted as RH-6), two were drilled along the Rittenhouse Channel perimeter (denoted as RH-7 and RH-8), five were drilled along the Power Road perimeter (denoted as RH-9 through RH-13), and four were drilled within the new basin area (denoted as RH-14 through RH-17). Bulk and relatively undisturbed soil samples were collected at selected intervals. Detailed descriptions of the soils encountered are presented in the boring logs in Appendix A.

The ground surface elevations and the lateral locations at each boring were measured by Consultant Engineering, Inc of Phoenix, Arizona after the drilling was finished. The elevations of each boring location are presented on the logs. The general locations of the borings are denoted on the Soil Boring Location Map (Figure 2).

### 5.2. Test Pits

Ninyo & Moore conducted a supplemental subsurface evaluation consisting of the excavation of three test pits from November 26 through 27, 2001 in order to further evaluate the existing subsurface conditions. The test pits were excavated along the EMF perimeter using a Ford 555E backhoe. Detailed descriptions of the soils encountered are presented in the boring logs in Appendix A, and the general locations of the test pits are denoted on Figure 2.

### 5.3. Piezometer Monitoring Wells

In order to monitor surface water seepage from the EMF after a large rain event, piezometer groundwater monitoring wells were installed in three of the boreholes after the boring was finished. Specifically, the piezometers were installed in borings RH-1, RH-3, and RH-5. In general, the bottom half of the wells consisted of screened PVC and the top half was solid. The annuli around the wells were backfilled with permeable sand and grouted near the surface. The tops of the wells were capped with an above-ground protective casing.

No substantial rainfall event occurred during our study period and no meaningful readings were taken; however, the wells were left in-place. Consequently, if a heavy rain event occurs during the final design phase, the piezometers may be read and the information could be useful.

### 5.4. Field Percolation Tests

In order to provide a preliminary evaluation of the infiltration rate near the bottom of the proposed basin, Ninyo & Moore conducted four infiltration tests in general accordance with the City of Chandler Typical Detail No. C-109. These tests were performed near the central portion of the site, adjacent to borings RH-14, RH-15, RH-16, and RH-17. The procedures used consisted of the insertion of a 12-inch diameter impermeable casing into undisturbed soil, to a depth of about 15 to 17 feet bgs, followed by prewetting of the soil. The test continued after the prewetting period by refilling the casing and monitoring the drop in water level as a function of time until steady-state conditions were achieved. The results of this testing are provided in Appendix C.

### 5.5. Field Screening for Volatile Organic Compounds (VOCs)

In order to provide a preliminary screening of soil for the possible presence of volatile organic compounds (VOCs), several collected samples were tested with a photoionization detector (PID). The Mini-Rae PID was calibrated at the beginning of each sampling day with 100 ppm isobutylene span gas. A zip-lock plastic bag was partially filled with a por-





tion of each collected soil sample, sealed, and allowed to volatilize for 10 minutes. The tip of the PID was then inserted into the headspace of the plastic bag.

The highest PID reading was noted and recorded on the field boring logs and in the field notebook. No elevated VOC readings were observed during our field work.

## 6. LABORATORY TESTING

The soil samples collected from our drilling activities were transported to the Ninyo & Moore laboratory in Phoenix, Arizona for geotechnical laboratory analysis. The analysis included in-situ moisture content and dry density, grain size analysis, Atterberg limits, hydro-consolidation (swell/collapse) tests, maximum density/optimum moisture relationship, expansion index, agronomic testing (growability), permeability tests, unconsolidated undrained Triaxial Compression tests and corrosivity characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the laboratory testing are presented on the logs in Appendix A and/or the laboratory sheets present in Appendix B.

Agronomic testing consisting of the testing of primary nutrients, secondary nutrients, micro nutrients, as well as other agricultural characteristics, was performed by Fruit Growers Laboratory, Inc. of Santa Paula, California. The results of these tests, which include planting recommendations, are presented in Appendix D.

## 7. GEOLOGY AND SUBSURFACE CONDITIONS

The geology and subsurface conditions at the site are described in the following sections.

### 7.1. Geologic Setting

The project site is located in the Sonoran Desert Section of the Basin and Range physiographic province, which is typified by broad alluvial valleys separated by steep, discontinuous, subparallel mountain ranges. The mountain ranges generally trend north-south and northwest-southeast. The basin floors consist of alluvium with thickness extending to several thousands of feet.

The basins and surrounding mountains were formed approximately 10 to 13 million years ago during the mid- to late-Tertiary. Extensional tectonics resulted in the formation of horsts (mountains) and grabens (basins) with vertical displacement along high-angle normal faults. Intermittent volcanic activity also occurred during this time. The surrounding basins filled with alluvium from the erosion of the surrounding mountains as well as from deposition from rivers. Coarser-grained alluvial material was deposited at the margins of the basins near the mountains. The surficial geology of the proposed canal is described as latest Quaternary age deposits (<10,000 years old) consisting of sand and silt, with local occurrences of fine gravels and coarse deposits that contain minimal soil development (Demsey, 1989).

### 7.2. Subsurface Conditions

Our knowledge of the subsurface conditions at the project site is based on our field exploration and laboratory testing, and our understanding of the general geology of the area. The following paragraphs provide a generalized description of the materials encountered. More detailed descriptions are presented on the boring logs in Appendix A.

Stratified desert alluvium was encountered at the surface of the borings and extended to the total depth explored. The alluvium consisted of clay (CL), silt (ML), and clayey/silty sand (SC/SM). Scattered caliche nodules, filaments, and stringers were present in many of the borings. Table 1 provides an estimated breakdown of the soil types encountered in our borings within the proposed basin excavation (e.g., from the ground surface to about 10 to 20 feet bgs):

**Table 1 – Approximate Percentage of Soil Types Encountered from Ground Surface to Anticipated Bottom of Basin**

| GP/GC/GM | SP | SC/SM | ML | CL |
|---|---|---|---|---|
| 0% | 0% | 20% | 16% | 64% |





Table 2 provides a breakdown of the soil types encountered in our borings at the anticipated bottom of the basin excavation (e.g., about 10 to 20 feet bgs):

**Table 2 – Approximate Percentage of Soil Types Encountered at the Anticipated Bottom of Basin Excavation**

| GP/GC/GM | SP | SC/SM | ML | CL |
|---|---|---|---|---|
| 0% | 0% | 53% | 12% | 35% |

The geological characteristics of the surface soils within the project site generally includes the presence of a Holocene "apron" overlying an older Late Pleistocene deposit. The Holocene deposits are typically of lower density and are relatively susceptible to collapse upon wetting. Consequently, the position of the contact between the Holocene and Late Pleistocene deposits is relevant. Based on our field work and laboratory testing, we estimate that this contact ranges from about elevation 1,299 to 1,320 feet MSL. Localized variations are largely attributable to erosion of the Late Pleistocene surface.

### 7.3. Groundwater

Groundwater was not encountered in our boring or test pit excavations. Based on well data from the Arizona Department of Water Resources (ADWR), the approximate depth to groundwater is in excess of about 180 feet bgs. Groundwater levels can fluctuate due to seasonal variations, irrigation, groundwater withdrawal or injection, and other factors. In general, groundwater is not expected to be a constraint to the construction of the project; however, given the occurrence of relatively pervious zones, perched tailwater resulting from flood irrigation of cropland might be encountered.

## 8. CONCLUSIONS

Based on the results of our subsurface evaluation, laboratory testing, and data analysis, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations of this report are incorporated into the design and construction of the proposed project, as appropriate. Based on this initial study, our summary of key geotechnical considerations includes the following:

- The on-site soils consist of stratified desert alluvium with a high degree of heterogeneity and anisotropy. The soils should generally be excavatable to planned depths with conventional earthmoving construction equipment in good working condition.
- A basin side slope angle of 4 horizontal to 1 vertical is feasible from a geotechnical standpoint. Our calculations show an acceptable factor of safety against appropriate failure modes.
- Of primary concern is the possibility of cracking, piping, and/or seepage through the natural levees. These concerns were addresses in the Failure Mode Analysis (FMA) performed for this project. As a result, one of the major findings revealed was that a crack-stopper barrier (located within the levee between the basin and the EMF and Rittenhouse Channel) would alleviate several of the potential failure modes discussed.
- We recommend that the weir be supported on a zone of engineered fill that extends through the Holocene alluvium soils to older Pleistocene deposits. Based on our field work, we estimate that the contact between the Holocene and Pleistocene deposits range from about elevation 1,299 to 1,320 feet MSL at the boring locations.
- Anti-seepage devices, like seepage collars, should be used for the installation of pipes or other penetrations that cross through or beneath the levees.

## 9. RECOMMENDATIONS

The following sections present our preliminary geotechnical recommendations for the proposed basin construction. We anticipate that more detailed recommendations will result from an additional design-phase geotechnical evaluation.

### 9.1. Earthwork

The following sections provide our earthwork recommendations.

#### 9.1.1. Excavation Characteristics

Our evaluation of the excavation characteristics of the on-site materials is based on the results of 17 widely-spaced exploratory borings, three test pits excavations, our site observations, and our experience with similar materials. In our opinion, excavation of the





on-site materials can generally be accomplished to the anticipated basin depth with conventional earthmoving equipment in good operating condition. However, scattered caliche nodules, filaments, and stringers were encountered in many of the borings, which may be somewhat more time-consuming to excavate. This cementation predominates in the older Pleistocene deposits, which were encountered below roughly elevation 1,299 to 1,320 feet MSL.

We recommend that trenches and excavations be designed and constructed in accordance with OSHA regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on a description of the soil types encountered. Trenches greater than 20 feet deep should be designed by the Contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that the OSHA soil classification for the encountered alluvial soil be considered as Type C.

#### 9.1.2. Grading, Fill Placement, and Compaction

Vegetation and debris from the clearing operation should be removed from the site and disposed of at a legal dumpsite. Demolition debris should be removed from the site and disposed of at a legal dumpsite. Obstructions that extend below finish grade, if present, should be removed and the resulting holes filled with compacted soil.

The geotechnical consultant should carefully evaluate areas of soft or wet soils prior to placement of fill or other construction. Drying or overexcavation and replacement of such materials may be anticipated.

We recommend that new fill be placed in horizontal lifts approximately 8 inches in loose thickness and compacted by appropriate mechanical methods, to 95 percent or more relative compaction, in accordance with ASTM D 698-91 at a moisture content within two percent of its above optimum.

Based on the laboratory tests we performed, it appears that an earthwork (shrinkage) factor of 10 to 25 percent is appropriate for the on-site soils within the basin area. This shrinkage factor range represents an average of the material tested. Potential bidders should consider this in preparing estimates and should review the available data to make their own conclusions regarding excavation conditions.

Although not apparent in our logs, because much of this site was used for farming, the top 6 to 12 inches may contain some organics. This layer may need to be segregated during construction and could be reused in non-structural area of the site.

### 9.1.3. Reuse of Excavated Material as Borrow

The composition of the soils that will likely be excavated for construction of the basin was outlined in Section 7.2. In addition to the index testing (grain size analysis and Atterberg limits) that was done to classify these soils, we also performed Expansion Index and corrosivity tests as a means to evaluate these soils for potential reuse. Table 3 outlines the results of these tests. Please note that given the very large volume of soil to be excavated and the heterogeneous nature of the natural soils, wider variations in soil characteristics than suggested by these results are likely.

**Table 3 – Summary of Expansion Index and Corrosivity Test Results**

| Sample Location | Sample Depth (ft) | Expansion Index | pH | Resistivity (ohm-cm) | Water-Soluble Sulfate Content in Soil (%) | Chloride Content (ppm) |
|---|---|---|---|---|---|---|
| RH-6 | 0-2 | 18 | -- | -- | -- | -- |
| RH-12 | 12-15 | 0 | -- | -- | -- | -- |
| RH-14 | 0-5 | 6 | 7.8 | 726 | 0.002 | 55.6 |
| RH-16 | 12-15 | 7 | 8.7 | 2,046 | 0.006 | 73.0 |

The Expansion Index test is used to evaluate the swell or expansion potential of a remolded soil sample that is inundated with water. Based on Uniform Building Code (UBC) Standard No. 18-2, an Expansion Index from 0 to 20 indicates a very low expansion potential, 21 to 50 indicates a low expansion potential, 51 to 90 indicates a medium expansion potential, 91 to 130 indicates a high expansion potential, and 130 or above





indicates a very high expansion potential. The soils that we tested exhibited a very low expansion potential.

The pH and minimum electrical resistivity tests were performed in general accordance with Arizona Test 236b, while sulfate and chloride tests were performed in accordance with Arizona Test 733 and 722, respectively. The soil pH values ranged from 7.8 to 8.7, which is considered to be alkaline. The minimum electrical resistivity measured in the laboratory varied from 726 to 2,046 ohm-cm, which is considered to be corrosive to ferrous materials. The chloride content of the sample tested ranged from about 56 to 73 ppm, which is also considered to be corrosive to ferrous materials.

Based on the UBC criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight (0 to 1,000 ppm), and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight (1,000 to 2,000 ppm). The potential for sulfate attack is severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight (2,000 to 20,000 ppm), and very severe for water-soluble sulfate contents over 2.00 percent by weight (20,000 ppm). The soluble sulfate content of the soil samples tested ranged from 0.002 to 0.006 percent, which represents a negligible sulfate exposure for concrete.

### 9.1.4. Imported Fill Material

Imported fill in contact with ferrous materials or concrete, if utilized, should consist of clean, granular material with a very low or low expansion potential. Import material that is in contact with buried ferrous materials or concrete should also have low corrosion potential (minimum resistivity greater than 2,000 ohm-cm or the average value for the site, chloride content less than 25 parts per million [ppm], and soluble sulfate content of less than 0.1 percent). The geotechnical consultant should evaluate such materials and details of their placement prior to importation.

### 9.2. Levee Stability and Seepage

The proposed construction of the new basin will create a natural levee along the perimeter of the basin, specifically along the EMF and the Rittenhouse Channel. Levees are usually constructed with select materials that are placed in a controlled manner and compacted to a specified density. For seepage and piping considerations, constructed levees will ordinarily be zoned and may contain internal drainage, and the embankment foundations are prepared with cut-offs extending below the embankment.

The composition of these natural levees will be highly heterogeneous and anisotropic, and could be subject to differential settlements, cracking, piping and/or seepage concerns. Although not disclosed in our limited sampling program, the natural levees and their foundations may contain defects such as desiccation cracks, open graded channels, etc. The following sections of the report address construction considerations with regards to the natural levees that will be constructed for this project and also address the basin infiltration that may be expected.

Due to the infrequent and transient nature of water storage and flow in the abutting channels, the embankment soils, constructed as proposed, will remain dry and (in some cases) brittle until a wetting front passes through during flood events. Given the short impoundment time, seepage through embankments is not expected to reach steady-state conditions.

#### 9.2.1. Side Slope Stability

Based on our conversations with your office and the conceptual plans we were given, we understand that the preliminary design of the side slopes around the perimeter of the basin calls for a 4 (horizontal) to 1 (vertical) slope. We performed preliminary slope stability analyses on a typical embankment section with this slope. The stability analyses were done using the computer program PCSTABL6H, which is a static and pseudostatic stability program using Bishop's modified circular failure surfaces. Based on the results of this analysis, we have calculated a factor of safety against failure in excess of 2.0. In determining this factor of safety, we assumed very conservative embankment soil parameters and a total stress analysis. Because saturated conditions





are not anticipated (except for the faces of the levees), rapid drawdown stability scenarios have been ruled out as highly unlikely.

On the basis of these analyses, we believe that the proposed 4:1 slope is feasible and safe from a geotechnical standpoint. A graphical representation of this slope stability analysis is given in Figure 3.

#### 9.2.2. Piping and Seepage

Because these natural levees will be constructed of native soils that are highly heterogeneous and not placed in a controlled manner, differential settlements, desiccation cracking, piping and seepage from the basin to the EMF and Rittenhouse Channel (or vice versa) are major design considerations. To better understand these and other potential risks associated with this type of construction, a failure mode assessment (FMA) was conducted for this project.

The outcome of this FMA will be summarized in a Failure Mode Report, which will be prepared by Kirkham Michael Consulting Engineers. One of the major findings revealed in this process was that a crack-stopper barrier (located within the levee between the basin and the EMF and Rittenhouse Channel) would alleviate several of the potential failure modes discussed, particularly those associated with differential settlement, cracking, piping and seepage. Detailed discussions and recommendations for crack-stopper barrier construction, including cost analysis and comparisons, will be provided in the final geotechnical report.

#### 9.2.3. Self-Weight Settlement of Levee and Basin Floor

As mentioned earlier, the project site is generally underlined with a Holocene "apron" overlying an older Late Pleistocene deposit. The Holocene deposits are typically of lower density and are relatively susceptible to collapse, under their own self-weight, upon wetting. If this settlement occurs under or within the levee, cracks may develop. As with the piping and seepage concerns discussed in the previous section, defensive measures like a crack-stopper barrier may alleviate this situation as well.

In addition, self-weight settlement within the basin may also occur, with the cracks that develop generally limited to the basin floor. As a result, a low spot could be created and the capacity of the basin may be locally affected. However, the overall performance of the basin as a result of this potential localized settlement will most likely not be compromised.

#### 9.2.4. Basin Base Infiltration

As mentioned earlier, four field percolation tests were performed for this basin. The tests were located within the central portion of the proposed basin area and extended 15 to 17 feet bgs. Table 4 summarizes these results of these percolation tests.

**Table 4 – Summary of Percolation Tests Within Rittenhouse Basin**

| Approximate Test Location | Test Depth (ft) | Average Percolation Rate ($ft^3/hr/ft^2$) | Soil Type at Test Depth |
|---|---|---|---|
| RH-14 | 15 | 0.08 | SC |
| RH-15 | 15 | 2.09 | SC |
| RH-16 | 15 | 0.88 | CL |
| RH-17 | 17 | 1.31 | SM |

The measured values should be viewed as highly approximate since soil permeability is among the more variable quantities used in soil mechanics. A conservative approach to seepage rates is recommended.

### 9.3. Side Weir and Outlet Works

As mentioned earlier, we understand that a 1,500-foot long side weir will be constructed near the northwest corner of the basin. This weir will enable stormwater to enter the basin from the EMF after it reaches about elevation 1,315 feet above MSL. To allow the water to transfer back into the EMF, an outfall is planned near the southern-most end of the side weir, about 2,100 feet southwest of the Power Road intersection with the EMF. This outfall is proposed to consist of multiple box culverts that will be incorporated structurally into the side weir.





In addition, we understand the weir will be concrete lined on both sides. The EMF side will be slightly battered toward the basin, and the basin side will be stepped. A plunge pool, extending 4 feet below the bottom of the basin, will be provided near the toe of the weir on the basin side. The plunge pool will be lined with rip-rap to mitigate erosion.

The conceptual drawings that we received also show two cut-off walls, located on either side of the weir and extending 6 feet below the bottom of the basin. We understand that these walls are proposed to discourage undermining of the side weir by water flow, but will also act in some capacity as piping and seepage control.

#### 9.3.1. Foundation Preparation

As part of our scope of work, the characteristics of the foundation soils supporting the new levees were evaluated. Particularly, the extent of a Holocene "apron" overlying the older Late Pleistocene deposits was considered. The Holocene deposits are typically of lower density and are relatively susceptible to collapse upon wetting. Consequently, the position of the contact between the Holocene and Late Pleistocene deposits is relevant.

In our evaluation of the Holocene/Late Pleistocene contact, the qualitative description of cementation stage proposed by Machette (1985) was used in conjunction with that proposed by Beckwith and Hanson (1982). The various stages of cementation are denoted on the logs in Appendix A. Based on our field work and laboratory testing, we estimate that this contact ranges from about elevation 1,299 to 1,320 feet MSL. Localized variations are largely attributable to erosion of the Late Pleistocene surface.

Relevant geologic information was provided during the FMA workshop. As a result, the presence of Holocene soils below the weir and the potential collapse of these soils was considered a potential failure mode and also a major finding. Consequently, it was recommended that the Holocene soils located below the weir should be removed and replaced with compacted, engineered fill.

As mentioned earlier, the thickness of the Holocene apron varies considerably across the project site. Therefore, the anticipated depth of removal for the construction of the weir should be further evaluated during the design phase of this project. This further evaluation should consist of more closely-spaced borings and/or test pits and additional laboratory testing.

Engineered fill should be placed in horizontal lifts approximately 8 inches in loose thickness and compacted by appropriate mechanical methods, to 95 percent or more relative compaction, in accordance with ASTM D 698-91 at a moisture content within two percent of its optimum moisture content. Selected low permeability, on-site soils could be reused for this purpose.

#### 9.3.2. Pipe Penetrations

An embankment breech can result from inadequately designed or constructed pipelines, utility conduits, or culverts (hereafter referred to as pipes) located beneath or within levees. During high water, seepage tends to concentrate along the outer surface of pipes resulting in piping (potential washing out) of fill or foundation material. Seepage may also occur because of leakage from the pipe. Consequently, we recommend that anti-seepage devices be employed to mitigate piping or erosion along the outside wall of the pipe. The term "anti-seepage device" usually refers to metal diaphragms or concrete collars that extend from the pipe into the backfill material. The diaphragms and collars are often referred to as "seepage rings". To reduce increased piping potential, great care should be taken when compacting backfill around these seepage rings.

In addition, the pipe should have adequate strength to withstand the applied earth loads. Consideration should also be given to live loads imposed from equipment during construction and the loads from traffic and maintenance equipment after the levee construction.





The pipe joints should be selected to accommodate movements resulting from foundation or fill settlement. In addition, the pipe joints, as well as the pipe itself, should be watertight.

#### 9.3.3. Concrete

As mentioned previously, the results of the sulfate content laboratory tests indicate the site soils present a negligible sulfate exposure to concrete. In accordance with Table 19-A-3 of the 1994 UBC, we believe that Type II cement can be used for the construction of concrete structures at this site. However, due to potential uncertainties as to the use of reclaimed irrigation water, or topsoil that may contain higher sulfate contents, sulfate-resistant cement, pozzalon, or admixtures may be considered.

The concrete should have a water-cement ratio no greater than 0.5 by weight for normal weight aggregate concrete. From a quality standpoint, a 28-day compressive strength of 4,000 psi or higher is desirable because it will improve concrete durability.

### 9.4. Pre-Construction Conference

We recommend that a pre-construction conference be held. Representatives of the owner, the civil engineer, the geotechnical consultant, and the contractor should be in attendance to discuss the project plans and schedule. Our office should be notified if the project description included herein is incorrect or if the project characteristics are significantly changed.

### 9.5. Construction Observation and Testing

During construction operations, we recommend that a qualified geotechnical consultant perform observation and testing services for the project. These services should be performed to evaluate exposed subgrade conditions, including the extent and depth of overexcavation if loose soils are encountered during construction, to evaluate the suitability of proposed borrow materials for use as fill, and to observe placement and test compaction of fill soils. We believe the design geotechnical consultant should be retained for construction services. However, if another geotechnical consultant is selected to perform observation and testing services for the project, we request that the selected consultant provide a letter to the owner, with a copy to Ninyo & Moore, indicating that they fully understand our recommendations and that they are in full agreement with the recommendations contained in this report. Qualified subcontractors utilizing appropriate techniques and construction materials should perform construction of the proposed improvements.

## 10. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only and may not provide sufficient data to prepare an accurate bid by some contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.





Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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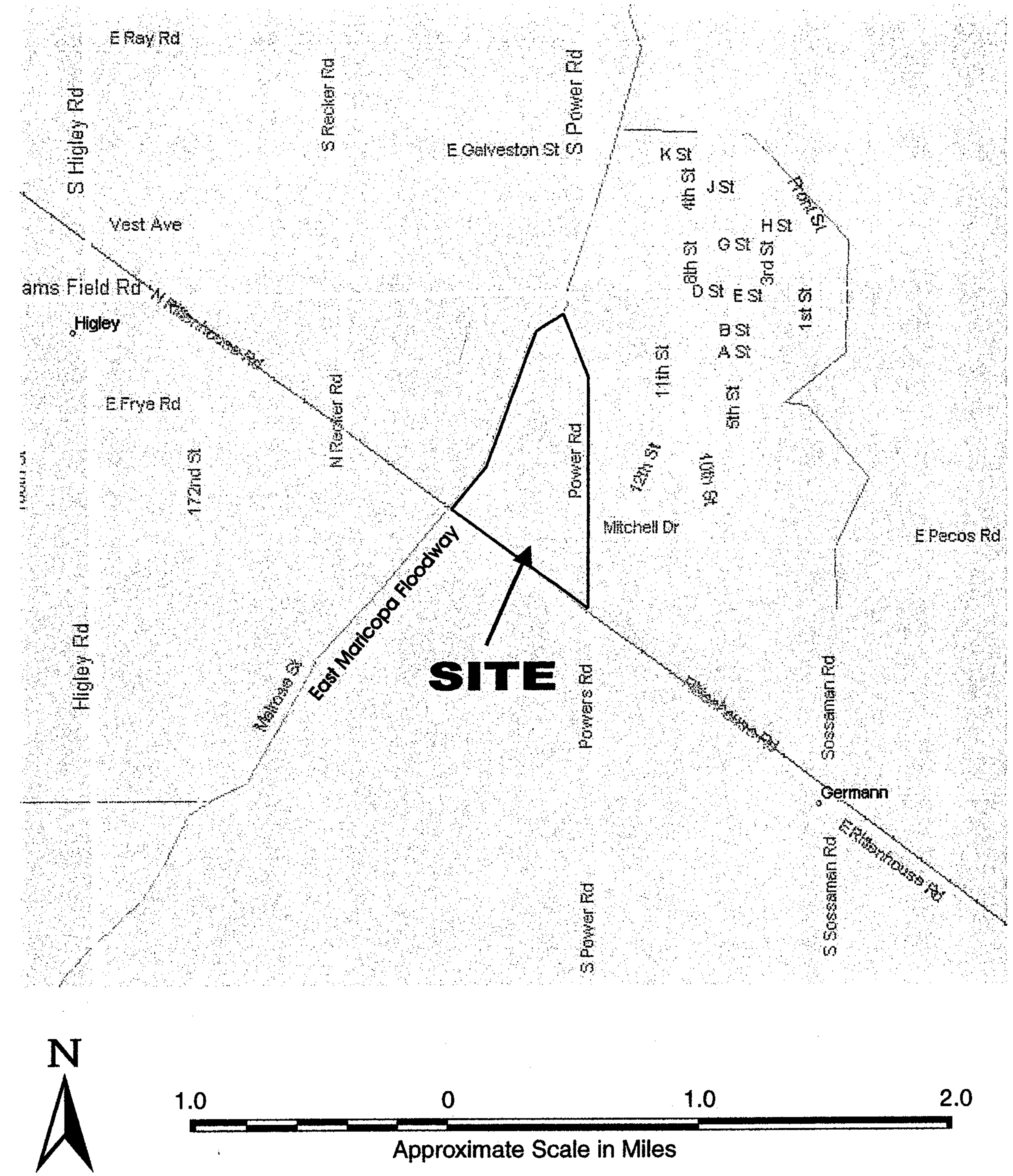


Reference: Microsoft Streets98, 1998.

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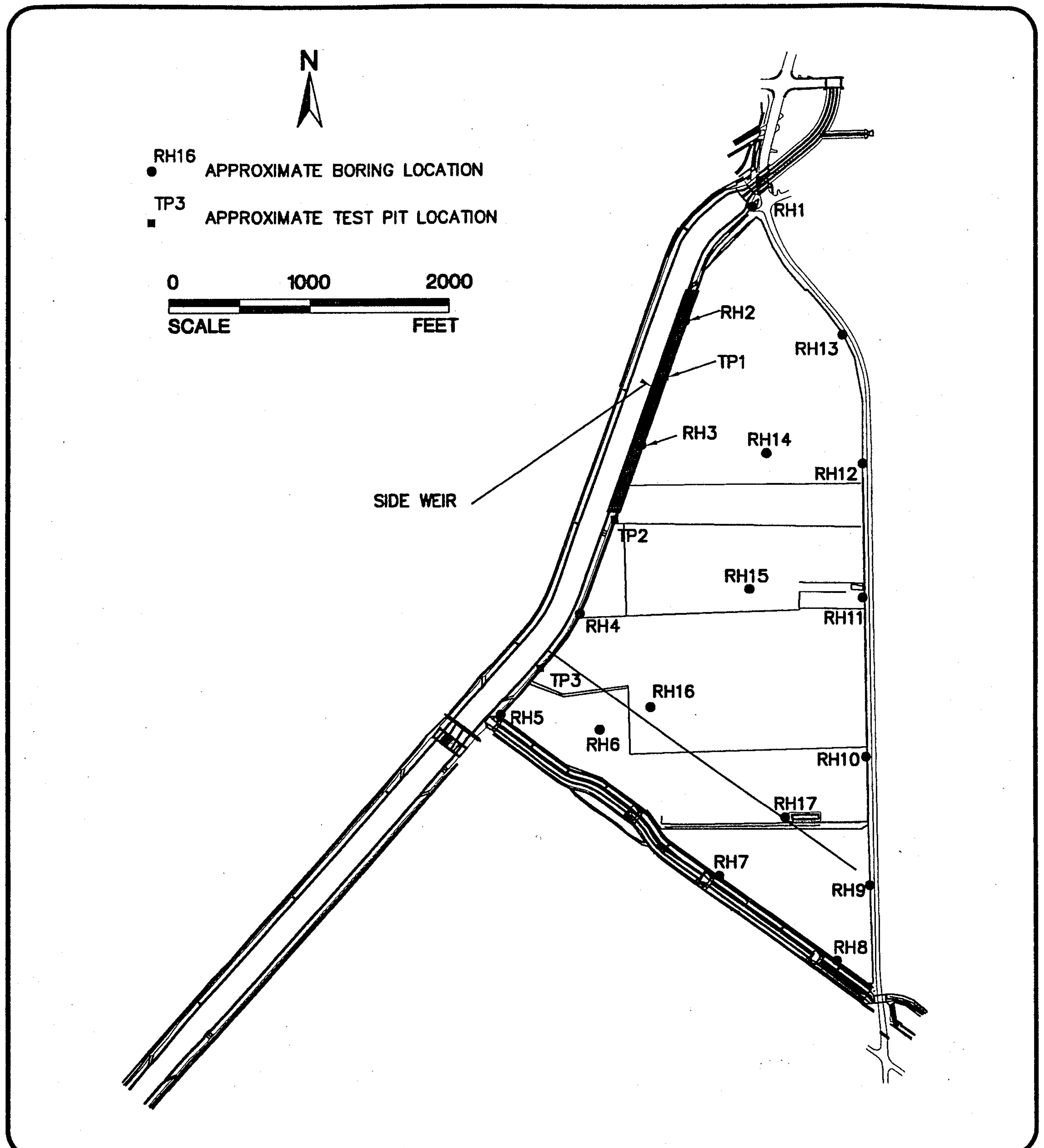
SITE LOCATION MAP

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | 1 |

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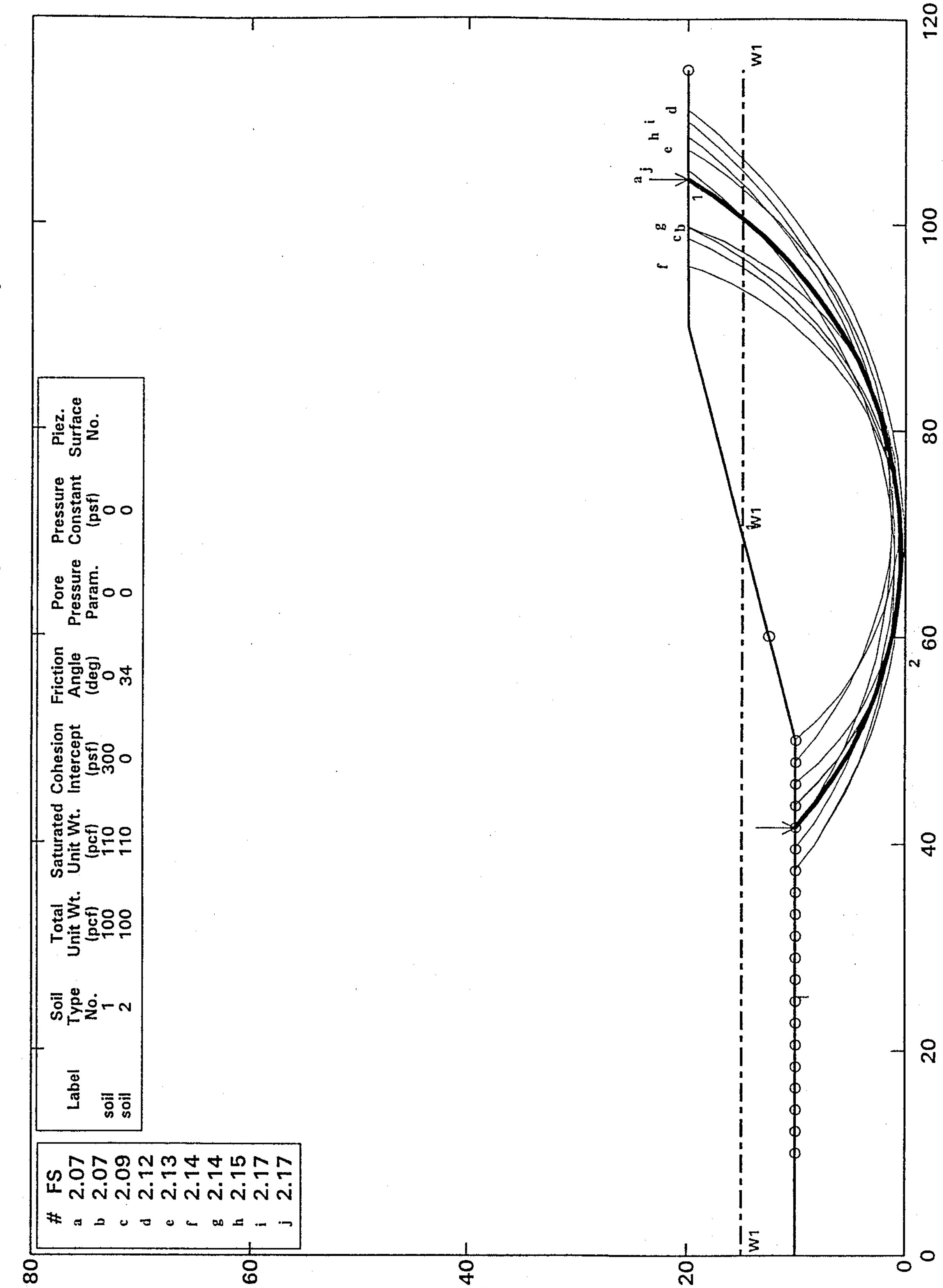
BORING/TEST PIT LOCATION MAP

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | 2 |

Figure 3: Slope Stability Analysis of Typical Embankment

Ten Most Critical. C:EMF-TYP.PLT By: Curt 09-28-01 3:52pm




| Label | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| soil | 1 | 100 | 110 | 300 | 0 | 0 | 0 | |
| soil | 2 | 100 | 110 | 0 | 34 | 0 | 0 | |

| # | FS |
|---|---|
| a | 2.07 |
| b | 2.07 |
| c | 2.09 |
| d | 2.12 |
| e | 2.13 |
| f | 2.14 |
| g | 2.14 |
| h | 2.15 |
| i | 2.17 |
| j | 2.17 |

Factors Of Safety Calculated By The Modified Bishop Method

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## APPENDIX A

## BORING/TEST PIT LOGS

**Field Procedure for the Collection of Disturbed Samples**

Disturbed soil samples were obtained in the field using the following methods.

**Bulk Samples**

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

**The Standard Penetration Test Spoon**

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test spoon sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The spoon was driven up to 18 inches into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-84. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the spoon, bagged, sealed, and transported to the laboratory for testing.

**Field Procedure for the Collection of Relatively Undisturbed Samples**

Relatively undisturbed soil samples were obtained in the field using the following method.

**The Modified Split-Barrel Drive Sampler**

The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-84. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.



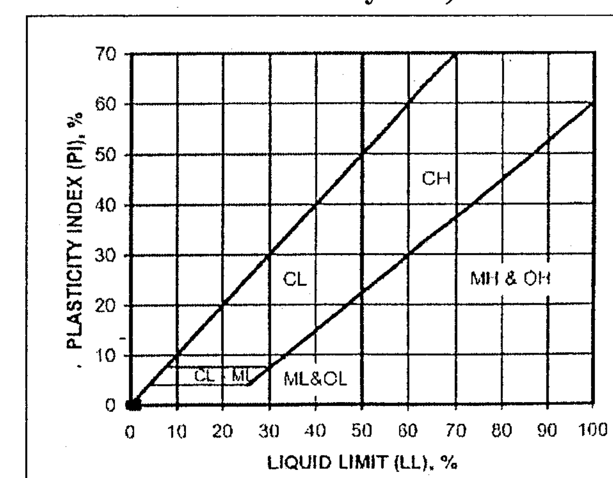
## U.S.C.S. METHOD OF SOIL CLASSIFICATION

| MAJOR DIVISIONS | | SYMBOL | TYPICAL NAMES |
|---|---|---|---|
| COARSE-GRAINED SOILS (More than 1/2 of soil >No. 200 sieve size) | GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size) | GW | Well graded gravels or gravel-sand mixtures little or no fines |
| | | GP | Poorly graded gravels or gravel-sand mixtures, little or no fines |
| | | GM | Silty gravels, gravel-sand-silt mixtures |
| | | GC | Clayey gravels, gravel-sand-clay mixtures |
| | SANDS (More than 1/2 of coarse fraction <No. 4 sieve size) | SW | Well graded sands or gravelly sands, little or no fines |
| | | SP | Poorly graded sands or gravelly sands, little or no fines |
| | | SM | Silty sands, sand-silt mixtures |
| | | SC | Clayey sands, sand-clay mixtures |
| FINE-GRAINED SOILS (More than 1/2 of soil <No. 200 sieve size) | SILTS & CLAYS Liquid Limit <50 | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity |
| | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays |
| | | OL | Organic silts and organic silty clays of low plasticity |
| | SILTS & CLAYS Liquid Limit >50 | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts |
| | | CH | Inorganic clays of high plasticity, fat clays |
| | | OH | Organic clays of medium to high plasticity, organic silty clays, organic silts |
| HIGHLY ORGANIC SOILS | | Pt | Peat and other highly organic soils |

## CLASSIFICATION CHART (Unified Soil Classification System)

| CLASSIFICATION | RANGE OF GRAIN SIZES | |
|---|---|---|
| | U.S. Standard Sieve Size | Grain Size in Millimeters |
| BOULDERS | Above 12" | Above 305 |
| COBBLES | 12" to 3" | 305 to 76.2 |
| GRAVEL<br>Coarse<br>Fine | 3" to No.4<br>3" to 3/4"<br>3/4" to No. 4 | 76.2 to 4.76<br>76.2 to 19.1<br>19.1 to 4.76 |
| SAND<br>Coarse<br>Medium<br>Fine | No. 4 to No. 200<br>No. 4 to No. 10<br>No. 10 to No. 40<br>No. 40 to No. 200 | 4.76 to 0.074<br>4.76 to 2.00<br>2.00 to 0.420<br>0.420 to 0.074 |
| SILT & CLAY | Below No. 200 | Below 0.074 |

GRAIN SIZE CHART




PLASTICITY CHART

Ninyo & Moore | U.S.C.S. METHOD OF SOIL CLASSIFICATION

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DATE DRILLED ____ BORING NO. PATTERNS<br>GROUND ELEVATION ____ SHEET 1 OF 2<br>METHOD OF DRILLING ____<br>DRIVE WEIGHT ____ DROP ____<br>SAMPLED BY ____ LOGGED BY ____ REVIEWED BY ____<br>**DESCRIPTION/INTERPRETATION** |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SOILS |
| | | | | | | | GW | (GW:G3N) = well graded GRAVEL |
| | | | | | | | GP | (GP:G) = poorly graded GRAVEL, sandy gravel, aggregate base |
| | | | | | | | GM | (GM:GZ) = silty GRAVEL |
| | | | | | | | GC | (GC:OG) = clayey GRAVEL |
| | | | | | | | SW | (SW:D) = well graded SAND |
| | | | | | | | SP | (SP:S) = poorly graded SAND |
| 5 | | | | | | | SM | (NZ) = silty SAND |
| | | | | | | | SC | (NO) = clayey SAND |
| | | | | | | | CL | (O) = low plasticity CLAY or just CLAY |
| | | | | | | | ML | (Z) = silt |
| | | | | | | | OL | (4) = low plasticity organic SILT |
| | | | | | | | CH | (C) = high plasticity CLAY |
| | | | | | | | MH | (M) = plastic SILT |
| 10 | | | | | | | OH | (5) = high plasticity organic CLAY |
| | | | | | | | PT | (Q) = peat |
| | | | | | | | | ROCKS AND CONCRETE |
| | | | | | | | | (I) = SILTSTONE (clayey SILTSTONE, sandy SILTSTONE, etc.) |
| | | | | | | | | (1) = SANDSTONE (silty SANDSTONE, clayey SANDSTONE, etc.) |
| | | | | | | | | (H) = CLAYSTONE (sandy CLAYSTONE, silty CLAYSTONE, etc.) |
| | | | | | | | | (O12) = BRECCIA rock with angular and/or gravel- or cobble-sized clasts |
| 15 | | | | | | | | (B) + (1) = CONGLOMERATE |
| | | | | | | | | (>) = SHALE or SLATE |
| | | | | | | | | (/) = GRANITIC ROCK or BONSALL TONALITE |
| | | | | | | | | (2) = METAVOLCANIC (or VOLCANIC) ROCK |
| | | | | | | | | (2+I) = VOLCANIC TUFF |
| | | | | | | | | (V) = GABBROIC ROCK or other intrusive igneous rock |
| | | | | | | | | (P) = ASPHALT CONCRETE |
| | | | | | | | | (9) = CONCRETE |
| 20 | | | | | | | | |

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**BORING LOG**

LEGEND FOR BORING LOGS

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| PATTERNS | REV. 5/99 | Legend-1 |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DATE DRILLED ____ BORING NO. PATTERNS<br>GROUND ELEVATION ____ SHEET 2 OF 2<br>METHOD OF DRILLING ____<br>DRIVE WEIGHT ____ DROP ____<br>SAMPLED BY ____ LOGGED BY ____ REVIEWED BY ____<br>**DESCRIPTION/INTERPRETATION** |
|---|---|---|---|---|---|---|---|---|
| 20 | | | | ▽ | | | | (WATER) Water table during drilling. |
| | | | | ▼ | | | | (FWATER) Water table at boring completion. |
| | | | | | | | | (%) = CALICHE |
| | | | | | | | | (.) = GYPSUM |
| | | | | | | | | ($) = SCHIST |
| | | | | | | | | (7) = Mudstone |
| | | | | | | | | (() Dolomite |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |

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**BORING LOG**

LEGEND FOR BORING LOGS

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| PATTERNS | REV. 5/99 | Legend-2 |

| DATE DRILLED | | BORING NO. | SYMBOL SAMPLES |
|---|---|---|---|
| GROUND ELEVATION | | SHEET 1 OF 1 | |
| METHOD OF DRILLING | | | |
| DRIVE WEIGHT | | DROP | |
| SAMPLED BY | LOGGED BY | REVIEWED BY | |

| DEPTH (feet) | BLOWS/FOOT | DESCRIPTION/INTERPRETATION |
|---|---|---|
| 0 | | Solid line denotes unit change. |
| | | Dashed line denotes material change. |
| | | Modified split-barrel drive sampler. |
| | | No recovery with modified split-barrel drive sampler. |
| 5 | | Seepage. |
| | | Groundwater encountered during drilling. |
| | | Groundwater measured after drilling. |
| 10 | | Standard Penetration Test (SPT). |
| | | No recovery with a SPT. |
| | XX/ XX | Shelby tube sample. Distance pushed in inches/length of sample recovered in inches. |
| | | No recovery with Shelby tube sampler. |
| 15 | | Bulk sample. |
| | | Continuous Push Sample. |
| | | The total depth line is a solid line that is drawn at the bottom of the boring. |
| 20 | | |

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**BORING LOG**

EXPLANATION OF BORING LOG SYMBOLS

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| SYMSAMP | Rev. 5/99 | Legend-3 |

| DATE DRILLED | 7/16/01 | BORING NO. | RH-1 |
|---|---|---|---|
| GROUND ELEVATION | 1324' | SHEET 1 OF 2 | |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY MDE | LOGGED BY MDE | REVIEWED BY LLG | |

| DEPTH (feet) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|
| 0 | | | | CL | ALLUVIUM: Light brown to brown (7.5 YR 6/4 to 7.5 YR 5/4), dry, hard, silty CLAY. Stage I cementation, weakly cemented by sparse calcium carbonate filaments and grain coatings. |
| | 49 | 8.3 | 114.9 | | |
| 5 | 44 | | | | |
| | 9 | 3.8 | | | Stiff. |
| 10 | | | | SC | Brown (7.5 YR 5/4), dry, medium dense to dense, clayey fine to coarse SAND. Stage I cementation, weakly cemented by sparse calcium carbonate and grain coatings. |
| | 30 | 6.9 | 99.4 | | |
| | 43 | | | | Very dense. |
| 15 | 85/11" | 5.2 | | | |
| | 44 | | | | |
| 20 | | | | | |

