EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS BASIN DESIGN
PREDESIGN STUDY

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

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FINAL REPORT
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The Chandler Heights Basin system will mitigate flows in the EMF by storing flows from Queen Creek (shown in this picture) and Sanokai Wash in a detention basin. The project also provides for potential for groundwater recharge. Recharge is occurring at the present time; this photo shows water in the channel disappearing into the streambed.

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INTRODUCTION

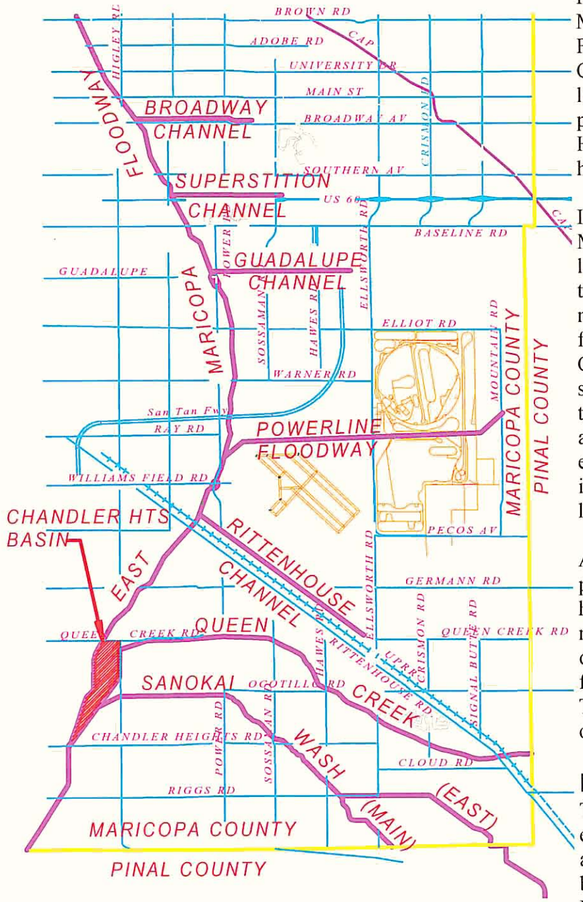


Figure 1. Map of the Chandler Heights Basin area.

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 Detention Basins and ultimately develop final construction plans. This report presents proposals for the Chandler Heights Basin, to be located north of Chandler Heights Road. A companion report presents similar information for the Rittenhouse 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 Chandler Heights 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 Chandler Heights Detention Basin is located in eastern Maricopa County within the Town of Gilbert and in unincorporated Maricopa County. The site is bounded by Higley Road to the east, Chandler Heights Road to the south, and by the EMF to the west. 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.

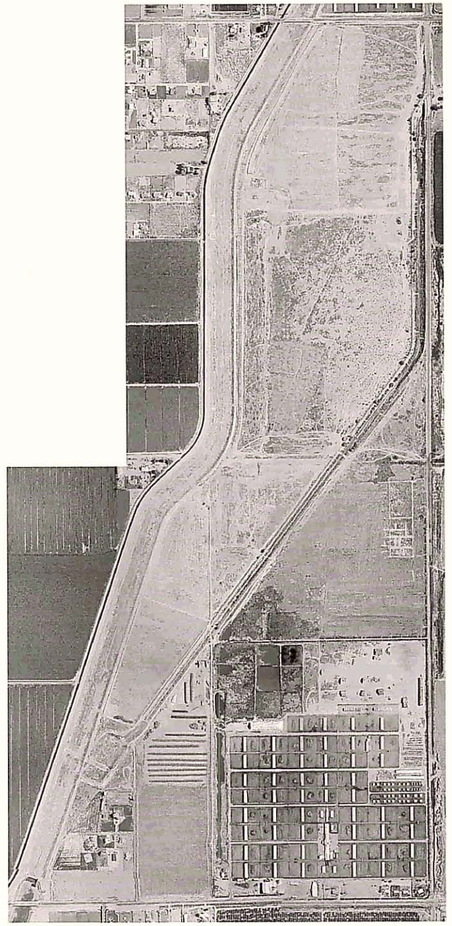


Figure 2. This aerial photo of the Chandler Heights Basin area was taken in June 2001. It shows the area where the Basin will be located as well as surrounding features.

Project design conditions

Two planning scenarios are managed by the FCDMC: existing conditions and full-development. In summary, flows in the full-development scenario are expected to be less than with the existing case as a result of development of the land and construction of associated on-site detention or retention facilities for each development. This project is designed for full-development conditions. This may result in higher flow conditions until more development occurs and the facilities may not reduce flows in the EMF to the extent provided by the design until that happens.

Multi-use opportunities

The site for the Chandler Heights Basin comprises approximately 250 acres—a very sizable area for flood detention. By comparison, common detention

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 south of the Queen Creek Road bridge. The Chandler Heights Basin will be constructed to the left of the left embankment shown here.

basins associated with specific developments are usually on the order of 1-20 acres.

Because the detention basin will often remain dry, the area is attractive for other compatible uses. However, while the design of the flood control facilities provide for and take into consideration these and possibly other uses, the resulting design and construction will only provide the flood control facilities themselves. Other parties will be responsible for design and construction of the other facilities consistent with design and operating criteria set forth here and consistent with operating agreements between those other parties. FCDMC funds will only be used for flood control facilities.

The Town of Gilbert recognized the potential for recreational development that could be built in and around the flood control basin, and the District is interested in working with the Town on this. The District would construct the flood control facilities and operate them. If a suitable interagency agreement is reached, the Town would probably construct any recreational facilities and operate them. Each party would be responsible for the design of their respective facilities.

The Roosevelt Water Conservancy District owns the existing Queen Creek Channel and plans to make improvements to it for groundwater recharge. The recharge project is compatible with the flood control project, and portions of our design anticipate the possibility of blending the two. Queen Creek is shown in Figures 4 and 5 below.

A regional trail system has been proposed that would incorporate a portion of the EMF into the system. The plan is to use the top of the east embankment of the EMF channel as part of the San Tan Regional Trail. This route covers approximately two miles in length between Chandler Heights Road to the south and Queen Creek Road to the north. The alternatives presented here include provisions that such a trail can be developed.

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 full development of the watershed and construction of selected planned flood control facilities as identified by FCDMC. The study recommended construction of two EMF detention basins; one in the vicinity of Rittenhouse Road and the other in the vicinity of Chandler Heights Road. This study and its hydrologic/hydraulic models provided the basis for the preliminary design of Rittenhouse Basin in this project.

Figure 4. A typical view of the Queen Creek channel. It has a bottom width of 50-70 feet, and much of it lies within raised levees, as the picture in Figure 5 shows.

Figure 5. This view shows the Queen Creek levee on the west side of the channel. The area to the left of the levee in this photo is in what will be the Chandler Heights Basin.

East Maricopa Floodway Capacity Mitigation Study Report, (Huitt-Zollars, 2000). This concept study developed and evaluated alternatives to resolve EMF conveyance capacity deficiencies and developed conceptual 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 capacity and flood control 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 (downstream to Hunt Highway) for the future watershed conditions from previous hydrologic study models and updated hydrology developed during the study. These future watershed conditions models served as the basis for the East Maricopa Floodway 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 of the channel. 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 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 study criteria and proposed alternatives were based upon the full development of the watershed according to local municipality General Plans and existing zoning boundaries. Future conditions hydrology from this study was incorporated into the EMF Capacity Mitigation Study by Huitt-Zollars.

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 Hydraulic Master Plan.

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 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.

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.

Figure 6. The EMF as seen from within the channel itself.

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.

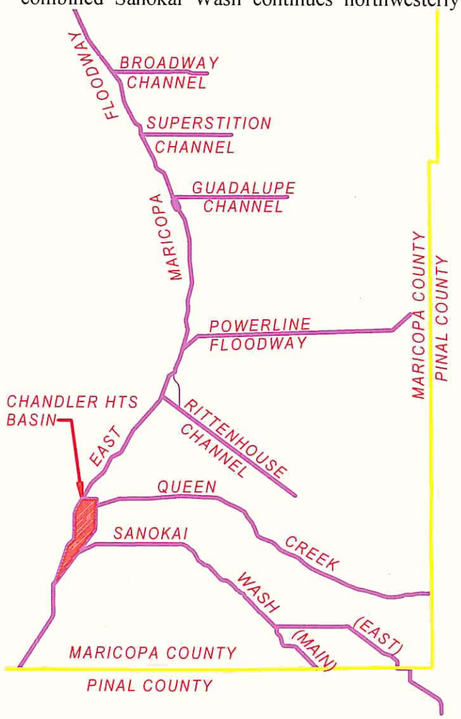


Figure 7. Map showing tributaries to the EMF in the Chandler Heights Basin area.

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 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. For the design hydrology, we modified the base hydrology to include the proposed Basin designs. Our work will define the changes to be made for both areas, and the base hydrology will be revised under this project.

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.

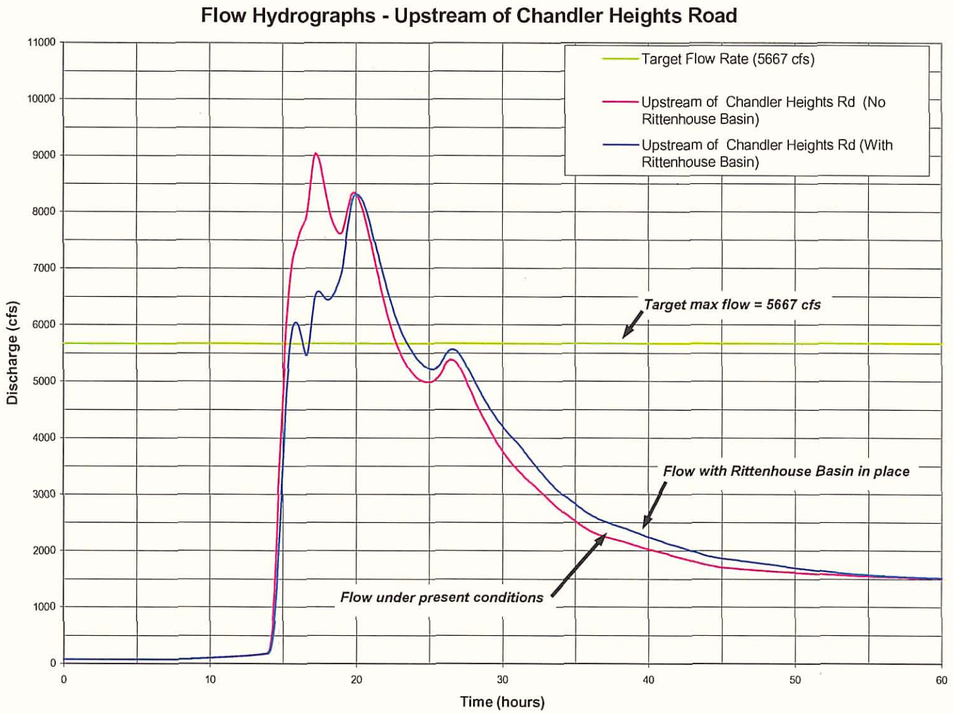


Figure 8. Flow hydrographs for future conditions at Chandler Heights Basin, with the current arrangement. The Target Discharge value is the design criterion for the EMF that applies to the Chandler Heights Basin system. The hydrograph for flow under the present conditions shows the situation in the EMF if nothing were done at either Rittenhouse Basin or Chandler Heights Basin. The hydrograph for the EMF above Chandler Heights Road with Rittenhouse Basin in place shows the effect of the Rittenhouse Basin and its related components.

Figure 9. Queen Creek flows into the project area from the left, crossing at the Higley Road Bridge. Queen Creek appears to have some flow most of the time, but it disappears into the stream bed within the first mile.

Central Arizona Project (CAP) Aqueduct Overchutes

The Salt-Gila CAP Aqueduct is a raised canal that interrupts natural westerly drainage of the EMF watershed. At several locations, pipe overchutes carry runoff across the aqueduct and contribute 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)

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.

Pilot studies were performed to compare the results of the two models and to determine how easy it was to prepare data and run the models. As reported in Rittenhouse Basin Design Report, it was determined that the two modeling systems gave similar

Figure 10. Queen Creek Road bridge crosses the EMF.

results. It was therefore decided to proceed using the HEC-RAS 3.0 model due to the ease of data preparation.

DESIGN HYDROLOGY

Hydrologic Impact of Rittenhouse Basin

As previous indicated, the hydrologic impact of the Rittenhouse Basin has a direct effect on the design of the Chandler Heights Basin. Therefore, it is necessary to determine, with a measure of assurance, the impact of the Rittenhouse Basin design on the EMF hydrology prior to initiating detailed alternative development and evaluation. Preliminary basin designs for the Rittenhouse Basin developed during the comparative modeling analysis (see FEQ vs. HEC-RAS 3.0) provided sufficient information to allow for preliminary alternative development for the Chandler Heights Basin. While the effectiveness and/or efficiency of some alternatives could also be assessed at this level, as the most efficient alternatives surfaced, it was necessary to have more accurate information on the hydrologic impact of the Rittenhouse Basin.

Development of design hydrology models in conjunction with unsteady-state hydraulic modeling

Generally, hydrologic and hydraulic modeling can be separated into distinct models such as HEC-1 for hydrology and HEC-2 and/or HEC/RAS for hydraulics. With the development and use of unsteady state hydraulic modeling, however, the distinction between hydraulic and hydrologic modeling is more difficult. For the design of the Rittenhouse and Chandler Heights Basins, the contributory EMF watershed is modeled using HEC-1, however, the hydrologic impact of the Rittenhouse and Chandler Heights Basins are determined in the unsteady state hydraulic analyses using HEC-RAS Version 3.0. The result is the development of hydrographs in the HEC-RAS models representing the hydrologic impact of the basins that are then imported or coded into the HEC-1 models in order to evaluate

Figure 11. Looking south from the northeast corner of what will be the Chandler Heights Basin. Higley Road is to the left.

To describe the process in more detail, HEC-1 hydrology models provided an input hydrograph, representing the EMF watershed upstream of the Rittenhouse Basin, for the Rittenhouse Basin HEC-RAS unsteady state hydraulic model. The Rittenhouse Basin HEC-RAS unsteady state analysis provided an output hydrograph representing the complete watershed upstream of the Rittenhouse Road including the impact of the Rittenhouse Basin. The output hydrograph was then hardcoded into a HEC-1 model for the EMF contributory watershed downstream of Rittenhouse Road.

For the Chandler Heights Basin, a similar process was performed. The hydrology for Queen Creek Wash and Sanokai Wash was determined using an HEC-1 model. The model provided hydrographs for both Queen Creek Wash and Sanokai Wash upstream of the Chandler Heights Basin that were then input into the HEC-RAS hydraulic model. In addition, the hydrograph from the HEC-1 model for the EMF upstream of the Chandler Heights Basin was input into the HEC-RAS model. The HEC-RAS unsteady state model used these hydrographs in the hydraulic analysis of the Chandler Heights Basin design and provided two output hydrographs which are then hardcoded into a HEC-1 model for the EMF contributory watershed downstream of the Chandler Heights Basin.

DESIGN HYDRAULICS

Development of design hydraulic models

To investigate detention basin alternatives, unsteady state analyses were performed to evaluate and size basin components. Because unsteady state analyses can easily become unstable in locations where the water surface elevation changes quickly, such as through bridges, culverts, weirs, and other hydraulic structures, only portions of previous HEC-RAS study models were used to develop the design hydraulic models rather than the complete HEC-RAS models for the EMF. The design hydraulic model boundary conditions and components included:

- A short reach of the EMF channel from north of Chandler Heights Road to south of Queen Creek Road
- The Chandler Heights Basin along the EMF, Queen Creek and Sanokai Wash
- A sideweir along the west bank of Queen Creek/ Sanokai Wash that is hydraulically connected with the detention basin
- A short reach of Queen Creek/Sanokai Wash from the EMF confluence to the Queen Creek/ Sanokai Wash confluence.
- A relocation of the drop structure in the EMF from just north of Chandler Heights Road to just north of the proposed Chandler Heights Basin primary outlet.
- Overflow spillways and basin outlets to the EMF
- The downstream boundary condition was set at normal depth using a friction slope of 0.00031 ft/ ft. This initial slope was selected to have non-erosive velocities of around 3 feet per second. We will conduct further study during design phase to determine "equilibrium slopes" for sediment transport in the channel.

Initial conditions

The hydrograph for the EMF at approximately Ocotillo Road from the EMF HEC-1 analysis was used as the initial flow data for the EMF in the HEC-RAS analysis. A hydrograph for Queen Creek/Sanokai Wash at their confluence was also used as initial flow data in the HEC-RAS analysis. As an initial condition, it is also assumed that the Chandler Heights Basin is empty prior to the start of the rainfall event.

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

Due to the large scale of the proposed detention basins, optimizing basin storage volume is important to minimize construction costs while still meeting the primary goal of flood attenuation and EMF capacity mitigation.

Provide for multiuse potential

Rather than provide a sterile detention basin solely for the use of flood control, the FCDMC and the Town of Gilbert desire a basin design that can provide multi-use opportunities such as recreational and/or recharge facilities. The design effort, therefore, has been coordinated with the FCDMC, the Town of Gilbert, the Roosevelt Water Conservation District and other stakeholders. While multi-use facilities are desirable, such facilities should not supplant the primary purpose of flood control. Information regarding coordination efforts are contained in Appendix F - Multiuse Planning and are described in the Alternatives Development section of this report.

DESIGN CRITERIA

Design criteria established by the FCDMC or established during the course of alternative development and analysis are identified and explained below.

Figure 12. 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.

Event size and frequency

The FCDMC established the 100-yr, 24-hr future watershed conditions as the design hydrology for the Rittenhouse and Chandler Heights Basins. The future watershed conditions hydrology is based upon the planned urban development of the watershed and includes capital improvement projects for flood control, retention and drainage facilities that FCMDC 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. It is provided as a factor of safety against overtopping that may reduce the structural stability of the embankment. Freeboard compensates for:

- Settlement of the embankment
- Flood levels higher than design
- Malfunction of outlet works
- Surface waves being generated to a height greater than the still water surface (see Provisions for Wave Action)

FCDMC does not have specific criteria for detention basin freeboard. The FCDMC defers to the standards of the community in which the basin is to be built, which is one foot of freeboard for the Town of Gilbert. That criterion may be suitable for smaller detention basins associated with residential and commercial developments, however, it was felt that one foot of freeboard was not prudent for a detention basin of this size.

It was decided to adopt the Arizona Department of Transportation (ADOT) unofficial policy regarding detention basin freeboard. ADOT uses the following rule-of-thumb for the design of detention basins:

- If feasible, provide all detention storage below the grade surrounding the basin site and provide one foot of freeboard
- If detention storage cannot be provide below the surrounding grade elevation, a minimum of three feet of freeboard should be provided.

Provisions for wave action

Sufficient freeboard should be provided to prevent erosion and overtopping of impoundment structures by wave action, a particular concern for large detention basins and dam structures. The provided freeboard should be sufficient to compensate for wave action, however, the freeboard need not be added to freeboard provided for other purposes described above. The most conservative estimate for freeboard based upon either situation is considered suitable.

The "Design of Small Dams" (DOSD) (US Dept of the Interior, Bureau of Reclamation, 1973) provides some guidance on the freeboard required for wave action. 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 (known as fetch), however; not just the height of waves is important. As waves approach a sloped face, water will run up the sloped face to a height greater than the wave height (known as uprush). Uprush is influenced by the depth of the water below the surface, the slope of the face being approached, and the roughness of the face material.

According to the DOSD, for a fetch of less than one mile a "Normal Freeboard" (based on 100 mph winds) of 4 feet and a "Minimum Freeboard" (based on 50 mph winds) of 3 feet is recommended for riprapped earthfill dams. This freeboard provides both protection against overtopping and protection against the effect of wave action.

For the Chandler Heights Basin, there are two situations for which freeboard should be provided:

- 1) embankment (levee) separates the basin from an adjacent channel (Figure 13)
- 2) the basin sideslope rises to natural ground (Figure 14)

In the first case, there is concern of overtopping the embankment, however, in the case of the Chandler Heights Basin, failure of the embankment would result in the impounded water being released to an adjacent flood control channel. Therefore, it is assumed that minimal factors of safety for freeboard are sufficient for this case. In the second case, there is no embankment to be weakened by overtopping, therefore, there is no concern of possible structural failure and, again, it is assumed that minimal factors of safety are sufficient. Therefore, we propose that a freeboard criteria of 3 feet be used for the Chandler Heights Basin to compensate for all factors. The actual provided freeboard is shown on the concept plan and typical sections in the Proposed Plan section of this report.

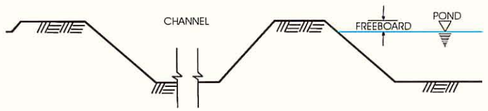


Figure 13. Freeboard can be in the lower end of the range where a flood control channel is on the other side of a pond embankment since overtopping is less likely to affect surrounding lands.

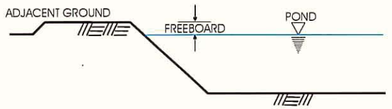


Figure 14. With adjacent ground levels at the same elevation as the pond embankment top, there is little concern over the effects of overtopping, so freeboard can be less.

Freeboard in channels

While the basis for the design of the Rittenhouse and Chandler Heights Basins is to attenuate flow in the EMF to meet target peak flow rates established by the FCDMC (see next section), the goal is 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

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

With the exception of areas through drop structures, it is expected that flows in the EMF, Queen Creek Wash and Sanokai Wash will be above critical depth (subcritical flow).

Target peak flow rates

FCDMC established, as the basis for the Rittenhouse and Chandler Heights Detention Basin designs, target instantaneous peak flow rates at three locations along the EMF:

Figure 15. Looking across the existing sedimentation basin for Queen Creek and Sanokai Wash. The entrance to the basin is in the left background.

Figure 16. Looking west at the north end of the proposed Chandler Heights Basin. Queen Creek Road is to the right.

- 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
- 8100 cfs just south of the Maricopa County Line.

Sediment capture

Previous studies have estimated the sediment load for the Queen Creek and Sanokai Wash systems, and these were updated in this study to reflect the current hydrologic modeling. This work is reported in detail in Appendix A - Hydrology and Hydraulics Report.

We analyzed the typical particle grain size found at various points in the existing sediment basin to get a better idea of what the system currently carries. More details of this sampling are found in Appendix B - Geotechnical Engineering Report. We used a particle size of 0.01 mm for our design, which means that the sediment basin should capture particles of this size or larger. The design criteria provide a volume of sufficient size and area that the design particle will settle within the basin rather than moving on through the outlet. We used a fall distance of 4 feet to determine the basin length needed. Our analysis showed that a basin must provide for at least 766,800 cubic feet of storage based on the 100-year, 24-hour event. Using a 6 foot storage depth, this results in a basin area of at least 2.9 acres. The resulting basin length will be approximately 600 feet.

Basin emptying

The detention basins should be drained as soon as possible after the passage of the rainfall event. Generally, the FCDMC requires that detention basins drain completely within 36 hours after the rainfall event to minimize standing water and vector control problems. Due to the size of the basin, the duration of flow released from the Rittenhouse Basin and the duration of flow in the EMF, Queen Creek and Sanokai Wash after the passage of the storm event, it may not be feasible to drain the Chandler Heights Basin within 36 hours. However, outlet facilities will be provided to drain the basin as soon as possible.

Landscaping and erosion control

While the FCDMC will not directly provide landscaping or other facilities not associated with necessary flood control features, the FCDMC will provide perimeter landscaping both to soften the visual impact of the flood control facilities and to help provide erosion control of slopes. However, it is in the best interests of the FCDMC, the Town of Gilbert and other stakeholders, that such landscaping and erosion control features be consistent with possible multi-use features. Details of planting criteria are described in Appendix F - Landscaping and Erosion Control, and are shown on the plans.

Basin side slopes

Based upon geotechnical investigations, the basin side slopes should be a maximum of 4:1 (H:V) (Appendix B - Geotechnical Engineering Report). This ratio will provide stable slopes that won't likely be affected by changing water depths in the basin. Flatter slopes may be desirable for landscaping, maintenance, or aesthetic purposes. If side slopes are to be grass or vegetated, the FCDMC prefers 5:1 slopes to facilitate mowing and maintenance. Maintenance access roads also require flatter slopes for vehicular traffic. Details concerning basin and embankment sideslopes and their uses are provided in Appendix F - Landscaping and Erosion Control.

Figure 17. Looking along the top of an existing levee forming Queen Creek. Levees should be avoided if at all possible because they present a greater risk to flooding if they should fail. Our design will place stored water below adjacent ground level to the greatest extent possible.

Figure 18. This photo shows the fine-grained particles that have previously settled out in the existing sedimentation basin. These particles were found about 300 feet from the basin inlet. Particles found closer to the inlet were typical of a larger size.

PROPOSED PROJECT

INTRODUCTION

While previous studies recommended conceptual plans to attenuate flow in the EMF using detention basins near Chandler Heights and Rittenhouse Road, no design details had been explored. In further developing the conceptual plan of detention basins at Rittenhouse Road and Chandler Heights Road, a number of different alternatives were developed and evaluated to identify a preferred alternative. In this section we discus the alternative development process we used and present an evaluation of various alternatives considered for the Chandler Heights Basin.

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 and Chandler Heights Basins, we reviewed the EMF hydrology to identify critical hydrologic components and to determine 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
- Rittenhouse Channel
- Queen Creek
- Sanokai Wash.

During the initial phase of alternative development, we evaluated a number of alternatives using the hydrology models to determine the effectiveness and efficiency of different scenarios. Alternatives included different basin designs (inline and offline), various basin combinations and sizes (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 reached the following conclusions relative to the Rittenhouse and Chandler Heights Basins:

- Neither a Rittenhouse Basin nor a Chandler Heights Basin alone can meet the target flows in the EMF. Both basins, therefore, must be utilized.
- Because the Rittenhouse Channel peaks earlier than the EMF at the confluence, storing flows from the Rittenhouse Channel is relatively ineffective in reducing peak flow in the EMF. The Rittenhouse Basin, therefore, should be dedicated to detaining flow directly from the EMF (see Rittenhouse Basin Predesign Study Report).
- The peak contributory flow from Queen Creek plus Sanokai Wash exceeds the target flow rate below Chandler Heights Road. Therefore, flow from Queen Creek and/or Sanokai Wash must be attenuated to meet the target flow rate downstream of Chandler Heights Road.
- While flow from Queen Creek and/or Sanokai Wash must be attenuated, diverting flow directly from the EMF into the Chandler Heights Basin is not a viable alternative and is not necessary. A Rittenhouse Basin will sufficiently reduce the peak flow and water surface elevation in the EMF, that further attenuation of the EMF is not necessary. Further, the low weir elevation needed would significantly reduce the amount of basin storage, EMF capacity and the Chandler Heights Basin would get wet during high frequency rainfall events. In addition, a Rittenhouse Basin would also "flatten" the EMF hydrograph so that while the peak flow is much lower, the duration of the highest flow rates are extended for a much longer period of time. Consequently, the amount of storage required to attenuate flow would increase due to the long duration of the peak flow rates.
- By meeting the target flow rate at Rittenhouse Road, there is sufficient capacity in the EMF downstream of Chandler Heights Road that some flow from Queen Creek and/or Sanokai Wash could bypass detention storage and flow directly into the EMF.

Figure 19. The Higley Road bridge crossing over Queen Creek was designed for future roadway widening. The present lanes use the eastern portion of the bridge, but room for a future lane is available to the west, as shown here.

Chandler Heights Basin Alternatives and preliminary evaluation

We investigated a number of preliminary alternatives and revisions of alternatives prior to selecting our preferred alternative. We developed alternatives to a level in which a qualitative evaluation could be made. The alternative was then either revised for further development or dropped from consideration.

A primary component of most alternatives was the concept of a detention basin "bypass" flow. We defined bypass flow as the amount of flow that can bypass detention storage unattenuated and enter the EMF without exceeding the target peak flow rate in the EMF at Chandler Heights Road. We set the minimum bypass flow equal to the difference between the target peak flow rate in the EMF at Chandler Heights Road and the peak flow rate in the EMF entering the Chandler Heights Basin area. The magnitude of the incoming EMF peak flow is dependent upon the Rittenhouse Basin design. However, we estimated prior to the final development of the Rittenhouse Basin alternative that the bypass flow could be approximately 2300 cfs. Actually, the bypass flow rate can increase as the incoming EMF flows decrease later in the storm.

Another component common to the basin alternatives is a sedimentation basin. We are concerned that inflows from Queen Creek and Sanokai Wash are likely to have significant sediment loads, and want to keep at least some of that load from entering the EMF. A sedimentation basin now exists just ahead of the confluence of the combined Queen Creek/ Sanokai Wash with the EMF.

Several of the preliminary alternatives are described here and show the progression of development of the preferred alternative. They are illustrated in figures on pages 9 and 10.

One factor that also entered into consideration of the alternatives is whether flows from all storms will enter the basin, or whether only flows from larger events will. In all cases except the recommended alternative most or all storm flows will enter some portion of the basin. This affects the availability of the basin for uses beyond flood control.

Queen Creek or Sanokai Wash Detention Basin Alternative.

In these alternatives we considered whether dedicating the Chandler Heights Basin to detaining flow from either Queen Creek or Sanokai Wash was feasible. Our initial investigations indicated such an alternative could not attenuate flows sufficiently to meet the EMF target flow rate at Chandler Heights Road when combined with the flows already present in the EMF. In addition, the volume of storage required significantly exceeded that for other alternatives. This alternative demonstrated to us that flows from both Queen Creek and Sanokai Wash must be handled. This alternative is illustrated in Figures 20 and 21.

Separate Queen Creek and Sanokai Wash Detention Basins Alternative.

We also considered an alternative that subdivided the Chandler Heights Basin into two separate basins, one handling Queen Creek flow and the other handling Sanokai Wash. This alternative initially appeared attractive in that it would eliminate the Queen Creek channel between the Higley Road bridge and the Sanokai Wash confluence. The expected future extension of Ocotillo Road would divide the Chandler Heights Basin area into two zones anyway. The arrangements are illustrated in Figure 22. Upon development, however, this alter-

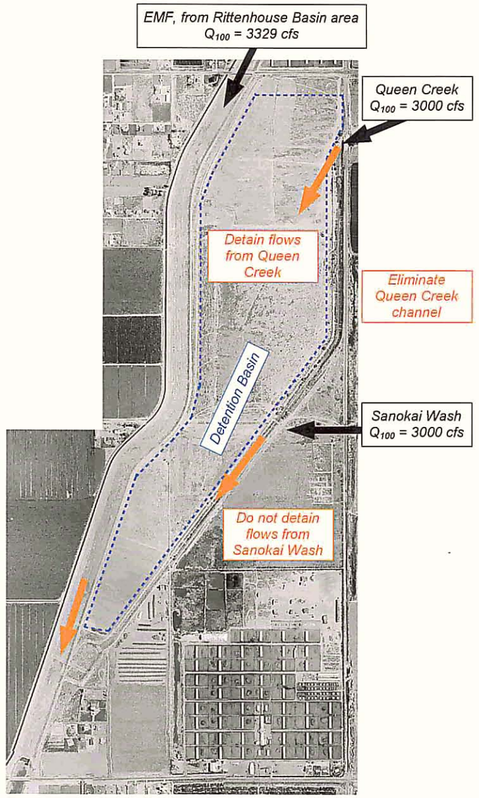


Figure 20. **Queen Creek Detention Basin Alternative.**

This figure shows detaining flows from Queen Creek but not Sanokai Wash.

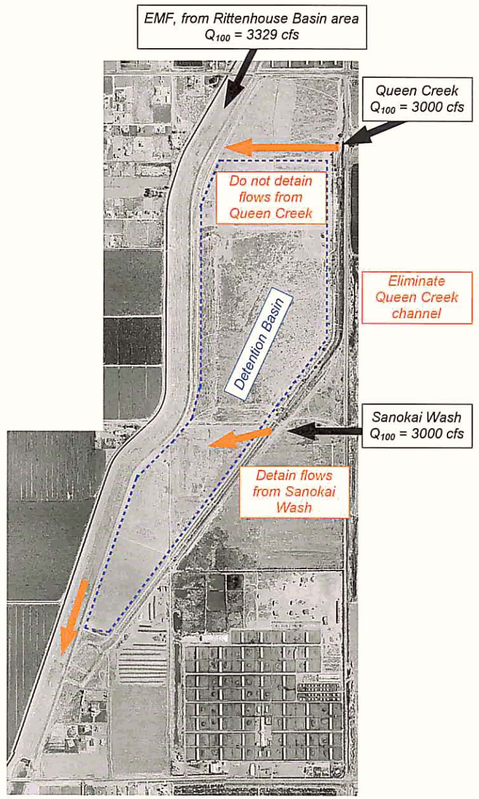


Figure 21. **Sanokai Wash Detention Basin Alternative.**

This figure shows detaining flows from Sanokai Wash but not Queen Creek.

Our initial investigations indicated that detaining flows from either Queen Creek or Sanokai Wash alone could not attenuate flows sufficiently to meet the EMF target flow rate at Chandler Heights Road when combined with the flows already present in the EMF. In addition, the volume of storage required significantly exceeded that for other alternatives. Further, in all of these alternatives even the smaller flows are sent to detention storage and thus the basins are not available for multiuse benefits. The basins would be wet for almost any rainfall event. This alternative demonstrated to us that flows from both Queen Creek and Sanokai Wash must be handled.

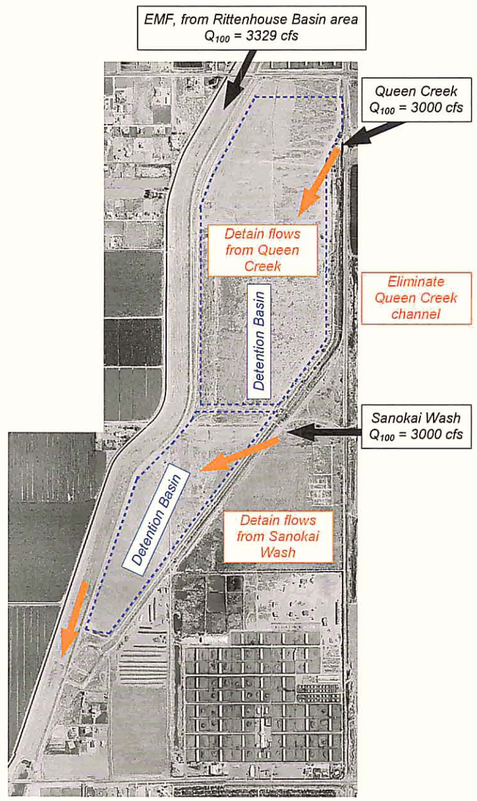


Figure 22. **Separate Queen Creek and Sanokai Wash Detention Basins Alternative.**

The possible benefits were outweighed by higher expected costs associated with duplicate inlet and outlet structures for each basin, the hydraulic inefficiency of detaining Queen Creek and Sanokai Wash separately, and the negative impact on possible multi-use opportunities. This alternative demonstrated that it was more efficient to handle the combined flow of Queen Creek and Sanokai Wash.

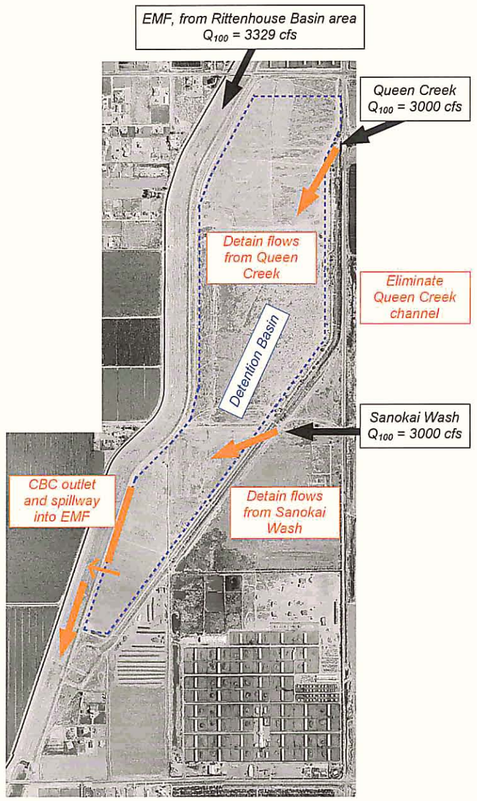


Figure 23. **Inline Chandler Heights Detention Basin Alternative.**

The simple concept and the expected costs savings made this an appealing alternative, however, an adequate basin and outlet configuration proved difficult to achieve during the preliminary analyses. The inability to effectively control the discharge from the outlet structure often resulted in either excessive amounts of detention storage or an excessive release of flow into the EMF that exceeded the target peak flow rate. Unable to develop an effective basin/outlet configuration based upon preliminary analyses, we dropped this alternative from further consideration.

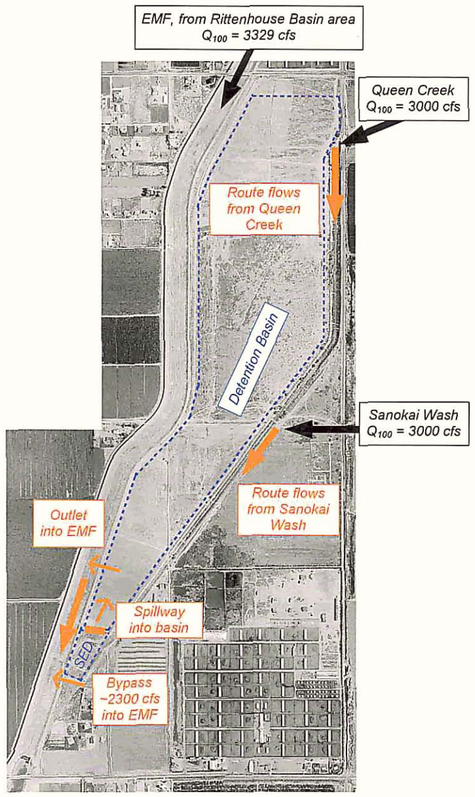


Figure 24. **Sedimentation Basin Weir Alternatives.**

This alternative was further developed to better estimate the length and size of the outlet/diversion structures. Upon further development, however, we found it difficult to control the amount of bypass flow through the CBC outlet due to changes in head. To minimize the fluctuation of head on the CBC outlet, we significantly increased the length of the diversion weir. Ultimately, this alternative appeared to offer a marginal reduction in weir length over a sideweir alternative.

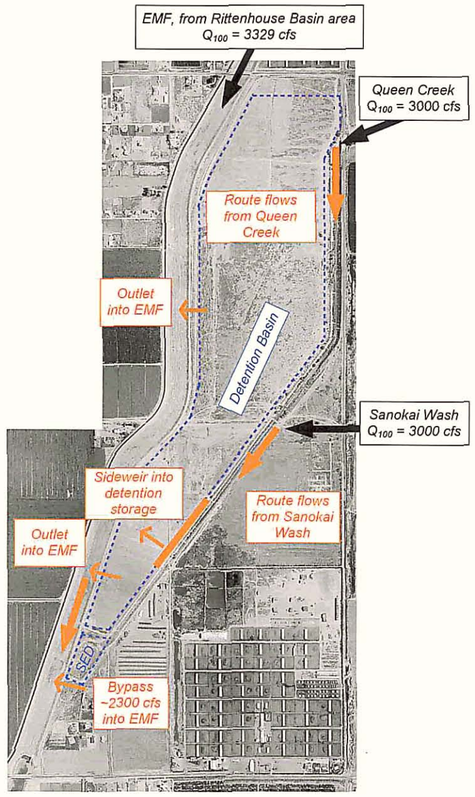


Figure 25. **Sideweir Alternatives.**

This alternative offered several advantages over other preliminary alternatives and was selected as the preferred alternative. Limiting the amount of bypass flow using a sideweir, and having a CBC outlet into the EMF, provides better control of the bypass flow. This decreases the likelihood that flow is not prematurely diverted into detention storage and reduces the probability of exceeding the maximum allowable bypass flow. A detailed description and discussion of this alternative is provided in the Preferred Alternative Section.

native was dropped from consideration. The possible benefits were outweighed by higher expected costs associated with duplicate inlet and outlet structures for each basin, the hydraulic inefficiency of detaining Queen Creek and Sanokai Wash separately, and the negative impact on possible multi-use opportunities. This alternative demonstrated that it was more efficient to handle the combined flow of Queen Creek and Sanokai Wash.

Inline Chandler Heights Detention Basin Alternative.

We considered providing a large inline detention basin for Queen Creek/Sanokai Wash and a outlet/spillway to the EMF, as shown in Figure 23, that would continually discharge into the EMF until the basin was completely drained. Unlike other alternatives that restricted basin discharge to a maximum of the "bypass" flow (~2300 cfs), the inline concept was to design a basin and outlet structure that would take advantage of the "excess bypass capacity" in the EMF prior to, and after, the passing of the peak flows in the EMF, thereby, minimizing basin storage and maximizing the drainage of the basin. The basin and outlet would essentially be configured to release as much flow as possible throughout the storm event provided the target peak flow rate was not exceeded. The simple concept and the expected costs savings made this an appealing alternative, however, an adequate basin and outlet configuration proved difficult to achieve during the preliminary analyses. The inability to effectively control the discharge from the outlet structure often resulted in either excessive amounts of detention storage or an excessive release of flow into the EMF that exceeded the target peak flow rate. Unable to develop an effective basin/outlet configuration based upon preliminary analyses, we dropped this alternative from further consideration.

Sedimentation Basin Weir Alternatives.

We developed several variations of this alternative, using different outlet/diversion structure configurations and mechanisms. We wanted to take advantage of the sedimentation basin to separate the "bypass flow" from flow to be diverted into detention storage. We expected this to minimize the detention diversion weir length and to provide a sedimentation basin for all Queen Creek and Sanokai Wash runoff. Several inventive outlet structures were considered, however, the most practical structure appeared to be a CBC outlet to the EMF for the bypass flow and a weir in the sedimentation basin to divert flow into detention storage. This alternative, illustrated in Figure 24, was further developed to better estimate the length and size of the outlet/diversion structures. Upon further development, however, we found it difficult to control the amount of bypass flow through the CBC outlet due to changes in head. To minimize the fluctuation of head on the CBC outlet, we significantly increased the length of the diversion weir. Ultimately, this alternative appeared to offer a marginal reduction in weir length over a sideweir alternative.

In addition, investigation into several existing residential structures and a proposed development immediately to the south of the Chandler Heights Basin revealed the finish floor elevations to be lower than the ground elevation around the existing sedimentation basin embankment. It was decided that alternatives that reduced the amount of flow and the water surface elevation at the existing sedimentation basin location would reduce the risk of potential flooding of existing and proposed developments. Therefore, this alternative was not developed further.

Sideweir Alternatives.

Variations of this alternative included separate sideweirs for Queen Creek and Sanokai Wash and a sideweir after the confluence of Queen Creek and Sanokai Wash. Separate sideweirs were determined to be inefficient and thus a single sideweir after the confluence of Queen Creek and Sanokai Wash was developed. The sideweir, as in the Rittenhouse Basin design, is designed to allow the bypass flow to continue past the sideweir and the detention basin, through the sedimentation basin and discharge unattenuated into the EMF though a CBC outlet. The weir elevation and length is set such that flow in excess of the bypass flow would be diverted into detention storage. This alternative offered several advantages over other preliminary alternatives and was selected as the preferred alternative. Limiting the amount of bypass flow using a sideweir, and having a CBC outlet into the EMF, provides better control of the bypass flow. This reduces the likelihood that flow is not prematurely diverted into detention storage and re-

duces the probability of exceeding the maximum allowable bypass flow. A detailed description and discussion of this alternative is provided in the Preferred Alternative Section, and the alternative is illustrated in Figure 25.

PREFERRED ALTERNATIVE

Description of proposed project

The proposed project is based upon the development of the sideweir alternative and consists of a 1647 acre-ft detention basin and a 1250 foot sideweir. The Queen Creek channel will be improved to contain the 100-yr event with freeboard and will vary from 200 feet to 70 feet in width. Four drop structures will allow the channel slope to be flattened so that velocities are lower in the channel. The two upper drop structures are likely to be not needed should the Roosevelt WCD undertake their planned changes to lower the Queen Creek channel.

A 1250 foot long sideweir will divert flow from an improved Queen Creek/Sanokai Wash channel into the detention basin. Flows in the channel below the sideweir will be conveyed into the sedimentation basin, and then discharged directly into the EMF. The height of the sideweir limits the amount of bypassed flow to the difference between the target flow at Chandler Heights Road bridge (5667 cfs) and the flow coming down the EMF from the Rittenhouse Basin area (3259 cfs). This approximately 2300 cfs bypass flow will pass through the sedimentation basin and be subject to sediment capture. With this arrangement "first flush" flows will probably be subject to sediment capture in the basin. Flows diverted into the detention basin will also be subject to sediment capture in that impoundment. The bypass flow will reach the EMF through thirteen 4'x4' box culverts. These will have flap gates that will prevent water from passing from the EMF into the sedimentation basin when water levels on the EMF side are higher than in the basin. These box culvert sizes are only preliminary, and the sizes will be optimized during final design.

The basic proposed arrangement is shown in more detail in Figures 34 through 46, at the end of this section. This provides a sideweir at elevation 1307.2, which is approximately eight feet above the Queen Creek channel bottom. The weir length is 1250 feet. The Basin will contain approximately 1647 acre-feet of water stored below the peak water surface elevation (1305.1 feet for the 100-year event, as determined in the model). This elevation is above the expected flow depth in Queen Creek 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 26 and 27.

Draining the Basin

FCDMC expressed a desire to avoid pumping for draining the basin if at all possible. While lowering

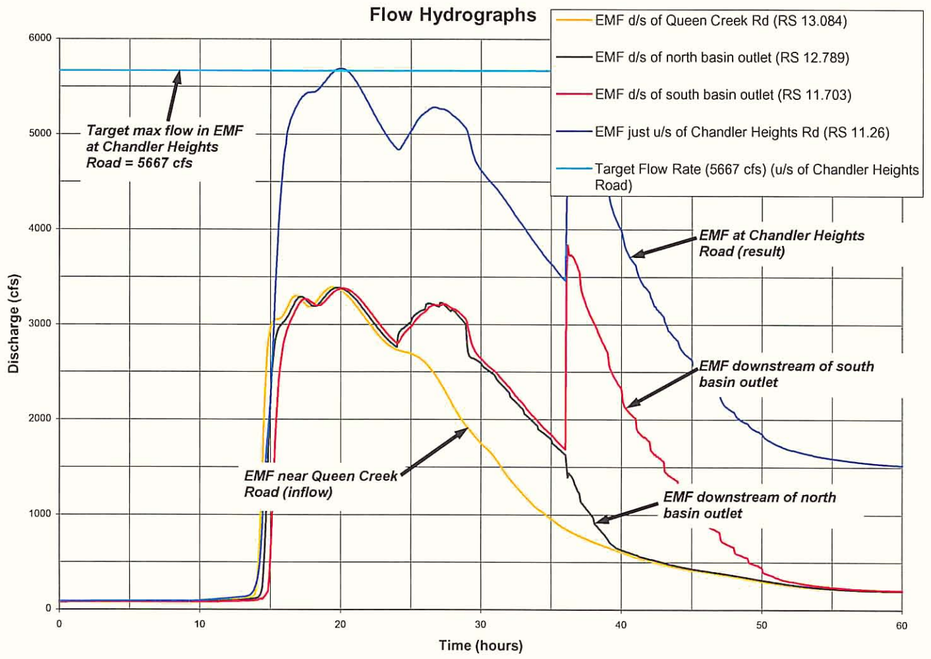


Figure 26. Flow hydrographs at Chandler Heights Basin . The hydrographs for the EMF at Queen Creek Road and for Queen Creek and Sanokai Wash represent inflows to the system. The Target Discharge value is the design criterion for the EMF that applies to the Chandler Heights Basin system. The hydrograph for the EMF near Chandler Heights Road includes the effect of the Chandler Heights Basin and its related components. Note: the sudden jump in flows in the EMF at approximately 36 hours results from opening the outlet gates at the lower outlet.

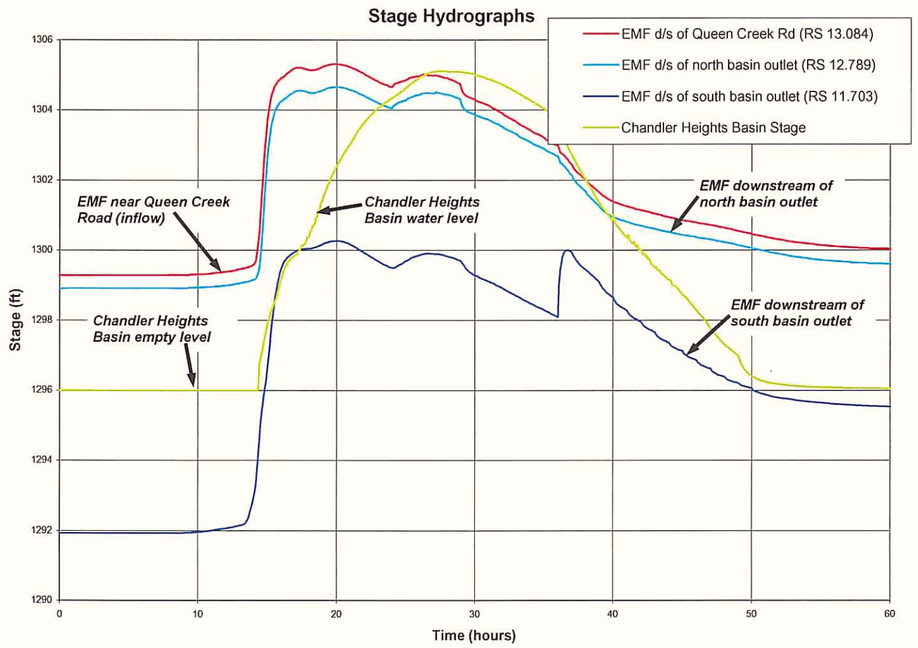


Figure 27. Stage hydrographs at Chandler Heights Basin system. The hydrograph for the EMF at Queen Creek Road represents inflow to the system. The hydrograph for downstream of the south basin outlet represents the results in the EMF due to the Chandler Heights Basin system. The hydrograph for Chandler Heights Basin water level shows changes in time for the pond. A sharp increase in water levels in the EMF downstream of the south outlet at around hour 36 is caused by opening those gates. The hydrograph for water levels within the Basin show it to be emptied before hour 60.

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.

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 two outlets and 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.

We decided on a gravity drain system for each outlet that would be made up of multiple box culvert sections. At the upstream outlet, 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. The relative water levels in the EMF and storage will not allow using flap gates for the downstream outlet. These will have to have operated slide gates.

Our analysis showed that a system of thirteen 4'x4' box culverts for the upstream outlet and twelve 4'x4' box culverts for the downstream outlet would provide enough capacity to drain the basin within the desired 36 hours following the storm event. This is illustrated in Figure 27. These box culvert sizes are only preliminary, and the sizes will be optimized during final design.

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

Sideweir seepage protection

In the area of the sideweir, we propose cutoff walls as an integral part of the weir structure and a crack-stopper barrier in other embankments. The 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.

The crack-stopper barrier system for the embankments outside of the sideweir will be designed during final design. See Appendix B — Geotechnical Engineering Report for more details.

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 sheetflow 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 Chandler Heights Basin from Queen Creek or Sanokai Wash 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 other recreation facilities should take this into account in planning for local drainage collection and disposal.

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 Queen Creek 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.

Subsidence and fissures

The preliminary analysis of the potential for fissures and for general settlement is described in detail in Appendix C - Subsidence and Fissures Evaluation. In summary, we found that there are not likely to be fissure action in the Chandler Heights Basin area that would be of concern.

There is a potential for differential subsidence, however. The subsidence is caused mostly be changes in groundwater levels and subsequent collapse of the soil structure. While in the past groundwater levels have sharply dropped, with recent focus on groundwater recharge these levels are recovering. For that reason, subsidence conditions that might have been present in the past may now be changing, making subsidence predictions difficult.

The evaluations completed for this report are not detailed enough to make specific design recommendations, but the report in Appendix C notes that a differential settlement in the range of 4-6 feet might be experienced over the length of the basin. The subsidence at the northern end is likely to be greater than subsidence in the southern end, hence the differential. Since the basin is more than 1 1/2 miles long, this amount of settlement is not great. However, for the EMF and the Basin, this amount of differential settlement could cause hydraulic problems. The total difference in elevation for the EMF channel bottom for the length of the proposed Chandler Heights Basin is approximately 4 feet, so a differential settlement of 4-6 feet could totally flatten or even reverse the EMF channel slope in that area. Our recommendation is to design as part of the basin project some monitoring points and propose a schedule of subsidence monitoring to be done by the Flood Control District in future. FCDMC could then anticipate problems should they begin to develop in time to be able to respond. A monitoring program is outlined in Appendix C — Subsidence and Fissures that has been recommended to FCDMC for other, similar structures.

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 piping. There is also the potential for burrowing animals to create paths for water to enter the embankment more quickly than by seepage.

Remedial measures to protect against piping include constructing an impervious core within the natural embankments that border on channels. This measure would be applied to the embankment along the EMF except where the sideweir is located, and the embankment along the Rittenhouse

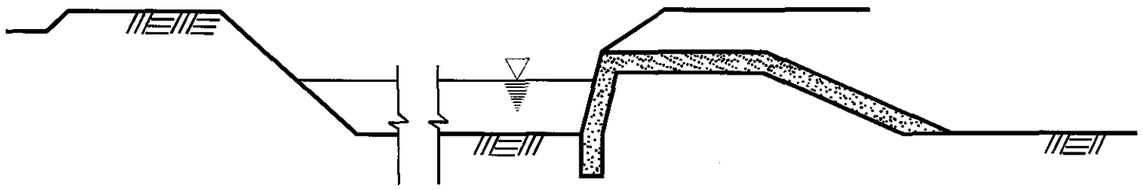


Figure 29. Flows in the Queen Creek channel below the sideweir crest level continue on downstream.

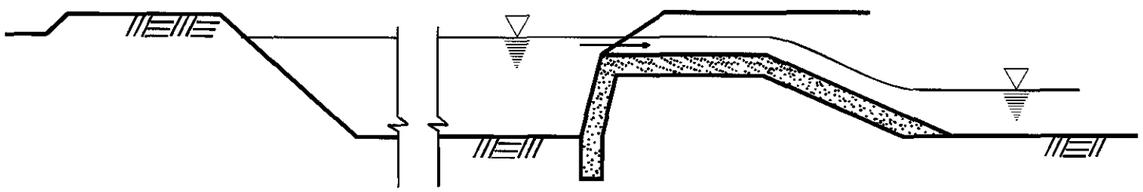


Figure 30. Once flows in Queen Creek pass the sideweir crest the excess is diverted into the storage basin. The bypass flows below the sideweir level continue down Queen Creek, through the sedimentation basin, and into the EMF.

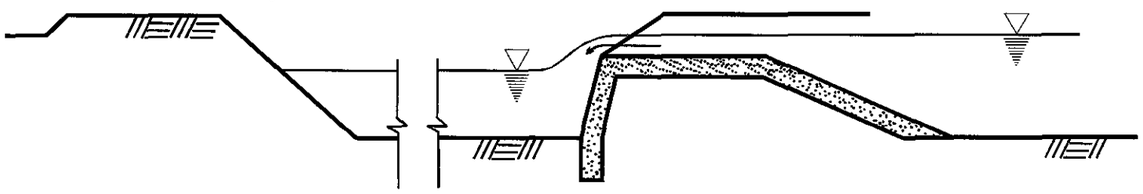


Figure 31. As flows in Queen Creek recede, stored water in the basin can initially pass back over the weir into Queen Creek.

Channel except next to the Kinder-Morgan property. The core wall is not needed at the sideweir because we provided structural cutoff walls with that structure. This core wall is described in more detail in Appendix G - Geotechnical Engineering Report.

Basin bottom

The basin bottom 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.

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 could flow back into the channel. A separate outlet is needed, however, to drain water stored below the weir crest elevation, but this outlet will discharge to the EMF rather than back to Queen Creek.

Several typical scenarios for the relative water levels between channel and storage are shown in Figures 29 through 31.

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 Queen Creek channel. 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 Queen Creek.

On the Queen Creek side, a face sloped at 3:1 is provided to create a sloped drop into the Queen Creek 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 Queen Creek 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.

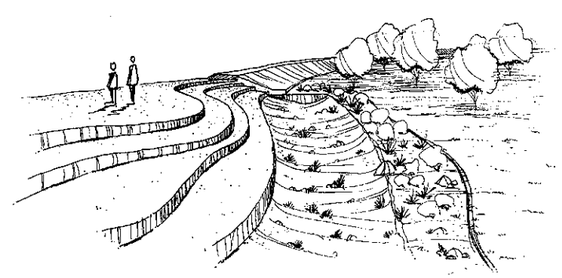


Figure 32. This sketch shows an approach using curved lines for the steps instead of straight edges.