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**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-1 |

## BORING LOG — RH-1 (Sheet 2 of 2)

DATE DRILLED 7/16/01 BORING NO. RH-1

GROUND ELEVATION 1324' SHEET 2 OF 2

METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Auto) DROP 30"

SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|
| 20 | 67 | 4.5 | 112.2 | SC | ALLUVIUM: (continued) Brown (7.5 YR 5/4), damp, dense, clayey fine to coarse SAND; few silty sand layers. |
| | 55 | 3.8 | | SM | Pale brown (10 YR 6/3), dry, very dense, silty SAND. |
| 25 | 50/3" | | | | |
| | | | | | Total Depth =25.3'<br>Groundwater not encountered.<br>Piezometer installed on 7/16/01. |
| 30 | | | | | |
| 35 | | | | | |
| 40 | | | | | |

**Ninyo & Moore**

**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-2 |

## BORING LOG — RH-2 (Sheet 1 of 2)

DATE DRILLED 7/9/01 BORING NO. RH-2

GROUND ELEVATION 1320' SHEET 1 OF 2

METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Auto) DROP 30"

SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|
| 0 | | | | CL | ALLUVIUM: Brown (7.5 YR 5/4), damp, hard, silty CLAY.<br>Stage I cementation, weakly cemented by sparse calcium carbonate filaments. |
| | 34 | 7.4 | | | |
| 5 | 41 | | | | |
| | 76/10" | 11.6 | 100.0 | | Scattered fine gravel. |
| | | | | SC-SM | Reddish brown (5 YR 5/4), damp, dense, silty, clayey SAND. |
| 10 | 26 | 11.9 | | | |
| | 75/11" | | | | Very dense.<br>Stage II cementation, moderate cementation by calcium carbonate nodules less than 1/4" in diameter. |
| 15 | 29 | 12.4 | | | Dense. |
| | | | | ML | Brown (7.5 YR 5/4), damp, very dense, sandy SILT. |
| | 90/10" | 9.8 | 103.6 | | Stage II cementation. |
| 20 | | | | | |

**Ninyo & Moore**

**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-3 |

| DATE DRILLED | 7/9/01 | BORING NO. | RH-2 |
|---|---|---|---|
| GROUND ELEVATION | 1320' | SHEET | 2 OF 2 |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY MDE | LOGGED BY MDE | REVIEWED BY | LLG |

| DEPTH (feet) | SAMPLES Bulk / Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 20 | Driven | 69 | | | | ML | ALLUVIUM: (continued) Brown (7.5 YR 5/4), damp, very dense, sandy SILT. |
| | | | | | | | Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/9/01. |
| 25 | | | | | | | |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |

Ninyo & Moore

**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-4 |

| DATE DRILLED | 7/16/01 | BORING NO. | RH-3 |
|---|---|---|---|
| GROUND ELEVATION | 1320' | SHEET | 1 OF 2 |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY MDE | LOGGED BY MDE | REVIEWED BY | LLG |

| DEPTH (feet) | SAMPLES Bulk / Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | SM | ALLUVIUM: Light brown to brown (7.5 YR 6/4 to 7.5 YR 5/4), dry to damp, silty, medium dense SAND; few fine gravel. Stage I cementation, weakly cemented and scattered filaments. |
| | Driven | 31 | 8.3 | 89.9 | | | |
| 5 | Driven | 74/10" | 9.9 | 93.8 | | | Very dense. |
| | Driven | 47 | 6.0 | | | | |
| | | | | | | ML | Very pale brown (10 YR 7/4), dry, hard, clayey SILT. Stage II cementation with scattered caliche nodules less than 1/4" in diameter. |
| 10 | Driven | 64 | | | | | |
| | Driven | 54 | | | | | |
| 15 | Driven | 90/10" | 8.4 | 86.0 | | | |
| | Driven | 81 | 6.2 | | | | |
| 20 | | | | | | | |

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**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-5 |

| DATE DRILLED | 7/16/01 | BORING NO. | RH-3 |
|---|---|---|---|
| GROUND ELEVATION | 1320' | SHEET | 2 OF 2 |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY | MDE | LOGGED BY | MDE |
| REVIEWED BY | LLG | | |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 20 | | ■ | 82 | 12.7 | 100.9 | | CL | ALLUVIUM: (continued)<br>Very pale brown (10 YR 7/4), dry, hard, silty CLAY. |
| | | | | | | | | Total Depth = 21.5'<br>Groundwater not encountered.<br>Piezometer installed on 7/16/01. |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |

**Ninyo & Moore**

**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-6 |

| DATE DRILLED | 7/9/011 | BORING NO. | RH-4 |
|---|---|---|---|
| GROUND ELEVATION | 1319' | SHEET | 1 OF 2 |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY | MDE | LOGGED BY | MDE |
| REVIEWED BY | LLG | | |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | CL | ALLUVIUM:<br>Brown (7.5 YR 5/4), dry, very stiff, silty CLAY.<br>Stage I cementation, weakly cemented scattered calcium carbonate filaments. |
| | | ■ | 24 | | | | | |
| 5 | | ■ | 22 | 8.0 | 98.3 | | | |
| | | ■ | 91/11" | 7.7 | | | | Very pale brown (10 YR 7/4), dry, hard, sandy CLAY.<br>Stage II cementation, trace to sparse caliche nodules less than 1/2" in diameter, moderately cemented. |
| 10 | | ■ | 66/11" | 9.0 | 91.4 | | | |
| | | ■ | 46 | 6.6 | | | | |
| 15 | | ■ | 47 | 7.3 | 98.3 | | SC | Reddish brown (5 YR 5/4), dry to damp, dense, clayey SAND; trace fine gravel.<br>Stage II cementation, moderately cemented, few to some calcium carbonate nodules less than 1/2" in diameter. |
| | | ■ | 33 | 4.8 | | | | Color change to very pale brown at 18.5'. |
| 20 | | | | | | | | |

**Ninyo & Moore**

**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-7 |

DATE DRILLED 7/9/011 BORING NO. RH-4

GROUND ELEVATION 1319' SHEET 2 OF 2

METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Auto) DROP 30"

SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 20 | | ■ | 64/11" | | | | SC | ALLUVIUM: (continued)<br>Reddish brown (5 YR 5/4), damp, dense, clayey SAND; trace fine gravel. |
| | | | | | | | | Total Depth = 20.9'<br>Groundwater not encountered.<br>Backfilled on 7/9/01. |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |

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## BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-8 |

DATE DRILLED 7/16/01 BORING NO. RH-5

GROUND ELEVATION 1320' SHEET 1 OF 2

METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Auto) DROP 30"

SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | CL | ALLUVIUM:<br>Light brown (7.5 YR 6/4), dry, hard, silty CLAY.<br>Stage I cementation, weakly cemented and scattered filaments. |
| | | ■ | 29 | | | | | |
| 5 | | ■ | 93/9" | 6.5 | | | | |
| | | ■ | 50/6" | 8.4 | | | | |
| 10 | | ■ | 48 | 8.0 | 97.1 | | | |
| | | | 64 | | | | | |
| 15 | | ■ | 91/9" | | | | | Sparse fine sand, cementation.<br>Stage II cementation, moderately cemented and scattered calcium carbonate nodules up to 1/4" in diameter. |
| | | ■ | 77 | 1.8 | | | SC | Pale brown, dry, very dense, clayey SAND; sparse fine gravel. |
| 20 | | | | | | | | |

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## BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-9 |

DATE DRILLED 7/16/01 BORING NO. RH-5
GROUND ELEVATION 1320' SHEET 2 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 20 | | ■ | 74 | | | | SC | ALLUVIUM: (continued) Pale brown (10YR 6/3), dry, very dense, clayey SAND; sparse fine gravel. Stage II cementation, moderately cemented. |
| | | | | | | | | Total Depth = 21.0' Groundwater not encountered. Piezometer installed on 7/16/01. |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |

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BORING LOG
East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-10 |

DATE DRILLED 7/9/01 BORING NO. RH-6
GROUND ELEVATION 1322' SHEET 1 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | ■ | | | | | | CL | ALLUVIUM: Light brown (7.5 YR 6/4), dry, hard, silty CLAY; few fine sand. Stage I cementation, scattered caliche filaments. |
| | | ■ | 51 | 5.4 | 100.2 | | | |
| 5 | | ■ | 50/5" | 6.1 | | | | |
| | | | | | | | | Brown (7.5 YR 5/4). |
| | | ◢ | 68/11" | | | | | |
| 10 | | ■ | 31 | 10.6 | 79.3 | | | |
| | | ◢ | 42 | 10.1 | | | | |
| | | | | | | | | Light to pale brown (7.5 YR 6/4 to 10 YR 6/3). |
| 15 | | ■ | 52 | 7.0 | 106.3 | | | |
| | | ◢ | 29 | 6.8 | | | SM | Light brown (7.5 YR 6/3), dry, dense, silty SAND. Stage II cementation, scattered caliche coatings. |
| 20 | | | | | | | | |

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BORING LOG
East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-11 |

## BORING LOG — RH-6 (Sheet 2 of 2)

DATE DRILLED 7/9/01 BORING NO. RH-6
GROUND ELEVATION 1322' SHEET 2 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 20 | | ■ | 44 | | | | ML | ALLUVIUM: (continued) Light brown to brown (7.5 YR 6/3 to 7.5 YR 5/3), damp, hard, clayey SILT. Stage II cementation, scattered caliche nodules. |
| | | | | | | | | Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/9/01. |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |

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**BORING LOG**
East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-12 |

## BORING LOG — RH-7 (Sheet 1 of 2)

DATE DRILLED 7/10/01 BORING NO. RH-7
GROUND ELEVATION 1325' SHEET 1 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | CL | ALLUVIUM: Pale brown (10 YR 6/3), dry, hard, silty CLAY. Stage I cementation, weakly cemented. |
| | | | | ♀ | | | | |
| | | ■ | 35 | 7.1 | 89.6 | | | |
| 5 | | ■ | 60 | 6.9 | | | | |
| | | ■ | 90/9" | | | | | |
| 10 | | ■ | 34 | 13.9 | | | | |
| | | ■ | 53 | 13.3 | 91.6 | | | Few fine sand. |
| | | | | | | | | Reddish brown (5 YR 5/4), damp. |
| 15 | | ■ | 40 | 10.6 | | | | |
| | | ■ | 71 | 13.2 | 81.9 | | | |
| 20 | | | | | | | | |

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**BORING LOG**
East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-13 |

| DATE DRILLED | 7/10/01 | BORING NO. | RH-7 |
|---|---|---|---|
| GROUND ELEVATION | 1325' | SHEET | 2 OF 2 |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY | MDE | LOGGED BY | MDE |
| REVIEWED BY | LLG | | |

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 20 | Driven | 50/5" | | | | ML | ALLUVIUM: (continued) Reddish brown (5 YR 5/4), damp, hard, clayey SILT; few sand. Stage I cementation, weakly cemented. |
| | | | | | | | Total Depth = 20.4' Groundwater not encountered. Backfilled on 7/10/01. |
| 25 | | | | | | | |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |

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## BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-14 |

| DATE DRILLED | 7/5/01 | BORING NO. | RH-8 |
|---|---|---|---|
| GROUND ELEVATION | 1329' | SHEET | 1 OF 2 |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY | MDE | LOGGED BY | MDE |
| REVIEWED BY | LLG | | |

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | CL | ALLUVIUM: Light brown (7.5 YR 6/3), dry, hard, silty CLAY; few fine to medium sand; scattered caliche filaments. |
| | Driven | 41 | 8.0 | 93.5 | | | |
| 5 | Driven | 97/10" | 8.7 | | | | |
| | Driven | 89/11" | 10.7 | | | | |
| 10 | | | | | | ML | Light brown (7.5 YR 6/3), dry, hard, clayey SILT; few fine sand. Stage II cementation, scattered caliche nodules less than 1/4" in diameter. |
| | Driven | 37 | | | | | |
| | Bulk / Driven | 54 | 10.5 | 84.7 | | | |
| 15 | Driven | 55 | | | | SM | Brown (7.5 YR 5/4), damp, very dense, silty SAND; few fine subrounded gravel. Stage II cementation. |
| | | | | | | CL | Light brown (7.5 YR 6/3), dry, hard, silty CLAY; few fine sand. Stage II cementation, scattered caliche nodules less than 1/4" in diameter. |
| | Driven | 95/11" | 11.3 | | | | |
| 20 | | | | | | | |

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## BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-15 |

## BORING LOG — RH-8 (Sheet 2 of 2)

DATE DRILLED 7/5/01 BORING NO. RH-8
GROUND ELEVATION 1329' SHEET 2 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 20 | | | | | | CL | ALLUVIUM: (continued) Light brown (7.5 YR 6/3), dry, hard, silty CLAY; few fine sand, scattered caliche nodules less than 1/2", scattered caliche stringers. Stage II cementation with scattered caliche nodules less than 1/2" in diameter. |
| | Driven | 76 | | | | | |
| | Driven | 63/11" | 10.8 | 102.6 | | | |
| 25 | Driven | 64 | | | | | |
| | | | | | | | Total Depth = 26.5'<br>Groundwater not encountered.<br>Backfilled on 7/9/01. |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |

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**BORING LOG**
East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-16 |

## BORING LOG — RH-9 (Sheet 1 of 2)

DATE DRILLED 7/10/01 BORING NO. RH-9
GROUND ELEVATION 1329' SHEET 1 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | ML | ALLUVIUM: Pale brown (10 YR 6/3), dry to damp, hard, clayey SILT. Stage I cementation, weakly cemented. |
| | Driven | 82 | | | | | |
| 5 | | | | | | CL | Pale brown (10 YR 6/3), dry to damp, hard, silty CLAY. Stage I cementation, weakly cemented. |
| | Driven | 55 | 7.9 | 109.0 | | | |
| | Driven | 48 | 9.4 | | | | |
| 10 | Driven | 84 | | | | | |
| | Driven | 34 | 18.8 | | | | |
| 15 | Driven | 32 | 18.2 | 103.3 | | | |
| | | | | | | SM | Brown to pale brown (7.5 YR 5/4 to 10 Yr 6/3), damp, medium dense, silty SAND; trace fine, subrounded gravel. Stage II cementation, gravel has thin coatings. |
| | Driven | 18 | 12.4 | | | | |
| 20 | | | | | | | |

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**BORING LOG**
East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/0101 | A-17 |

| DATE DRILLED | 7/10/01 | BORING NO. | RH-9 |
|---|---|---|---|
| GROUND ELEVATION | 1329' | SHEET | 2 OF 2 |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY | MDE | LOGGED BY | MDE |
| REVIEWED BY | LLG | | |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 20 | | | | | | | SC | ALLUVIUM: (continued)<br>Brown (7.5 YR 5/4), damp, medium dense, clayey fine to coarse SAND; trace subangular fine gravel.<br>Stage II cementation, gravel has thin coatings. |
| | | | 40 | 7.3 | 107.8 | | | |
| | | | 33 | | | | | Dense to very dense. |
| 25 | | | | | | | | |
| | | | 90/11" | | | | | Very dense. |
| | | | | | | | | Total Depth = 27.8'<br>Groundwater not encountered.<br>Backfilled on 7/10/01. |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |

# BORING LOG

Ninyo & Moore

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/0101 | A-18 |

| DATE DRILLED | 7/9/011 | BORING NO. | RH-10 |
|---|---|---|---|
| GROUND ELEVATION | 1327' | SHEET | 1 OF 1 |
| METHOD OF DRILLING | CME 75, 8" Diameter Hollow-Stem Auger | | |
| DRIVE WEIGHT | 140 lbs. (Auto) | DROP | 30" |
| SAMPLED BY | MDE | LOGGED BY | MDE |
| REVIEWED BY | LLG | | |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | CL | ALLUVIUM:<br>Light brown (7.5 YR 6/4), dry, hard, silty CLAY.<br>Stage I cementation, weakly cemented with scattered caliche filaments. |
| | | | 31 | | | | | |
| 5 | | | 53 | | | | | |
| | | | 94/10" | 12.3 | | | | |
| | | | | | | | ML | Pale brown (10 YR 6/3), dry, hard, clayey SILT.<br>Stage II cementation, scattered nodules. |
| 10 | | | 62 | 7.3 | | | | |
| | | | 66 | 10.5 | 86.9 | | | |
| 15 | | | 59 | 7.1 | | | | |
| | | | | | | | | Total Depth = 16.5<br>Groundwater not encountered.<br>Backfilled on 7/9/01. |
| 20 | | | | | | | | |

# BORING LOG

Ninyo & Moore

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-19 |

DATE DRILLED 7/10/01 BORING NO. RH-11

GROUND ELEVATION 1325' SHEET 1 OF 2

METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Auto) DROP 30"

SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | CL | ALLUVIUM: Pale brown (10 YR 6/3), dry, hard, silty CLAY. Stage I cementation, weakly cemented with scattered filaments. |
| | | 71 | 7.5 | 10.9 | | | |
| 5 | | 85 | 9.3 | | | | |
| | | 21 | 14.4 | | | | |
| 10 | | 30 | 13.8 | 98.2 | | | |
| | | 33 | | | | | |
| 15 | | 23 | 16.6 | | | | Few sand. |
| | | | | | | SC | Light brown to very pale brown (7.5 YR 6/3 to 10 YR 7/4), damp, medium dense, clayey SAND. Stage II cementation below 17' bgs. |
| | | 32 | | | | | |
| 20 | | | | | | | |

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# BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-20 |

DATE DRILLED 7/10/01 BORING NO. RH-11

GROUND ELEVATION 1325' SHEET 2 OF 2

METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Auto) DROP 30"

SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 20 | | 36 | 9.3 | 104.8 | | SC | ALLUVIUM: (continued) Light brown (7.5 YR 6/3), damp, medium dense, clayey SAND. |
| | | | | | | | Total Depth = 21.5'. Groundwater not encountered. Backfilled on 7/10/01. |
| 25 | | | | | | | |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |

Ninyo & Moore

# BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-21 |

DATE DRILLED 7/9/01 BORING NO. RH-12
GROUND ELEVATION 1322' SHEET 1 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|
| 0 | | | | ML | ALLUVIUM: Pale brown (10 YR 6/3), dry to damp, hard, clayey SILT. Stage I cementation, scattered filaments. |
| | 46 | 6.7 | 97.5 | | |
| 5 | 50/6" | 6.7 | 91.6 | | |
| | | | | SM | Pale brown (10 YR 6/3), dry to damp, very dense, silty SAND; trace fine gravel. Stage I cementation, scattered filaments. |
| | 36 | 3.8 | | | |
| 10 | 76/11" | | | ML | Pale brown (10 YR 6/3), dry to damp, very hard, SILT. |
| | 50/5" | 5.2 | | SM | Pale brown (10 YR 6/3), dry to damp, very dense, silty SAND; scattered caliche filaments. |
| 15 | 50/5" | 7.5 | 84.1 | CL | Pale brown (10 YR 6/3), dry to damp, hard, silty CLAY; trace fine gravel. Stage II cementation below 15' bgs. |
| | 40 | | | | |
| 20 | | | | | |

**Ninyo & Moore**

## BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-22 |

DATE DRILLED 7/9/01 BORING NO. RH-12
GROUND ELEVATION 1322' SHEET 2 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|
| 20 | 65 | | | SM | ALLUVIUM: (continued) Reddish brown (5 YR 5/4), dry, dense, silty SAND; trace fine gravel. |
| | | | | | Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/9/01. |
| 25 | | | | | |
| 30 | | | | | |
| 35 | | | | | |
| 40 | | | | | |

**Ninyo & Moore**

## BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-23 |

DATE DRILLED 7/10/01 BORING NO. RH-13
GROUND ELEVATION 1324' SHEET 1 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | CL | ALLUVIUM: Light to dark brown (7.5 YR 6/4 to 7.5 YR 3/4), damp, very stiff, silty CLAY. Stage I cementation, scattered filaments. |
| | Driven | 24 | 8.7 | 89.3 | | | |
| 5 | Driven | 30 | 12.0 | | | | Hard. |
| | Driven | 21 | 11.2 | | | | |
| 10 | Driven | 50 | 13.7 | 101.3 | | | |
| | Driven | 31 | 9.9 | | | | Scattered subrounded fine gravel. |
| 15 | Driven | 86/11" | | | | | |
| | Driven | 56 | 9.1 | | | | |
| | | | | | | ML | Reddish brown (5 YR 5/4), damp to dry, hard, clayey SILT. |
| 20 | | | | | | | |

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**BORING LOG**
East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-24 |

DATE DRILLED 7/10/01 BORING NO. RH-13
GROUND ELEVATION 1324' SHEET 2 OF 2
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY MDE LOGGED BY MDE REVIEWED BY LLG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 20 | | | | | | ML | Stage II cementation, weakly cemented, scattered nodules. |
| | Driven | 60 | 8.4 | 97.6 | | | ALLUVIUM: (continued) Reddish brown (5 YR 5/4), damp to dry, hard, clayey SILT. |
| | | | | | | | Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/10/01. |
| 25 | | | | | | | |
| 30 | | | | | | | |
| 35 | | | | | | | |
| 40 | | | | | | | |

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**BORING LOG**
East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-25 |

DATE DRILLED 7/5/01 BORING NO. RH-14
GROUND ELEVATION 1323' SHEET 1 OF 1
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY EMS LOGGED BY EMS REVIEWED BY LLG

| DEPTH (feet) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|
| 0 | | | | CL | ALLUVIUM: Light brown (7.5 YR 6/4), dry, hard, silty CLAY; trace sand. Stage I cementation, weakly cemented. |
| | 38 | 7.6 | 94.9 | | |
| | | | | | Little fine to coarse sand. |
| 5 | 39 | 6.8 | | | |
| | 50/4" | 4.5 | 103.9 | | Few gravel. |
| | | | | ML | Light brown (7.5 YR 6/4), dry, very dense, fine sandy SILT. |
| 10 | 47 | 5.1 | | | |
| | | | | SC | Light brown (7.5 YR 6/4), damp, very dense, clayey fine to coarse SAND. Stage I cementation, scattered filaments. |
| 15 | 83/9" | 7.5 | 99.7 | | |
| | | | | | Total Depth = 15.8'<br>Groundwater not encountered.<br>Backfilled on 7/5/01. |
| 20 | | | | | |

**Ninyo & Moore**

# BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-26 |

DATE DRILLED 7/5/01 BORING NO. RH-15
GROUND ELEVATION 1322' SHEET 1 OF 1
METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs. (Auto) DROP 30"
SAMPLED BY EMS LOGGED BY EMS REVIEWED BY LLG

| DEPTH (feet) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|
| 0 | | | | ML | ALLUVIUM: Brown (7.5 YR 5/4), damp, hard, clayey SILT; few fine sand. Stage I cementation, scattered filaments. |
| | 44 | 9.6 | 86.7 | | |
| 5 | 70/11" | 10.4 | 96.5 | | Weakly to moderately cemented by caliche. |
| | 22 | 15.5 | | | |
| | | | | CL | Brown (7.5 YR 5/4), damp, hard, silty CLAY. Stage II cementation, scattered caliche filaments and nodules. |
| 10 | 45 | 15.4 | 101.7 | | |
| | | | | SC | Brown (7.5 YR 5/4), damp, medium dense to dense, clayey fine to medium SAND. Stage II cementation, scattered nodules. |
| 15 | 20 | 5.7 | | | |
| | | | | | Total Depth = 16.5'<br>Groundwater not encountered.<br>Backfilled on 7/5/01. |
| 20 | | | | | |

**Ninyo & Moore**

# BORING LOG

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-27 |

DATE DRILLED 7/5/01 BORING NO. RH-16

GROUND ELEVATION 1322' SHEET 1 OF 1

METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Auto) DROP 30"

SAMPLED BY EMS LOGGED BY EMS REVIEWED BY LLG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | CL | ALLUVIUM: Brown (7.5 YR 5/4), damp, hard, silty CLAY; little fine to medium sand. Stage I cementation, scattered filaments. |
| | | 51 | 7.2 | 92.7 | | | |
| 5 | | | | | | | |
| | | 79 | 12.5 | | | | |
| | | | | | | SM | Brown (7.5 YR 5/4), damp, very dense, silty fine to medium SAND. Stage II cementation, scattered to numerous caliche filaments and nodules. |
| | | 93/9" | 20.5 | 94.5 | | | |
| | | | | | | CL | Brown (7.5 YR 5/2), moist, very stiff, silty CLAY. Stage II cementation, scattered caliche nodules. |
| 10 | | | | | | | |
| | | 16 | 18.1 | | | | |
| 15 | | | | | | | |
| | | 32 | 17.1 | 108.3 | | | Hard. |
| | | | | | | | Total Depth = 16.5' Groundwater not encountered. Backfilled on 7/5/01. |
| 20 | | | | | | | |

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**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-28 |

DATE DRILLED 7/5/01 BORING NO. RH-17

GROUND ELEVATION 1327' SHEET 1 OF 1

METHOD OF DRILLING CME 75, 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Auto) DROP 30"

SAMPLED BY EMS LOGGED BY EMS REVIEWED BY LLG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | SC | ALLUVIUM: Brown (7.5 YR 5/4), damp, medium dense, clayey fine to coarse SAND; few gravel. Stage I cementation, weakly cemented. |
| | | 17 | 4.3 | 100.7 | | | |
| 5 | | | | | | | |
| | | 27 | 4.7 | 106.4 | | | |
| | | | | | | SM | Brown (7.5 YR 5/4), damp, very dense, silty fine to coarse SAND; few gravel; trace clay. Stage I cementation, weak cementation with scattered caliche filaments. |
| | | 73/10" | 5.8 | | | | |
| 10 | | | | | | | |
| | | 72 | | | | | |
| 15 | | | | | | | |
| | | 85 | 7.2 | | | | |
| | | | | | | | Total Depth = 16.5' Groundwater not encountered. Backfilled on 7/5/01. |
| 20 | | | | | | | |

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**BORING LOG**

East Maricopa Floodway
Rittenhouse Detention Basin

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | A-29 |

## Ninyo & Moore

# TEST PIT LOG

East Maricopa Floodway
Rittenhouse Detention Basin

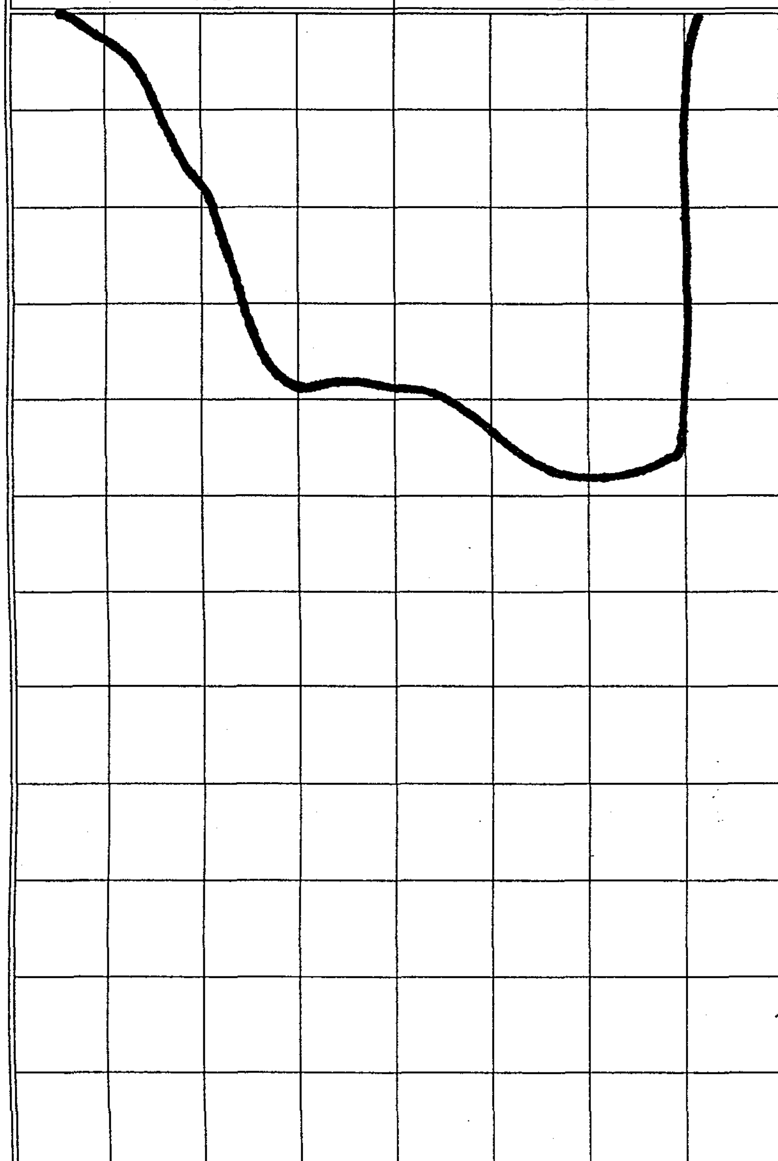
| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

DATE EXCAVATED 11/26/01 TEST PIT NO. TP-1

GROUND ELEVATION -- LOGGED BY MDE

METHOD OF EXCAVATION Backhoe - Ford 555 E

LOCATION 0.4 Mi. N/NE of TP-3, E Side of EMF Rd. at Fenceline



| DEPTH (FEET) | SAMPLES: Bulk / Driven / Sand Cone | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION |
|---|---|---|---|---|---|
| 0 | | | | SM | FILL: Brown (7.5 YR 5/4), dry to damp, loose, silty fine to medium sand, scattered fine GRAVEL. |
| | | | | CL | ALLUVIUM: Brown, damp, very stiff, CLAY. @ 2-2.5 feet, scattered calcium carbonate filaments less than 1/4" long, scattered rootlets, scattered caliche nodules less than 1/2" in diameter at 2.0 to 2.5 feet, weakly cemented (Class 1). |
| 5 | | | | ML | Strong brown (7.5 YR 4/6), loose to medium dense, damp, SILT. Stage I cementation, weakly cemented. @ 4 feet bgs, becomes loose, dry to damp. @ 6 feet bgs, becomes dense, with increased calcium carbonate cementation in abundant stringers less than 1" long and scattered rootlet casts, color lightens to brown (7.5 YR 4/4). @ 7 feet, becomes reddish brown (5 Y/R 4/4), with trace to few fine sand, higher observed porosity, strongly reactive with HCL, open pinhole porosity coated with calcium carbonate in-fill. @ 8 feet, pervasive calcium carbonate stringers, degree of cementation increases, color hue lightens to reddish brown (5 YR 5/4), dense. @ 10.5 to 12 feet, medium dense, damp, sparse fine SAND, (7.5 YR 4/6), strong brown, strong reaction with HCL. Stage I cementation decreases. Strong reaction with HCL. |
| 10 | | | | | |
| 15 | | | | | Total Depth = 12 feet. Groundwater not encountered during drilling. Backfilled on 11/26/01. Excavation Bearing: 201° |
| 20 | | | | | |
| 25 | | | | | |

SCALE = 1 in./5 ft.

FIGURE A-30

## Ninyo & Moore

# TEST PIT LOG

East Maricopa Floodway
Rittenhouse Detention Basin

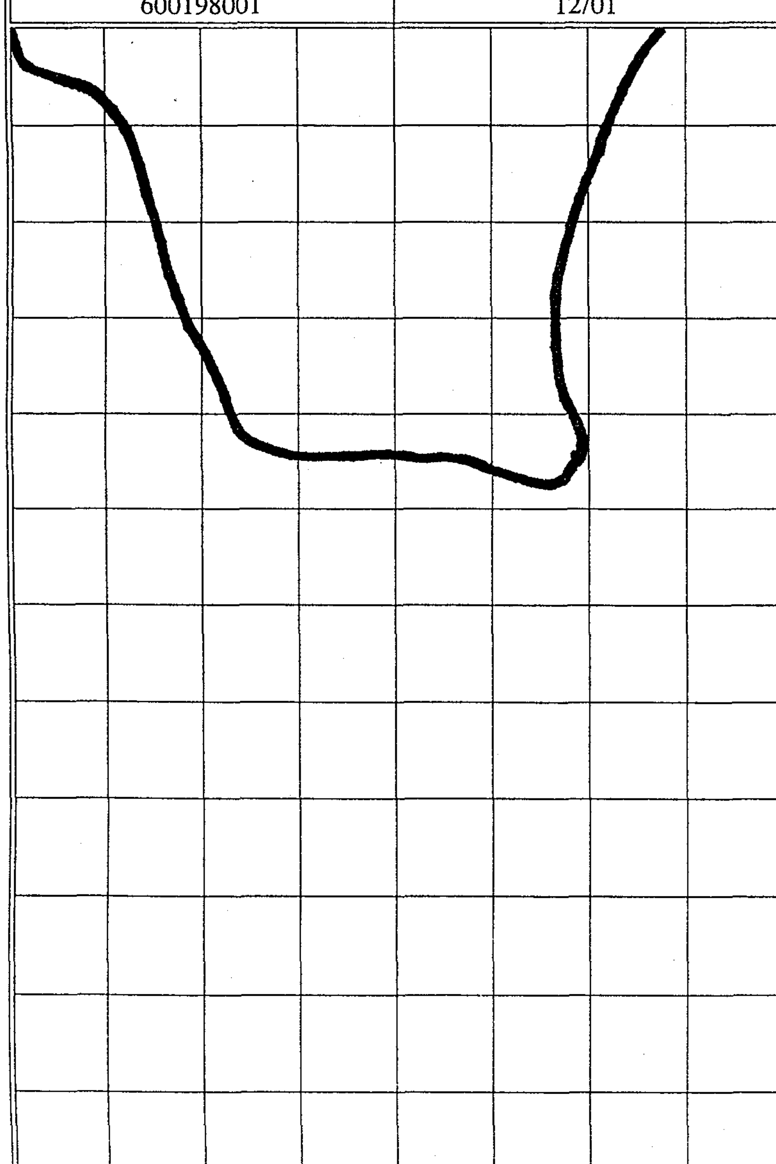
| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

DATE EXCAVATED 11/27/01 TEST PIT NO. TP-2

GROUND ELEVATION -- LOGGED BY MDE

METHOD OF EXCAVATION Backhoe - Ford 555 E

LOCATION 0.2 Mi. S of TP-1, E Side of EMF Rd., E of Road 8'.



| DEPTH (FEET) | SAMPLES: Bulk / Driven / Sand Cone | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION |
|---|---|---|---|---|---|
| 0 | | | | CL | ALLUVIUM: Strong brown (7.5 YR 4/6), stiff, damp, silty CLAY; scattered rootlets, scattered pinhole porosity, trace fine sand, trace fine gravel, weak reaction with HCL. Stage I cementation, weakly to non-cemented. |
| | | | | ML | Brown (7.5 YR 5/4), loose to medium dense, dry to damp SILT, trace fine sand, trace fine gravel, scattered rootlets, scattered pinhole porosity, scattered root casts up to 1/8" in diameter. Stage I cementation, scattered filaments less than 1/4" long. @ 4 feet bgs, becomes dense with higher degree of calcium carbonate cementation, silt color lightens to light brown (7.5 YR 6/4), moderate reaction with HCL. @ 7 feet, Stage I cementation with abundant calcium carbonate filaments, very dense pockets of calcium carbonate cementation within sandy silt up to 6" in diameter by 2" thick, surrounding silt is weakly cemented and weakly reactive with HCL. |
| 5 | | | | | |
| | | | | SM | Strong brown (7.5 YR 5/6), dry to damp, silty SAND; scattered fine gravel, abundant pinhole porosity. Stage II cementation, moderately cemented, scattered to sparse pockets less than 6" in diameter of strong cementation. |
| 10 | | | | ML | Brown (7.5 YR 4/4), damp, medium dense, sandy SILT. |
| 15 | | | | | Total Depth = 12 feet. Groundwater not encountered during drilling. Backfilled on 11/27/01. Excavation Bearing: 200° |
| 20 | | | | | |
| 25 | | | | | |

SCALE = 1 in./5 ft.

FIGURE A-31

**Ninyo & Moore**

## TEST PIT LOG

East Maricopa Floodway
Rittenhouse Detention Basin

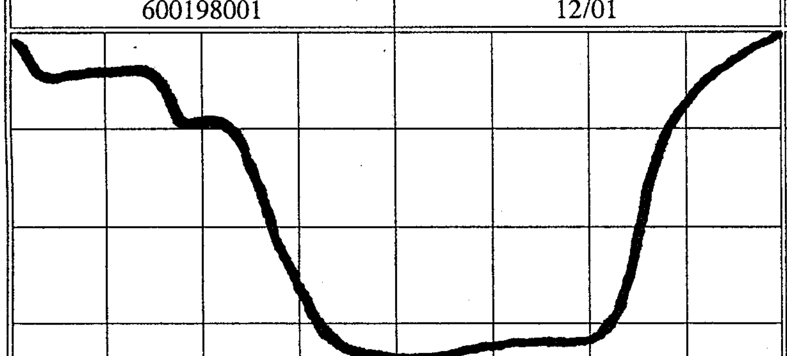
| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

DATE EXCAVATED 11/26/01 TEST PIT NO. TP-3

GROUND ELEVATION -- LOGGED BY MDE/HV

METHOD OF EXCAVATION Backhoe - Ford 555 E

LOCATION E Side of EMF, approx. 500'N of RH-5, E of Road 8'.



| DEPTH (FEET) | SAMPLES: Bulk | SAMPLES: Driven | SAMPLES: Sand Cone | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | SM | FILL: Brown (7.5 YR 5/4), dry to damp, loose, silty fine -to medium SAND; scattered fine gravel. |
| | ■ | | | | | CL | ALLUVIUM: Dark brown (7.5 YR 3/3), damp, stiff to very stiff, silty CLAY; scattered rootlets. Stage I cementation, weakly to non-cemented. |
| 5 | | | | | | ML | Strong brown (7.5 YR 4/6), loose to medium dense, damp, SILT; trace fine sand and clay, scattered pinhole porosity, scattered rootlets and roots. Stage I cementation, scattered filaments less than 1/2" long. @ 4 feet bgs, becomes loose. @ 6 feet bgs, becomes hard with higher degree of cementation. @ 7 feet bgs, (10 YR 6/6), changes to fine sandy scattered pockets of silt, higher porosity. Stage I cementation, abundant filaments. |
| 10 | | | | | | SM | Strong brown (7.5 YR 5/6), dense to medium dense, dry, silty SAND; scattered fine gravel, scattered pinhole porosity. Stage II cementation, moderately cemented, increased calicum carbonate coatings on root casts and open pore space. Refusal on strongly cemented, Stage II material with 555 backhoe. |
| 15 | | | | | | | Total Depth = 8.5 feet. Groundwater not encountered during drilling. Backfilled on 11/26/01. Excavation Bearing: 215° |
| 20 | | | | | | | |
| 25 | | | | | | | |

SCALE = 1 in./5 ft.

FIGURE A-32

# APPENDIX B

# LABORATORY TESTING

**Classification**
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-93. Soil classifications are indicated on the logs of the exploratory excavations in Appendix A.

**Moisture Content**
The moisture content of samples obtained from the exploratory excavations was evaluated in accordance with ASTM D 2216-92. The test results are presented on the logs of the exploratory excavations in Appendix A.

**In-Place Moisture and Density Tests**
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory excavations were evaluated in general accordance with ASTM D 2937-94. The test results are presented on the logs of the exploratory excavations in Appendix A.

**Gradation Analysis**
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422-63. The grain-size distribution curves are shown on Figures B-1 through B-33. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System.

**Atterberg Limits**
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318-98. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results and classifications are shown on Figures B-34 through B-37.

**Consolidation Tests**
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435-90. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-38 through B-40.





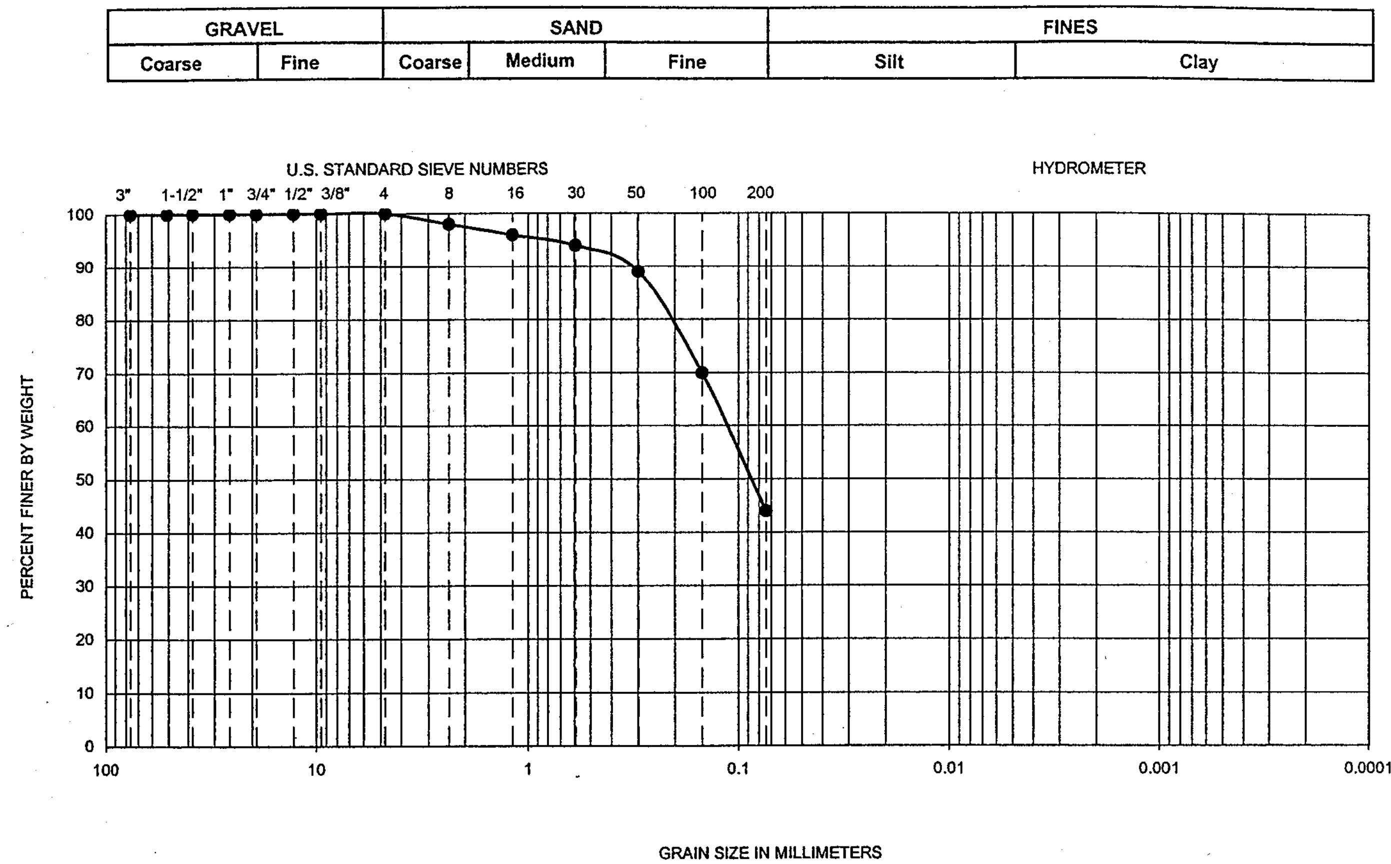
**Maximum Dry Density and Optimum Moisture Content Tests**
The maximum dry density and optimum moisture content of selected representative soil samples were evaluated in general accordance with ASTM D 1557-91. The results of these tests are summarized on Figures B-41 through B-43.

**Expansion Index Tests**
The expansion index of selected materials was evaluated in general accordance with U.B.C. Standard No. 18-2. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure B-44.

**Soil Corrosivity Tests**
Soil pH and minimum resistivity tests were performed on representative samples in general accordance with Arizona Test 236b. The chloride content of selected samples was evaluated in general accordance with Arizona Test 722. The sulfate content of selected samples was evaluated in general accordance with Arizona Test 733. The test results are presented on Figure B-45.

**Permeability Tests**
Constant head permeability tests were performed on selected remolded soil samples in general accordance with ASTM D 2434-68. The samples were placed in the apparatus and saturated. Water flow through the soil was sustained using a pneumatically induced head at specified pressures. The quantity of flow, the elapsed time, and the hydraulic gradient were recorded. The permeability was then calculated using Darcy's equation. The results of the tests are presented on Figure B-46.

**Unconsolidated Undrained Triaxial Compression Tests**
Triaxial compression tests were performed on selected remolded and undisturbed samples in general accordance with ASTM D 2850-95. The test results are shown on Figures B-47 and B-49.




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-1 | 10-11.5 | -- | -- | NP | -- | -- | -- | -- | -- | 44 | SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

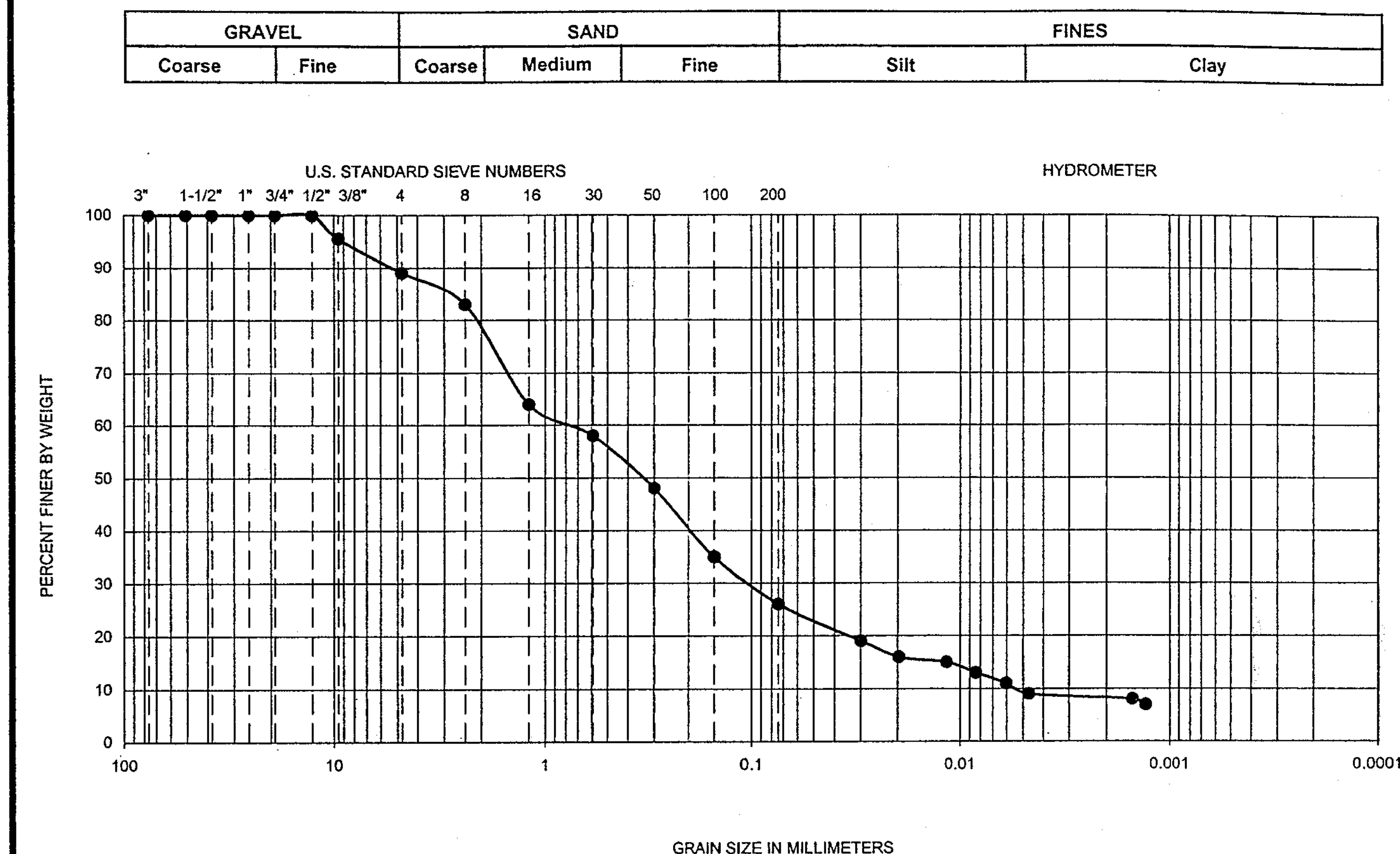
Ninyo & Moore

**GRADATION TEST RESULTS**

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-1 |

SIEVE RH-01@10.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-1 | 25.0-26.5 | -- | 18 | -- | 0.01 | 0.11 | 0.84 | 167.2 | 2.8 | 26 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

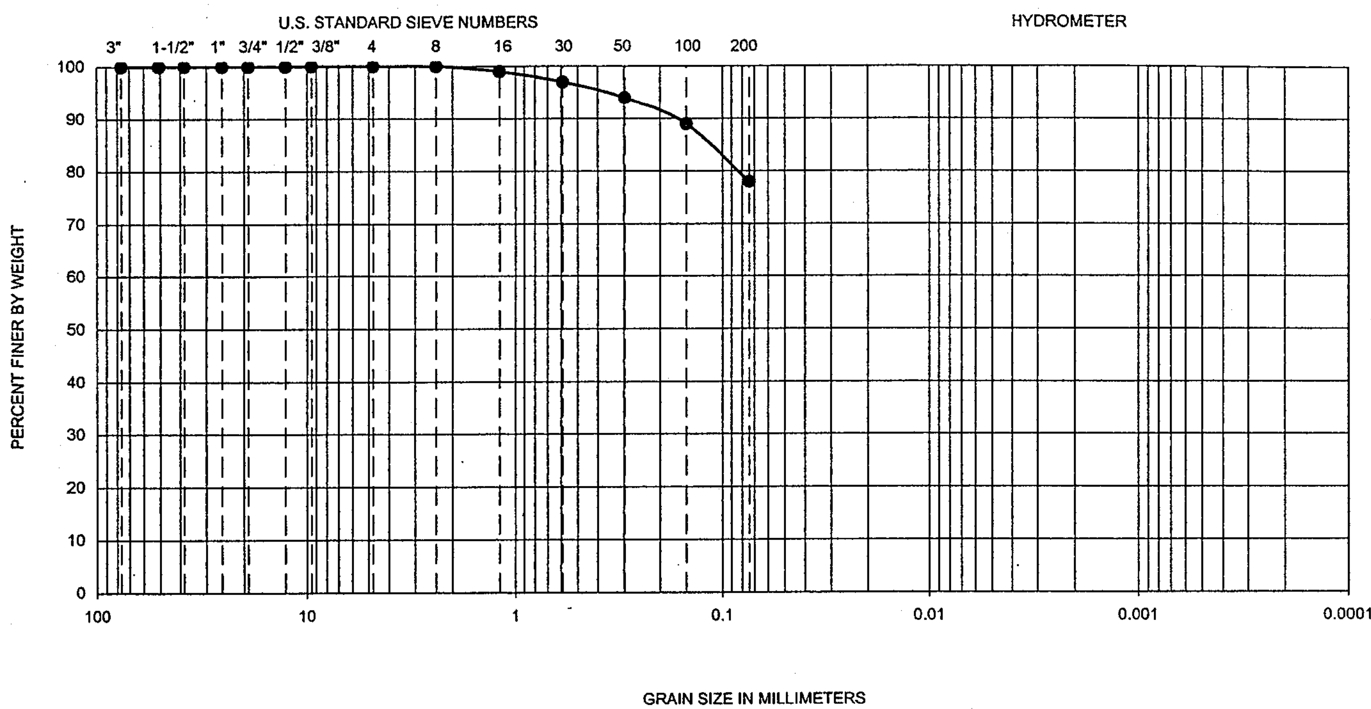
**GRADATION TEST RESULTS**

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-2 |

RH-1 25 TO 26.5 SIEVE+HYDRO1.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-2 | 2.5-4 | 34 | 8 | 26 | -- | -- | -- | -- | -- | 78 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

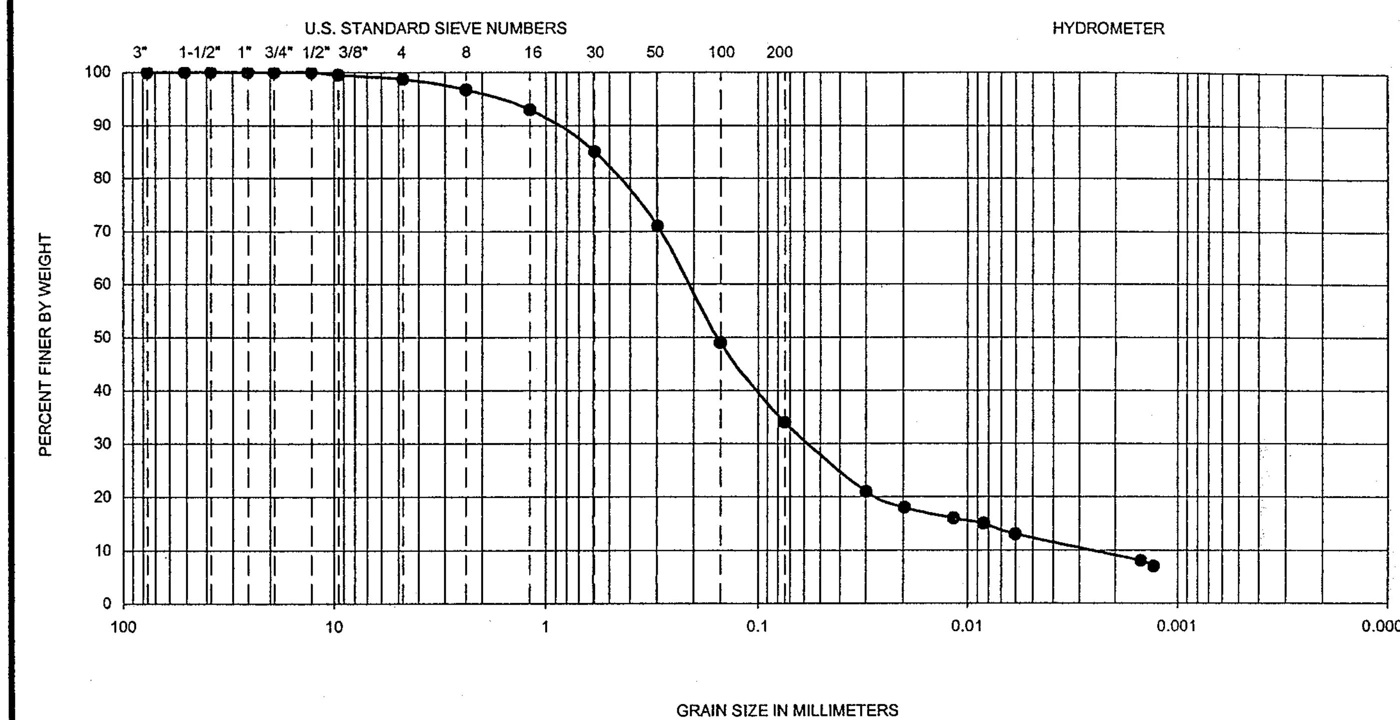
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-3 |

SIEVE RH-02@2.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-2 | 12.5-14.0 | 23 | 17 | 6 | 0.004 | 0.07 | 0.22 | 56.0 | 4.7 | 34 | SC-SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

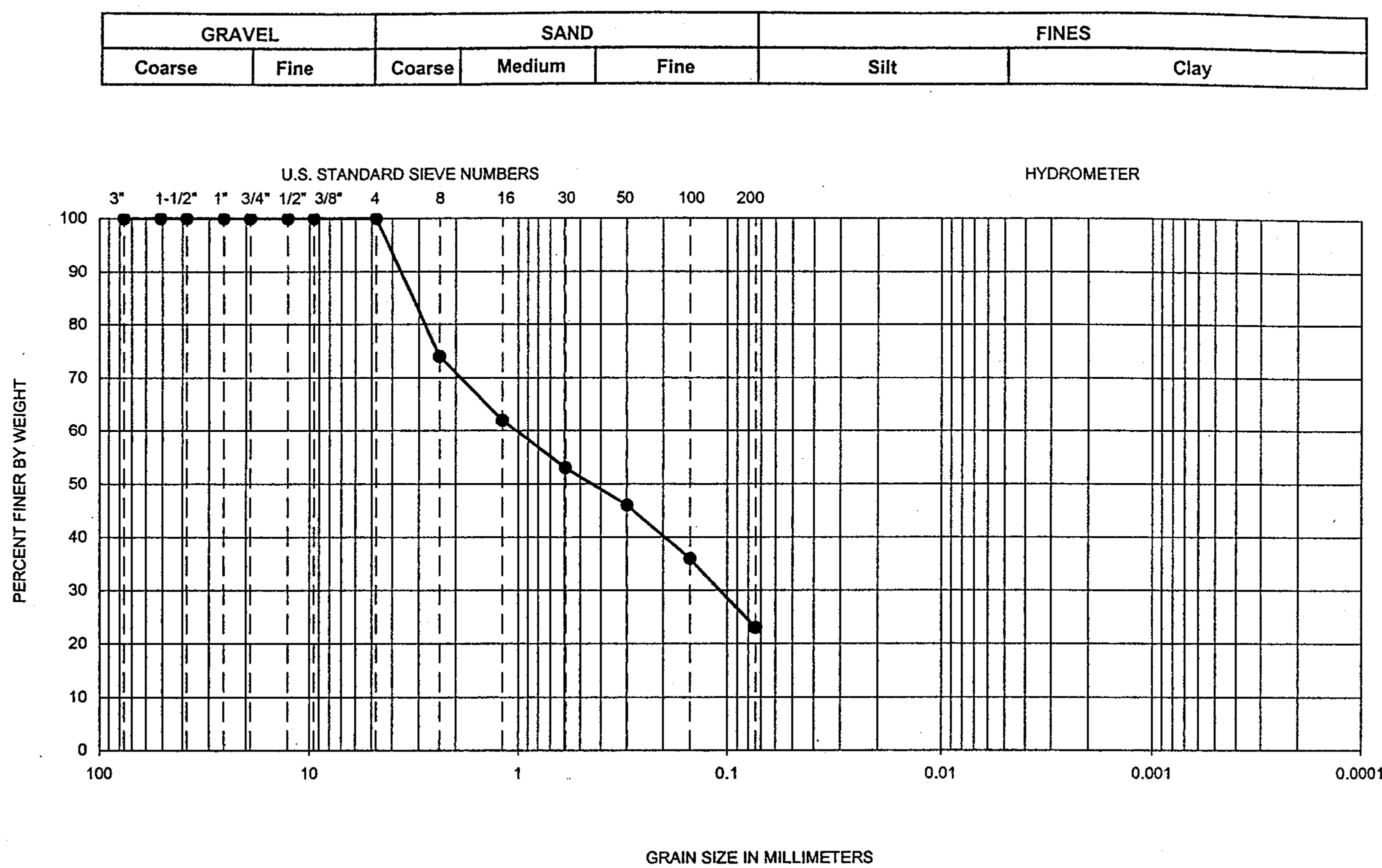
Ninyo & Moore

## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-4 |

RH-2 12.5 TO 14 SIEVE+HYDRO1.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-3 | 5-6.5 | 28 | 24 | 4 | -- | -- | -- | -- | -- | 23 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

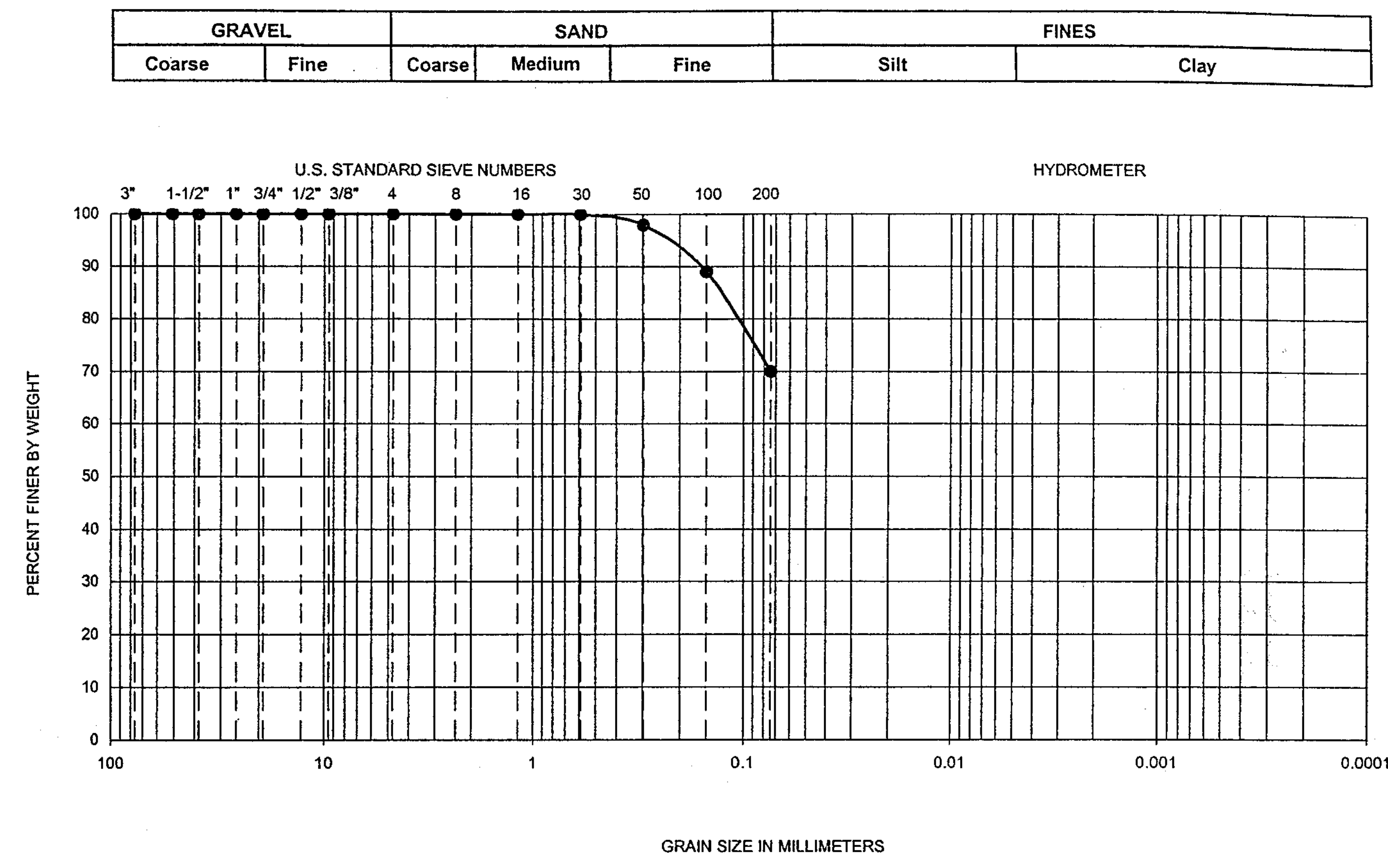
Ninyo & Moore

## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-5 |

SIEVE RH-03@5.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-4 | 5-6.5 | 27 | 15 | 12 | -- | -- | -- | -- | -- | 70 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

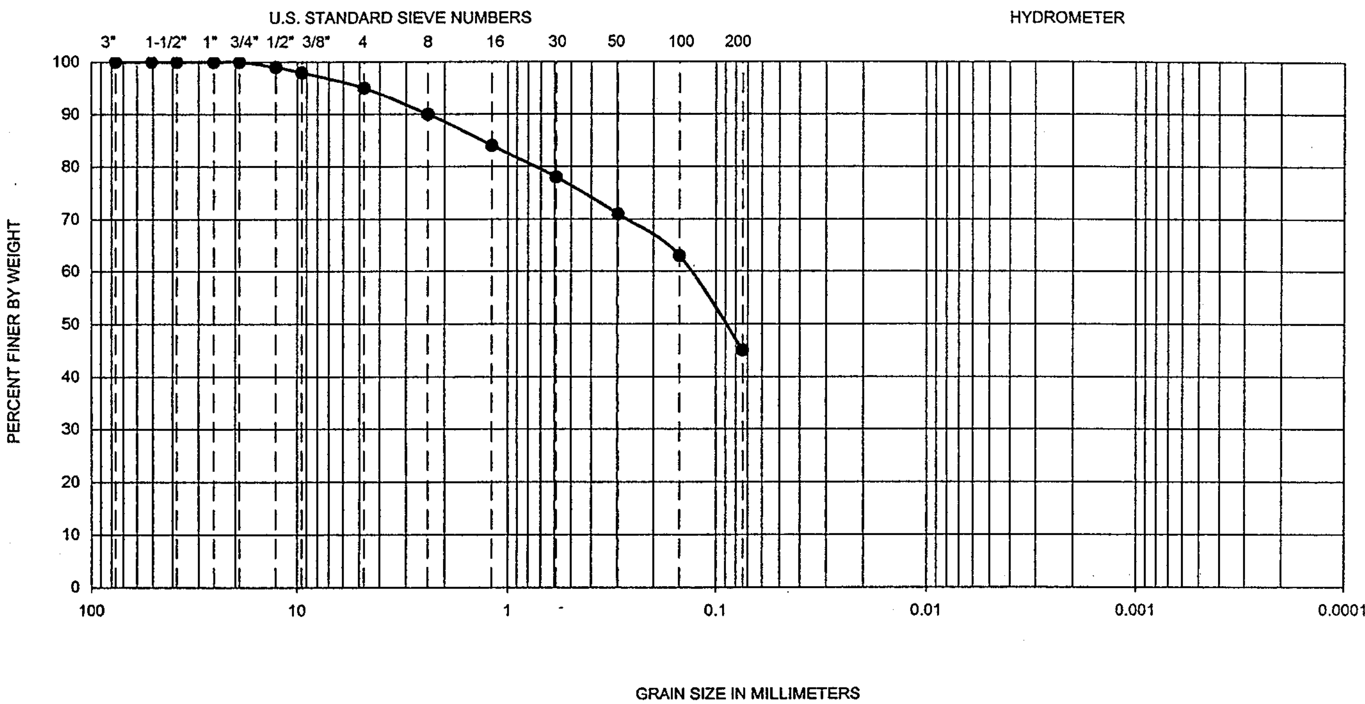
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-6 |

SIEVE RH-04@5.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-4 | 15-16.5 | 29 | 16 | 13 | -- | -- | -- | -- | -- | 45 | SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

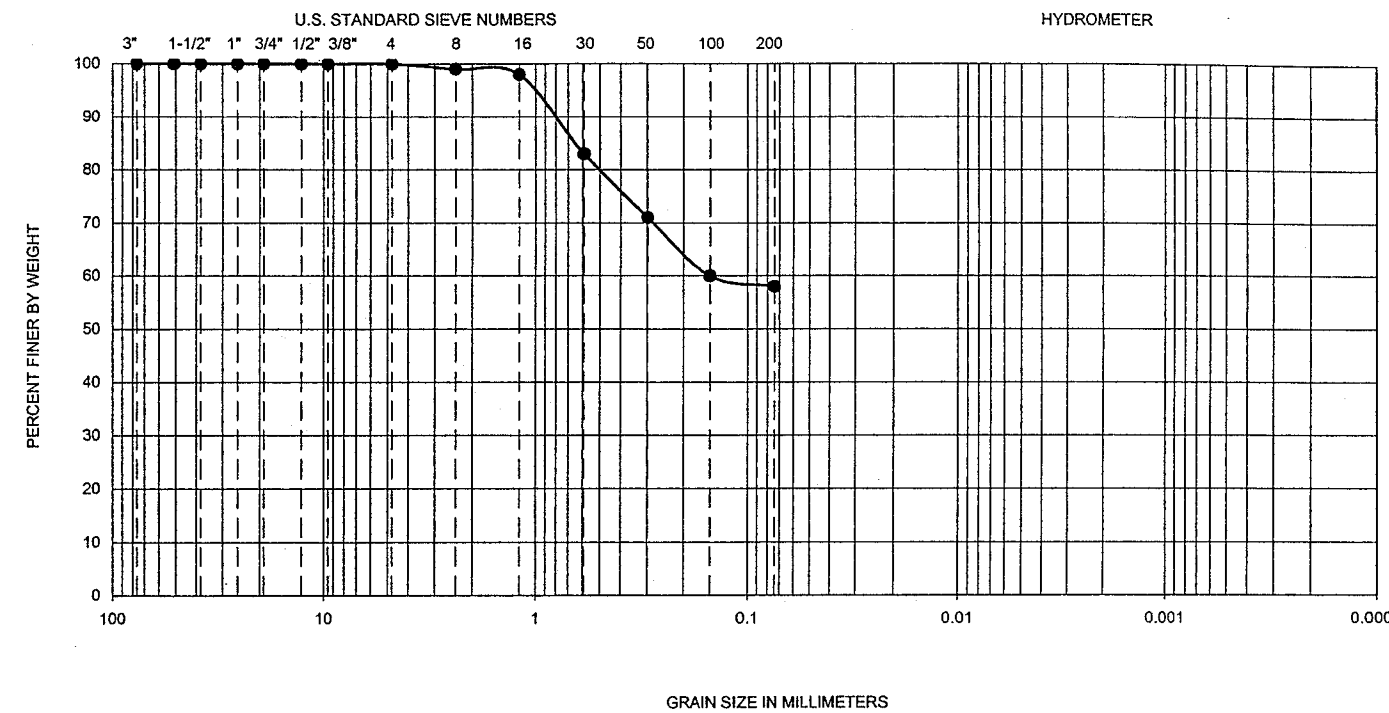
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-7 |

SIEVE RH-04@15.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-5 | 5-6.5 | 25 | 20 | 5 | -- | -- | -- | -- | -- | 58 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

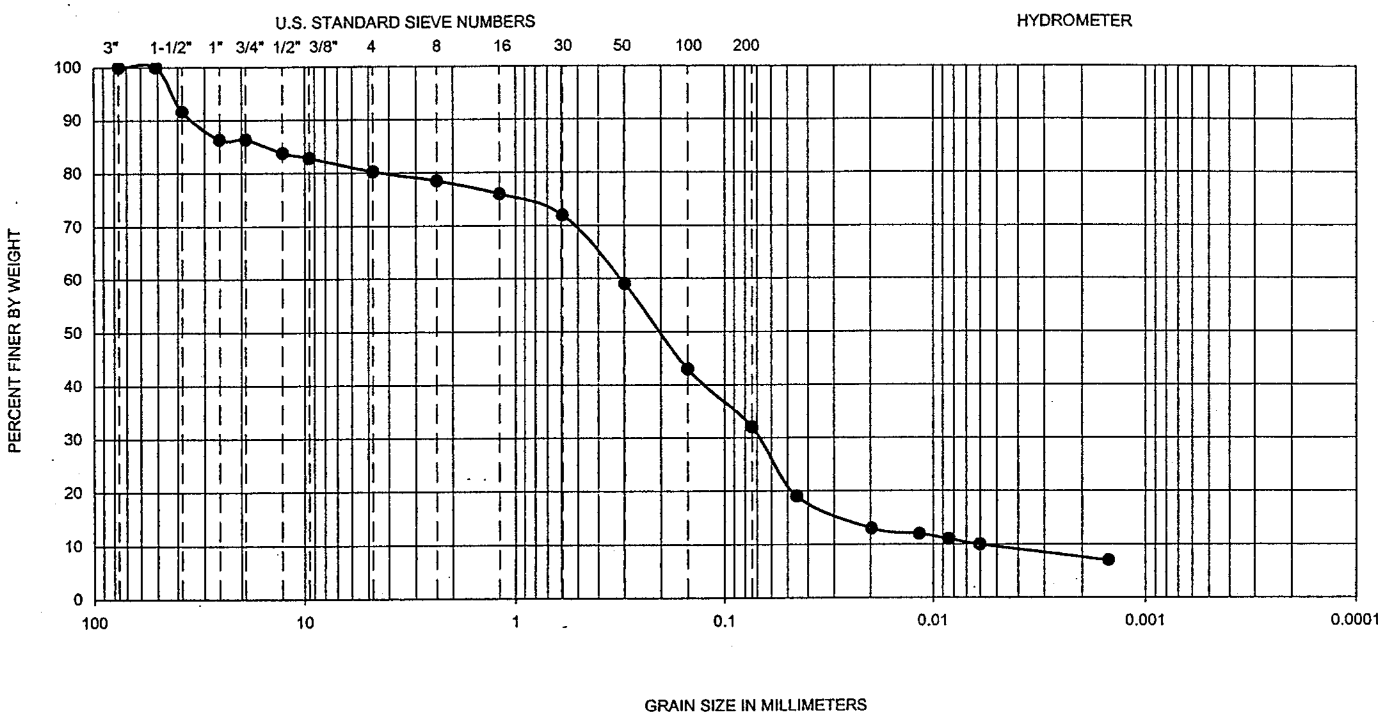
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-8 |

SIEVE RH-05@5.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-5 | 20.0-21.5 | 27 | 19 | 8 | 0.006 | 0.07 | 0.32 | 53.5 | 2.6 | 32 | SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

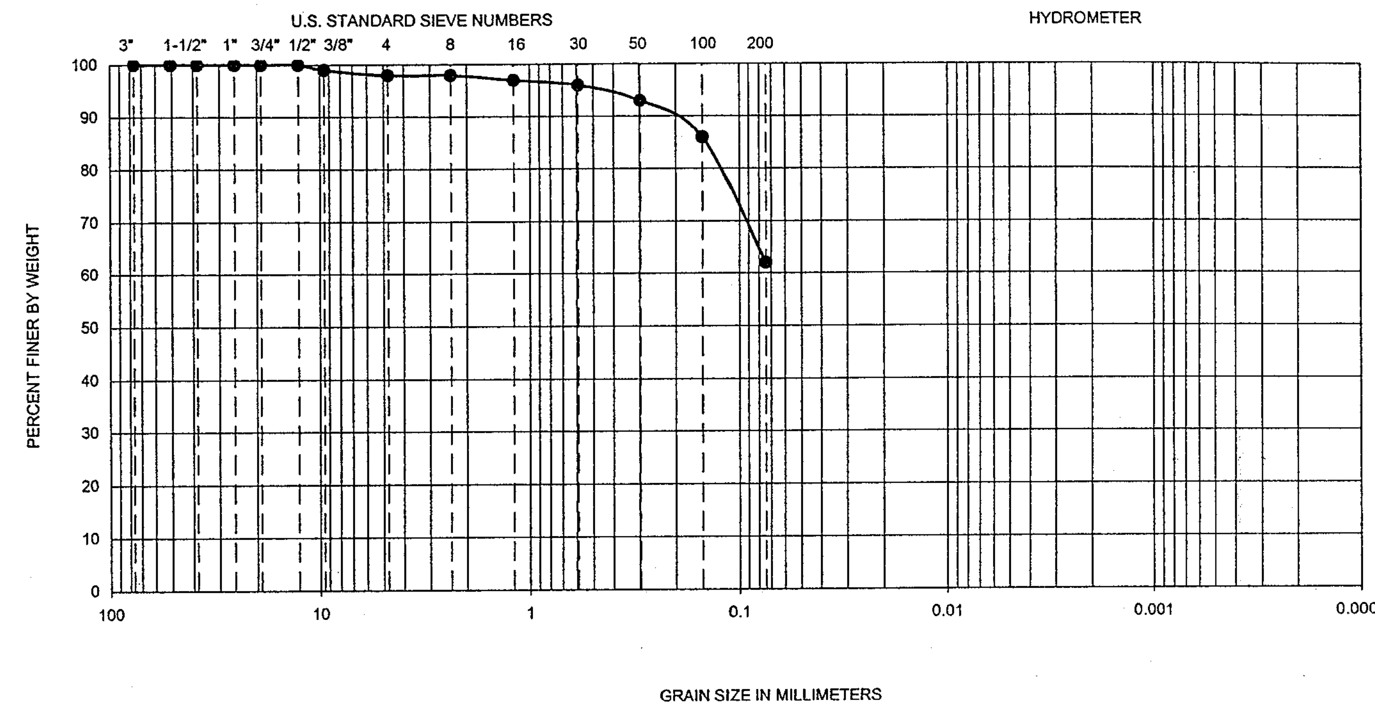
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-9 |

RH-5 20.0 TO 21.5 SIEVE+HYDRO1.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-6 | 10-11.5 | 28 | 19 | 9 | -- | -- | -- | -- | -- | 62 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

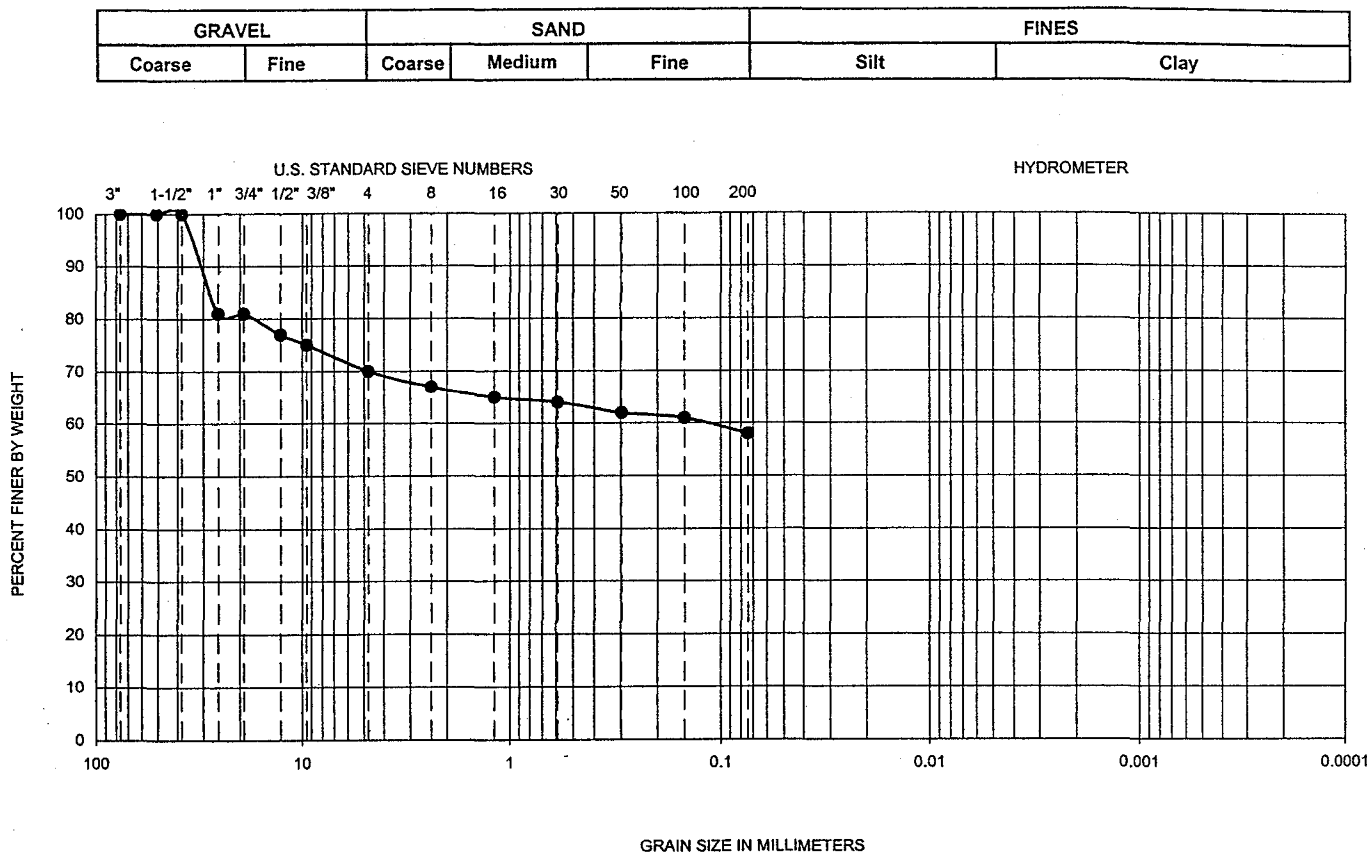
Ninyo & Moore

## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-10 |

SIEVE RH-06@10.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-6 | 15-16.5 | 32 | 19 | 13 | -- | -- | -- | -- | -- | 58 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

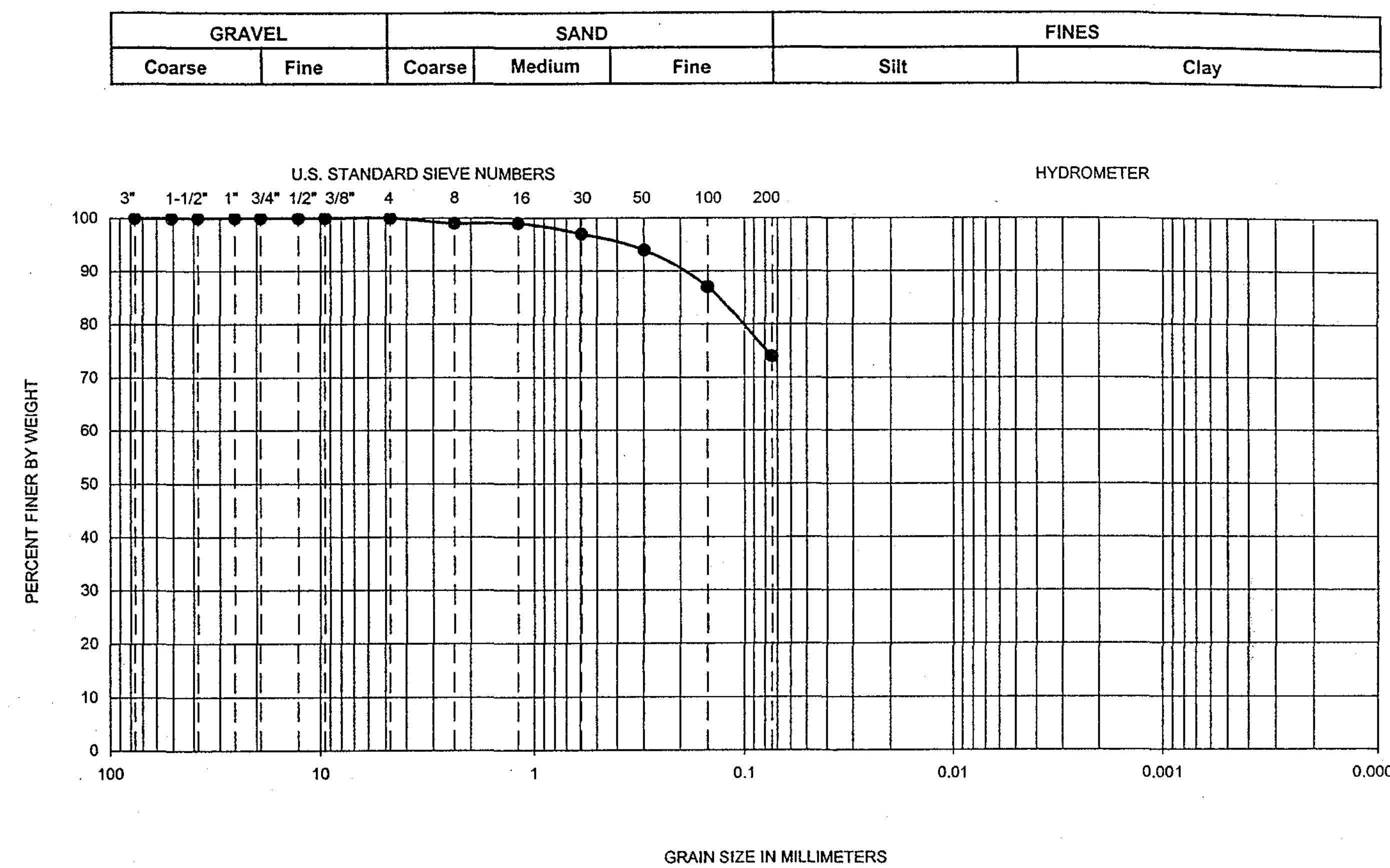
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

FIGURE B-11

SIEVE RH-06@15.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-7 | 2.5-4 | 30 | 16 | 14 | -- | -- | -- | -- | -- | 74 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

## GRADATION TEST RESULTS

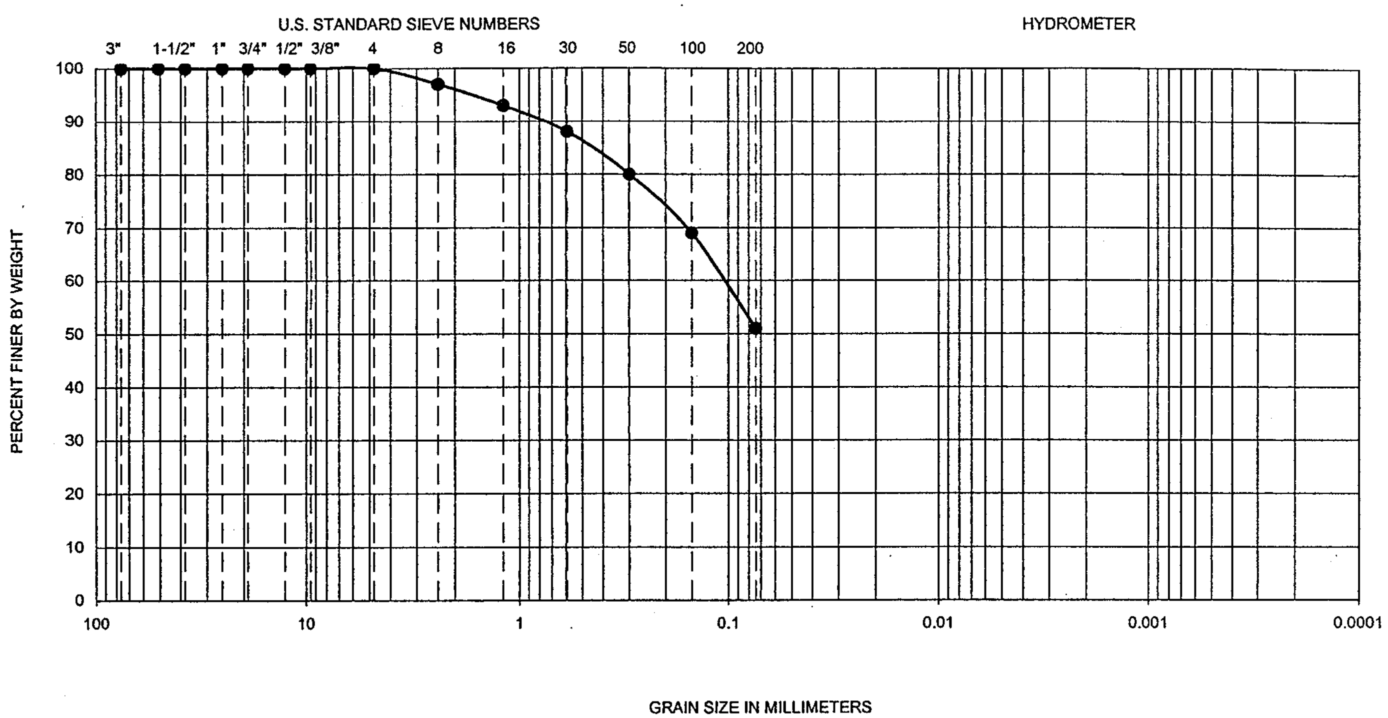
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

FIGURE B-12

SIEVE RH-07@2.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-7 | 17.5-18.5 | 32 | 19 | 13 | -- | -- | -- | -- | -- | 51 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

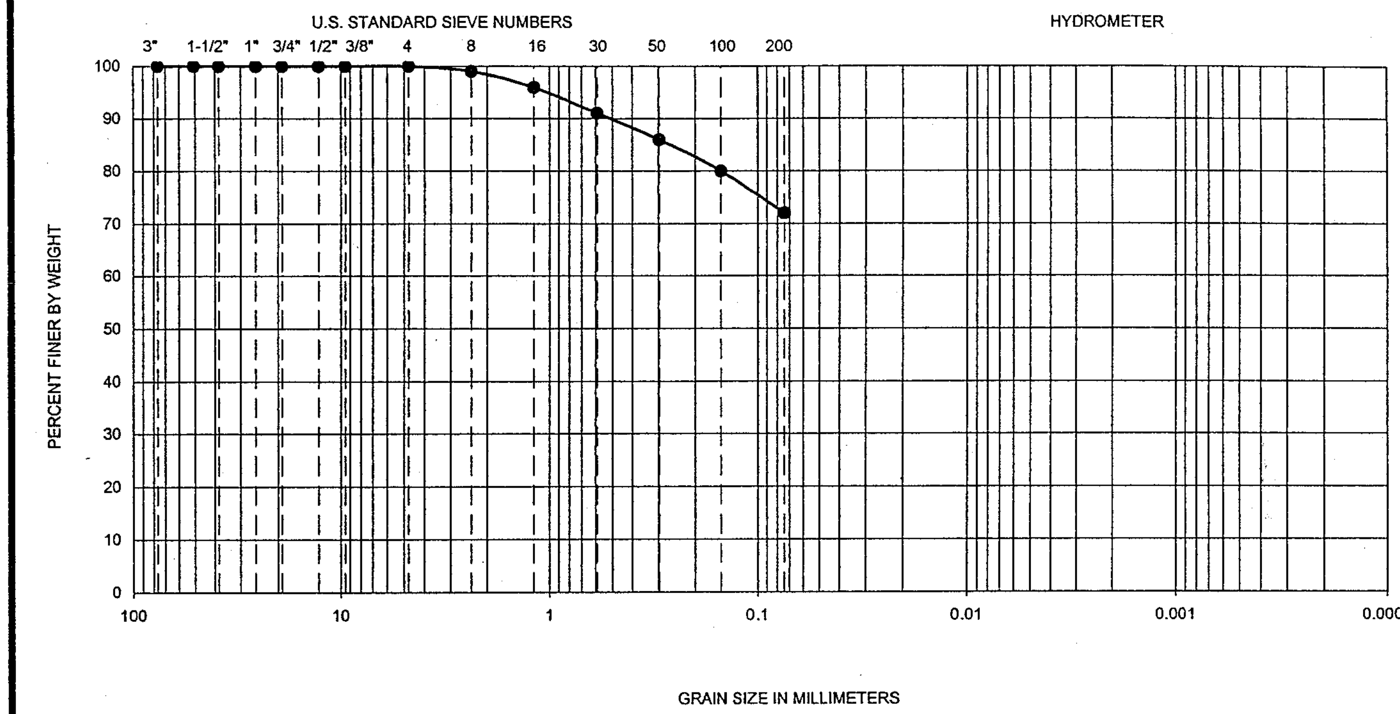
**GRADATION TEST RESULTS**

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-13 |

SIEVE RH-07@17.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-8 | 7.5-8.9 | 32 | 21 | 11 | -- | -- | -- | -- | -- | 72 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