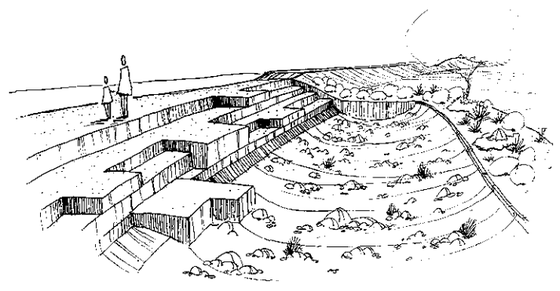


Figure 33. Steps can also be done in an irregular fashion instead of having continuous edges.

While the sideweir is 1250 feet long, it is only 4 feet high when viewed from the west (5 feet high when viewed from the Queen Creek side). From the basin bottom it will appear as a long, low structure which would probably be mostly screened with plantings associated with multiuse development. Viewed from normal ground elevations, such as along Higley Road, the structure will be almost a half mile away and would be hardly visible.

We feel the face along Queen Creek and the sideweir crest and roadway areas should be poured concrete. The face along the Queen Creek 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.

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.

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 future uses

The Town of Gilbert is evaluating development of recreational facilities that would coexist with the flood control facilities at Chandler Heights Basin. Our project team has worked with Gilbert officials to develop some potential recreational concepts that are compatible with flood control needs. An Illustrative Plan sketch and a Site Analysis sketch are shown at the end of this section.

QUEEN CREEK CHANNEL

The preferred plan provides for continued use of the Queen Creek Channel to convey flows from Queen Creek and Sanokai Wash into the system. While the existing channel performs essentially this same function, the Channel would be modified extensively under our plan.

Our hydraulic analyses show that the existing channel does not have the capacity to contain and convey the design flows. Accordingly, we propose widening the channel to a 100-foot bottom width from just downstream of the Higley Road bridge to the Ocotillo alignment. We also propose drop structures along this channel so that bottom slopes can be managed to keep flow velocities down. Our proposed bottom slope is 0.0003 feet per foot, similar to the EMF. Where Sanokai Wash flows join, the channel must have a 200-foot bottom width to provide the needed capacity. We proposed channel side slopes to be 4:1.

The existing Queen Creek channel is contained within raised levees, as shown in several photos in this report. We proposed to deepen the channel, and the height of raised levee will be greatly reduced in length and also in height. We expect that design flows will actually be contained within the portion of the channel that lies below adjacent grades. Thus levees where needed will only serve to extend freeboard.

The proposed sideweir will be built on the west side of the channel, below the Ocotillo Road alignment. The purpose of the sideweir, as explained earlier, is to restrict the amount of flow that continues in the channel to that permitted in bypass. The sideweir does this by conveying water higher than its crest to the side and into the detention basin. The depth of flow at sideweir crest provides just the conveyance needed for the bypass flow rate (approximately 2300 cfs). If the channel were to continue with the same bottom width, the water surface would drop as flow passed over the sideweir crest and the sideweir would be less efficient further along unless we sloped the sideweir crest along its length. Instead, we taper the channel bottom width from 200 feet to 70 feet for the length of the sideweir. Below the sideweir, the channel continues with a 70-foot bottom width and 4:1 side slopes until the entrance to the sedimentation basin.

In order to make Queen Creek channel wider than at present, the east side outside toe of slope will be kept within the property boundary. This will result in the channel to be located further west than at present. See the discussion below regarding the Roosevelt Water Conservation District for further details on this topic.

SANOKAI WASH ALIGNMENT

At present, Sanokai Wash is a largely undefined watercourse upstream from the Queen Creek channel. It has no defined bed, and is mostly defined by where it crosses north-south streets. However, development plans for the lands east and west of Higley Road call for changes to Sanokai Wash.

We have been told by officials of the Town of Gilbert that Sanokai Wash will be contained within a defined channel in this area. Though Sanokai Wash is now considered to run south of the Ocotillo Road alignment east and west of Higley Road, they plan to relocate it to the north of future Ocotillo Road. We expect to have further definition of this alignment at the confluence with Queen Creek so that we can incorporate it into our final design.

RECHARGE AND THE ROOSEVELT WATER CONSERVATION DISTRICT

The Roosevelt Water Conservation District (RWCD), based in Higley AZ, owns the Queen Creek channel in its present configuration. Their ownership extends both above and below the Higley Road bridge. RWCD is already planning to improve the channel in order to use it for intentional groundwater recharge. They will acquire the sources of water and convey it to their channel, where recharge would take place within the channel. Their plans also provide that storm waters will continue to be conveyed to the EMF, but that none of the introduced water would make its way to the EMF.

We have had preliminary discussions with RWCD officials and consultants, and the consensus seems to be that our proposed flood control improvements would be very similar to, and compatible with, their improvements. The intent is to try during final design to provide a project that will jointly meet the needs of both RWCD and FCDMC.

Some minor differences between the two plans is that RWCD would construct temporary berms across the channel to make small impoundments for recharge. These berms would be washed away during larger storm events and would be reconstructed after the storm had passed in order to restore the recharge facility.

Our design calls for permanent drop structures in order to manage channel bottom slopes. These permanent structures can replace the RWCD temporary berms providing that a different low-flow notch be provided in permanent drop structures. The normal notch would allow even the smallest flows to pass through the structure, though perhaps in a throttled manner. Having such a notch would not allow water to be impounded above the structure for recharge functions.

Our preliminary investigations on the feasibility of groundwater recharge in the Chandler Heights Basin area are reported in Appendix G - Groundwater Recharge Capabilities. The conclusions in that report show that recharge seems to be feasible with the soils present, and would not be impeded by groundwater since levels are low. Further evaluation may be needed during final design, depending upon whether the recharge project for RWCD is used or FCDMC does its own.

OCOTILLO ROAD

Ocotillo Road is not yet constructed in our project area, though officials for the Town of Gilbert have told us they plan to construct Ocotillo Road from Higley Road west and across the proposed Chandler Heights Basin. We have allowed for this by leaving an embankment in the Ocotillo Road alignment upon which the future road can be built. Our plans provide for two openings in this embankment, however. One opening will allow the Queen Creek channel to pass through, the other will allow the upper and lower parts of the detention basin to be connected. The opening for the Queen Creek channel will have to have a 200-foot bottom width if Sanokai Wash is to be located north of Ocotillo Road, or 100-foot bottom width if Sanokai Wash stays to the south. The opening connecting the basins must have a 200-foot bottom width to properly handle the flows between basin segments.

The width of our proposed embankment allows for the full right-of-way width needed by the Town of Gilbert for Ocotillo Road (130 feet). We have added top width to provide for constructing approach ramps to the future bridges; our initial design allows for an additional four feet of fill at these points, with 4:1 side slopes.

Bridges will be needed

Though not a part of the flood control project, several long bridges will be needed to carry Ocotillo Road.

- The crossing of Queen Creek will span 200-foot bottom width with Sanokai Wash to the north. The top width will be approximately 325 feet if the 4:1 side slopes are used.
- The crossing of the basin connection will also span a 200-foot bottom width, with a top width of approximately 300 feet using 4:1 side slopes.
- The crossing of the EMF will also span a 200-foot bottom width, and the existing top width outside the access roads on each side is approximately 340 feet.
- Immediately west of the EMF is the RWCD canal. The apparent top width to be spanned will be approximately 50 feet.

Double section corner monument

The south quarter corner for Section 15, Township 2 South, Range 6 East, Gila and Salt River Meridian lies within the EMF channel and on the Ocotillo Road alignment. Our surveyors found a discrepancy in that two different monument locations are described in different surveys, and various parcels of land have been defined in relation to one or the other monument. As described in Appendix D - Survey and Control, the difference in location of these two monuments is around 75 feet. It will not be the function of this project to resolve this difference, but we must account for it in planning the facilities. The most significant impact would be on the future location of Ocotillo Road, since the 75± foot different is in a north-south direction. FCDMC has proposed to compensate for this by adding width to the Ocotillo Road embankment to accommodate the offset.

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.

PREDESIGN COST ESTIMATE
CHANDLER HEIGHTS BASIN

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	AMOUNT
EARTHWORK				
Basin excavation (incl. haul and disposal)	CY	6,003,500	$3.75	$22,513,125
Queen Creek channel excavation (incl. haul and disposal)	CY	246,200	$3.75	$923,250
EMF channel - remove existing drop structure	CY	1,000	$15.00	$15,000
EMF channel excavation (to new drop structure)	CY	246,200	$3.75	$923,250
Structural excavation (incl. haul and disposal)	CY	18,300	$8.00	$146,400
Structure backfill (placed and compacted)	CY	14,200	$15.00	$213,000
EMBANKMENT				
Reconstruct embankment southeast of sed basin	CY	37,000	$15.00	$555,000
Core walls	LF	10,000	$265.00	$2,650,000
STRUCTURES				
Sideweir				
Sideweir (Queen Creek to storage)	LF	1,250	$2,100	$2,625,000
Wingwalls	CY	35	$300	$10,500
Toewall	CY	600	$300	$180,000
Outlets				
CBC, 13-4x4x105 (Bypass)	LF	81	$1,650	$133,650
CBC, 13-4x4x110 (North pond outlet)	LF	86	$1,650	$141,900
CBC, 12-4x4x100 (South pond outlet)	LF	76	$1,500	$114,000
Flap gates	EA	26	$5,000	$130,000
Slide gates	EA	12	$7,500	$90,000
Trash racks	EA	51	$4,000	$204,000
Toewalls	CY	80	$300	$24,000
Wingwalls	CY	40	$300	$12,000
Spillway				
Lower pond	LF	1,000	$700	$700,000
Sediment basin	LF	350	$700	$245,000
Drop structures				
Queen Creek channel	SF	15,000	$15	$225,000
EMF channel (new drop structure)	CY	1,000	$300	$300,000
Inlet to sediment basin	SY	180	$75	$13,500
Plunge pools				
Rip rap (at sideweir, 12")	CY	1,000	$35.00	$35,000
EMF at drop structure	CY	10,000	$35.00	$350,000
Miscellaneous				
Decomposed granite maintenance road	SY	58,100	$23.85	$1,385,685
RELOCATION AND REMOVALS				
Clearing and grubbing	AC	250	$150	$37,500
Miscellaneous removals	LS	1	$50,000	$50,000
LANDSCAPING AND EROSION CONTROL				
Temporary erosion control	LS	1	$750,000	$750,000
Hydroseeding	AC	324	$2,000	$648,000
Two-strand wire perimeter fence	LF	21,000	$10	$210,000
Irrigation system	LS	1	$485,000	$485,000
Trees (15 gal)	EA	1,700	$150	$255,000
Shrubs	EA	2,720	$20	$54,400
MISCELLANEOUS CONSTRUCTION ITEMS				
Mobilization/demobilization	LS	1	$300,000	$300,000
Survey and construction staking	LS	1	$300,000	$300,000
Construction, materials and quality control testing	LS	1	$440,000	$440,000
SUB-TOTAL CONSTRUCTION COSTS				**$38,388,160**
ENGINEERING (5%)				$1,919,408
CONTINGENCY (30%)				$11,516,448
TOTAL ESTIMATED COST				**$51,824,016**

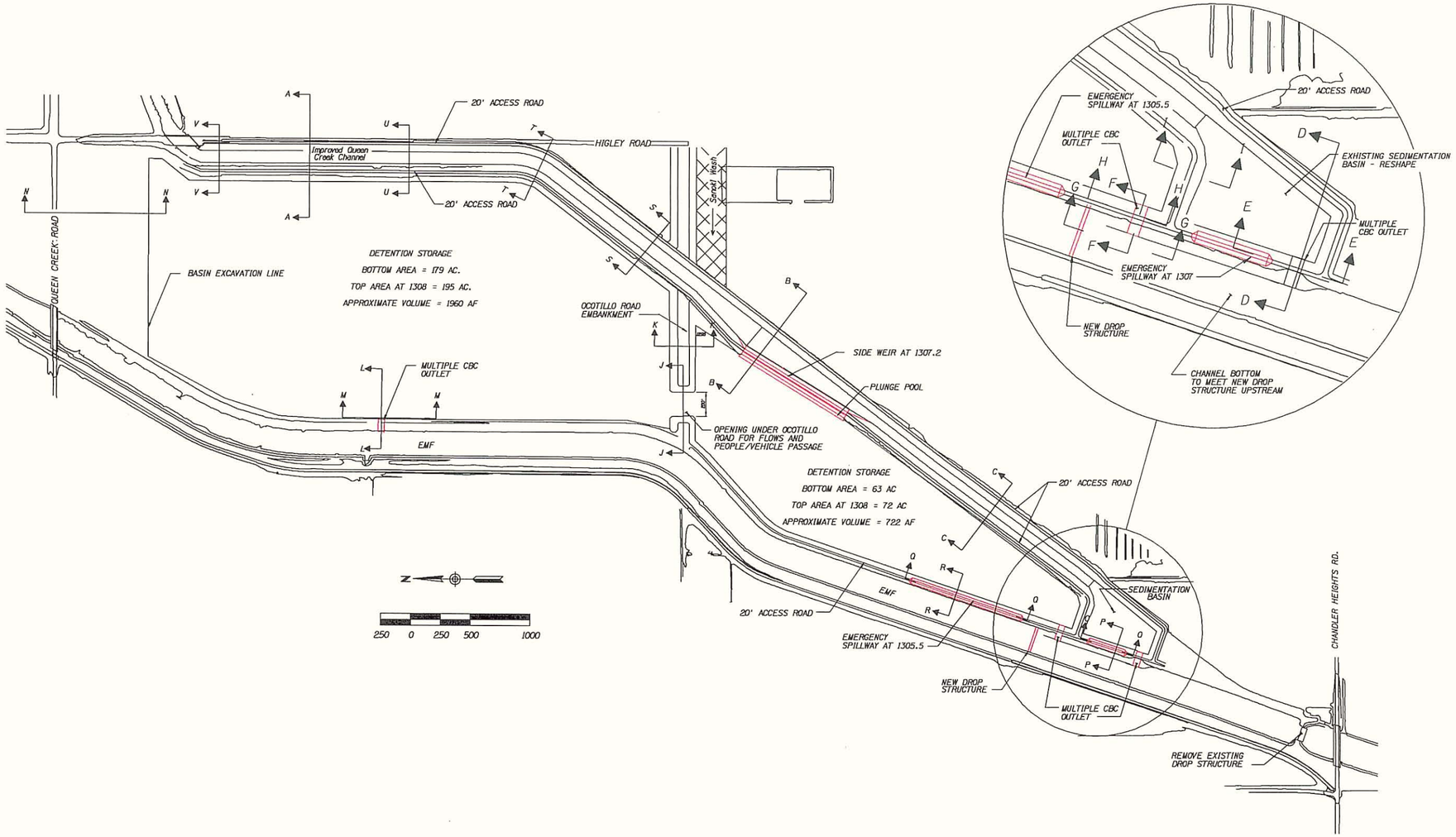


Figure 34. Chandler Heights Basin concept plan. See also the larger scale plan in the pocket at the rear of this report.. This plan shows the general arrangements for the preferred plan, and shows where typical sections are cut. Those typical sections are shown on the pages that follow. Note that these typical sections are not the modeled cross sections but simply show typical arrangements.

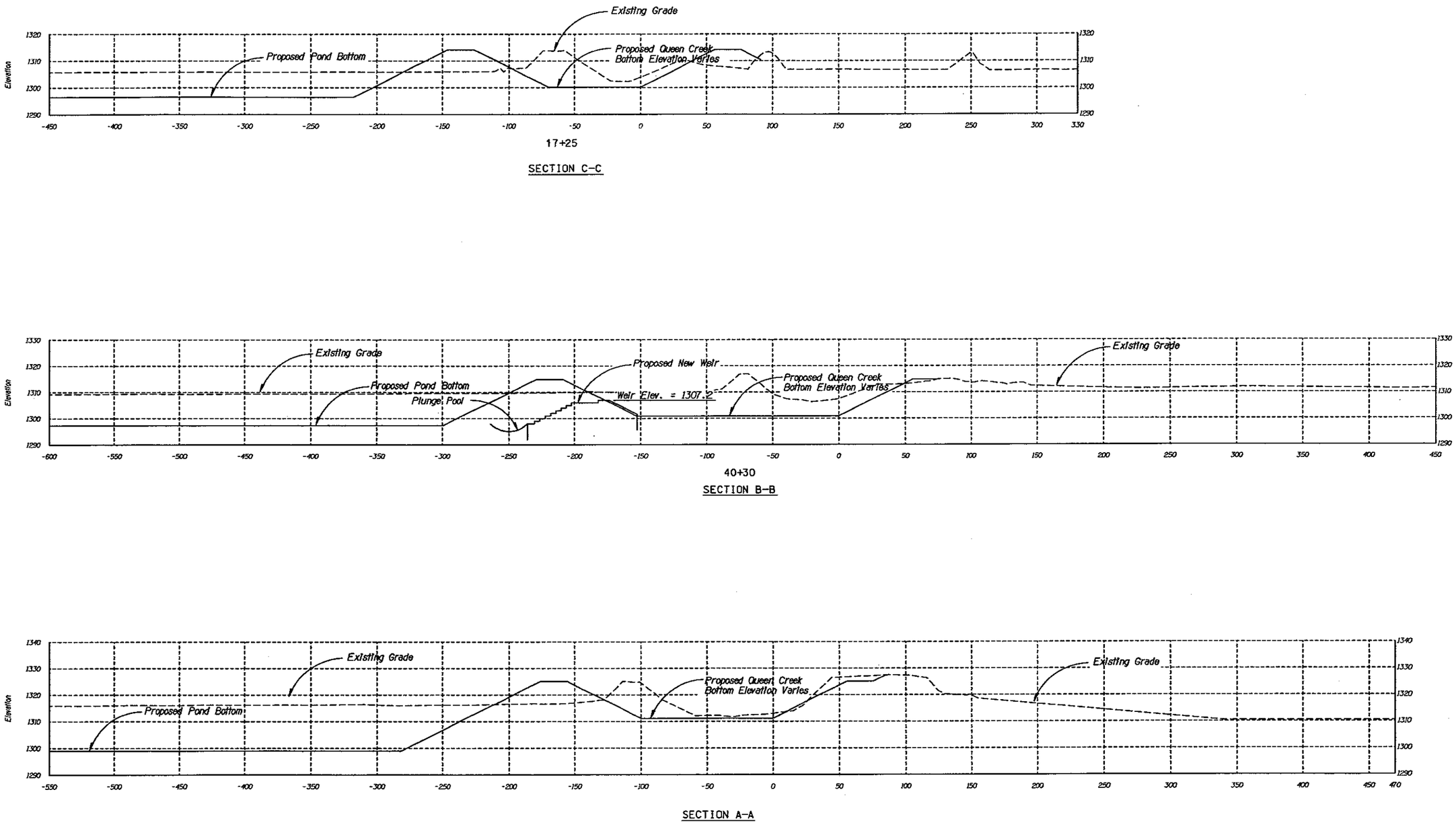
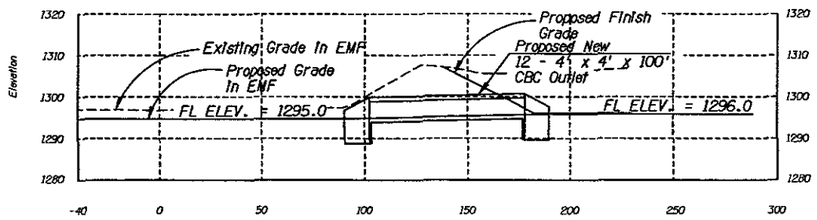
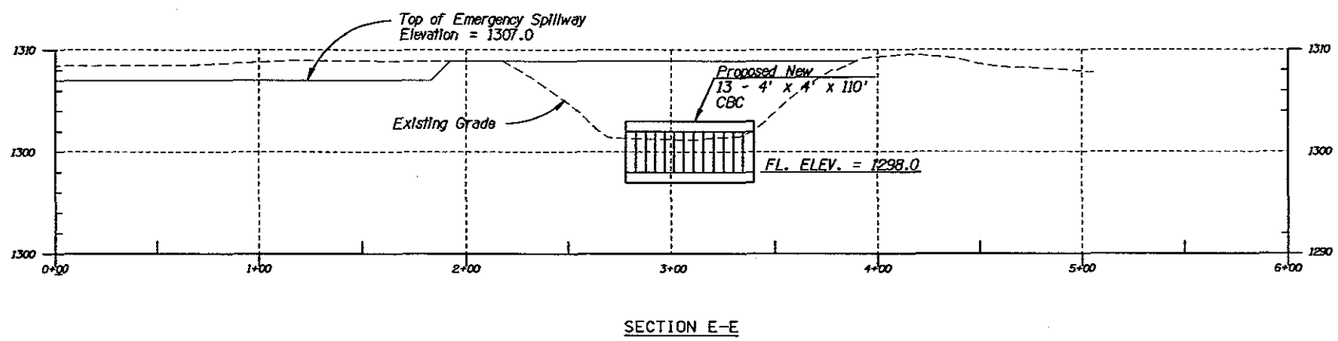


Figure 35. Chandler Heights Basin typical sections.





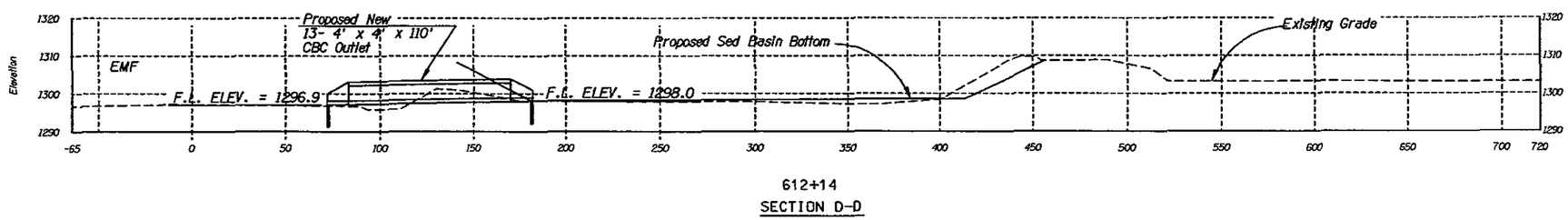
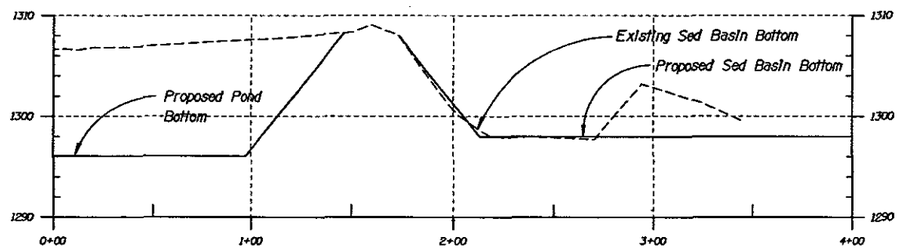
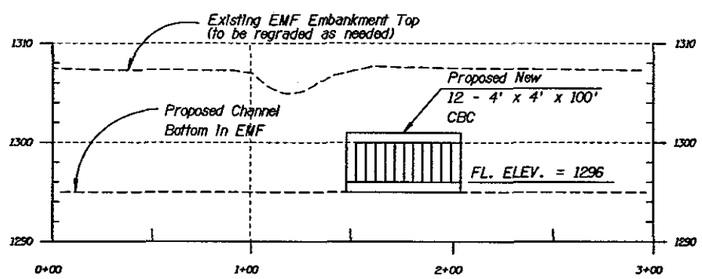


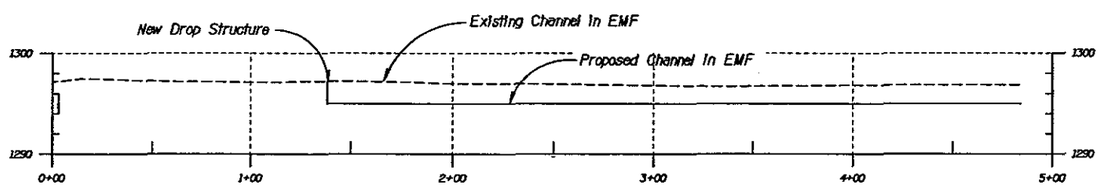
Figure 36. Chandler Heights Basin typical sections.



SECTION I-I

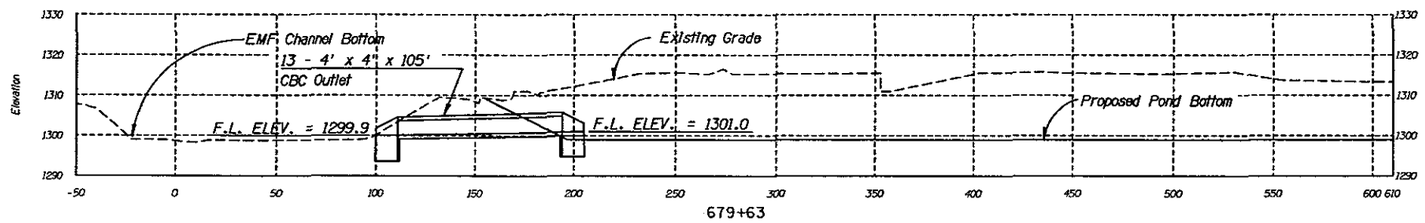


SECTION H-H



SECTION G-G

Figure 37. Chandler Heights Basin typical sections.



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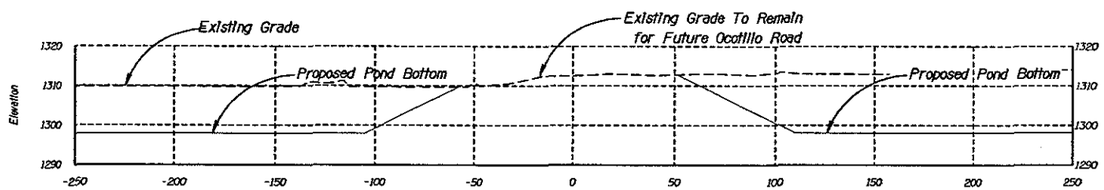
SECTION L-L



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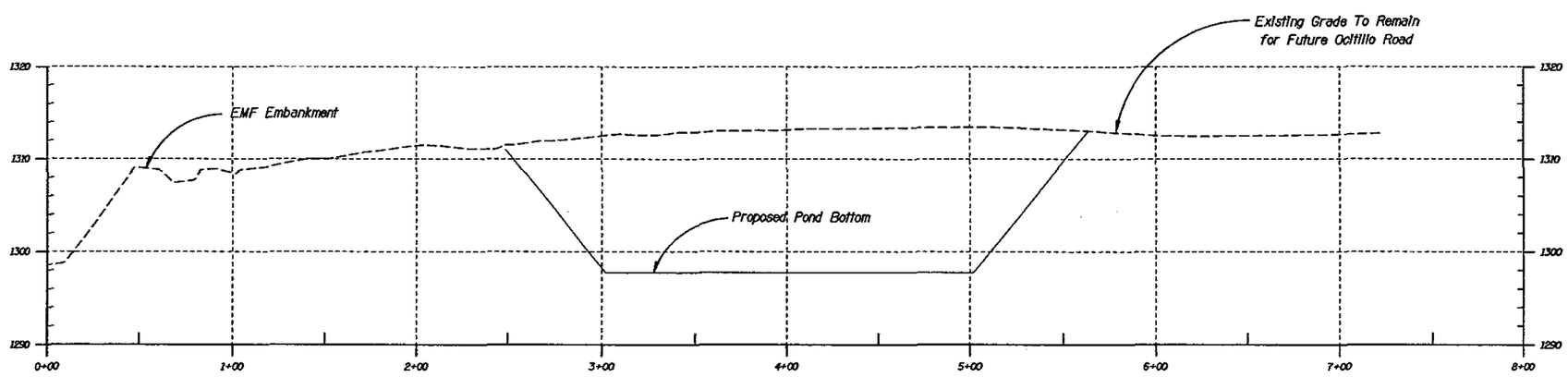
SECTION K-K



SECTION J-J

Figure 38 Chandler Heights Basin typical sections.

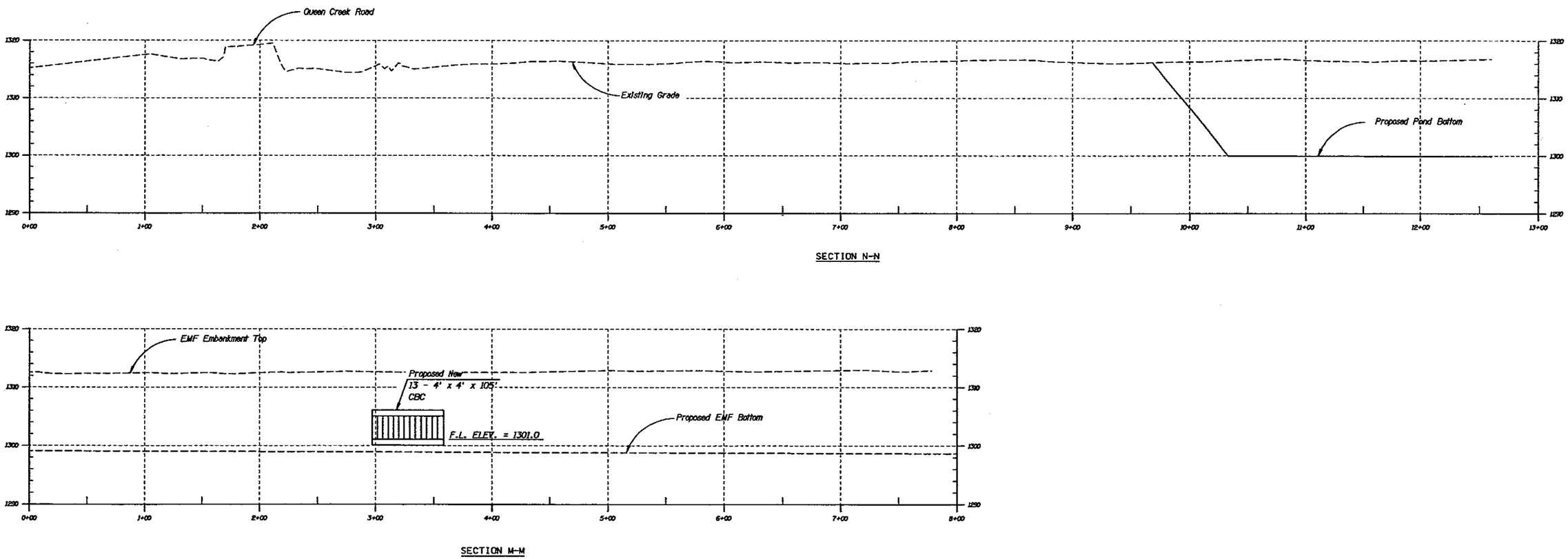
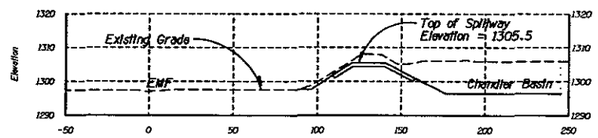
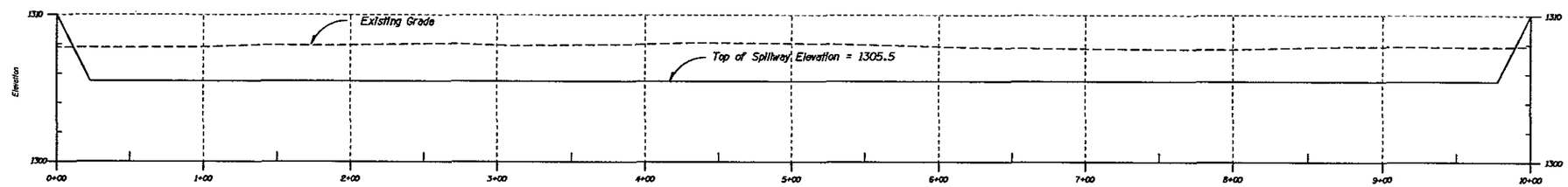


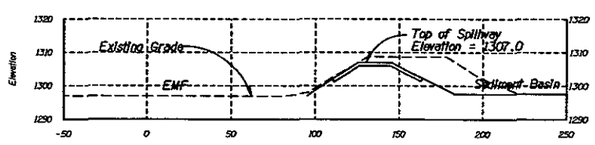
Figure 39 Chandler Heights Basin typical sections.



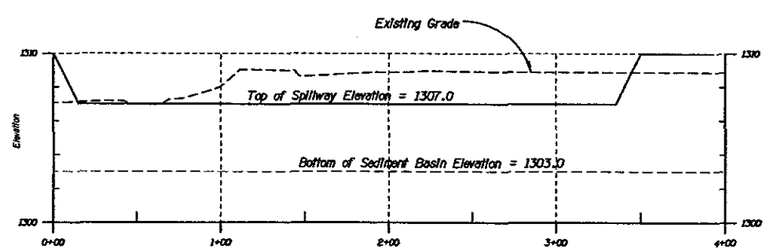
SECTION R-R



SECTION Q-Q
(Spillway at Lower Basin)

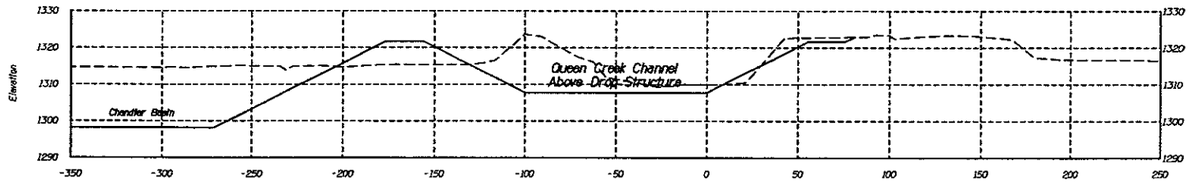


SECTION P-P

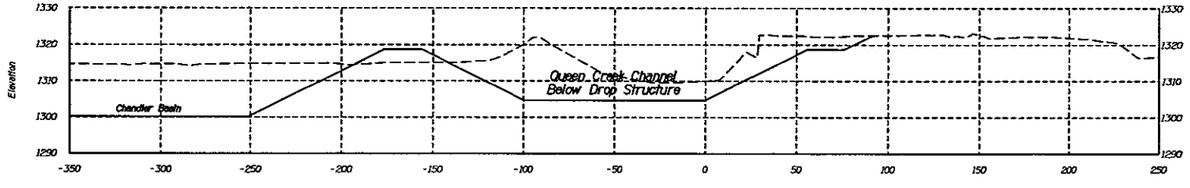


SECTION O-O
(Spillway at Sedimentation Basin)

Figure 40. Chandler Heights Basin typical sections.



68+00
SECTION T-T
(Above Drop Structure)



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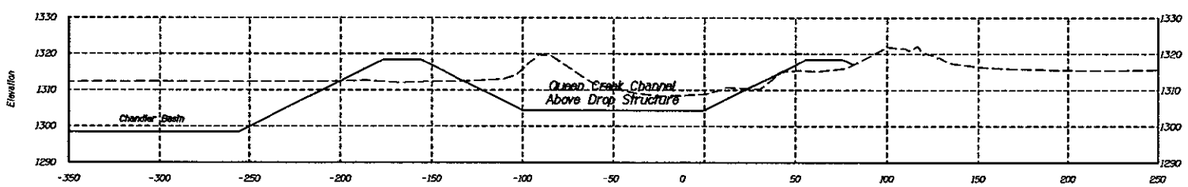
(Below Drop Structure)



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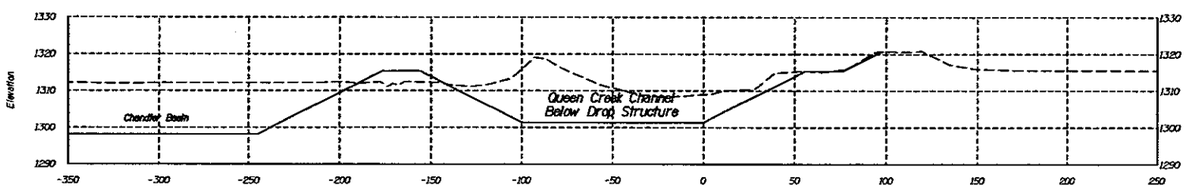
SECTION S-S
(Above Drop Structure)



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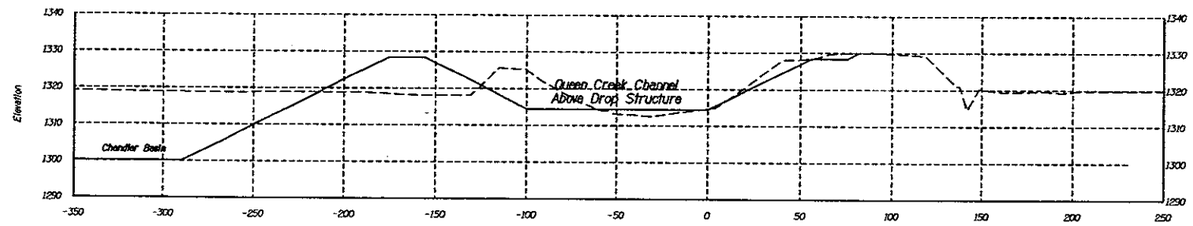
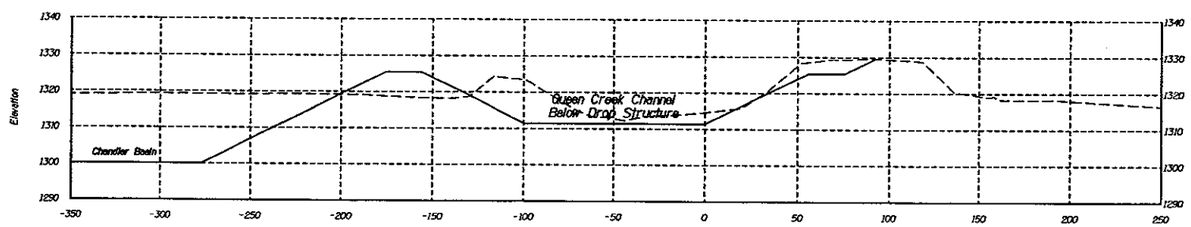
SECTION S-S
(Below Drop Structure)

Figure 41. Chandler Heights Basin typical sections.



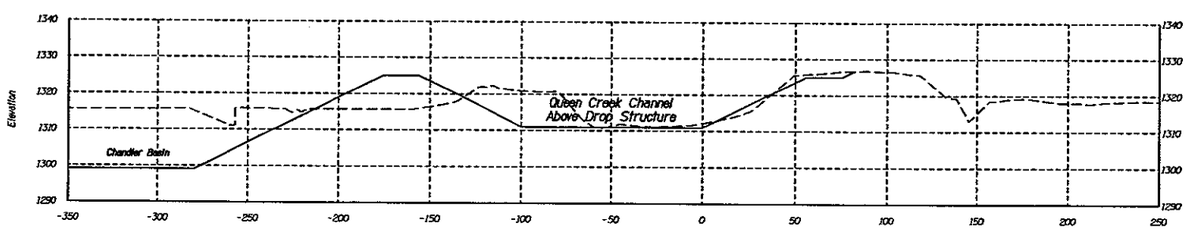
95+00

SECTION V-V
(Above Drop Structure)



94+00

SECTION V-V
(Below Drop Structure)



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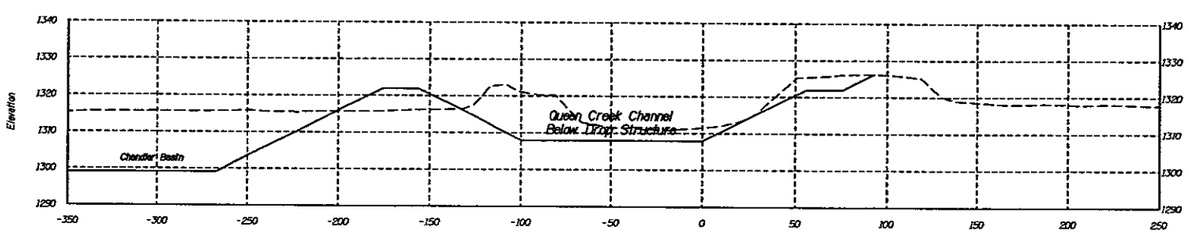
SECTION U-U
(Above Drop Structure)



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SECTION U-U
(Below Drop Structure)

Figure 42. Chandler Heights Basin typical sections.

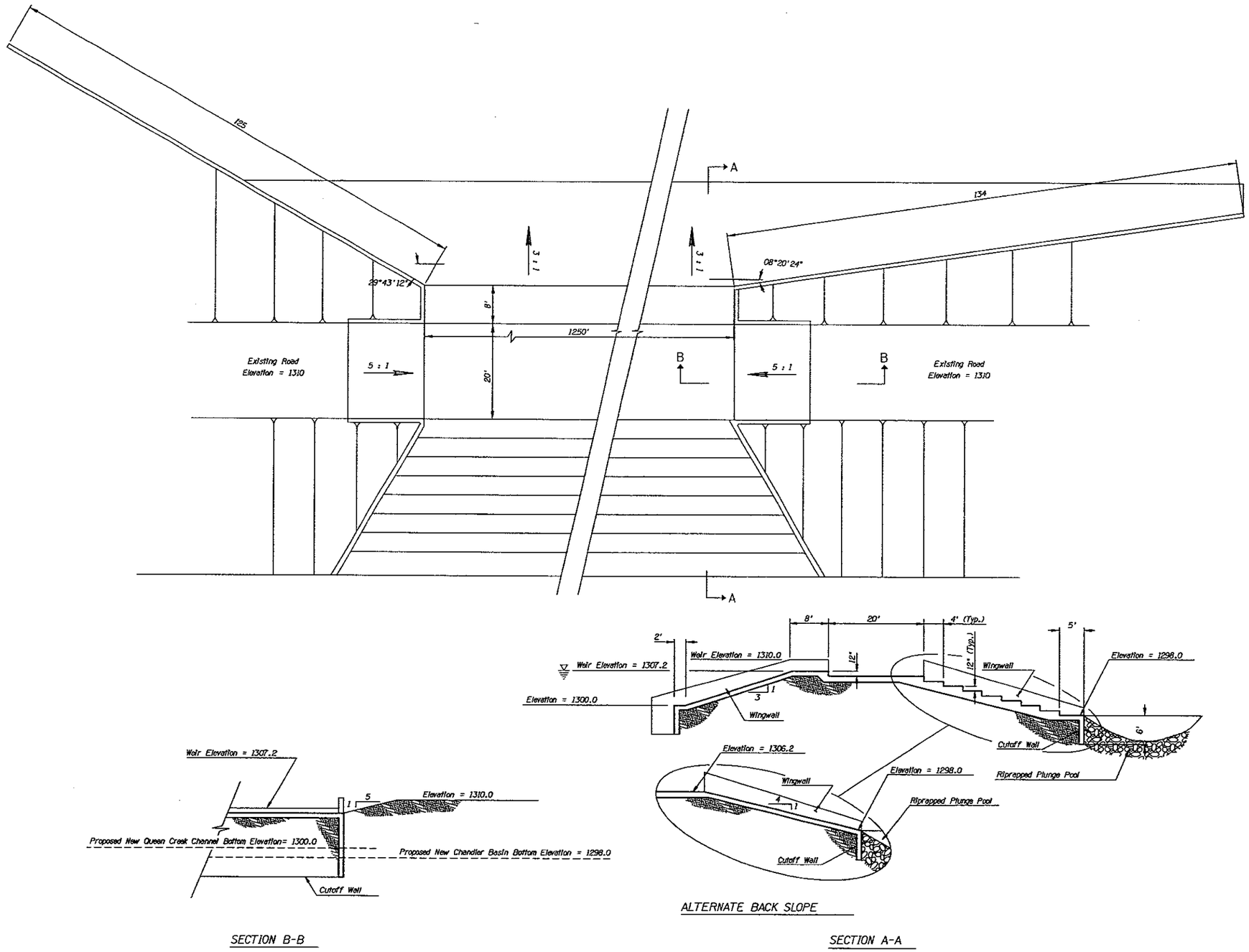

Figure 43. Chandler Heights Basin typical sideweir details.

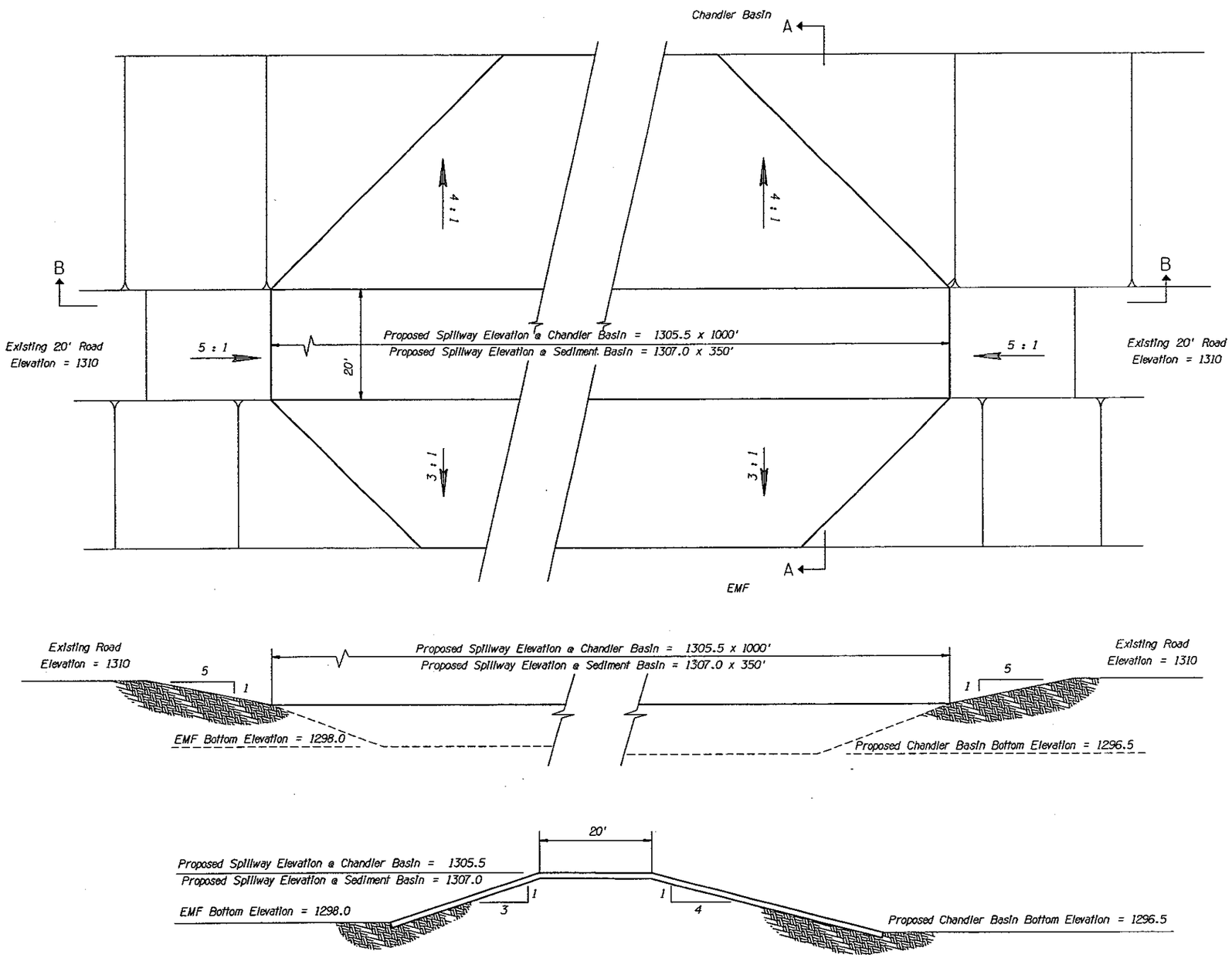

Figure 44. Chandler Heights Basin typical spillway details.

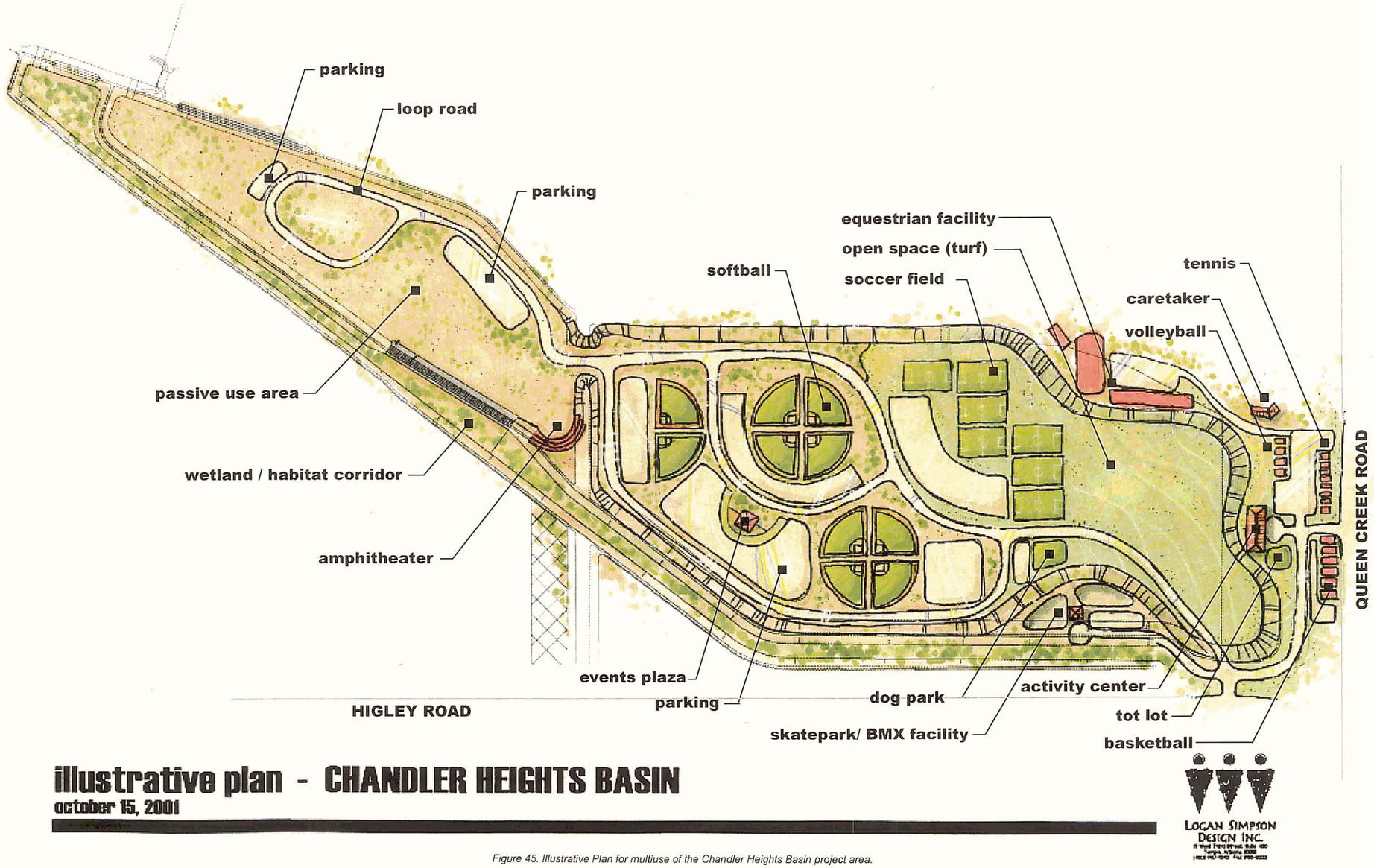

Figure 45. Illustrative Plan for multiuse of the Chandler Heights Basin project area.

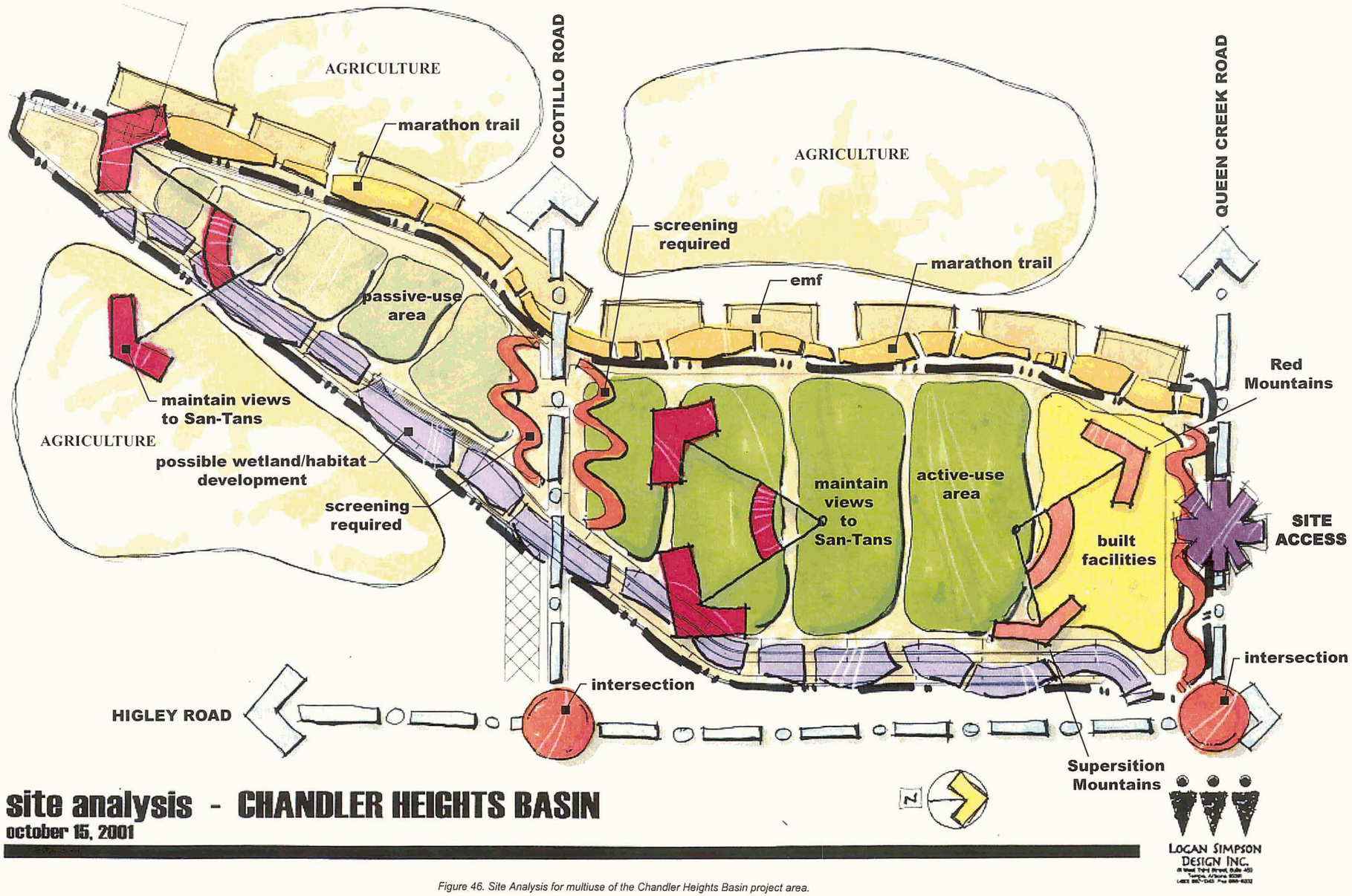

Figure 46. Site Analysis for multiuse of the Chandler Heights Basin project area.

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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

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APPENDIX G: GROUNDWATER RECHARGE CAPABILITIES

HYDROLOGY AND HYDRAULICS REPORT

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EAST MARICOPA FLOODWAY

Hydraulic Report

Chandler Heights Basin

January 8, 2002

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1. DESIGN PARAMETERS AND BASIN MODELING APPROACH

1.1. Design Goals and Parameters

The objective of the hydraulic design is to limit the total flow in the East Maricopa Floodway (EMF) to approximately 5670 cfs by intercepting flows from the combined Queen Creek and Sanokai Wash in the Chandler Heights basin. The flow in the EMF approaching from downstream of the Rittenhouse Channel is 3400 cfs. This limits the flow from the Chandler Heights basin to approximately 2300 cfs when the EMF approach flow peaks at 3400 cfs. This is accomplished by using a lateral weir in the combined flow channel that is designed to limit the peak bypass flow to 2300 cfs. The diverted flows are stored in the detention basin, which is to drain within 36 hours after the end of the storm. A sediment basin is included downstream of the bypass flow channel to capture sediment in excess of 0.01 mm.

1.2. HEC-RAS 3.0 Unsteady Flow Model

Due to the complex hydraulic interaction among the elements of the detention basin, unsteady flow is the method used to analyze the basin behavior. This results in a final hydrograph that describes the interaction between the EMF approach flow, the Queen Creek/Sanokai Wash bypass flow, and the discharges from the north and south outlets from the Chandler Heights basin.