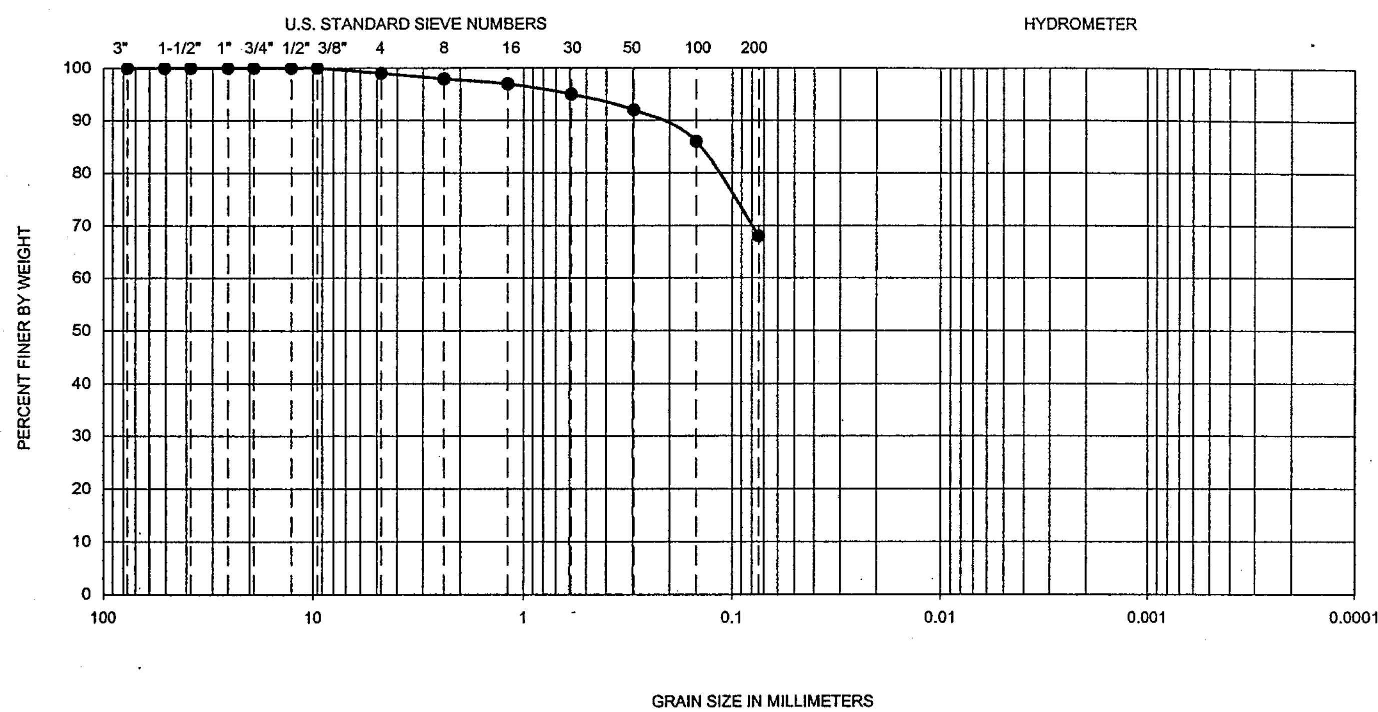
**GRADATION TEST RESULTS**

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-14 |

SIEVE RH-08@7.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-8 | 17.5-18.9 | 36 | 16 | 20 | -- | -- | -- | -- | -- | 68 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

## GRADATION TEST RESULTS

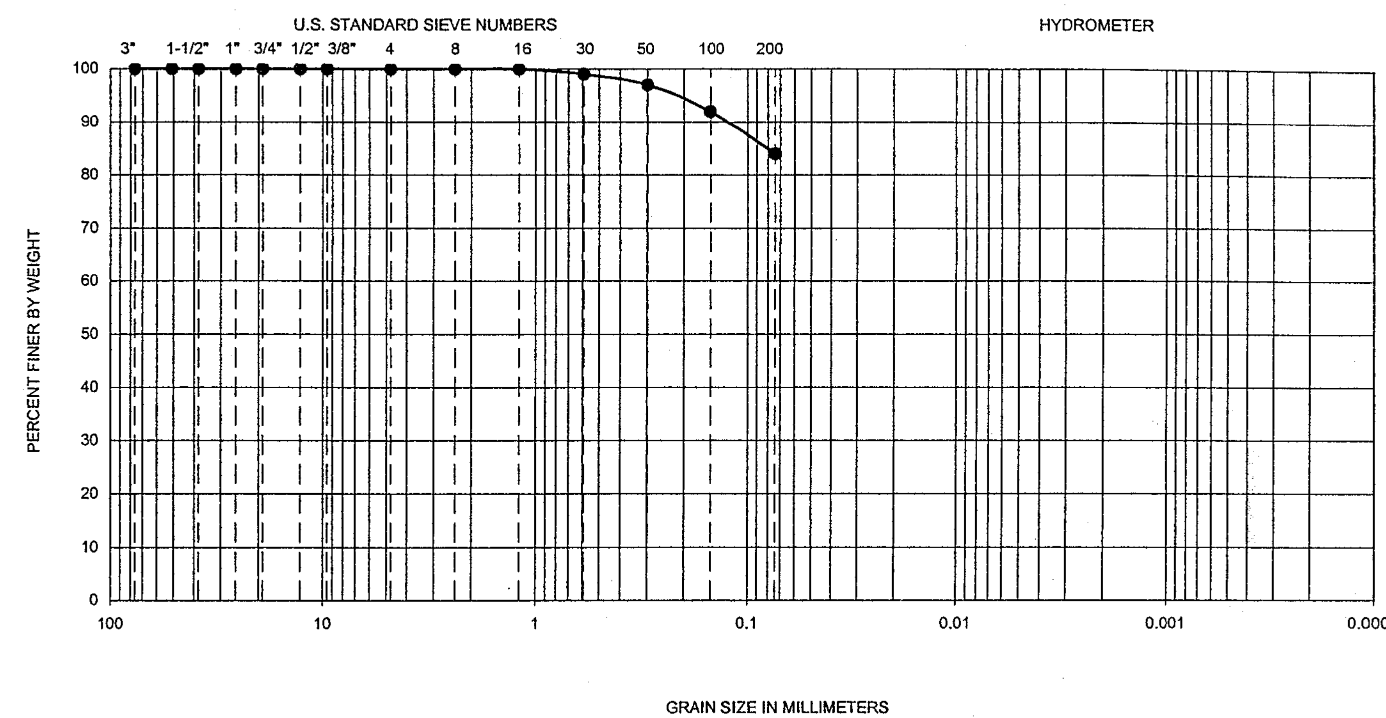
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

FIGURE
B-15

SIEVE RH-08@17.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-9 | 5-6.5 | 28 | 17 | 11 | -- | -- | -- | -- | -- | 84 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

## GRADATION TEST RESULTS

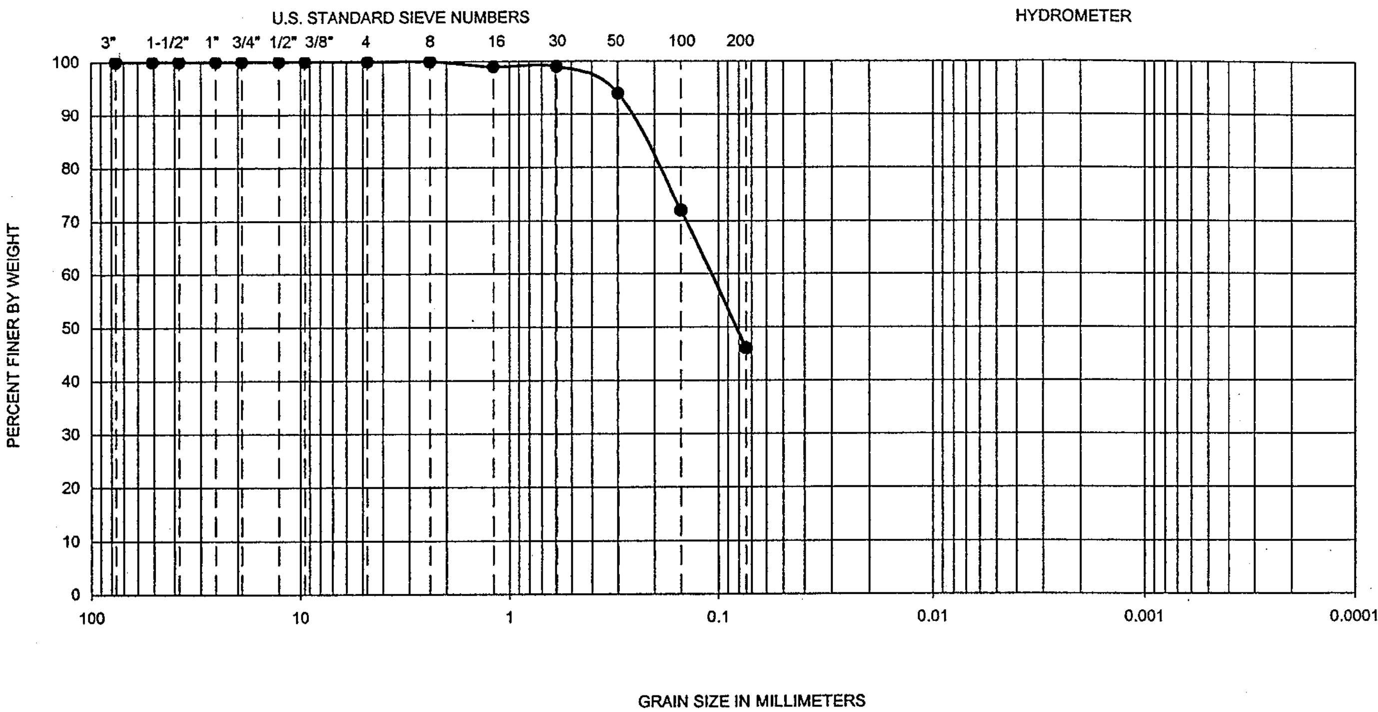
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

FIGURE
B-16

SIEVE RH-09@5.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-9 | 20-21.5 | -- | -- | NP | -- | -- | -- | -- | -- | 46 | SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

## GRADATION TEST RESULTS

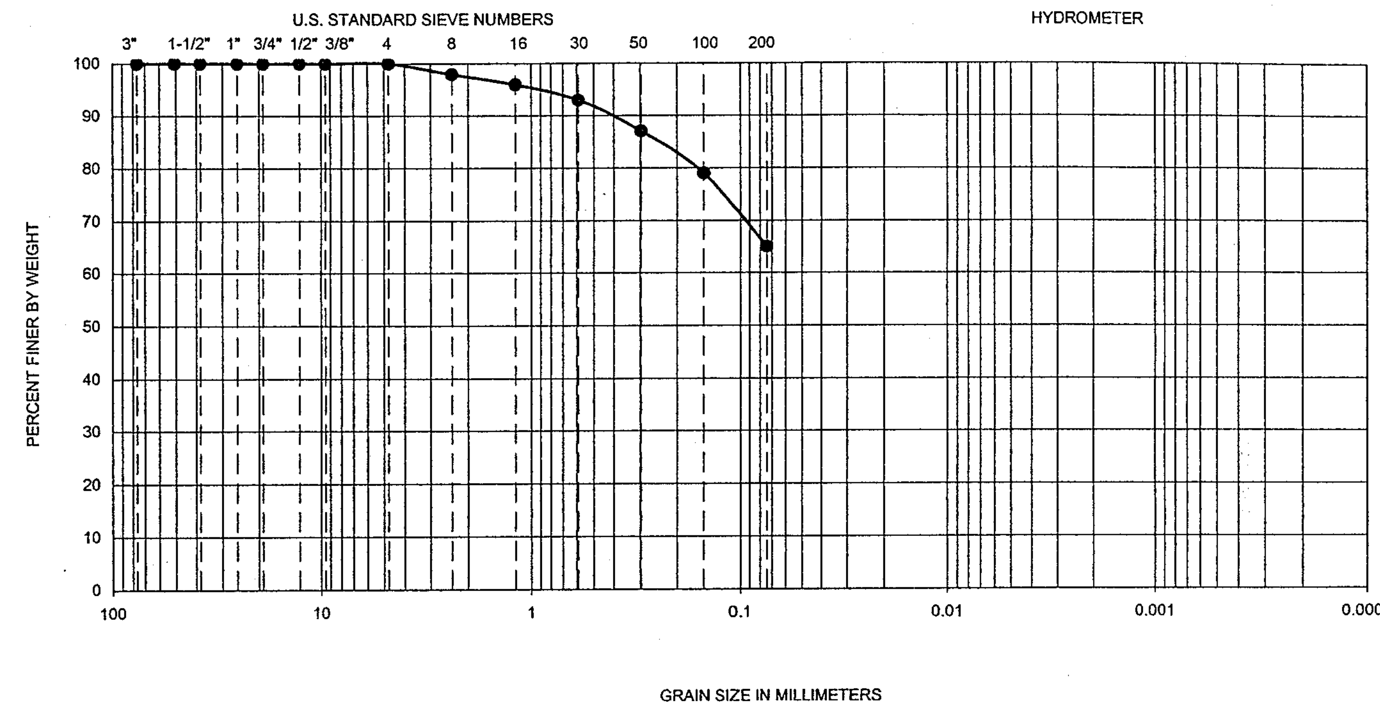
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

FIGURE B-17

SIEVE RH-09@20.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-10 | 12.5-14 | 30 | 23 | 7 | -- | -- | -- | -- | -- | 65 | ML |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

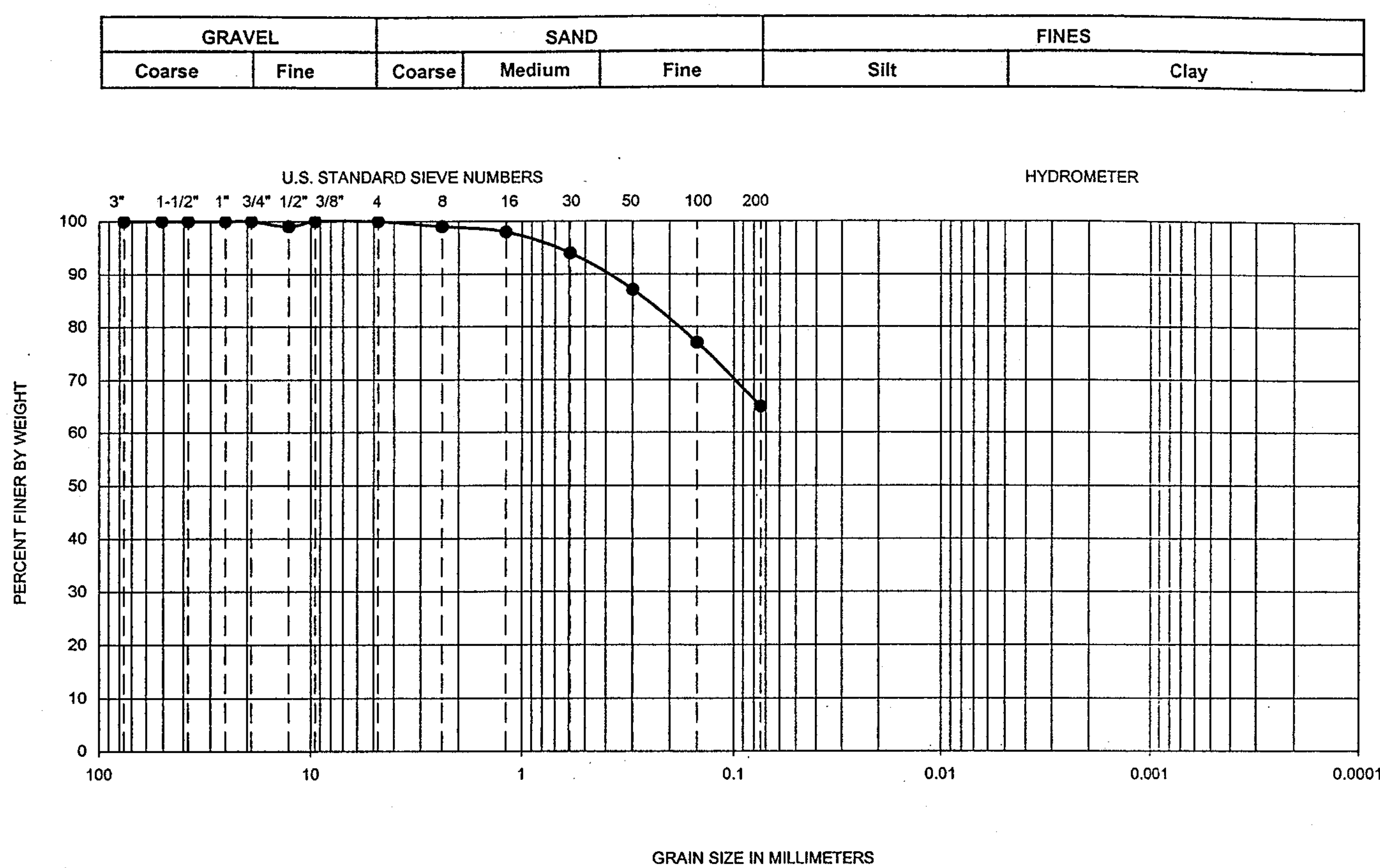
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

FIGURE B-18

SIEVE RH-10@12.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-11 | 10-11.5 | 36 | 19 | 17 | -- | -- | -- | -- | -- | 65 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

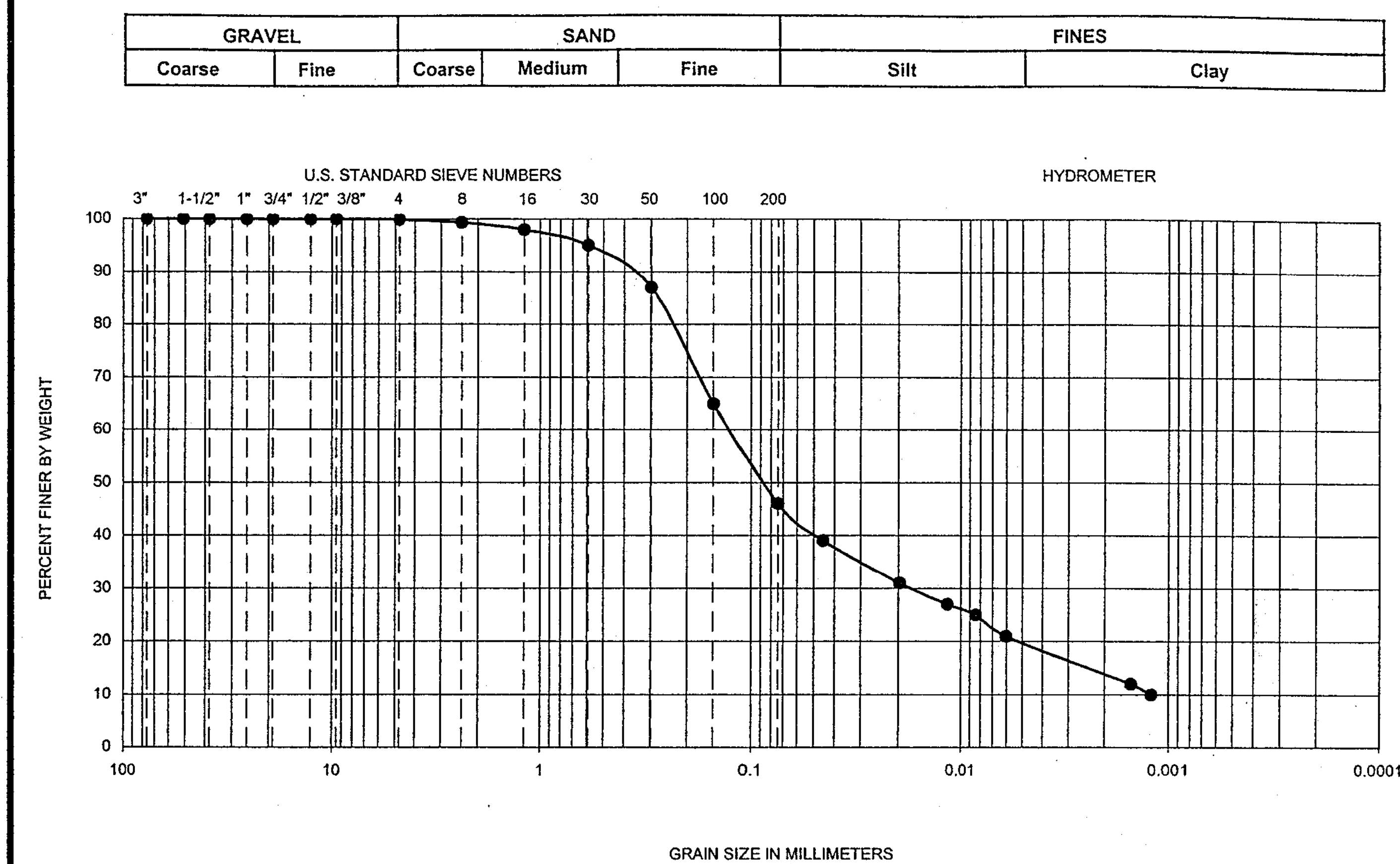
Ninyo & Moore

## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-19 |






| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-11 | 17.5-19.0 | 35 | 17 | 8 | 0.001 | 0.02 | 0.13 | 129.0 | 2.8 | 46 | SC |

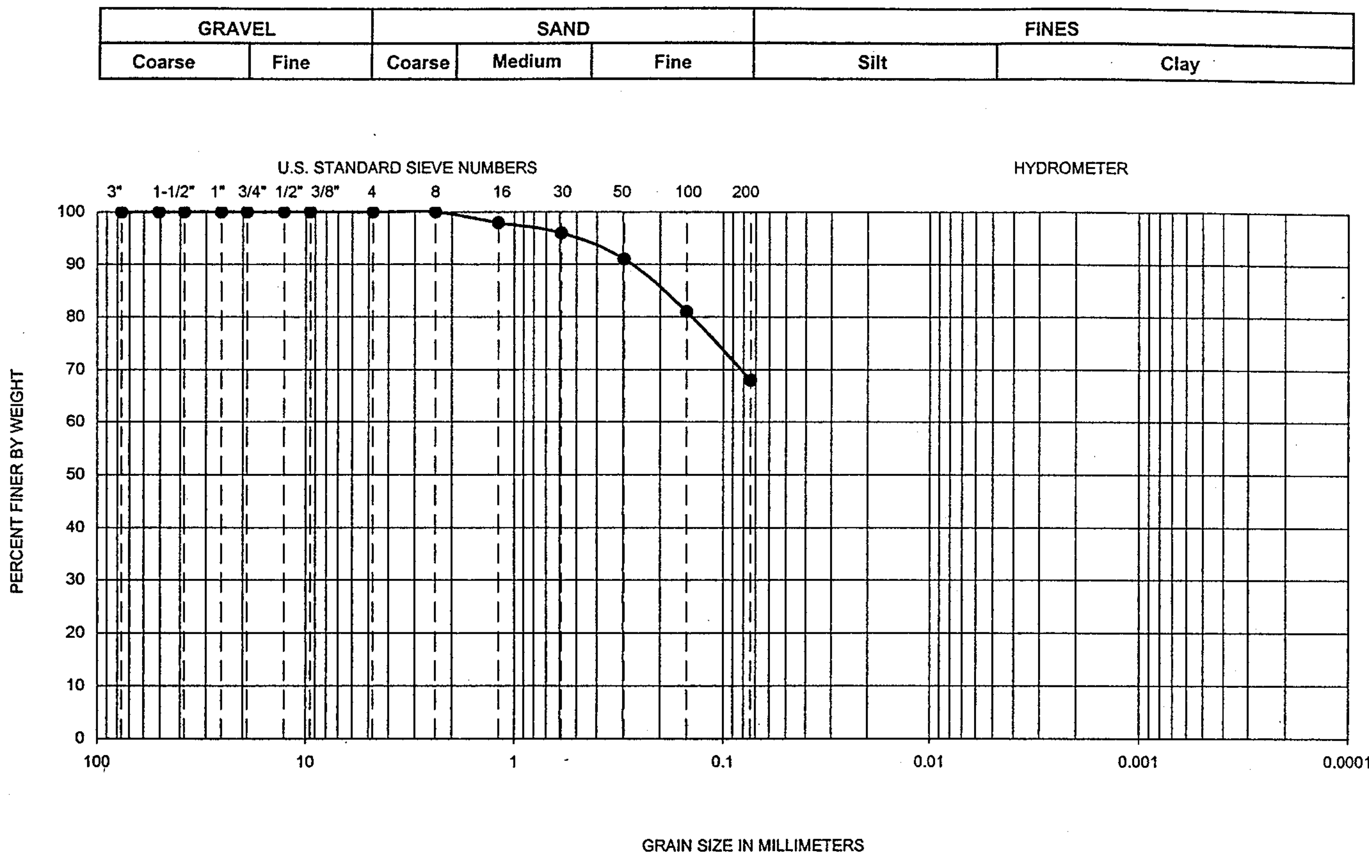
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-20 |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-12 | 5-5.5 | -- | -- | NP | -- | -- | -- | -- | -- | 68 | ML |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

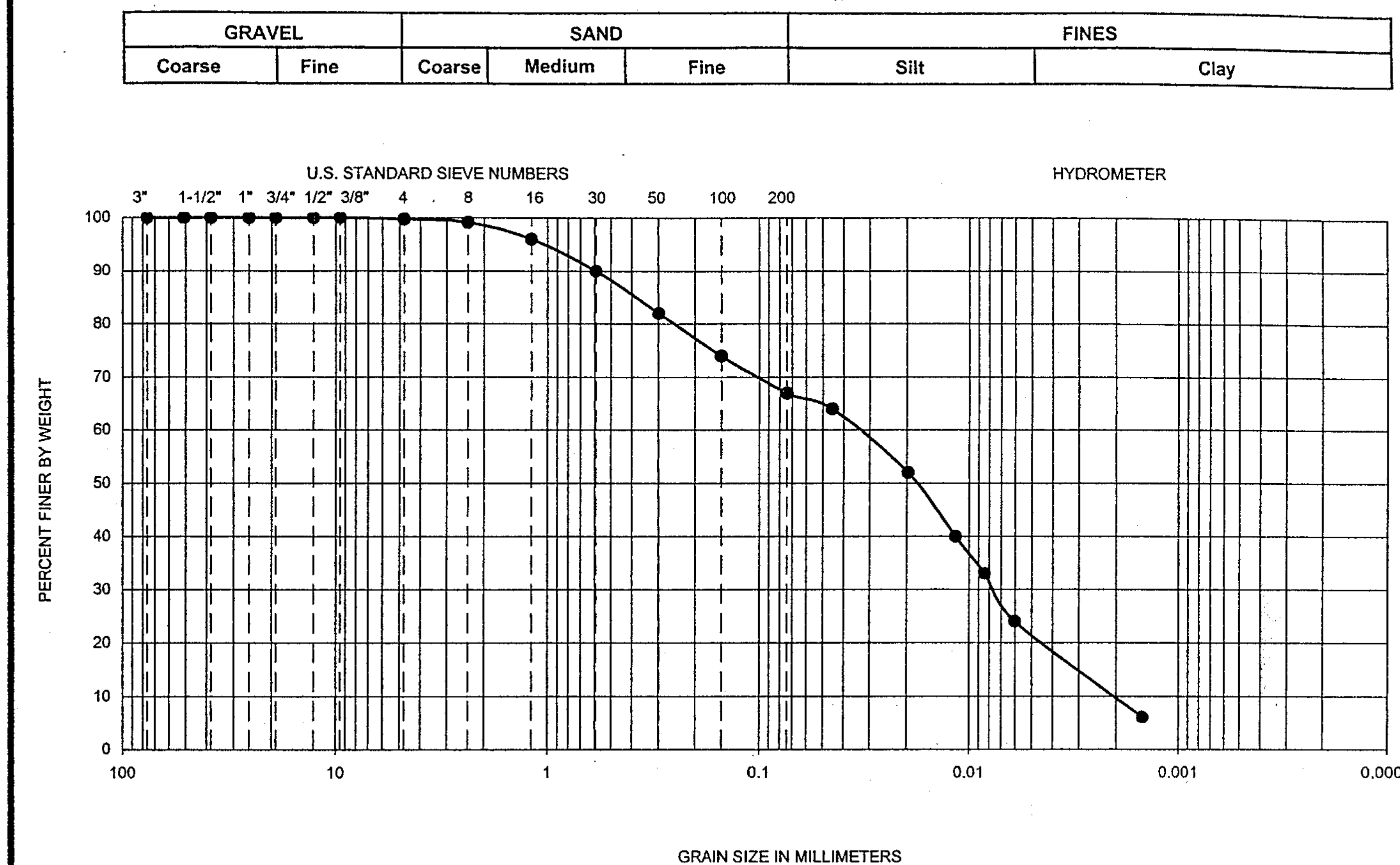
Ninyo & Moore

## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-21 |

SIEVE RH-12@5.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-12 | 10.0-11.5 | -- | -- | NP | 0.002 | 0.01 | 0.03 | 15.0 | 1.1 | 67 | ML |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

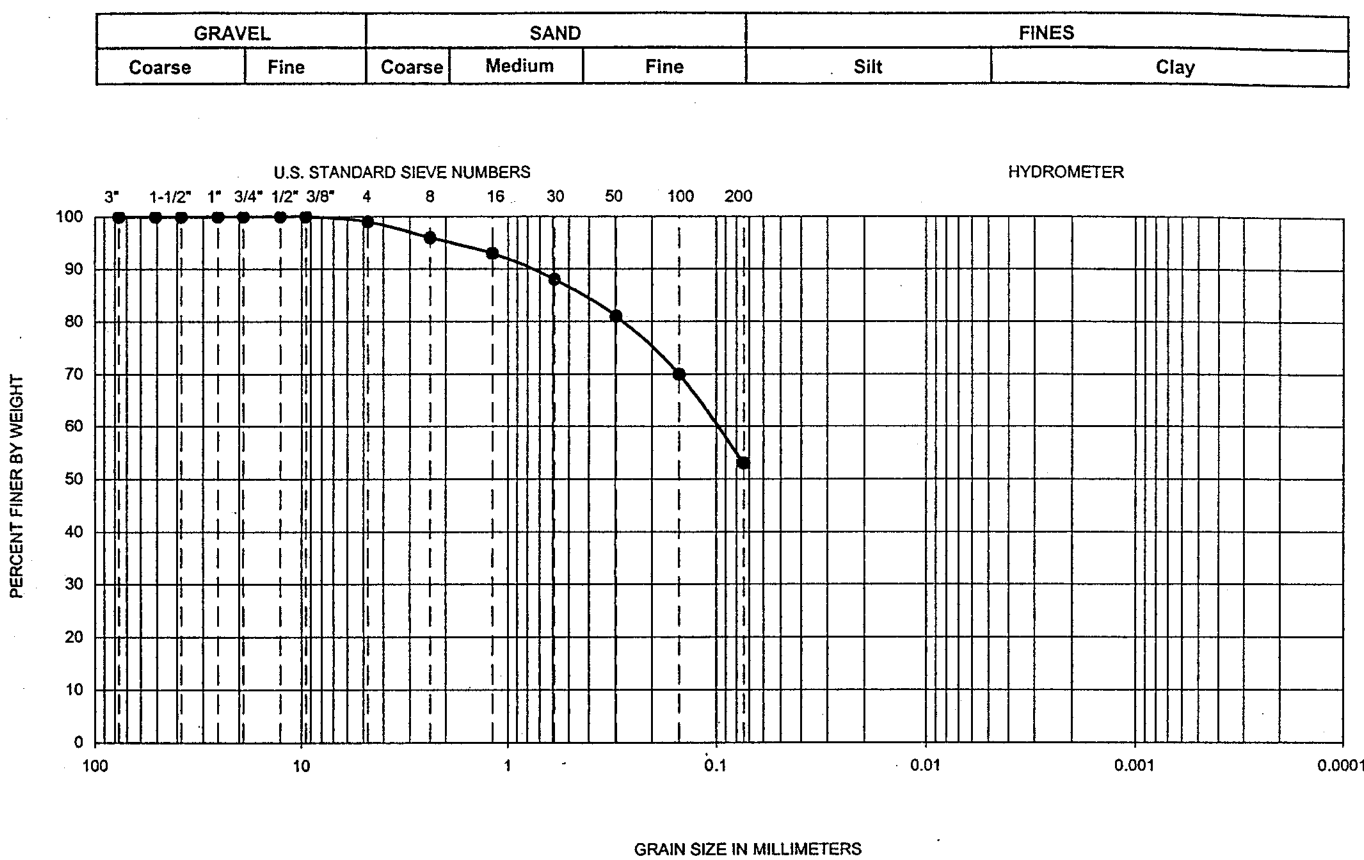
Ninyo & Moore

## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-22 |

RH-12 10.0 TO 11.5 SIEVE+HYDRO1.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-12 | 15-15.4 | 26 | 18 | 8 | -- | -- | -- | -- | -- | 53 | CL |

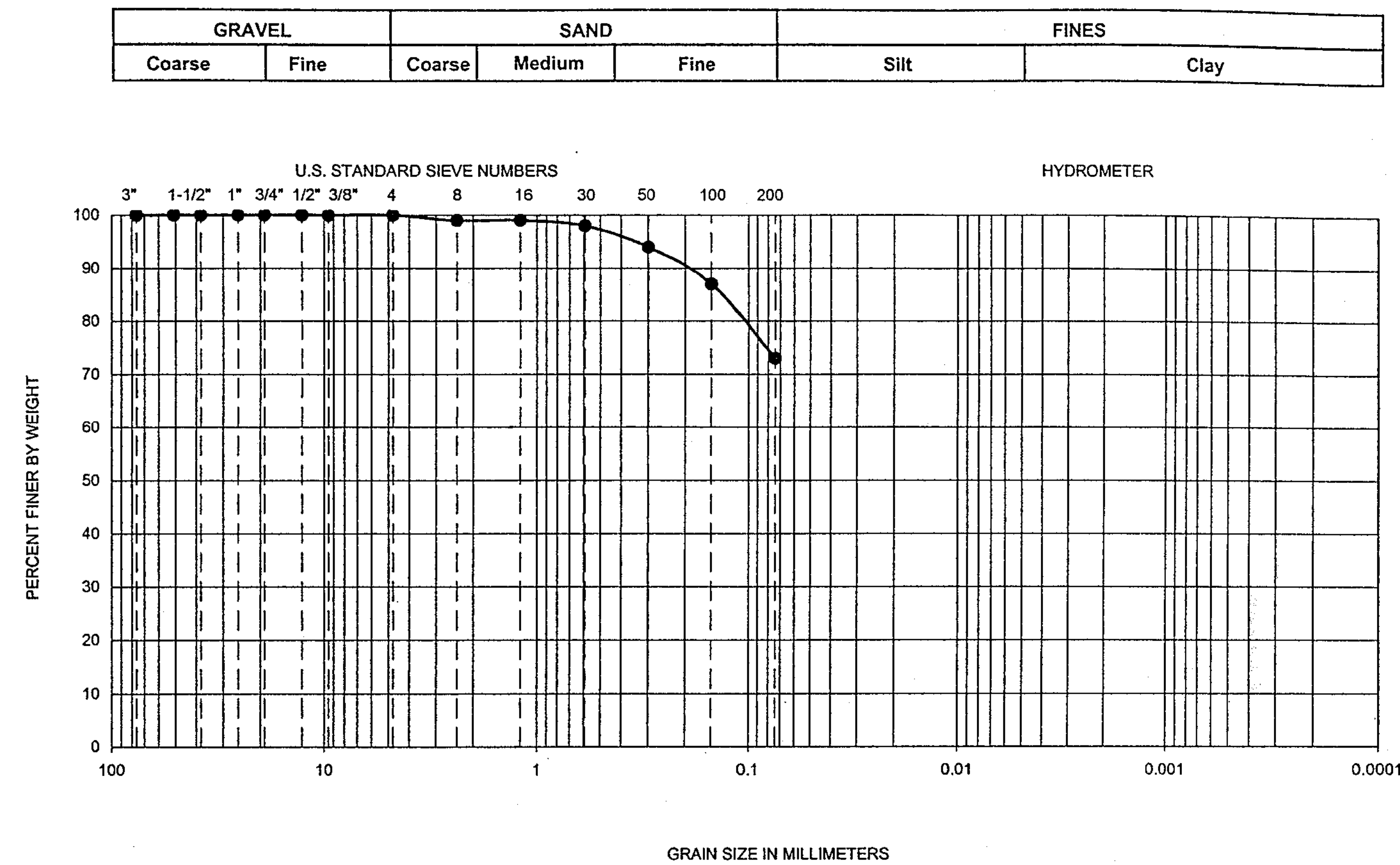
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

**GRADATION TEST RESULTS**
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-23 |

SIEVE RH-12@15.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-13 | 5-6.5 | 43 | 17 | 26 | -- | -- | -- | -- | -- | 73 | CL |

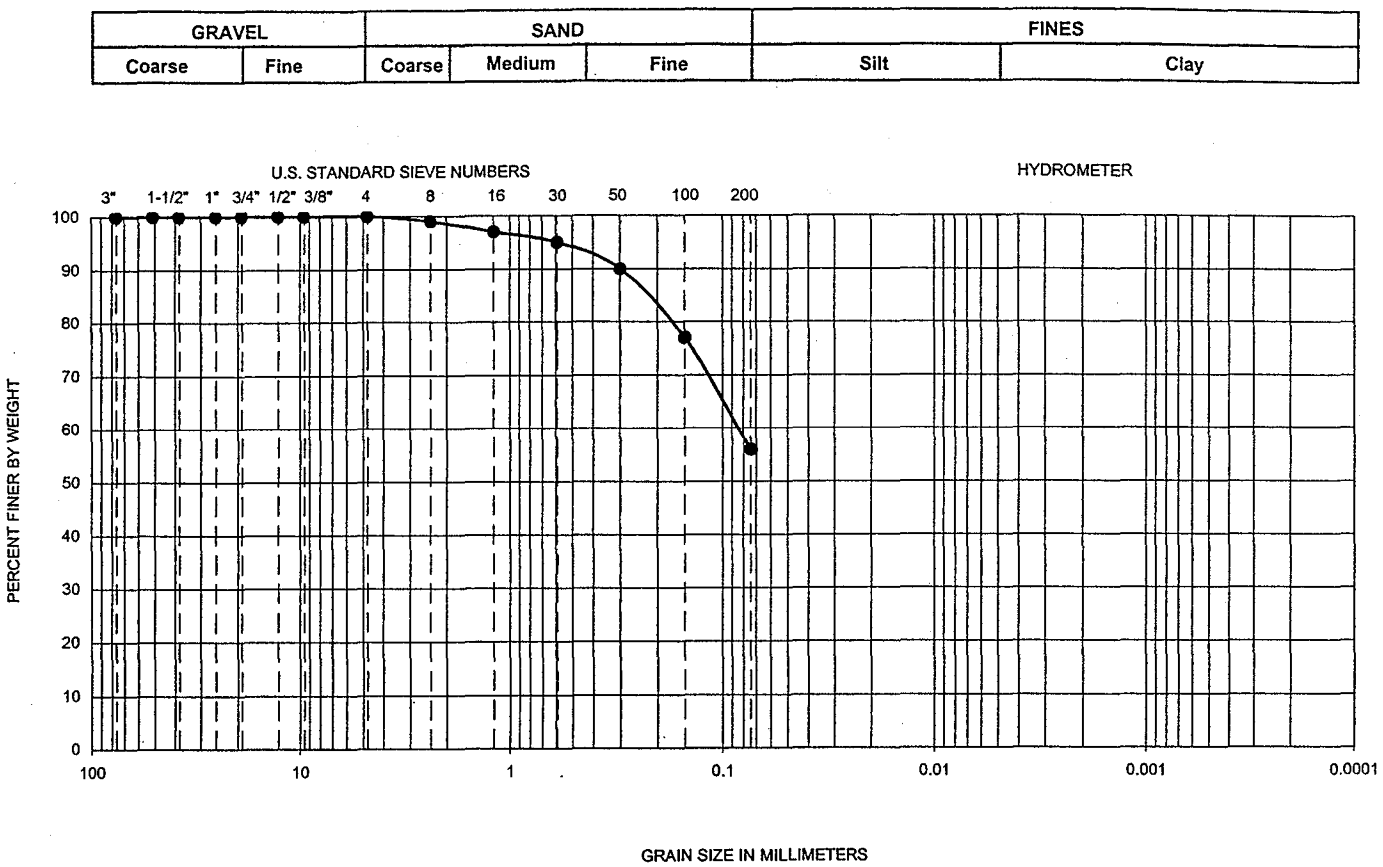
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

**GRADATION TEST RESULTS**
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-24 |

SIEVE RH-13@5.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-13 | 20-21.5 | -- | -- | NP | -- | -- | -- | -- | -- | 56 | ML |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

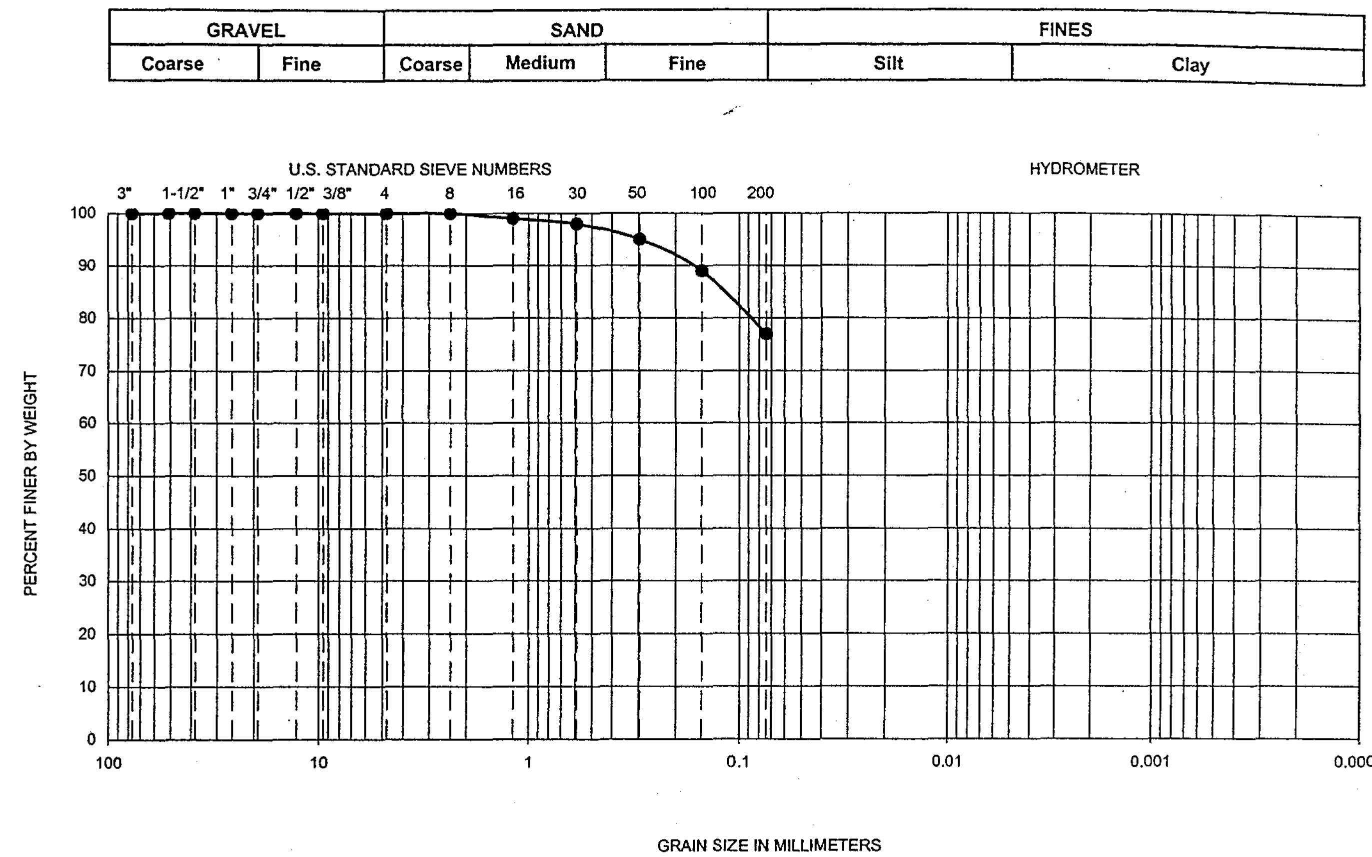
Ninyo & Moore

## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-25 |

SIEVE RH-13@20.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-14 | 2.5-3.5 | 29 | 16 | 13 | -- | -- | -- | -- | -- | 77 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