The US Army Corps of Engineers computer program, HEC-RAS Version 3.0, is used to calculate water surface profiles for both steady and unsteady flow. The program uses an unsteady flow equation solver adapted from Dr. Robert L. Barlauu's UNET model. The unsteady flow simulation can be used to model one-dimensional unsteady flow through complicated open channel systems with split flow, in-line weirs, side weirs, and other hydraulic structures.

Modeling of flows through culverts with flap gates or manually operated gates and over side weirs requires adjustment of the modeling methods used in HEC-RAS. Side weir flow is modeled by HEC-RAS using the same equation as for regular weir flow. Only one discharge coefficient is allowed for each weir used. The coefficient for side weir flow used in the analysis is derived by application of an adjustment to the inline weir flow coefficient. Flow through a culvert with flap gate or manual operated gate is not available as an option in HEC-RAS; it is modeled as a sluice gate for which the opening is varied over time to match the discharge through a culvert with a gate computed external to HEC-RAS.

HEC-RAS does occasionally demonstrate instability when drop structures are encountered, affecting how the models are developed. Some special modeling techniques were used to avoid the instability, which will be discussed in the following sections.

2. HYDROLOGY MODEL

2.1. Base Model

A Year 2020 future conditions HEC-1 model for a 100-year, 24-hour storm event was provided by the Flood Control District of Maricopa County (District) as the base model for this project. This model consists of five sub-models and was updated by the District based on models from East Mesa Area Drainage Master Plan Study prepared by Dibble & Associates and Queen Creek/Sanokai Wash Hydraulic Master Plan & East Maricopa Floodway Capacity Mitigation Study prepared by Huitt-Zollars.

In addition to the base model, three target peak flows were provided by the 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 by District, Kirkham-Michael and Primatech during this study are listed below:

- Revision of the flows from CAP over chutes, CAP1a and CAP1b.
- Revision to incorporate revised hydrograph (QI record) out of Sanokai Flood Retarding Structure at Queen Creek.
- Replace the flow in the EMF downstream of the Rittenhouse basin with the resultant hydrograph derived from the Rittenhouse basin HEC-RAS analysis (using QI record).
- Replace the flow in the EMF downstream of the Chandler Heights basin with the resultant hydrograph derived from the Chandler Heights basin HEC-RAS analysis (using QI record).
- Create HEC-1 models for the 2-yr. and 5-yr. events for computation of the corresponding hydrographs in Queen Creek/Sanokai Wash. These hydrographs are used for computation of sediment volumes.

The HEC-1 models and the resultant hydrographs for the proposed conditions (with Rittenhouse Basin and Chandler Heights Basin) at Queen Creek Road, Chandler Heights Road, and Hunt Highway are presented in the appendix. The hydrographs for the existing conditions (without the detention basins) at the same locations will also listed in the appendix.

2.3. Input Hydrographs

Hydrographs were obtained for both Queen Creek and Sanokai Wash (HEC-1 Station ID: CO508). The flow in the EMF upstream of the Chandler Heights basin is derived from the Rittenhouse basin analysis. That analysis determined the flow downstream of the Rittenhouse channel (Refer Hydraulic Report for Rittenhouse Basin). That hydrograph was translated using HEC-1 downstream to the Chandler Heights basin (HEC-1 Station ID: RQCS). Both Queen Creek/Sanokai Wash and the EMF hydrographs have residual flows that exist for several days.

2.4. Resultant Hydrograph from the HEC-RAS Modeling

Resultant hydrograph from the Chandler Heights Basin HEC-RAS modeling was coded into HEC-1 using QI Card. This hydrograph is input in the HEC-1 model to replace the hydrograph generated in the base HEC-1 model in the EMF at the same location (HEC-1 ID: CHNBAS).

3. HYDRAULIC MODEL

3.1. Model Elements

The hydraulic structures at the Chandler Heights basin include a two level detention basin, a sedimentation basin, an approach open channel, a side weir to divert excess flows, and three outlet structures, one un-gated, one with a flap gate, and the third with a manually operated gate. The embankment of the future Ocotillo Road separates the two level basin. There is a 200-foot opening in the Ocotillo Road embankment to allow the hydraulic interaction of the two basins.

3.2. Channel

Two channel segments are simulated in the Chandler Heights Basin HEC-RAS model. One is a section of EMF located along the west side of the Chandler Heights Basin. The second is the combined Queen Creek/Sanokai Wash channel, which is located along the east side of the Chandler Heights Basin downstream of the confluence of the Queen Creek and Sanokai Wash.

3.2.1. EMF Channel

The geometry of the EMF Channel was modified based on the original base EMF HEC-RAS model Reach 3 provided by District. The existing drop structure in EMF at approximately 200 feet upstream Chandler Heights Bridge was relocated 2350 feet further upstream at Station 11.845. This results in sufficient drop at the outlet to enable the detention basin to completely drain.

To overcome the program instability associated with the unsteady flow analysis, the low

flow channel in the cross-section at the top of the drop was blocked. Steady flow analyses were performed to investigate the impact of the low flow channel blockage on the basin hydraulic modeling. The steady flow modeling was conducted with different flow configurations occurred during the operation period of the north basin outlet. The results show that the impacts are insignificant. Generally, with the blockage the water surface elevation in EMF at the outlet is around 0.05 feet higher than that without the blockage.

3.2.2. The combined Queen Creek/Sanokai Wash Channel

The combined Queen Creek/Sanokai Wash channel is sized to maintain the flow below the existing ground surface and allow connection of the Queen Creek and Sanokai Wash channels. This criterion results in an invert elevation of 1300.0 at the entrance to the sedimentation basin and a slope of 0.0003 ft/ft. The width was based on two discharges: 6000 cfs downstream of the confluence of Queen Creek and Sanokai Wash, and 2300 cfs downstream of the side weir diversion. A side weir of 1250 feet in length is located at the west bank of the channel to divert excessive peak flow into the detention basin. The low flow or bypass flow from a 100-year event will be conveyed by the channel into a sediment basin. Since the storage volume of the sediment basin is relatively small, it is not included in the HEC-RAS model. An outlet culvert was modeled at the end of the channel. Physically, this culvert is proposed to discharge water from the sediment basin into the EMF. The culvert was sized to allow discharging the bypass flow.

To avoid the program instability, this culvert is not connected to the EMF channel. The hydrograph obtained at the downstream end of the culvert was manually added into the EMF channel at the location where the culvert should be connected. It is an iterative modeling process to ensure the whole system being modeled properly.

3.3. Side Weir

The side weir is located downstream of the confluence of Sanokai Wash with Queen Creek. As the two washes have differently shaped hydrographs, using the combined hydrographs allows for maximum passage of the allowable flow (2300 cfs) and, therefore, the minimum storage required. The side weir elevation is set to achieve the bypass of 2300 cfs. The goal was to maximize the side weir discharge efficiency while maintain a constant elevation for the weir, which results in a narrowing of the channel as flow is diverted. A 1250-foot-long weir with the channel narrowing from 200 feet to 70 feet meets the desired hydraulic operation of bypassing 2300 cfs. Computation of flow over the side weir was performed in the same manner as for the Rittenhouse basin.

<u>Side Weir Coefficient:</u>

For broad 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 the 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:

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

Where, q = lateral outflow intensity over side weir (cfs/ft)
c = Weir shape correct factor.

For a vertical broad-crested weir with length B,

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

H = upstream energy head
w = height of side weir
h = water depth upstream side weir

$$y = \frac{h}{H}$$

$$W = \frac{w}{H}$$

Reorganizing Equation (1) we obtain:

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

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

$$Q = CLH^{3/2} \qquad (3)$$

Where, H = the upstream water depth above the crest (in the side weir modeling, the water surface option was selected for the weir flow calculation.)
L = length of the weir
C = discharge coefficient

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

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

Discharge coefficient was calculated for each time period. An average discharge coefficient of 2.44 was determined for the overall discharge time (See appendix for the detailed calculations and results).

3.4. Detention Basin Volume & Drain Operation

The final stage volume data for the two basins are:

Elevation	North Basin, Ac-Ft.	South Basin, Ac-Ft.	Total Storage, Ac-Ft.
1307	1470	631	2101
1306	1282	595	1877
1305	1096	525	1621
1304	910	456	1366
1303	726	388	1114
1302	543	320	863
1301	361	253	614
1300	180	187	367
1299	41	122	163
1298	0	58	58
1297	0	10	10
1296	0	0	0

Low point of north basin, 1297.8. Low point of south basin, 1296.0.

The basin is to drain within 36 hours from the end of the 24-hour storm event. The goal is to achieve this by a gravity system. The draining of the basins is accomplished using two outlet structures. The north basin has flap-gates with the invert at 1301.0; the south basin utilizes manually operated gates with the invert at 1296.0.

The sizing of both outlet structures involves an iterative procedure. The HEC-RAS program does not model the unsteady flow through a flap gate. It does allow for modeling of a sluice gate for which one can vary the height of the opening at different times. This is used to describe the flow condition. Using an Excel spreadsheet for flow through a culvert, which includes the losses incurred by using a flap gate, a discharge is calculated at a time step. This discharge is then used to determine the height of the opening to be input into HEC-RAS at that time step. HEC-RAS uses this opening data to calculate the discharge from the basin, the volume change in the basin, and the stage in the basin at each time step. At the completion of the HEC-RAS run, a comparison is made of the computed stage-time data for the current trial with the prior trial. If the difference is too great, the most recent trial is used to compute new openings, and re-entered into HEC-RAS. A solution has been achieved within 8 trials, the maximum difference between the last two computed stage-time data is 0.04 feet.

An Emergency Spillway was set in the HEC-RAS model at the west bank of the detention basin as proposed in the basin plan to discharge excess water from basin to the EMF for a super flood event, The elevation of the spillway is determined to be 1305.5, which is 0.4 feet above the maximum water surface elevation in the basin computed for a 100-year event. Therefore, there is no flow over the emergency spillway for a 100-year event modeled in this study. No modeling was performed for a super flood event such as a 500-year flood at this time.

3.5. Flow Process

The flow process for the HEC-RAS model is described below:

A. The Queen Creek and Sanokai Wash hydrographs are summed.

B. The combined flow is entered into the 200-foot open channel.

C. Downstream of the junction, the flow is routed through a section of channel that has a side weir and narrows to 70 feet. This allows the by-pass of flows less than 2300 cfs, and diversion into the south detention basin of excess flows.

D. The bypass flows are routed into a sedimentation basin which outlets to the EMF through a group of unregulated box culverts.

E. The diverted flow is routed into the two detention basins. The south basin is one-foot lower than the north basin. A 200-foot opening connects the two basins.

F. The north basin has a group of outlet culverts with flap gates. These are modeled using a sluice gate that is computationally set at each time step to match the discharge that is computed externally for a culvert with a flap gate operating under varying head conditions. The sluice gate is opened when the stage in the basins is higher than the stage in the EMF at the outlet location.

G. The south basin has a group of outlet culverts with manually operated gates that are opened when there is capacity for the discharge in the EMF. These outlets are also modeled using a sluice gate that is computationally set at each time step to match the discharge that is computed externally for a culvert operating under varying head conditions.

H. The hydrograph from EMF is combined with hydrograph from the three group of outlets described in D., F., and G. The combined hydrograph will be used in the HEC-1 model to check if the peak flows meet the target flow requirements at Chandler Heights Road and at Hunt Highway.

3.6. Unsteady Flow Data

The combined Queen Creek/Sanokai Wash hydrograph is used as input into the model. The derivation of the combined hydrograph is described in Section 2.3.

3.7. Boundary Conditions

The boundary conditions set are normal depth at the most downstream section. In the model these occur: 1) at the most downstream cross-section of the EMF channel, and 2) downstream of the outlet from the sediment basin. For the EMF, since there is a rather long and uniform channel section downstream of the most downstream cross-section used in the unsteady flow model, it is reasonable to use the channel bottom slope (0.0003) to estimate the energy slope for the boundary condition. For the outlet form the sediment basin, analysis showed that inlet control would dominate over the water level range in the EMF. Therefore, the accuracy of the boundary water surface elevation is not important for the modeling. Initial Conditions

For the basin model, a starting water surface elevation in the basins is required. Since the basins are dry at the beginning of the storm event, the water surface is set at the bottom of the basins.

3.8. Outlet Modeling Approach

Similar to other available unsteady flow modeling programs, 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 a culvert in the system. The option of a sluice gate was used in the study to model the outlet culverts with a flap gate or a manually operated 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 set for each gate. The 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. The FHWA manual "*Hydraulic Design of Highway Culverts*" (FHWA, 1985), HDS-5, Appendix A – Design Methods and Equations, Table 9, Chart No. 10, Scale 3, provides the following two equations for unsubmerged and submerged culvert design:

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

Submerged: $H_w/D = c\left[\frac{Q}{AD^{0.5}}\right]^M + Y - 0.5S$ for $Q/AD^{0.5} \ge 4.0$ (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 vary depending on culvert shape and entrance condition. For this study, 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/per barrel, which is the case experienced in the Chandler Heights basin outlet hydraulics for the inlet control flow condition.

Rewriting Equation (4) with B=1' (unit width), we obtained:

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

Outlet Control Equation

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

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

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:

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

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:

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

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:

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

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
α_{ex} = coefficient velocity weighting coefficient, using α_{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-inch 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:

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

HEC-RAS Sluice Gate Equation

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

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

Where, H = upstream energy head above the spillway crest
C = coefficient of discharge

When the downstream tail water increases to form a submergence condition; that is, when the tail water 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 reaches 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

Outlet Modeling Procedure

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

Step 2. Obtain the resultant stage hydrographs at EMF and in the basin starting from the time when the 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:

- Determine average water surface elevation in the basin;
- Determine the difference of the water surface elevation in the basin and in EMF;
- Calculate discharge using both inlet and outlet control equations;
- 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. Enter gate-opening values determined in Steps 4 and 5 for each time interval that corresponds to each section of the hydrograph into HEC-RAS.

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

Step 8. Review the new resultant stage hydrograph in EMF from HEC-RAS to check whether or not it is close enough to the previous ones.

If NO, go back to Step 3.

If YES, the outlet modeling is complete.

The discharge coefficient C selected in this analysis for outlets with flap gate and manually operated gate are 0.8 and 0.6 respectively.

The flap gate in the north basin was initially opened at 24 hours when the basin stage rose above the EMF stage. It was closed when the north basin stage dropped below the outlet invert elevation of 1301.0. An evaluation was made for the time to open the manually opened gate in the south basin. The gate can be opened whenever there is capacity for some discharge into the EMF. Based on the hydrographs being used for the 100-year event, an opening time of 36 hours was selected.

4. SEDIMENT BASIN

West Consultants, Inc. estimated the sediment transport capacities of Queen Creek and Sanokai Wash as part of the East Maricopa Floodway Capacity Mitigation Study in a Final Report for the sediment transport analysis (West Consultants, 2000). The analysis provided estimates of the aggradation-degradation and armoring processes along the two streams. The results are based on the sediment transport capacity of the

downstream reaches in Queen Creek and Sanokai Wash upstream of the Chandler Heights Basin. It is stated in the report that sedimentation basins would be required to capture the sediment load corresponding to a 100-year, 24-hour storm event.

The West Consultants, Inc. study estimated specific bed material discharge, q_s, specific bulked sediment volume V_t for both the 5-year, 24-hour and the 100-year, 24-hour events, and overall bed load volume for the 100-year, 24-hour event. The specific bulked sediment volume V_t for the 2-year, 24-hour event was estimated in the present study based on existing hydraulic and sediment parameters presented in the Final Report, Sediment Transport Analysis by West Consultants. The overall bed load volume for both the 2-year, 24-hour and the 5-year, 24-hour events were evaluated further based on the corresponding specific bulked sediment volume. The 100-year, 24-hour overall bed load volume was used for the design of the sediment basin at the end of the combined Queen Creek/Sanokai Wash channel. The 2-year, 24-hour, and 5-year, 24-hour overall bed load volumes are not used for the design of a sediment basin. However, they are important in terms of the maintenance required for the sedimentation basin.
The specific bulk sediment volume V_t for the 2-year, 24-hour event was derived by correlating sediment transport parameters (q_s) with appropriate hydrographs. Table S1 presents the hydraulic parameters for Queen Creek and Sanokai Wash. The parameters describe Reach 1 of Queen Creek and Reach 2 of Sanokai Wash. Table S2 presents the sedimentation parameters for Queen Creek and Sanokai Wash, for the corresponding reaches. For the combined Queen Creek/Sanokai Wash, the 2-year, 5-year and 100-year sediment volumes are 10,800 cy, 17400 cy and 71000 cy respectively.

Table S1 – Hydraulic Parameters for Queen Creek and Sanokai Wash

	Peak Flow Rate	Mean Velocity	Hydraulic Depth	Slope	Manning's Roughness Coefficient	Top Width
	Q	V	Yh	S	n	B
	(cfs)	(ft/s)	(ft)	(ft/ft)		(ft)
Queen Creek						
2-year	1055	3.52	2.572	0.00229	0.035	130.54
5-year	1459	4.52	3.102	0.00229	0.035	136.30
100-year	2934	5.29	4.000	0.00229	0.034	153.00
Sanokai Wash						
2-year	302	2.36	1.543	0.003307	0.042	102.85
5-year	1254	3.3	1.950	0.003307	0.042	147.90
100-year	3141	4.69	3.320	0.003307	0.042	199.00

Table S2 - Sedimentation Parameters for Queen Creek and Sanokai Wash

	Bed Material Discharge	Bulked Sediment Volume	Degradation Depth	Overall Sediment Volume
	q_s	V_t	Depth	
	(cfs/ft)	(cfs)	(ft)	(cy)
Queen Creek				
2-year	0.0288	12.1	1.1	7400
5-year	0.0314	13.2	1.2	8000
100-year	0.0619	27.1	2.1	45000
Sanokai Wash				
2-year	0.0028	0.1	0.0	3400
5-year	0.0076	0.4	0.0	9400
100-year	0.0347	1.5	0.1	26000

The sediment basin is sized to capture sediment larger than 0.01 mm when the flow is 2400 cfs. The process used is:

1) Determine the fall velocity of the 0.01 mm particle.
2) Determine the fall distance required from top of water surface to 4 feet below the invert of outlet structure. Divide the fall distance by the sediment fall velocity to determine the fall time.
3) Calculate the average flow velocity in the sediment basin.
4) Multiply the average flow velocity times the required fall time to calculate the length of basin required.

The volume of sediment to be stored was based on the ratio of 2400 cfs to the total peak discharge of 6000 cfs. The design volume of 766,800 cf, is 40% of the total volume. The desired storage volume is provided by a depth of 6 feet below the outlet invert.

The 2-year and 5-year volumes are 292,000 cf and 470,800 cf respectively. The sediment basin should need cleaning at an interval greater than 5 years.

5. RESULTS

The combination of outlet structure sizes, weir length and elevation resulted in achieving the design goals. The peak bypass flow is 2360 cfs, and the peak water surface elevations are 1305.1 in the detention basins, and 1305.9 in the sediment basin. The peak storage in the detention basin is 1647 AC-FT. The sediment basin will capture particles larger than 0.01 mm and the peak discharge downstream of the Chandler Heights basin is 5678 cfs. Below is a summary of the final geometry for those designed elements:

Side weir:	Length = 1250 ft, Elevation 1307.2 ft
QC/SW Channel at side weir:	Beginning width = 200 ft, ending width70 ft Max. water surface elevation at Sta. 4000 – 1308.3
Sediment Basin:	Elevation at channel – 1300.0 Elevation at box culvert invert – 1298.0 Elevation at bottom of basin – 1292.0

5.1. Outlet Sizing Results

The outlet culverts are sized to drain the basin within 36 hours 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 12-barrel 4'x4' box culvert at the south basin, there will be only about 1 inch water remaining in the basin at the time of the end of 36 hours.

The three outlet structures are determined to be:

Sedimentation Basin:	13-4x4 box culvert Invert Elevation: 1298.0 Slope 0.01
North basin:	13-4x4 box culvert with flap gate Invert Elevation: 1301.0 Slope 0.01
South basin:	12-4x4 box culvert Invert Elevation: 1296.0 Slope 0.01

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PRIMATECH

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Job No. _____ Project East Maricopa Floodway Sheet ____ of ____

Subject CHANDLER HEIGHTS BASIN - "C" coefficient

By G. Lopez-Capuo Date 10/26/01 Checked By _____ Date _____

To Determine "C" coefficient:

From a trial run:

1) Get Stage at u/s end at time step.
2) At same time step, get flow, cfs.
3) Determine values of depth of flow, velocity, energy head.
4) Use above values to calculate "C".

Example: at time 15100

$Q = 3230$, stage $= 1307.95$

Channel bottom $= 1300.9$, Weir Elev. $= 1307.2$

h ∴ Depth of flow $h = 1307.95 - 1300.9 = 7.05'$

Area $= 7.05(200 + z(4)(7.05)/z) = 1608.81\ ft^2$

$v = 3230/1608.81 = 2.008\ ft/s$

Energy Head $= (2.008)^2/64.4 = 0.063$

H = Total Head $= 7.05 + 0.063 = 7.113$

w = Height of weir $= 1307.2 - 1300.9 = 6.3'$

$$Y = \frac{h}{H} = \frac{7.05}{7.113} = 0.9912$$

$$W = \frac{w}{H} = \frac{6.3}{7.113} = 0.8858$$

$$C = 2.622\left(\frac{1-W}{3-2Y-W}\right)^{1/2} = 2.622\left(\frac{1-0.8858}{3-2(0.9912)-0.8858}\right)^{1/2} =$$

$$= 2.622\left(\frac{0.1142}{0.1318}\right)^{1/2} = 2.441$$

See table for other time steps.

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS BASIN
SIDE WEIR COEFFICIENT--C

MFC027
GLC
10/26/01

Channel Bottom= 1300.9
Weir Elevation= 1307.2
Weir Height= 6.300

Time	Discharge	Elevation	Depth	Velocity	Energy Head	Total Head	y	W	C
12	10	1302	1.1						
13	15	1302	1.1						
14	860	1306.5	5.6						
15	3230	1307.95	7.05	2.008	0.063	7.113	0.9912	0.8858	2.441
16	4260	1308.2	7.3	2.546	0.101	7.401	0.9864	0.8513	2.411
17	3820	1308.1	7.2	2.319	0.083	7.283	0.9885	0.8650	2.424
18	4570	1308.25	7.35	2.710	0.114	7.464	0.9847	0.8440	2.398
19	5480	1308.5	7.6	3.130	0.152	7.752	0.9804	0.8127	2.384
20	5050	1308.4	7.5	2.928	0.133	7.633	0.9826	0.8254	2.394
21	4080	1308.2	7.3	2.438	0.092	7.392	0.9875	0.8522	2.425
22	3360	1308.02	7.12	2.065	0.066	7.186	0.9908	0.8767	2.446
23	3000	1307.86	6.96	1.892	0.056	7.016	0.9921	0.8980	2.439
24	2880	1308.5	7.6	1.645	0.042	7.642	0.9945	0.8244	2.544
25	3480	1309	8.1	1.849	0.053	8.153	0.9935	0.7727	2.550
26	3360	1309	8.1	1.785	0.049	8.149	0.9939	0.7731	2.555
27	3030	1308.9	8	1.633	0.041	8.041	0.9949	0.7834	2.562
28	2750	1308.8	7.9	1.503	0.035	7.935	0.9956	0.7939	2.567
29	2580	1307.4	6.5	1.756	0.048	6.548	0.9927	0.9621	2.227
30	2450	1307.7	6.8	1.586	0.039	6.839	0.9943	0.9212	2.450
31	2300	1307.64	6.74	1.504	0.035	6.775	0.9948	0.9299	2.447
32	2120	1307.57	6.67	1.402	0.031	6.701	0.9954	0.9402	2.442
33	1980	1307.5	6.6	1.325	0.027	6.627	0.9959	0.9506	2.428
34	1890	1307.42	6.52	1.282	0.026	6.546	0.9961	0.9625	2.386
35	1820	1307.36	6.46	1.247	0.024	6.484	0.9963	0.9716	2.334
36	1780	1307.32	6.42	1.229	0.023	6.443	0.9964	0.9777	2.276

Avg. Value — 2.433

PRIMATECH

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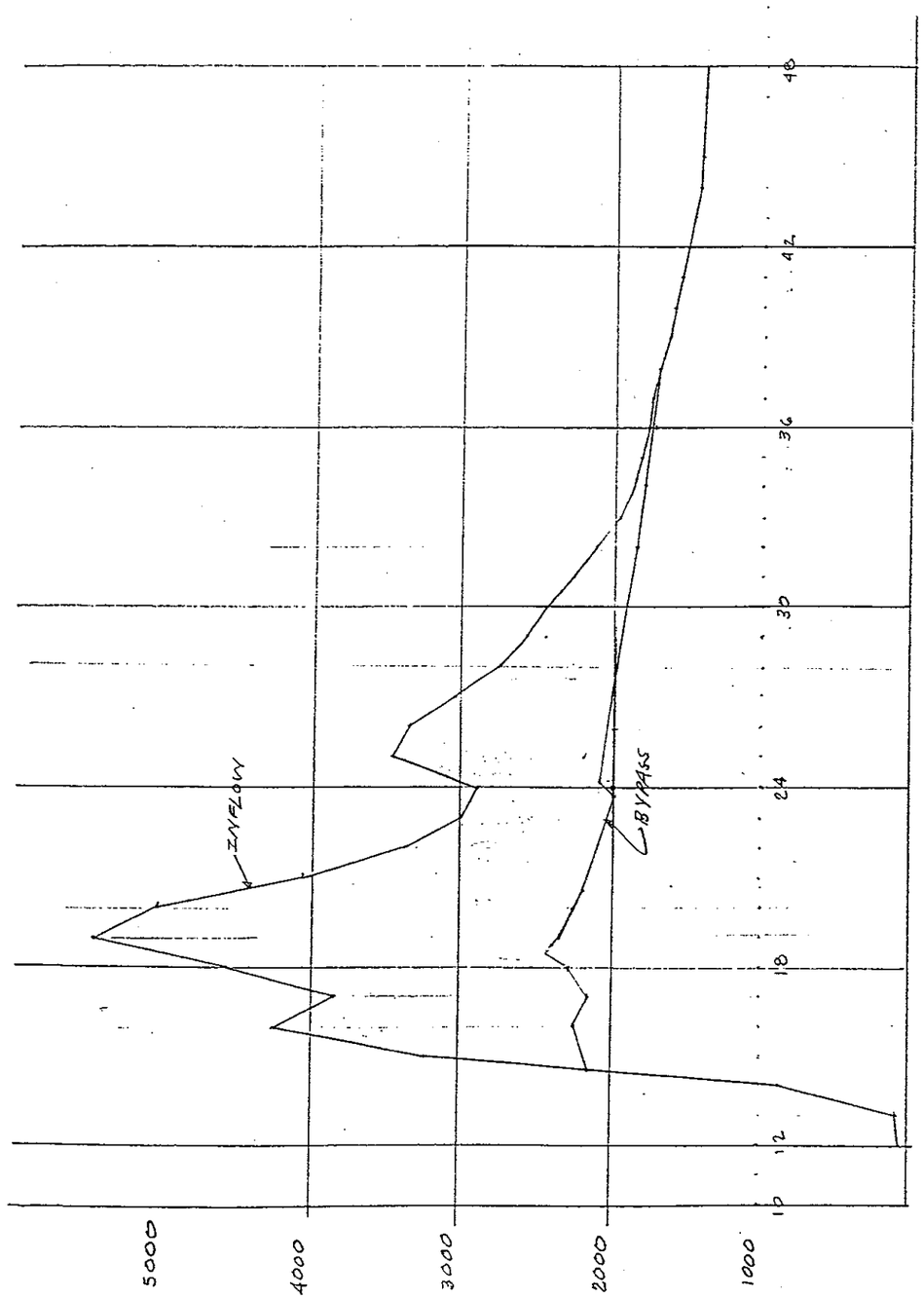
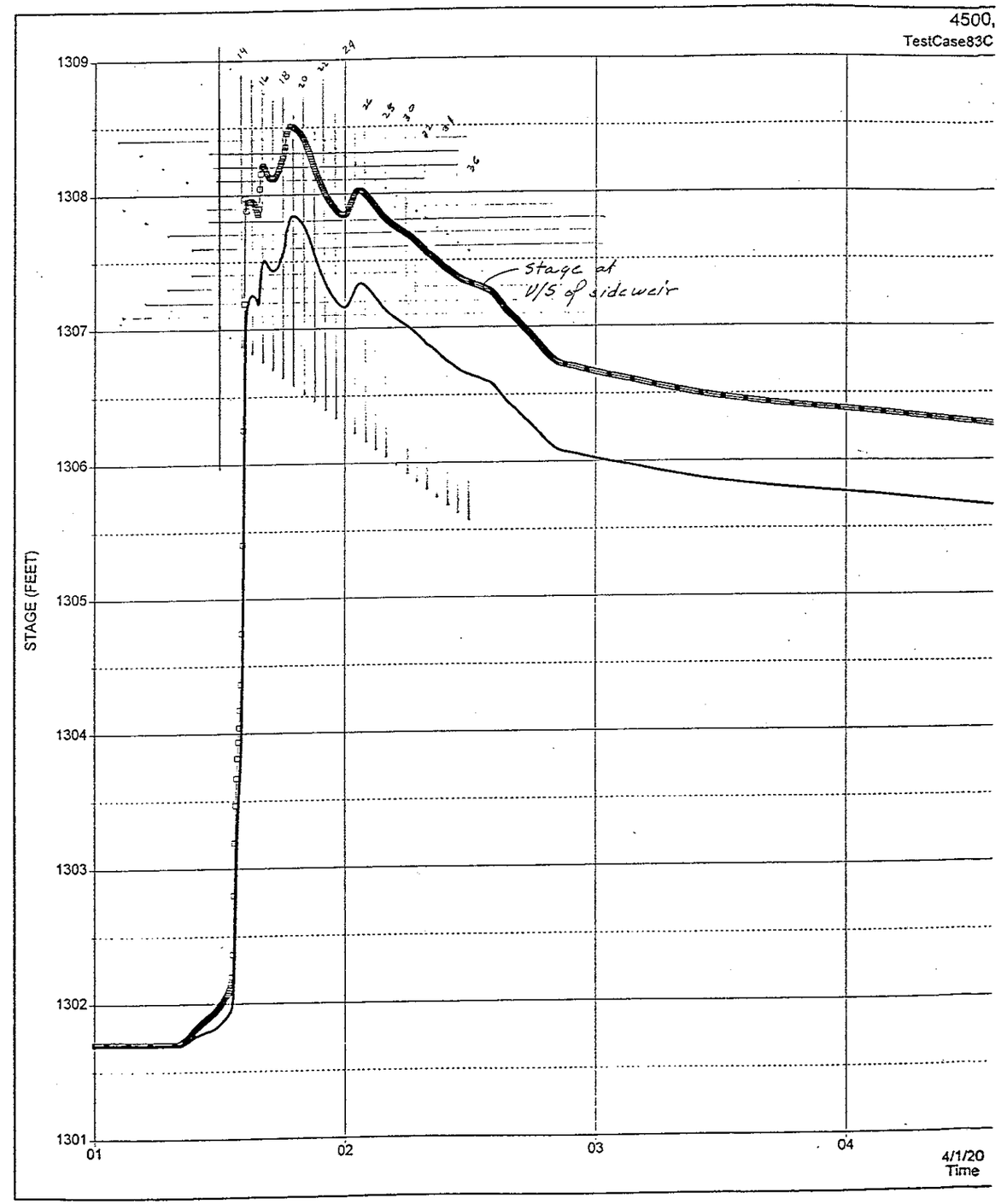
Job No. _____ Project EMF-CHANDLER HEIGHTS BASIN Sheet ____ of ____

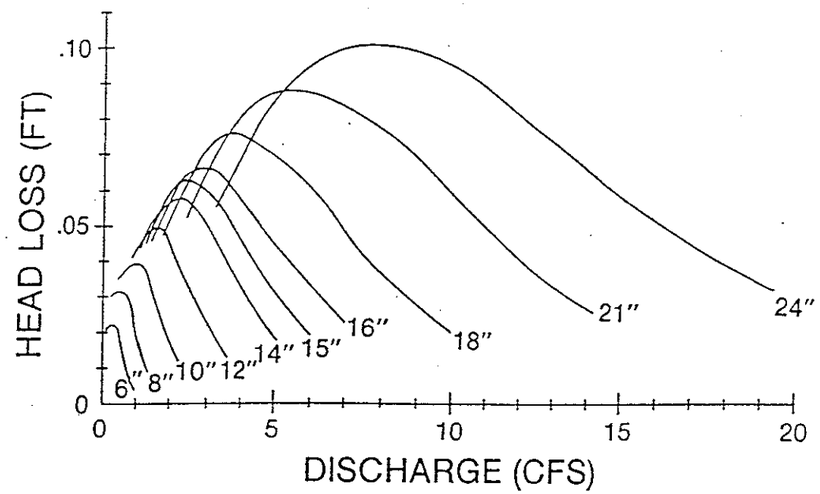
Subject Combined Hydrographs – Queen Creek & Sonokai Wash

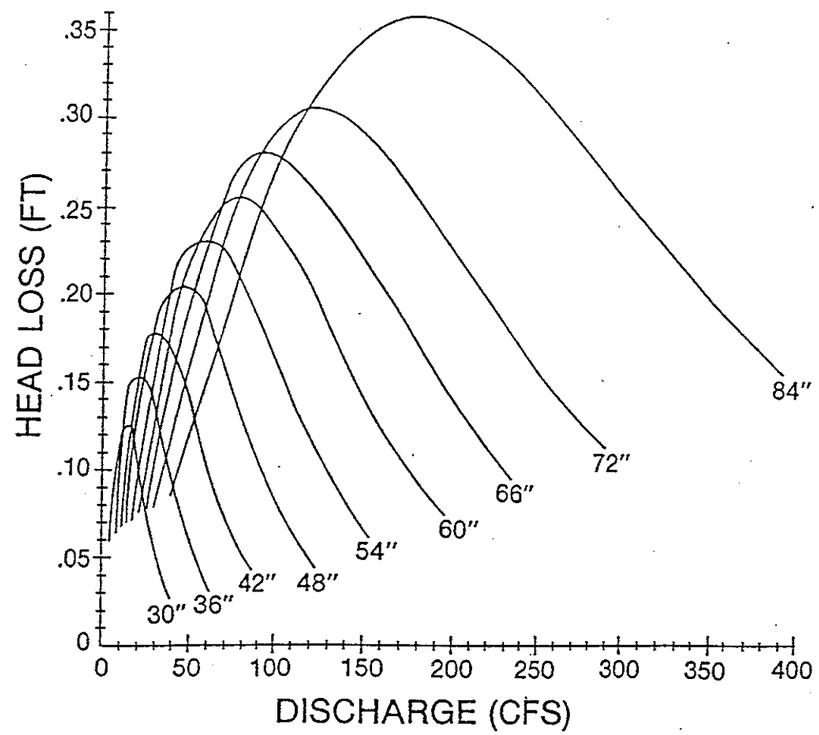
By GLC Date 9/21/01 Checked By _____ Date _____





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.

Waterman INDUSTRIES, INC.

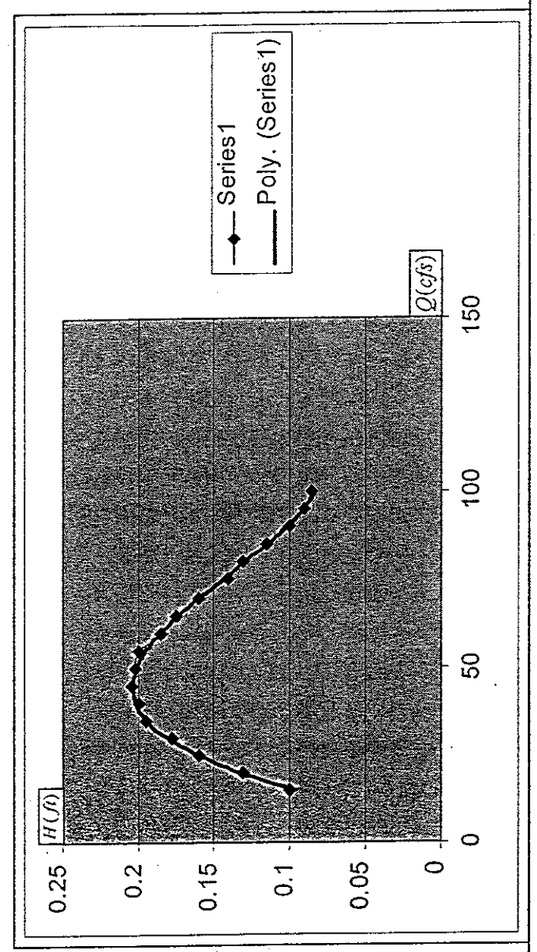
EXETER, CA • LUBBOCK, TX • GARDEN CITY, KS • BOISE, ID
MEMPHIS, TN • GRAND ISLAND, NB • INDIANAPOLIS, IN
RED TOP WATER CONTROL GATES, VALVES and EQUIPMENT

NOTE: FOR PRELIMINARY DESIGN PURPOSES ONLY
DO NOT USE FOR INSTALLATION
UNLESS PART OF CERTIFIED & APPROVED SUBMITTAL

$H_{exG}=0.000001Q^3-0.00023Q^2+0.0145Q-0.072$

Where: h_{exG} = Engergy loss due to a 4'x4' flap gate

Loss Curve based on Waterman Industries, Inc Catalog Dwg. No. 0049, 48" steel Drainage Gate



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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 Cont #12 - Scale 3

Coefficients -

Unsubmerged: Eqn Form = 2, $K = 0.435$, $M = 0.667$

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; $AD^{.5} = BD^{1.5} = (4)(4)^{1.5} = 32$

$\therefore$ 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 H_w

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

for B=1', D=4',

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

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

$$Q = D^{1.5}\left[\frac{H_w}{0.435D}\right]^{1/0.67} = \frac{D^{1.5}H_w^{1.5}}{(0.337)(D^{1.5})} \ldots$$

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

I. Conditions

1. Volume of sediment interested

From the Final Report of Sedimentation Analysis for Queen Creek & Sanokai Wash
For 100-year event. for 24-hour

Sediment contribution from QC = 45,000 yd³
Sediment contribution from SW = 26,000 Yd³
Tototal sediment volume = 71,000 Yd³
(1,917,000 ft³)

2. Flow rate

For 100-year event

Total Flow rate Q_T = 6000 cfs
Flow through the sedimentation Basin Q = 2400 cfs
Assume that the sediment distribution in the flow is uniformed. Therefore,
the sediment into the SB, $V = \frac{2400}{6000}(1917000) = 766.800 ft^3$

3. Assume the particles are spereical shape, and the size of particle to be trapped by the sedimentation basin is equal or greater than 0.01 mm (diameter).

4. Maximum water surface Elev. = 1306

5. BCB, which connects to EMF, Inv. Elev = 1298

6. The sedimentation Basin bottom Elev = 1294

7. The Basin side slope. H:V = 4:1

8. channel side slope H:V = 4:1

9 channel slope = 0.03%

10. The basin configuration — Scenario II

II. Approach to Sizing the Sedimentation Basin.

1. Determine the fall velocity (w) of the particles with a diameter of 0.01 mm

1). $w = \sqrt{\frac{4gd}{3C_D}\frac{\gamma_s - \gamma}{\gamma}}$

where
g — gravity acceration (9.81 m/s²)
d — particle diameter (0.01mm)
γ_s — particle specific weight
γ — water specific weight
C_D — drag co-efficient

2) $R_e = \frac{w d \rho}{\mu}$

where
Re — Reynolds number
w — particle fall velocity
d — particle diameter
ρ — water desity (988.2 kg/m³ @ 20°c)
μ — dynamic viscosity of water (1.002×10^{-3} kg/m·s @ 20°c)

By trial and error
w = 0.02472 ft/s (see Table 2)

2. Determine the length of the SB

1) particle fall velocity, w

$w = \frac{d}{t}$

where
d — allowable water depth (ft)
t — time for settling particle's with 0.01mm diameter

2) Flow velocity, v

$$v = \frac{L}{t}$$

where
L – length of the sedimentation Basin (ft)
t – time for settling particles with 0.01 mm diameter

By running HEC-RAS on the conditions indicated in Section I. Use the maximum flow velocity in the SB

$v = 0.83$ ft/s.

3) Length of SB

$$L = \frac{v}{w} d$$

$$= \frac{0.83 \text{ ft/s}}{0.02472 \text{ ft/s}} (1306 - 1294) = 402.9 \text{ ft.}$$

(Say 410 ft)

3. Determine the depth of the SB.

1) SB Bottom Total Area, A

$$A = \frac{1}{2}(a+b)h$$
$$= \frac{1}{2}(200+400)(500) = 150{,}000 \text{ ft}^2$$

2). SB Bottom Effective Area, Ae

$$A_{eff} = \frac{1}{2}(240+400)(410) = 131{,}200 \text{ ft}^2$$

3). Sediment Depth in terms of the entire basin

$$d = \frac{V}{A}$$
$$= \frac{776{,}800 \text{ ft}^3}{150{,}000 \text{ ft}^2} = 5.2 \text{ ft}$$

4). Sediment Depth in terms of the effective area

$$d_{eff} = \frac{776{,}800 \text{ ft}^3}{131{,}200 \text{ ft}^2} = 5.9 \text{ ft} \quad (\text{Say } 6 \text{ ft})$$

4. Size of the Sedimentation Basin.

Length of the current basin configuration

$L = 500$ ft. > 410 ft (calculated) OK

Basin bottom Elev = 1294, which will provide with a depth of 6 ft

(channel bottom Elev − basin bottom Elev = 1300 − 1294 = 6 ft)

5. Check SB Length required when in full capacity of the SB, where: SB bottom Elev = 1298

Run REC-RAS

Max. Flow velocity, $v = 1.2$ ft/s

SB Length, $L = \frac{v}{w}(d)$

$$L = \frac{1.2 \text{ ft/s}}{0.02472 \text{ ft/s}}(1306 - 1298)\text{ft} = 388.3 \text{ ft} < 410 \text{ ft. } \underline{\underline{OK}}$$

Conclusion

To handle the sediment from Qc and SW for 100-year event, and a flow of 2400 cfs, a 6-foot depth of sedimentation basin is required, in terms of the current basin configuration shown on Figure 2.

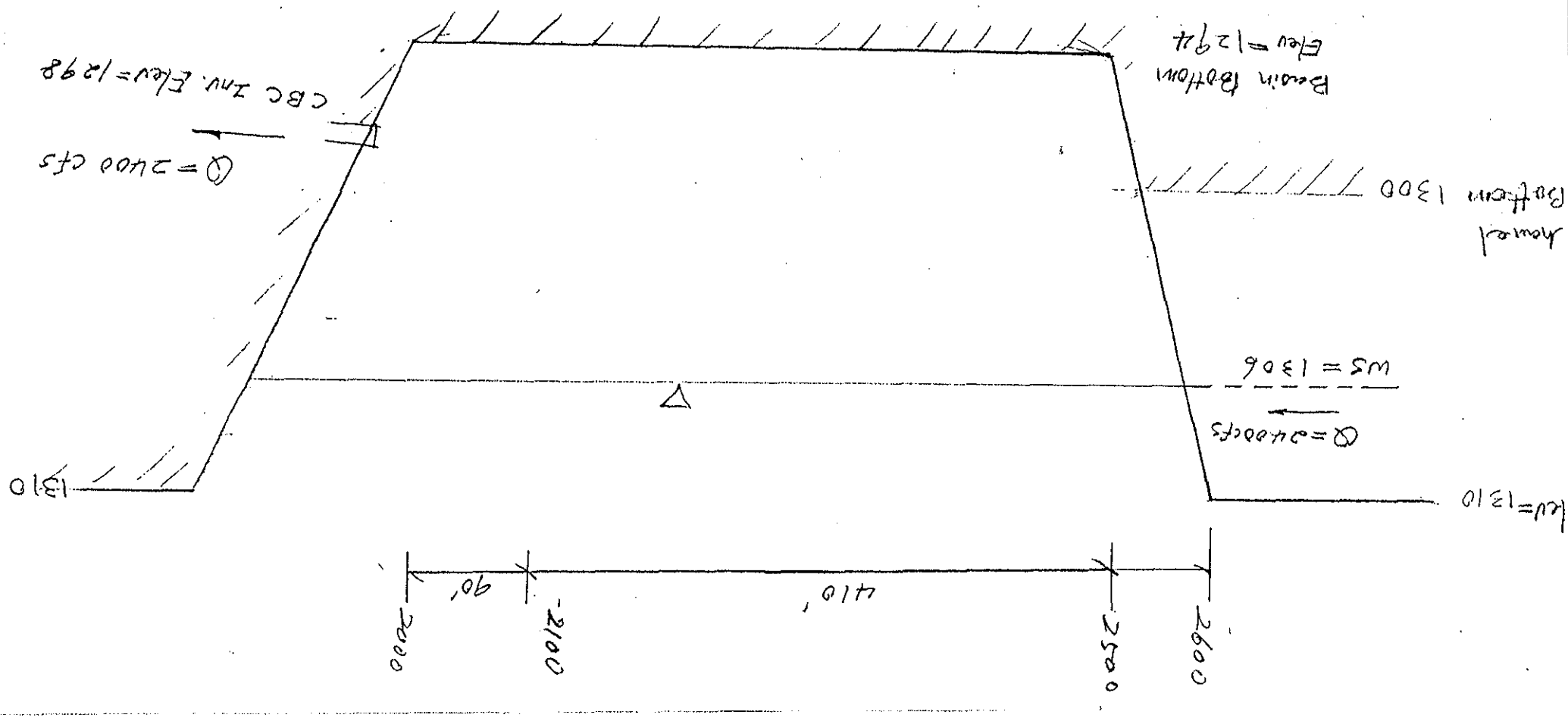
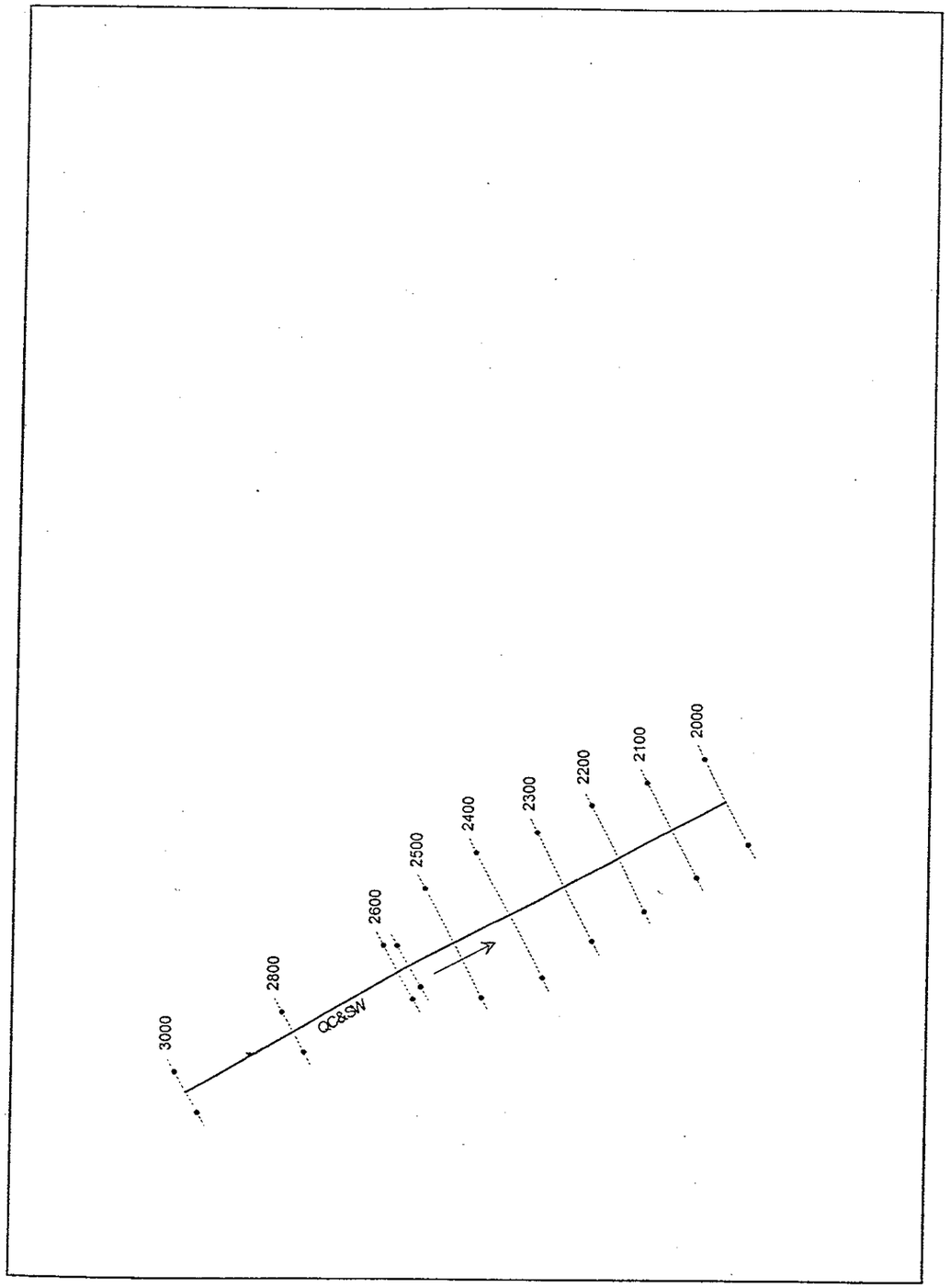


TABLE 2 CALCULATION FOR LENGTH OF SEDIMENTATION BASIN (SCENARIO II, Q=2400cfs)

w	w	Cd	Re	d	d	t	t	v	v	L
m/s	ft/s			ft	m	s	min	ft/s	m/s	ft
0.00379	0.01244	15.00	0.03779	13.0	3.962	1044.626	17.410	0.750	0.229	783.474
0.00585	0.01920	63.00	0.05831	13.0	3.962	676.995	11.283	0.750	0.229	507.749
0.00735	0.02410	40.00	0.07318	13.0	3.962	539.442	8.991	0.750	0.229	404.584
0.00744	0.02441	39.00	0.07411	13.0	3.962	532.657	8.878	0.750	0.229	399.495
0.00754	0.02472	38.00	0.07508	13.0	3.962	525.783	8.763	0.750	0.229	394.340
0.00759	0.02489	37.50	0.07558	13.0	3.962	522.313	8.705	0.750	0.229	391.737
0.00764	0.02506	37.00	0.07608	13.0	3.962	518.819	8.647	0.750	0.229	389.117

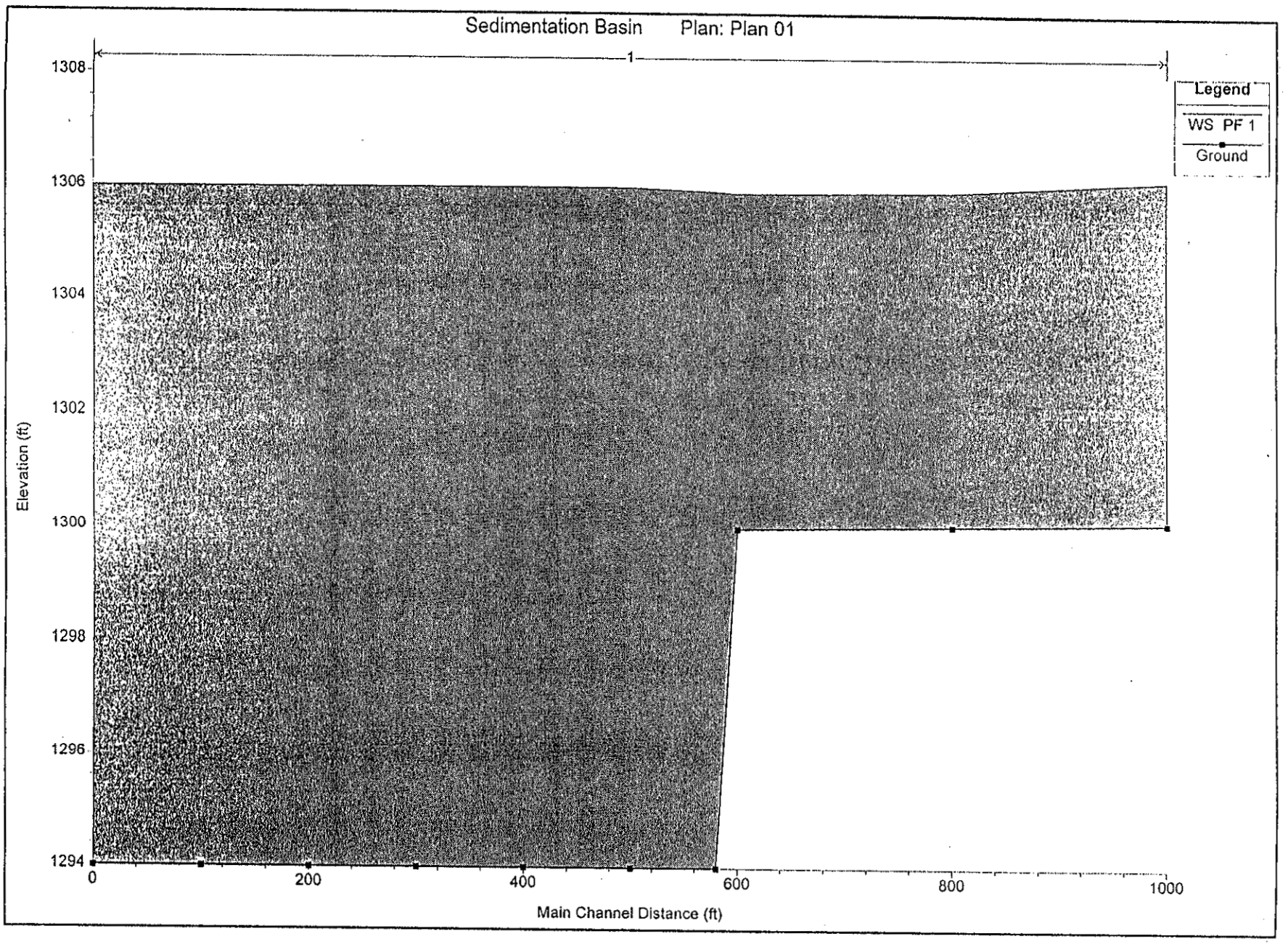
where

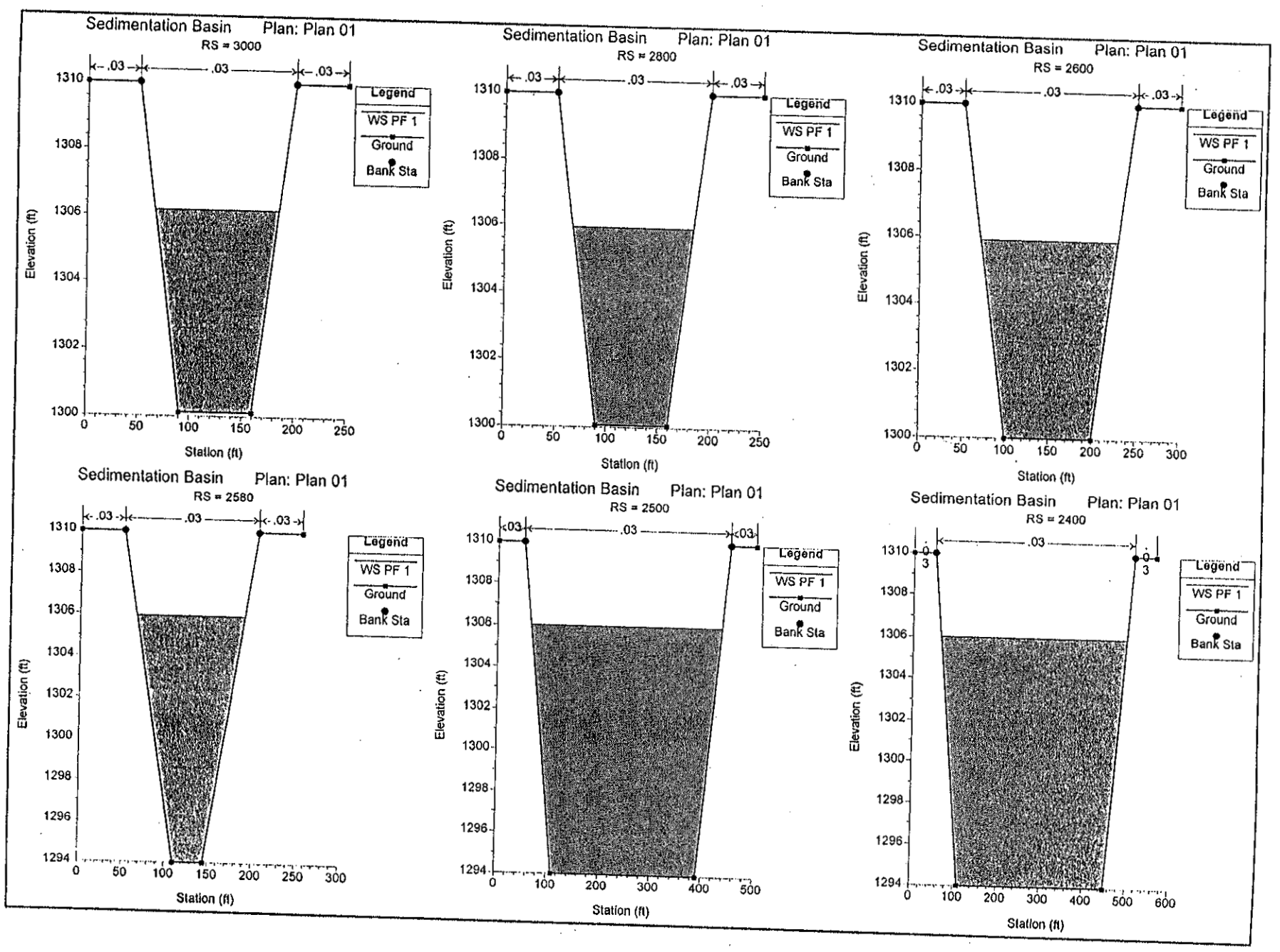
w	Falling Velocity
Cd	Drag Co-efficient
Re	Renolds Number
d	Allowable Water Depth
t	Minimum Time for Settling the particles with a diameter of 0.01 mm
v	Flow velocity in the proposed Sedimentation Basin
L	Minimum Length of the proposed Sedimentation Basin