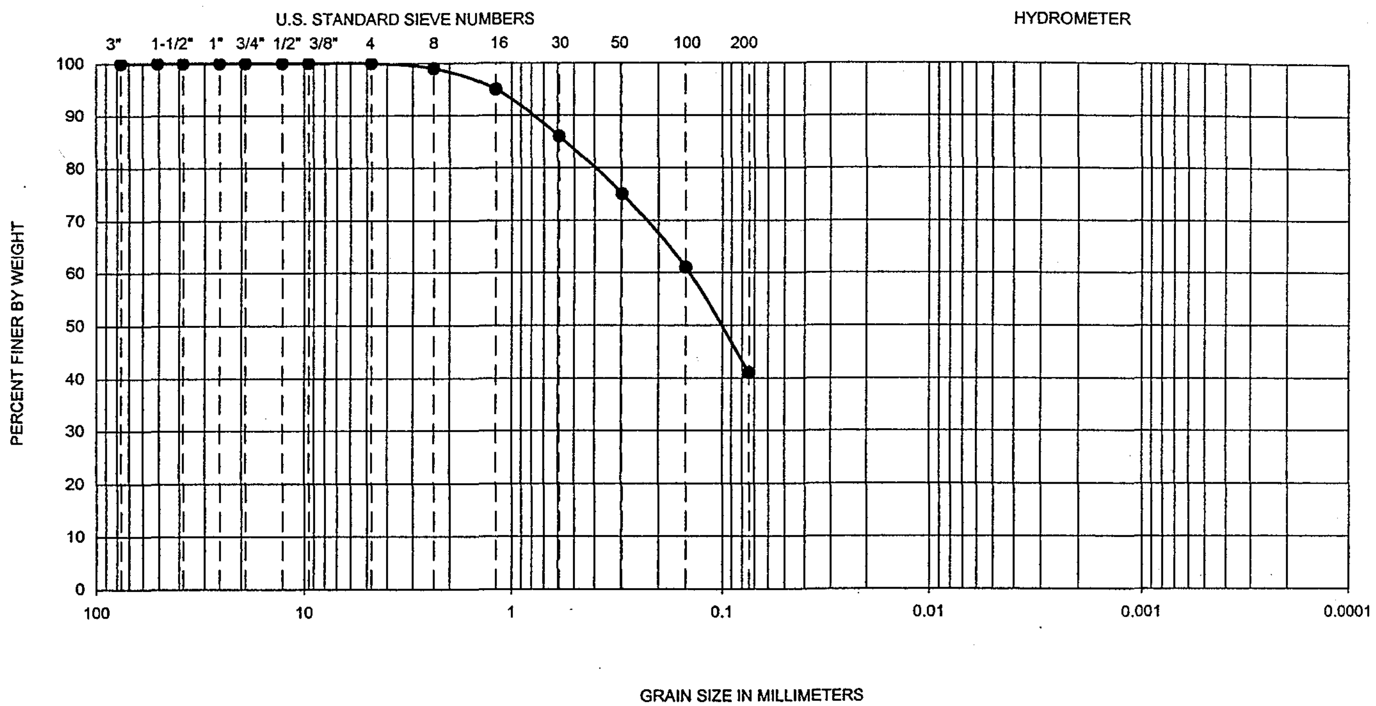
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-26 |

SIEVE RH-14@2.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-14 | 15-15.8 | 30 | 18 | 12 | -- | -- | -- | -- | -- | 41 | SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

## GRADATION TEST RESULTS

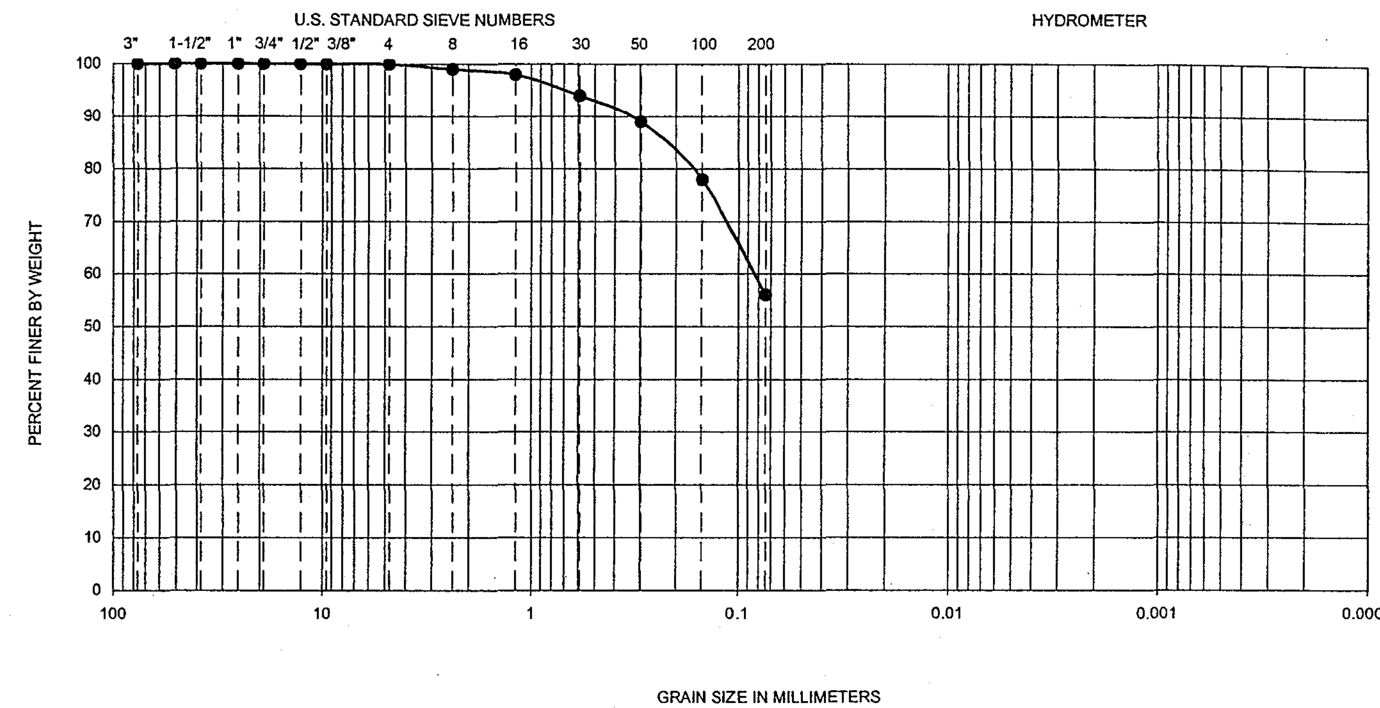
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

FIGURE B-27

SIEVE RH-14@15.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-15 | 5-5.9 | -- | -- | NP | -- | -- | -- | -- | -- | 56 | ML |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

## GRADATION TEST RESULTS

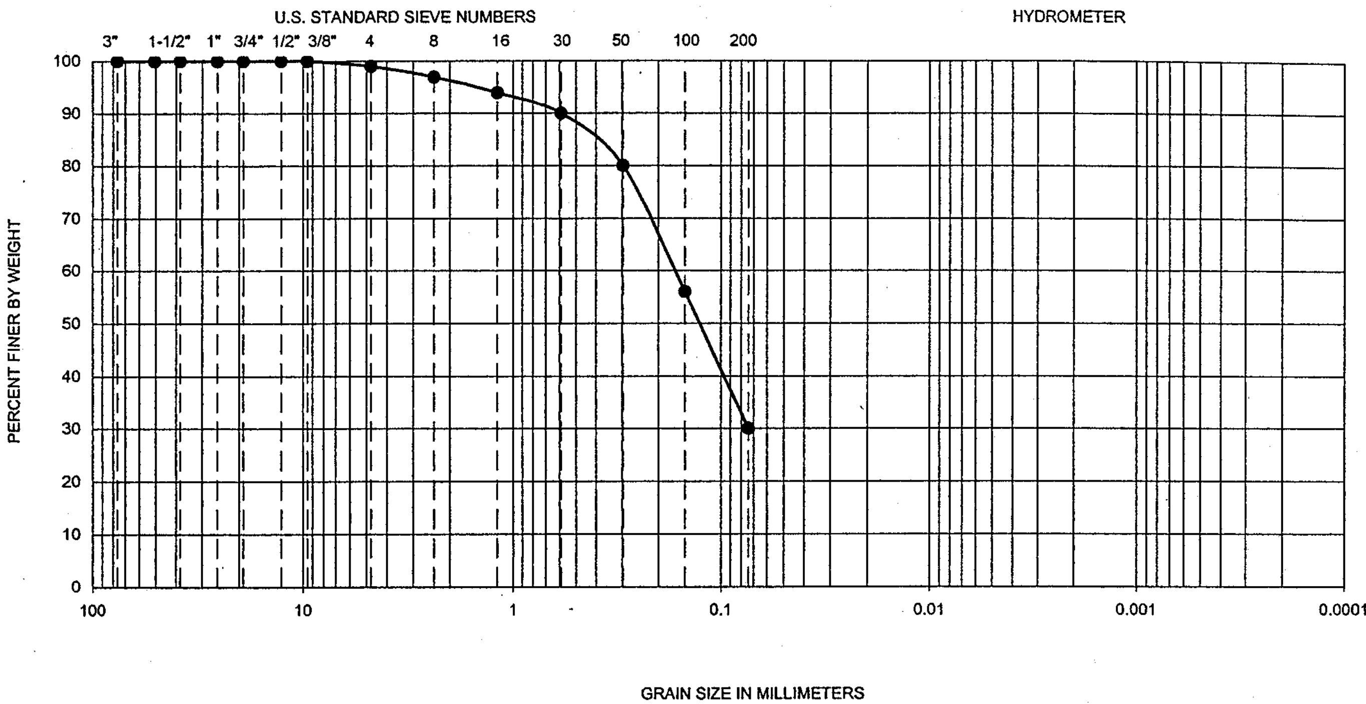
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE |
|---|---|
| 600198001 | 12/01 |

FIGURE B-28

SIEVE RH-15@5.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-15 | 15-16.5 | -- | -- | NP | -- | -- | -- | -- | -- | 30 | SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

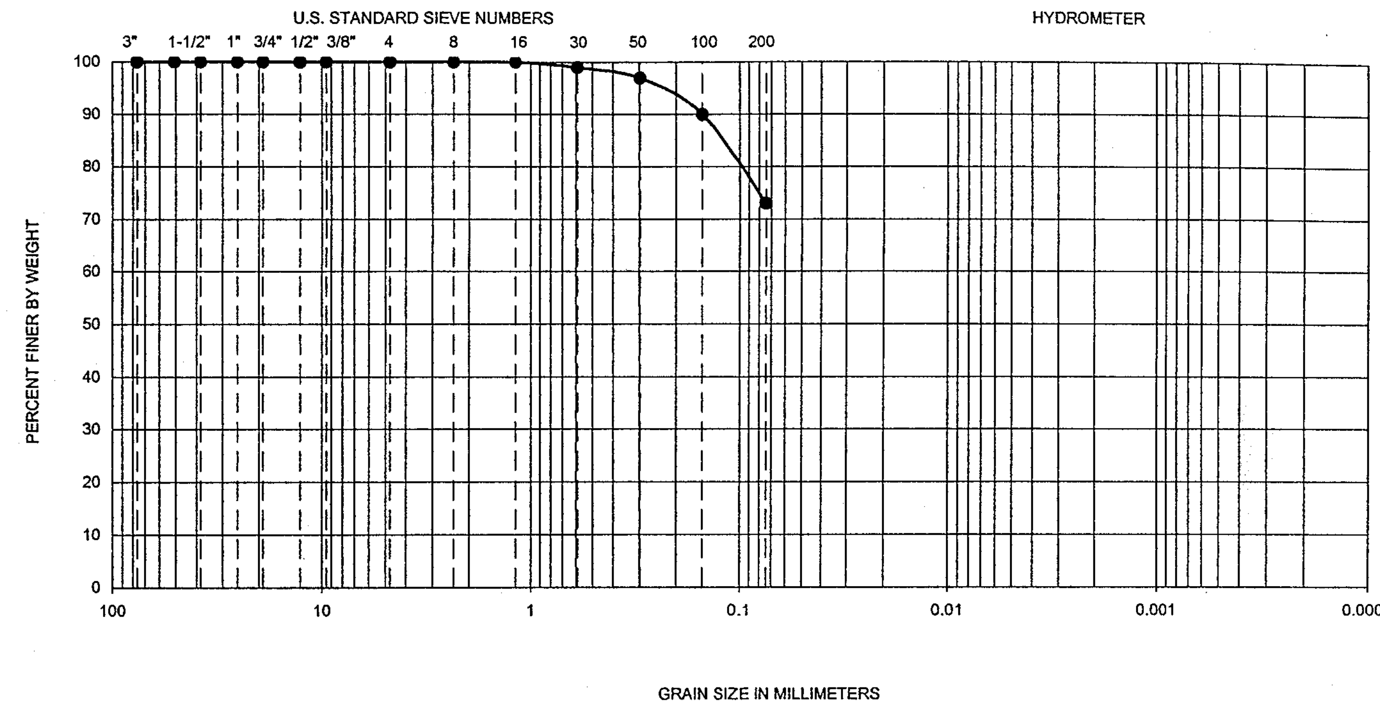
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-29 |

SIEVE RH-15@15.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-16 | 2.5-4 | 27 | 16 | 11 | -- | -- | -- | -- | -- | 73 | CL |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

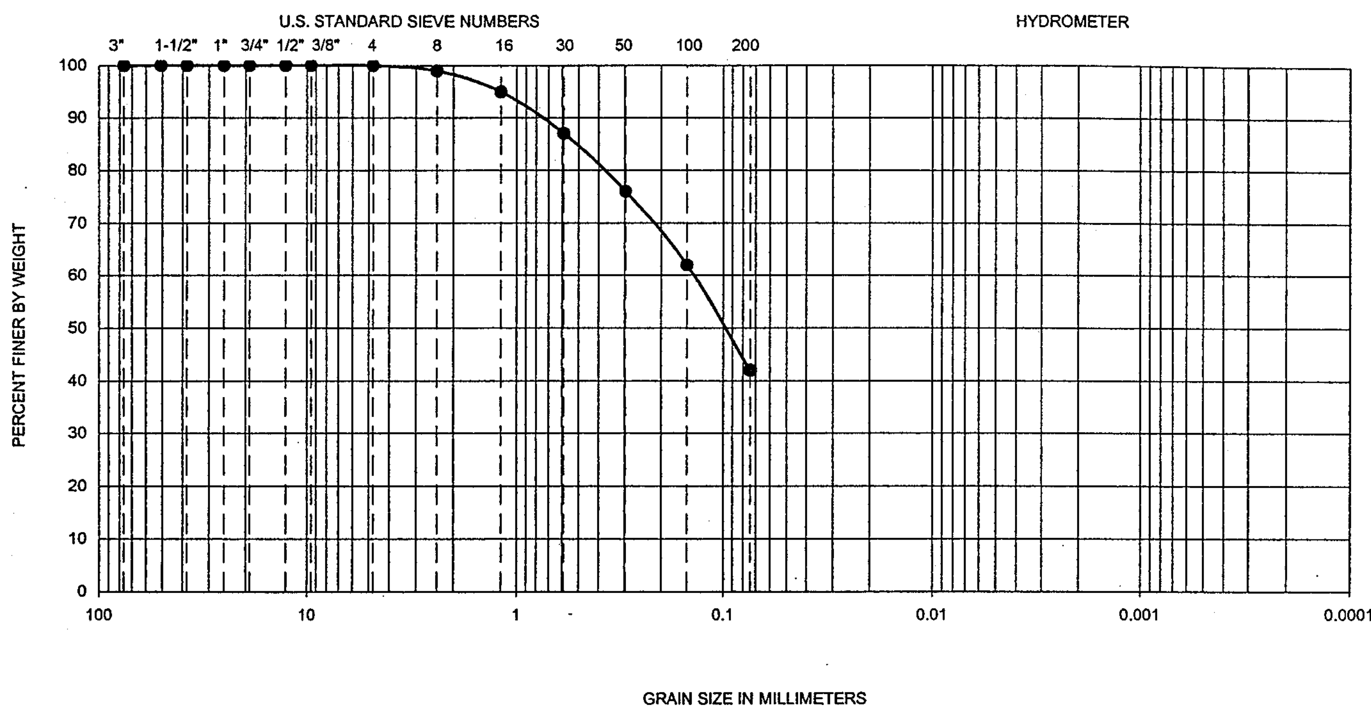
## GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-30 |

SIEVE RH-16@2.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-16 | 7.5-8.8 | -- | -- | NP | -- | -- | -- | -- | -- | 42 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

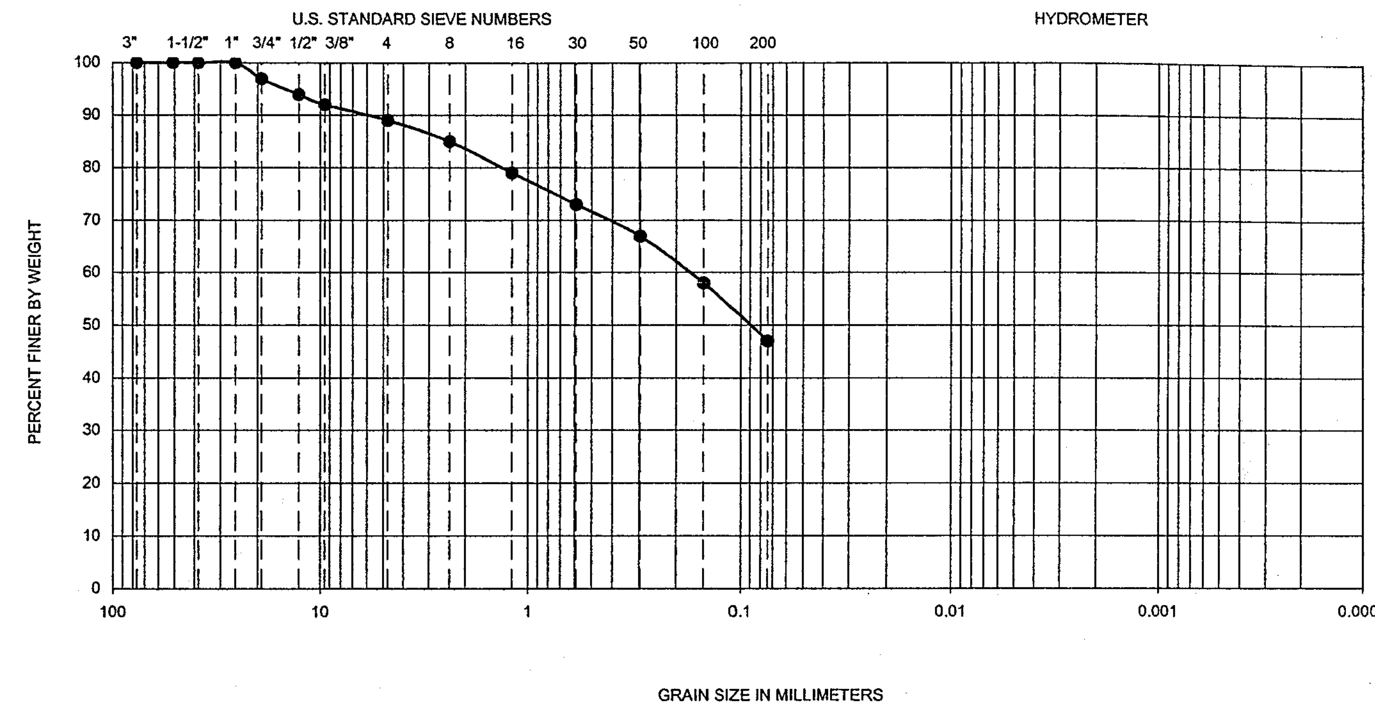
**GRADATION TEST RESULTS**

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-31 |

SIEVE RH-16@7.xls

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-17 | 2.5-4 | 22 | 15 | 7 | -- | -- | -- | -- | -- | 47 | SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

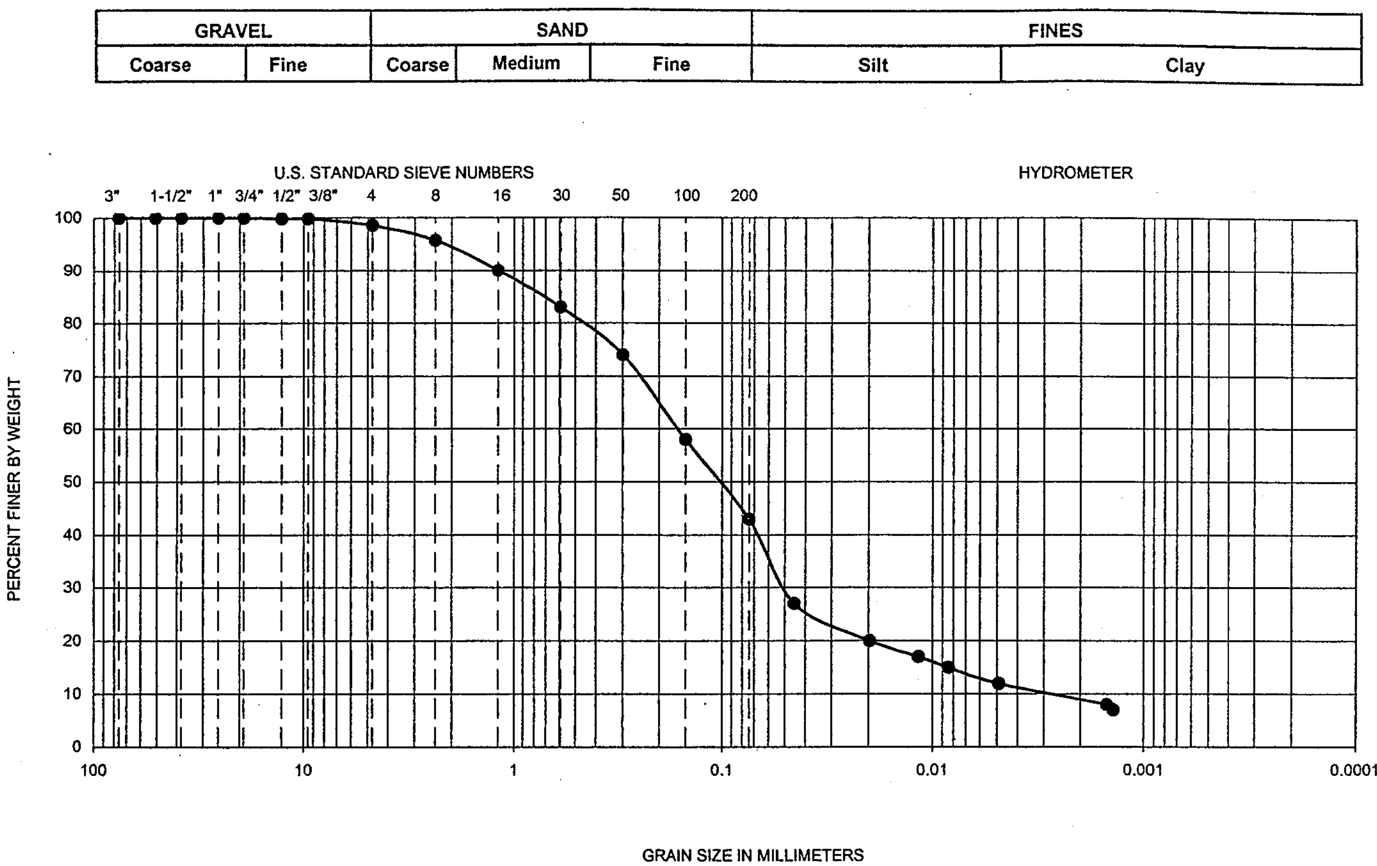
Ninyo & Moore

**GRADATION TEST RESULTS**

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-32 |

SIEVE RH-17@2.xls




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | U.S.C.S |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | RH-12 | 10.0-11.5 | -- | -- | NP | 0.004 | 0.05 | 0.17 | 41.3 | 4.1 | 43 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

**Ninyo & Moore**

## GRADATION TEST RESULTS

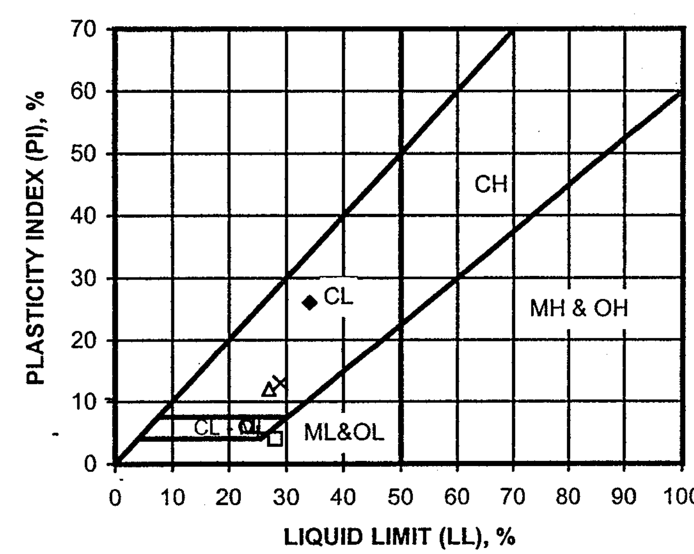
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-33 |

RH-17 10.0 TO 11.0 SIEVE+HYDRO1.xls

| SYMBOL | LOCATION | DEPTH (FT) | LL (%) | PL (%) | PI (%) | U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction) | U.S.C.S. (Entire Sample) |
|---|---|---|---|---|---|---|---|
| ● | RH-1 | 10-11.5 | - | - | NP | SC | SC |
| ■ | RH-1 | 25-26.5 | - | - | NP | ML | SM |
| ◆ | RH-2 | 2.5-4 | 34 | 8 | 26 | CL | CL |
| ○ | RH-2 | 12.5-14 | 23 | 17 | 6 | CL-ML | SC-SM |
| □ | RH-3 | 5-6.5 | 28 | 24 | 4 | ML | SM |
| Δ | RH-4 | 5-6.5 | 27 | 15 | 12 | CL | CL |
| X | RH-4 | 15-16.5 | 29 | 16 | 13 | CL | SC |
| + | RH-5 | 5-6.5 | 25 | 20 | 5 | ML-CL | CL |

NP - Indicates non-plastic




PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

**Ninyo & Moore**

## ATTERBERG LIMITS TEST RESULTS

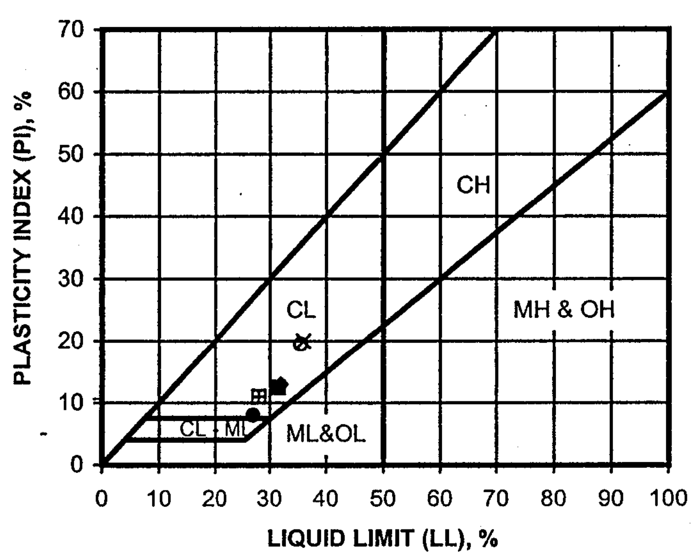
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-34 |

RH ATTERBERG 1.xls

| SYMBOL | LOCATION | DEPTH (FT) | LL (%) | PL (%) | PI (%) | U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction) | U.S.C.S. (Entire Sample) |
|---|---|---|---|---|---|---|---|
| ● | RH-5 | 20-21.5 | 27 | 19 | 8 | CL | SC |
| ■ | RH-6 | 10-11.5 | 32 | 19 | 13 | CL | CL |
| ◆ | RH-6 | 15-16.5 | 32 | 19 | 13 | CL | CL |
| ○ | RH-7 | 2.5-4 | 36 | 16 | 20 | CL | CL |
| □ | RH-7 | 17.5-19 | 28 | 17 | 11 | CL | CL |
| Δ | RH-8 | 7.5-9 | 32 | 21 | NP | CL | CL |
| X | RH-8 | 17.5-19 | 36 | 16 | 20 | CL | CL |
| + | RH-9 | 5-6.5 | 28 | 17 | 11 | CL | CL |

NP - Indicates non-plastic




PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

Ninyo & Moore

## ATTERBERG LIMITS TEST RESULTS

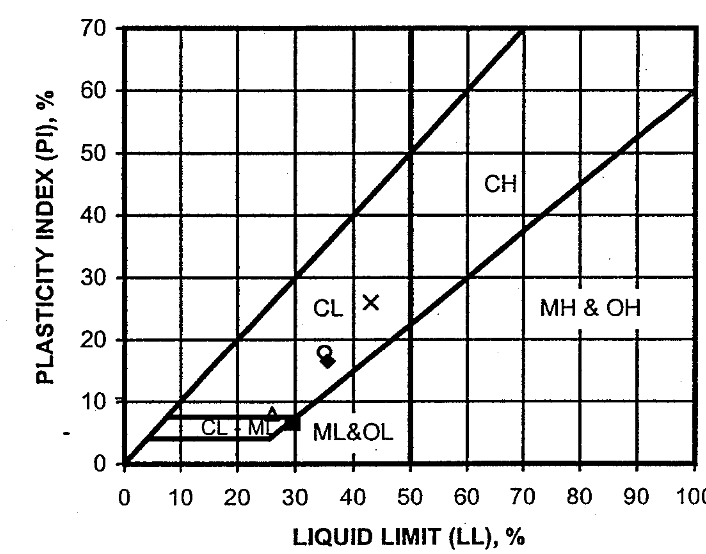
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-35 |

RH ATTERBERG 2.xls

| SYMBOL | LOCATION | DEPTH (FT) | LL (%) | PL (%) | PI (%) | U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction) | U.S.C.S. (Entire Sample) |
|---|---|---|---|---|---|---|---|
| ● | RH-9 | 20-21.5 | - | - | NP | SC | SC |
| ■ | RH-10 | 12.5-14 | 30 | 23 | 7 | ML | ML |
| ◆ | RH-11 | 10-11.5 | 36 | 19 | 17 | CL | CL |
| ○ | RH-11 | 17.5-19 | 35 | 17 | 18 | CL | SC |
| □ | RH-12 | 5-6.5 | - | - | NP | ML | ML |
| Δ | RH-12 | 15-16.5 | 26 | 18 | 8 | CL | CL |
| X | RH-13 | 5-6.5 | 43 | 17 | 26 | CL | CL |
| + | RH-13 | 20-21.5 | - | - | NP | ML | ML |

NP - Indicates non-plastic




PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

Ninyo & Moore

## ATTERBERG LIMITS TEST RESULTS

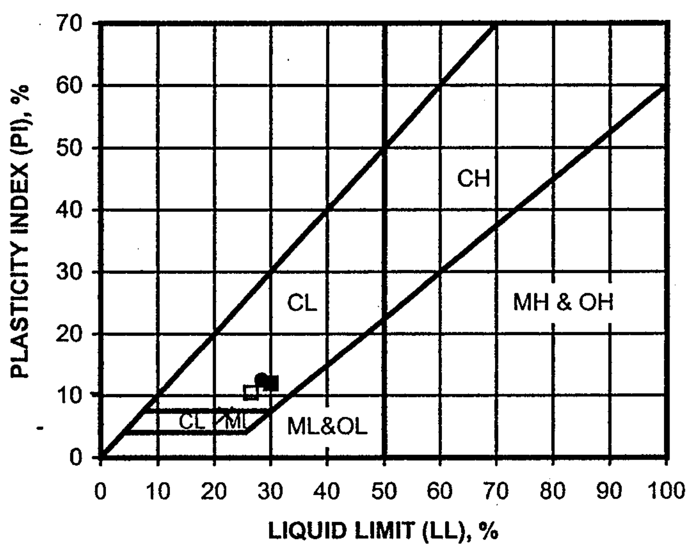
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-36 |

RH ATTERBERG 3.xls

| SYMBOL | LOCATION | DEPTH (FT) | LL (%) | PL (%) | PI (%) | U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction) | U.S.C.S. (Entire Sample) |
|---|---|---|---|---|---|---|---|
| ● | RH-14 | 2.5-3.5 | 29 | 16 | 13 | CL | CL |
| ■ | RH-14 | 15-15.8 | 30 | 18 | 12 | CL | SC |
| ◆ | RH-15 | 5-5.9 | - | - | NP | ML | ML |
| ○ | RH-15 | 15-16.5 | - | - | NP | SC | SC |
| □ | RH-16 | 2.5-4 | 27 | 16 | 11 | CL | CL |
| Δ | RH-16 | 7.5-8.8 | - | - | NP | SM | SM |
| X | RH-17 | 2.5-4 | 22 | 15 | 7 | CL-ML | SC |
| + | RH-17 | 10-11 | - | - | NP | ML | SM |

NP - Indicates non-plastic




PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

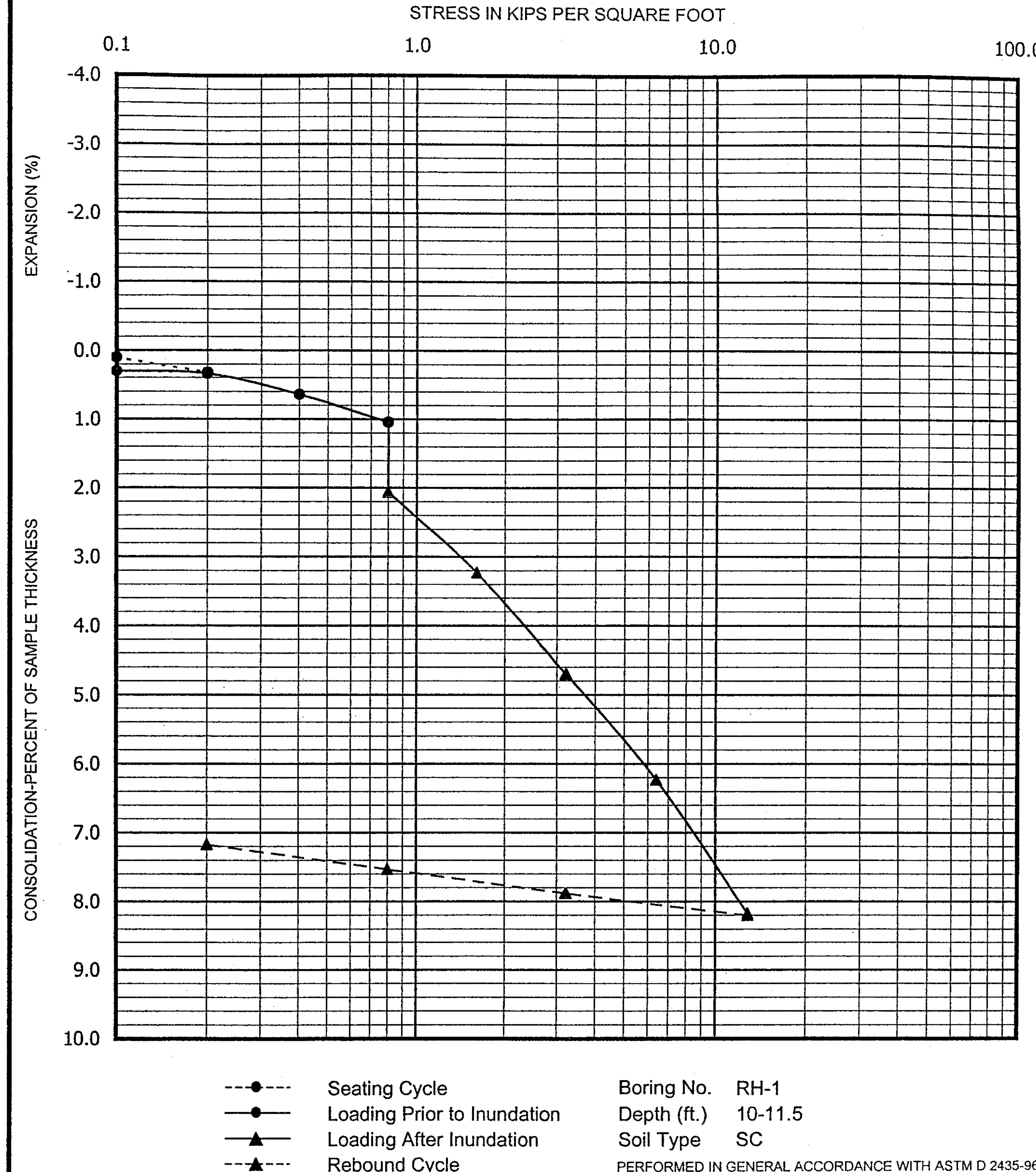
Ninyo & Moore

## ATTERBERG LIMITS TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-37 |

RH ATTERBERG 4.xls




Boring No. RH-1
Depth (ft.) 10-11.5
Soil Type SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

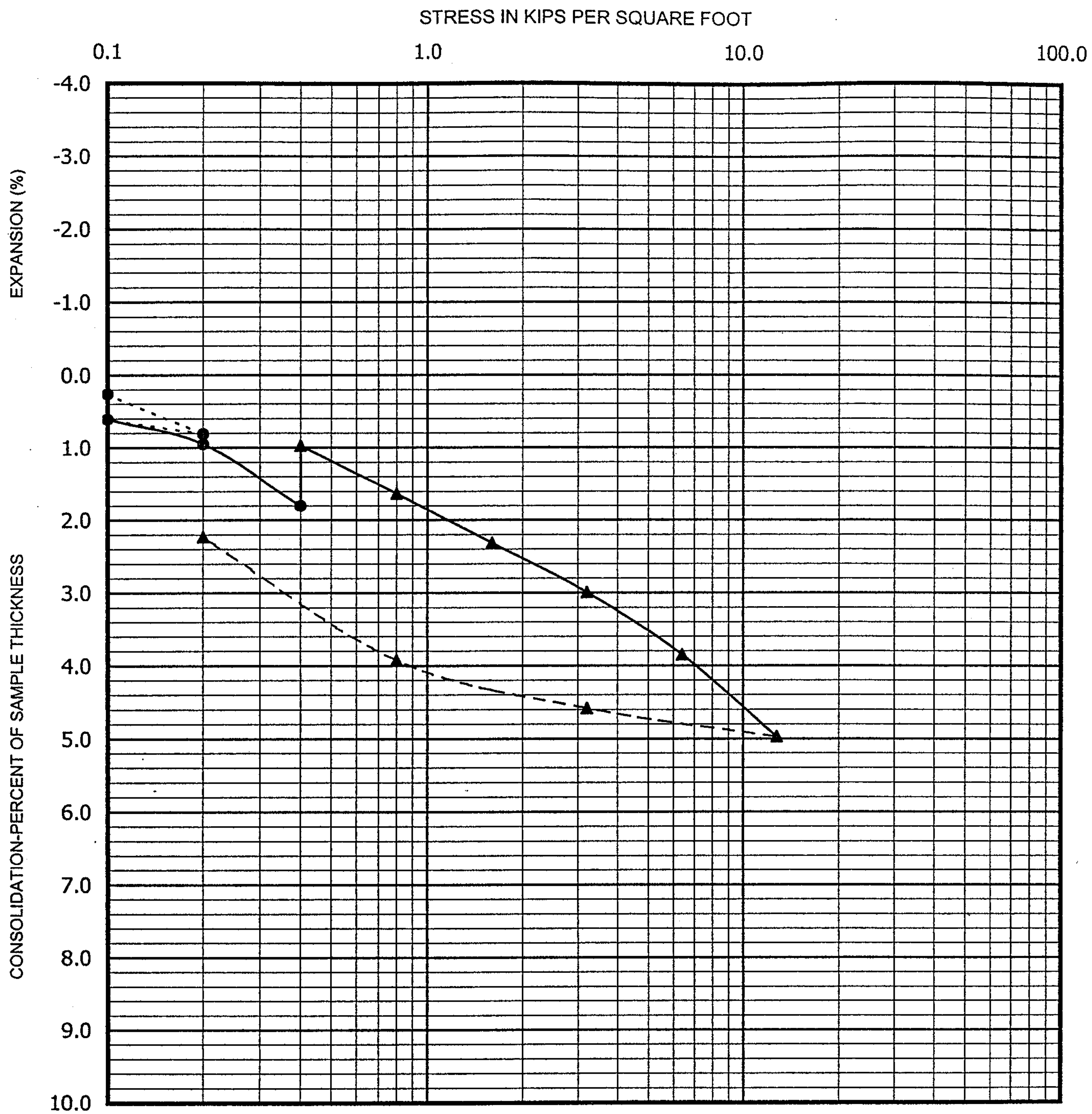
Ninyo & Moore

## CONSOLIDATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-38 |

CONSOLIDATION RH-1 10-11.xls




| | | | |
|---|---|---|---|
| Seating Cycle | | Boring No. | RH-3 |
| Loading Prior to Inundation | | Depth (ft.) | 2.5-4 |
| Loading After Inundation | | Soil Type | SM |
| Rebound Cycle | | | |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

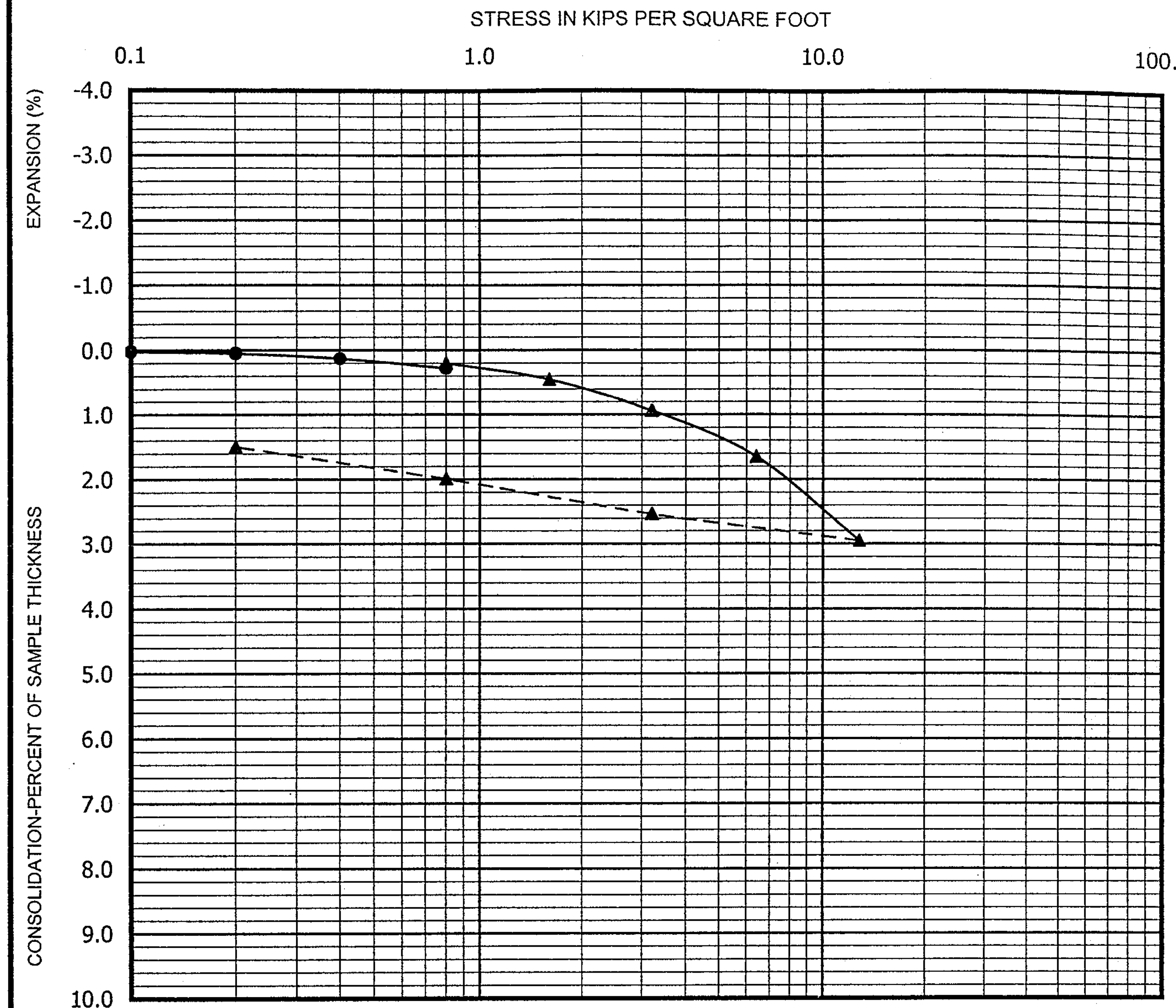
Ninyo & Moore

## CONSOLIDATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-39 |

CONSOLIDATION RH-3 2.5-4.xls




| | | | |
|---|---|---|---|
| Seating Cycle | | Boring No. | RH-4 |
| Loading Prior to Inundation | | Depth (ft.) | 5-6.5 |
| Loading After Inundation | | Soil Type | CL |
| Rebound Cycle | | | |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