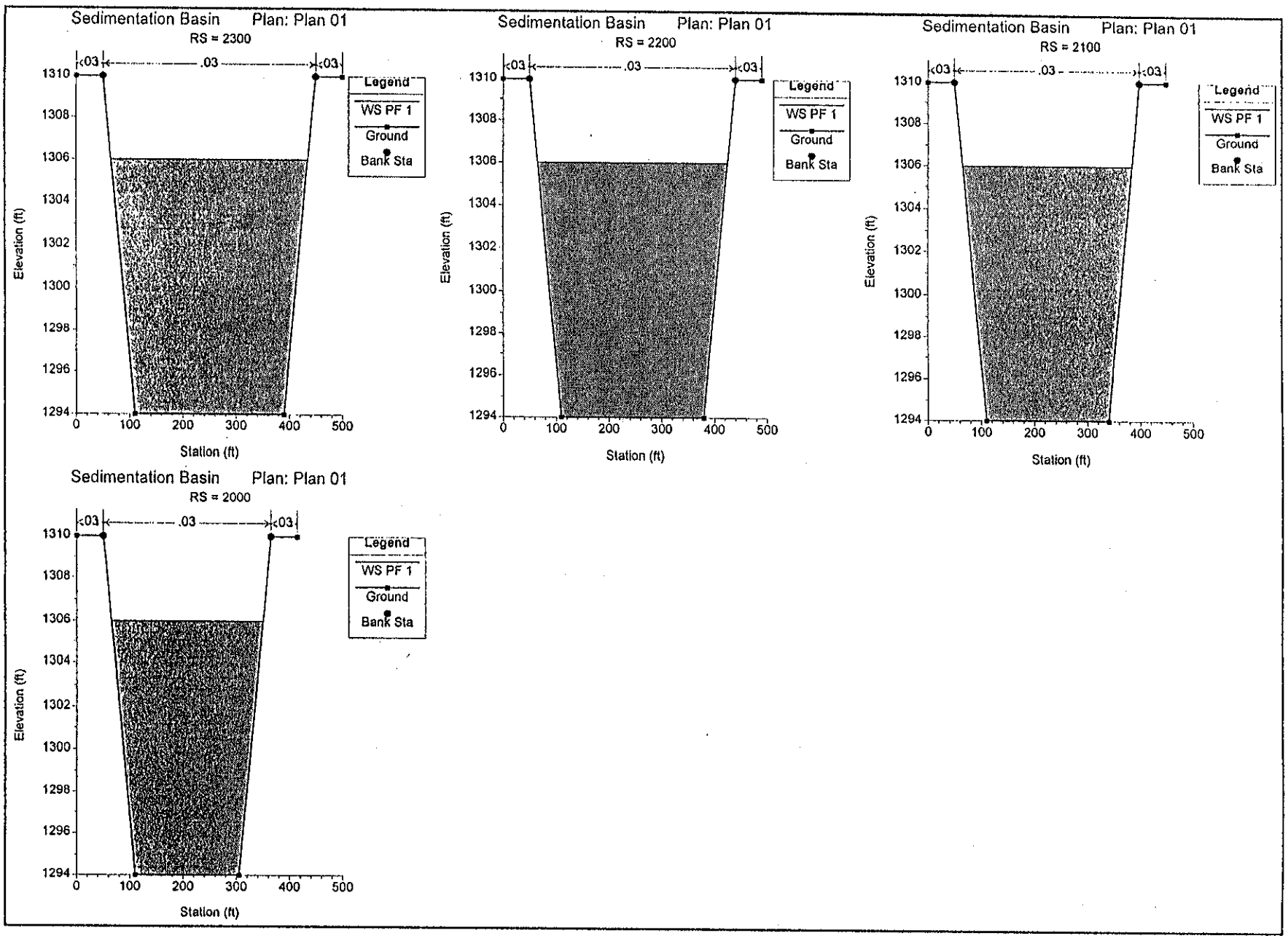


HEC-RAS Plan: SB S1 River: QC&SW Reach: 1 Profile: PF 1

Reach	River Sta.	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
		(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	3000	2400.00	1300.12	1306.16		1306.44	0.000904	4.20	570.87	118.93	0.34
1	2800	2400.00	1300.06	1305.95		1306.25	0.000993	4.35	552.28	117.43	0.35
1	2600	2400.00	1300.00	1305.92		1306.07	0.000496	3.13	766.54	159.16	0.25
1	2580	2400.00	1294.00	1305.95		1306.05	0.000177	2.52	953.49	124.61	0.16
1	2500	2400.00	1294.00	1306.01		1306.02	0.000007	0.61	3904.11	370.08	0.03
1	2400	2400.00	1294.00	1306.01		1306.02	0.000005	0.52	4624.62	430.08	0.03
1	2300	2400.00	1294.00	1306.01		1306.01	0.000007	0.61	3902.98	370.06	0.03
1	2200	2400.00	1294.00	1306.01		1306.01	0.000007	0.63	3781.93	360.04	0.03
1	2100	2400.00	1294.00	1306.00		1306.01	0.000010	0.73	3300.86	320.02	0.04
1	2000	2400.00	1294.00	1306.00	1295.66	1306.01	0.000013	0.83	2880.00	285.00	0.05

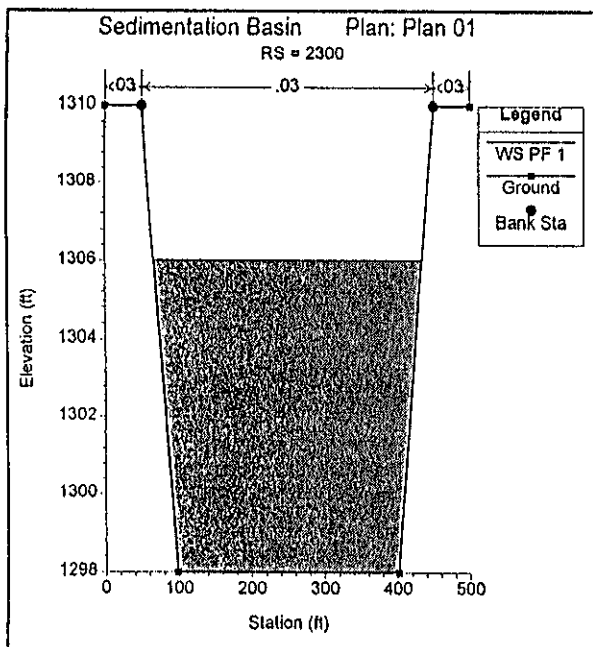

Sedimentation Basin Plan: Plan 01
Legend
WS PF 1
Ground
Elevation (ft)
Main Channel Distance (ft)

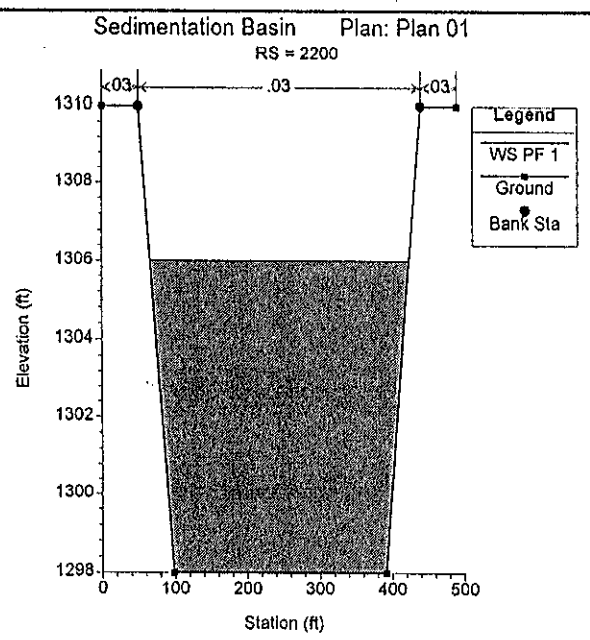

Sedimentation Basin Plan: Plan 01
RS = 3000
RS = 2800
RS = 2600
RS = 2580
RS = 2500
RS = 2400
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)

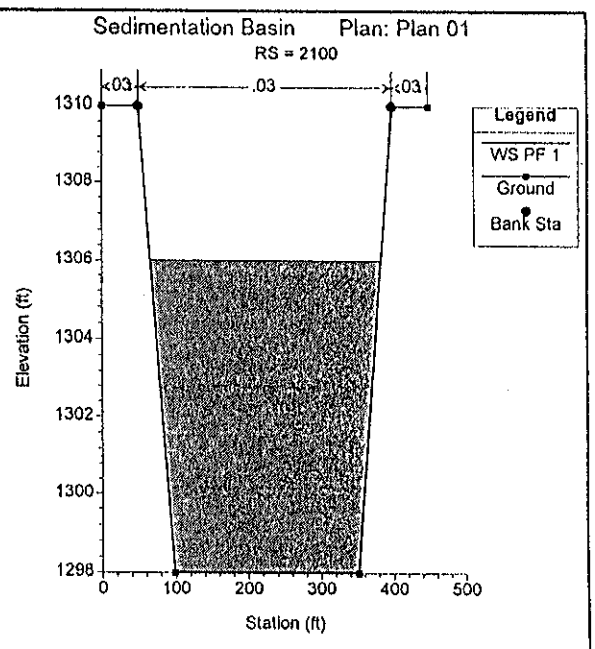


HEC-RAS Plan: SB S1 River: QC&SW Reach: 1 Profile: PF 1

Reach	River Sta	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
		(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	3000	2400.00	1300.12	1306.19		1306.46	0.000890	4.18	574.15	119.16	0.34
1	2800	2400.00	1300.06	1305.99		1306.27	0.000974	4.32	556.03	117.69	0.35
1	2600	2400.00	1300.00	1305.95		1306.10	0.000486	3.11	771.97	159.50	0.25
1	2580	2400.00	1298.00	1305.91		1306.09	0.000440	3.34	717.49	122.31	0.24
1	2500	2400.00	1298.00	1306.02		1306.04	0.000023	0.89	2696.27	368.18	0.06
1	2400	2400.00	1298.00	1306.02		1306.03	0.000016	0.76	3177.20	428.17	0.05
1	2300	2400.00	1298.00	1306.02		1306.03	0.000023	0.89	2694.34	368.14	0.06
1	2200	2400.00	1298.00	1306.01		1306.03	0.000024	0.92	2612.63	358.10	0.06
1	2100	2400.00	1298.00	1306.01		1306.02	0.000032	1.05	2290.06	318.05	0.07
1	2000	2400.00	1298.00	1306.00	1299.53	1306.02	0.000043	1.20	2008.00	283.00	0.08


Sedimentation Basin Plan: Plan 01
RS = 2300
.03
.03
.03
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)


Sedimentation Basin Plan: Plan 01
RS = 2200
.03
.03
.03
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)


Sedimentation Basin Plan: Plan 01
RS = 2100
.03
.03
.03
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)

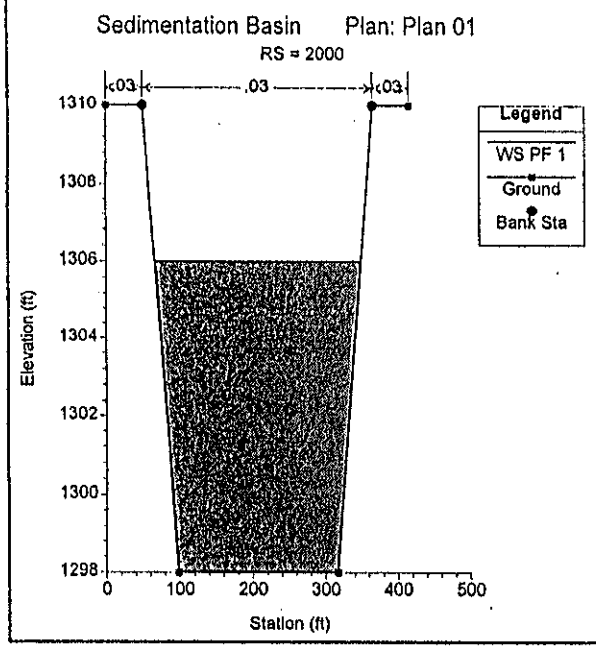
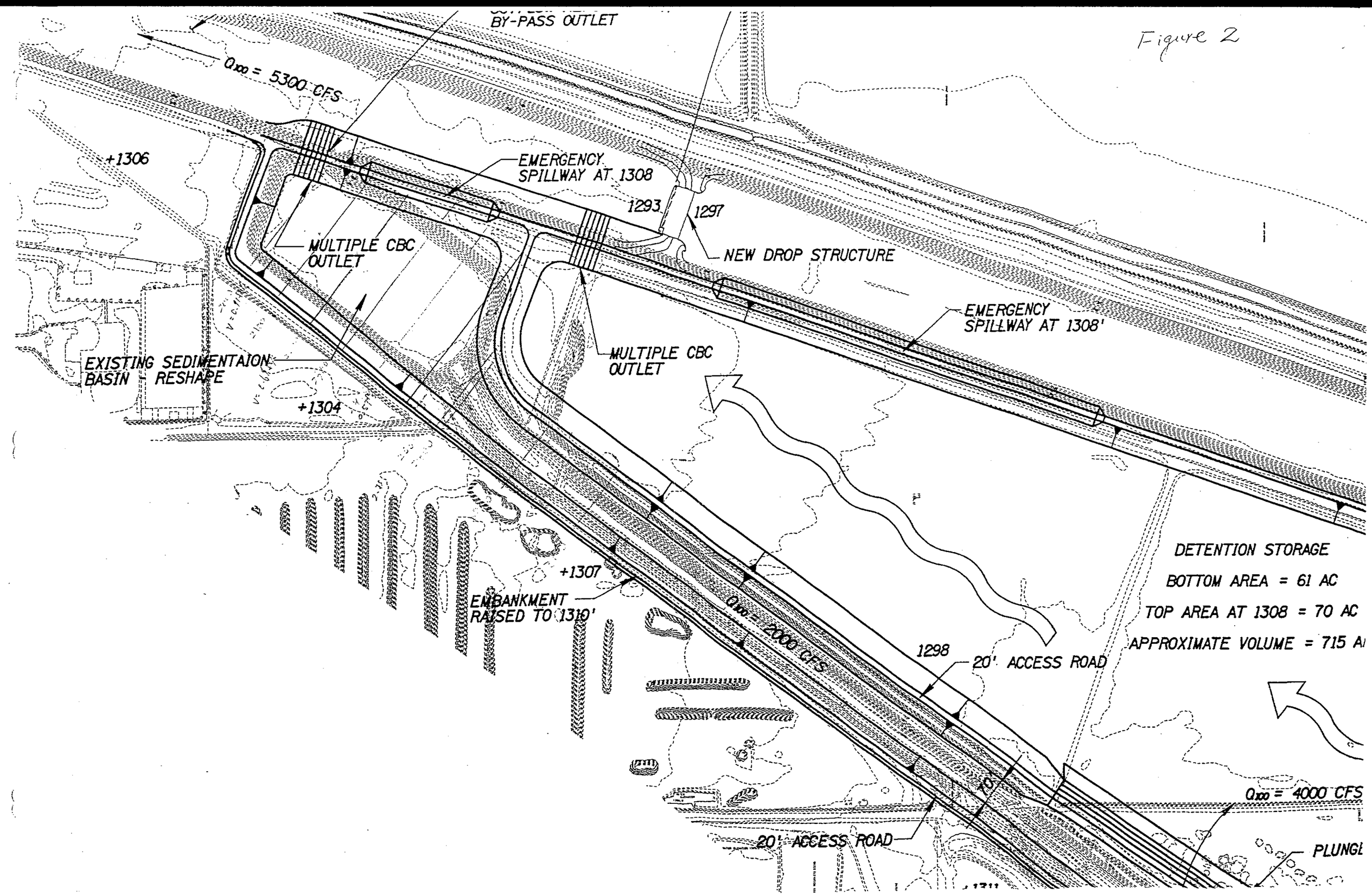

Sedimentation Basin Plan: Plan 01
RS = 2000
.03
.03
.03
Legend
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)

Figure 2

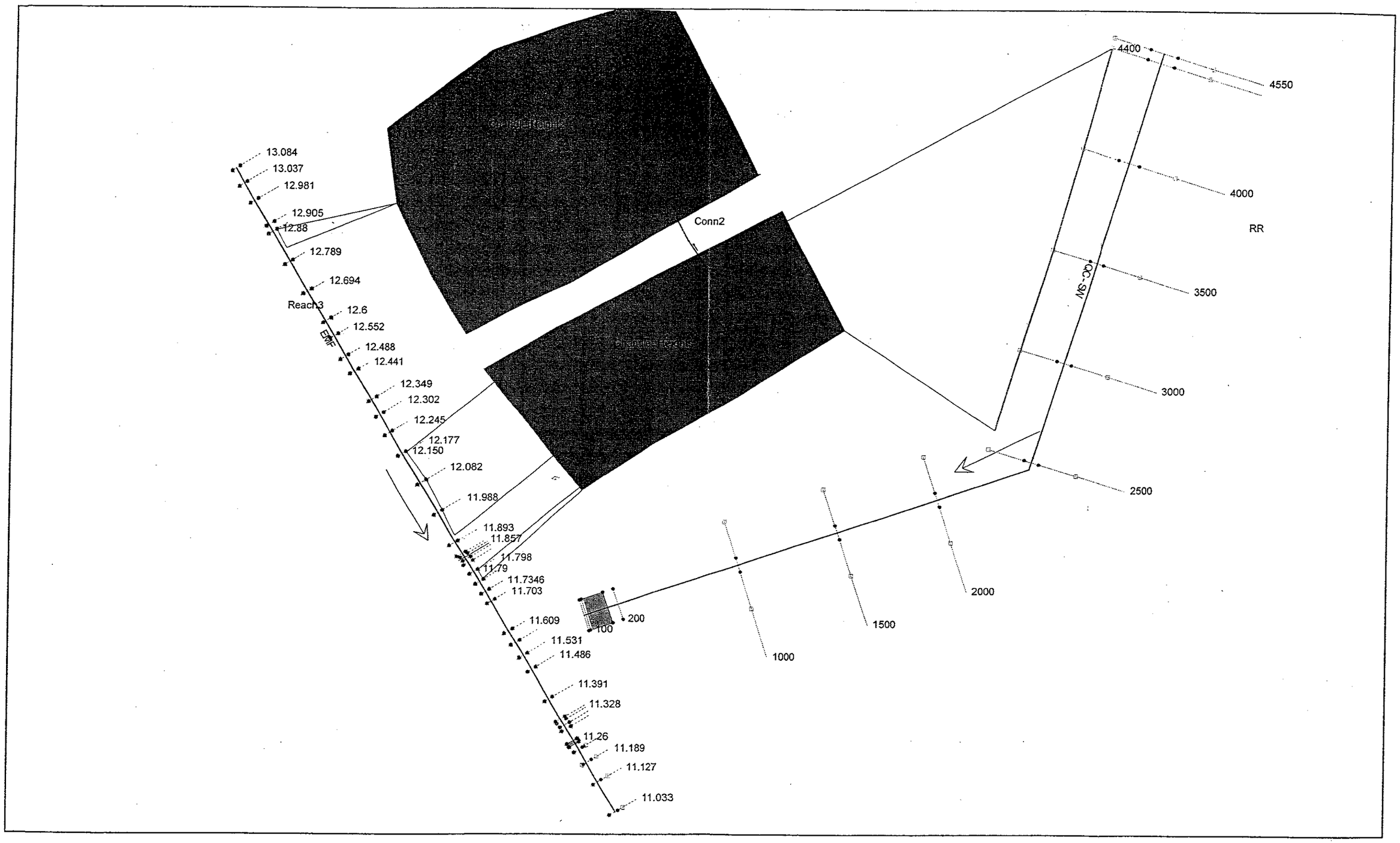

BY-PASS OUTLET
Q100 = 5300 CFS
+1306
EMERGENCY SPILLWAY AT 1308
1293
1297
NEW DROP STRUCTURE
MULTIPLE CBC OUTLET
EMERGENCY SPILLWAY AT 1308'
MULTIPLE CBC OUTLET
EXISTING SEDIMENTAION BASIN RESHAPE
+1304
+1307
EMBANKMENT RAISED TO 1310'
Q100 = 2000 CFS
1298
20' ACCESS ROAD
70'
20' ACCESS ROAD
Q100 = 4000 CFS
PLUNGE
DETENTION STORAGE
BOTTOM AREA = 61 AC
TOP AREA AT 1308 = 70 AC
APPROXIMATE VOLUME = 715 A

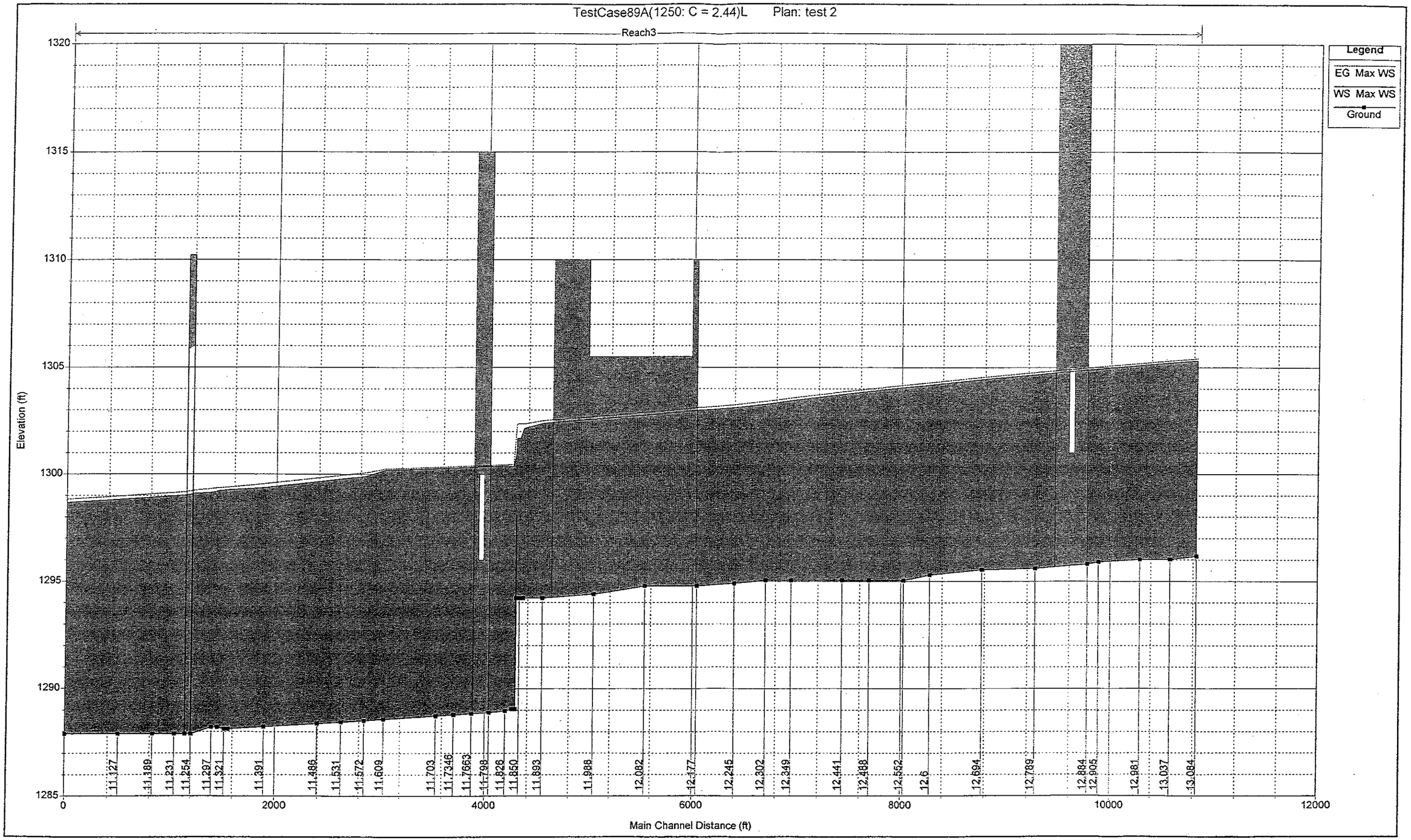
HEC-RAS Plan: T2 Profile: Max WS

River	Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
EMF	Reach3	13.084	Max WS	3386.07	1296.17	1305.32		1305.41	0.000420	2.51	1349.33	234.30	0.18
EMF	Reach3	13.037	Max WS	3385.93	1296.02	1305.22		1305.31	0.000386	2.44	1386.86	236.00	0.18
EMF	Reach3	12.981	Max WS	3385.74	1296.02	1305.10		1305.19	0.000424	2.52	1345.27	233.99	0.18
EMF	Reach3	12.905	Max WS	3385.43	1295.90	1304.93		1305.03	0.000421	2.52	1342.63	231.70	0.18
EMF	Reach3	12.884	Max WS	3385.33	1295.80	1304.88		1304.98	0.000426	2.52	1343.55	234.27	0.19
EMF	Reach3	12.88		Lat. Weir									
EMF	Reach3	12.789	Max WS	3384.82	1295.60	1304.66		1304.77	0.000436	2.55	1328.50	232.66	0.19
EMF	Reach3	12.694	Max WS	3382.00	1295.54	1304.44		1304.54	0.000469	2.60	1302.04	233.32	0.19
EMF	Reach3	12.6	Max WS	3381.86	1295.29	1304.18		1304.30	0.000515	2.69	1258.00	231.48	0.20
EMF	Reach3	12.552	Max WS	3381.74	1295.04	1304.06		1304.17	0.000489	2.64	1279.99	231.49	0.20
EMF	Reach3	12.488	Max WS	3381.52	1295.04	1303.89		1304.00	0.000531	2.71	1250.02	232.67	0.21
EMF	Reach3	12.441	Max WS	3381.32	1295.04	1303.74		1303.86	0.000583	2.80	1209.55	229.95	0.21
EMF	Reach3	12.349	Max WS	3379.19	1295.04	1303.45		1303.57	0.000614	2.87	1179.22	224.15	0.22
EMF	Reach3	12.302	Max WS	3379.14	1295.04	1303.28		1303.41	0.000672	2.95	1146.69	223.51	0.23
EMF	Reach3	12.245	Max WS	3379.04	1294.90	1303.11		1303.25	0.000413	2.98	1135.38	219.80	0.23
EMF	Reach3	12.177	Max WS	3378.86	1294.78	1302.98		1303.11	0.000379	2.87	1178.87	226.58	0.22
EMF	Reach3	12.150		Lat. Weir									
EMF	Reach3	12.082	Max WS	3378.55	1294.78	1302.77		1302.91	0.000430	2.98	1132.66	225.47	0.23
EMF	Reach3	11.988	Max WS	3378.18	1294.40	1302.58		1302.70	0.000378	2.86	1182.38	227.86	0.22
EMF	Reach3	11.893	Max WS	3376.90	1294.21	1302.38		1302.51	0.000408	2.92	1154.82	227.43	0.23
EMF	Reach3	11.857	Max WS	3376.86	1294.21	1302.14		1302.35	0.001288	3.67	920.41	215.82	0.31
EMF	Reach3	11.850	Max WS	3376.85	1294.21	1301.67	1300.59	1302.36	0.000634	6.66	507.08	141.72	0.62
EMF	Reach3	11.845		Inline Weir									
EMF	Reach3	11.838	Max WS	3377.05	1289.05	1300.34		1300.45	0.000041	2.66	1269.40	179.50	0.18
EMF	Reach3	11.826	Max WS	3377.02	1288.96	1300.39		1300.45	0.000181	1.94	1737.89	234.09	0.13
EMF	Reach3	11.798	Max WS	3376.93	1288.88	1300.36		1300.42	0.000175	1.96	1727.27	218.22	0.12
EMF	Reach3	11.79		Lat. Weir									
EMF	Reach3	11.7663	Max WS	3376.84	1288.83	1300.33		1300.39	0.000174	1.95	1731.73	218.33	0.12
EMF	Reach3	11.7346	Max WS	3376.75	1288.78	1300.30		1300.36	0.000172	1.94	1736.78	218.50	0.12
EMF	Reach3	11.703	Max WS	3376.67	1288.73	1300.28		1300.33	0.000171	1.94	1741.54	218.62	0.12
EMF	Reach3	11.609	Max WS	3376.43	1288.58	1300.19		1300.25	0.000168	1.92	1760.30	222.23	0.12
EMF	Reach3	11.572	Max WS	5693.56	1288.52	1299.90		1300.08	0.000500	3.34	1705.89	217.63	0.21
EMF	Reach3	11.531	Max WS	5693.41	1288.45	1299.79		1299.96	0.000526	3.36	1696.58	217.38	0.21
EMF	Reach3	11.486	Max WS	5693.23	1288.38	1299.66		1299.84	0.000540	3.38	1685.65	218.40	0.21
EMF	Reach3	11.391	Max WS	5692.84	1288.23	1299.38		1299.56	0.000570	3.44	1657.13	217.77	0.22
EMF	Reach3	11.328	Max WS	5692.54	1288.13	1299.27		1299.40	0.000389	2.92	1951.44	251.16	0.18
EMF	Reach3	11.321	Max WS	5692.50	1288.12	1299.25		1299.39	0.000389	2.92	1950.50	251.15	0.18
EMF	Reach3	11.308	Max WS	5692.44	1288.21	1299.23		1299.36	0.000304	2.92	1948.03	251.12	0.18
EMF	Reach3	11.297	Max WS	5692.38	1288.21	1299.15		1299.34	0.000629	3.51	1623.79	230.69	0.23
EMF	Reach3	11.26	Max WS	5692.19	1287.90	1299.09	1293.75	1299.23	0.000452	3.05	1866.73	254.99	0.20
EMF	Reach3	11.254		Bridge									
EMF	Reach3	11.249	Max WS	5691.41	1287.90	1299.02		1299.18	0.000502	3.17	1796.72	251.38	0.21

HEC-RAS Plan: T2 Profile: Max WS (Continued)

River	Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
EMF	Reach3	11.231	Max WS	5691.36	1287.90	1298.99		1299.14	0.000272	3.06	1862.47	253.63	0.20
EMF	Reach3	11.189	Max WS	5691.25	1287.90	1298.92		1299.08	0.000299	3.15	1804.23	251.64	0.21
EMF	Reach3	11.127	Max WS	5691.08	1287.90	1298.83		1298.98	0.000292	3.13	1819.94	252.01	0.21
EMF	Reach3	11.033	Max WS	5690.83	1287.90	1298.67	1293.55	1298.83	0.000301	3.18	1792.09	249.05	0.21
QC-SW	RR	4550	Max WS	5535.00	1301.07	1308.53		1308.69	0.000472	3.23	1713.83	259.72	0.22
QC-SW	RR	4500	Max WS	5534.80	1301.05	1308.50		1308.67	0.000473	3.23	1712.79	259.62	0.22
QC-SW	RR	4400		Lat. Weir									
QC-SW	RR	4000	Max WS	4086.05	1300.90	1308.34		1308.51	0.000536	3.33	1226.00	194.53	0.23
QC-SW	RR	3500	Max WS	2884.33	1300.75	1308.05		1308.30	0.000892	3.98	724.54	128.42	0.30
QC-SW	RR	3000	Max WS	2354.27	1300.60	1307.83		1308.00	0.000617	3.29	715.19	127.84	0.25
QC-SW	RR	2500	Max WS	2348.19	1300.45	1307.50		1307.68	0.000674	3.39	691.93	126.38	0.26
QC-SW	RR	2000	Max WS	2348.07	1300.30	1307.12		1307.32	0.000759	3.54	663.95	124.59	0.27
QC-SW	RR	1500	Max WS	2347.72	1300.15	1306.69		1306.91	0.000886	3.73	629.13	122.34	0.29
QC-SW	RR	1000	Max WS	2345.35	1300.00	1306.12		1306.39	0.001173	4.18	560.46	113.10	0.33
QC-SW	RR	200	Max WS	2345.00	1298.00	1305.92		1305.93	0.000036	0.98	2396.29	305.28	0.06
QC-SW	RR	150	Max WS	2344.95	1298.00	1305.92		1305.93	0.000036	0.98	2396.13	305.31	0.06
QC-SW	RR	120		Culvert									
QC-SW	RR	100	Max WS	2344.75	1297.00	1298.41		1298.89	0.010872	5.54	423.60	300.84	0.82
QC-SW	RR	60	Max WS	2344.75	1296.90	1298.31	1298.14	1298.79	0.010872	5.54	423.60	300.84	0.82


13.084
13.037
12.981
12.905
12.88
12.789
12.694
Reach3
12.6
12.552
12.488
12.441
12.349
12.302
12.245
12.177
12.150
12.082
11.988
11.893
11.857
11.798
11.79
11.7346
11.703
11.609
11.531
11.486
11.391
11.328
11.26
11.189
11.127
11.033
Conn2
4400
4550
4000
RR
3500
3000
2500
2000
1500
1000
200
100


TestCase89A(1250: C = 2.44)L Plan: test 2
Reach3
Legend
EG Max WS
WS Max WS
Ground
Elevation (ft)
1320
1315
1310
1305
1300
1295
1290
1285
0
2000
4000
6000
8000
10000
12000
Main Channel Distance (ft)
11.127
11.189
11.231
11.254
11.297
11.321
11.391
11.486
11.531
11.572
11.609
11.703
11.7346
11.7663
11.798
11.826
11.850
11.893
11.988
12.082
12.177
12.245
12.302
12.349
12.441
12.488
12.552
12.6
12.694
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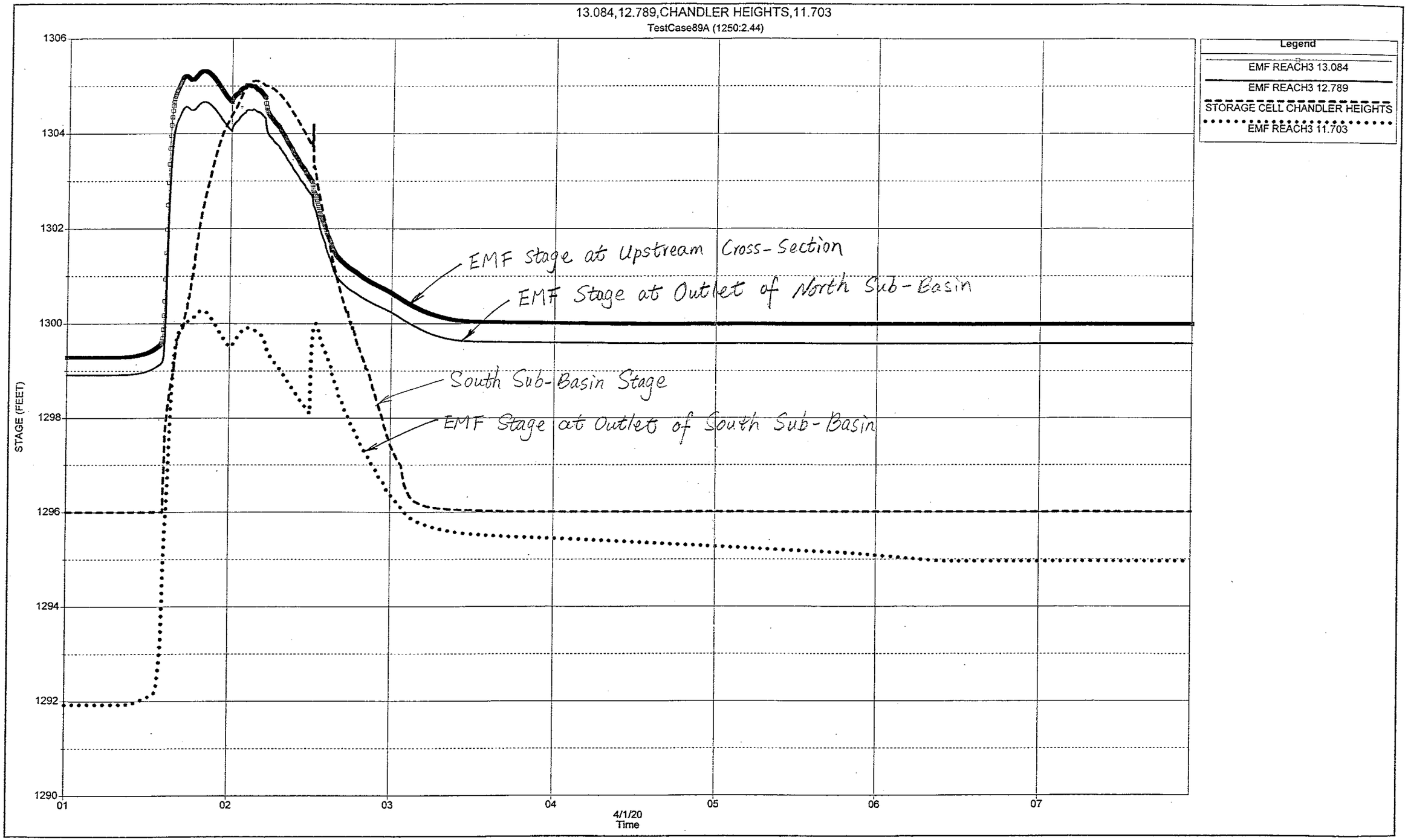
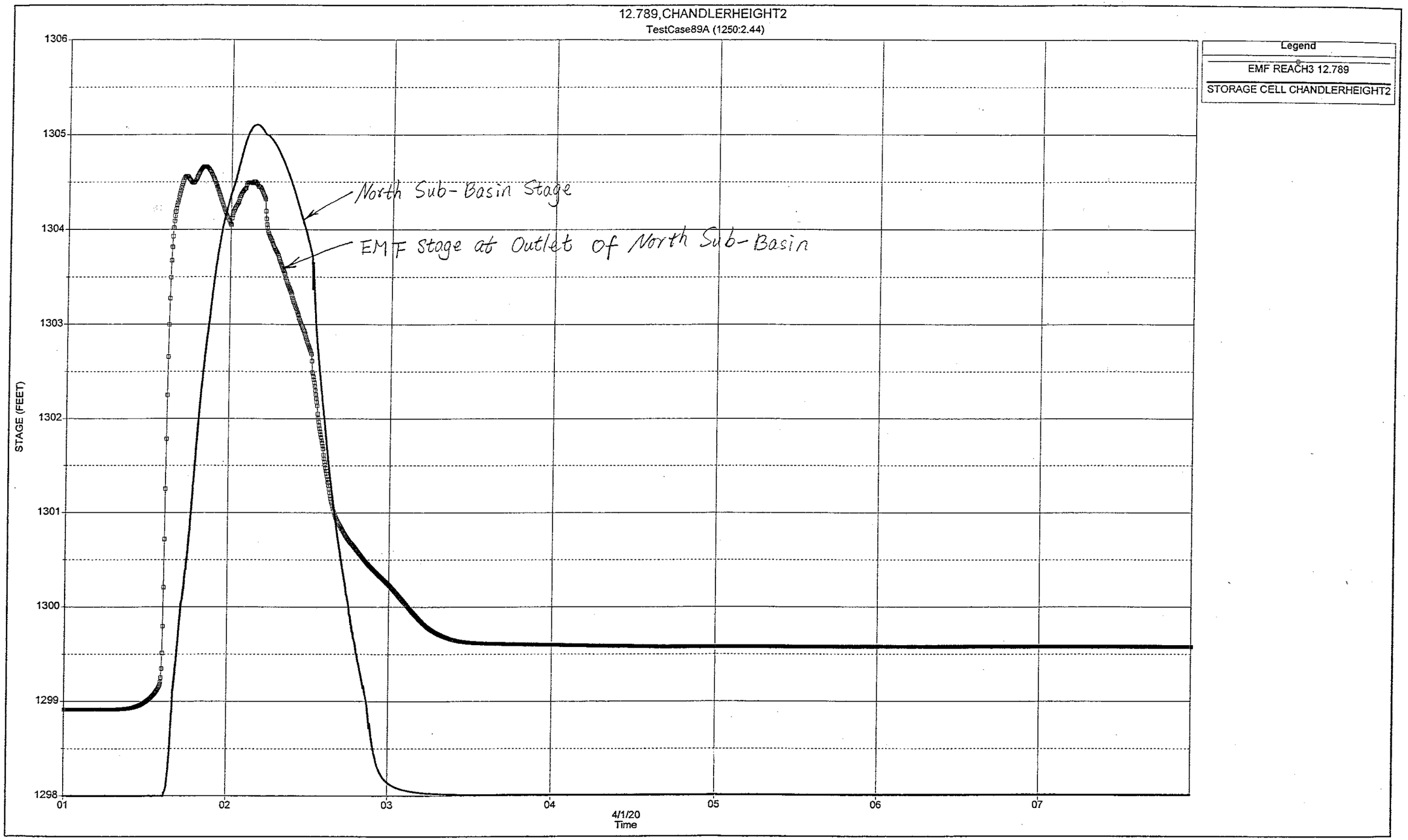
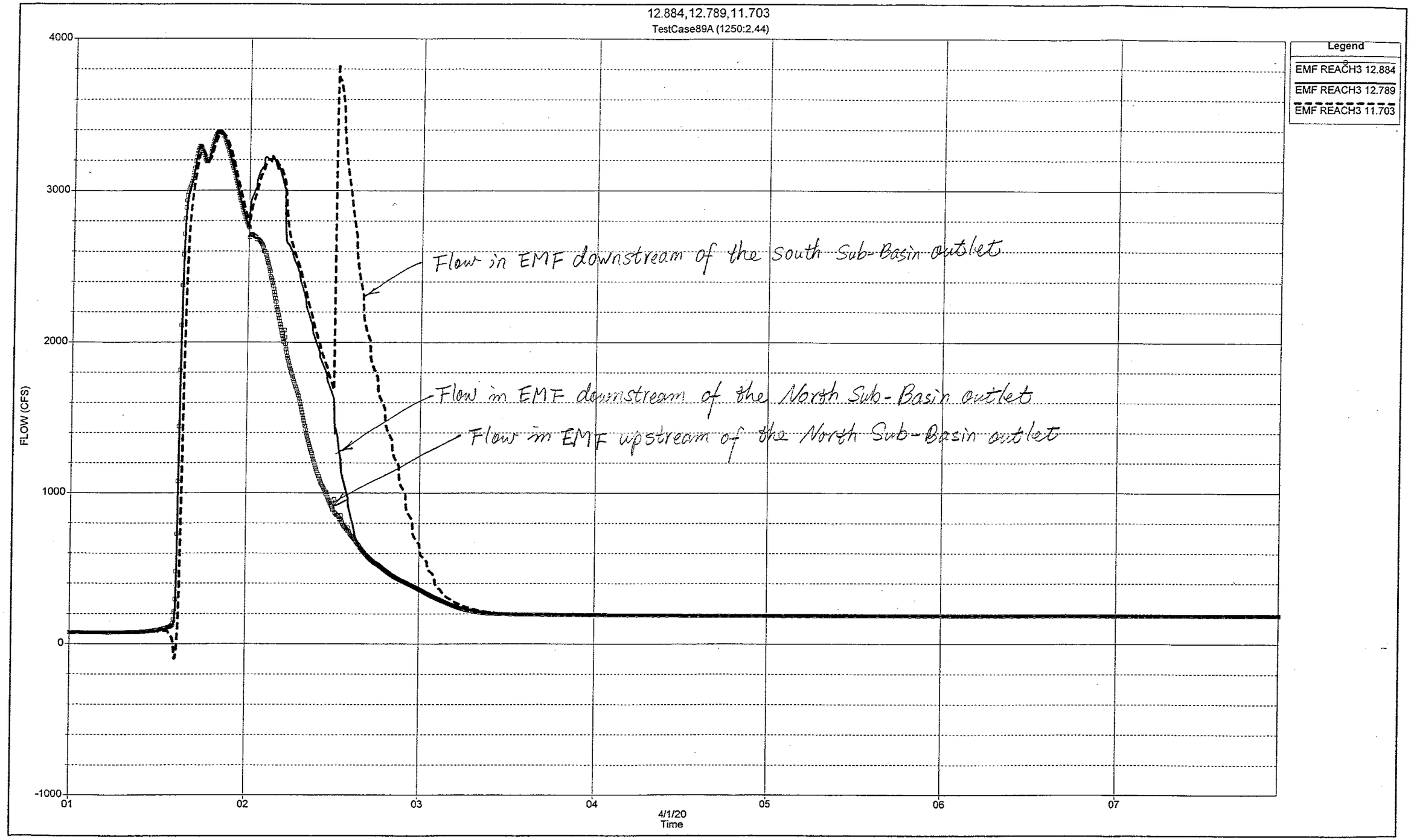
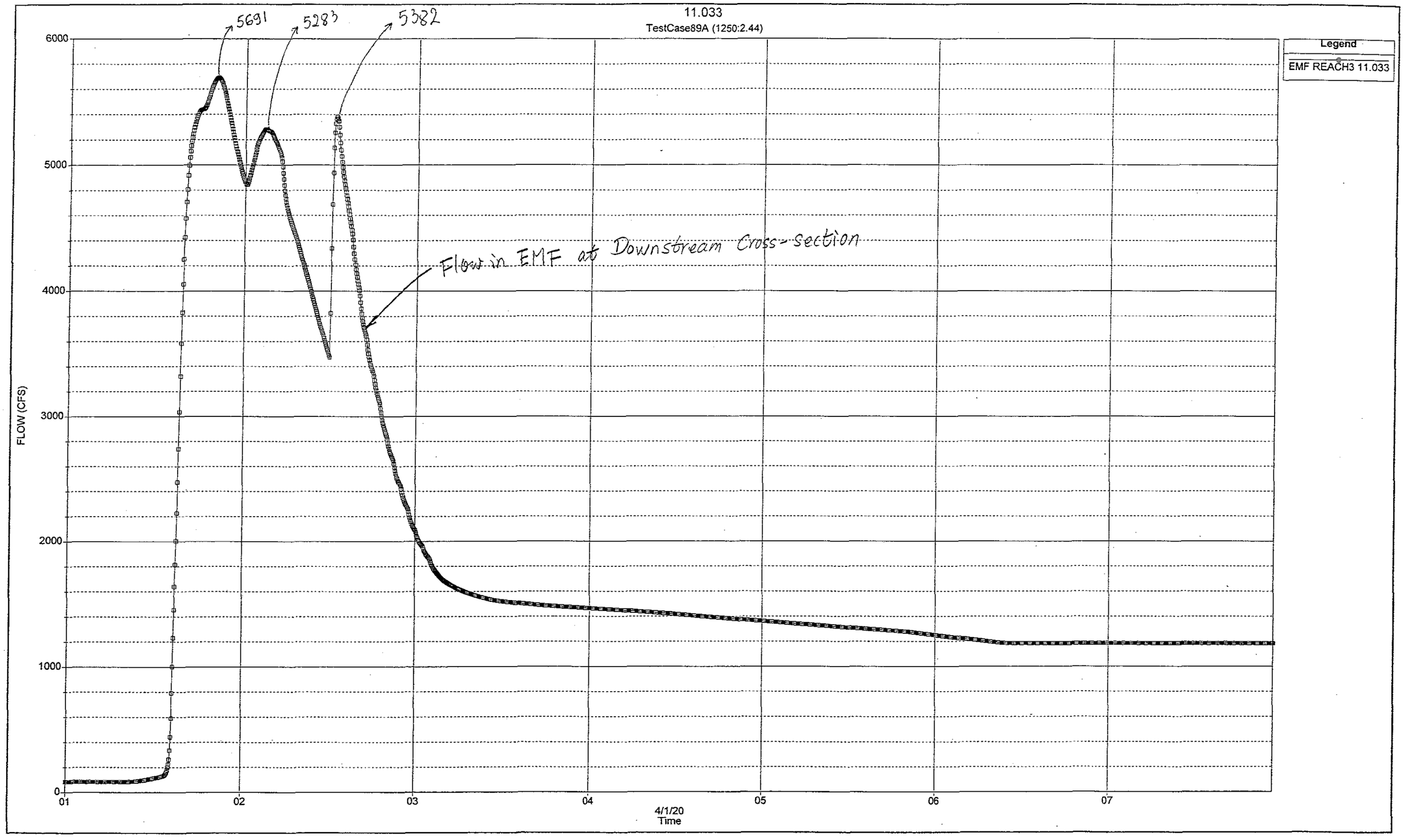
12.884
12.905
12.981
13.037
13.084


13.084,12.789,CHANDLER HEIGHTS,11.703
TestCase89A (1250:2.44)
Legend
EMF REACH3 13.084
EMF REACH3 12.789
STORAGE CELL CHANDLER HEIGHTS
EMF REACH3 11.703
STAGE (FEET)
1306
1304
1302
1300
1298
1296
1294
1292
1290
01
02
03
04
05
06
07
4/1/20
Time
EMF Stage at Upstream Cross-Section
EMF Stage at Outlet of North Sub-Basin
South Sub-Basin Stage
EMF Stage at Outlet of South Sub-Basin


12.789,CHANDLERHEIGHT2
TestCase89A (1250:2.44)
Legend
EMF REACH3 12.789
STORAGE CELL CHANDLERHEIGHT2
North Sub-Basin Stage
EMF Stage at Outlet of North Sub-Basin
STAGE (FEET)
1306
1305
1304
1303
1302
1301
1300
1299
1298
01
02
03
04
05
06
07
4/1/20
Time


12.884,12.789,11.703
TestCase89A (1250:2.44)
Legend
EMF REACH3 12.884
EMF REACH3 12.789
EMF REACH3 11.703
FLOW (CFS)
4000
3000
2000
1000
0
-1000
01
02
03
04
05
06
07
4/1/20
Time
Flow in EMF downstream of the South Sub-Basin outlet
Flow in EMF downstream of the North Sub-Basin outlet
Flow in EMF upstream of the North Sub-Basin outlet


11.033
TestCase89A (1250:2.44)
5691
5283
5382
Flow in EMF at Downstream Cross-section
Legend
EMF REACH3 11.033
FLOW (CFS)
6000
5000
4000
3000
2000
1000
0
01
02
03
04
05
06
07
4/1/20
Time

GEOTECHNICAL ENGINEERING REPORT

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INITIAL GEOTECHNICAL EVALUATION
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

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January 4, 2002
Project No. 600198001

Ninyo & Moore
Geotechnical and Environmental Sciences Consultants

January 4, 2002
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
Chandler Heights 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 preliminary 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, P.E.
Senior Project Engineer

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

SDN/MDE/RM/avv

Distribution: (2) Addressee

Registered Professional Engineer (Civil)
CERTIFICATE NO. 34866
STEVEN D. NOWACZYK
Date Signed 1-4-02
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 Chandler Heights 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 26 small-diameter exploratory borings to depths of about 16 to 33 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 12 feet bgs. The test pit logs are also presented in Appendix A.
- Performing six field infiltration tests at the anticipated bottom-of-basin level, in general accordance with the City of Chandler method. The results of this testing 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 preliminary recommendations regarding the design and construction of the new basin.

3. SITE DESCRIPTION

The project site is located in Sections 15 and 22 of Township 2 South, Range 6 East. The project area covers approximately 320 acres of land and is situated in the Town of Gilbert, Arizona. The project area is bounded by Higley Road to the east, Queen Creek Road to the north, Queen Creek Wash to the southwest, and the EMF to the west, and is depicted on the Site Location Map (Figure 1).

At the time of our evaluation, the project area was vacant. Farming apparently occurred on the site in the past, particularly in its northern and extreme southern portions. Scattered trees, small brush, and weeds were observed during our site visits. In addition, several unpaved roads randomly crossed the site, although there were several unpaved roads that are on east-west alignments. One of these appeared to coincide with the alignment of Ocotillo Road in the south-central portion of the project area and is also coincident with an existing east-west aligned fence line. Some scattered piles of soil were also observed. We understand that some spoils from the original construction of the EMF were dumped and 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 1,315 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 east to the west, toward the EMF, with a vertical relief of less than 10 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* indicated a denser vegetation and small shrubs across the central and northern portion of the project area than exists at the site presently. In addition, a small area of row crops is recorded in the same photograph near the extreme 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. We also observed some public use of recreational vehicles during our field activities within the project area, with associated trails tracking across the project area.

4. PROPOSED CONSTRUCTION

The project generally includes the construction of a new detention basin along the southeast side of the EMF, from Queen Creek Road to just north of Chandler Heights Road. The basin will collect runoff from Queen Creek and Sanokai Wash, temporarily retain the water and ultimately discharge it into the EMF. The basin elevation will vary from about 1,300 feet above MSL near the north side, to about 1,296 feet above MSL near the south side. Consequently, the excavations needed to create the basin area will extend to about 10 to 20 feet bgs. Natural soil berms, which will act as a levees, will be created (from the excavation) between the basin and the EMF and Queen Creek. Based on our conversations with Kirkham Michael Consulting Engineers and the Flood Control District of Maricopa County, we understand that this construction is not considered a jurisdictional dam because the impounded water will be situated below to existing ground surface.

A segment of the Queen Creek channel, from its crossing under Higley Road to the existing sedimentation basin to the north of Chandler Heights Road, will be improved. Specifically, Queen Creek will be widened to 100 feet from Higley Road to about the alignment of Ocotillo Road. Immediately to the south of the Ocotillo Road alignment, Queen Creek will be widened to 200 feet and then gradually taper over about 2,500 linear feet to a channel width of 70 feet. This width will be maintained until it terminates into the sedimentation basin. The vertical alignment of the creek bottom will also be improved. From Higley Road to the Ocotillo Road alignment, there will be four drop structures, each 3 feet high, that will lower the level of the creek from about 1,315 to 1,303 feet above MSL. As a result of the improvements to Queen Creek, some of the natural soil berm created between Queen Creek and the EMF, specifically about the top 2 to 3

feet, will extend above the existing ground surface. This portion of the levee will be constructed of new engineered fill.

The sediment basin at the end of Queen Creek will be re-shaped to accommodate the anticipated water volumes. The existing sediment basin outflow, located near the southwest corner of the basin, will be replaced with 13, 4 feet wide by 4 feet high concrete box culverts (CBC). The discharge of these CBCs will be controlled with one-way flap gates.

A 1,800-foot long, concrete side weir will be constructed along the northwest side of Queen Creek, specifically where it tapers from 200 feet to 70 feet wide. This side weir will enable water to enter the basin from Queen Creek. The side weir crest elevation is tentatively planned to be at about 1,307 feet above MSL.

To allow the water to transfer into the EMF, two outlets are planned. One outlet will be located near the southwestern tip of the basin and will consist of 12, 4 feet wide by 4 feet high CBCs. This outlet will be controlled with manual gates. The other outlet will be located about 2,700 feet south of the EMF intersection with Queen Creek Road and will consist of 13, 4 feet wide by 4 feet high CBCs. This outlet will be controlled with one-way flap gates.

Two emergency spillways will be constructed. One will be located between the new basin and the EMF, near the southwestern most tip of the basin (just north of the new outlet structure), and the other will be located between the sediment basin and the EMF. These emergency spillways will consist of concrete-surfaced embankments.

The side slopes around the perimeter of the basin are proposed to be constructed with a 4 vertical to 1 horizontal slope. The land use within the new basin is tentatively planned to accommodate multiple-use facilities, and could include several baseball and soccer fields. A small portion of the basin located along the Ocotillo Road alignment will not be excavated. This area is reserved for future roadway development.

5. FIELD EXPLORATION

5.1. Soil Borings

Ninyo & Moore conducted a subsurface evaluation at the site between July 11 and July 19, 2001 in order to evaluate the existing subsurface conditions and to collect soil samples for laboratory testing. Our evaluation consisted of the excavating, logging, and sampling of 26 small-diameter borings. The borings were drilled using a CME-75 truck-mounted drill rig. Of these borings, ten were drilled along the EMF perimeter (denoted as CH-1 through CH-10), nine were drilled along the Queen Creek perimeter (denoted as CH-11 through CH-19), one was drilled along the Queen Creek Road perimeter (denoted as CH-20), and six were drilled within the new basin area (denoted as CH-21 through CH-26). 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 determined 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 each 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 on November 26 and 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, three of the boreholes were completed as piezometers. Specifically, the piezometers were installed in borings CH-2, CH-6, and CH-9. In general, the bottom half of the piezometer well casing

consisted of bottom capped screened PVC, and the top half solid impermeable PVC. The annuli around the wells were backfilled with permeable sand and grouted near the ground surface using cement-bentonite slurry. The above ground exposures of the well casings were enclosed and capped with an above-ground lockable protective steel casing.

No substantial rainfall occurred during our study period and no meaningful readings were taken; however, the wells were left in-place. Consequently, if a heavy sustained 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 six infiltration tests in general accordance with the City of Chandler Typical Detail No. C-109. This method was selected because it is commonly considered to be a standard throughout metropolitan Phoenix. These tests were performed near the central portion of the proposed basin at the site, adjacent to borings CH-21, CH-22, CH-23, CH-24, CH-25, and CH-26. The procedures used consisted of the insertion of a 12-inch diameter solid riser into undisturbed soil, to a depth of approximately 15 to 20 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 in the field as drilling progressed 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 portion of each collected soil sample, sealed, the soil disturbed,

and allowed to volatilize for 10 minutes. The tip of the PID was then inserted into the head-space 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 detention basin is described as latest Quaternary age deposits (<10,000 years old) and Pleistocene deposits (<250,000 years old) consisting of sand, clay, and silt with local occurrences of fine gravels and coarse deposits that contain minimal to moderate soil development (Pearthree, 1994).

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), clayey/silty sand (SC/SM) and clayey/silty fine to coarse gravel (GP/GC/GM). Scattered caliche nodules, filaments, and stringers were present in many of the borings. Table 1 provides a 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
2%	0%	16%	7%	75%

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
8%	0%	50%	8%	34%

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 1,286 to 1,308 feet MSL. Localized variations may be greater than the given range and are largely attributable to erosion of the Late Pleistocene surface.

7.3. Groundwater

Groundwater was not encountered in our boring 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 preliminary recommendations of this report are incorporated into the design and construction of the proposed project, as appropriate. Geotechnical considerations include 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 primary conclusions was that a crack-stopper barrier (located within the levee between the basin and the EMF and Queen Creek) would alleviate several of the potential failure modes discussed.
- We recommend that the side weir (and possibly the emergency spillway) 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 1,286 to 1,308 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. PRELIMINARY 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 26 widely spaced exploratory borings and three test pits, 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 1,286 to 1,308 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.

A 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 5 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 conducted to classify these soils, we performed Expansion Index and corrosivity tests as a means to evaluate these soils for potential reuse. Table 3 outlines the results of these tests. 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 possible.

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)
CH-11	0-2	1.5	7.6	508	0.0025	160
CH-21	12-15	6	8.4	1,320	0.0004	10
CH-23	0-2	1.5	--	--	--	--
CH-25	12-15	0	--	--	--	--

The Expansion Index test is used to evaluate the intrinsic swell or expansion potential of a remolded soil sample upon saturation 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 7.6 to 8.4, which is considered to be alkaline. The minimum electrical resistivity measured in the laboratory varied from 508 to 1,320 ohm-cm, which is considered to be corrosive to ferrous materials. The chloride content of the sample tested ranged from about 10 to 160 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 0.0004 to 0.0025 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 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 Queen Creek. Levees are usually constructed with select materials that are placed in an engineered 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 likely 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 levee 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 these levees 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 establishing 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 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 Queen Creek (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 Queen Creek) 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 to about 15 to 17 feet bgs. Table 4 summarizes these results of these percolation tests.

Table 4 – Summary of Percolation Tests Within Chandler Heights Basin

Test Location	Test Depth (ft)	Average Percolation Rate ($ft^3/hr/ft^2$)	Soil Type at Test Depth
near CH-21	20	0.91	SM
near CH-22	20	0.33	CL
near CH-23	15	0.10	CL
near CH-24	20	0.30	CL
near CH-25	20	0.13	SM
near CH-26	15	0.07	GM

The reported 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. New Levee Construction

As a result of the proposed improvements to Queen Creek, some of the natural levee created between Queen Creek and the EMF, specifically about the top 2 to 3 feet, will extend above the existing ground surface. Consequently, this layer will be engineered and compacted in lifts.

We recommend that the new fill needed for this top segment of the levee 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. Selected low permeability on-site soils could be reused for this purpose. We recommend that this segment be keyed into the native soils.

9.4. Side Weir and Outlet Works

As mentioned earlier, we understand that a 1,800-foot long, concrete side weir will be constructed along the northwest side of Queen Creek, specifically where it tapers in width from 200 feet to 70 feet. The side weir crest elevation is tentatively planned to be at about 1,307 feet above MSL. To allow the water to transfer into the EMF, three outfalls are planned. These outfalls are proposed to consist of multiple box culverts that will be incorporated into the natural levee, which will be created between the EMF and the new basin.

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

The conceptual drawings that we received also show two cut-off walls, located on either side of the side weir and extending about 6 feet below the bottom of the basin. We understand that these walls were employed to discourage undermining of the side weir by water flow, but will also provide a measure of piping and seepage control.

9.4.1. 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 1,286 to 1,308 feet MSL. Localized variations are largely attributable to erosion of the Late Pleistocene surface.

Relevant geologic information was shared during the FMA workshop. As a result, the presence of Holocene soils below the side 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 side weir (and possibly the emergency spillway) 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 side 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.

9.4.2. 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.

9.4.3. 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.

9.5. 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.

9.6. 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.

11. SELECTED REFERENCES

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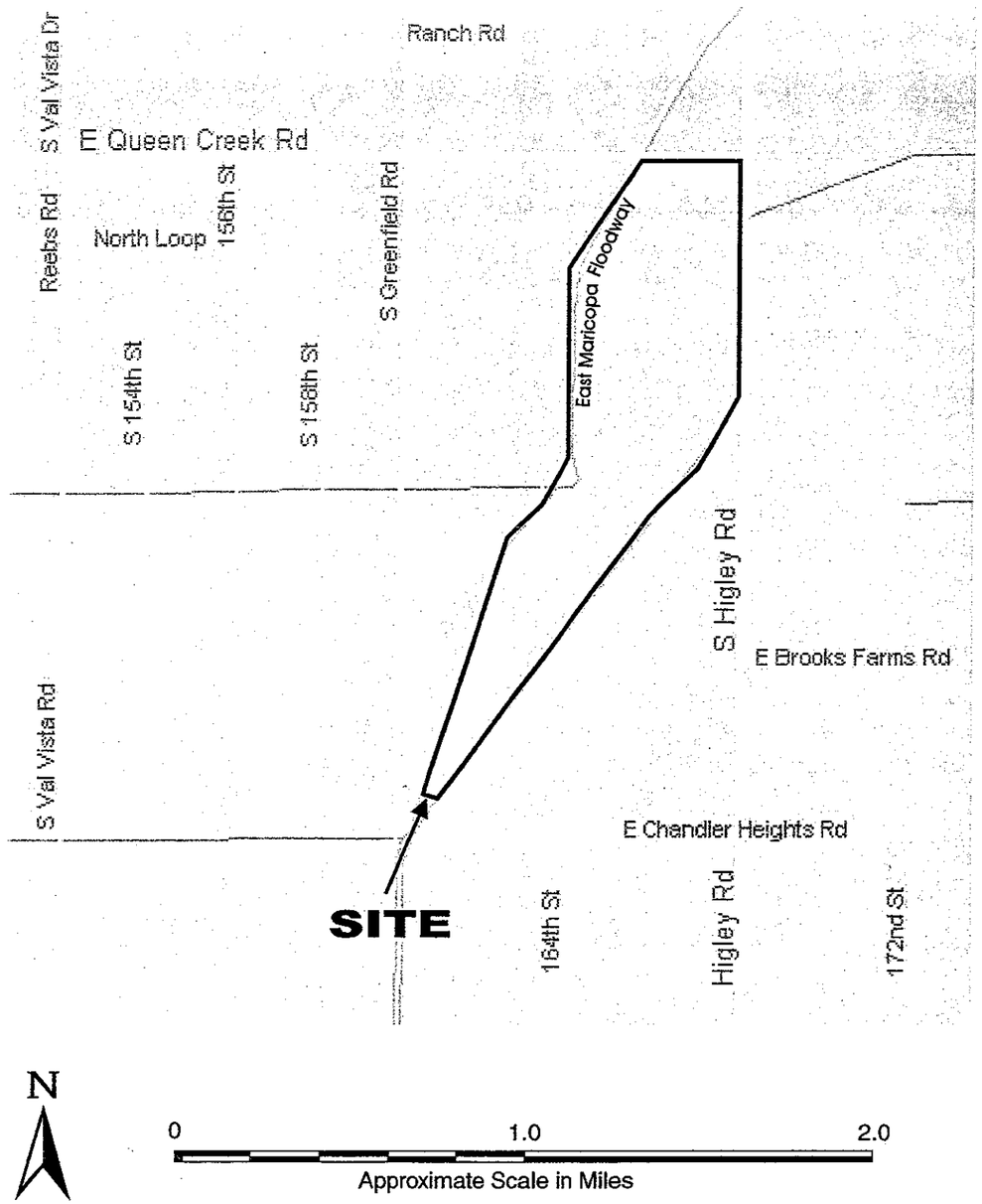

Source: Microsoft Streets98, 1998.

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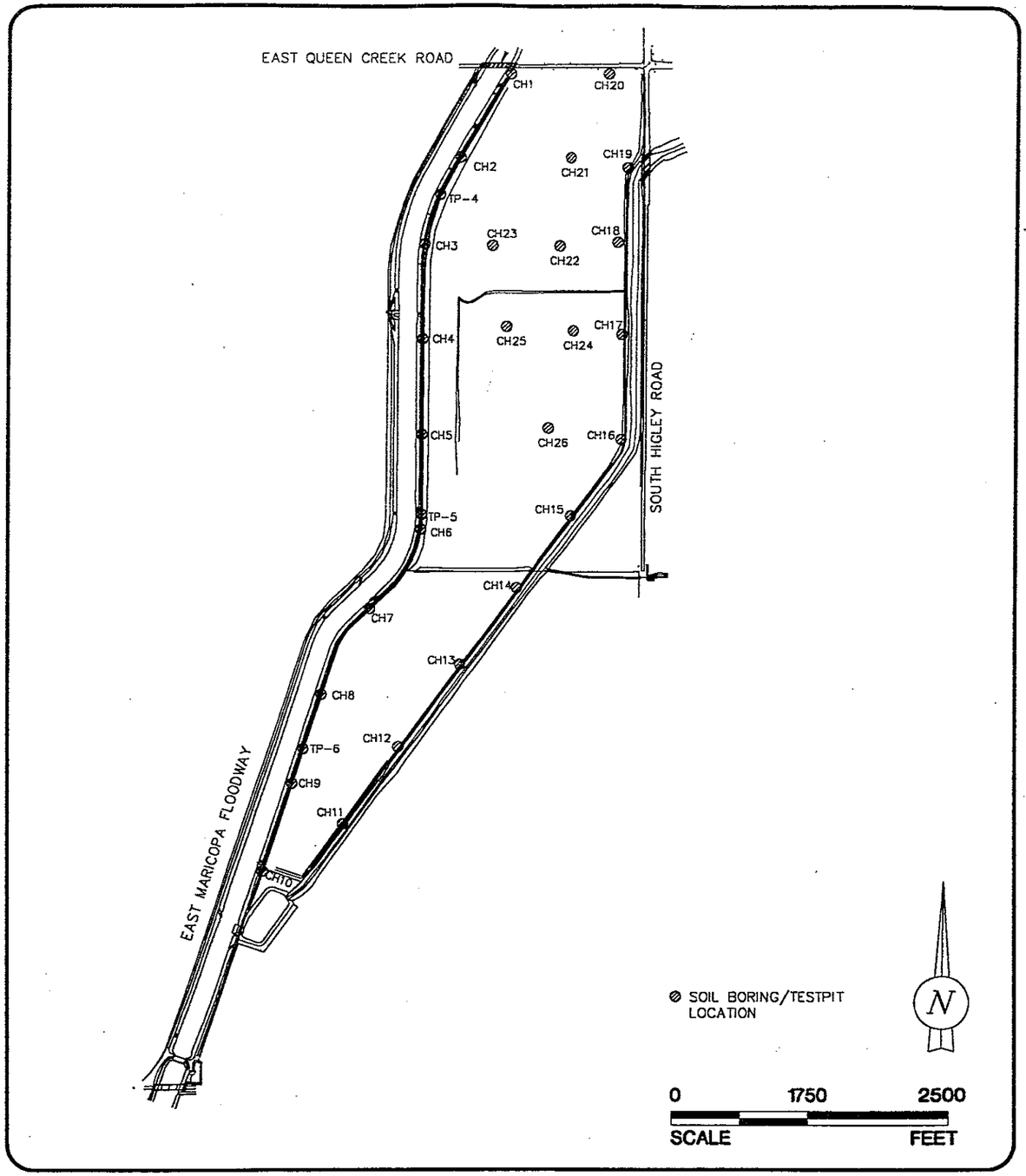
SITE LOCATION MAP

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	1



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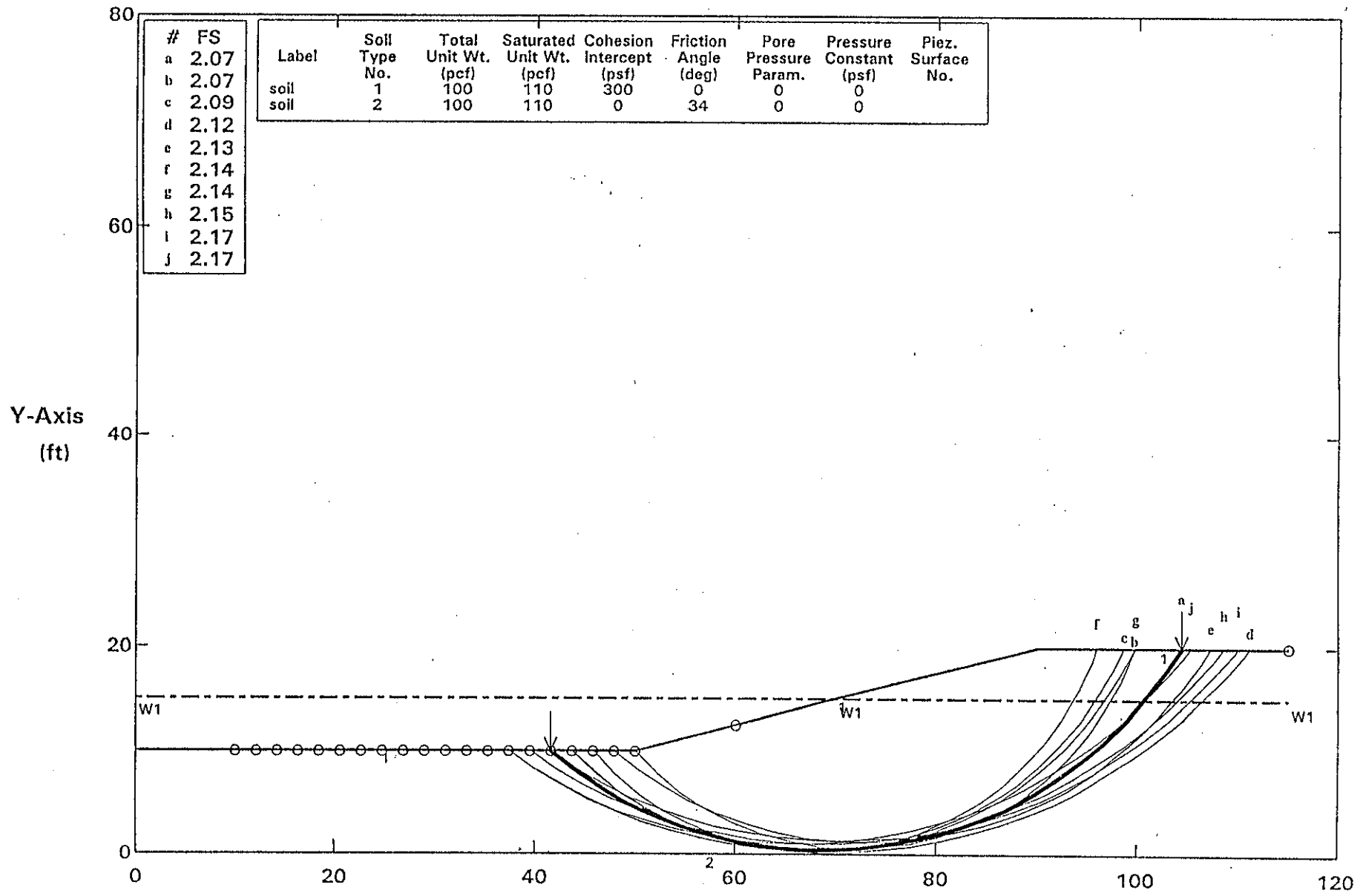
BORING AND TEST PIT LOCATION MAP

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT #	DATE	FIGURE
600198001	01/02	2

Figure 3: Slope Stability Analysis of Typical Embankment

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



#	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

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	

STABL6H FSmin = 2.07 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

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APPENDIX A

BORING 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.

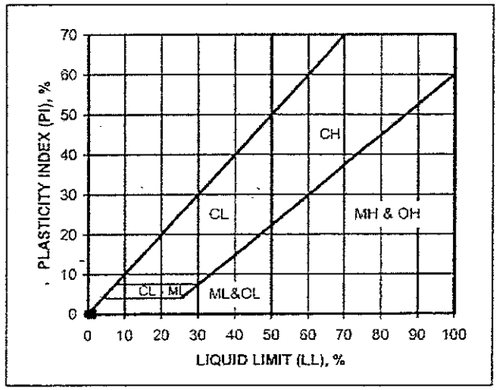
U.S.C.S. METHOD OF SOIL CLASSIFICATION

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	3" to No.4	76.2 to 4.76
Coarse	3" to 3/4"	76.2 to 19.1
Fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
Coarse	No. 4 to No. 10	4.76 to 2.00
Medium	No. 10 to No. 40	2.00 to 0.420
Fine	No. 40 to No. 200	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.	DESCRIPTION/INTERPRETATION
								DATE DRILLED _____ BORING NO. PATTERNS
								GROUND ELEVATION _____ SHEET 1 OF 2
								METHOD OF DRILLING _____
								DRIVE WEIGHT _____ DROP _____
								SAMPLED BY _____ LOGGED BY _____ REVIEWED BY _____
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
20								(9) = CONCRETE

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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.	DESCRIPTION/INTERPRETATION
								DATE DRILLED _____ BORING NO. PATTERNS
								GROUND ELEVATION _____ SHEET 2 OF 2
								METHOD OF DRILLING _____
								DRIVE WEIGHT _____ DROP _____
								SAMPLED BY _____ LOGGED BY _____ REVIEWED BY _____
20								(WATER) Water table during drilling.
								(FWATER) Water table at boring completion.
								(%) = CALICHE
								(.) = GYPSUM
								($) = SCHIST
								(7) = Mudstone
25								(() Dolomite
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)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	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.
							Standard Penetration Test (SPT).
10							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.
20							The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

EXPLANATION OF BORING LOG SYMBOLS

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

DATE DRILLED 7/10/01 BORING NO. CH-1

GROUND ELEVATION 1312' 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: Brown (7.5 YR 5/4), damp, loose to dense, clayey SILT. Stage I cementation, weakly cemented by calcium carbonate, weak reaction with HCL, sparse calcium carbonate filaments.
		42					
5		9	10.7				Medium dense.
		11	11.1	88.4			
10		26	8.9			SM	Pale brown (10 YR 6/3), dry to damp, dense, silty SAND; trace fine, subrounded gravel. Stage I cementation, weakly cemented by calcium carbonate, weak reaction with HCL, sparse calcium carbonate filaments.
		28	10.2	90.1		CL	Reddish brown (5 YR 5/4), damp, hard, silty CLAY. Stage I cementation, weakly cemented by calcium carbonate, weak reaction with HCL, sparse calcium carbonate filaments.
15		16	15.9				Very stiff.
		9				ML	Brown (7.5 YR 5/4), damp, loose, clayey SILT. Stage I cementation, non-cemented, no reaction to HCL.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-1

DATE DRILLED 7/10/01 BORING NO. CH-1
GROUND ELEVATION 1312' 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		21				SM	ALLUVIUM: (continued) Very pale brown (10 YR 7/4), dry, medium dense, silty SAND. Soil type change from ML to SM within sample interval.
							Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/10/01.
25							
30							
35							
40							

Ninyo & Moore

BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-2

DATE DRILLED 7/17/01 BORING NO. CH-2
GROUND ELEVATION 1309' 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: Brown (7.5 YR 5/4), dry, stiff, silty CLAY; scattered caliche filaments, weakly cemented. Stage I, weak reaction with HCL.
		12	9.8	85.5			
5		45					Hard.
		12	8.0				Very stiff.
10		37	4.7	100.8			Hard.
		82/9"	13.3				Stage I cementation, scattered caliche filaments, weakly cemented, weak to no reaction with HCL.
15		39				SM	Pale brown (10 YR 6/3), dry, medium dense, silty SAND.
		53	5.5			CL	Brown (7.5 YR 5/4), dry, hard, silty CLAY. Stage II cementation, scattered caliche nodules, moderately cemented.
20							

Ninyo & Moore

BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-3

DATE DRILLED 7/17/01 BORING NO. CH-2
GROUND ELEVATION 1309' 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		27	6.0	94.2		CL	ALLUVIUM: (continued) Brown (7.5 YR 5/4), dry, hard, silty CLAY. Stage II cementation, scattered calcium carbonate nodules.
							Total Depth = 21.5' Groundwater not encountered. Piezometer installed on 7/17/01.
25							
30							
35							
40							

Ninyo & Moore

BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-4

DATE DRILLED 7/11/01 BORING NO. CH-3
GROUND ELEVATION 1309' 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	FILL: Brown (7.5 YR 5/4), damp, stiff, silty CLAY. Stage II cementation, scattered calcium carbonate filaments, weakly cemented.
		12	14.6	84.2			
5		17					Very stiff.
		34	8.4				Hard; few sand.
10		35	5.4	102.1		ML	ALLUVIUM: Reddish brown (5 yr 5/4), damp, medium dense, sandy SILT. Stage I cementation, weakly cemented, moderage reaction with HCL.
		55	3.0			SM	Brown (7.5 YR 5/4), damp, dense to very dense, silty SAND; some fine subrounded gravel. Stage II cementation, moderate cementation by caliche, color change to light gray.
15		67/11"	6.3				
		68	7.4	95.8		CL	Brown (7.5 YR 5/4), damp, hard, silty CLAY. Stage II cementation, trace caliche nodules less than 1/2" in diameter.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-5

DATE DRILLED 7/11/01 BORING NO. CH-3
GROUND ELEVATION 1309' 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		■	70	7.1			ML	ALLUVIUM: (continued) Brown (7.5 YR 5/4), damp, hard, silty CLAY, trace nodules. Stage II cementation, trace carbonate nodules less than 1/2" in diameter.
								Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/11/01.
25								
30								
35								
40								

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-6

DATE DRILLED 7/11/01 BORING NO. CH-4
GROUND ELEVATION 1308' 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: Brown (7.5 Yr 5/4), damp, very stiff, silty CLAY. Stage I cementation, scattered calcium carbonate filaments.
		■	14					
5		◥	13	6.0				Thin layer of silty fine sand.
		■	35	13.0	72.5			Hard, scattered calcium carbonate filaments.
10		◥	12	7.7				Very stiff.
		■	90/11"	6.3	93.8			Hard.
15		◥	21	3.1			ML	Very pale brown (10 YR 7/4), dry to damp, dense, clayey SILT; few fine gravel, trace coarse gravel. Stage II cementation below 15 feet moderate reaction with HCL.
20		■	82/11"	2.3	117.0		SP-SM	Light bluish gray (10B 8/1), dry to damp, very dense, silty SAND. Stage II cementation, grains coated by calcium carbonate, matrix loose, moderate reaction with HCL on coatings.

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-7

DATE DRILLED 7/11/01 BORING NO. CH-4
GROUND ELEVATION 1308' 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		30				SM	ALLUVIUM: (continued) Light bluish gray (10B 8/1), dry, dense, silty SAND.
							Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/11/01.
25							
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-8

DATE DRILLED 7/11/01 BORING NO. CH-5
GROUND ELEVATION 1307' 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: Brown (7.5 YR 5/4), damp, very stiff, silty CLAY. Stage I cementation, scattered calcium carbonate filaments.
		15	7.0	90.3			
5		14	12.8				
		83	7.4			ML	Pale brown (10 YR 6/3), dry, dense to very dense, clayey SILT. Stage I cementation, scattered calcium carbonate filaments.
10		85/11"					
		33	5.3				
15		43					
		37	3.9			CL	Reddish brown (5 YR 5/4) to pale brown (10 YR 6/3), damp, hard, silty CLAY; some medium to fine sand.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-9

DATE DRILLED 7/11/01 BORING NO. CH-5
GROUND ELEVATION 1307' 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		54	4.6	96.1		CL	ALLUVIUM: (continued) Pale brown (10 YR 6/3), damp to dry, hard, silty CLAY. Stage I cementation, scattered calcium carbonate filaments.
							Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/11/01.
25							
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-10

DATE DRILLED 7/17/01 BORING NO. CH-6
GROUND ELEVATION 1308' 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		22				CL	ALLUVIUM: Brown (7.5 YR 5/4), dry, very stiff, silty CLAY. Stage I cementation, sparse calcium carbonate filaments, non -to weakly cemented.
5		13	6.6			ML	Brown (7.5 YR 5/4), dry to damp, medium dense to very dense, clayey SILT; scattered caliche stringers. Stage I cementation, sparse calicium carbonate filaments, non -to weakly cemented.
		29	4.9	96.1			
10		41	4.2				
15		24				SM	Brown (7.5 YR 5/4) to reddish brown (5 YR 5/4), dry to damp, medium dense, silty SAND; few fine gravel. Stage I cementation, trace calcium carbonate filaments, non -to weakly cemented, weak to no reaction with HCL.
		11	0.6				
20		34	1.8	121.5			Stage II cementation below 17.5 feet; continuous coatings of calcium carbonate on fine gravel grains, matrix loose.

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-11

DATE DRILLED 7/17/01 BORING NO. CH-6
GROUND ELEVATION 1308' 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		63	1.6	127.1		SM	ALLUVIUM: (continued) Pale brown (10 YR 6/3), dry to damp, dense, silty SAND; few fine gravel. Stage II cementation.
							Total Depth = 21.5' Groundwater not encountered. Piezometer installed on 7/17/01.
25							
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-12

DATE DRILLED 7/11/01 BORING NO. CH-7
GROUND ELEVATION 1306' 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		21	6.9	92.7		ML	ALLUVIUM: Brown (7.5 YR 5/4), damp, medium dense, clayey SILT. Stage I cementation, weakly cemented by scattered calcium carbonate filaments.
5		24	5.9	96.8		CL	Brown (7.5 YR 5/4), damp, very stiff, silty CLAY; some fine sand. Stage I cementation, weakly cemented by scattered calcium carbonate filaments.
		51	5.6				Hard.
10		62	5.3	99.8			
		15	1.8			SM	Pale brown (5 YR 6/2), dry to damp, medium dense, silty SAND; scattered fine gravel.
15		74	1.0	129.6		SP	Pale brown, dry to damp, very dense, SAND with fine to coarse gravel. Stage II cementation below 15 feet; weak to moderate reaction with HCL.
		51	1.2				
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-13

DATE DRILLED 7/11/01 BORING NO. CH-7
GROUND ELEVATION 1306' 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		66				GM	ALLUVIUM: (continued) Pale brown (10 YR 6/3), dry, dense, silty GRAVEL. Stage II cementation, thin calcium carbonate coating, matrix loose, weak to no reaction with HCL.
							Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/11/01.
25							
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-14

DATE DRILLED 7/11/01 BORING NO. CH-8
GROUND ELEVATION 1306' 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: Brown (7.5 YR 5/4), damp, very stiff, silty CLAY. Stage I cementation, sparse calcium carbonate filaments, weakly cemented, moderate reaction with HCL.
		22	6.0	91.8			
5		6					Stiff.
		9	2.8			SM	Pale brown (10 YR 6/3), dry to damp, medium dense, silty SAND; trace fine gravel. Stage I cementation, no calcium carbonate coatings on gravel grains, weak reaction with HCL on sand particles.
10		24	2.8	105.1			
		12					Loose.
15		20	3.6				Medium dense to dense.
		52	2.8	117.5		SM/GM	Light yellowish brown (10 YR 6/4), dry to damp, dense, silty SAND with fine gravel; increase in gravel content. Stage II cementation, moderately cemented by calcium carbonate.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-15

DATE DRILLED 7/11/01 BORING NO. CH-8
GROUND ELEVATION 1306' 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		27	3.9			SM/GM	ALLUVIUM: (continued) Light yellowish brown (10 YR 6/4), dry to damp, dense, silty SAND with gravel. Stage II cementation.
							Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/11/01.
25							
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-16

DATE DRILLED 7/17/01 BORING NO. CH-9
GROUND ELEVATION 1306' 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		13				SM	ALLUVIUM: Pale brown (10 YR 6/3), dry, loose, silty SAND; few fine gravel. Stage I cementation.
5		31				ML	Pale brown (10 YR 6/3) to brown (7.5 YR 5/4), dry to damp, medium dense, clayey SILT.
		11	9.3			SM	Pale brown (10 YR 6/3) to brown (10 YR 5/3), dry to damp, medium dense, silty SAND; few fine gravel.
10		30	1.7	107.4			
		32	1.5				Dense.
15		56					Few fine gravel; trace cobbles. Stage II cementation below 15 feet, continuous calcium carbonate coatings on gravel grains, matrix loose.
		21	0.7				
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-17

DATE DRILLED 7/17/01 BORING NO. CH-9
GROUND ELEVATION 1306' 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		46	12.0	108.2		SC	ALLUVIUM: (continued) Pale brown (10 YR 6/3), damp, dense, clayey SAND. Stage II cementation.
							Total Depth = 21.5' Groundwater not encountered. Piezometer installed on 7/17/01.
25							
30							
35							
40							

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BORING LOG
East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-18

DATE DRILLED 7/11/01 BORING NO. CH-10
GROUND ELEVATION 1306' 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: Brown (7.5 YR 5/4), dry to damp, hard, silty CLAY. Stage II cementation, scattered caliche filaments, weakly cemented.
		72/11"					
5		40					
		83/11"					
10		25	5.8				
		50/4"	7.4	92.0			
15		72	13.2			SM	Pale brown (10 YR 6/3), dry to damp, very dense, silty SAND. Stage II cementation below 14.5 feet; moderately cemented.
20		71	11.3	105.9		SC	Pale brown (10 YR 6/3), dry to damp, very dense, clayey SAND. Scattered fine gravel. Stage II cementation, continuous calcium carbonate coatings on gravel grains.

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BORING LOG
East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-19

DATE DRILLED 7/11/01 BORING NO. CH-10
GROUND ELEVATION 1306' 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		24	3.3			SM	Pale brown (10 YR 6/3), dry to damp, medium dense, silty SAND; scattered fine gravel. Stage II cementation, moderately cemented.
							Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/11/01.
25							
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-20

DATE DRILLED 7/13/01 BORING NO. CH-11
GROUND ELEVATION 1307' 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 / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						CL	ALLUVIUM: Brown (7.5 YR 5/4), dry, very stiff to hard, silty CLAY. Stage I cementation, scattered caliche filaments, weakly cemented by calcium carbonate, moderate reaction with HCL.
		24	6.2	101.1			
5		57					Hard.
		23	11.1				
10		32	16.7	104.4			Few sand.
		15	0.3			SW-SM	Very pale brown (10 YR 7/4), dry, medium dense, SAND with silt; few fine gravel. Stage II cementation.
15		32	4.4	110.7			
							Total Depth = 17.0' Groundwater not encountered. Backfilled on 7/13/01.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-21

DATE DRILLED 7/13/01 BORING NO. CH-12
GROUND ELEVATION 1309' 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 / 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, very stiff, silty CLAY. Stage I cementation, weakly cemented by calcium carbonate.
		16					
5		16	2.2				
		31	3.4	111.1		SM	Light brown (7.5 YR 6/4) to reddish brown (5 YR 5/4), dry, dense to medium dense, silty SAND; few fine gravel. Stage II cementation.
10		19	0.8				
		50/5""					
15		67/11"	3.8				
20							Total Depth = 16.5' Groundwater not encountered. Backfilled on 7/13/01.

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-22

DATE DRILLED 7/13/01 BORING NO. CH-13
GROUND ELEVATION 1310' 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 / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						CL	ALLUVIUM: Brown (7.5 YR 5/4) to light brown (7.5 YR 6/4), damp, very stiff, silty CLAY. Stage I cementation, non-cemented to weakly cemented, few calcium carbonate filaments.
		22	7.0	86.8			
5		14	5.0	71.0		ML	Brown (7.5 YR 5/4), damp, loose to medium dense, clayey SILT.
		16				CL	Brown (7.5 YR 5/4) to light brown (7.5 YR 6/4), damp, very stiff, silty CLAY.
10		17	6.2	87.1			
		43				ML	Light brown (7.5 YR 6/4), dry, dense, clayey SILT; few sand. Stage II cementation, scattered calcium carbonate filaments, continuous Stage II cementation, few sand.
15		40	5.0			SM	Light brown (7.5 YR 6/4), damp, medium dense, silty SAND.
20							Total Depth = 16.5' Groundwater not encountered. Backfilled on 7/13/01.

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-23

DATE DRILLED 7/13/01 BORING NO. CH-14
GROUND ELEVATION 1310' 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 / 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 to damp, very stiff, silty CLAY. Stage I cementation, trace calcium carbonate filaments, weakly cemented by calcium carbonate; weak reaction with HCL.
		15	9.0				
5		9	5.4				Stiff.
		47					Hard; scattered caliche filaments.
10		47	3.9				
		31	5.2				Few sand.
15		31	1.3	105.3		SW-SM	Light brown (7.5 YR 6/4) to light bluish gray (10 B 8/1), dry, medium dense, SAND with silt and sand; few gravel, trace cobbles. Stage II cementation below 15 feet; calcium carbonate coatings on gravel grains.
		27					Shoe plugged by cobble.
20							Total Depth = 19.0' Groundwater not encountered. Backfilled on 7/13/01.

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-24

DATE DRILLED 7/13/01 BORING NO. CH-15
GROUND ELEVATION 1313' 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: Brown (7.5 YR 5/4), dry to damp, stiff to very stiff, silty CLAY. Stage I cementation, scattered calcium carbonate nodules less than 1/4" in diameter, weakly cemented.
		22	5.5	82.4			
5		13	3.3				
		35					Hard.
10		76/9"	5.7				
		63					Stage II cementation below 12.5 feet; color changes to pale brown (10 YR 6/3), moderate reaction to HCL, calcium carbonate nodules less than 1/4" in diameter.
15		52	2.9				
		86/9"	3.4	99.7		ML	Light brown to reddish brown, dry to damp, very dense, sandy SILT; sparse fine gravel, calcium carbonate nodules less than 1/4"in diameter, gravel fraction coated by calcium carbonate. Stage II cementation.
20							Total Depth = 18.3'

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-25

DATE DRILLED 7/13/01 BORING NO. CH-15
GROUND ELEVATION 1313' 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
							Groundwater not encountered. Backfilled on 7/13/01.
25							
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-26

DATE DRILLED 7/13/01 BORING NO. CH-16
GROUND ELEVATION 1315' 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 6/4), dry to damp, very stiff, silty CLAY. Stage I cementation, weakly cemented by calcium carbonate, moderate reaction with HCL.
		15					
5		24	5.6	81.1			
		15					
10		34					Hard.
		16	2.0			SM	Light brown (7.5 YR 6/4) to light bluish gray (10 B 8/1), dry, medium dense, silty SAND. Stage I cementation, weakly cemented by calcium carbonate, moderate reaction with HCL.
15		40	2.0				
						CL	Light brown (7.5 YR 6/4), dry, hard, silty CLAY. Stage I cementation.
		45				SM	Light bluish gray (10 B 8/1), dry, very dense, silty SAND; scattered fine gravel. Stage II cementation below 18 feet.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-27

DATE DRILLED 7/13/01 BORING NO. CH-16
GROUND ELEVATION 1315' 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		54	3.8	112.8		SM	Light bluish gray (10 B 8/1) to reddish brown (5 YR 5/4), dry to damp, dense, silty SAND. Stage II cementation, trace to few cobbles, calcium carbonate coatings on cobbles.
		49	0.1				Very dense; poor recovery, cobble fragments only.
25		87/9"				CL	Brown (7.5 YR 5/4) to pale brown (10 YR 6/3), dry to damp, hard, silty CLAY. Stage II cementation, scattered caliche nodules.
		50/4"					
30		64	1.9	116.7		SM	Reddish brown (5 YR 5/4), dry to damp, dense, silty SAND; sparse fine gravel. Stage II cementation, gravel fraction has thin calcium carbonate coatings on all sides.
		81/9"	3.0			CL	Pale brown (10 YR 6/3) to reddish brown (5 YR 5/4), damp, hard, silty CLAY. Stage II cementation, thin calcium carbonate layers less than 1/8" thick.
35							Total Depth = 33.3' Groundwater not encountered. Backfilled on 7/13/01.
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-28

DATE DRILLED 7/13/01 BORING NO. CH-17
GROUND ELEVATION 1316' 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: Brown (7.5 YR 5/4), dry to damp, very stiff, silty CLAY with fine sand. Stage I cementation, scattered caliche filaments.
		15	2.8	103.1			
5		12	3.7				
		30	9.5	85.3			Hard.
10		18	4.2				Very stiff.
		38					Hard. Stage I cementation, scattered fine gravel, trace filaments of calcium carbonate.
15		68				SM	Light bluish gray (10 B 8/1), dry to damp, very dense, silty SAND. Stage II cementation, sparse fine gravel, gravel fraction has calcium carbonate coatings.
		64	3.3	107.4			Dense.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-29

DATE DRILLED 7/13/01 BORING NO. CH-17
GROUND ELEVATION 1316' 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		21	1.3			SM	ALLUVIUM: (continued) Brown (7.5 YR 5/4), dry to damp, dense, silty SAND; sparse fine gravel.
		68	6.0			CL	Light brown (7.5 YR 6/3), damp, dense, sandy CLAY; sparse fine gravel. Stage II cementation, moderately cemented by calcium carbonate.
25							Total Depth = 24.0' Groundwater not encountered. Backfilled on 7/13/01.
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-30

DATE DRILLED 7/13/01 BORING NO. CH-18
GROUND ELEVATION 1318' 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	FILL: Brown (7.5 YR 5/4), dry to damp, hard, silty CLAY; trace fine sand, no rootlets.
		56	7.7	105.0			
5		15					Very stiff.
							No recovery.
10		24	2.7	108.6		SM	Brown (7.5 YR 5/4), dry to damp, dense, silty SAND. Stage I cementation, weakly cemented, weak reaction with HCL. Few fine sand.
15		93/10"	3.1			CL	ALLUVIUM: Pale brown (10 YR 6/3), dry to damp, hard, silty CLAY with sand. Stage I cementation, scattered caliche filaments.
		36					
		88/8"	5.1				
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-31

DATE DRILLED 7/13/01 BORING NO. CH-18
GROUND ELEVATION 1318' 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		61	11.5	90.4		CL	ALLUVIUM: (continued) Pale brown (10 YR 6/3), dry to damp, hard, silty CLAY with sand and fine gravel. Stage II cementation, scattered caliche filaments, sand and gravel grains coated on all sides.
							Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/13/01.
25							
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-32

DATE DRILLED 7/12/01 BORING NO. CH-19
GROUND ELEVATION 1318' 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	FILL: Brown (7.5 YR 5/4), damp, dense, silty SAND.
		67	9.3	113.4			
5		9	6.8			CL	ALLUVIUM: Light brown (7.5 YR 6/4), damp, stiff, silty CLAY. Stage I cementation, scattered caliche filaments.
		15					Very stiff.
10		38					Hard.
		78	6.2	98.8			
15		54	9.1				
		65					Few sand.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-33

DATE DRILLED 7/12/01 BORING NO. CH-19
GROUND ELEVATION 1318' 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		38	4.1			SM	ALLUVIUM: (continued) Pale brown (10 YR 6/3), dry to damp, very dense, silty SAND. Stage II cementation, moderately cemented by calcium carbonate, continuously cemented matrix.
		77	3.1	106.6			
25							Total Depth = 24.0' Groundwater not encountered. Backfilled on 7/12/01.
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-34

DATE DRILLED 7/11/01 BORING NO. CH-20
GROUND ELEVATION 1316' 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	FILL: Pale brown (10 YR 5/4) to brown (7.5 YR 5/4), dry to damp, very stiff, silty and sandy CLAY. Stage I cementation.
		20					
5		12	3.9	98.6			Stiff.
		13	4.7				Very stiff.
10						ML	ALLUVIUM: Pale brown (10 YR 6/3), dry, medium dense, SILT; few gravel and sand. Stage I cementation, scattered caliche filaments.
		31	3.9	109.6		SM	Very pale brown (10 YR 7/4), dry to damp, medium dense, silty SAND; few gravel. Stage II cementation.
		48	4.7				
15		50/6"	7.8	88.5			
						CL	Pale brown (10 YR 6/3), dry, hard, silty CLAY. Stage II cementation, few caliche nodules less than 1/2" in diameter.
		75					
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-35

DATE DRILLED 7/11/01 BORING NO. CH-20
GROUND ELEVATION 1316' 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		49				CL	ALLUVIUM: (continued) Pale brown (10 YR 6/3), dry, hard, silty CLAY. Stge II cementation, few caliche layers with contiuous cementation
		14	1.5			SP	Pale brown (10 YR 6/3), dry, medium dense, SAND; few fine gravel. Stage II cementation, continuous coatings on gravel grains, moderate to weak reaction with HCL.
25		36	1.3				
							Total Depth = 26.5' Groundwater not encountered. Backfilled on 7/11/01.
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-36

DATE DRILLED 7/12/01 BORING NO. CH-21
GROUND ELEVATION 1315' 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	FILL: Brown (7.5 YR 5/4), dry to damp, very stiff, silty CLAY. Stage I cementation.
		21	6.0	91.4			
5		12					
		35	5.1	85.9		ML	ALLUVIUM: Brown (7.5 YR 5/4), dry to damp, medium dense, SILT. Hard. Stage I cementation.
10		55	4.7			CL	Brown (7.5 YR 5/4), dry to damp, very stiff, silty CLAY. Stage I cementation.
		34					
15		32	2.8			SM	Pale brown (10 YR 6/3), dry, dense, silty SAND. Stage II cementation.
		49					Trace fine gravel.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-37

DATE DRILLED 7/12/01 BORING NO. CH-21
GROUND ELEVATION 1315' 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		34	1.9			SM	ALLUVIUM: (continued) Pale brown (10 YR 6/3), dry, medium dense, silty SAND. Stage II cementation.
		45	1.4	108.5			Dense; scattered fine gravel.
25		38					Medium dense.
							Total Depth = 25.5' Groundwater not encountered. Backfilled on 7/12/01.
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-38

DATE DRILLED 7/12/01 BORING NO. CH-22
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 / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						CL	FILL: Brown (7.5 YR 5/4), dry to damp, hard, silty CLAY. Stage I cementation, weak cementation by trace of caliche filaments.
		81/11"					
5		18					Very stiff.
		13					
10		26	5.8	91.6		CL	ALLUVIUM: Light brown (7.5 YR 6/4), dry to damp, very stiff to hard, silty CLAY. Stage I cementation, few to some caliche filaments.
		27	6.9				Hard.
15		79	10.9	106.6			
		100	6.6				
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-39

DATE DRILLED 7/12/01 BORING NO. CH-22
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 / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20		89				CL	ALLUVIUM: (continued) Light brown (7.5 YR 6/4), dry to damp, hard, silty CLAY.
		44	7.9				
25		44	6.3	108.6		SM	Brown (7.5 YR 5/4), dry to damp, dense, silty SAND. Stage II cementation, carbonate grain coatings.
30							Total Depth = 26.5' Groundwater not encountered. Backfilled on 7/12/01.
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-40

DATE DRILLED 7/12/01 BORING NO. CH-23
GROUND ELEVATION 1318' 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 / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						CL	FILL: Brown (7.5 YR 5/4), dry to damp, hard, silty CLAY.
		87	3.4	109.6			
5		30	6.1				
		50	3.7	100.9		CL-ML	Brown (7.5 YR 5/4), damp, hard, silty CLAY. Stage I cementation, trace caliche.
10		13	6.2			CL	ALLUVIUM: Pale brown (10 YR 6/3), dry to damp, very stiff, silty CLAY. Stage II cementation, few caliche nodules less than 1/2" in diameter.
		65					Hard.
15		91	5.2				
		73	4.3	107.2		ML	Pale brown (10 YR 6/3), dry to damp, very dense, sandy SILT. Stage II cementation, scattered caliche nodules less than 1/2" in diameter.
20							Total Depth = 19.0' Groundwater not encountered. Backfilled on 7/12/01.

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-41

DATE DRILLED 7/12/01 BORING NO. CH-24
GROUND ELEVATION 1316' 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: Brown (7.5 YR 5/4), dry to damp, very stiff, silty CLAY. Stage I cementation.
		22	3.4	94.7			
5		10					
		17	6.4				
10		27					Hard.
		65/11"	5.8	91.0			
15		32	5.2				Weak cementation by caliche.
		90	6.6	109.4		SM	Pale brown (10 YR 6/3), dry to damp, very dense, silty SAND; scattered fine subrounded to rounded gravel. Stage II cementation, carbonate coatings on grains.
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-42

DATE DRILLED 7/12/01 BORING NO. CH-24
GROUND ELEVATION 1316' 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		37	5.8			CL	ALLUVIUM: (continued) Pale brown (10 YR 6/3), dry to damp, hard, sandy CLAY.
		89				SM	Pale brown (10 YR 6/3), dry, very dense, silty SAND. Stage II cementation, carbonate coatings on grains.
25							Total Depth = 24.0' Groundwater not encountered. Backfilled on 7/12/01.
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-43

DATE DRILLED 7/12/01 BORING NO. CH-25
GROUND ELEVATION 1312' 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, silty CLAY.
		15	1.1	96.1		SM	Pale brown (10 YR 6/3) to light bluish gray (10 B 8/1), dry, medium dense, silty SAND. Stage I cementation.
5		16	1.3				Trace fine gravel.
		49				CL	Pale brown (10 YR 6/3) to brown (7.5 YR 5/4), damp, hard, silty CLAY. Stage I cementation.
10		71/10"					
		36	5.5	105.7			
15		32					
		92				GM	Pale brown (10 YR 6/3) to light bluish gray (10 B 8/1), dry to damp, very dense, silty GRAVEL with sand. Stage II cementation.
20							

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BORING LOG
East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-44

DATE DRILLED 7/12/01 BORING NO. CH-25
GROUND ELEVATION 1312' 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		76/11"	4.0	116.0		SM	ALLUVIUM: (continued) Pale brown (10 YR 6/3) to light bluish gray (10 B 8/1), dry to damp, very dense, silty SAND with gravel. Stage II cementation.
							Total Depth = 21.4' Groundwater not encountered. Backfilled on 7/12/01.
25							
30							
35							
40							

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BORING LOG
East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-45

DATE DRILLED 7/12/01 BORING NO. CH-26
GROUND ELEVATION 1313' 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 6/4) to brown (7.5 YR 5/4), dry to damp, very stiff, silty CLAY. Stage I cementation.
		19	3.7	94.6			
5		18					
		29	6.0				Hard.
10		60	4.8	104.9			
		17	1.5			SM	Pale brown (10 YR 6/3), dry, medium dense, silty SAND with fine gravel.
15		84				GM	Pale brown (10 YR 6/3) to light bluish gray (10 B 8/1), dry, very dense, silty GRAVEL with fine sand; trace cobbles. Stage II cementation.
		68	1.7				
20							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-46

DATE DRILLED 7/12/01 BORING NO. CH-26
GROUND ELEVATION 1313' 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						SM	ALLUVIUM: (continued)
		50/4"	5.3	105.0		CL	Pale brown (10 YR 6/3) to light bluish gray (10 B 8/1), dry, very dense, silty SAND with fine gravel; trace cobbles. Stage II cementation, carbonate coatings on grains.
							Brown, dry to damp, hard, silty CLAY with sand.
25							Total Depth = 21.5' Groundwater not encountered. Backfilled on 7/12/01.
30							
35							
40							

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BORING LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE	FIGURE
600198001	1/02	A-47

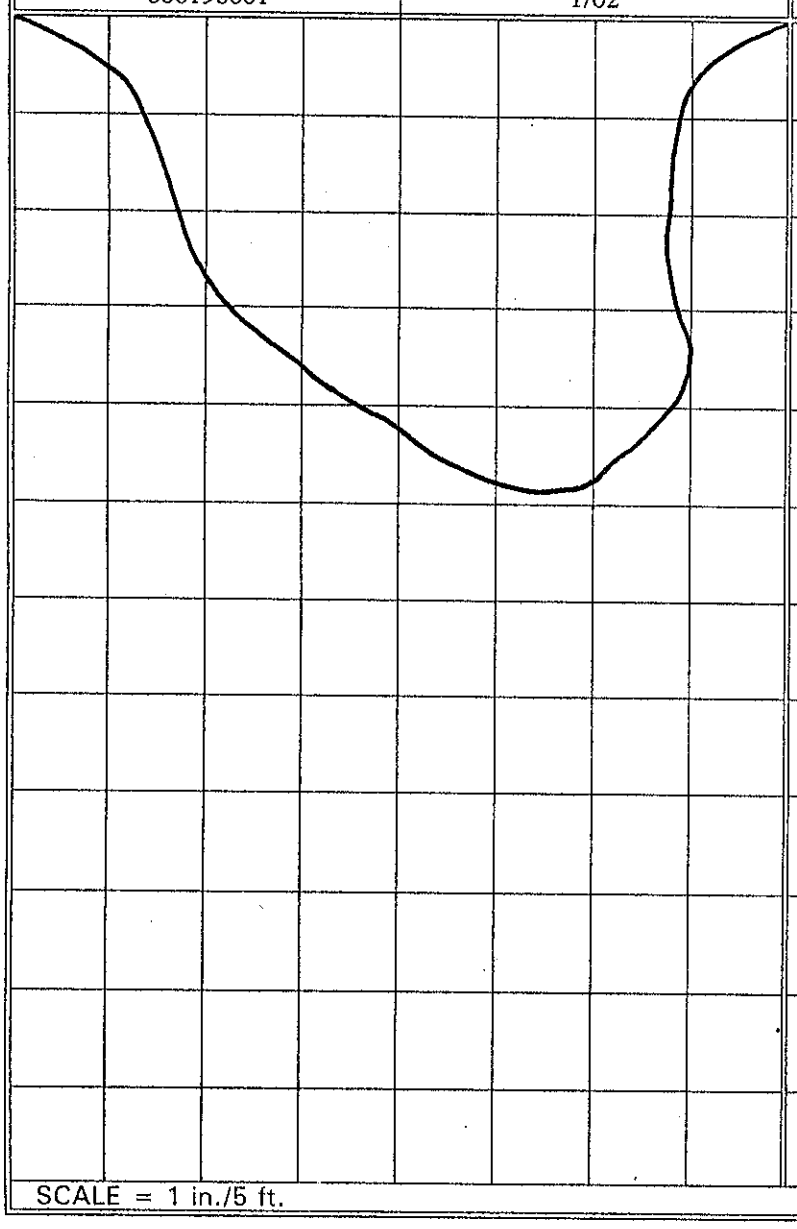
Ninyo & Moore

TEST PIT LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE
600198001	1/02

DATE EXCAVATED 11/26/01 TEST PIT NO. TP-4
GROUND ELEVATION -- LOGGED BY MDE
METHOD OF EXCAVATION Backhoe, Ford 555 E
LOCATION 0.1 Mi. N of CH-3 on EMF Access Rd., E of Fill Section



DEPTH (FEET)	SAMPLES (Bulk / Driven / Sand Cone)	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0				ML	ALLUVIUM: Light reddish brown (5 YR 6/4), loose to dense, dry to damp, clayey SILT. Few rootlets from 0-6 feet bgs. Stage I cementation, scattered caliche stringers less than 1/4" long, moderate reaction with HCL, very weakly cemented by calcium carbonate.
5					
10					@ 10 feet bgs, Stage II cementation with scattered caliche filaments, forming discontinuous, hard, cobble to fine gravel size lenses of cemented clayey silt up to 1/2" thick and 5" diameter, strong reaction with HCL within silt and lenses.
15					Total Depth = 12 feet. Groundwater not encountered during drilling. Backfilled on 11/26/01. Excavation Bearing: 200°
20					
25					

SCALE = 1 in./5 ft.

FIGURE A-48

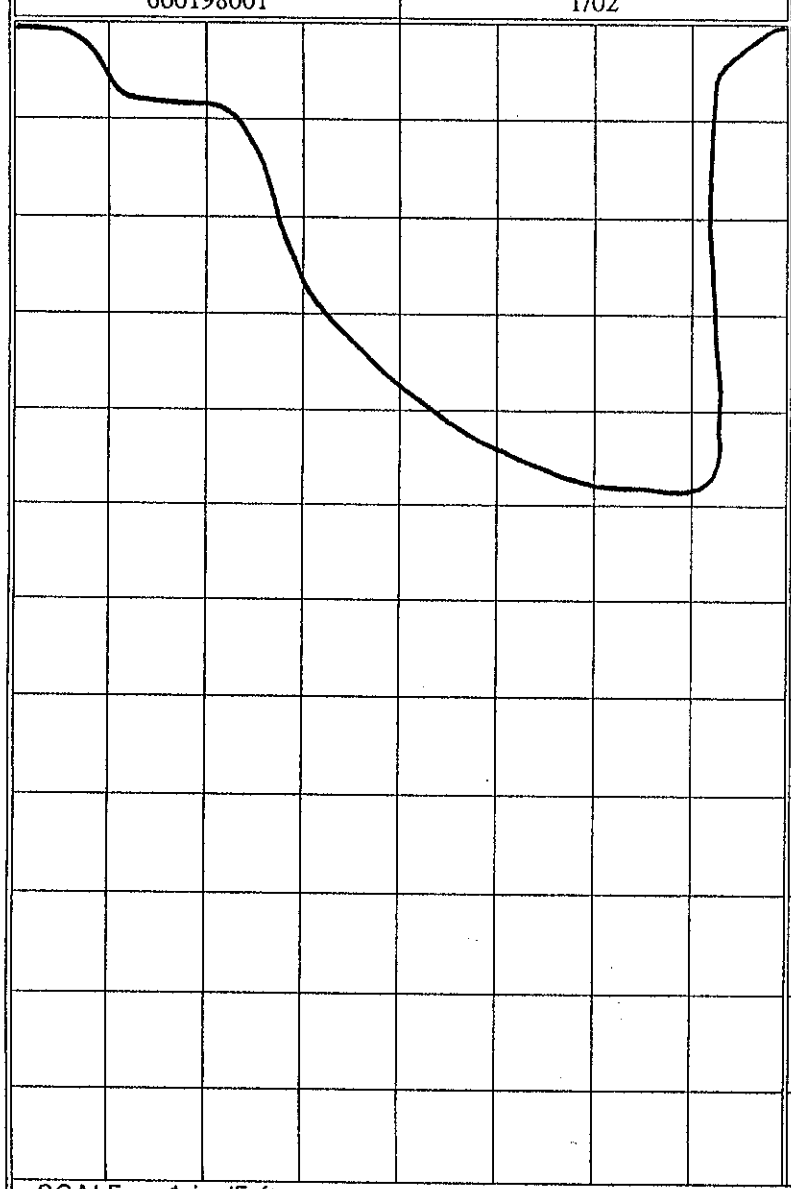
Ninyo & Moore

TEST PIT LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE
600198001	1/02

DATE EXCAVATED 11/27/01 TEST PIT NO. TP-5
GROUND ELEVATION -- LOGGED BY MDE
METHOD OF EXCAVATION Backhoe, Ford 555 E
LOCATION 40'N of CH-6 (MW), E Side of EMF Access Road



DEPTH (FEET)	SAMPLES (Bulk / Driven / Sand Cone)	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0				CL	ALLUVIUM: Dark yellowish brown (10 YR 4/4), dry to damp, stiff to very stiff, silty CLAY. Stage I cementation, weakly cemented, scattered calcium carbonate filaments less than 1/4" long, moderate reaction with HCL.
5				ML	Strong brown (7.5 YR 4/6), dry to damp, medium dense to dense, clayey SILT; scattered rootlets, scattered pinhole porosity. Stage I cementation, moderate reaction to HCL, weakly to non-cemented, scattered calcium carbonate filaments.
10					@ 10-12 feet, cementation increases slightly to weakly cemented by calcium carbonate.
15					Total Depth = 12 feet. Groundwater not encountered during drilling. Backfilled on 11/27/01. Excavation Bearing: 185°
20					
25					

SCALE = 1 in./5 ft.