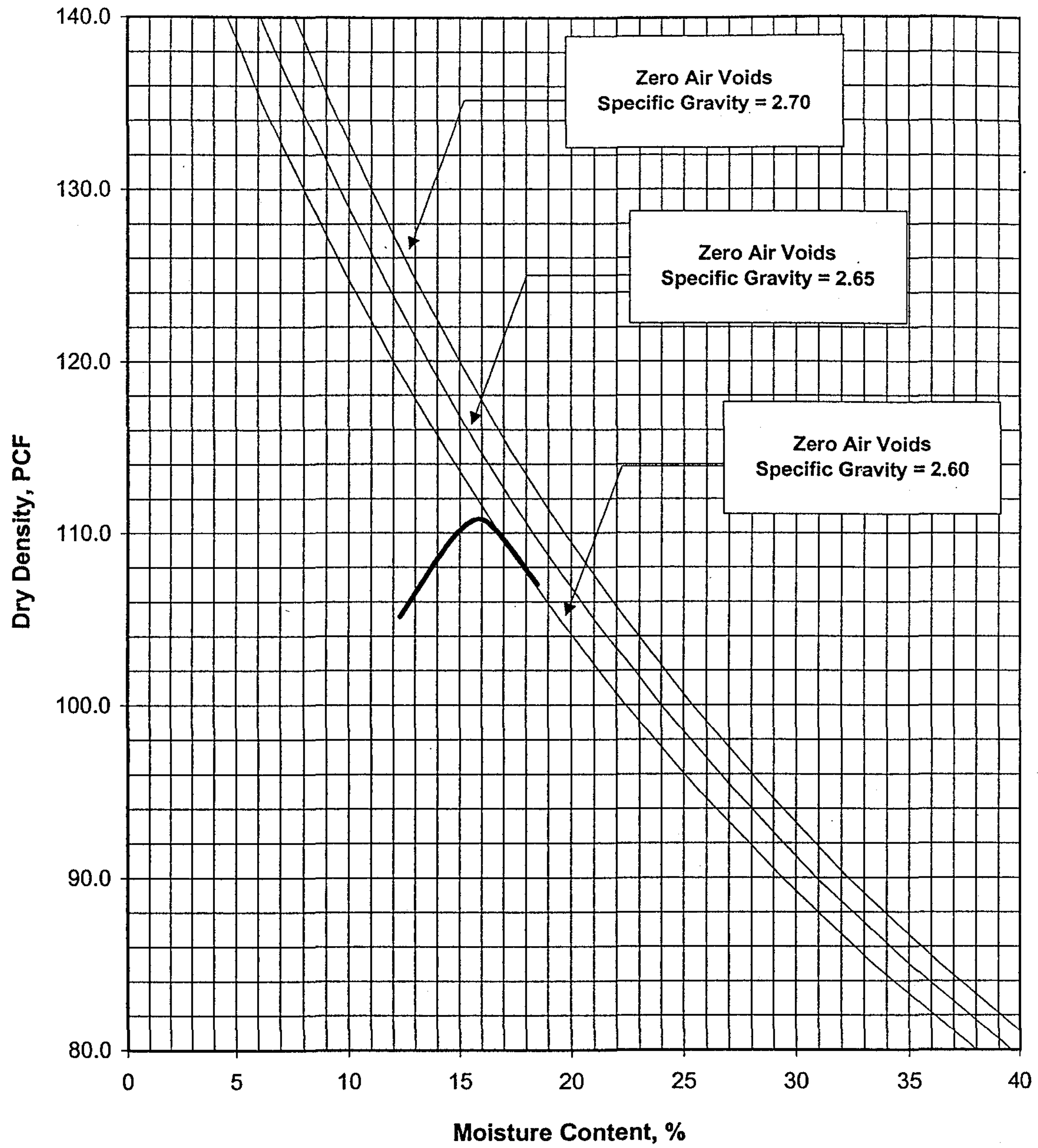
Ninyo & Moore

## CONSOLIDATION TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-40 |

CONSOLIDATION RH-4 5-6.5.xls




| SAMPLE LOCATION | DEPTH (FT) | SOIL DESCRIPTION | MAXIMUM DENSITY (PCF) | OPTIMUM MOISTURE CONTENT (%) |
|---|---|---|---|---|
| RH-6 | 0-2 | Silty CLAY | 110.8 | 15.8 |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1557-91

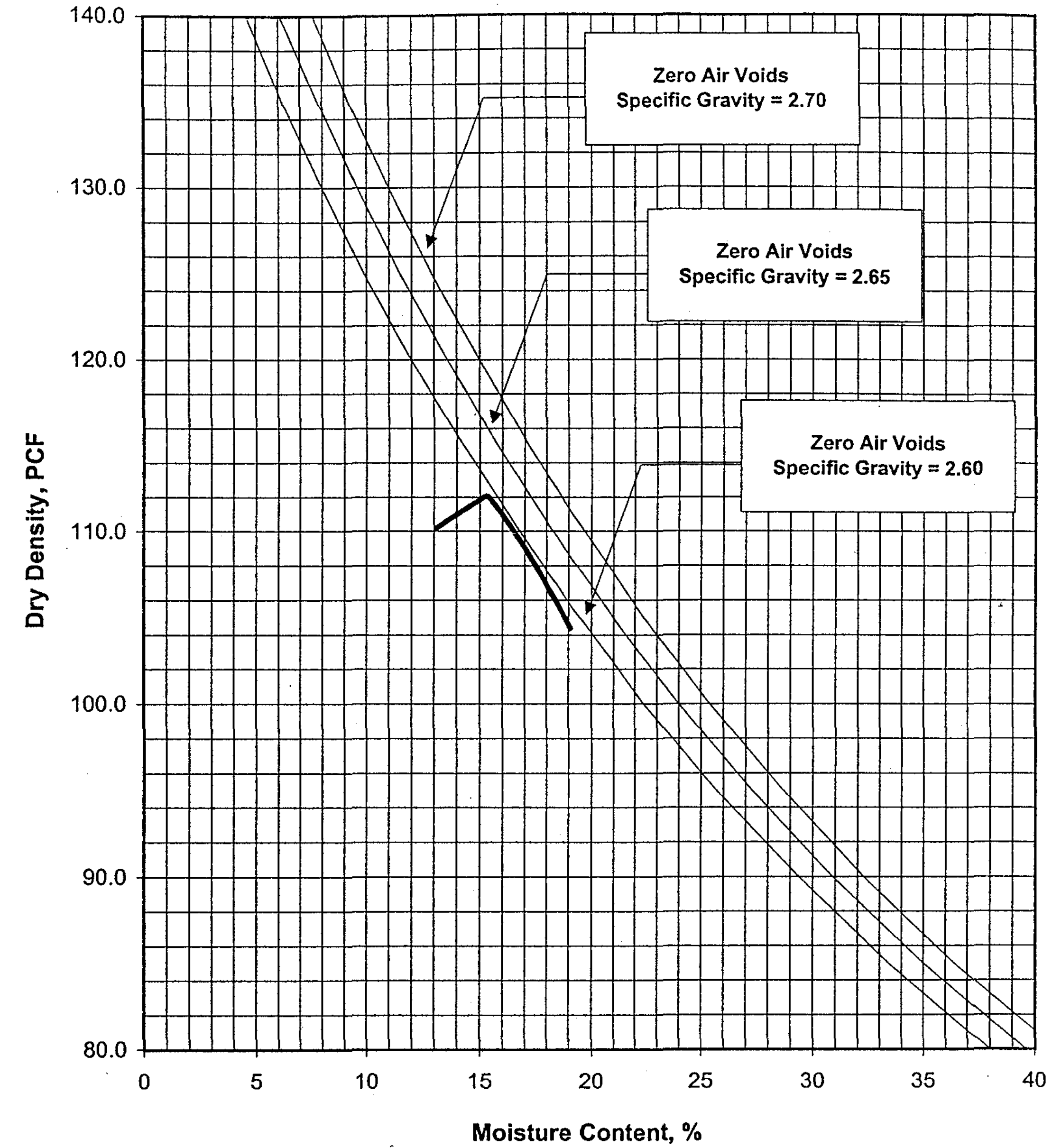
Ninyo & Moore

MAXDENSITY RH-6@0.xls

## MAXIMUM DENSITY TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-41 |




| SAMPLE LOCATION | DEPTH (FT) | SOIL DESCRIPTION | MAXIMUM DENSITY (PCF) | OPTIMUM MOISTURE CONTENT (%) |
|---|---|---|---|---|
| RH-12 | 12-15 | Silty SAND | 112.0 | 15.4 |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1557-91

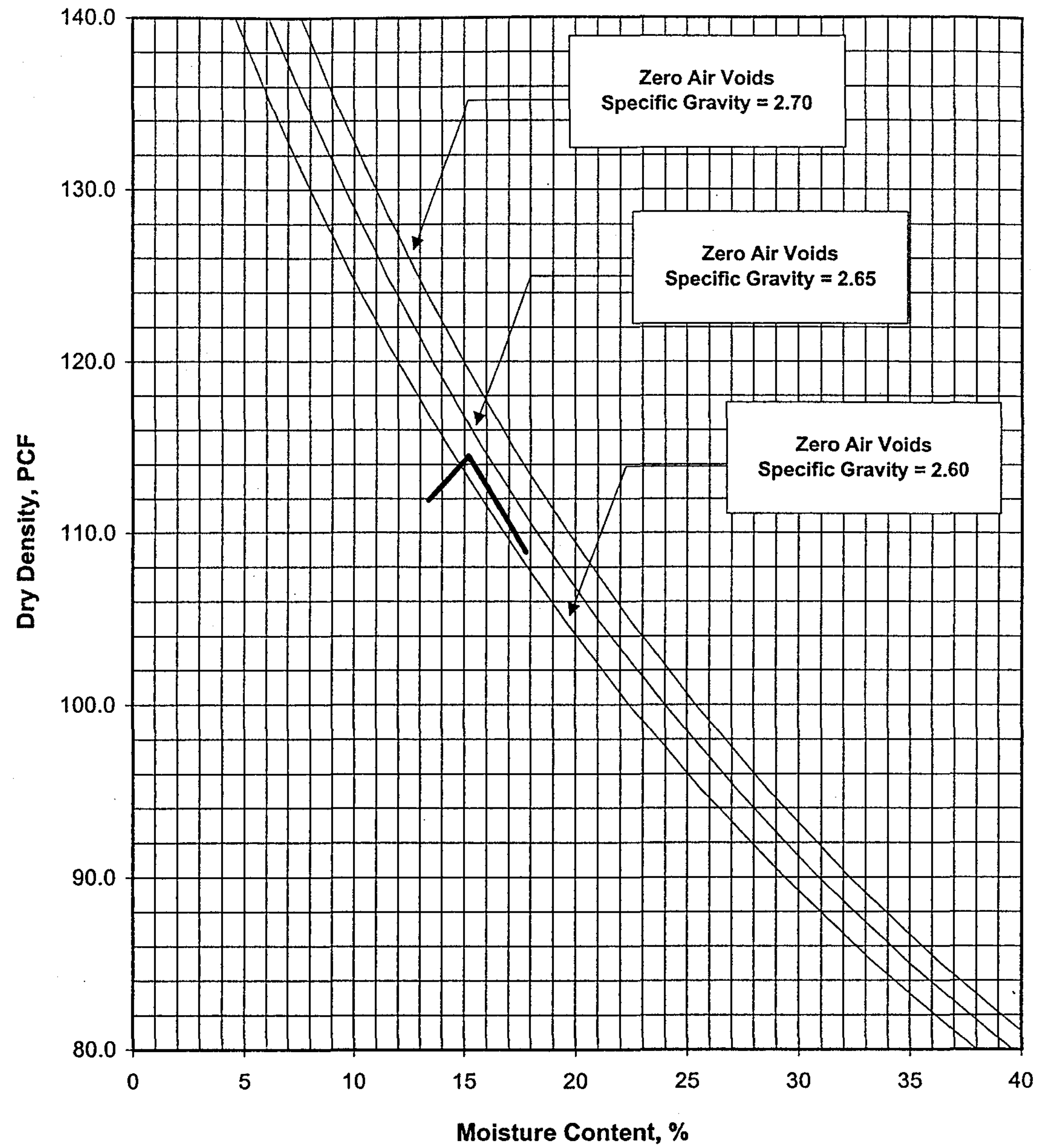
Ninyo & Moore

MAXDENSITY RH-12@12.xls

## MAXIMUM DENSITY TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-42 |




| SAMPLE LOCATION | DEPTH (FT) | SOIL DESCRIPTION | MAXIMUM DENSITY (PCF) | OPTIMUM MOISTURE CONTENT (%) |
|---|---|---|---|---|
| RH-14 | 0-5 | Silty CLAY | 114.5 | 15.2 |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1557-91

Ninyo & Moore

## MAXIMUM DENSITY TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-43 |

MAXDENSITY RH-14@0.xls

## EXPANSION INDEX TEST RESULTS

| SAMPLE LOCATION | SAMPLE DEPTH (FT) | INITIAL MOISTURE (%) | COMPACTED DRY DENSITY (PCF) | FINAL MOISTURE (%) | VOLUMETRIC SWELL (IN) | EXPANSION INDEX | EXPANSION POTENTIAL |
|---|---|---|---|---|---|---|---|
| RH-6 | 0-2 | 11.0 | 101.3 | 15.7 | 0.0175 | 18 | Very Low |
| RH-12 | 12-15 | 12.0 | 107.7 | 18.0 | 0.0003 | 0 | Very Low |
| RH-14 | 0-5 | 10.0 | 99.5 | 22.1 | 0.0063 | 6 | Very Low |
| RH-16 | 12-15 | 15.7 | 96.8 | 22.6 | 0.0074 | 7 | Very Low |

PERFORMED IN GENERAL ACCORDANCE WITH UBC STANDARD 18-2

Ninyo & Moore

## EXPANSION INDEX TEST RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-44 |

EXPANSION #240.xls

## CORROSIVITY TEST RESULTS

| SAMPLE LOCATION | SAMPLE DEPTH (FT) | pH * | RESISTIVITY * (ohm-cm) | WATER-SOLUBLE SULFATE CONTENT IN SOIL ** (%) | CHLORIDE CONTENT *** (ppm) |
|---|---|---|---|---|---|
| RH-14 | 0-5 | 7.8 | 726 | 0.002 | 55.6 |
| RH-16 | 12-15 | 8.7 | 2,046 | 0.006 | 73.0 |

* PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 236b
** PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 733
*** PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 722

Ninyo & Moore

CORROSIVITY TEST RESULTS
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-45 |

CORROSIVITY #240.xls

## PERMEABILITY TEST RESULTS

| SAMPLE LOCATION | SAMPLE DEPTH (FT) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) | DRY DENSITY (PCF) | VARIATION IN HEAD (cm) | AVERAGE PERMEABILITY (cm/sec) |
|---|---|---|---|---|---|---|
| RH-1 | 25.0-26.5 | 8.1 | 8.9 | 79.2 | 0.6 - 22.8 | $1.47 \times 10^{-3}$ |
| RH-2 | 12.5-14.0 | 8.7 | 9.5 | 86.0 | 2.7 - 12.8 | $1.02 \times 10^{-4}$ |
| RH-5 | 20.0-21.5 | 4.4 | 4.6 | 86.2 | 2.1 - 13.4 | $5.20 \times 10^{-4}$ |
| RH-17 | 10.0-11.0 | 11.1 | 12.5 | 74.7 | 2.4 - 16.8 | $6.27 \times 10^{-4}$ |

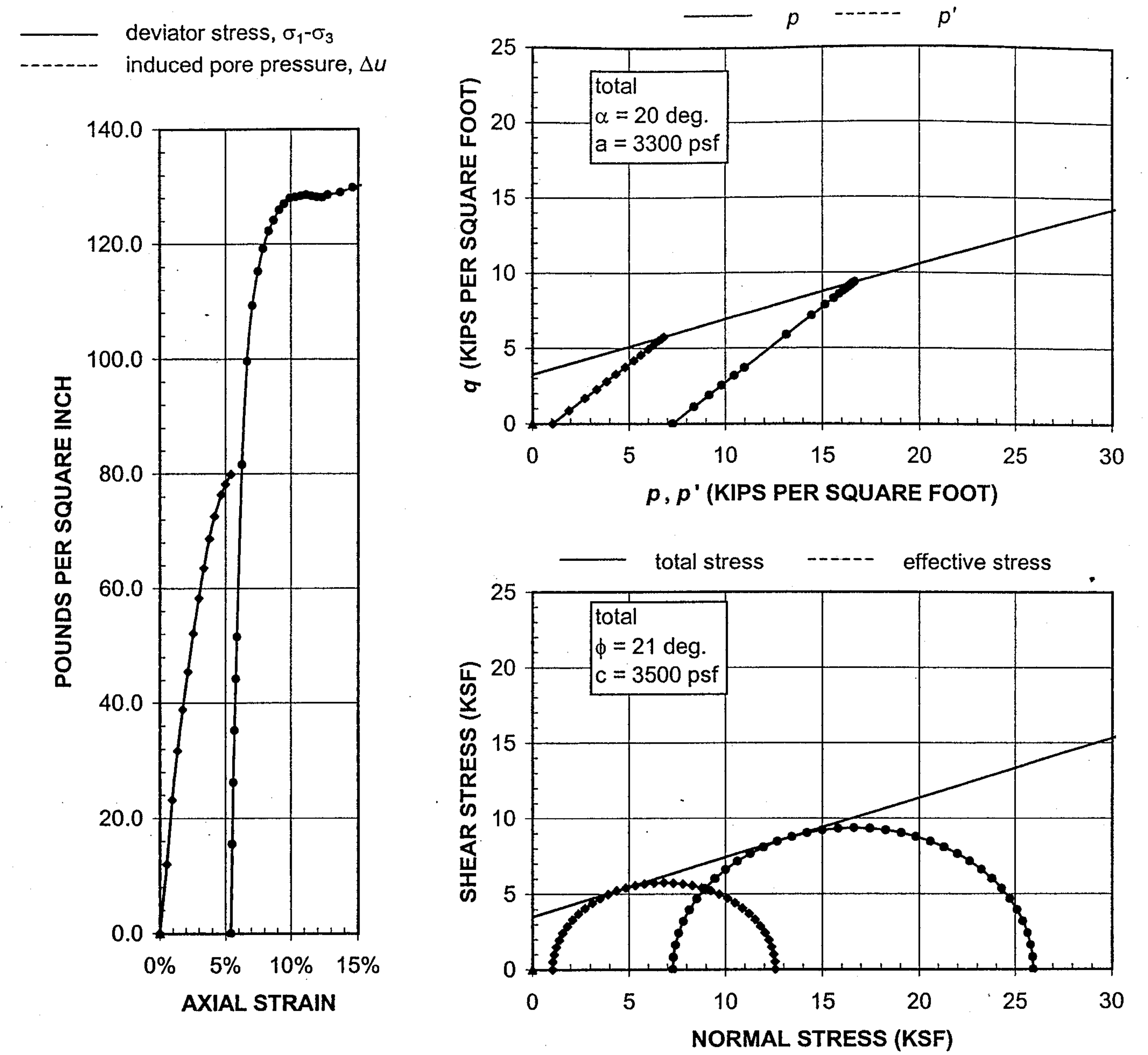
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2434-68

Ninyo & Moore

PERMEABILITY TEST RESULTS
EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-46 |

PERMEAB1.xls




| Sym. | Description | Soil Type | Sample Location | Sample Depth (ft.) | Initial Moisture (%) | Initial Dry Density (pcf) | Final Degree Saturation | Confining Stress (ksf) | Rate of Strain (%/min) |
|---|---|---|---|---|---|---|---|---|---|
| ◆ | Clayey Sand | SC | RH-11 | 17.5-19.0 | 5.6% | 111.6 | 104% | 1.05 | 1.1% |
| ● | Clayey Sand | SC | RH-11 | 17.5-19.0 | 5.6% | 111.6 | 104% | 7.27 | 0.9% |
| | | | | | | | | | |

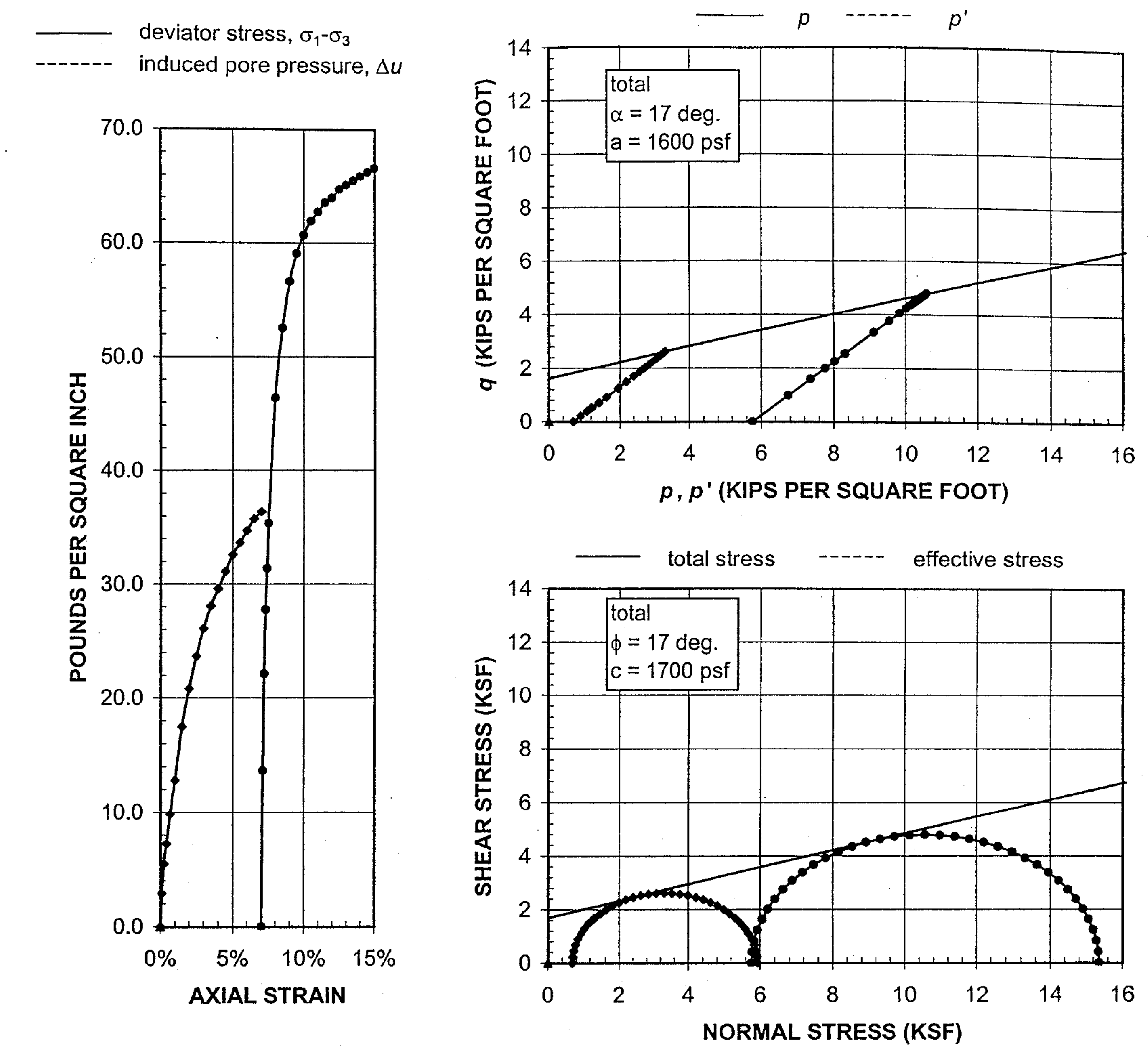
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850

Ninyo & Moore

## UU TRIAXIAL COMPRESSION RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-47 |




| Sym. | Description | Soil Type | Sample Location | Sample Depth (ft.) | Initial Moisture (%) | Initial Dry Density (pcf) | Final Degree Saturation | Confining Stress (ksf) | Rate of Strain (%/min) |
|---|---|---|---|---|---|---|---|---|---|
| ◆ | Silt | ML | RH-12 | 10.0-11.5 | 15.4% | 81.3 | 96% | 0.69 | 1.2% |
| ● | Silt | ML | RH-12 | 10.0-11.5 | 15.4% | 81.3 | 96% | 5.76 | 1.0% |
| | | | | | | | | | |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850

Ninyo & Moore

## UU TRIAXIAL COMPRESSION RESULTS

EAST MARICOPA FLOODWAY
RITTENHOUSE DETENTION BASIN
MARICOPA COUNTY, ARIZONA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 600198001 | 12/01 | B-48 |

APPENDIX C

PERCOLATION TEST RESULTS

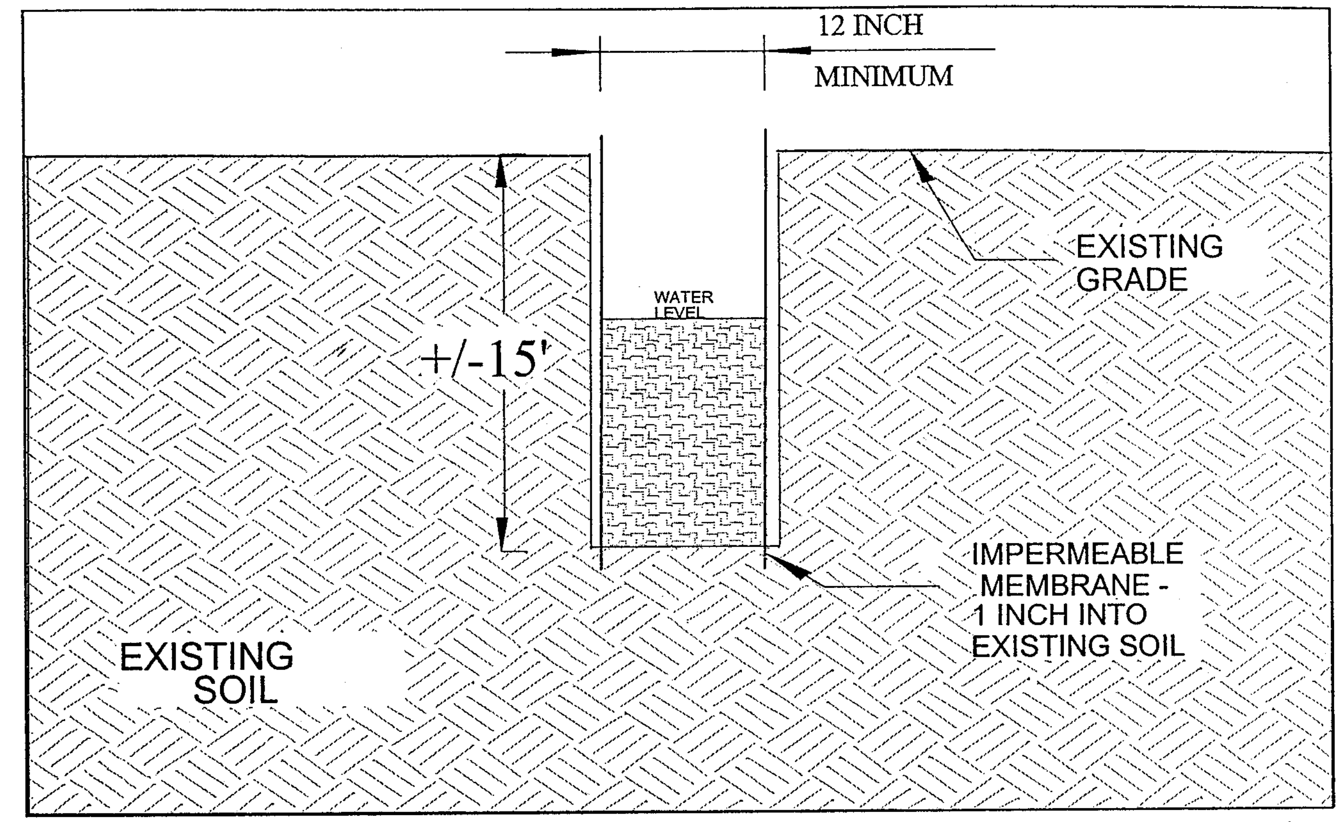


Ninyo & Moore

# SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Rittenhouse Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-1 (Near RH-14)




| START TIME (Hr:Min) | ENDING TIME (Hr:Min) | ELAPSED TIME (Hr:Min) | INITIAL READING (Feet) | FINAL READING (Feet) | CHANGE IN WATER LEVEL (Feet) | PERCOLATION RATE* |
|---|---|---|---|---|---|---|
| 11:00 | 11:28 | 0:28 | 0.35 | 0.36 | 0.01 | 0.02 |
| 11:28 | 11:47 | 0:19 | 0.36 | 0.40 | 0.04 | 0.13 |
| 11:47 | 12:11 | 0:24 | 0.40 | 0.44 | 0.04 | 0.10 |
| 12:11 | 12:30 | 0:19 | 0.44 | 0.46 | 0.02 | 0.06 |
| 12:30 | 12:50 | 0:20 | 0.46 | 0.49 | 0.03 | 0.09 |

* Note: Percolation Rate is reported in Cubic Feet per Hour per Square Foot of percolation area.

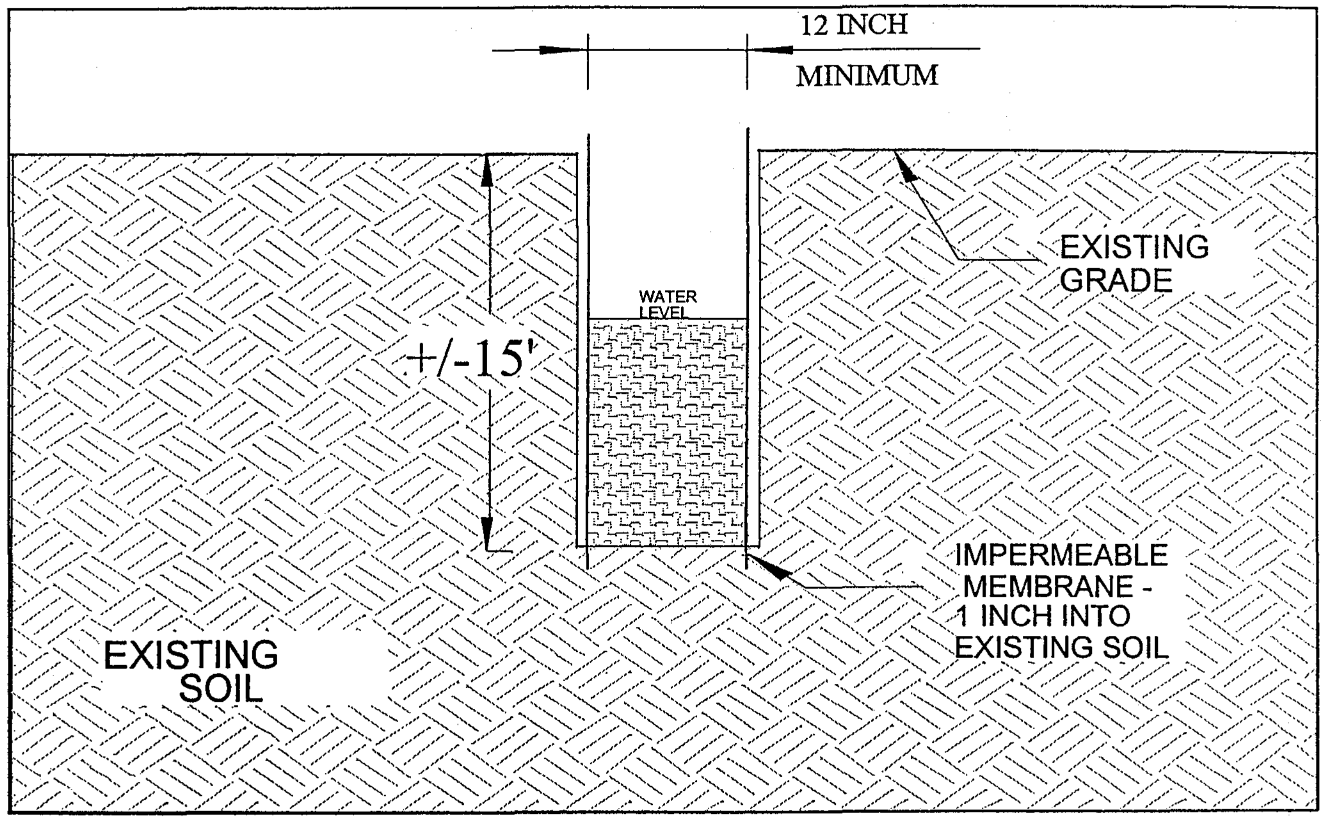
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **0.08** $FT^3/HOUR/FT^2$

Ninyo & Moore

# SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Rittenhouse Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-2 (Near RH-15)




| START TIME (Hr:Min) | ENDING TIME (Hr:Min) | ELAPSED TIME (Hr:Min) | INITIAL READING (Feet) | FINAL READING (Feet) | CHANGE IN WATER LEVEL (Feet) | PERCOLATION RATE* |
|---|---|---|---|---|---|---|
| 10:48 | 11:24 | 0:36 | 0.90 | 4.40 | 3.50 | 5.83 |
| 11:24 | 11:43 | 0:19 | 4.40 | 5.40 | 1.00 | 3.16 |
| 11:43 | 12:00 | 0:17 | 5.40 | 6.11 | 0.71 | 2.51 |
| 12:00 | 12:25 | 0:25 | 6.11 | 6.99 | 0.88 | 2.11 |
| 12:25 | 12:45 | 0:20 | 6.99 | 7.54 | 0.55 | 1.65 |

* Note: Percolation Rate is reported in Cubic Feet per Hour per Square Foot of percolation area.

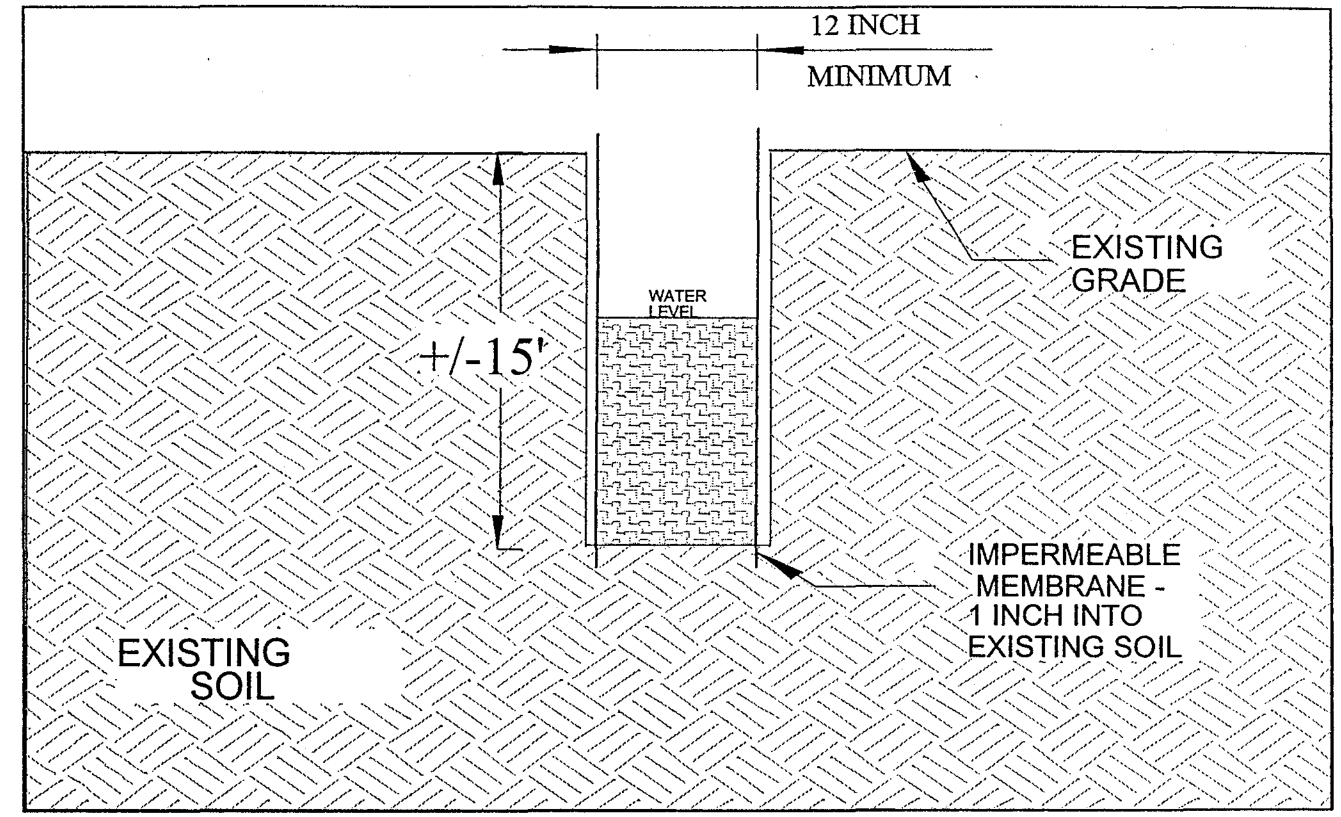
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **2.09** $FT^3/HOUR/FT^2$

Ninyo & Moore

# SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Rittenhouse Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-3 (Near RH-16)




| START TIME (Hr:Min) | ENDING TIME (Hr:Min) | ELAPSED TIME (Hr:Min) | INITIAL READING (Feet) | FINAL READING (Feet) | CHANGE IN WATER LEVEL (Feet) | PERCOLATION RATE* |
|---|---|---|---|---|---|---|
| 10:36 | 11:17 | 0:41 | 0.40 | 1.20 | 0.80 | 1.17 |
| 11:17 | 11:36 | 0:19 | 1.20 | 1.52 | 0.32 | 1.01 |
| 11:36 | 11:54 | 0:18 | 1.52 | 1.81 | 0.29 | 0.97 |
| 11:54 | 12:19 | 0:25 | 1.81 | 2.20 | 0.39 | 0.94 |
| 12:19 | 12:39 | 0:20 | 2.20 | 2.45 | 0.25 | 0.75 |

* Note: Percolation Rate is reported in Cubic Feet per Hour per Square Foot of percolation area.

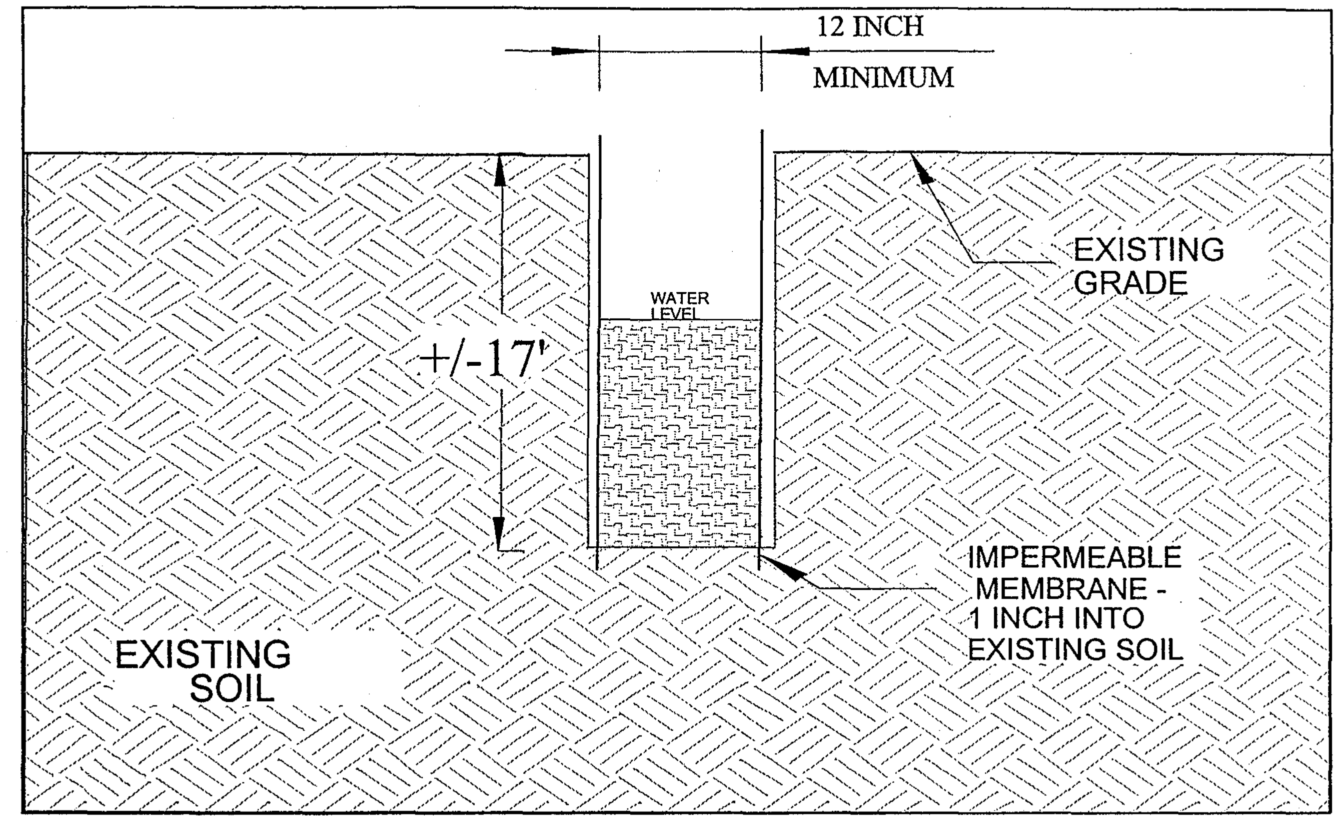
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **0.88** $FT^3/HOUR/FT^2$

Ninyo & Moore

# SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Rittenhouse Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-4 (Near RH-17)




| START TIME (Hr:Min) | ENDING TIME (Hr:Min) | ELAPSED TIME (Hr:Min) | INITIAL READING (Feet) | FINAL READING (Feet) | CHANGE IN WATER LEVEL (Feet) | PERCOLATION RATE* |
|---|---|---|---|---|---|---|
| 10:27 | 11:12 | 0:45 | 3.10 | 4.40 | 1.30 | 1.73 |
| 11:12 | 11:39 | 0:27 | 4.40 | 4.85 | 0.45 | 1.00 |
| 11:39 | 11:51 | 0:12 | 4.85 | 5.22 | 0.37 | 1.85 |
| 11:51 | 12:15 | 0:24 | 5.22 | 5.78 | 0.56 | 1.40 |
| 12:15 | 12:35 | 0:20 | 5.78 | 6.01 | 0.23 | 0.69 |

* Note: Percolation Rate is reported in Cubic Feet per Hour per Square Foot of percolation area.

AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **1.31** $FT^3/HOUR/FT^2$

## APPENDIX D

## AGRONOMIC TESTS RESULTS





August 21, 2001

Lab #: SP 107342-01

**Ninyo & Moore**
5035 South 33rd St.
Phoenix, AZ 85040

### Recommendations for Rittenhouse Basin

The following report presents the results of analyses conducted on your soil. See page 4 for sample information and analyses results. The following recommendations are based upon the current conditions of the soil. All application recommendations are for each 1,000 square feet of growing area. Please be sure to read the standard application notes presented on page 3.

### I. Plant Selection

The analyses of this soil indicates the following plant selection requirements:

A. Select only non-acidic loving plants for this soil.
B. Select only those plants that have a slight or greater tolerance to free limestone for planting at this site.