FIGURE A-49

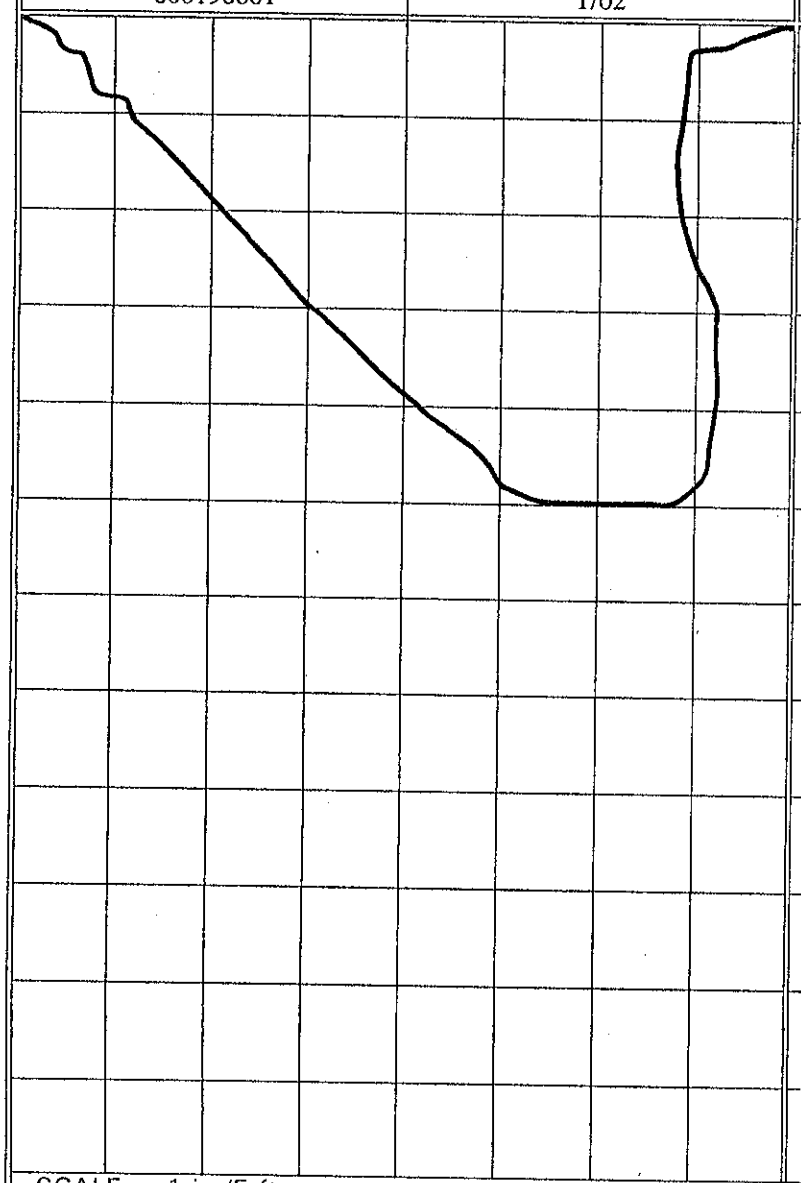
Ninyo & Moore

TEST PIT LOG

East Maricopa Floodway
Chandler Heights Detention Basin

PROJECT NO.	DATE
600198001	1/02

DATE EXCAVATED 11/26/01 TEST PIT NO. TP-6
GROUND ELEVATION -- LOGGED BY MDE
METHOD OF EXCAVATION Backhoe, Ford 555 E
LOCATION 500'S of CH-6 , E Side of EMF Access Road



DESCRIPTION

ALLUVIUM:

Reddish brown (5YR 4/4), stiff to very stiff, dry to damp, silty CLAY; scattered calcium carbonate filaments less than 1/4"long, scattered pinhole voids, trace sand.
Stage I cementation, weakly cemented.

Yellowish-red (5 YR 5/6), soft to dense, damp, clayey SILT; scattered calcium carbonate filaments up to 1/4" long, trace to few pinhole voids, trace fine sand, weakly cemented.
@ 3 feet bgs, increase in amount of cementation by calcium carbonate, few weakly cemented caliche nodules less than 0.5" in diameter, color change to reddish-yellow (7.5 YR 7/6), loose to medium dense, dry to damp, clayey SILT; moderate reaction with HCL, few to no calcium carbonate filaments.
@ 4 feet bgs, color changes to light brown (7.5 YR 6/4), fine sand increases from trace to sparse, damp, loose, sandy SILT; trace fine gravel.
Stage I cementation, little to no calcium carbonate cementation.

Reddish-brown (5 YR 5/3), damp, loose to medium dense, silty fine SAND with sparse fo few fine gravel; trace coarse gravel.
Stage I cementation, strong reaction with HCL, cementation weak to non-cemented.
@ 12 feet bgs, Stage I cementation, increase in amount of weak calcium carbonate cementation, sand breaks into hard fragments up to 4" across, no filaments or nodules observed.

Total Depth = 12.5 feet.
Groundwater not encountered during drilling.
Backfilled on 11/26/01.

Excavation Bearing: 197°

SCALE = 1 in./5 ft.

FIGURE A-50

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-48. 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-49 through B-54.

Hydroconsolidation (Settlement Potential) Tests
Hydroconsolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435-96. 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-55 through B-59.

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-60.

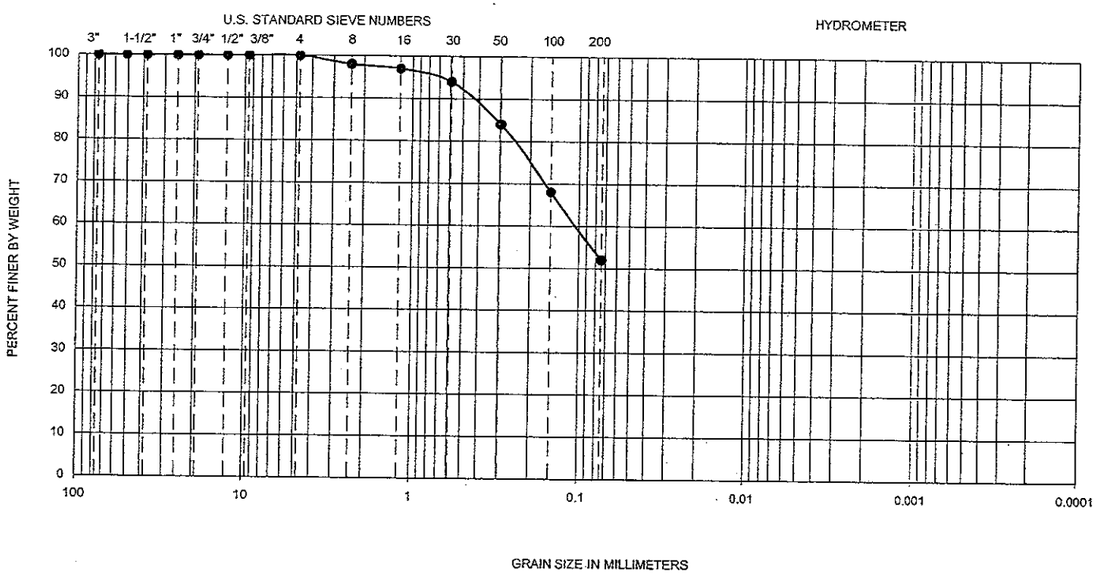
Maximum Dry Density and Optimum Moisture Content Tests
The maximum dry density and optimum moisture content of selected representative soil samples were evaluated in general accordance with ASTM D 698. The results of these tests are summarized on Figures B-61 through B-64.

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-65.

Permeability Tests
Constant head permeability tests were performed on selected undisturbed (and 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-66.

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 Figure B-67.

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
●	CH-1	7.5-9	–	–	–	–	–	–	–	–	52	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

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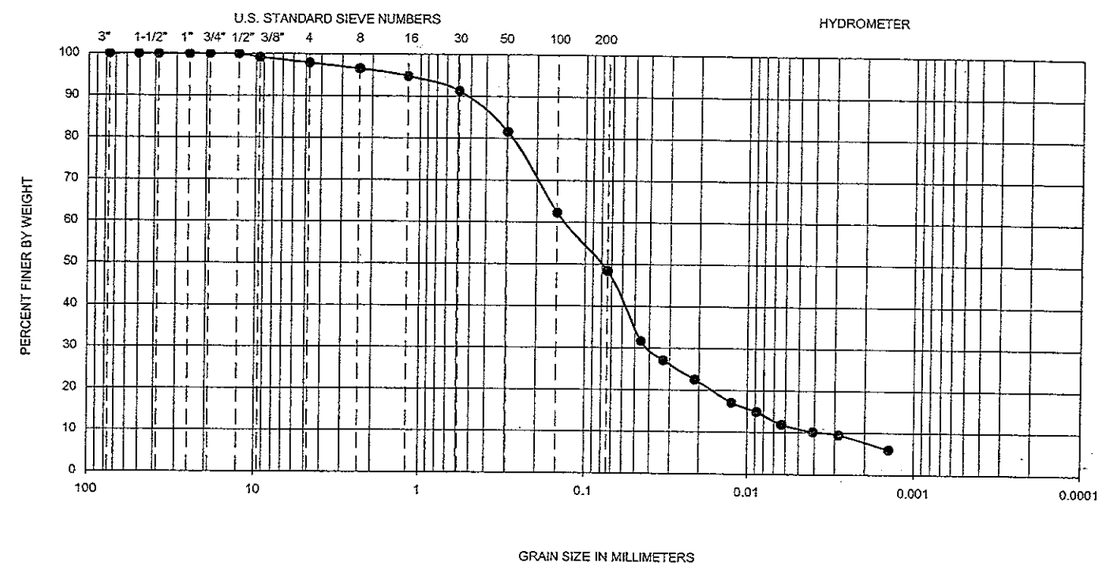
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-1

SIEVE CH-1 7.5-9 #248.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
●	CH-1	20.0-21.5	-	-	NP	0.004	0.04	0.14	37.0	3.3	48.38847	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

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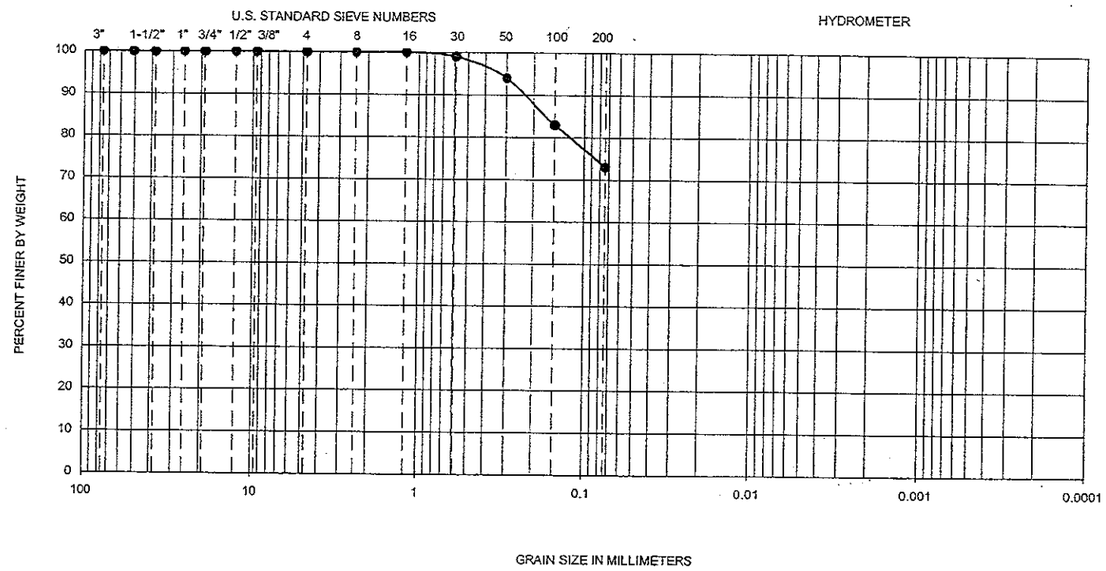
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-2

SIEVE CH-1 20-21.5 HYDRO2.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
●	CH-2	5-6.5	21	16	5	--	--	--	--	--	73	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

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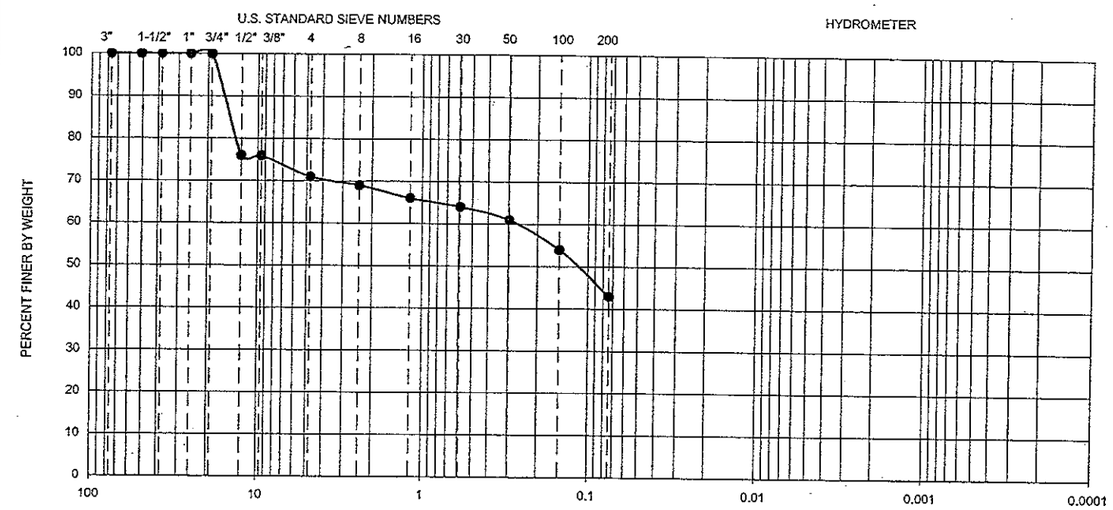
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-3

SIEVE CH-2@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
●	CH-2	15-16.5	28	16	12	--	--	--	--	--	43	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

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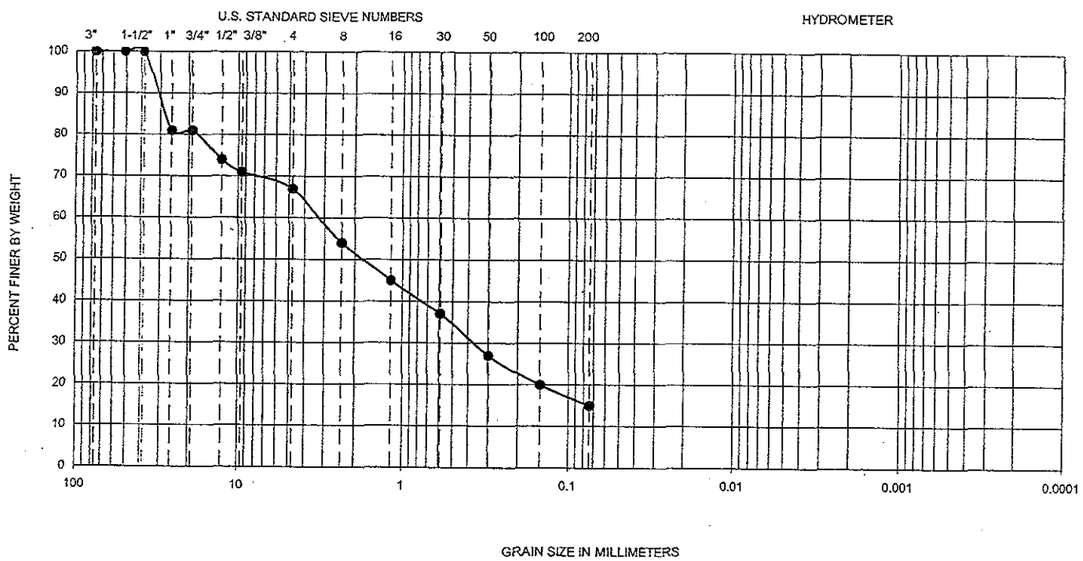
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-4

SIEVE CH-2@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
●	CH-3	15-16.5	--	--	--	--	--	--	--	--	15	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

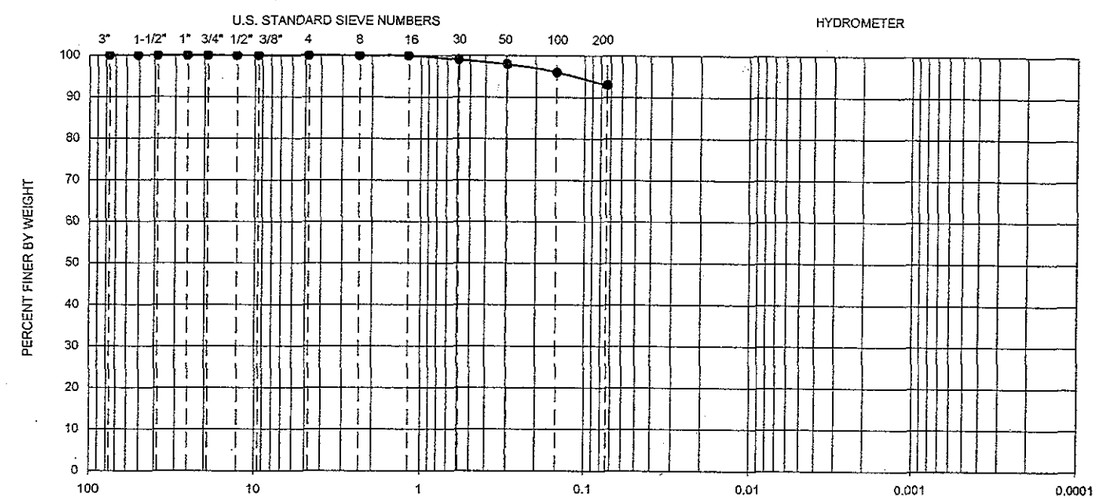
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-5

SIEVE CH-3@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
●	CH-4	7.5-9	42	22	20	--	--	--	--	--	93	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

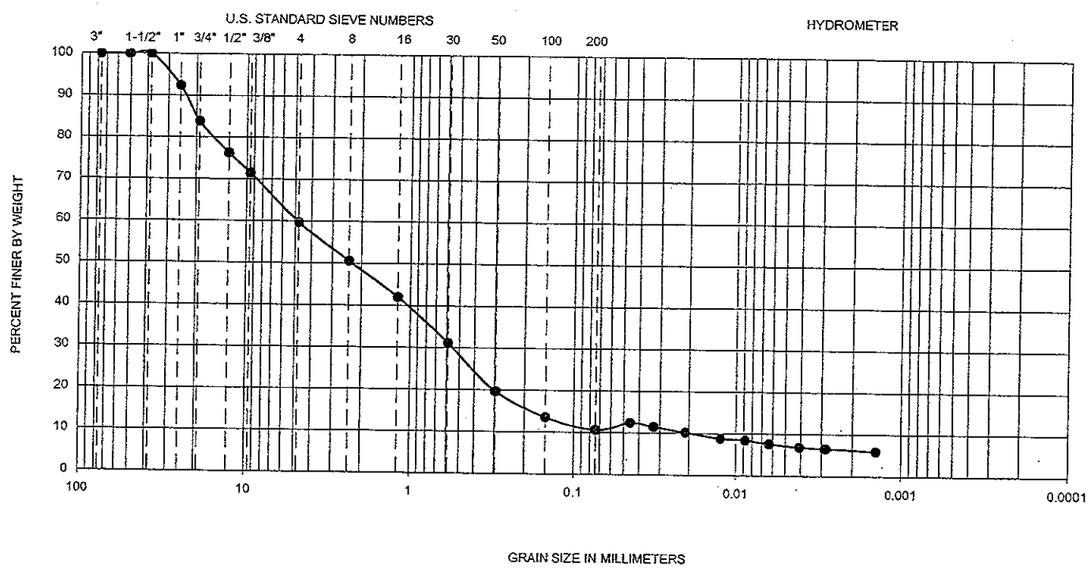
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-6

SIEVE CH-4@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
●	CH-4	17.5-19.0	-	-	NP	0.019	0.57	4.87	249.7	3.4	11	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

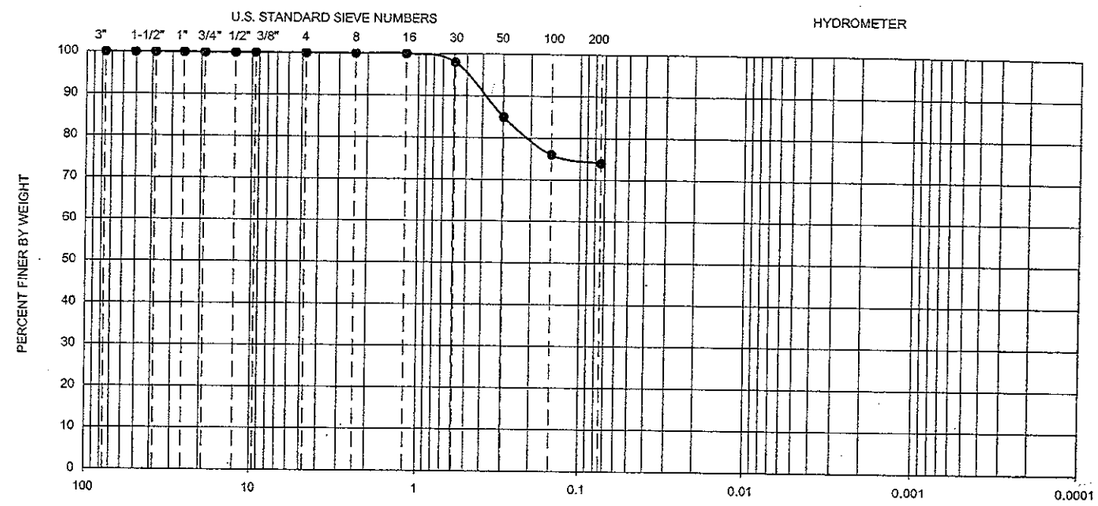
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-7

SIEVE CH-4 17.5-19.0 HYDRO2.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
●	CH-5	5-6.5	29	19	10	--	--	--	--	--	74	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

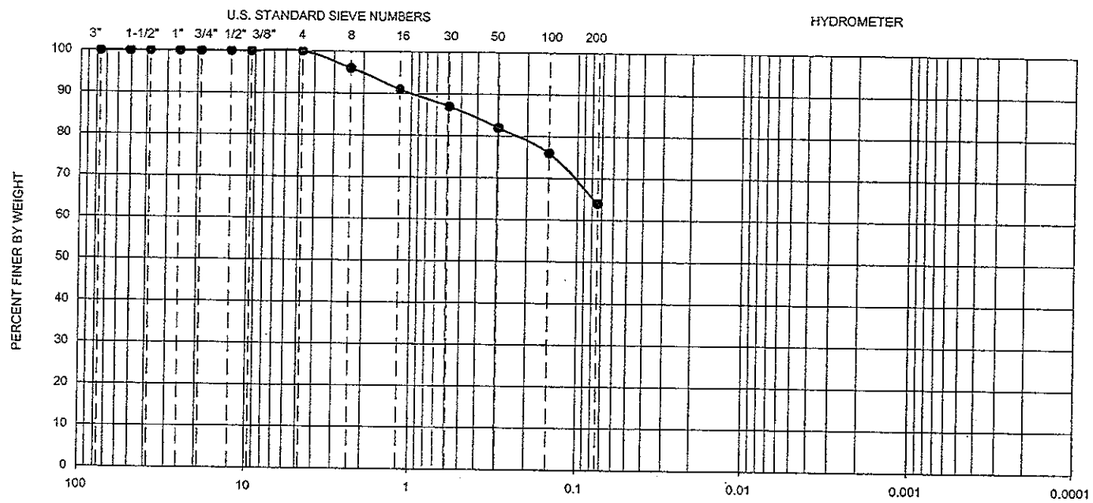
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-8

SIEVE CH-5@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
●	CH-5	15-16.5	--	--	--	--	--	--	--	--	64	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

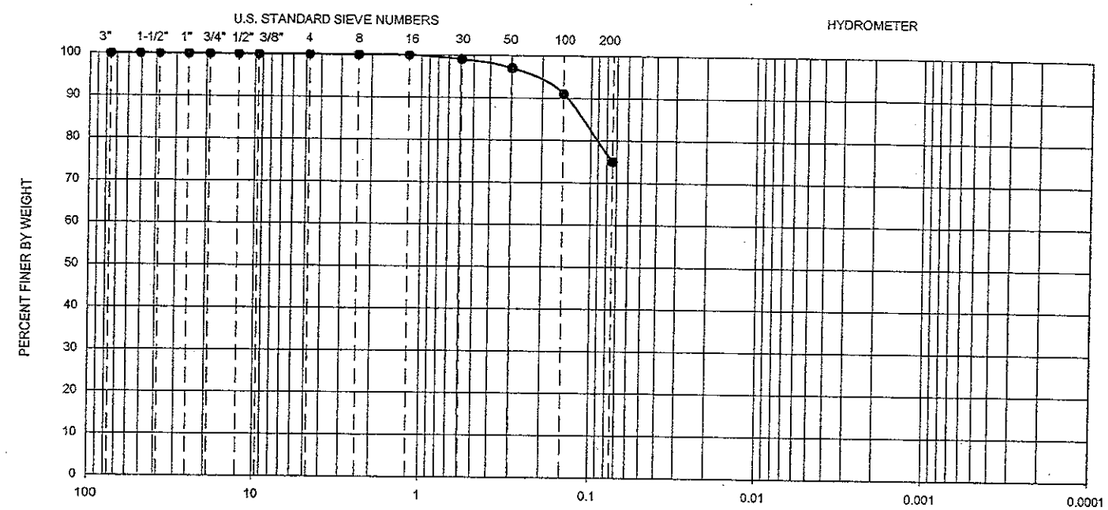
Ninyo & Moore

GRADATION TEST RESULTS
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-9

SIEVE CH-5@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
●	CH-6	7.5-9	--	--	--	--	--	--	--	--	75	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

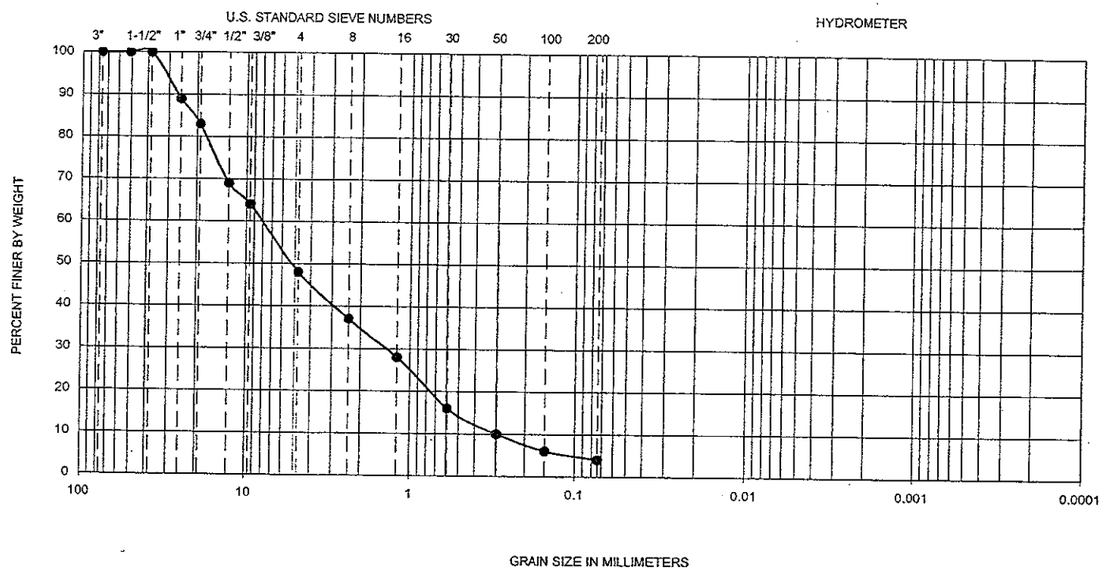
Ninyo & Moore

GRADATION TEST RESULTS
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-10

SIEVE CH-6@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
●	CH-6	17.5-19	--	--	--	--	--	--	--	--	4	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

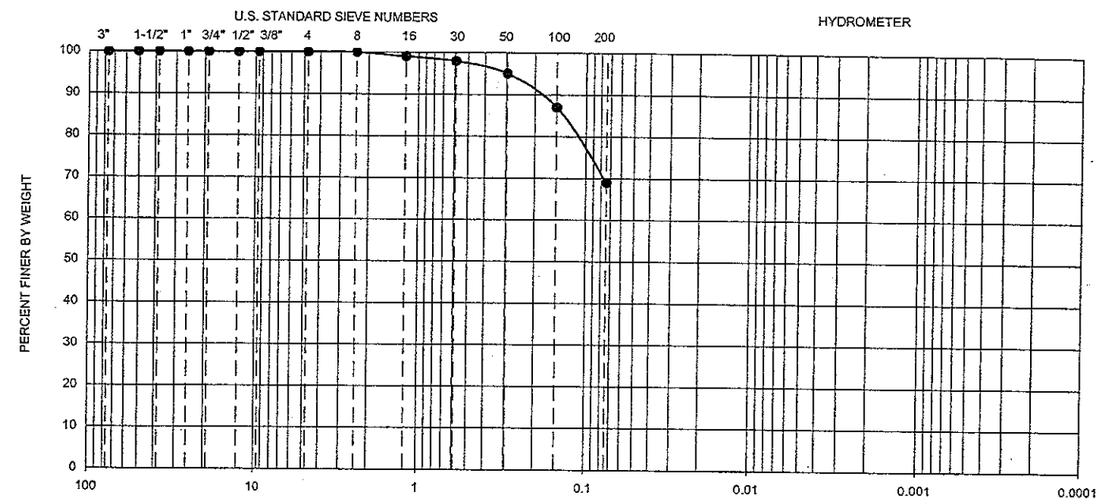
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-11

SIEVE CH-6@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
●	CH-7	5-6.5	--	--	--	--	--	--	--	--	69	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

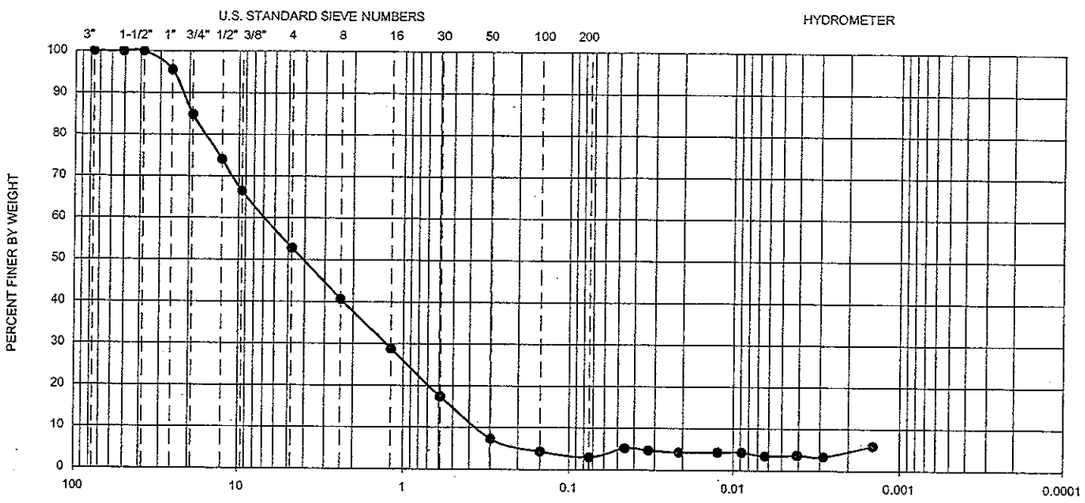
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-12

SIEVE CH-7@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
●	CH-7	15.0-16.5	-	-	NP	0.379	1.30	7.25	19.1	0.6	3	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

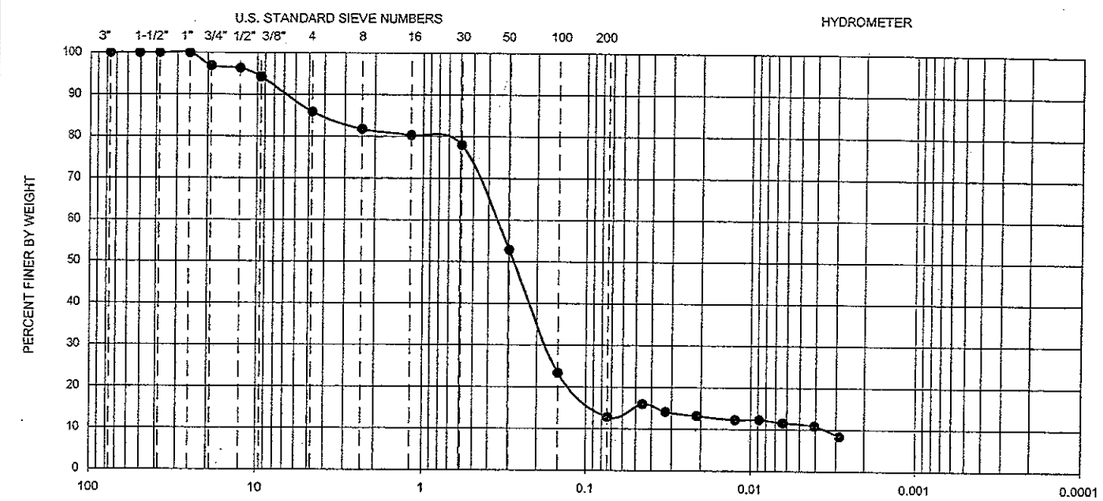
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-13

SIEVE CH-7 15.0-16.5 HYDRO2.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
●	CH-8	10.0-11.5	-	-	NP	0.004	0.18	0.38	107.2	24.4	13	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

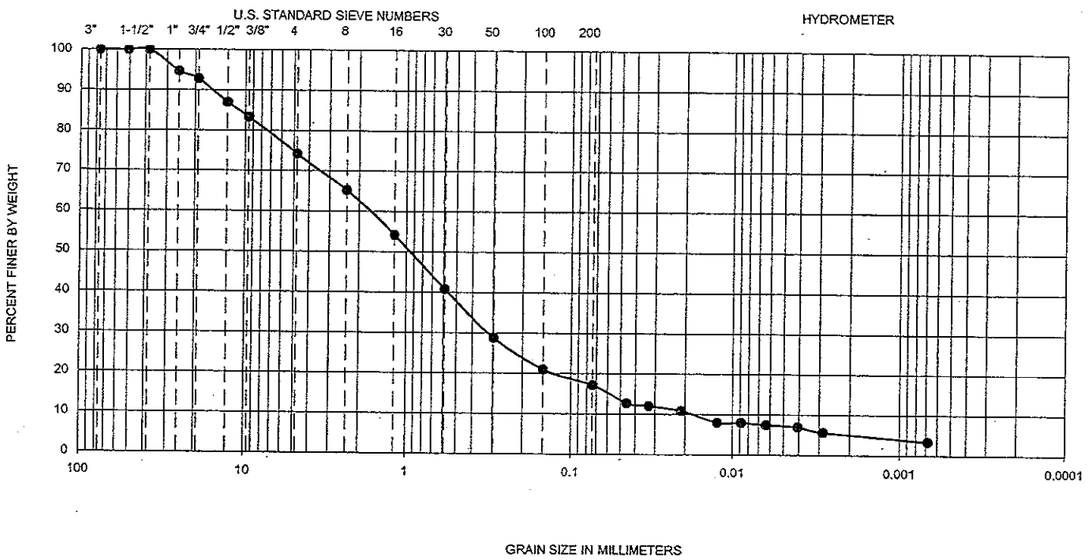
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-14

SIEVE CH-8 10.0-11.5 HYDRO2.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
●	CH-8	17.5-19.0	-	-	NP	0.019	0.33	1.80	95.0	3.2	17	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

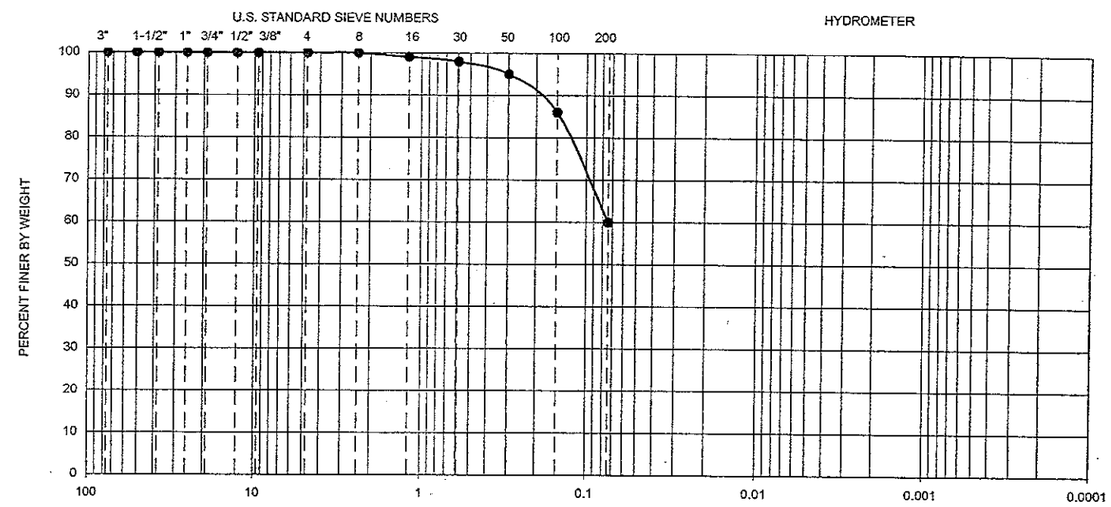
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-15

SIEVE CH-8 17.5-19.0 HYDRO2.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
●	CH-9	5-6.5	--	--	--	--	--	--	--	--	60	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

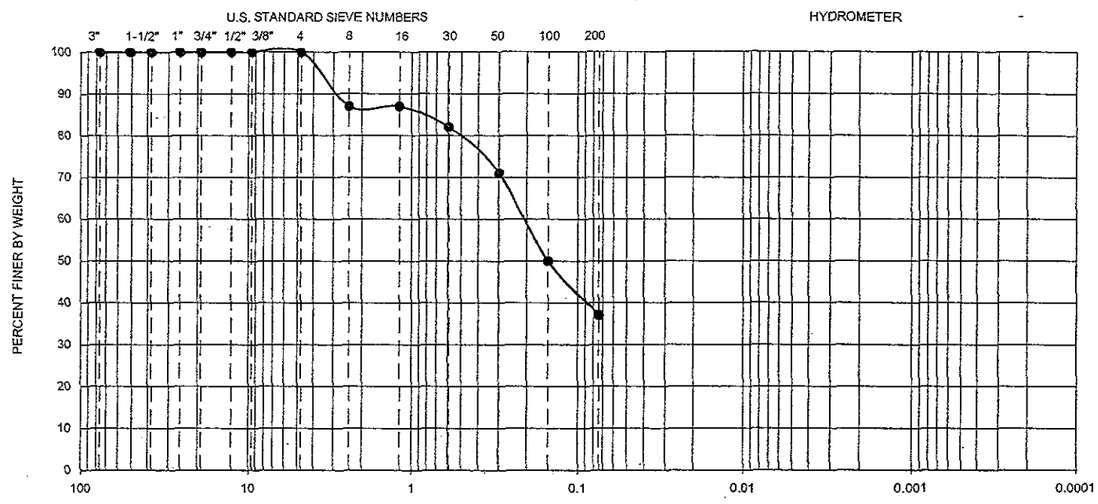
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-16

SIEVE CH-9@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
●	CH-9	20-21.5	34	17	17	–	–	–	–	–	37	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

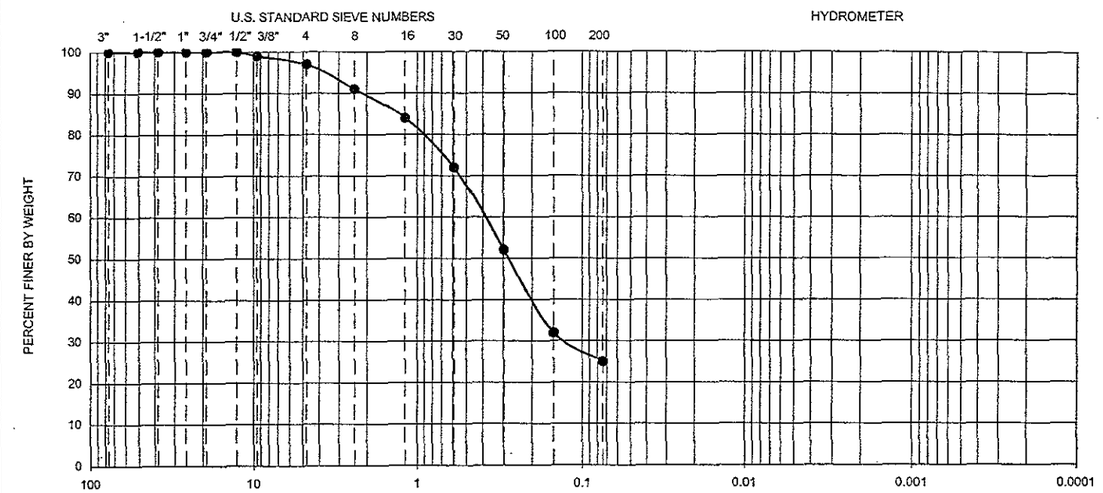
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-17

SIEVE CH-9@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
●	CH-10	17.5-19	30	21	9	–	–	–	–	–	25	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

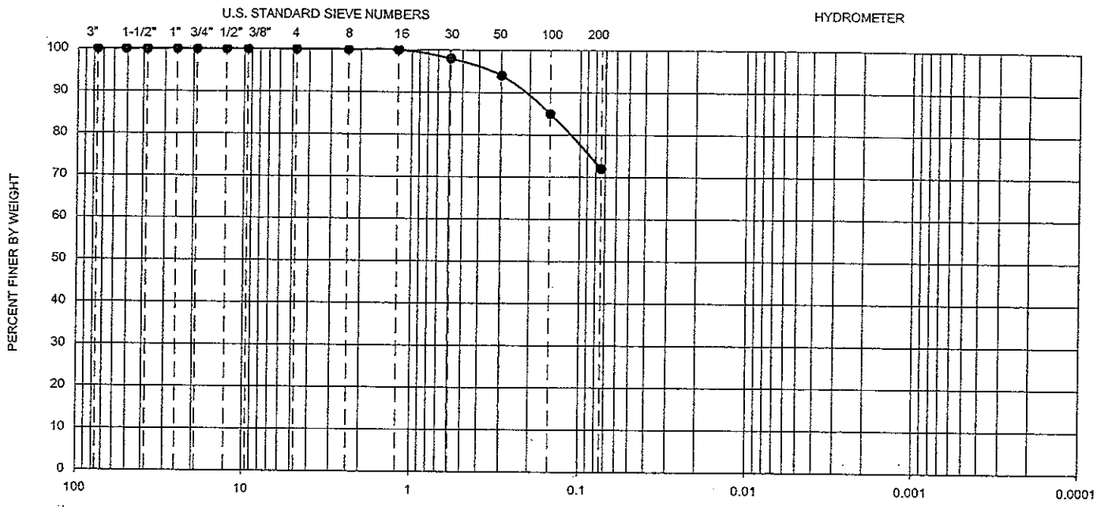
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-18

SIEVE CH-10@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
●	CH-11	2.5-4	24	14	10	–	–	–	–	–	72	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

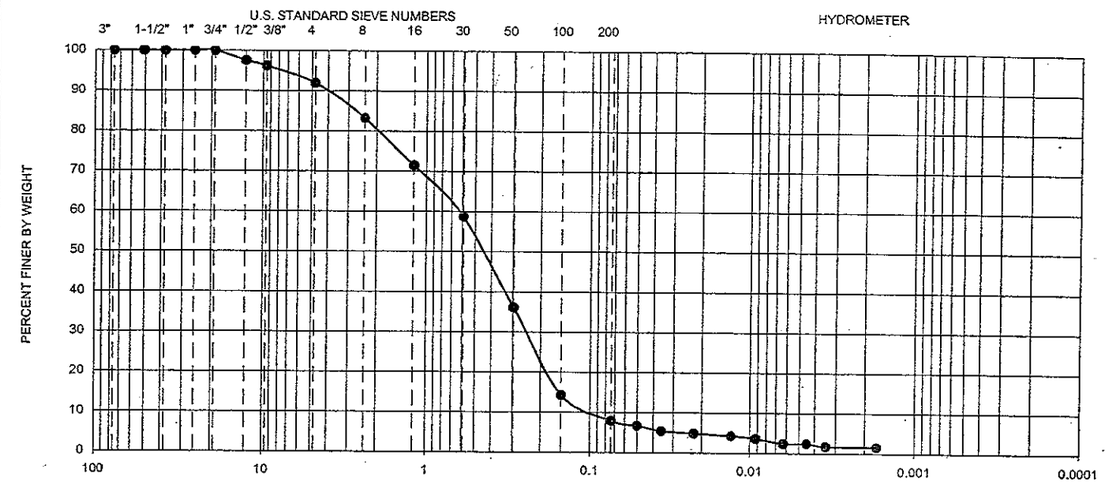
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-19

SIEVE CH-11@2.5.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
●	CH-11	15.5-17.0	-	-	NP	0.100	0.26	0.66	6.6	1.0	8	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

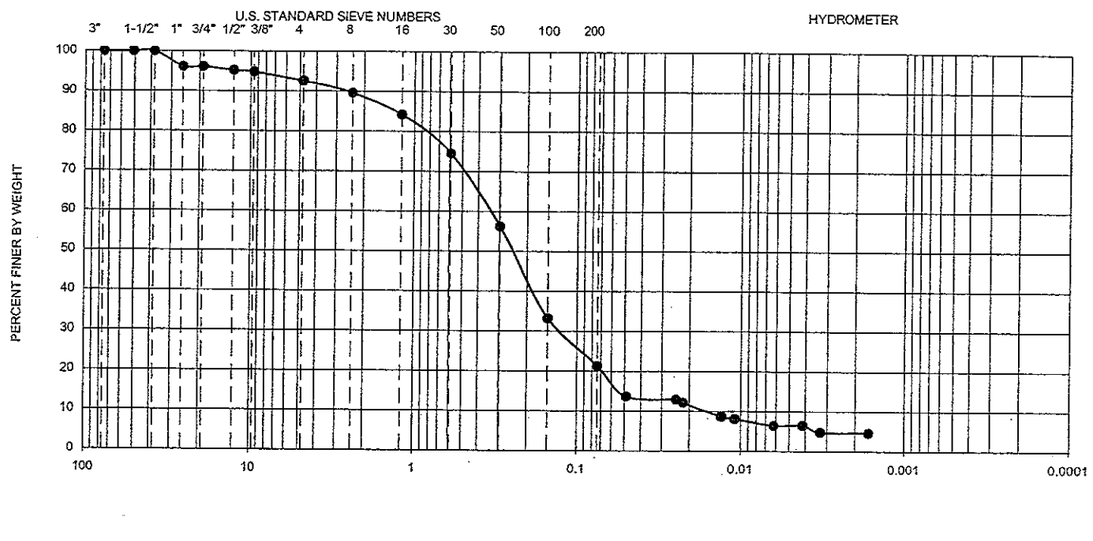
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-20

SIEVE CH-11 15.5-17.0 HYDRO3.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
●	CH-12	7.5-9.0	-	-	NP	0.016	0.13	0.36	22.1	2.8	21	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

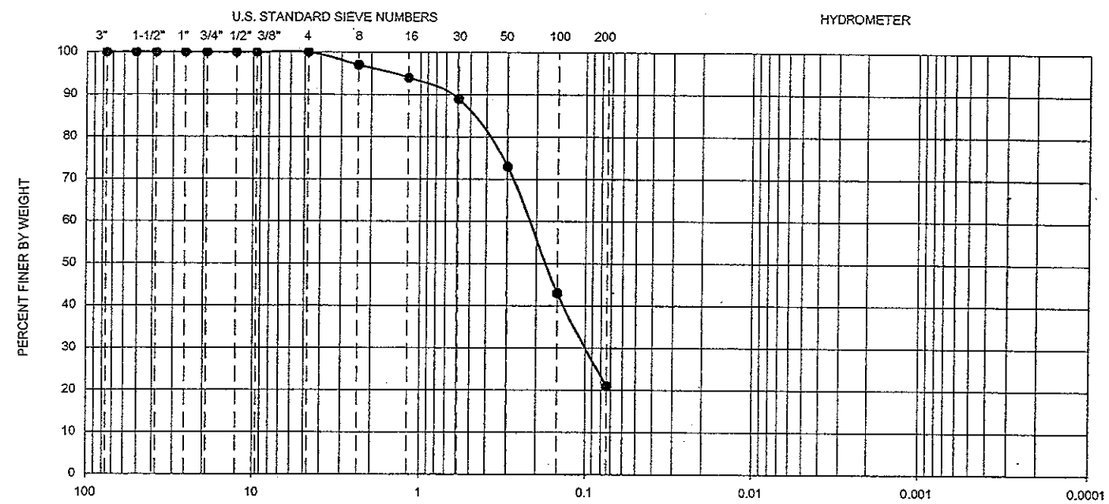
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-21

SIEVE CH-12 7.5-9.0 HYDRO3.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
●	CH-12	15-16.5	--	--	--	--	--	--	--	--	21	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

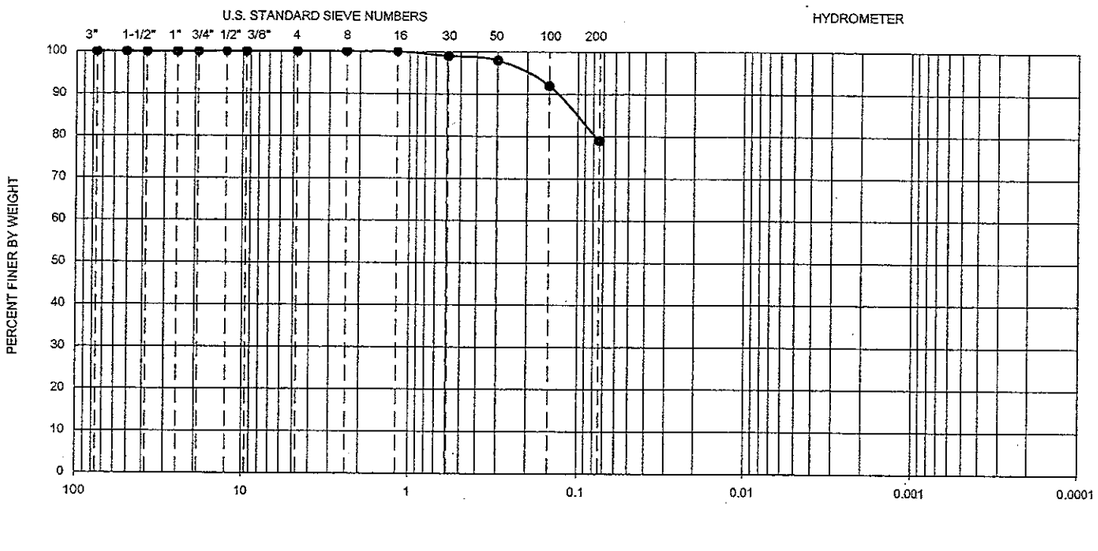
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-22

SIEVE CH-12@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
●	CH-13	5-6.5	30	28	2	--	--	--	--	--	79	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

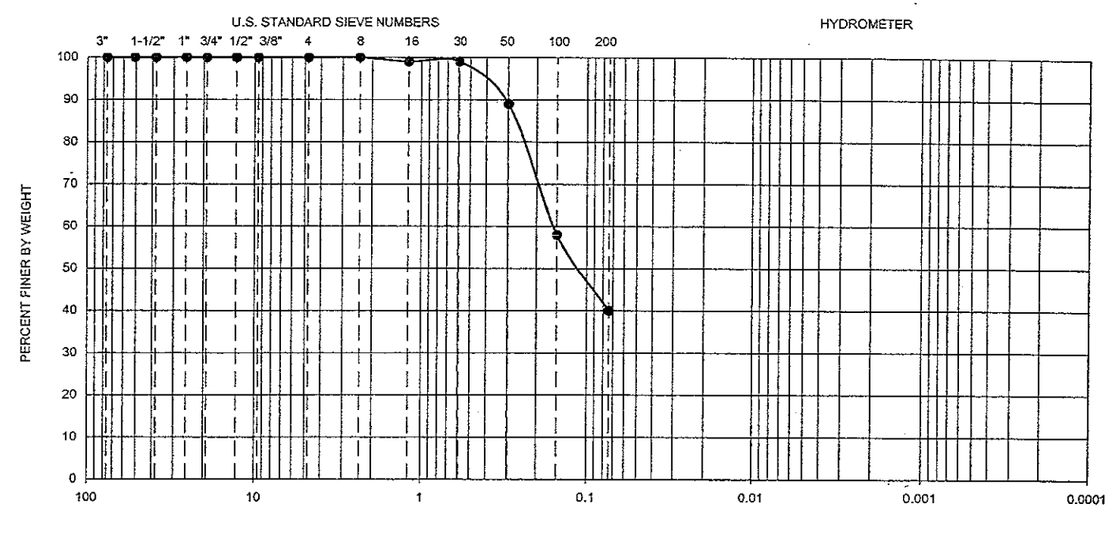
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-23

SIEVE CH-13@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
●	CH-13	15-16.5	25	22	3	--	--	--	--	--	40	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

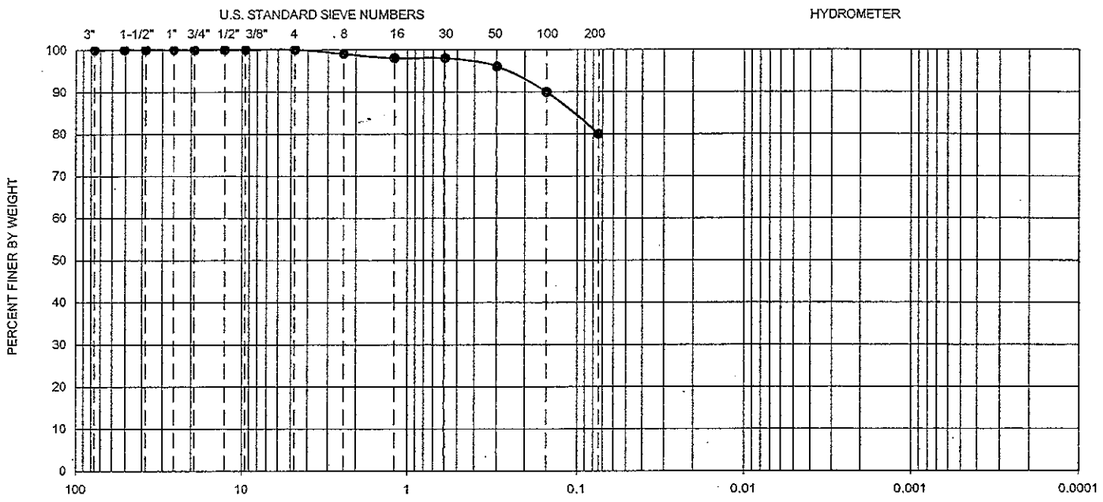
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-24

SIEVE CH-13@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
●	CH-14	2.5-4	36	19	17	–	–	–	–	–	80	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

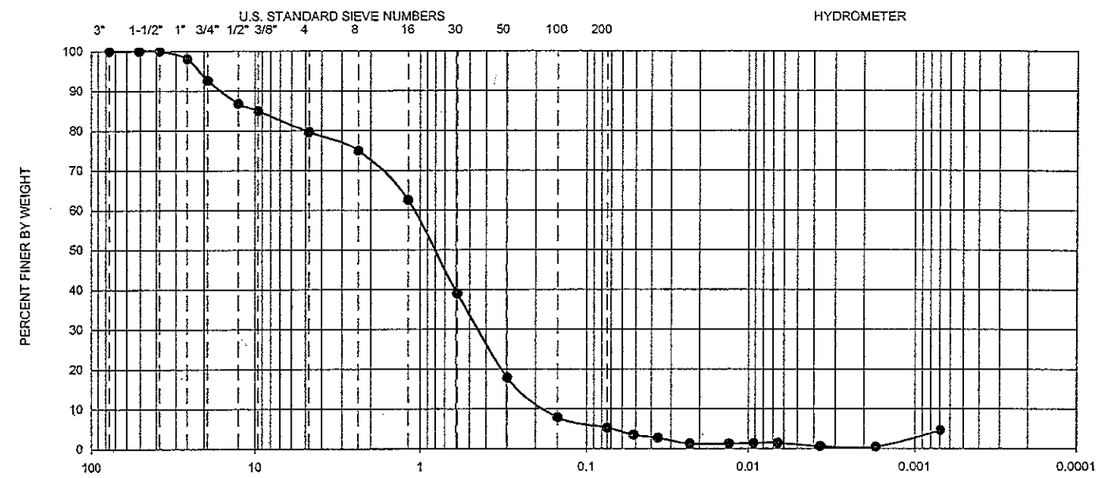
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-25

SIEVE CH-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
●	CH-14	15.0-16.5	-	-	NP	0.182	0.47	1.12	6.1	1.1	5	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

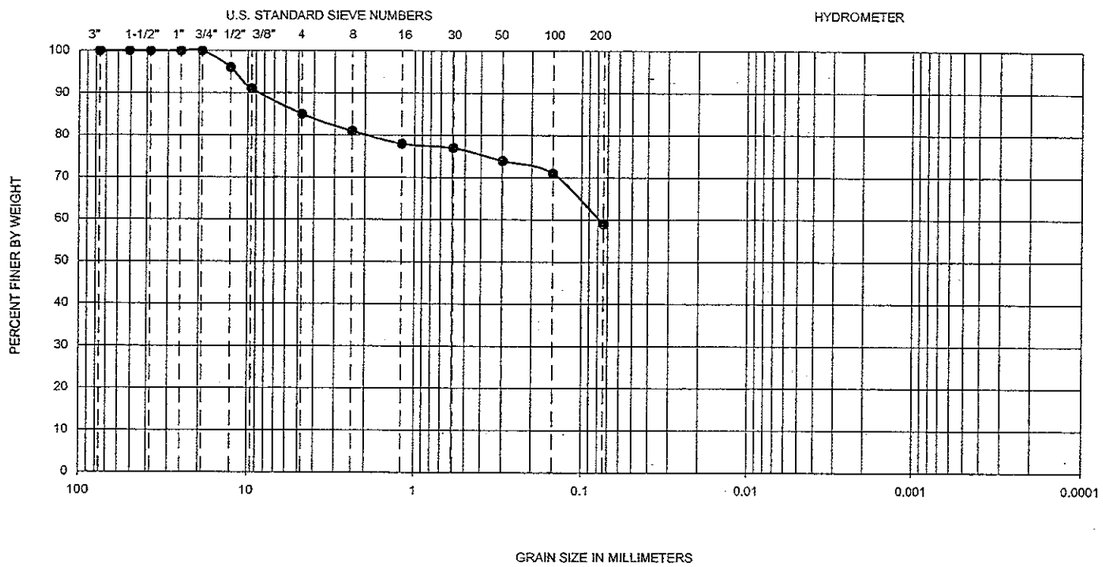
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-26

SIEVE CH-14 15.0-16.5 HYDRO3.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
●	CH-15	17.5-19	28	23	5	--	--	--	--	--	59	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

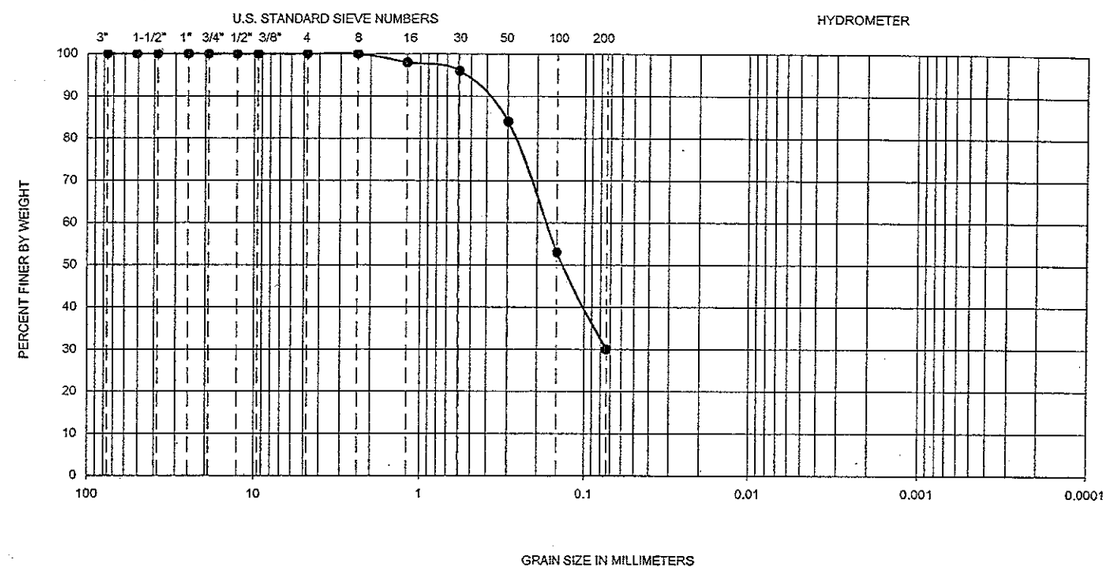
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-27

SIEVE CH-15@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
●	CH-16	15-16.5	--	--	--	--	--	--	--	--	30	SM+CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

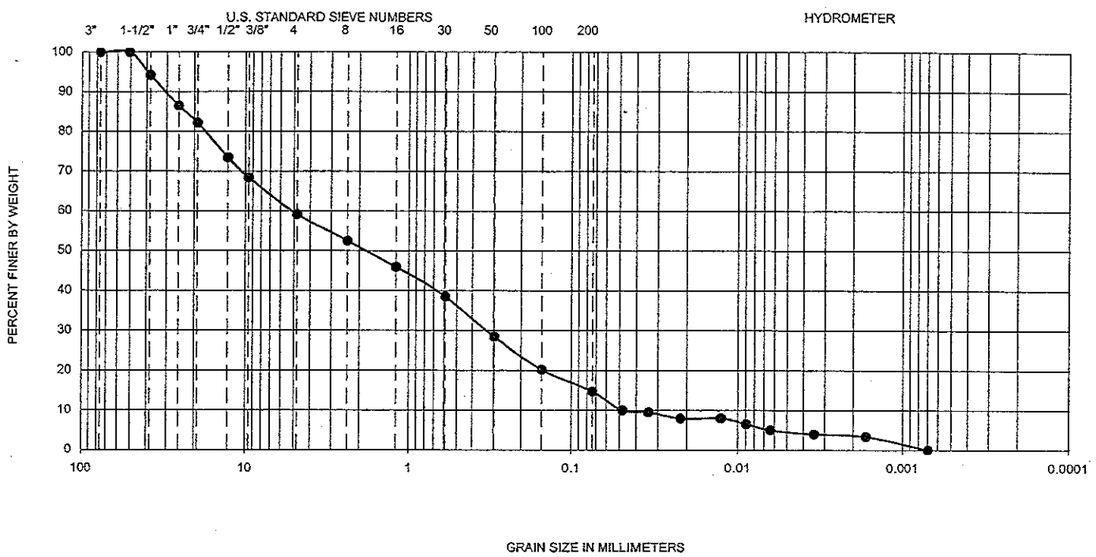
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-28

SIEVE CH-16@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
●	CH-16	20.0-21.5	-	-	NP	0.048	0.35	5.19	108.1	0.5	15	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

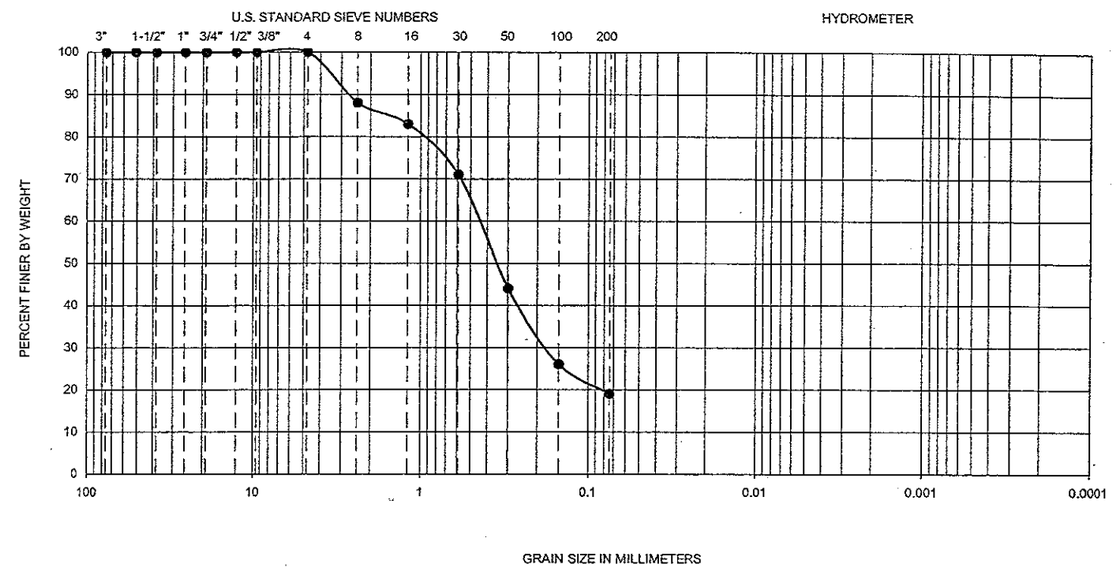
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-29

SIEVE CH-16 20.0-21.5 HYDRO3.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
●	CH-16	32.5-34	--	--	--	--	--	--	--	--	19	SM+CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

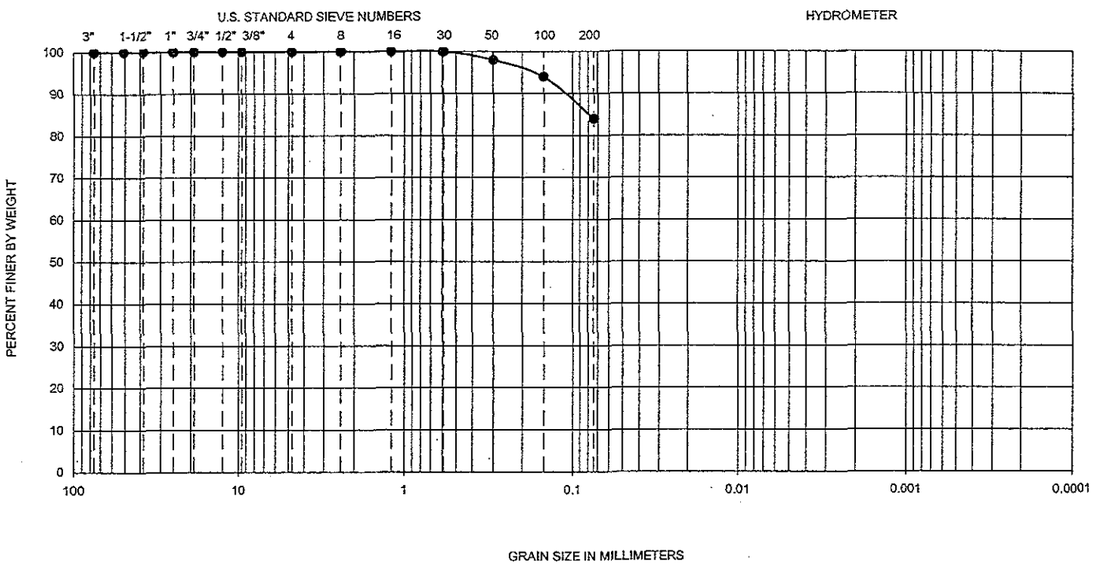
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-30

SIEVE CH-16@32.5-34.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
●	CH-17	7.5-9	40	23	17	--	--	--	--	--	84	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

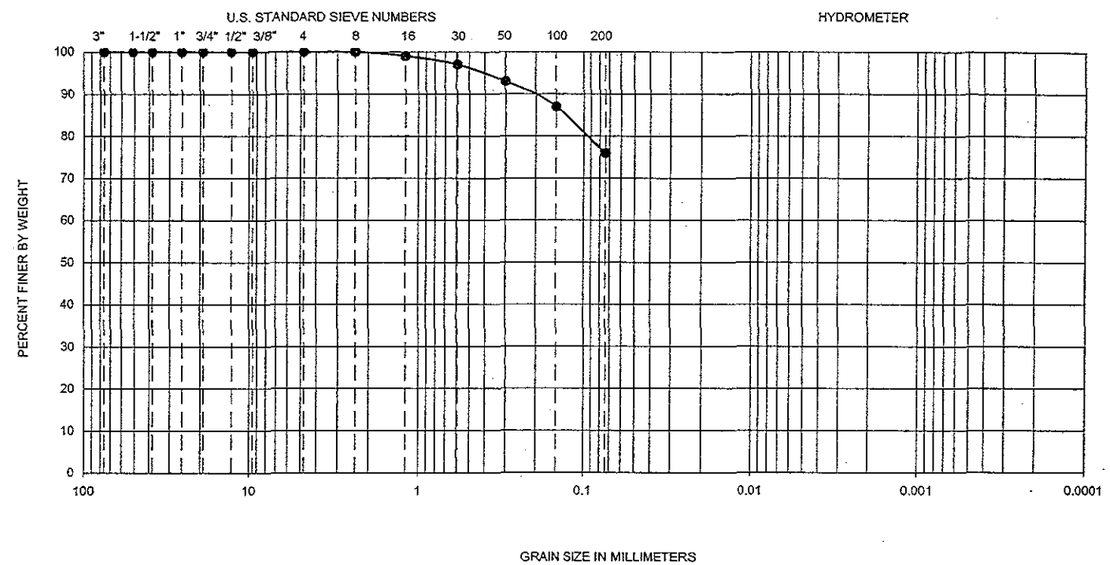
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-31

SIEVE CH-17@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
●	CH-17	22.5-24	29	19	10	--	--	--	--	--	76	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

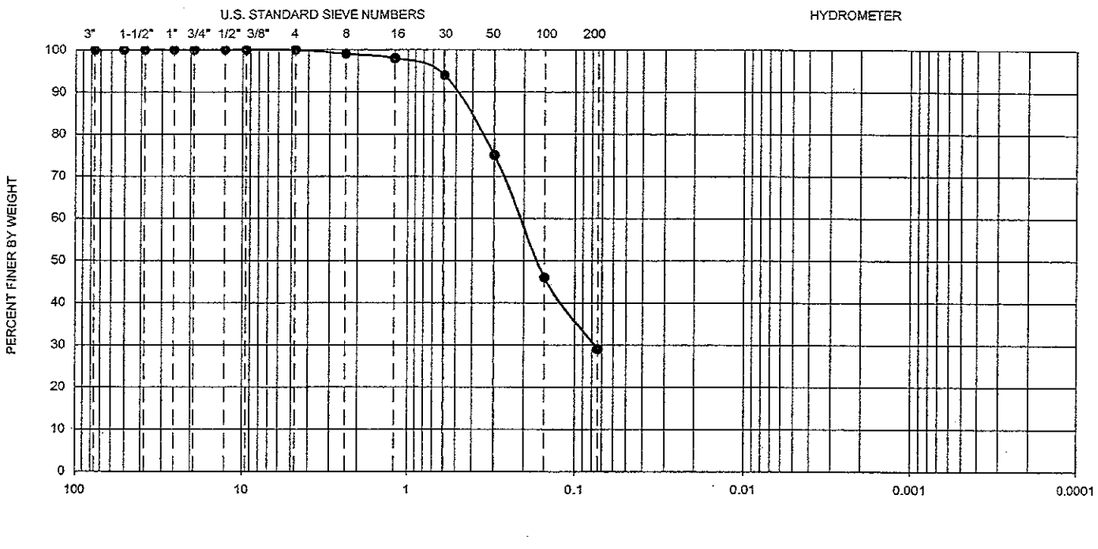
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-32

SIEVE CH-17@22.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
●	CH-18	10-11.5	–	–	–	–	–	–	–	–	29	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

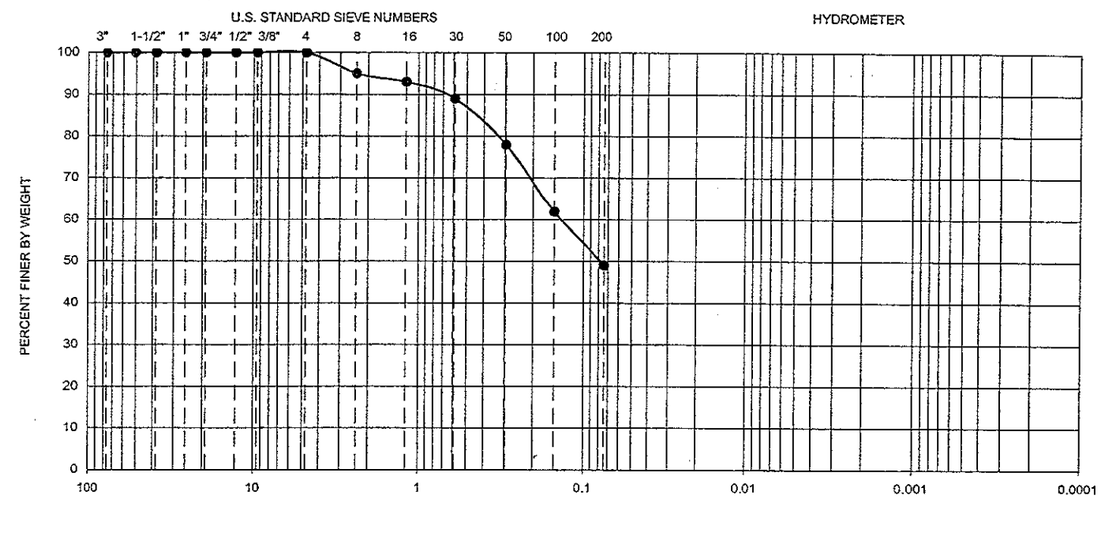
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-33

SIEVE CH-18@10.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
●	CH-19	2.5-4	–	–	–	–	–	–	–	–	49	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

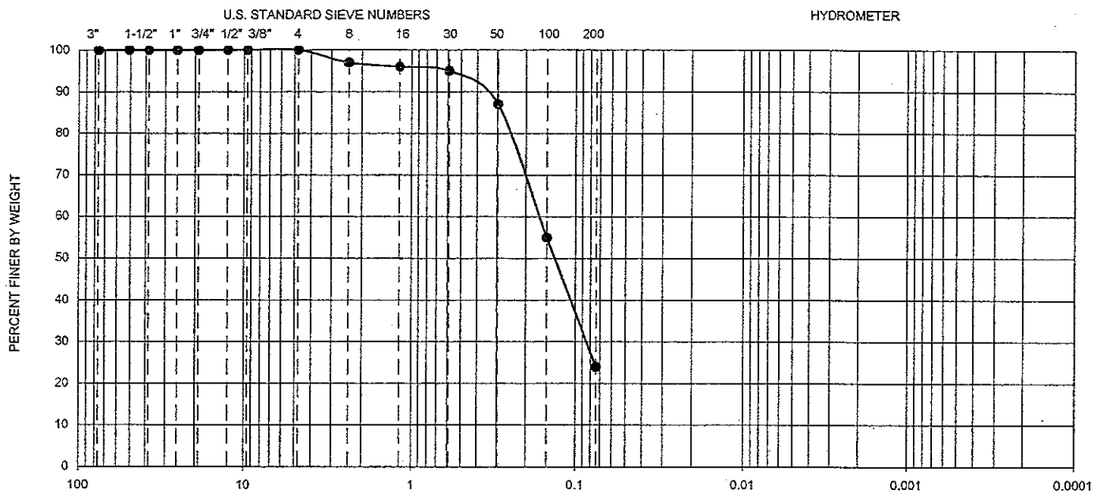
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-34

SIEVE CH-19@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
●	CH-19	22.5-24	--	--	--	--	--	--	--	--	24	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

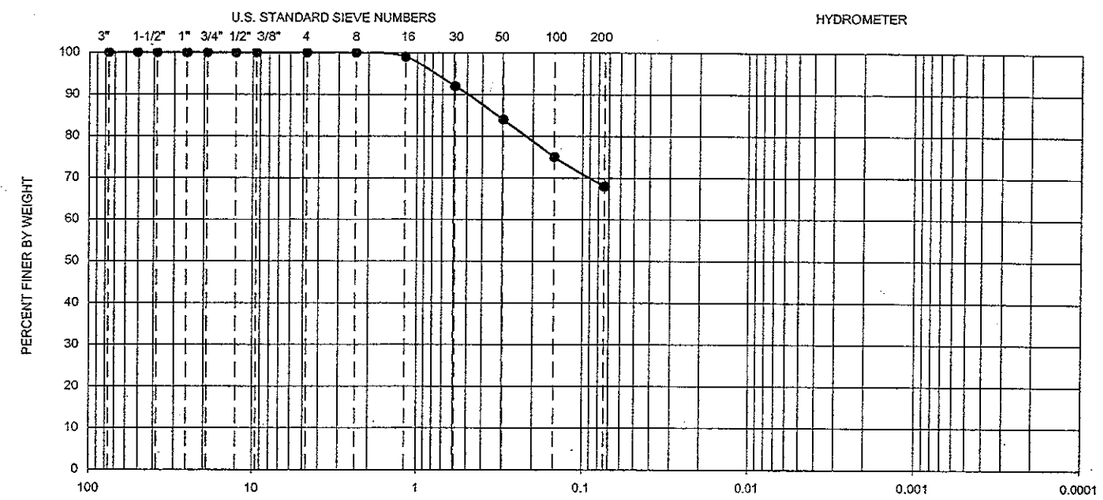
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-35

SIEVE CH-19@22.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
●	CH-20	10-11.5	--	--	--	--	--	--	--	--	68	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

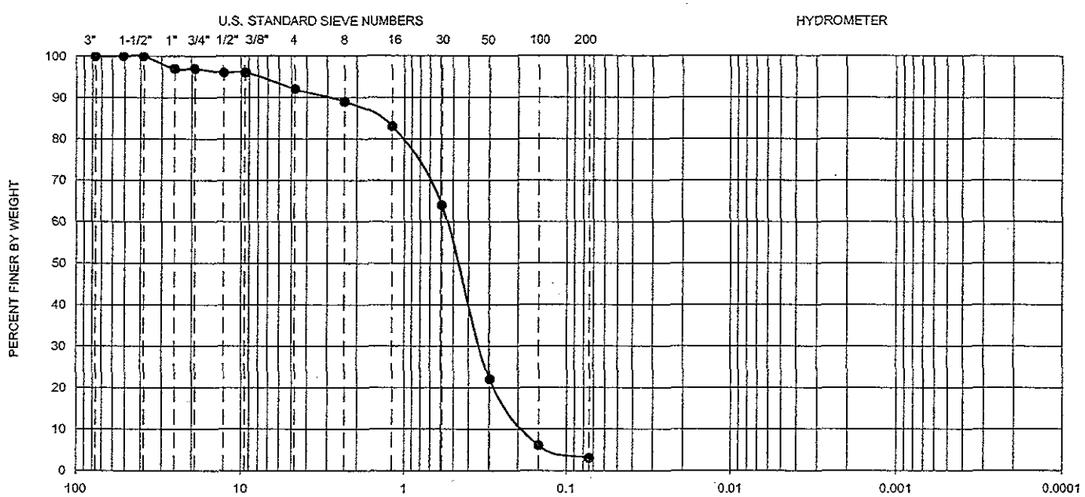
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-36

SIEVE CH-20@10.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
●	CH-20	25-26.5	--	--	--	--	--	--	--	--	3	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

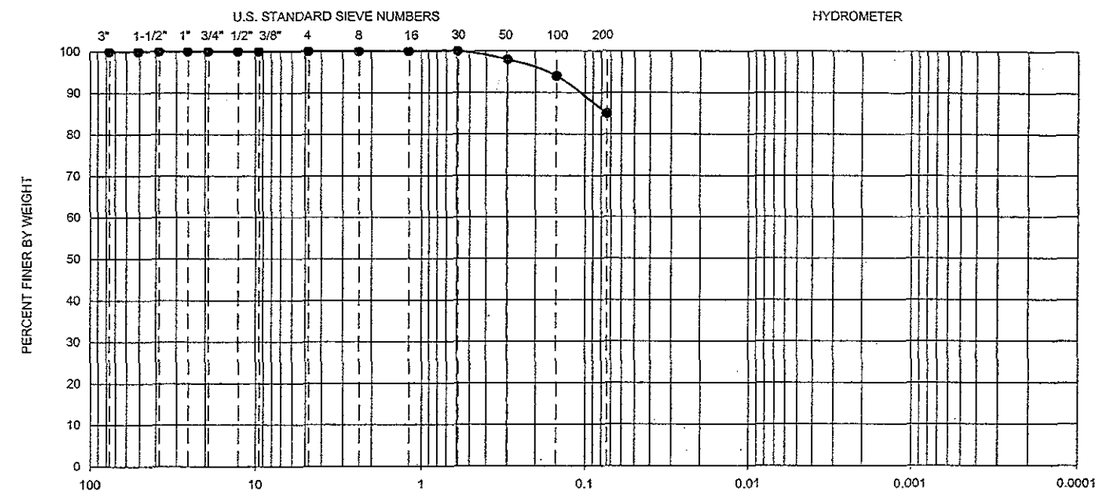
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-37

SIEVE CH-20@25.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
●	CH-21	7.5-9	--	--	--	--	--	--	--	--	85	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

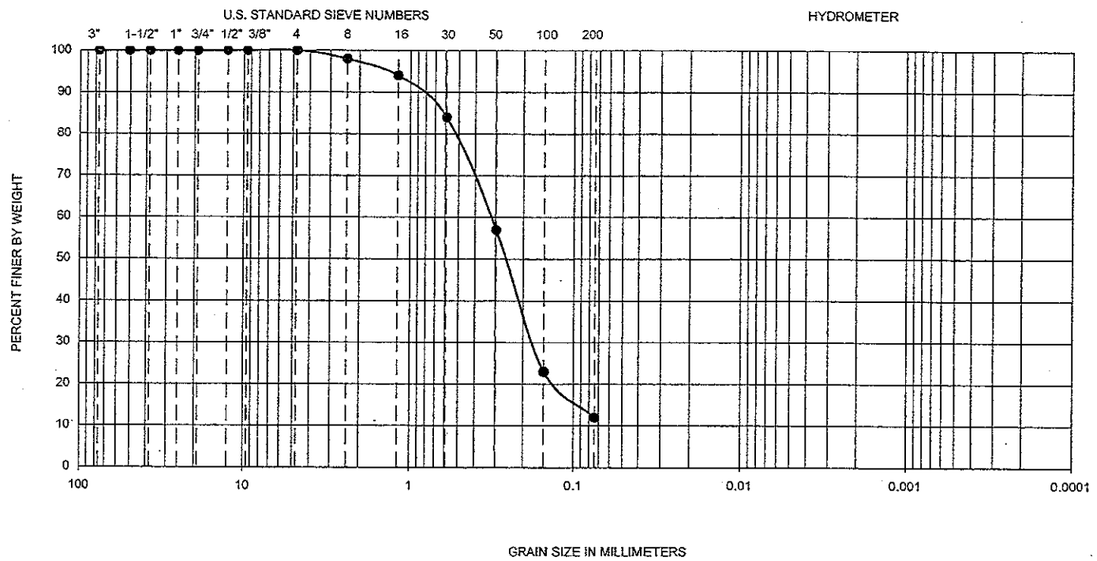
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-38

SIEVE CH-21@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
●	CH-21	22.5-24	--	--	--	--	--	--	--	--	12	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

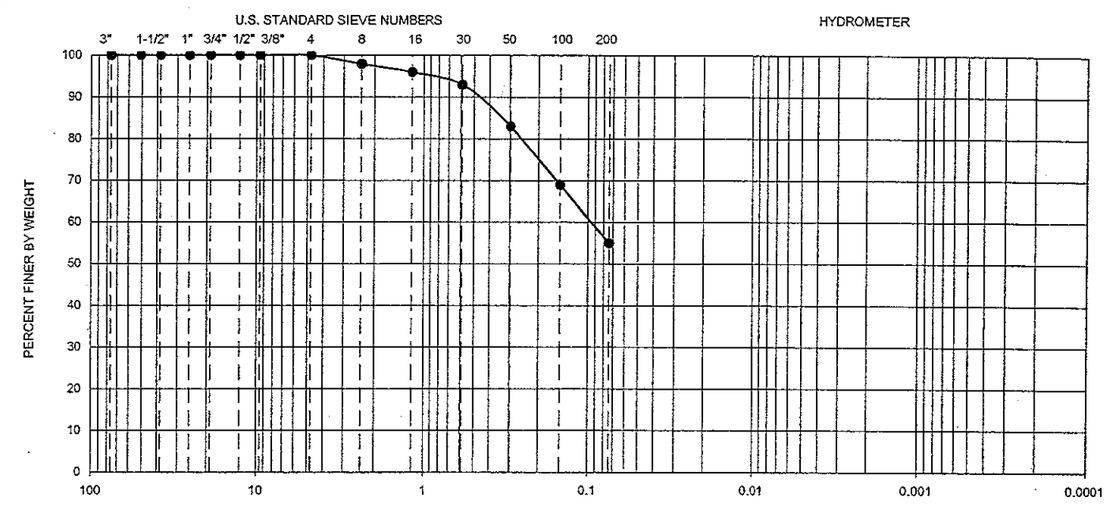
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-39

SIEVE CH-21@22.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
●	CH-22	15-16.5	44	18	26	--	--	--	--	--	55	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

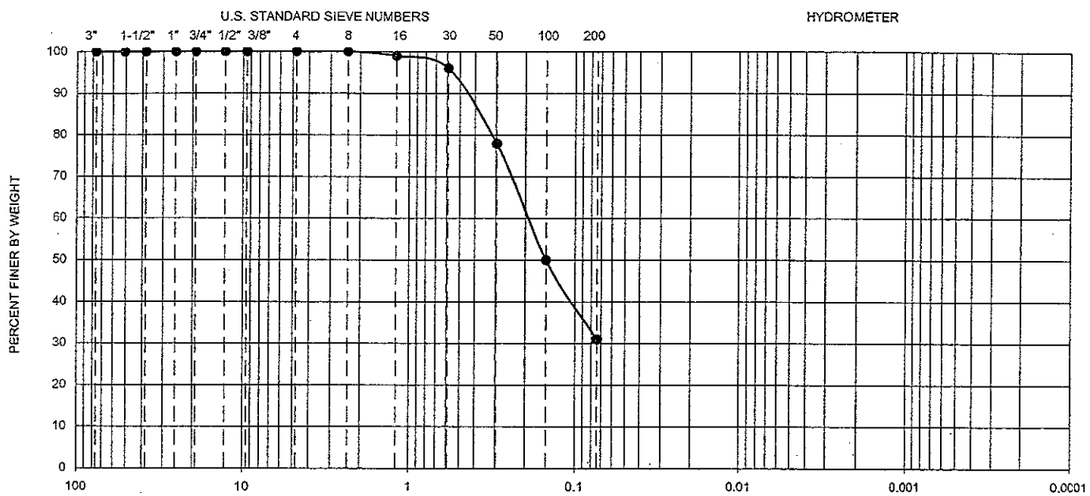
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-40

SIEVE CH-22@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
●	CH-22	25-26.5	--	--	--	--	--	--	--	--	31	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

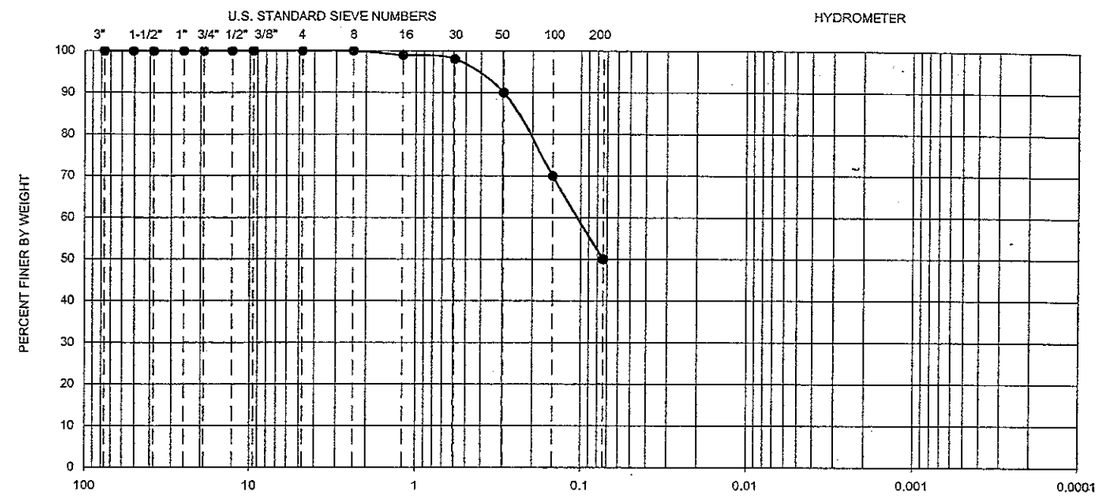
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-41

SIEVE CH-22@25.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
●	CH-23	7.5-9	21	16	5	--	--	--	--	--	50	CL-ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

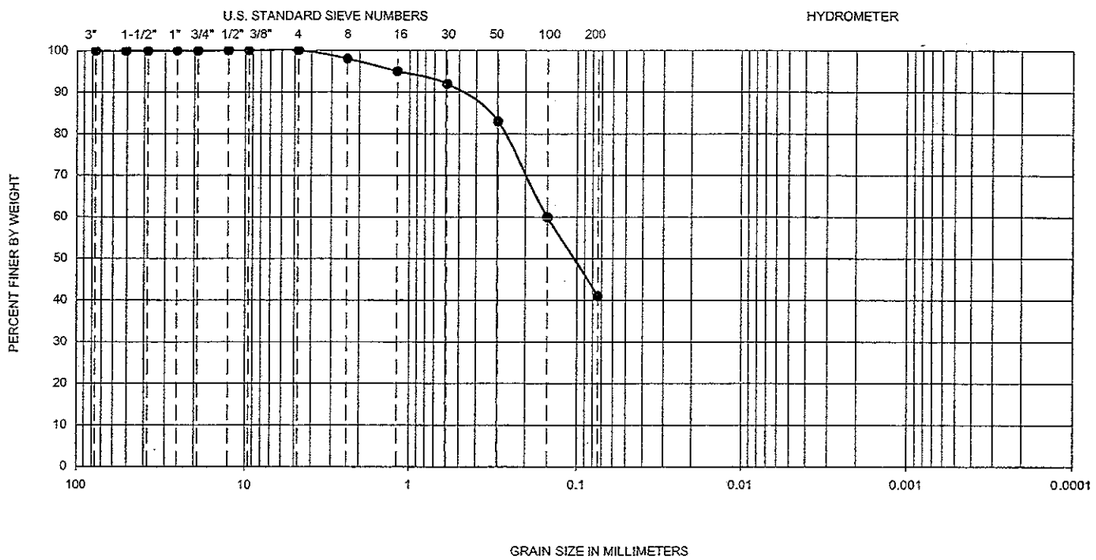
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-42

SIEVE CH-23@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
●	CH-24	17.5-19	--	--	--	--	--	--	--	--	41	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

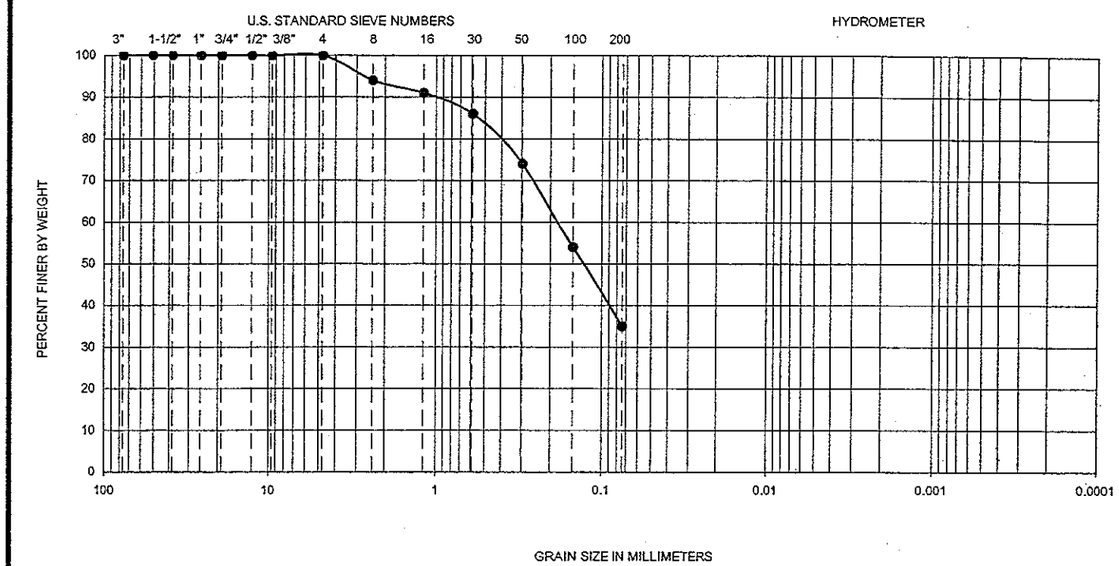
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-43

SIEVE CH-24@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
●	CH-24	22.5-24	--	--	--	--	--	--	--	--	35	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

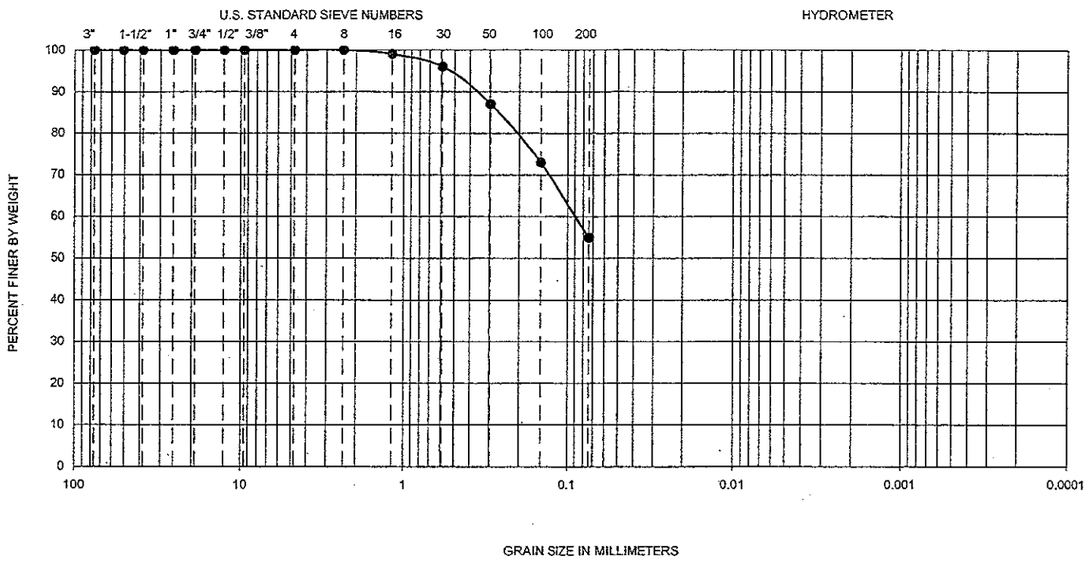
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-44

SIEVE CH-24@22.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
●	CH-25	12.5-14	21	16	5	–	–	–	–	–	55	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

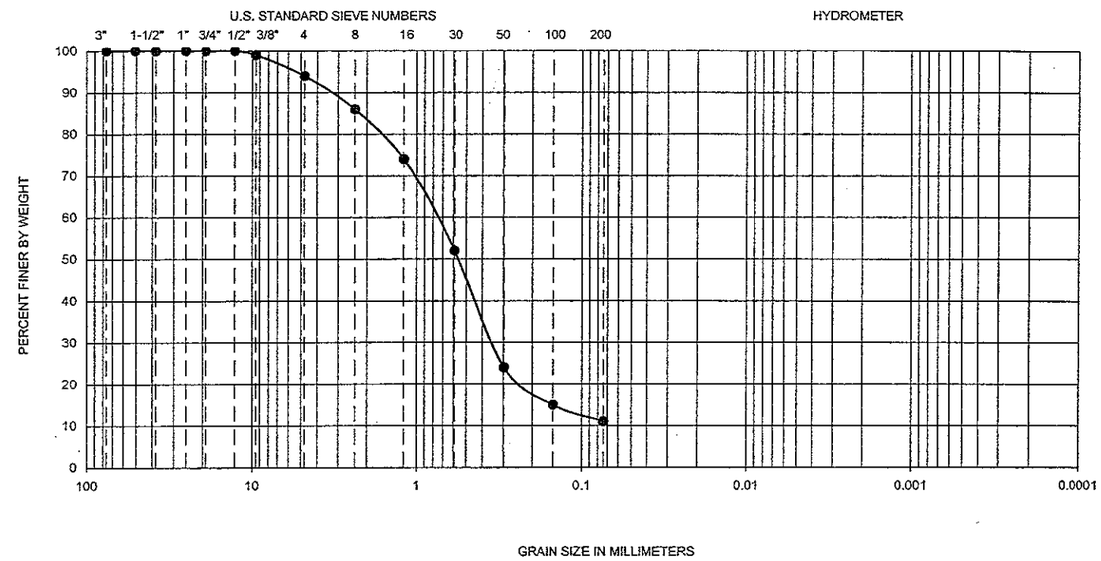
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-45

SIEVE CH-25@12.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
●	CH-25	20-21.5	–	–	–	–	–	–	–	–	11	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

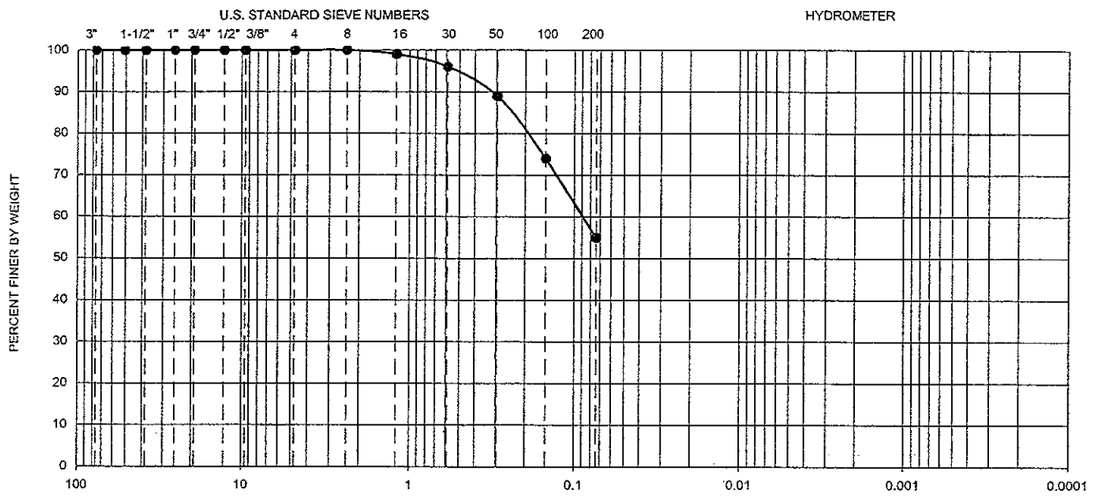
GRADATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-46

SIEVE CH-25@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
●	CH-26	2.5-4	26	21	5	--	--	--	--	--	55	CL-ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

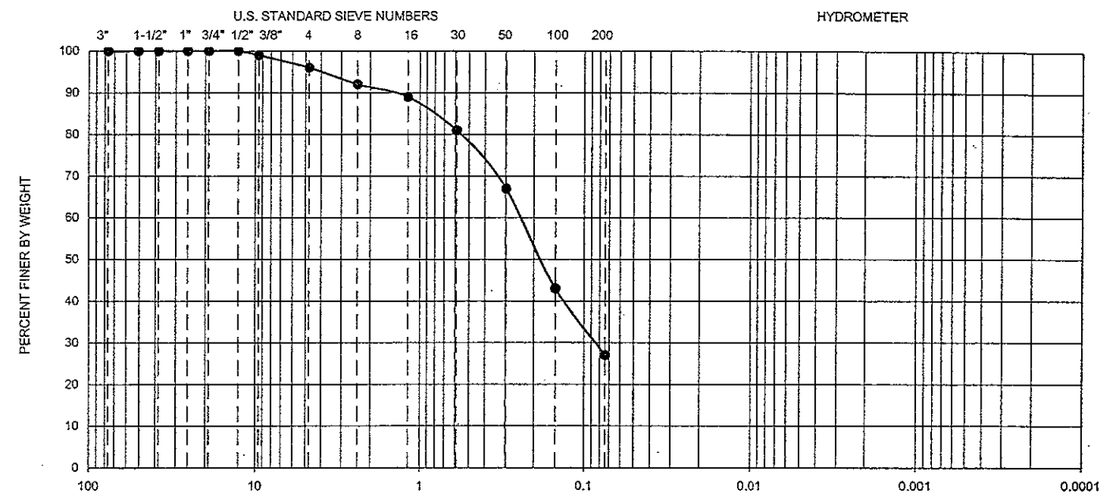
Ninyo & Moore

GRADATION TEST RESULTS
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-47

SIEVE CH-26@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
●	CH-26	20-21.5	--	--	--	--	--	--	--	--	27	SM+CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

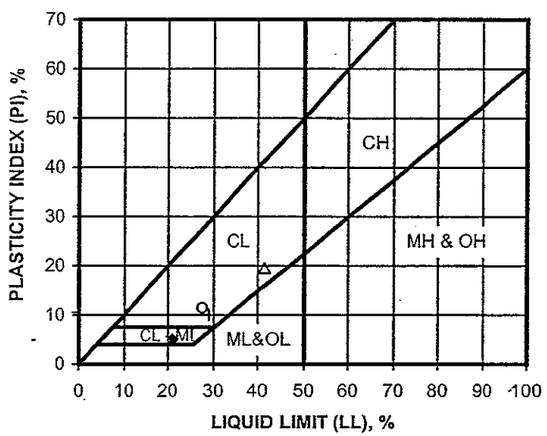
GRADATION TEST RESULTS
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-48

SIEVE CH-26@20.xls

SYMBOL	LOCATION	DEPTH (FT)	LL (%)	PL (%)	PI (%)	U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction)	U.S.C.S. (Entire Sample)
●	CH-1	7.5-9	-	-	-	NP	ML
■	CH-1	20.0-21.5	-	-	-	NP	SM
◆	CH-2	5-6.5	21	16	5	CL-ML	CL
○	CH-2	15-16.5	28	16	12	CL	SM
□	CH-3	15-16.5	-	-	-	NP	SM
△	CH-4	7.5-9	42	22	20	CL	CL
X	CH-4	17.5-19.0				NP	SP-SM
+	CH-5	5-6.5	29	19	10	CL	CL

NP - Indicates non-plastic



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

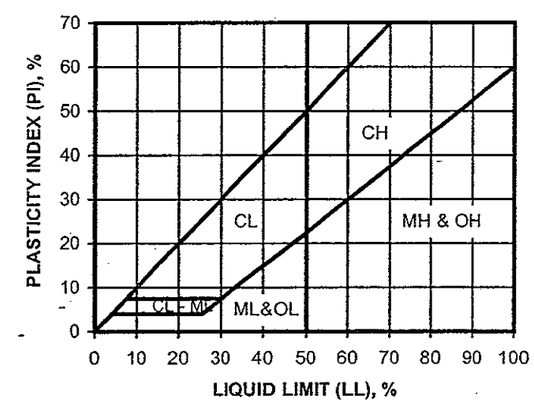
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-49

ATTERBERG #261.xls

SYMBOL	LOCATION	DEPTH (FT)	LL (%)	PL (%)	PI (%)	U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction)	U.S.C.S. (Entire Sample)
●	CH-5	15-16.5	-	-	-	NP	ML
■	CH-6	7.5-9	-	-	-	NP	ML
◆	CH-6	17.5-19	-	-	-	NP	SM
○	CH-7	5-6.5	-	-	-	NP	ML
□	CH-7	15-16.5	-	-	-	NP	SP
△	CH-8	10-11.5	-	-	-	NP	SM
X	CH-8	17.5-19.0	-	-	-	NP	SM
+	CH-9	5-6.5	-	-	-	NP	ML

NP - Indicates non-plastic



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

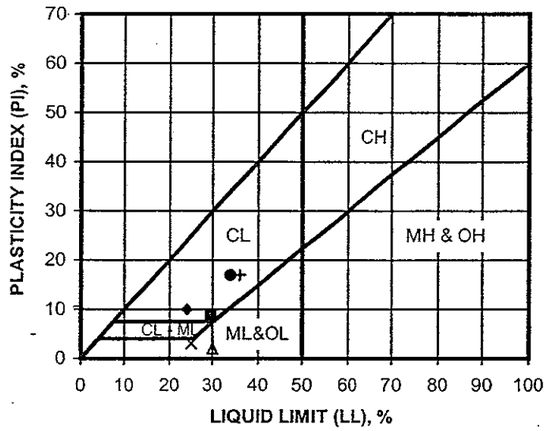
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-50

ATTERBERG #255.xls

SYMBOL	LOCATION	DEPTH (FT)	LL (%)	PL (%)	PI (%)	U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction)	U.S.C.S. (Entire Sample)
●	CH-9	20-21.5	34	17	17	CL	SC
■	CH-10	17.5-19	30	21	9	CL	SC
◆	CH-11	2.5-4.0	24	14	10	CL	CL
○	CH-11	15.5-17	-	-	-	NP	SW-SM
□	CH-12	15-16.5	-	-	-	NP	SM
Δ	CH-13	5-6.5	30	28	2	ML	ML
X	CH-13	15-16.5	25	22	3	ML	SM
+	CH-14	2.5-4	36	19	17	CL	CL

NP - Indicates non-plastic



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

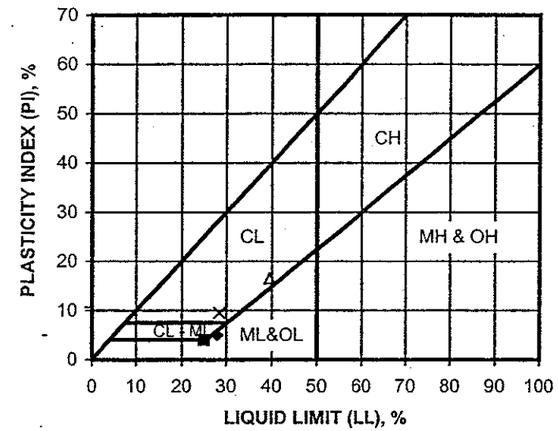
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-51

CH ONLY #260A ATTERBERG.xls

SYMBOL	LOCATION	DEPTH (FT)	LL (%)	PL (%)	PI (%)	U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction)	U.S.C.S. (Entire Sample)
●	CH-14	15.0-16.5	-	-	-	NP	SW-SM
■	CH-15	7.5-9	25	21	4	CL-ML	CL-ML
◆	CH-15	17.5-19	28	23	5	ML	ML
○	CH-16	15-16.5	-	-	-	NP	SM+CL
□	CH-16	32.5-34	-	-	-	NP	SM+CL
Δ	CH-17	7.5-9	40	23	17	CL	CL
X	CH-17	22.5-24	29	19	10	CL	CL
+	CH-18	10-11.5	-	-	-	NP	SM+CL

NP - Indicates non-plastic



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

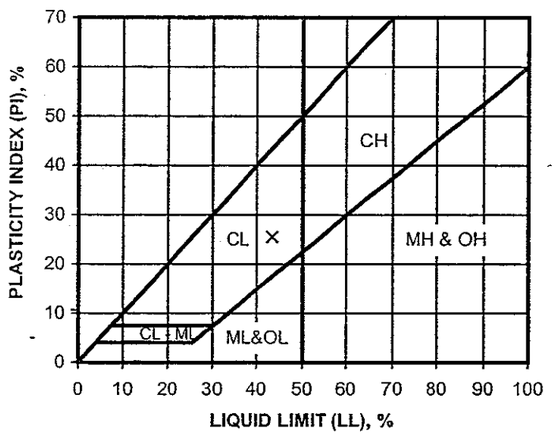
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-52

CH ONLY #260B ATTERBERG.xls

SYMBOL	LOCATION	DEPTH (FT)	LL (%)	PL (%)	PI (%)	U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction)	U.S.C.S. (Entire Sample)
●	CH-19	2.5-4	-	-	-	NP	SM
■	CH-19	22.5-24	-	-	-	NP	SM
◆	CH-20	10-11.5	-	-	-	NP	ML
○	CH-20	25-26.5	-	-	-	NP	SP
□	CH-21	7.5-9	-	-	-	NP	ML
△	CH-21	22.5-24	-	-	-	NP	SM
X	CH-22	15-16.5	44	18	26	CL	CL
+	CH-22	25-26.5	-	-	-	NP	SM

NP - Indicates non-plastic



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

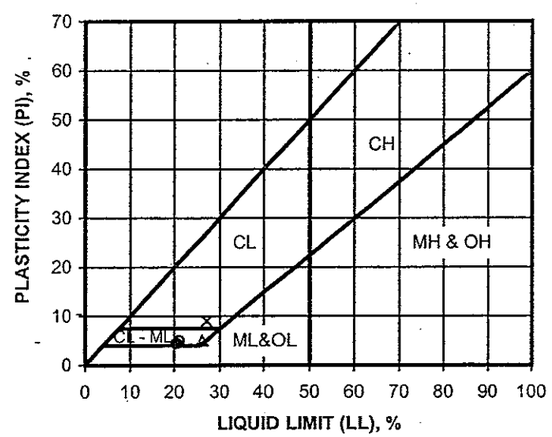
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-53

ATTERBERG #259.xls

SYMBOL	LOCATION	DEPTH (FT)	LL (%)	PL (%)	PI (%)	U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction)	U.S.C.S. (Entire Sample)
●	CH-23	7.5-9	21	16	5	CL-ML	CL-ML
■	CH-24	17.5-19	-	-	-	NP	SM
◆	CH-24	22.5-24	-	-	-	NP	SM
○	CH-25	12.5-14	21	16	5	CL-ML	CL
□	CH-25	20-21.5	-	-	-	NP	SM
△	CH-26	2.5-4	26	21	5	CL-ML	CL
X	CH-26	20-21.5	27	18	9	CL	SM+CL

NP - Indicates non-plastic



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-98

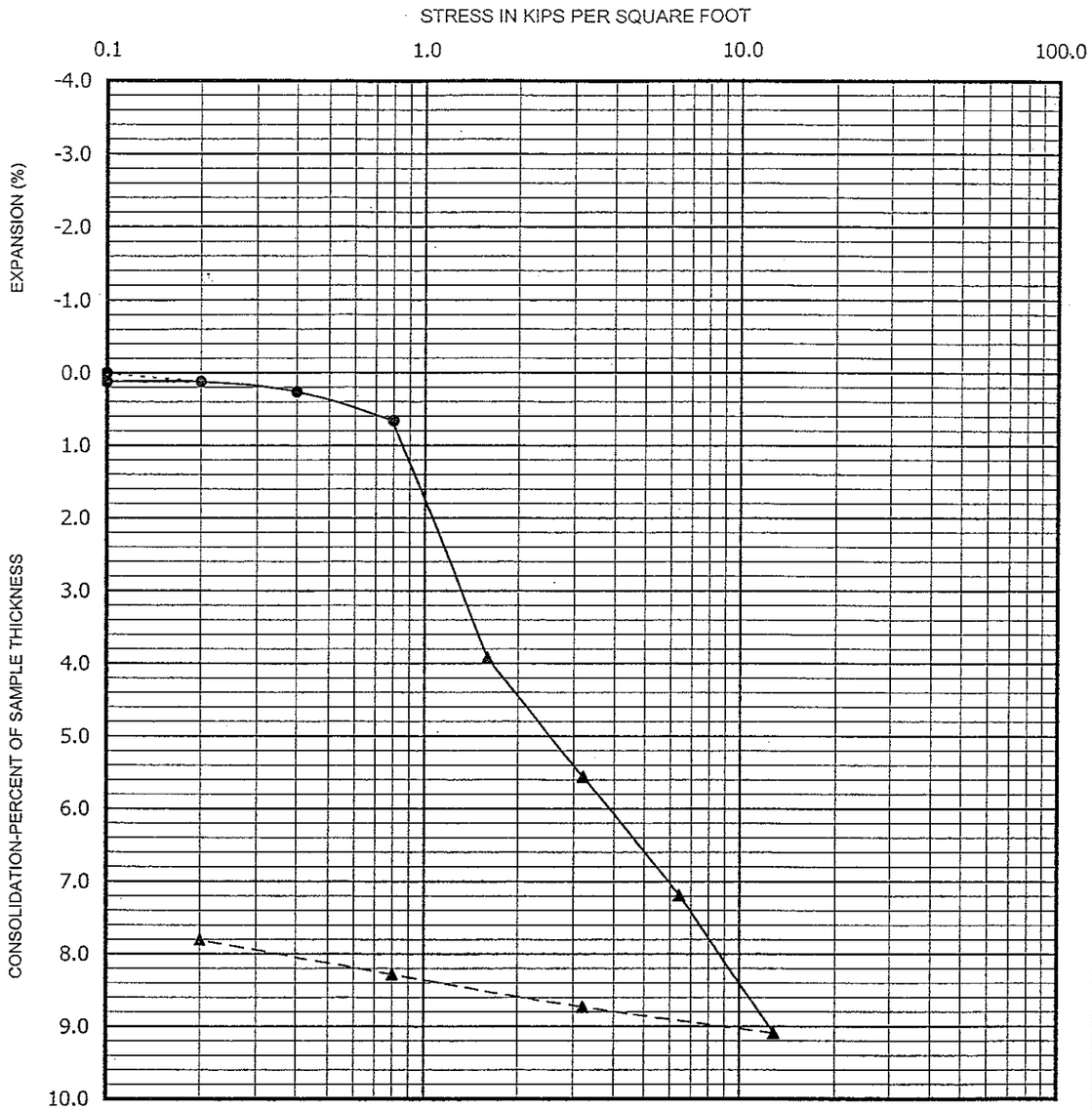
Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-54