### II. Preplant Soil Amendments and Fertilizers

#### A. Turf and Groundcover

| | Apply per 1000 sq. ft. |
|---|---|
| 1. Soil amendments | |
| a. Organic (well-composted) | 2.00 cu. yds. |
| b. Limestone | 0.00 lbs. |
| c. Soil Sulfur | 25.0 lbs. |

| | Apply per 1000 sq. ft. |
|---|---|
| 2. Fertilizers | |
| a. Nitrogen (N) | 1.00 lbs. |
| b. Phosphorus ($P_2O_5$) | 4.10 lbs. |
| c. Potassium ($K_2O$) | 3.40 lbs. |
| d. Magnesium (Mg) | 0.00 lbs. |
| e. Zinc (Zn) | 1.30 lbs. |
| f. Manganese (Mn) | 0.00 lbs. |
| g. Iron (Fe) | 0.55 lbs. |
| h. Copper (Cu) | .025 lbs. |
| i. Boron (B) | .009 lbs. |



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TEL: 805/659-0910

Office & Laboratory
2500 Stagecoach Road
Stockton, CA 95215
TEL: 209/942-0181

Field Office
Visalia, CA
TEL: 559/734-9473
FAX: 559/734-8435

B. **Tree and Shrub Backfill Mix**

| | | |
|---|---|---|
| 1. | Native (site) soil | 66% |
| 2. | Nitrogen Fertilized Organic Material | 33% |
| 3. | Commercial Fertilizer (8-8-4) | 1 lb./cu. yd. |
| 4. | Iron | 2 oz./cu. yd. |
| 5. | Zinc | 1 oz./cu. yd. |
| 6. | Manganese | 1 oz./cu. yd. |

When planting specifications do not call for a separate backfill mix then backfill the holes that are excavated to install containerized plants using the native (site) soil amended according to the preplant recommendations given on page 1.

## III. Leaching Requirement

No Leaching Requirement for this soil.

## IV. Post-Plant Fertilization - lbs./1000 sq. ft.

| | |
|---|---|
| Nitrogen | 1/2 lb. |
| Phosphorus | 1/2 lb. |
| Potassium | 1/2 lb. |

The actual post-plant requirements for fertilizers and soil amendments will vary depending upon the specific site conditions. Periodic post-plant analyses can be used to assure proper soil conditions and balanced levels of plant nutrition.

## V. Irrigation

Make certain that the irrigation water being applied is penetrating to a depth slightly greater than the root zone of the plants being grown. Water with a frequency needed to maintain moist soil at all times - never wet for long periods and never let the soil dry out.



# Application Notes

The application instructions listed below apply only if the material(s) is recommended in this report on page 1. Materials not included in the recommendations are excluded either because the analyses data did not indicate a need or the analysis to determine if a need existed was not requested.

### Organic Materials

Nitrolized redwood compost is preferred but other organic mixes may be substituted depending upon the site requirements. Organic materials should be spread uniformly over the surface soils and when possible should be incorporated to a depth of two to three inches.

### Limestone, Dolomite & Sulfur

These materials should be broadcast uniformly over the surface soils and then incorporated to a depth of two to three inches.

### Gypsum

This material should be broadcast uniformly over surface soils for water penetration. For best results do not incorporate.

### Preplant Phosphorous, Zinc, Manganese, Iron & Copper

These materials should be broadcast uniformly over the surface soils and then incorporated to a depth of two to three inches. Post-plant applications can be surface applied for water penetration.

### Nitrogen, Potassium & Magnesium

These materials are highly water soluble and can be applied uniformly over the surface soils for water penetration or they can be incorporated with the other materials. Magnesium sources for plant nutrition include Epsom salts (Magnesium Sulfate), and the double salt of Potasium-Magnesium Sulfate (Sulfate of Potash-magnesia).

FRUIT GROWERS LABORATORY, INC.

ANALYTICAL CHEMISTS

August 21, 2001

**Ninyo & Moore**
5035 South 33rd St.
Phoenix, AZ 85040

Description : RH-8
Project : Rittenhouse Basin

Lab ID : SP 107342-01
Customer ID: 2-18569

Sampled On : July 11, 2001
Sampled By : Ninyo & Moore
Received On: August 15, 2001
Depth : 12-15'
Meth. Irrg. : S.S. Sprinklers

## LANDSCAPE SOIL ANALYSIS

| Test Description | Result | Optimum Range | Graphical Results Presentation |
|---|---|---|---|
| | | | Very Low / Moderately Low / Optimum / Moderately High / Very High |
| **Primary Nutrients** | | | |
| Nitrate-Nitrogen | 5.8 PPM | 10 - 70 | |
| Phosphorus | 2 PPM | 12 - 60 | |
| Potassium (Exch) | 300 PPM | 81 - 500 | |
| Potassium (Sol) | 0.17 meq/L | 0.25 - 1.0 | |
| **Secondary Nutrients** | | | |
| Calcium (Exch) | 3800 PPM | --- | |
| Calcium (Sol) | 1.2 meq/L | 2.0 - 50 | |
| Magnesium (Exch) | 780 PPM | --- | |
| Magnesium (Sol) | 1.0 meq/L | 1.5 - 60 | |
| Sodium (Exch) | 200 PPM | --- | |
| Sodium (Sol) | 4.7 meq/L | See SAR | |
| Sulfate | 2.1 meq/L | 0.6 - 20 | |
| **Micro Nutrients** | | | |
| Zinc | 0.2 PPM | 0.7 - 50 | |
| Manganese | 4.1 PPM | 1.4 - 50 | |
| Iron | 9.7 PPM | 8.0 - 100 | |
| Copper | 0.8 PPM | 0.2 - 15 | |
| Boron | 0.23 PPM | 0.3 - 2.1 | |
| Chloride | 1.42 meq/L | 0.1 - 4.0 | |
| CEC | 26.8 meq/100g | Variable | |
| **% Base Saturation** | | | |
| CEC - Calcium | 70.1 % | 60 - 80 | |
| CEC - Magnesium | 23.9 % | 10 - 20 | |
| CEC - Potassium | 2.8 % | 2 - 5 | |
| CEC - Sodium | 3.2 % | 0 - 5 | |
| CEC - Hydrogen | 0.0 % | 0 - 3 | |
| | | | Strongly Acidic / Moderately Acidic / Near Neutral / Moderately Alkaline / Strongly Alkaline |
| pH | 8.2 --- | 5.8 - 8.2 | |

Good — Problem

Table continued next page...

**Corporate Offices & Laboratory**
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**Field Office**
Visalia, CA
TEL: 559/734-9473
FAX: 559/734-8435
Mobile: 559/737-2399

August 21, 2001

**Ninyo & Moore**

Lab ID : SP 107342-01
Customer ID: 2-18569
Description : RH-8

## LANDSCAPE SOIL ANALYSIS

| Test Description | Result | Optimum Range | Graphical Results Presentation |
|---|---|---|---|
| | | | Satisfactory / Possible Problem / Moderate Problem / Increasing Problem |
| **Others** | | | |
| Soil Salinity | 0.75 mmhos/cm | 0.5 - 2.0 | |
| SAR | 4.5 | 0.1 - 6 | |
| Limestone | 3.0 % | 0 - 0.1 | |
| | | | 0 / 1 / 2 / 3 / 4 / 5 / 6 |
| Lime Requirement | 0.0 Tons/AF | --- | |
| | | | Very Low / Moderately Low / Optimum / Moderately High / Very High |
| Moisture | 11.2 % | 1/2 Satn. % | |
| | | | Loamy Sand / Sandy Loam / Loam / Silt Loam / Clay Loam / Clay / Organic |
| Saturation | 38.8 % | 20 - 60 | |

Good — Problem

Soil pH & Limestone levels are important to consider when making plant selections. Soil pH levels above 7.0 are not suitable for acid loving plants. Soils containing limestone are not suitable for plants sensitive to Limestone.

FRUIT GROWERS LABORATORY, INC.

Darrell H. Nelson, President

DHN:md

# SUBSIDENCE AND FISSURES EVALUATION

GEOLOGICAL CONSULTANTS, INC
2333 WEST NORTHERN AVENUE, SUITE 1A
PHOENIX AZ 85021
(602) 864-1888

GEOLOGICAL CONSULTANTS INC.

Kenneth M. Euge, R.G.


# GROUND SUBSIDENCE & EARTH FISSURES EVALUATION

## EAST MARICOPA FLOODWAY RITTENHOUSE BASIN DESIGN

## MARICOPA COUNTY, ARIZONA

Prepared for:

Mr. Barry Ling, P.E.
Kirkham Michael Consulting Engineers
9201 North 25th Avenue, Suite 195
Phoenix, Arizona 85021

Prepared by:

Geological Consultants Inc.
2333 West Northern Avenue, Suite 1A
Phoenix, Arizona 85021

REGISTERED GEOLOGIST
CERTIFICATE NO.
10338
KENNETH M.
EUGE
Date Signed 12/21/01
ARIZONA, U.S.A.

Project No. 2001-101

December 21, 2001





## NOTICE

The geologic and soils observations, findings, conclusions and recommendations presented in this report are based on (1) data from published and unpublished sources available at the time of this study, (2) photo-geological interpretation, and (3) a cursory geological field reconnaissance of the project site. The services provided by Geological Consultants to Kirkham Michael Consulting Engineers were performed according to generally accepted geological principals and standard practices used by members of the geological profession in this locale at the time of this study.

It must be recognized that subsurface geologic and soil conditions may vary from place to place and from those interpreted at locations where evaluations are made by the investigator. No warranty or representation, either expressed or implied, is or should be construed regarding geological or soil conditions at locations other than those observed by the investigator.

This report was prepared by the scope of work outlined in Geological Consultants proposal for geological services dated February 26, 2001 (revised) and as authorized by Kirkham & Michael on June 7, 2001.

TABLE OF CONTENTS



FIGURES



REGISTERED GEOLOGIST
CERTIFICATE NO.
10338
KENNETH M.
EUGE
Date Signed 12/21/01
ARIZONA, U.S.A.





# GROUND SUBSIDENCE & EARTH FISSURES EVALUATION

## EAST MARICOPA FLOODWAY RITTENHOUSE BASIN DESIGN

## MARICOPA COUNTY, ARIZONA

## 1.0 INTRODUCTION

This report presents the results of an assessment of ground subsidence and earth fissures (ground cracks) in the vicinity the Rittenhouse water retention basin that is part of the East Maricopa Floodway Project. The Rittenhouse Basin is located in eastern Maricopa County, Arizona (Figure 1). The main purpose of this study is to:

1) Conduct an overview fatal-flaw evaluation of ground subsidence and earth fissures in the project area.

2) Make recommendations to mitigate the known subsidence and earth fissures that could impact the basins.

### 1.1 Scope of Work

The scope of work for the ground subsidence and earth fissure evaluation included the following activities designed to satisfy the objectives of the study:

- Review and summarize available data concerning site geology, ground subsidence, groundwater withdrawal, and earth fissuring in the vicinity of the proposed Rittenhouse Basin.

- Recent aerial photography (provided by Kirkham Michael) was used for geological photo interpretation to identify suspect earth fissures that may be present within and adjacent to the study area.

- o A geologic reconnaissance of the proposed basin area.

- o Compilation and analysis of the data gathered to document subsidence and earth fissures within the project area.

- o Preparation of this report documenting study findings and conclusions.

Geological Consultants Inc. used available research reports and maps from various sources including the Arizona Geological Survey, the U.S. Geological Survey, and unpublished consultant reports as part of its geological research data base for this study.





## 2.0 CONCLUSIONS AND RECOMMENDATION

**2.1** Based on our ground subsidence and earth fissure research and the field reconnaissance conducted at the Rittenhouse Basin site, the following opinions are provided.

- The proposed basin is within an active ground subsidence area. The data indicates ground subsidence has been active for more than 50 years.

- Groundwater level declines on the order of 150 to 300 feet (Schumann, 1986) within the study area triggered active subsidence in the basin. Residual ground subsidence, which continues after excessive pumping ceases and water level in the aquifer either stabilize or begin to rise, is expected to continue at a diminished rate until the aquifer system in this area achieves equilibrium.

- The ground subsidence is related to deep consolidation of compressible basin fill sediments in response to the lowering of the groundwater table due to excessive pumping for agricultural and domestic uses within the East Salt River Valley (ESRV). However, in the past 10 to 15 years, water level measurements in wells shows a static to rising water table condition has continued due to increased recharge inflow to the aquifer and a reduction in groundwater use.

- Ground subsidence could be exacerbated if water level decline is reinitiated through renewed excessive groundwater use.

- No ground subsidence-related earth fissures were observed at the time of the geologic field reconnaissance within the Rittenhouse basin area. Likewise, no earth fissures were observed outside the basin perimeter that project toward the basin.

  The closest documented earth fissures are about 6-miles south of the Rittenhouse basin. None of these fissures trend toward the project area.

**2.2** The amount of total subsidence to date that has occurred in the basin area is not known. However, ground subsidence, which is documented in nearby areas, may be extrapolated to the project area with reasonable confidence. Documented ground subsidence is 3.9 feet for the period 1934 to 1967 (Strange, 1983) near the Town of Queen Creek, which is about five miles southeast of the Rittenhouse Basin site. This represents an average ground subsidence rate of about 0.12 feet per year. The hydrogeological conditions and basin configuration in the Queen Creek area are similar to that of the Rittenhouse basin area. Therefore, in our opinion, it is conceivable that both areas could have experienced similar ground subsidence during the same period. According to Strange (1983), ground subsidence in the William Air Field area east of the Rittenhouse Basin site was about 0.7 feet for the period 1971 through 1981, which represents an average subsidence rate in the project area of about 0.07 feet.

**2.3** Residual subsidence is expected to continue in the basin area which will result in a lowering of level surface elevation. This phenomenon is expected to continue at a diminishing rate until the aquifer system in this area achieves equilibrium. However, it is also expected that the subsidence will be relatively uniform because this site is located near the central part of the basin, outside of the bedrock boundary where the basin fill sediments are several thousands of feet thick.

**2.4** Although we recommend anticipated future potential ground subsidence should be monitored, in our opinion, it should not be necessary to factor ground subsidence into the design of the Rittenhouse Basin.

The Rittenhouse Basin is located near the central portion of the Chandler Basin. This area is also near the center of the regional bowl of depression caused by ground subsidence. In this area of the subsidence bowl, the ground subsidence is expected to cause a vertical lowering of the ground surface. No land surface tilt is expected in this area as it might be near the margins of the subsiding basin. Therefore, differential ground subsidence across the Rittenhouse Basin site is not expected to adversely impact Basin design.

**2.5** As part of the design of the Rittenhouse Basin, we recommend an ongoing ground subsidence monitoring program be implemented. The program should include periodic





monitoring of both ground subsidence at established benchmarks and groundwater level monitoring of wells in the Basin vicinity. We recommend the subsidence monitoring of the EMF Rittenhouse Basin be conducted in accordance with the recommendations for subsidence monitoring provided in the Program and Policy report prepared for FCDMC as part of the Phase I Structures Assessment Program (Kimley Horn, 2000).

The subsidence monitoring can be tied into the Arizona Department of Water Resources (ADWR) or the Maricopa County networks if benchmarks used to monitor ground subsidence within the East Salt River Valley. Water level data can be obtained from wells in the project vicinity and could be integrated into the ADWR well monitoring program.

# 3.0 GEOGRAPHIC SETTING

## 3.1 Location

The proposed Rittenhouse basin is located in southeast Gilbert, covering portions of the western half of Section 36 in Township 1 South, Range 6 East in the eastern portions of Maricopa County. (Figure 2). The Rittenhouse basin can be accessed by Rittenhouse Road at the Power Road intersection.

## 3.2 Physical Features

Regionally, the project area is situated within and near the south-centered margin of the East Salt River Valley Basin. The basin is bounded on the north and east by the McDowell, Usery, Goldfield, and Superstition Mountains, on the south by the Santan and Sacaton Mountains, and on the west by the South Mountains, the Papago Buttes, the Phoenix Mountains, the Union Hills, and the Deem Hills. Surface runoff toward the site flows to the southwest where drainage channels are intercepted by the East Maricopa Floodway. The Floodway presently discharges flows to the south-southwest and carries the runoff to the Gila River.

At the Rittenhouse basin, the east boundary parallels Power Road, the southwest boundary by the Southern Pacific rail and Rittenhouse road and the north boundary by the Roosevelt canal and EMF alignment. Residential properties and Williams Gateway Airport are east of the basin site. The remainder of the site is presently surrounded by agricultural land.

## 3.3 Climate and Vegetation

The climate of the area is arid to semiarid. Average annual temperature ranges from about 72 degrees to 74 degrees Fahrenheit (F) with summer maximums reaching more than 100 degrees F and winter minimums below freezing (32 degrees F). The precipitation is confined to essentially two seasons during the year, one in the summer and the other in the winter. Average annual rainfall is about six to 8 inches. Although natural desert vegetation, dominated by creosotebush, mesquite, paloverde, annual grasses, cacti (Adams, 1972), is present in isolated areas, the basins are essentially surrounded by either agricultural or residential properties.





# 4.0 GEOLOGICAL EVALUATION

## 4.1 Regional Geologic Setting

The site is located within the Sonoran Desert region in the north-central portion of the Basin and Range Physiographic Province near its boundary with the Transition Zone. The Basin and Range Province is characterized by northwest, north, and northeast trending mountains that rise abruptly from broad, elongated, deep sediment-filled valleys produced by block faulting and folding. The mountains and hills south of the EMF project, the Santan Mountains, are composed predominately of old, Pre-Cambrian age (570 million years ago (mya)) metamorphic schist and igneous granodiorite bedrock, intruded by younger dikes (Ferguson, 1996). The bedrock, which underlies the basin, is overlain by thick sequences of Quaternary age (younger than 1.6 mya) alluvium. The basin fill within the valley commonly makes up the principle groundwater aquifer of the region.

Structurally, the basin-bounding mountain ranges have been uplifted to its present position by episodes of mountain/basin bounding fault movements (Cooley, 1977). The tectonic episodes and deformation, evident in the orientation of foliation planes and joint dip set discontinuities exposed in the bedrock terrain, have provided the mechanics necessary to form grabens, or down-thrown bedrock blocks to create deep intermontane basins that were subsequently filled with sediment.

### 4.1.1 Basin Stratigraphy

The study area is situated near the central portion of a broad alluvium-filled valley that is bounded on the south by the Santan and Sacaton Mountains, on the north and east by the McDowell, Usery, Goldfield, and Superstition Mountains, and on the west by the South Mountains, the Papago Buttes, the Phoenix Mountains, the Union Hills, and the Deem Hills. This portion of the basin is filled with sedimentary deposits that range in thickness from several thousands of feet reaching to more than 11,000 feet southeast of Gilbert (ADWR, VII, 1994).

In the Rittenhouse Basin area, the estimated basin fill thickness is more than 8,000 feet (Oppenheimer et al, 1980). The alluvial deposits contained in the basin can be grossly subdivided into three zones: an upper sand and gravel unit that ranges in thickness from nil to more than 300 feet, a middle silt and clay unit that ranges in thickness from less than 100 feet to more than 1,800 feet, and a lower conglomerate unit that ranges in thickness from less than 100 feet to more than 9,000 feet (ADWR VII, 1994).





# 5.0 GROUND SUBSIDENCE

## 5.1 Overview

Ground subsidence is known to occur in alluvium filled valleys of Arizona where agricultural activities and urban development have caused substantial over-drafting or removal of groundwater from thick basin aquifers. The magnitude of subsidence is directly related to the subsurface geology, the thickness, and compressibility of the alluvial sediments deposited in the valleys, and the net groundwater decline. According to Bouwer (1977), ground subsidence rates range from about one-hundredth to one-half foot per 10-foot drop in groundwater level, depending on the thickness and compressibility of the formation.

## 5.2 Groundwater

The major human-induced factor contributing to subsidence is the large scale pumping and removal of groundwater. Nearly all of the populated southern Arizona basins from Phoenix to Tucson have experienced at least a 100+ foot drop in groundwater level, and an area surrounding the town of Stanfield, Arizona has dropped more than 500 feet. The groundwater level near the study area has dropped from 100 feet to 300 feet (Schumann 1986). Analysis of water level data (ADWR, 2001) from nearby wells (Figure 3) indicates water levels have indeed dropped from 100 to 300 feet from an average depth of groundwater that averaged about 90 feet below ground surface in the late 1930's. The greatest water level declines in the basin occurred through the 1960's to 1980's (Figure 4). Water levels in the Rittenhouse Basin area have either stabilized or increased from about 50 to 75 feet in the last 20 years. The net water level decline for the area is unknown, however, it is expected to be less than 100 feet for the Rittenhouse Basin area.

### 5.2.1 Groundwater Use in the East Salt River Valley Sub-Basin

This portion of the East Maricopa Floodway project is within the East Salt River Valley Sub-Basin (ESRV), one of the seven groundwater sub-basins within the Phoenix Active Management Area (AMA) as defined by the Arizona Department of Water Resources (ADWR). Prior to 1923, the groundwater system in the East Salt River Valley was in equilibrium because the groundwater recharge and outflow were balanced. By 1950, 2.3 million acre-feet per year were needed to meet agricultural demands. As a result, groundwater flow directions were impacted due to the lowering of the water table. Currently, most of the groundwater flows toward three large cones of depression, which have been created by the large scale pumping of groundwater. The cones of depression are located near Scottsdale, Mesa, and Queen Creek (ADWR, VII, 1994).

Groundwater pumping estimates prior to 1984 are not readily available for the ESRV, but are available for the entire Salt River Valley (SRV). In 1915, 15,000 acre-feet of groundwater were pumped from wells in the SRV. By 1942, the annual volume of groundwater withdrawn had increased to approximately one million acre-feet. Approximately 2.3 million acre-feet per year were pumped from the aquifer when groundwater withdrawal peaked in the 1950's. By 1992, annual usage in the SRV had decreased to 1.1 million acre-feet. Approximately 304,900 acre-feet of groundwater were pumped from the ESRV in 1990 (ADWR, VII, 1994).

## 5.3 Regional Subsidence

Prior to the utilization of groundwater resources within the Phoenix area, the water table was higher and hydrogeologic conditions were in equilibrium. Water levels within the aquifer were lowered when pumping was initiated and the basin fill sediments were dewatered. In the arid southwest, the water in the aquifer may be removed by pumping faster than it can be naturally replenished causing a net water table decline. As a result, the weight of the soil column is gradually increased as the buoyant effects and aquifer pressures induced by the water acting on the soil column are decreased. This condition causes increased loading stresses to consolidate portions of the thick compressible sediment that result in the lowering (subsidence) of the land surface over a large area.

Land subsidence was first documented in Arizona in 1934 following the releveling of first-order survey lines by the Coast and Geodetic Survey (now the National Geodetic Survey (NGS)). Subsequent levels by the NGS, the U.S. Geological Survey, the Bureau of Reclamation, and the ADOT has documented substantial ground surface subsidence in south central Arizona including the Salt River Valley, the Queen Creek - Apache Junction area, and the Eloy - Casa Grande - Stanfield area as overdrafting of the aquifer continues.





Subsidence and earth fissures in urban areas can cause a variety of problems. Structures built across fissures may be damaged, street cracks, flow in gravity water and sewer lines can be reversed, and differential subsidence (although rare) can rupture buried utilities (Arizona Geological Survey, 1987). However, design measures can be implemented to mitigate the effects of ground subsidence. Some of these measures can include additional structural reinforcement, oversized pipes, surface drainage controls, and bridging the subsidence feature.

### 5.3.1 Study Area Subsidence

No formal ground subsidence studies have been completed within or directly adjacent to the East Maricopa Floodway alignment. Schumann (1974) estimated that subsidence for the area (including both basins) was between one and 3 feet as of that year. From 1934 to 1967, 3.9 feet of subsidence had been recorded in the town of Queen Creek and 3.8 feet at Buckhorn. At an area near Williams Airport, 0.7 feet of subsidence was measured between 1971 and 1981 (Strange, 1983). Additional subsidence has likely occurred since that time over the entire study area. Assuming the rate of ground subsidence for the 1971 to 1981 period remains the same, the estimated residual ground subsidence for the Rittenhouse Basin area through the year 2001 could total an additional 1.4 feet.

## 5.4 Earth Fissures

Fissures occur in unconsolidated sediments, typically near the margins of alluvial valleys or near the bedrock pediment edge where ground water levels have dropped from about 200 feet to 500 feet below ground surface (Schumann, 1986).

Fissures are initiated underground when tensile stresses exceed the strength of the soils. Tensile stresses induced by the subsidence continue to increase until the ground breaks to form earth fissures. The fissures then propagate upwards to intersect the ground surface (Figure 5). Early signs of earth fissuring are small en echelon hairline cracks and irregular spaced depressions. As fissures develop, they grow in length to several hundreds to thousands of feet and may extend to bedrock or to the water table. The fissures often have vegetation growing in them because the ground is commonly more moist along the earth fissure. In the same area, other physical features associated with fissures may be slump-related escarpments from one inch to a few feet in height, as well as a drainage pattern associated with the fissure that does not conform to the areas local drainage pattern.

### 5.4.1 Earth Fissures at Rittenhouse Basin

The estimated the depth to bedrock (Oppenheimer, 1981) under the Chandler basin is 2,000 to 9,600 feet below ground surface, with the basin fill thickening to the north. The estimated depth to bedrock under the Rittenhouse basin ranges from 8,000 to 9,600 feet below ground surface (Oppenheimer, 1981). A well drilled just south of the Rittenhouse basin was advanced approximately 9,600 feet without hitting bedrock. The Oppenheimer depth-to-bedrock map suggests a relatively uniform deep bedrock surface without protuberances such as isolated buried bedrock highs in the vicinity of, or directly under the proposed basin that could be a focus for earth fissure development.

Interpretation of aerial photography provided by Kirkham Michael did not discern any earth fissures or earth fissure-like features within or adjacent to the proposed basin. This was also confirmed during the field reconnaissance. No earth fissures are known to exist within a five-mile radius of this property.





## 6.0 BIBLIOGRAPHY

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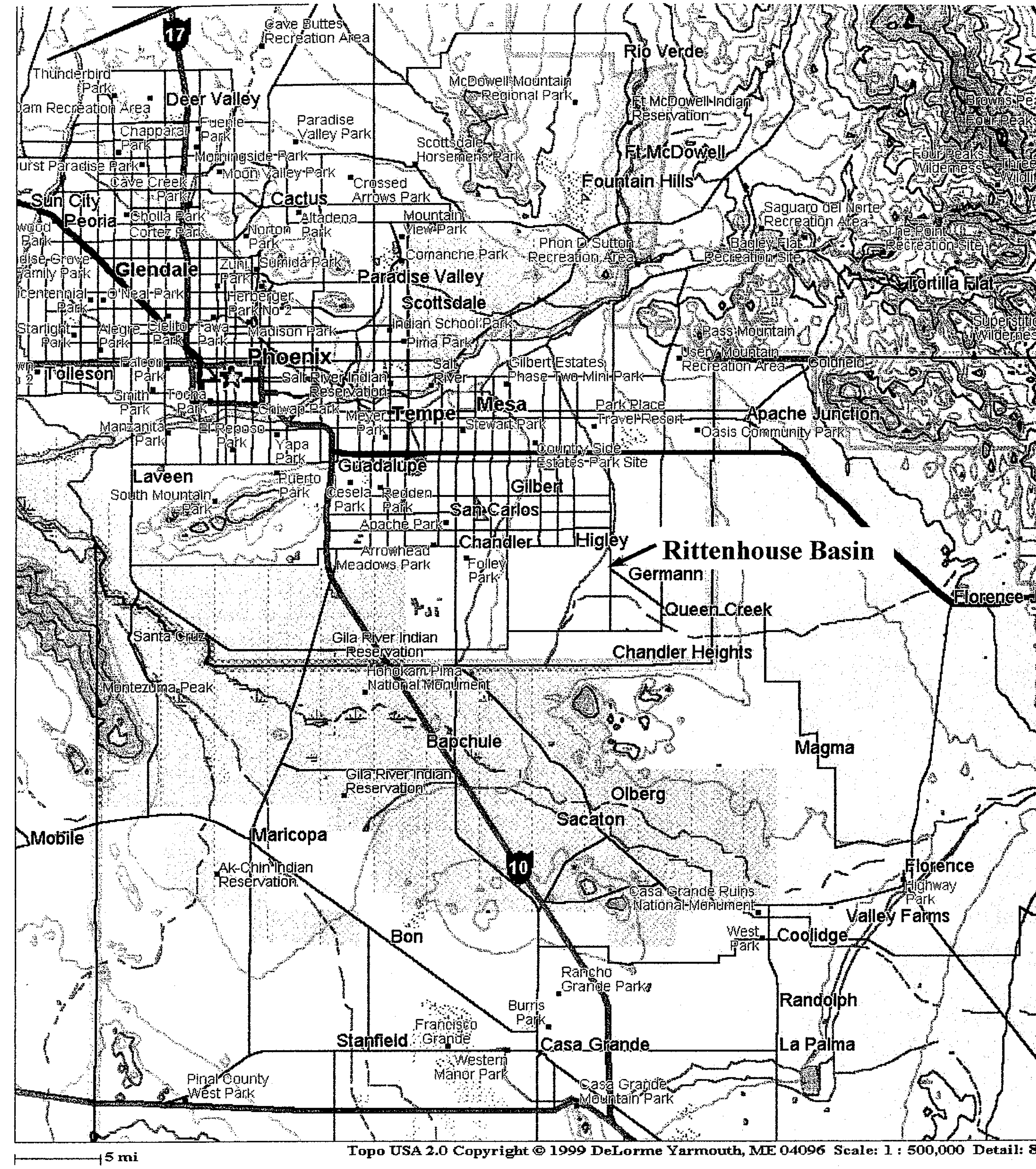
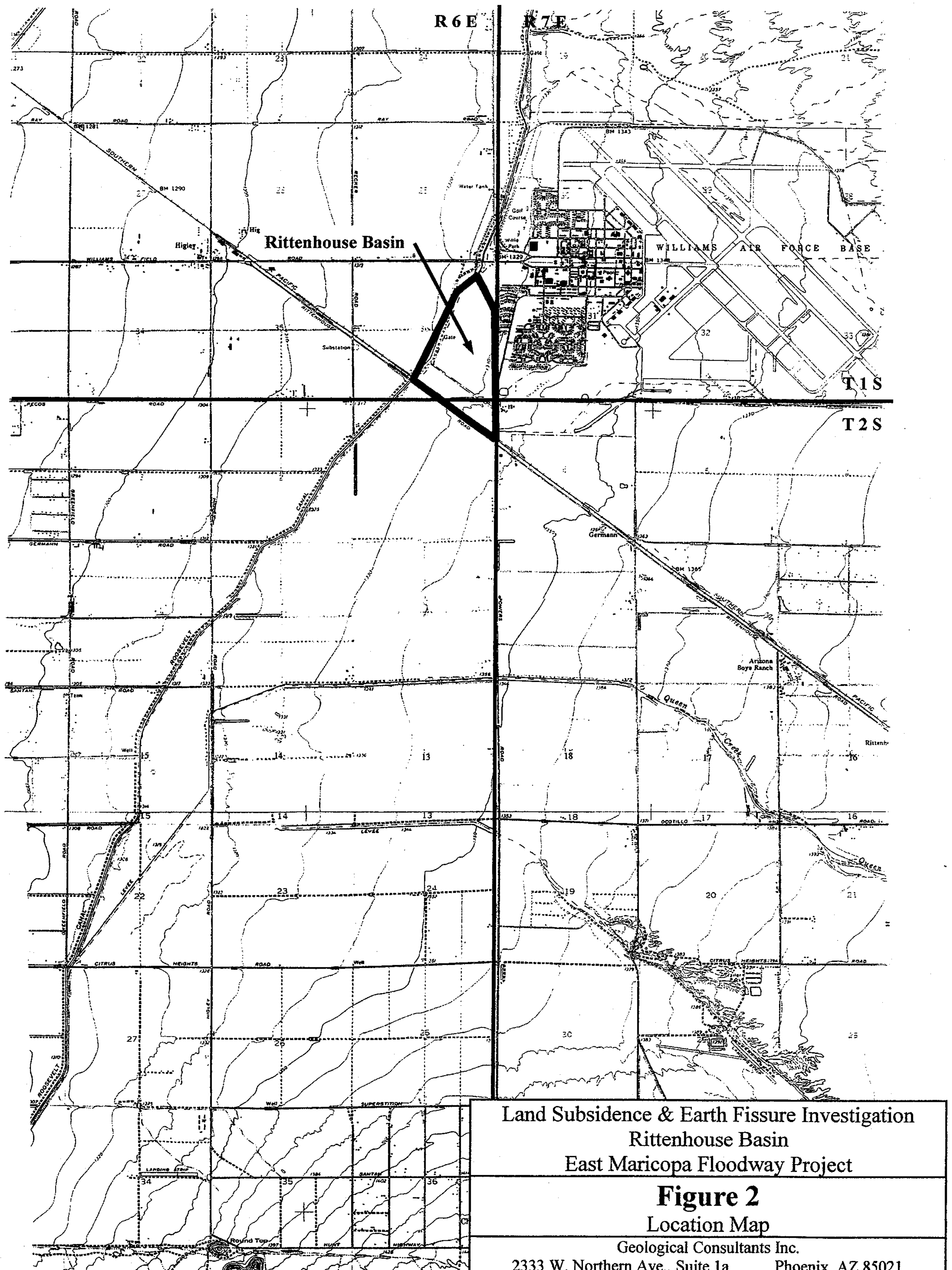



Land Subsidence & Earth Fissure Investigation
Rittenhouse Basin
East Maricopa Floodway Project

**Figure 1**
Site Vicinity Map

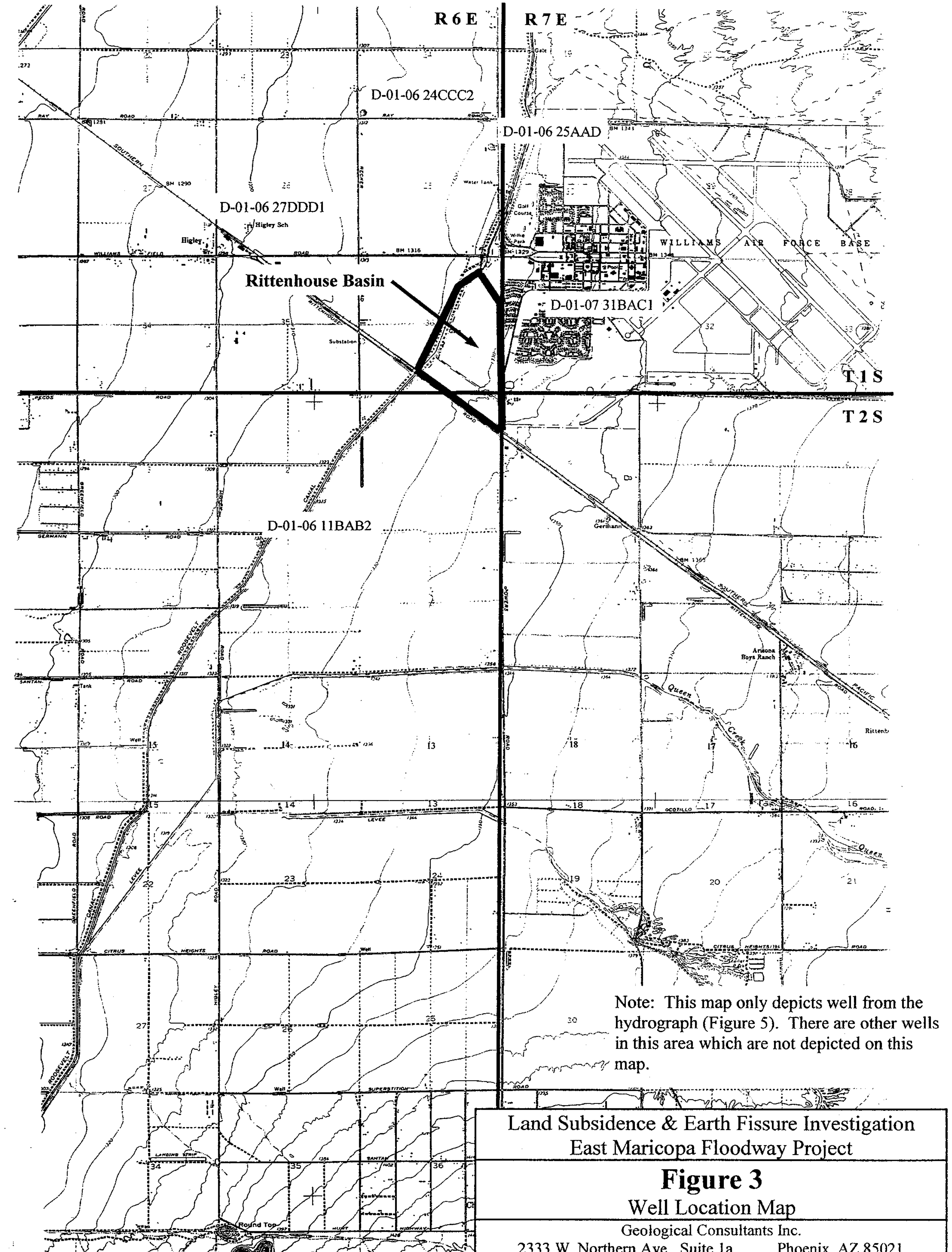
Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021




Land Subsidence & Earth Fissure Investigation
Rittenhouse Basin
East Maricopa Floodway Project

**Figure 2**
Location Map

Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021

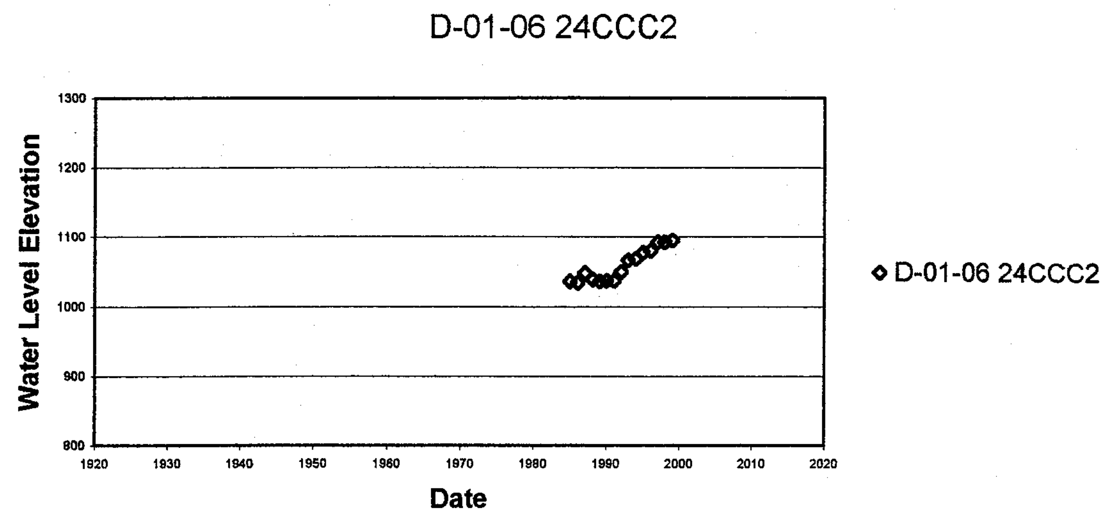


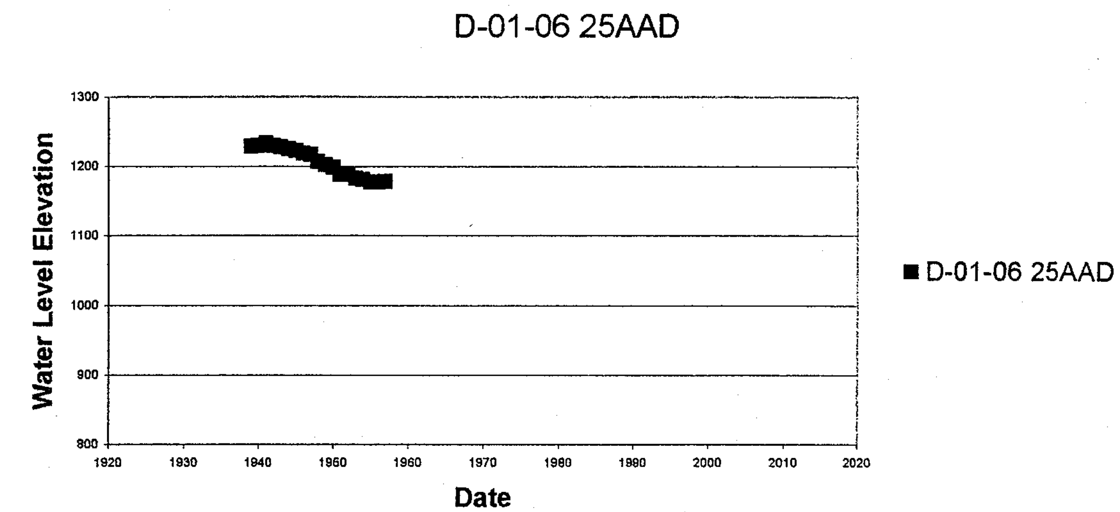

Note: This map only depicts well from the hydrograph (Figure 5). There are other wells in this area which are not depicted on this map.

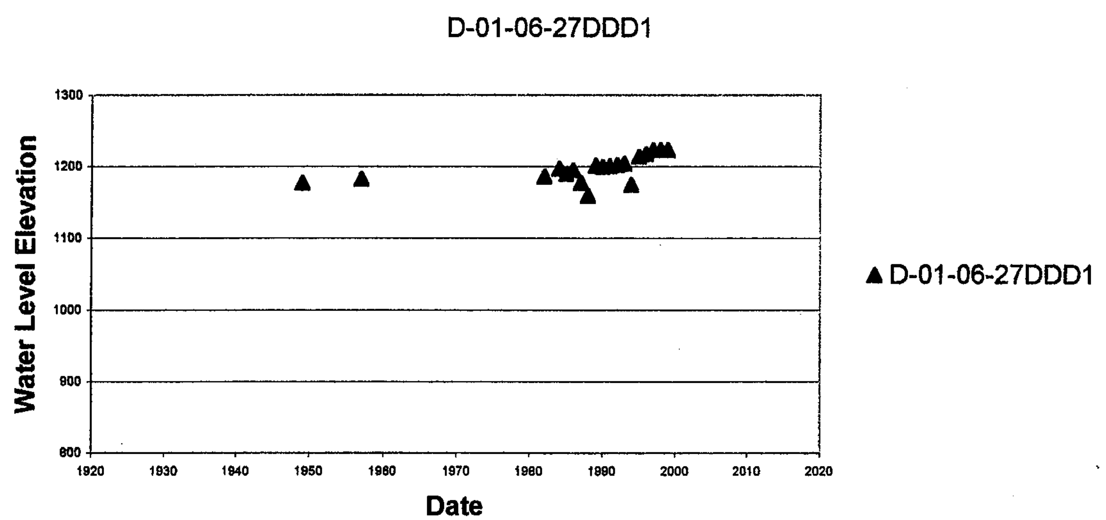
Land Subsidence & Earth Fissure Investigation
East Maricopa Floodway Project

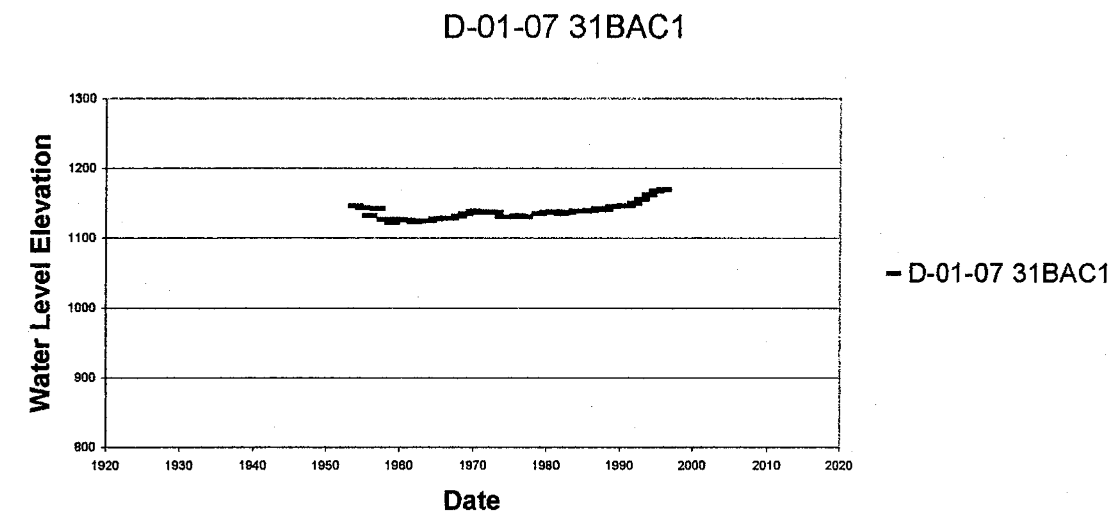
**Figure 3**
Well Location Map

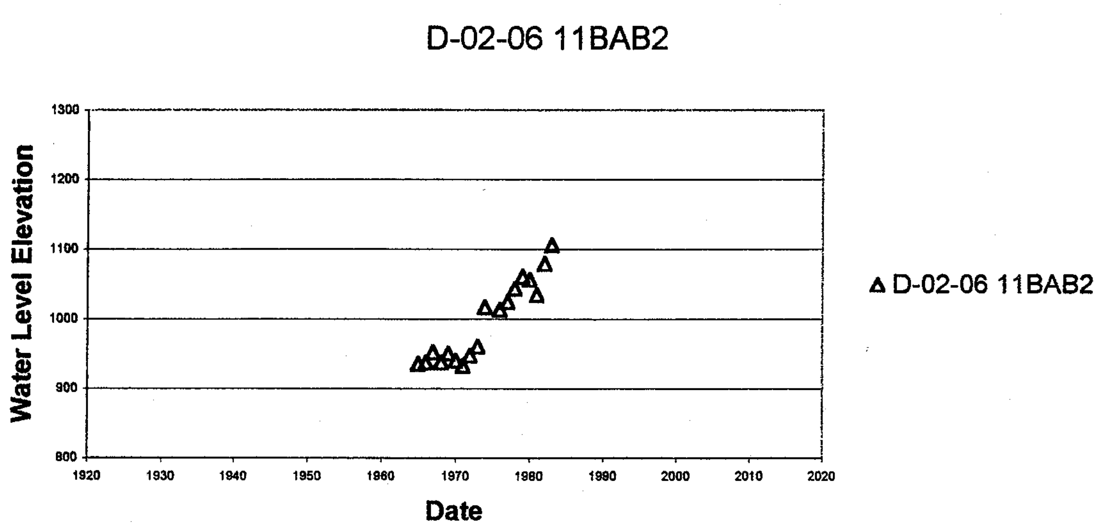
Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021



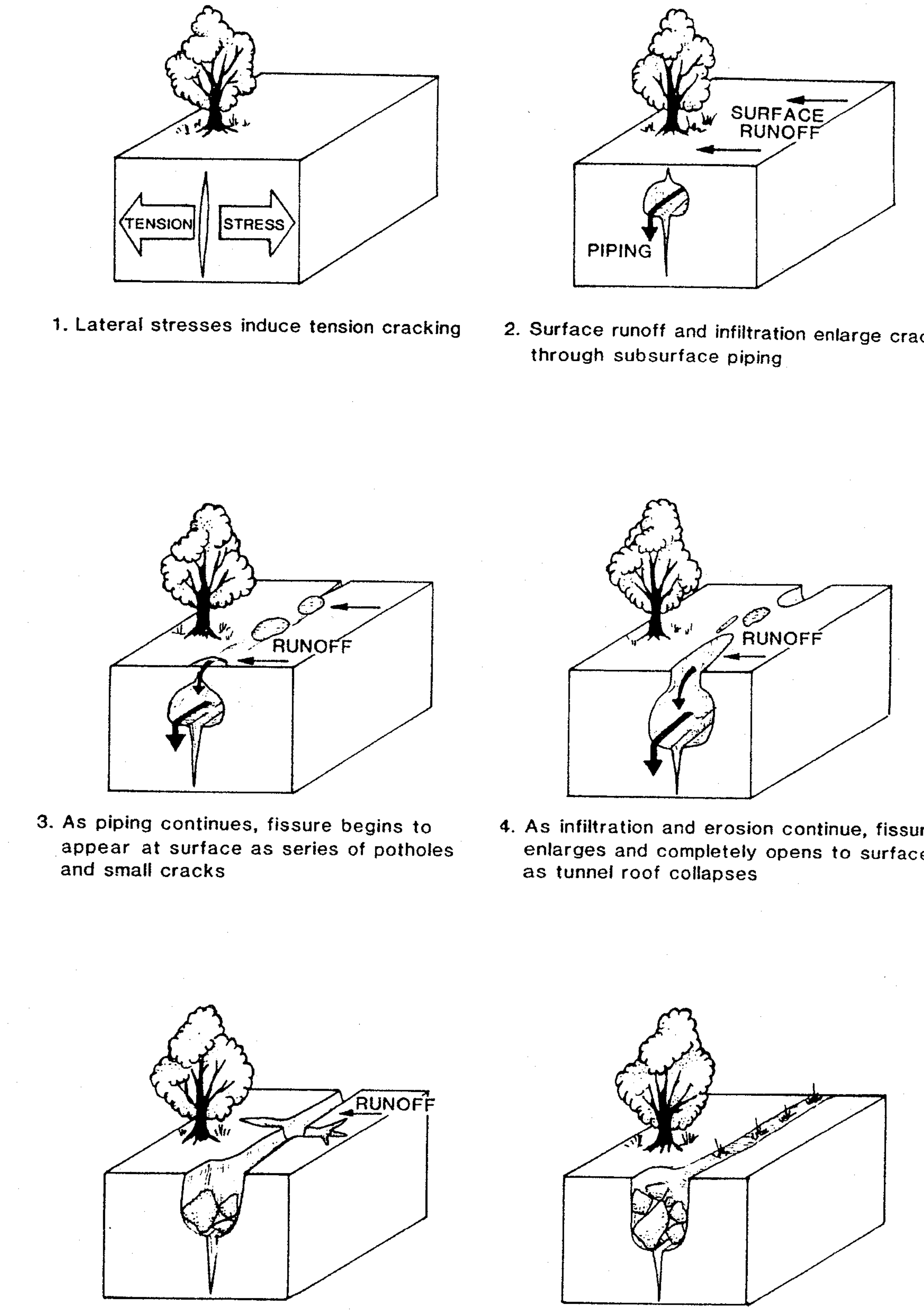




Land Subsidence & Earth Fissure Investigation
Rittenhouse Basin
East Maricopa Floodway Project

**Figure 4**
Well Hydrographs

Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021




1. Lateral stresses induce tension cracking
2. Surface runoff and infiltration enlarge crack through subsurface piping
3. As piping continues, fissure begins to appear at surface as series of potholes and small cracks
4. As infiltration and erosion continue, fissure enlarges and completely opens to surface as tunnel roof collapses
5. The entire fissure is opened to the surface and enlargement continues as fissure walls are widened, extensive slumping and side-stream gullying occur
6. Fissure becomes filled with slump and runoff debris and is marked by vegetation lineament and slight surface depression, it may become reactivated upon renewal of tensile stress

Figure 5. Generalized states of fissure development (from Pewe, 1981).

# SURVEY AND CONTROL

CONSULTANT ENGINEERING, INC
3404 WEST CHERYL DRIVE, SUITE A-255
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PHOENIX AZ 85069-7167
(602) 866-5090

## FLOOD CONTROL BASIN SURVEY METHODS

The horizontal control for these basins was determined to be referenced to the Maricopa County Geodetic Densification and Cadastral Survey (GDACS) control network, which is based upon the National Geodetic Survey, North American Datum of 1983. This was also the datum used for mapping the East Maricopa Floodway. The vertical control for these basins was determined to be referenced to the North American Vertical Datum of 1929. The monuments used are published on the stated East Maricopa Floodway Mapping Sheets completed in 1993 by HNTB.

The mapping for this project was completed through a combination of Aerial and Terrestrial methods. Global Positioning System (GPS) surveying equipment was used to establish locations of: boundary (section) and right of way monuments; aerial panels; check ground elevations; and to map the current EMF channel. Conventional terrestrial survey equipment was used to measure vertical elevations for the aerial panel control and to collect measurements for the, more vertical critical, existing paving and bridge data.

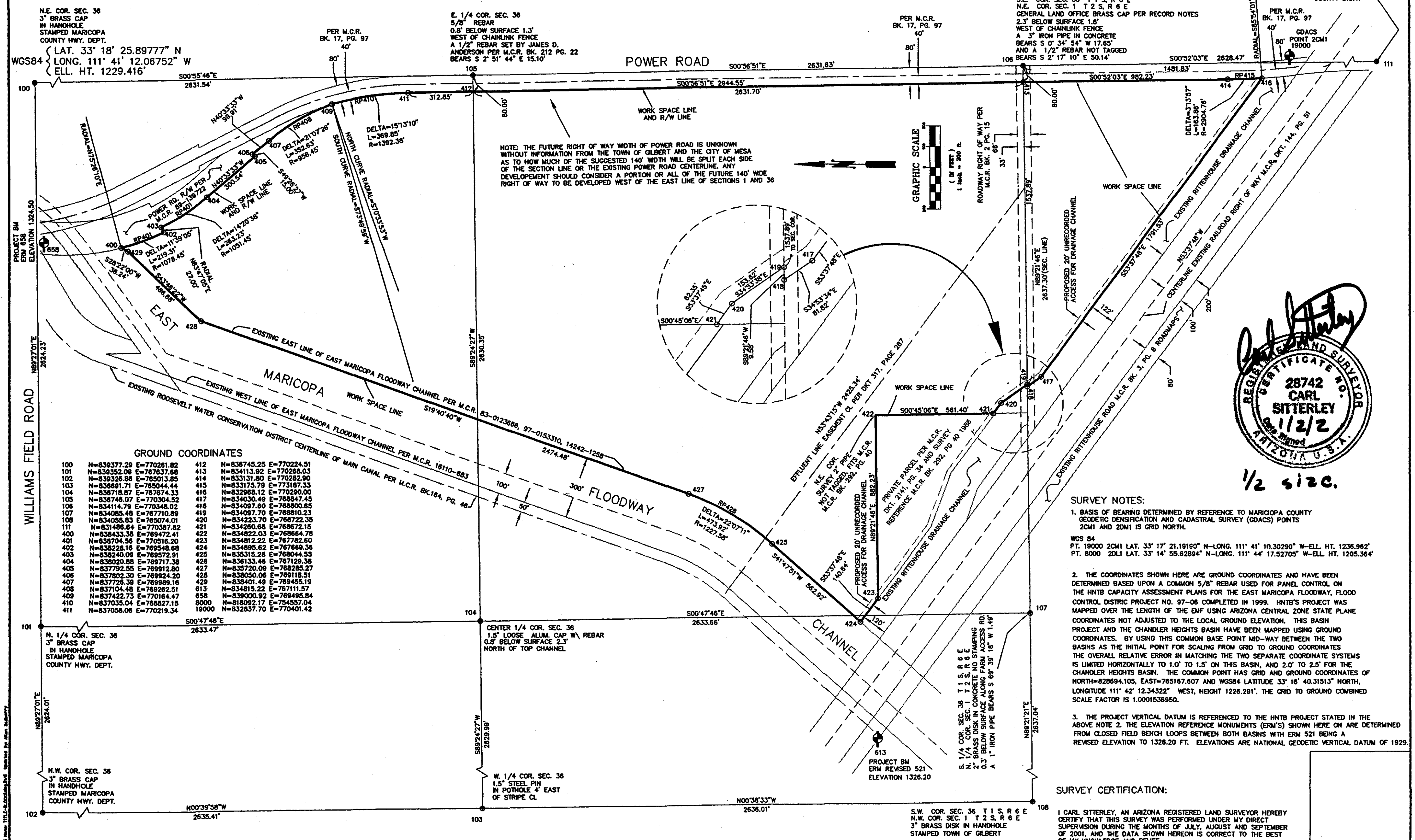
Boundary measurement data and researched record deeds, plats and maps were analyzed by an Arizona Registered Land Surveyor to identify boundary and easement issues affecting the development of each site. A results of survey and a results of survey control drawing is being produced for each site to provide data for current design, current issue resolution, future land transfers, and future construction survey control.

## MAPS OF SURVEY

The following maps of survey show the boundary control system used for the Rittenhouse Basin and the property boundary established for the Rittenhouse Basin.

# RESULTS OF CONTROL SURVEY-RITTENHOUSE DETENTION BASIN

## TOWNSHIPS 1 SOUTH AND 2 SOUTH, RANGE 6 EAST, GILA AND SALT RIVER MERIDIAN




### GROUND COORDINATES

| | | | |
|---|---|---|---|
| 100 | N=839377.29 E=770261.82 | 412 | N=836745.25 E=770224.51 |
| 101 | N=839352.09 E=767637.68 | 413 | N=834113.92 E=770268.03 |
| 102 | N=839326.86 E=765013.85 | 414 | N=833131.80 E=770282.90 |
| 103 | N=836691.71 E=765044.44 | 415 | N=833175.79 E=773187.33 |
| 104 | N=836718.87 E=767674.33 | 416 | N=832968.12 E=770290.00 |
| 105 | N=836746.07 E=770304.52 | 417 | N=834030.49 E=768847.45 |
| 106 | N=834114.79 E=770348.02 | 418 | N=834097.60 E=768800.65 |
| 107 | N=834085.48 E=767710.89 | 419 | N=834097.70 E=768810.23 |
| 108 | N=834055.83 E=765074.01 | 420 | N=834223.70 E=768722.35 |
| 111 | N=831486.64 E=770387.82 | 421 | N=834260.68 E=768672.15 |
| 400 | N=838433.38 E=769472.41 | 422 | N=834822.03 E=768664.78 |
| 401 | N=838704.56 E=770516.20 | 423 | N=834812.22 E=767782.60 |
| 402 | N=838228.16 E=769548.68 | 424 | N=834895.62 E=767669.36 |
| 403 | N=838240.09 E=769572.91 | 425 | N=835315.28 E=768044.55 |
| 404 | N=838020.88 E=769717.38 | 426 | N=836133.46 E=767129.38 |
| 405 | N=837792.55 E=769912.80 | 427 | N=835720.09 E=768285.27 |
| 406 | N=837802.30 E=769924.20 | 428 | N=838050.06 E=769118.51 |
| 407 | N=837726.39 E=769989.16 | 429 | N=838401.49 E=769455.19 |
| 408 | N=837104.48 E=769262.51 | 613 | N=834815.22 E=767111.57 |
| 409 | N=837422.73 E=770164.47 | 658 | N=839000.92 E=769495.84 |
| 410 | N=837035.04 E=768827.15 | 8000 | N=818092.17 E=754557.04 |
| 411 | N=837058.06 E=770219.34 | 19000 | N=832837.70 E=770401.42 |

### SURVEY NOTES:

1. BASIS OF BEARING DETERMINED BY REFERENCE TO MARICIOPA COUNTY GEODETIC DENSIFICATION AND CADASTRAL SURVEY (GDACS) POINTS 2CM1 AND 2DM1 IS GRID NORTH.

WGS 84
PT. 19000 2CM1 LAT. 33° 17' 21.19190" N-LONG. 111° 41' 10.30290" W-ELL. HT. 1236.962'
PT. 8000 2DL1 LAT. 33° 14' 55.62894" N-LONG. 111° 44' 17.52705" W-ELL. HT. 1205.364'

2. THE COORDINATES SHOWN HERE ARE GROUND COORDINATES AND HAVE BEEN DETERMINED BASED UPON A COMMON 5/8" REBAR USED FOR PANEL CONTROL ON THE HNTB CAPACITY ASSESSMENT PLANS FOR THE EAST MARICOPA FLOODWAY, FLOOD CONTROL DISTRIC PROJECT NO. 97-06 COMPLETED IN 1999. HNTB'S PROJECT WAS MAPPED OVER THE LENGTH OF THE EMF USING ARIZONA CENTRAL ZONE STATE PLANE COORDINATES NOT ADJUSTED TO THE LOCAL GROUND ELEVATION. THIS BASIN PROJECT AND THE CHANDLER HEIGHTS BASIN HAVE BEEN MAPPED USING GROUND COORDINATES. BY USING THIS COMMON BASE POINT MID-WAY BETWEEN THE TWO BASINS AS THE INITIAL POINT FOR SCALING FROM GRID TO GROUND COORDINATES THE OVERALL RELATIVE ERROR IN MATCHING THE TWO SEPARATE COORDINATE SYSTEMS IS LIMITED HORIZONTALLY TO 1.0' TO 1.5' ON THIS BASIN, AND 2.0' TO 2.5' FOR THE CHANDLER HEIGHTS BASIN. THE COMMON POINT HAS GRID AND GROUND COORDINATES OF NORTH=828694.105, EAST=765167.607 AND WGS84 LATITUDE 33° 16' 40.31513" NORTH, LONGITUDE 111° 42' 12.34322" WEST, HEIGHT 1226.291'. THE GRID TO GROUND COMBINED SCALE FACTOR IS 1.0001536950.

3. THE PROJECT VERTICAL DATUM IS REFERENCED TO THE HNTB PROJECT STATED IN THE ABOVE NOTE 2. THE ELEVATION REFERENCE MONUMENTS (ERM'S) SHOWN HERE ON ARE DETERMINED FROM CLOSED FIELD BENCH LOOPS BETWEEN BOTH BASINS WITH ERM 521 BEING A REVISED ELEVATION TO 1326.20 FT. ELEVATIONS ARE NATIONAL GEODETIC VERTICAL DATUM OF 1929.

### SURVEY CERTIFICATION:

I CARL SITTERLEY, AN ARIZONA REGISTERED LAND SURVEYOR HEREBY CERTIFY THAT THIS SURVEY WAS PERFORMED UNDER MY DIRECT SUPERVISION DURING THE MONTHS OF JULY, AUGUST AND SEPTEMBER OF 2001, AND THE DATA SHOWN HEREON IS CORRECT TO THE BEST OF MY KNOWLEDGE AND BELIEF.

| DATE | REVISIONS |
|---|---|
| | |

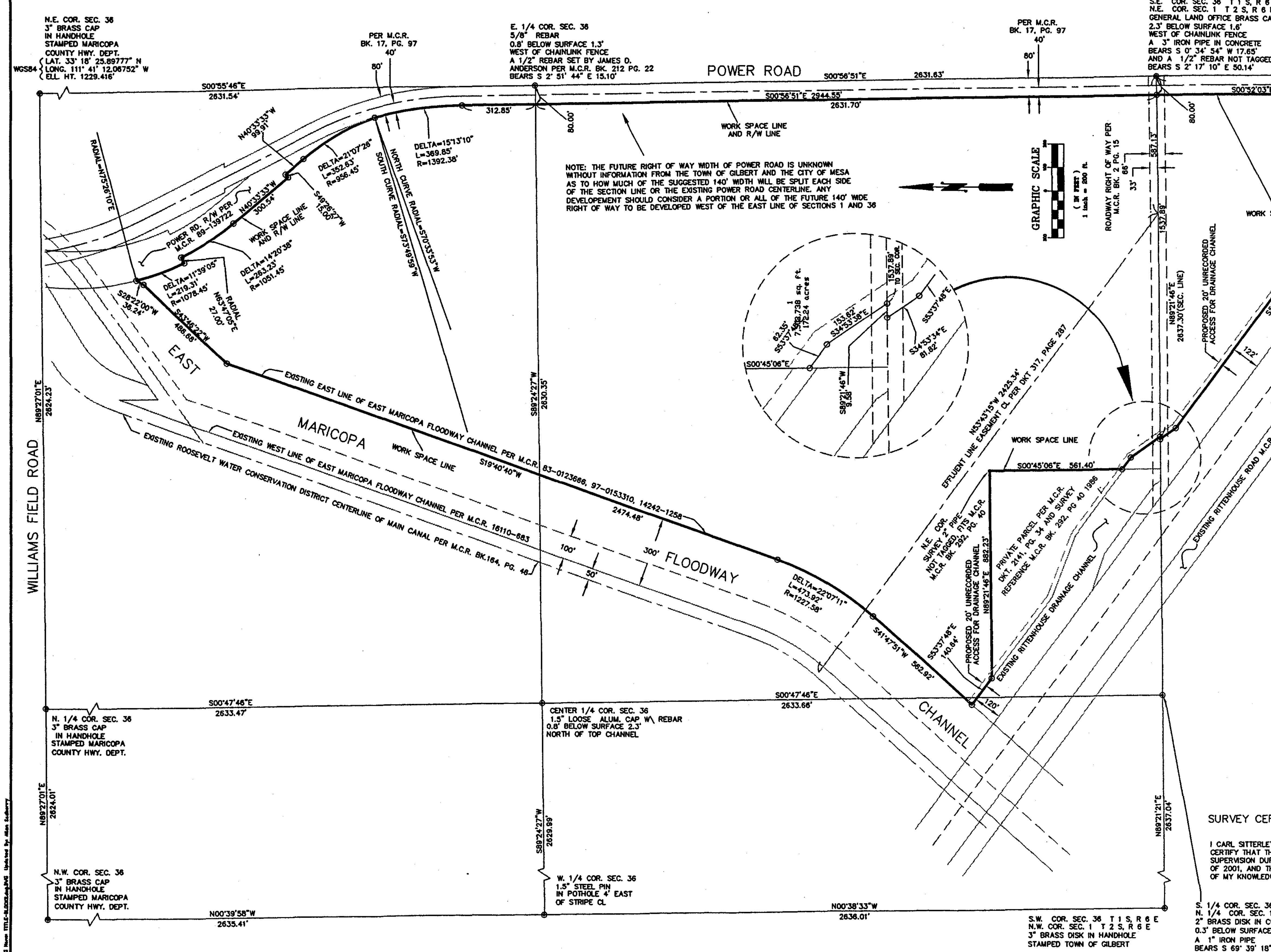

CEI
CONSULTANT ENGINEERING, INC.
3404 W. CHERYL DRIVE
PHOENIX, ARIZONA
602-866-5090


RESULTS OF CONTROL SURVEY
FOR
MARICOPA COUNTY FLOOD CONTROL

| | |
|---|---|
| DATE | 01-02-02 |
| SCALE | 1" = 200' |
| DRAWN BY | CRS |
| APPROVED BY | CRS |
| SHEET | 1 OF 1 |

# RESULTS OF SURVEY–RITTENHOUSE DETENTION BASIN

TOWNSHIPS 1 SOUTH AND 2 SOUTH, RANGE 6 EAST, GILA AND SALT RIVER MERIDIAN




REGISTERED LAND SURVEYOR
CERTIFICATE NO. 28742
CARL SITTERLEY
12/28/01
ARIZONA U.S.A.

1/2 SIZE

## SURVEY NOTES:

1. BASIS OF BEARING DETERMINED BY REFERENCE TO MARICOPA COUNTY GEODETIC DENSIFICATION AND CADASTRAL SURVEY (GDACS) POINTS 2CM1 AND 2DM1 IS GRID NORTH.

   WGS84
   2CM1 LAT. 33° 17' 21.19190" N–LONG. 111° 41' 10.30290" W–ELL. HT. 1236.962'
   2DL1 LAT. 33° 14' 55.62894" N–LONG. 111° 44' 17.52705" W–ELL. HT. 1205.364'

2. THE PURPOSE OF THIS DRAWING IS TO PROVIDE:
   A. A RELATIONSHIP BETWEEN THE FOUND SECTIONAL MONUMENTS
   B. IDENTIFY POTENTIAL ISSUES AFFECTING THE AVAILABLE WORK AREA FOR CONSTRUCTION OF THE FUTURE BASIN, SUCH AS THE EXISTING POWER ROAD RIGHT OF WAY, EAST MARICOPA FLOODWAY CHANNEL RIGHT OF WAY, RITTENHOUSE CHANNEL RIGHT OF WAY.
   C. EVIDENCE TO SUPPORT EXISTING BOUNDARY LINES FOR THE PRIVATE PARCEL ADJOINING THE MARICOPA COUNTY FLOOD CONTROL FEE OWNERSHIP.

3. THIS DRAWING DOES NOT INDICATE BOUNDARY LINES SHOWING THE EXTENTS OF THE FLOOD CONTROL CONTIGUOUS OWNERSHIP INTERESTS. SUCH INTERESTS EXTEND BEYOND THE LIMITS SHOWN. THE LINES LABELED "WORK SPACE LINE" INDICATE EXISTING RIGHT OF WAY LINES FOR THE CHANNELS AND ROAD WAYS WHICH MAY EFFECT THE BASIN DEVELOPMENT. THIS INCLUDES THE NORTH AND EAST BOUNDARY LINE FOR THE PRIVATELY HELD OWNERSHIP OF THE SANTA FE PACIFIC PIPELINE. THESE LINES PROVIDE A REFERENCE FOR ANY FUTURE TRANSFER OF OWNERSHIP AND TO FURTHER IDENTIFY ANY POTENTIAL AREAS UNAVAILABLE FOR THE BASIN IMPROVEMENTS. THIS INCLUDES THE FOLLOWING: ADDITIONAL POWER ROAD RIGHT OF WAY REQUIREMENTS NEEDED TO MEET CITY AND COUNTY TRAFFIC STANDARDS; ADDITIONAL RIGHT OF WAY FOR THE MAINTENANCE OF THE RITTENHOUSE CHANNEL OR THE EAST MARICOPA FLOOD WAY CHANNEL; AND OF SPECIAL NOTE THE REQUIRED LEGAL ACCESS TO THE MENTIONED PRIVATE PARCEL WHICH MAY HAVE ACCESS BLOCKED WITH THE EXISTING CHANNELS AND/OR ANY BASIN DESIGNS.

4. LEGAL DESCRIPTIONS REFERENCED FOR THE RITTENHOUSE DRAINAGE CHANNEL ARE SHOWN ADJUSTED TO MATCH AND ADJOIN THE EXISTING SANTA FE RAILROAD RIGHT OF WAY AS DETERMINED TO BE 100 FEET EACH SIDE OF THE CENTERLINE OF THE LOCATED RAILS.

## SURVEY CERTIFICATION:

I CARL SITTERLEY, AN ARIZONA REGISTERED LAND SURVEYOR HEREBY CERTIFY THAT THIS SURVEY WAS PERFORMED UNDER MY DIRECT SUPERVISION DURING THE MONTHS OF JULY, AUGUST AND SEPTEMBER OF 2001, AND THE DATA SHOWN HEREON IS CORRECT TO THE BEST OF MY KNOWLEDGE AND BELIEF.

S. 1/4 COR. SEC. 36 T 1 S, R 6 E
N. 1/4 COR. SEC. 1 T 2 S, R 6 E
2" BRASS DISK IN CONCRETE NO STAMPING
0.3' BELOW SURFACE ALONG FARM ACCESS RD.
A 1" IRON PIPE
BEARS S 69° 39' 18" W 1.49'

| DATE | REVISIONS |
|---|---|
| | |

CEI
CONSULTANT ENGINEERING, INC.
3404 W. CHERYL DRIVE
PHOENIX, ARIZONA
602-866-5090

RESULTS OF SURVEY
FOR
MARICOPA COUNTY FLOOD CONTROL

| | |
|---|---|
| DATE | 12-28-01 |
| SCALE | 1" = 200' |
| DRAWN BY | CRS |
| APPROVED BY | CRS |
| SHEET | 1 OF 1 |

# MAPPING

**AERIAL MAPPING, INC**
**3141 WEST CLARENDON AVENUE**
**PHOENIX AZ 85017**
**(602) 263-5728**

## TOPOGRAPHIC MAPPING PROCEDURES

The topographic mapping used for these projects was compiled from stereo aerial photography using conventional first order analytical stereo plotters. We flew multiple flight lines of black & white stereo imagery at each location. Photographs and mapping diapositives were printed and the control points identified. Pass points were marked on each photo. The field control was densified for mapping purposes by using a simultaneous large block adjustment of the control and marked pass points.

We digitized the standard planimetric features, including roads, bridges and other structures, buildings and utilities, in 3d MicroStation DGN files using the layering codes necessary for the FCDMC GIS submittals. The ground topography was defined with digitized 3d breaklines along all sudden changes of elevation. Closely spaced mass points were measured to capture the general ground surface changes. The breaklines and mass points were used to create a Digital Terrain Model, from which the 1' contour information was generated. Elevation labels were added to the index contours, and spot elevations were placed where needed.

After the final editing, the complete set of mapping data was prepared for GIS conversion. We examined the data set for pseudo nodes, closed polygons and correct layer tags. The data sets were then converted to FCDMC CAD delivery file specifications.

We created ortho rectified photo overlays using the mapping DTM as a rectification guide. The digital imagery was aligned to the control, and mosaiced into seamless tone matched sheet windows.

## MAPPING

The two sheets in this Appendix show the aerial photography for the Rittenhouse Basin area and the topographic mapping developed from it.

Photogrammetry was augmented with supplemental ground survey points using GPS methods, as reported in Appendix D - Survey and Control.

# LANDSCAPING AND EROSION CONTROL MULTIUSE COORDINATION

**LOGAN SIMPSON DESIGN**
**51 WEST THIRD STREET, SUITE 450**
**TEMPE AZ 85281**
**(480) 967-1343**

## I. *Existing Physical Characteristics*

### A. Vegetation, terrain, existing features.

Existing vegetation is primarily composed of native mesquite, sporadic desert scrub and small undergrowth. The arrangement of mesquites on the site indicates past uses as a tree farm, due to the arrangement of the plants into defined rows. As part of the proposed drainage improvements, the site will be cleared and grubbed of all existing vegetation. Some of the existing trees are being salvaged by the FCD; the trees will be auctioned to private parties. No salvaged trees are intended for replanting after completion of the drainage improvements.

The existing terrain of the site is predominately flat. There are no identifiable high or low points, and are few noticeable man-made features. Fencing, both barbed wire and split rail, are located around portions of the site's perimeter. An abandoned corral is located in the southeast portion of the site. There is evidence of wildcat dumping, around the site. This site has few natural features; it has likely been impacted by previous agriculture, off-road activities and general use. Due to the future use of this site and the associated excavation, the existing terrain will be entirely modified into a large basin configuration.

### B. Adjacent Land Uses and Prominent Features.

The site is surrounded by several different land use types. The most prominent land use is agriculture. To the north of the site, land uses include agriculture, light industrial associated with the agriculture, recreation, in the form of a golf course, Williams Gateway Airport and future residential areas. Contiguous to the western edge of the site is the EMF. Further west, beyond the EMF, are primarily agriculture uses. Rittenhouse Road abounds the southern edge of the site. South of Rittenhouse Road agriculture is the primary land use, although this area is planned as a future residential addition to the existing Power Ranch development. The eastern edge of the site is bounded by Power Road which serves residential properties built immediately east of the roadway.

### C. Views and Viewsheds.

Views of the EMF and of airplanes from Williams Gateway Airport are visible from the site, otherwise on-site views are very limited by the scarcity of natural and man-made features. Significant views to off-site landforms include those looking southward to the San Tan Mountains, northeasterly towards the Superstition Mountains, and northward to Red Mountain and the McDowell Mountains beyond. The vistas to the San Tan Mountains are the most prominent, and should be considered during development of the basin site plan. The site will be completed re-graded and excavated below its existing elevation, so views of the EMF and off-site vistas will be eliminated, except from higher elevations around the perimeter of the site (i.e., along the EMF, etc.).

## II. *Existing Surrounding Land Uses/Zoning/Multi-Use Opportunities*

### A. Land Use Planning.

The site is currently undeveloped. Later sections in this report will elaborate on the Town of Gilbert's intended use of this site as a golf course. Adjacent land uses are predominantly agriculture and residential. The site is currently zoned as Open Space by the Town of Gilbert.

### B. Recreation/Multi-Use Opportunities.

Recreation and multi-use opportunities are somewhat constrained by the site's locations within the FAA's Wildlife Hazard Area surrounding the Williams Gateway Airport, in that no activities can be undertaken that might create bird habitat areas. This means that the development within the basin will need to be more urban in character and will have few opportunities for any naturalistic-type landscaping installation. Possibilities for use of the site for wildlife viewing or environmental education are substantially reduced by this constraint. However, the Town of Gilbert has conducted studies of the site for development of a golf course and is currently preparing initial course layouts for the basin area. Further elaboration of that proposal is provided below. Mutli-use opportunities are afforded by the proximity and potential connection to the East Maricopa Floodway (EMF). The Maricopa County Trail Commission has designated the eastern maintenance road of the EMF as it passes by the site as the location for the regionally-based Marathon Trail. While this trail will serve a regional purpose, it will also accommodate local community recreation users and is consistent with the *Town of Gilbert, 1996-2001 Parks, Open Space and Trails Plan.*

## III. *Proposed Land Use (by Town of Gilbert)*

### A. Land Use and Landscaping.

The Town of Gilbert has partnered with a local golf course architect, in association with the National Golf Foundation (NGF), to prepare initial plans for development of a regulation size golf course to be located within the Rittenhouse Basin. The golf course architect is working in conjunction with the NGF to insure compatibility of the layout to national golf course standards. The NGF previously completed a Feasibility Study to determine if the proposed golf course is needed at this location, if it would be profitable, and what level of play would be offered. The results of this survey indicate that a regulation size, 18-hole golf course at the Rittenhouse Basin would be feasible for the Town of Gilbert to build, maintain and operate.

### B. Golf Course Layout and Character.

The golf course architect has developed an initial layout plan depicting both the layout and character of the proposed golf course. The layout was prepared after coordination with the Rittenhouse Basin engineers.

### C. Access.

Access to the site can be gained by many modes of travel including motor vehicle, bicycle, rollerblade/skateboard and by foot. The primary form of transportation to the site will be by motor vehicle. Vehicular access to the site will be from Power Road, about mid-way between Rittenhouse Road and Williams Field Road. Maintenance personnel will also be able to access the site from Rittenhouse Road where an access route will lead to the Maintenance Facility proposed for the southwest corner of the Basin. Bicyclists will have access to the site via the vehicular access off Power Road; it is expected that this user group will also have access through connections to the EMF/Marathon Trail. There may also be additional points of entrance along the perimeter based on further refinement of the course site plan. Pedestrians, other than those on bicycles, will have access to the facility via sidewalks and paths adjacent to roadways leading to the site and via connections to the EMF/Marathon Trail. It is assumed that the trail connections will be "roughed in" during construction of the Rittenhouse Basin and improvements made by the Town of Gilbert as necessary to satisfy the golf course use.

There may be times when access to the site will not be allowed, for example; after operations hours, during times of basin inundation and when more extensive maintenance is being completed. During these circumstances, access to the site will need to be controlled. This will be accomplished through a combination of gates, fences and signage installed by the Town of Gilbert.

**D. Landform/Grading.**

Golf course grading within the site will be primarily as proposed by the golf course architect. This grading could include berming and contouring to aid in the creation of a functioning and aesthetically pleasing golf course. The features associated with a golf course all require special grading or contouring for proper drainage, ball play and aesthetics. These may include Tee boxes, sand traps, water hazards, greens and fairways. Accent grading by the Town of Gilbert around the perimeter of the site, if desired, will include berming around parking lots to meet screening requirements and also for aesthetics.

## *IV. Proposed Rittenhouse Basin Features (by Flood Control District of Maricopa County)*

**A. Basin Grading.**

Excavation for the basin will begin at the right-of-way limits. Slopes along the perimeter of the basin will generally be a constant 4:1 slope ratio, although some slight variations in the slopes can be expected at various locations. A 5-6 acre platform of land will be retained along Power Road ensuring that a "dry" area for the golf course clubhouse, associated structures and parking can be retained during times of basin inundation. These facilities would be designed and built by the Town of Gilbert to meet all municipal codes, regulations and standards. No "aesthetic grading" is anticipated by the FCD for the clubhouse area.

The floor of the Rittenhouse Basin will be gently sloped to the outlet structures that daylight into the EMF to create positive drainage within of the basin. The basin bottom grade will be so slight as to appear flat. Maintenance roads or paths may be built into the side slopes of the basin if identified on the golf course architect's layout, to allow for vehicles to enter or exit the basin at specified locations. Pedestrian paths, if called for by the golf course architect, would also require special grading to traverse the side slopes and to conform to applicable ADA requirements.

**B. Drainage Features.**

Due to the use of the site as a flood control basin, there will be certain infrastructure features constructed as part of the project. The proposed features include an approximately 1,500' long concrete side-weir along the EMF (northwest portion of the basin). The side-weir is proposed to have a 20' concrete top width. As part of the side weir, there will be a concrete stair-stepped energy dissipater and 4' deep by 30' wide plunge pool on the basin side for the weir's entire length. In addition, a single pipe outlet with concrete headwalls will be provided to drain the basin.

Even though these elements as planned are utilitarian, both in function and appearance, there exists an opportunity to influence their overall appearance, and in some cases, how the element is integrated into the landscape. Some examples of how the appearance or location of a device may be manipulated are as follows:

- Side-weir and apron. The proposed side-weir is located along the East edge of the EMF, and the subsequent West edge of the Rittenhouse Basin. The main purpose of this device is to set a predetermined elevation at which flows in the EMF will spill over into the basin via the side-weir, thus alleviating some demands on the EMF, and allowing for increased volume down-channel. The side-weir is approximately 1,500' in length, and will have a access or maintenance road atop it, with a width of about 20'. The weir steps into the basin and terminates into a apron or plunge pool, that will function as a erosion control device by spreading the water out, and reducing the velocity of the incoming flows. This structure can be constructed of several different materials, including concrete, gabions, geo-blocks and similar pre-fabricated devices. The overall shape and location of the weir will conform to standards set by the Engineer, however, the appearance may be modified to produce a more aesthetic feature. This may be accomplished through the use of any of the following, or a combination of two:
  - 1. Implementation of colored concrete. The use of colored concrete can alter the appearance of any concrete structure, while not influencing the overall structural integrity of the feature. Colored concrete can be applied into the mix, thus allowing for an integral color, or applied to the surface of the material after installation, as a dust coating. The integral color would be preferred, due to the high risk of chipping and fading of the surface application. The color additive can take on the role of disguising the structure, thus blending it into the surrounding environment, giving it a natural look, or the color can be used to create designs and conspicuous patterns attracting attention. One or several colors can be installed, depending on the desired aesthetics of the structure.
  - 2. Texture, as related to concrete. With the use of concrete as the primary material in the side-weir, the opportunity to manipulate the texture of the surface is another way to influence it's appearance. Smooth finish to heavy broom finish, and varying levels of exposed aggregate allow for a wide array of textures.
  - 3. Construction and Control Joints, as related to concrete. The manipulation of construction or control joints in the surface of the concrete can create designs or patterns, even without the use of colors or unique textures. One example of this approach is to mimic flow patterns that may cascade over the structure, this would not only give the viewer a sense of what the function of the structure is, but also add some interesting detail to it as well.
  - 4. Stamps and Impressions, as related to concrete. The use of "stamps" to create interesting patterns and designs in concrete surfaces has become quite common as of late. These stamps can include any number of objects, including specially fabricated stamps, tire treads and custom made elements created of any number of materials. After the impression is made into the surface of the concrete, the stamp or stencil is removed, thus revealing the intended 'footprint" of the stamp. The features can include depictions of plant material, recreated petroglyphs, text or any other desired image.
  - 5. Gabions. The use of gabion baskets would limit access to the structure to foot-traffic only. Bikes and equestrians would not be able to safely traverse such surfaces. Gabions could however, be used at the area identified as the apron for the weir, which

is to act as erosion mitigation and to dissipate the energy in the overflow water from the EMF.

- 6. Geo-blocks. Similar in function as the gabion baskets, geo-blocks allow for water to spill over them, all the while maintaining the integrity of the underlying soil. Unfortunately, these blocks also posses similar issues as related to accessibility as gabions. Interesting patterns can be achieved through careful design and layout of such blocks.
- 7. Rocks and Boulders. Through the use of actual boulders and possibly artificial rocks, the weir could be given a natural appearance of outcroppings. This option may be quite nice during times of overflow from the EMF. During in-active flows, the boulders could be used by local rock climbing clubs or basic rock scrambling by hikers.
- 8. Introduction of Plant Materials. The possibility also exists to introduce some varieties of plant material. Of course any large massing of plants along the crest of the weir would be counter productive to its overall purpose, however, through careful planning and design, vegetation could be brought into this structure. This could be achieved through a few scenarios, one being the creation of high points, or "islands" that would provide enough high ground that could support plant material all the while keeping vegetation out of the path of flows. Other options include planting grasses and small shrubs that will also lend a natural look to the structure. Trees and large shrub material would likely be kept to the edges of the structure and on high points, thus eliminating the chance of catching debris and subsequently causing back-ups.

- Rip-rap areas. These areas will be located at primary and secondary inlets to the basin to prevent soil erosion caused by actively flowing runoff. Installation of rip-rap will be as per FCDMC standards, although the general shape and possibly the size of the cobbles can be influenced to create interesting elements, as well as maintaining it's intended purpose.
- Flood gates. The planned outlet from the basin, that would allow for limited re-flow back into the EMF would be achieved via floodgates installed near the southern edge of the site, adjacent to the EMF. These gates are purely utilitarian in design and function, and could possibly create a hazardous situation for site users. It is recommend that no aesthetic treatments be installed on this equipment.

### C. Landscaping.

The perimeter of the site, including areas along trails/paths and at vehicular access points to the site, will be landscaped in accordance with the approved Town of Gilbert plant palette (see below) and within the parameters of the FCDMC's Aesthetic Guidelines. The plants will be primarily desert adapted, low water use trees, with minimal use of shrubs, groundcovers and accent material. The plant selection will be coordinated with the Town of Gilbert's Parks and Recreation representatives. The ground plane will be seeded with native plant species; no turf will be installed as part of the FCDMC's construction activities. The plantings will be watered via truck watering or through a rudimentary automatic, underground drip system designed to be abandoned once the Town of Gilbert initiates work on the golf course.

Town of Gilbert Approved Plant List for Trees:

| Common Name: | Botanical Name: |
|---|---|
| Brazilian Pepper | Schinus terebinthifolius |
| Jacaranda | Jacaranda mimosifolia |
| Palo Brea | Cercidium praecox |
| Desert Mus. Palo Verde | Cercidium Hybrid 'Desert Museum' |
| Blue Palo Verde | Cercidium floridum |
| Native Mesquite | Prosopis velutina |
| Texas Honey Mesquite | Prosopis glandulosa |
| Fan-Tex Ash | Fraxinus velutina 'Rio Grande' |
| Mulga Acacia | Acacia aneura |
| Guajillo | Acacia berlandieri |
| Sweet Acacia | Acacia minuta |
| Southern Live Oak | Quercus virginiana |
| Cascalote | Caesalpinia cacalaco |
| Mexican Bird of Paradise | Caesalpinia mexicana |
| Lysiloma | Lysiloma thornberi |
| Texas Mtn. Laurel | Sophora secundiflora |
| Texas Ebony | Pithecellobium flexicaule |
| Mexican Fan Palm | Washingtonia robusta |
| Date Palm | Phoenix dactylifera |
| Sissoo Tree | Dalbergia sissoo |
| Tipu Tree | Tipuana tipu |
| Saguaro Cactus | Carnegiea gigantea |

### D. Erosion Control/Protection.

Soil erosion can occur at two distinct phases of development and operation of the Rittenhouse Basin. The initial period is during the construction phase. Because more than 5 acres will be disturbed by the basin construction, the work is regulated by the National Pollutant Discharge Elimination System (NPDES) regulations. The Maricopa County Drainage Design Manual (Volume III) identifies the steps necessary to be in compliance with the NPDES regulations and to reduce erosion from construction sites (i.e., preparation of a Stormwater Pollution Prevention Plan (SWPPP), use of Best Management Practices (BMP) and completion of Notice of Intent (NOI) and Notice of Termination (NOT) forms). The FCD drainage improvements will be designed and installed based on the procedures contained in Design Manual.

The second phase where erosion can occur at the site is during the operations phase. When the basin is filled or filling during storm events, sediment will be carried into the basin from off-site flows or spillage into the basin can cause localized erosion near the side-weir. During periods of inundation, water can also begin to erode the basin banks due to wave action. Suspended materials in the water can then be transported into the EMF during the draining process either through the pipe outlet or when water spills back over the side weir. While little can be done to eliminate the sediment import from off-site water, erosion created in the basin has been addressed. The stepped spillway associated with the side-weir has been designed to spread the flows entering the basin to reduce potential erosion of basin soils.

One form of erosion control is the planting of side slopes. The extent of plantings, and the sophistication of the associated irrigation system will depend largely on the anticipated lag time between the basin completion, and breaking ground for the golf course, there after. In the event that the golf course will be more than 5 years out, it is suggested that heavier plantings be installed, including the possibility of turf. In addition, a more complex irrigation system would also be recommended. Should the proposed golf course be scheduled for installation earlier than 5 years after completion of the basin, hydroseed would be recommended for those areas that are at risk to erosion. Only a rudimentary irrigation system would be installed at that time, with the anticipation that future plantings and permanent irrigation system would be installed along with construction of the golf course, thus resulting in the abandonment of the temporary irrigation system.

### E. Section 404.

An Individual Permit will be used to authorize construction of the Rittenhouse Basin. After discussions with FCD staff, it was determined that because of the hydraulic interrelationship of the Rittenhouse and Chandler Heights Basins and the need to construct drainage features within the EMF (which may be jurisdictional to the Corps of Engineers) during the Rittenhouse Basin activities, both basins will be permitted under a single permit. Alternatives analysis will be prepared by the consultant for use by the FCD staff. FCD will prepare the permit application. Compensatory mitigation required to offset lost habitat values will be addressed during the permitting phase. No costs have been assumed in the Engineers Estimate prepared for the project because of the uncertainty of the mitigation requirements.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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

EAST MARICOPA FLOODWAY
RITTENHOUSE BASIN DESIGN

PRELIMINARY NOT FOR CONSTRUCTION OR RECORDING

| | BY | DATE |
|---|---|---|
| DESIGNED | D. VIOLETTE | 10/01 |
| DRAWN | R. MALOTT | 10/01 |
| CHECKED | B. LING | 10/01 |

KIRKHAM MICHAEL

SHEET OF