ATTERBERG #259 B.xls



---●---	Seating Cycle	Boring No.	CH-1
—●—	Loading Prior to Inundation	Depth (ft.)	7.5-9
—▲—	Loading After Inundation	Soil Type	ML
---▲---	Rebound Cycle		

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

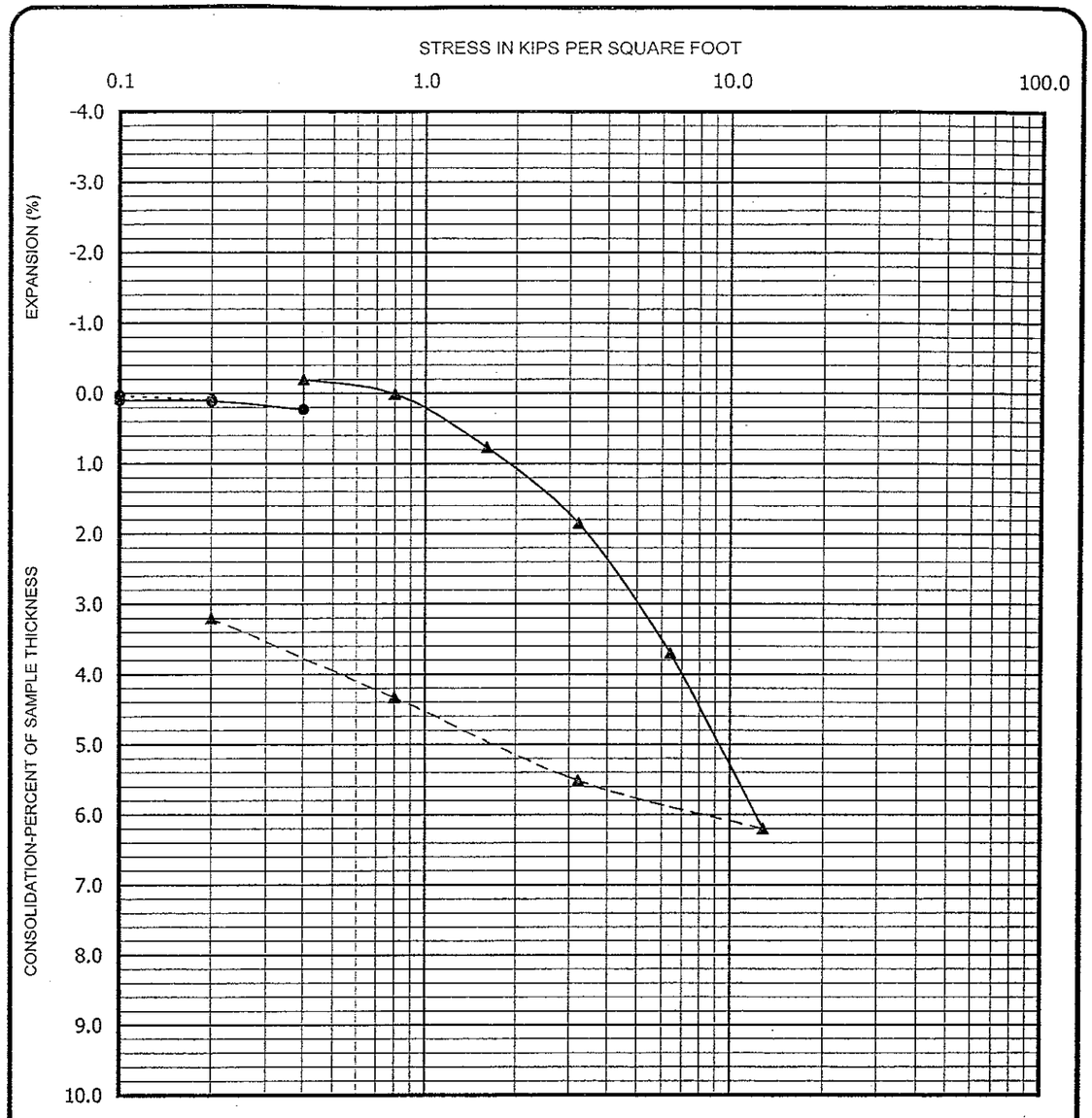
Ninyo & Moore

CONSOLIDATION #248 CH-1 7.5-9.xls

CONSOLIDATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-55



---●---	Seating Cycle	Boring No.	CH-2
—●—	Loading Prior to Inundation	Depth (ft.)	2.5-4
—▲—	Loading After Inundation	Soil Type	CL
---▲---	Rebound Cycle		

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

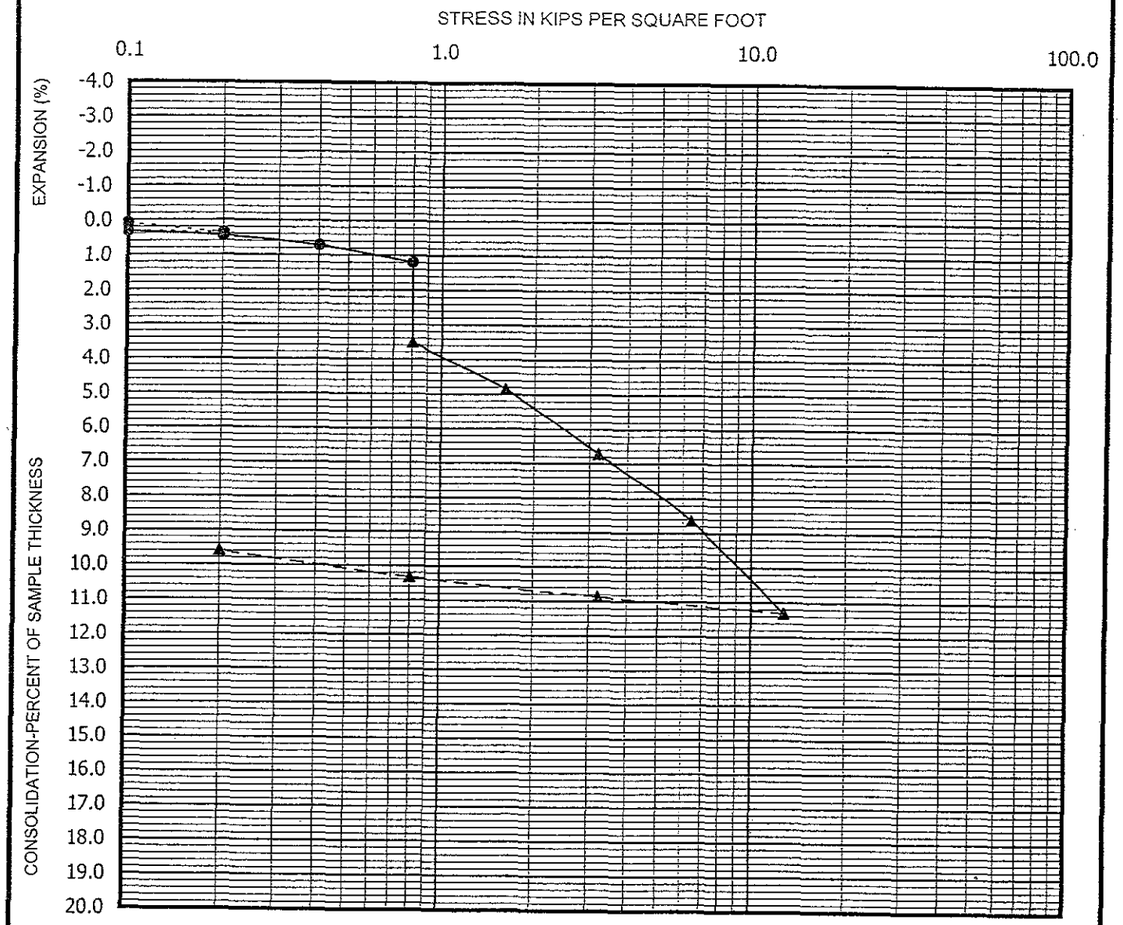
Ninyo & Moore

CONSOLIDATION CH-2 2.5-4.xls

CONSOLIDATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-56



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Boring No. CH-6
Depth (ft.) 7.5-9
Soil Type ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

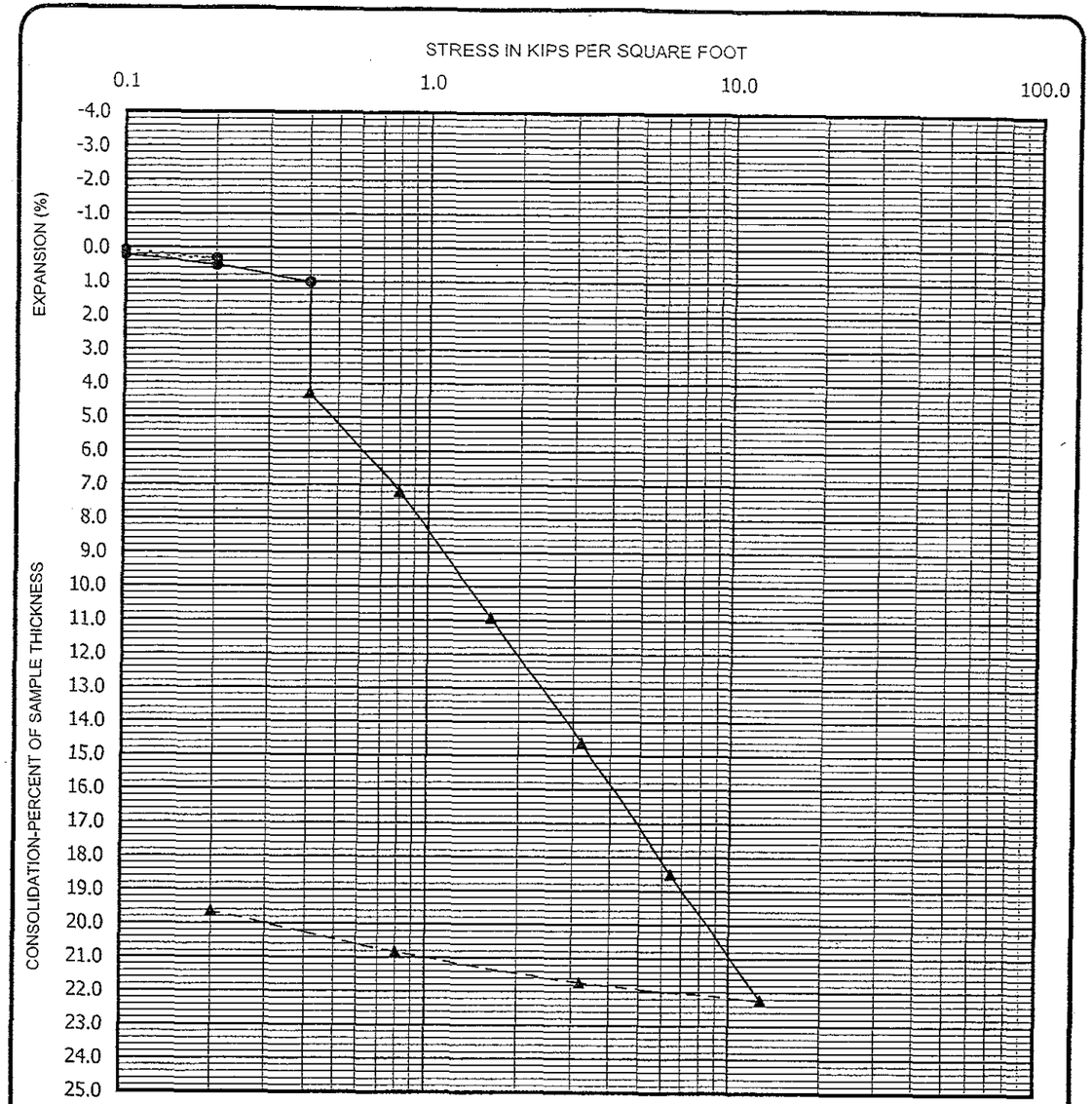
Ninyo & Moore

CONSOLIDATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-57

CONSOLIDATION CH-6 7.5-9.xls



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Boring No. CH-13
Depth (ft.) 5-6.5
Soil Type CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

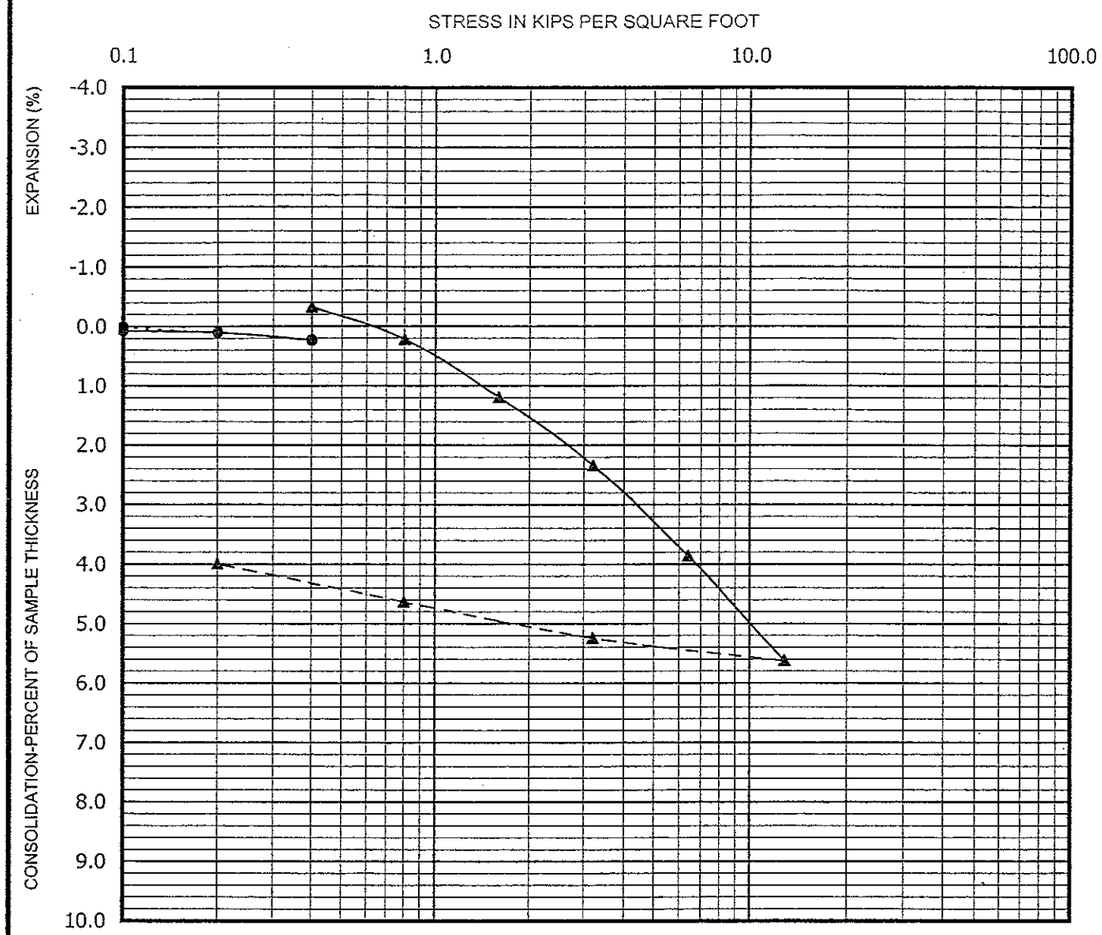
Ninyo & Moore

CONSOLIDATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-58

CONSOLIDATION CH-13 5-6.5.xls



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Boring No. CH-16
Depth (ft.) 2.5-4
Soil Type CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

Ninyo & Moore

CONSOLIDATION TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-59

CONSOLIDATION CH-16 2.5-4.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
CH-11	0-2	11.1	111.1	17.5	0.0158	16	Very Low
CH-21	12-15	11.0	108.8	18.7	0.0058	6	Very Low
CH-23	0-2	10.2	106.9	18.9	0.0171	17	Very Low
CH-25	12-15	7.8	113.0	14.4	0.0000	0	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH UBC STANDARD 18-2

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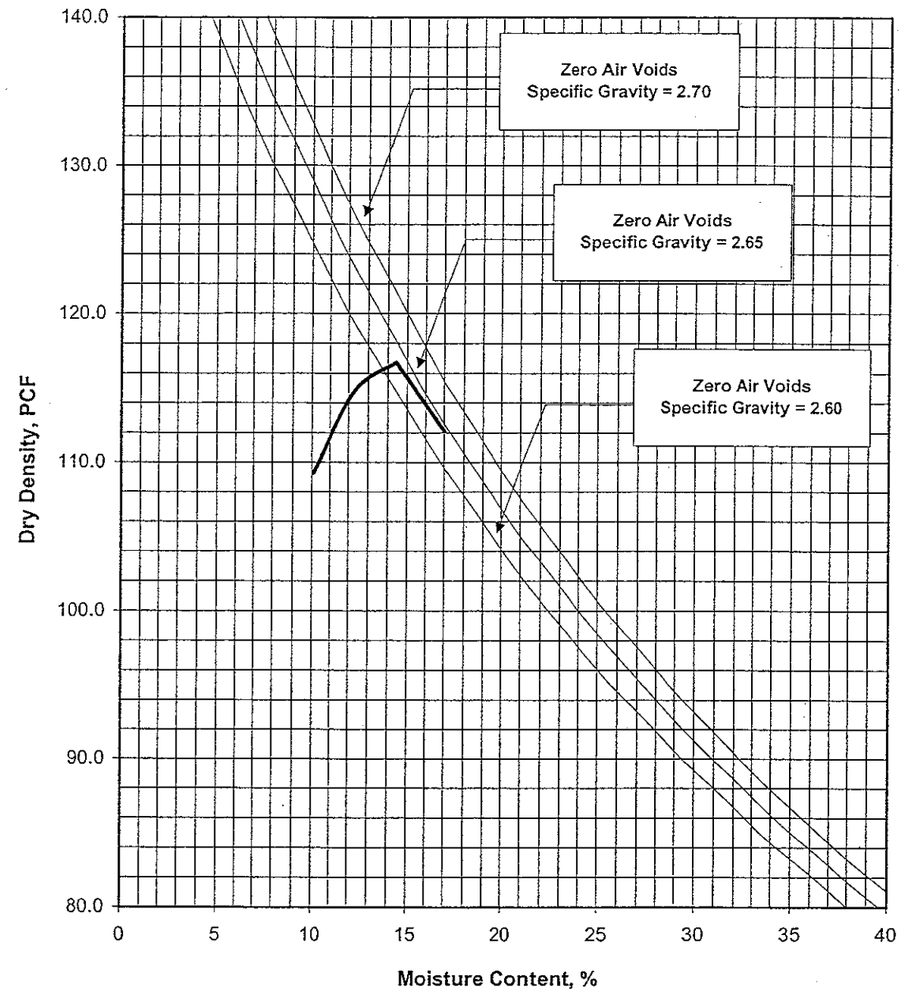
EXPANSION INDEX TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-60

EXPANSION #259.xls



SAMPLE LOCATION	DEPTH (FT)	SOIL DESCRIPTION	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
CH-11	0-2	Sandy Clay	116.7	14.4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 698

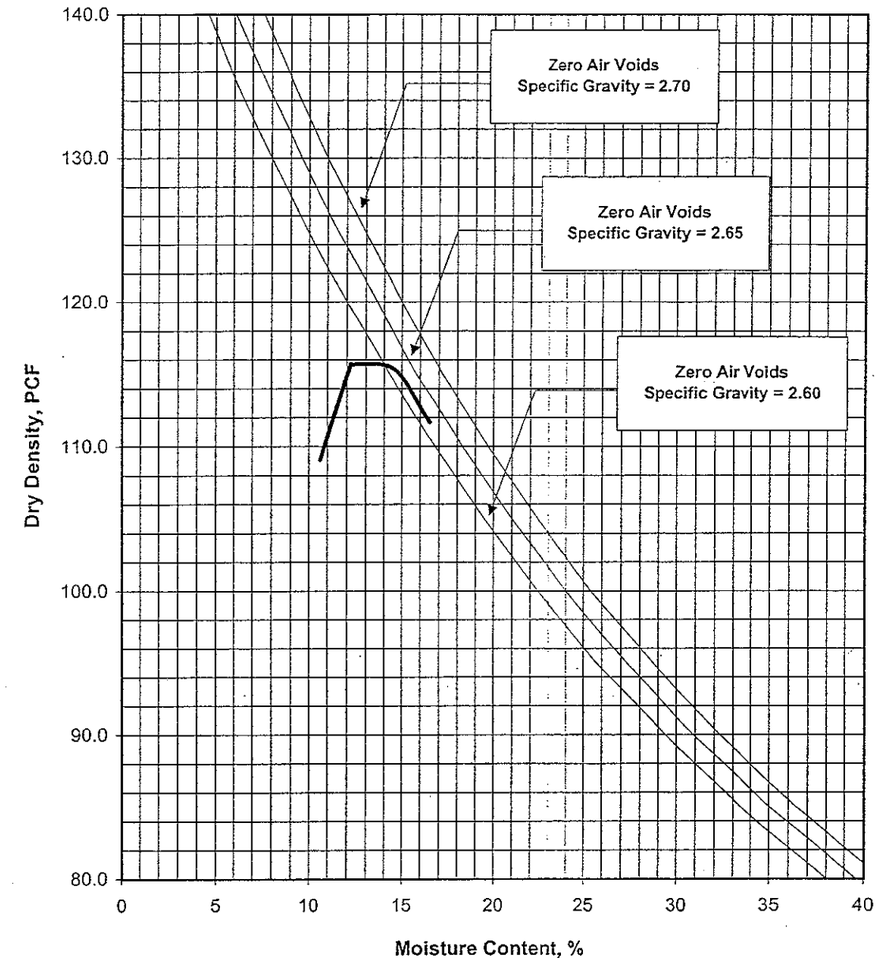
Ninyo & Moore

MAXDENSITY CH-11@0 - 2 update.xls

MAXIMUM DENSITY TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-61



SAMPLE LOCATION	DEPTH (FT)	SOIL DESCRIPTION	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
CH-21	12-15	Sandy Clay - Silty Sand	115.7	12.2

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 698

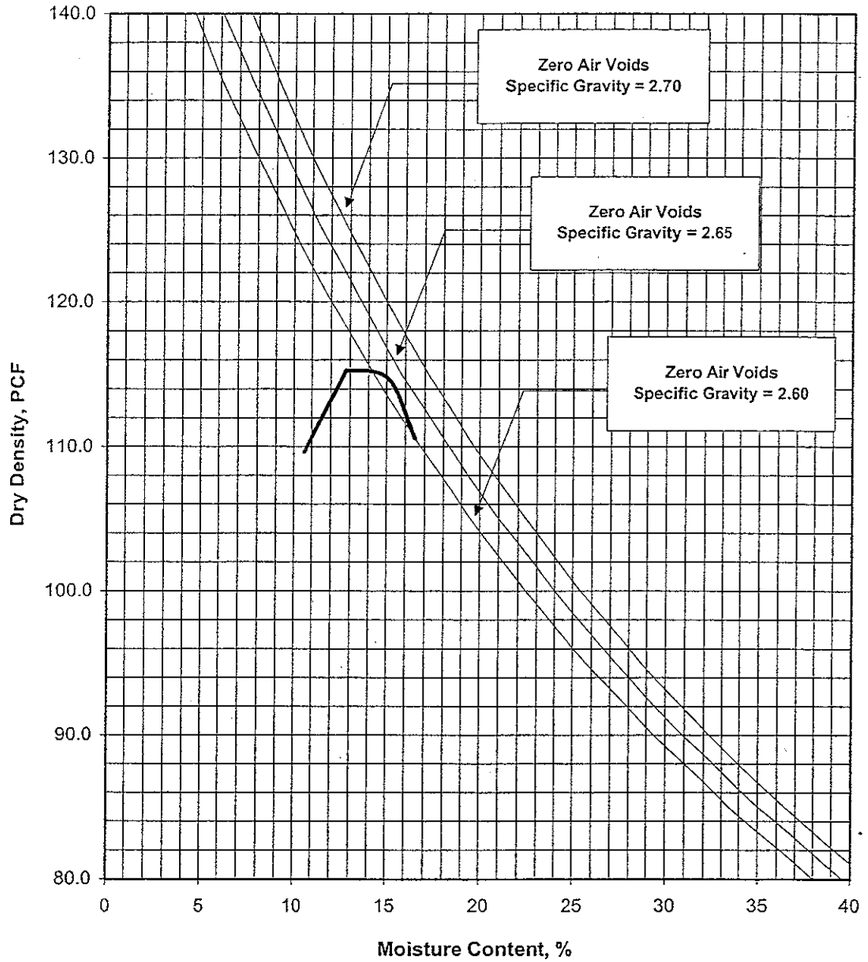
Ninyo & Moore

MAXDENSITY CH-21@12.xls

MAXIMUM DENSITY TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-62



SAMPLE LOCATION	DEPTH (FT)	SOIL DESCRIPTION	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
CH-23	0-2	Silty Clay	115.3	12.8

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 698

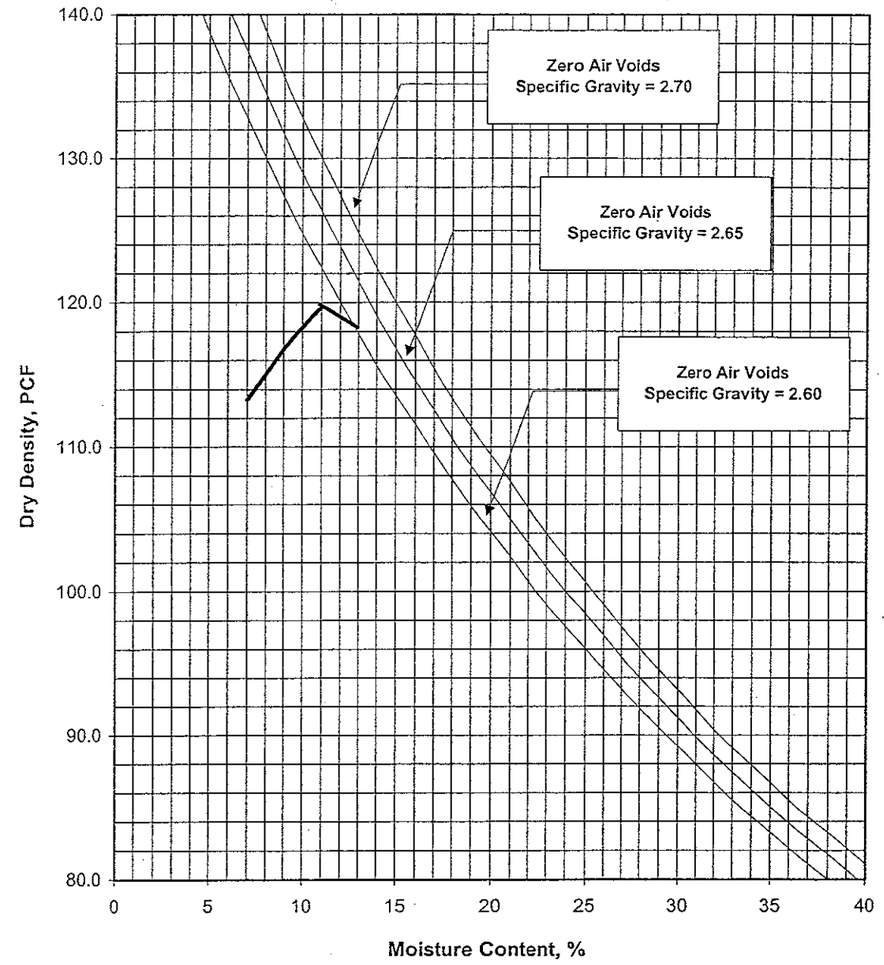
Ninyo & Moore

MAXIMUM DENSITY TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-63

MAXDENSITY CH-23@0.xls



SAMPLE LOCATION	DEPTH (FT)	SOIL DESCRIPTION	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
CH-25	12-15	Silty Clay	119.7	11.0

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 698

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MAXIMUM DENSITY TEST RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-64

MAXDENSITY CH-25@12.xls

CORROSIVITY TEST RESULTS

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH *	RESISTIVITY * (ohm-cm)	WATER-SOLUBLE SULFATE CONTENT IN SOIL ** (%)	CHLORIDE CONTENT *** (ppm)
CH-11	0-2	7.6	508	0.0025	160
CH-21	12-15	8.4	1,320	0.0004	10

* 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

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CORROSIVITY TEST RESULTS
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-65

CH only CORROSIVITY.xls

PERMEABILITY TEST RESULTS

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	DRY DENSITY (PCF)	RANGE IN HEAD (cm)	AVERAGE PERMEABILITY (cm/sec)
CH-1	20.0-21.5	9.7	20.6	101.1	30.0-40.0	8.5×10^{-5}
CH-4	17.5-19.0	2.3	12.5	109.7	2.0-12.0	1.4×10^{-3}
CH-7	15.0-16.5	0.1	10.4	111.2	2.6 - 12.4	1.3×10^{-2}
CH-8	10.0-11.5	2.8	18.2	103.3	2.0 - 12.4	9.8×10^{-4}
CH-8	17.5-19.0	2.8	10.9	104.4	2.0 - 2.7	2.8×10^{-4}
CH-11	15.5-17.0	4.4	17.8	102.6	2.5 - 11.4	2.8×10^{-3}
CH-12	7.5-9.0	3.4	16.5	96.7	2.0 - 12.3	7.8×10^{-4}
CH-14	15.0 -16.5	1.3	16.1	107.7	2.4 - 12.3	1.4×10^{-2}
CH-16	20.0-21.5	3.8	N.M.	107.3	2.4 - 11.9	2.9×10^{-4}

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2434-68

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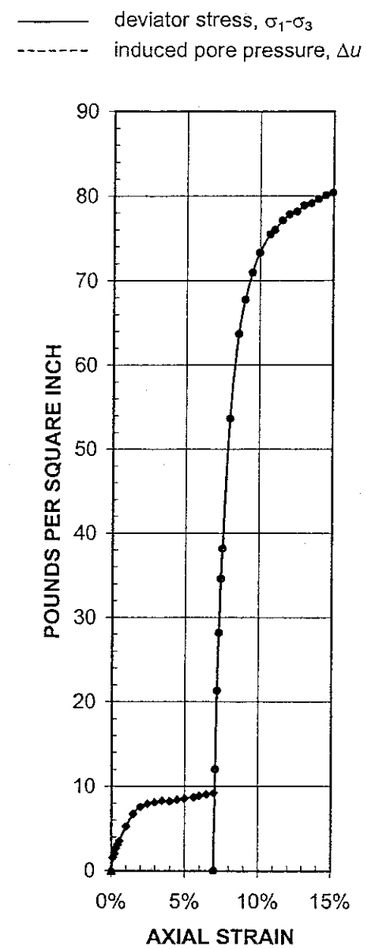
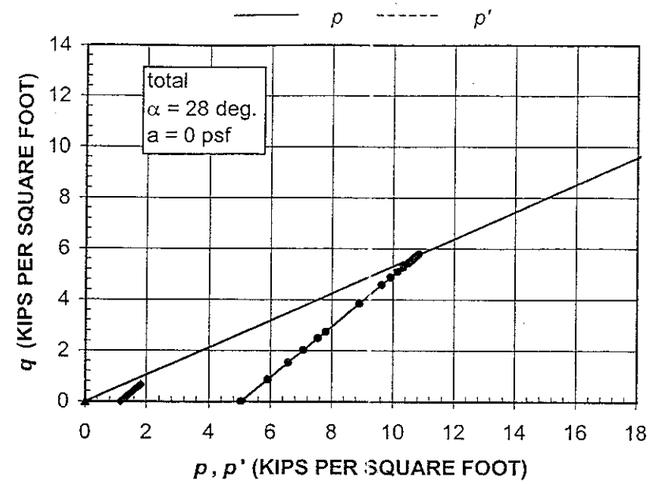
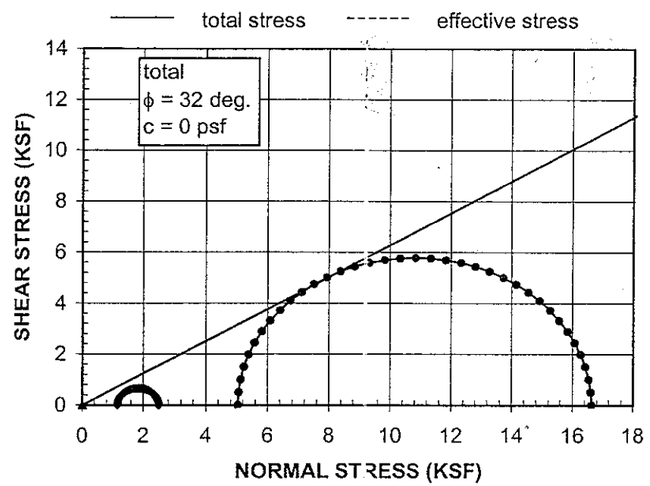
PERMEABILITY TEST RESULTS
EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-66

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)
◆	Silty Sand	SM	CH-1	20.0-21.5	9.7%	101.1	83%	1.15	0.9%
●	Silty Sand	SM	CH-1	20.0-21.5	9.7%	101.1	83%	5.04	1.0%
▲									

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850

Ninyo & Moore

UU TRIAXIAL COMPRESSION RESULTS

EAST MARICOPA FLOODWAY
CHANDLER HEIGHTS DETENTION BASIN
MARICOPA COUNTY, ARIZONA

PROJECT NO.	DATE	FIGURE
600198001	01/02	B-67

UUTRIAXchn.xls

APPENDIX C

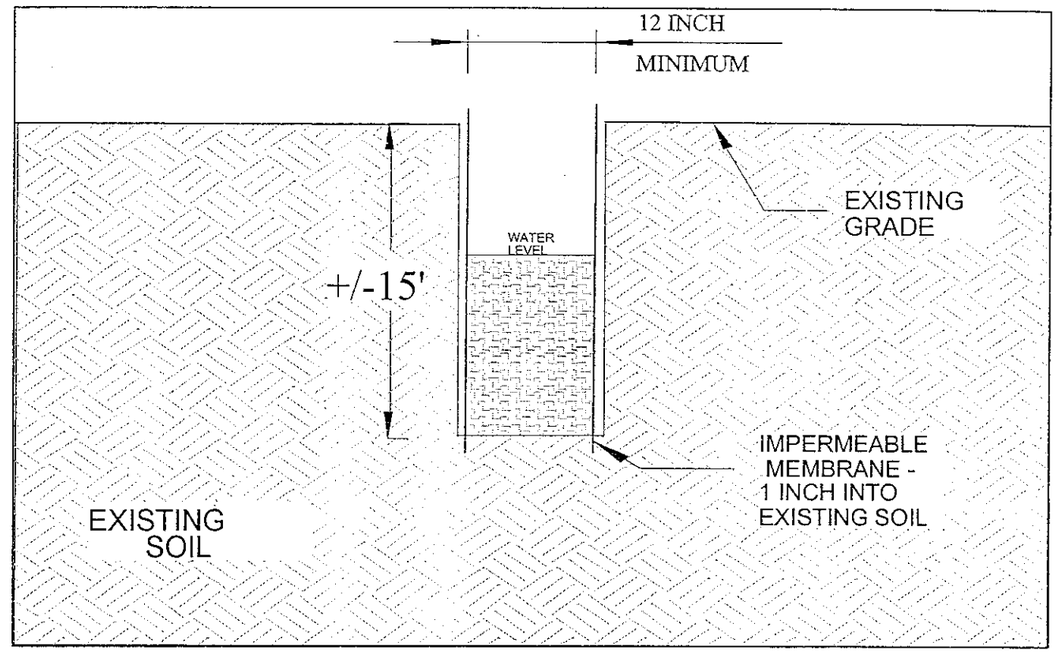
PERCOLATION TEST RESULTS

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.

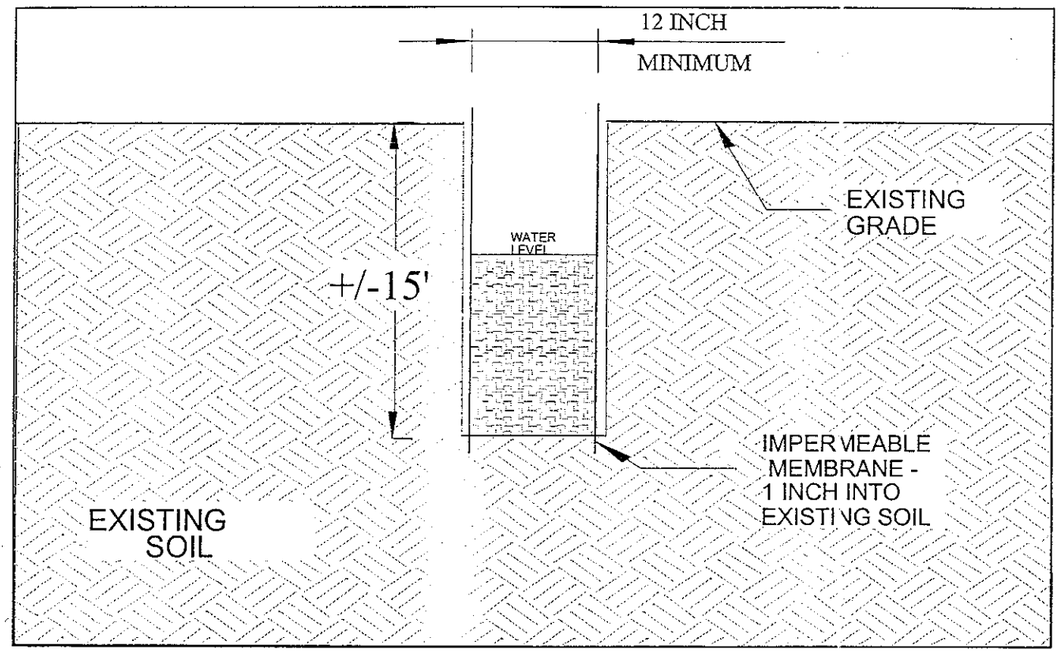
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS	**0.08**	$FT^3/HOUR/FT^2$

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.

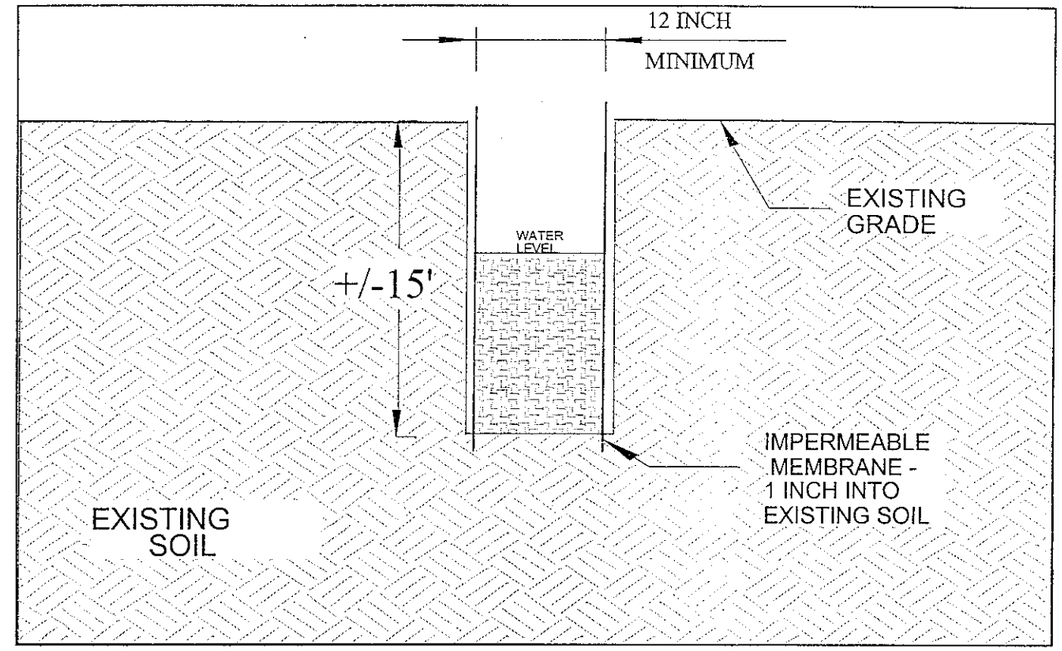
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **2.09** $FT^3/HOUR/FT^2$

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.

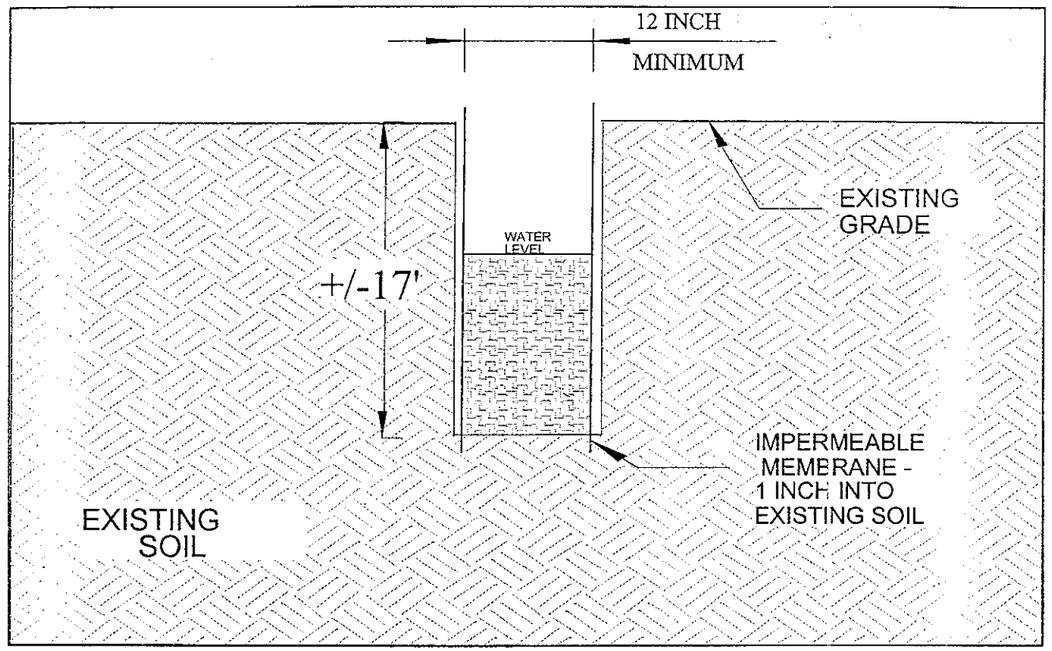
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **0.88** $FT^3/HOUR/FT^2$

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.

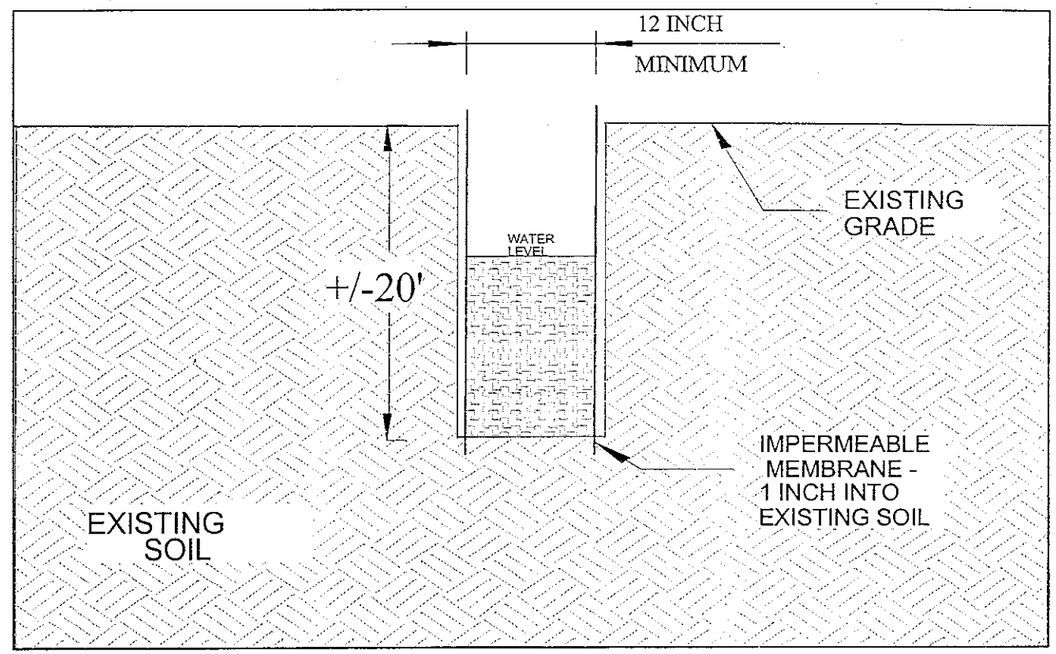
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **1.31** $FT^3/HOUR/FT^2$

Ninyo & Moore

SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Chandler Heights Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-5 (Near CH-21)



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*
14:22	14:54	0:32	0.32	0.88	0.56	1.05
14:54	15:15	0:21	0.88	1.22	0.34	0.97
15:15	16:21	1:06	1.22	2.26	1.04	0.95
16:21	16:44	0:23	2.26	2.58	0.32	0.84
16:44	17:03	0:19	2.58	2.88	0.30	0.95

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

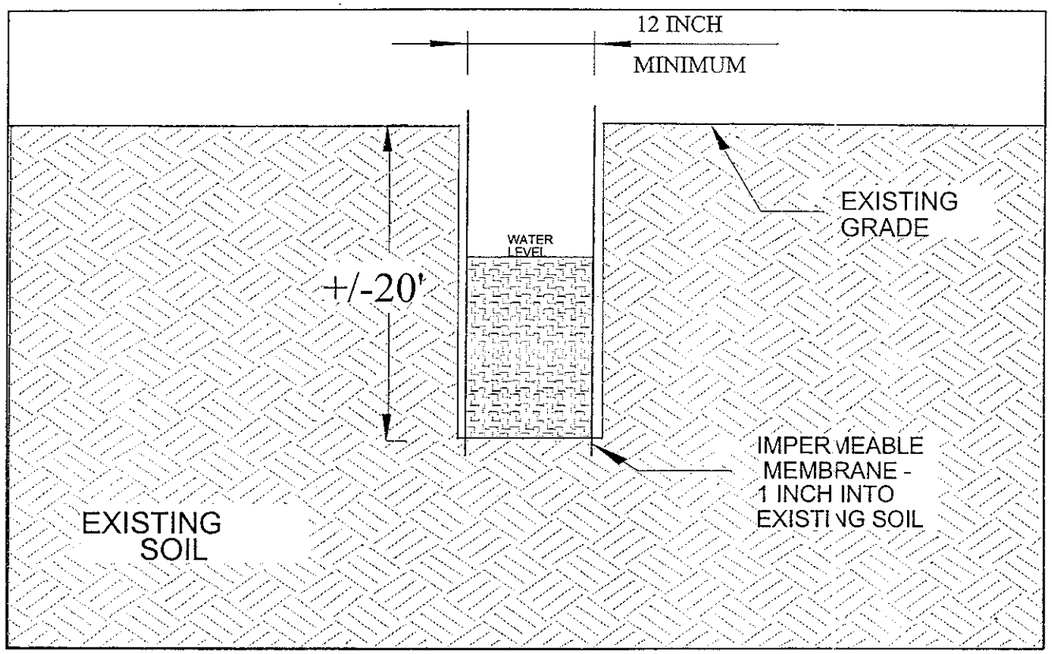
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **0.91** $FT^3/HOUR/FT^2$

Ninyo & Moore

SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Chandler Heights Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-6 (Near CH-22)



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*
16:37	16:54	0:17	4.54	4.61	0.07	0.25
16:54	17:12	0:18	4.61	4.72	0.11	0.37
17:12	17:30	0:18	4.72	4.82	0.10	0.33
17:30	17:49	0:19	4.82	4.93	0.11	0.35
17:49	18:09	0:20	4.93	5.03	0.10	0.30

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

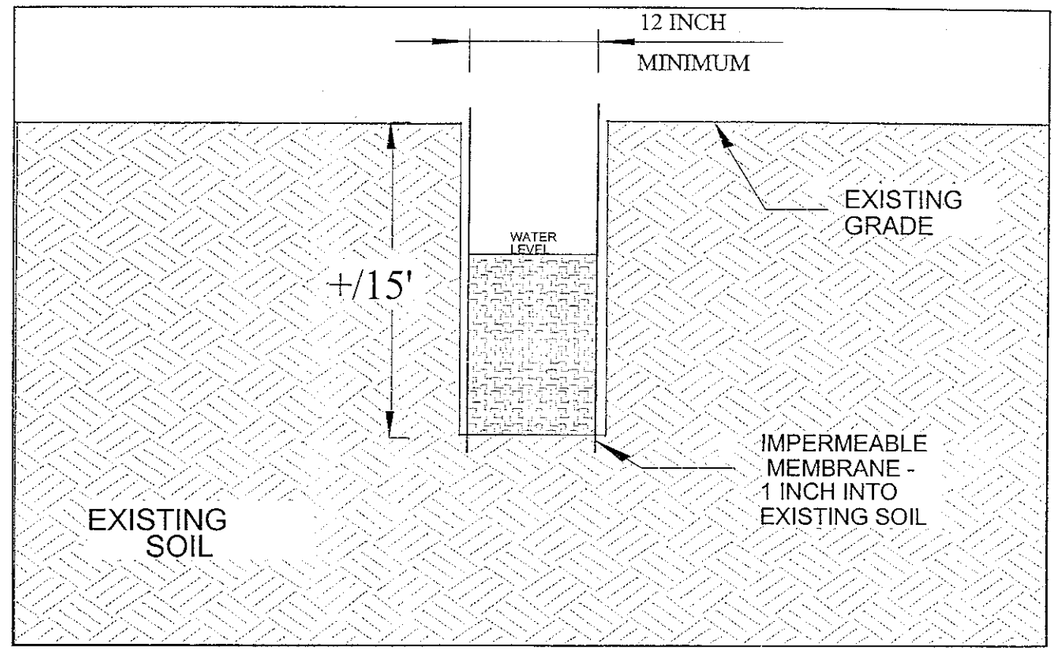
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **0.33** $FT^3/HOUR/FT^2$

Ninyo & Moore

SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Chandler Heights Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-7 (Near CH-23)



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*
17:26	17:44	0:18	4.09	4.12	0.03	0.10
17:44	18:02	0:18	4.12	4.15	0.03	0.10
18:02	18:18	0:16	4.15	4.17	0.02	0.08
18:18	18:35	0:17	4.17	4.21	0.04	0.14
18:35	18:49	0:14	4.21	4.23	0.02	0.09

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

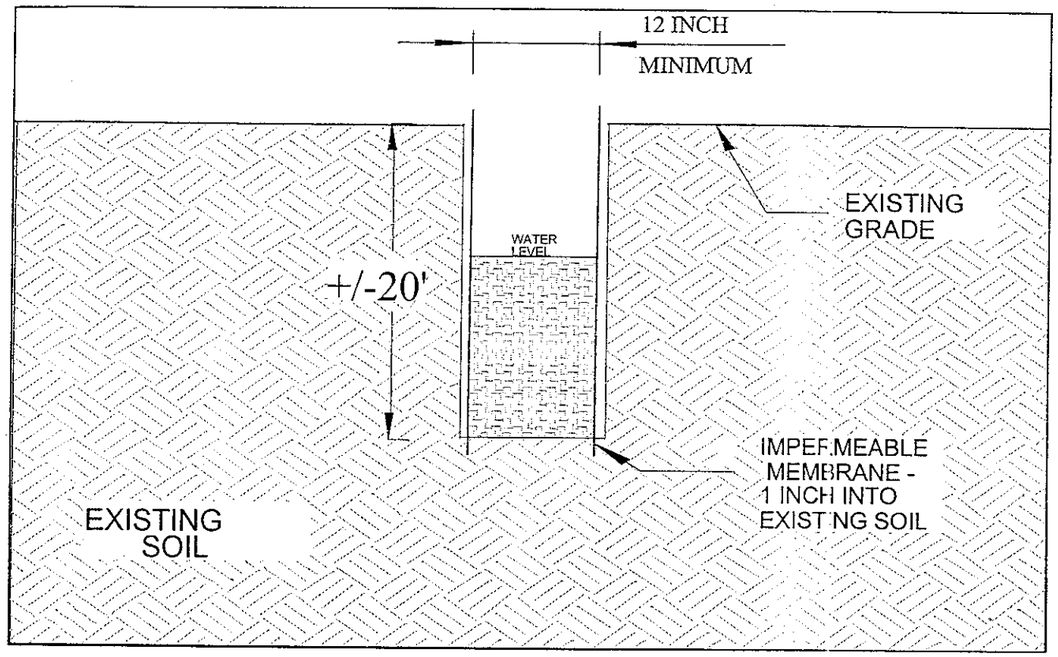
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **0.10** $FT^3/HOUR/FT^2$

Ninyo & Moore

SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Chandler Heights Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-8 (Near CH-24)



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*
14:39	16:28	1:49	0.34	0.89	0.55	0.30
16:28	16:48	0:20	0.89	0.98	0.09	0.27
16:48	17:07	0:19	0.98	1.09	0.11	0.35
17:07	17:20	0:13	1.09	1.15	0.06	0.28
17:20	17:37	0:17	1.15	1.23	0.08	0.28

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

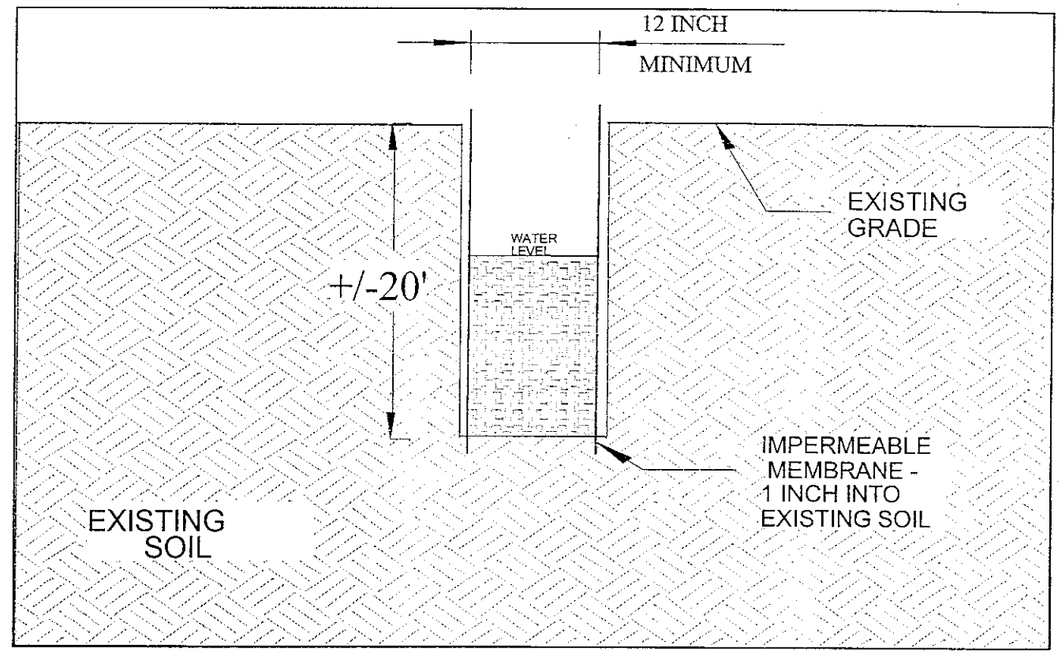
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **0.30** $FT^3/HOUR/FT^2$

Ninyo & Moore

SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Chandler Heights Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-9 (Near CH-25)



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*
19:04	19:17	0:13	9.67	9.70	0.03	0.14
19:17	19:30	0:13	9.70	9.73	0.03	0.14
19:30	19:40	0:10	9.73	9.76	0.03	0.18
19:40	19:53	0:13	9.76	9.78	0.02	0.09
19:53	20:07	0:14	9.78	9.81	0.03	0.13

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

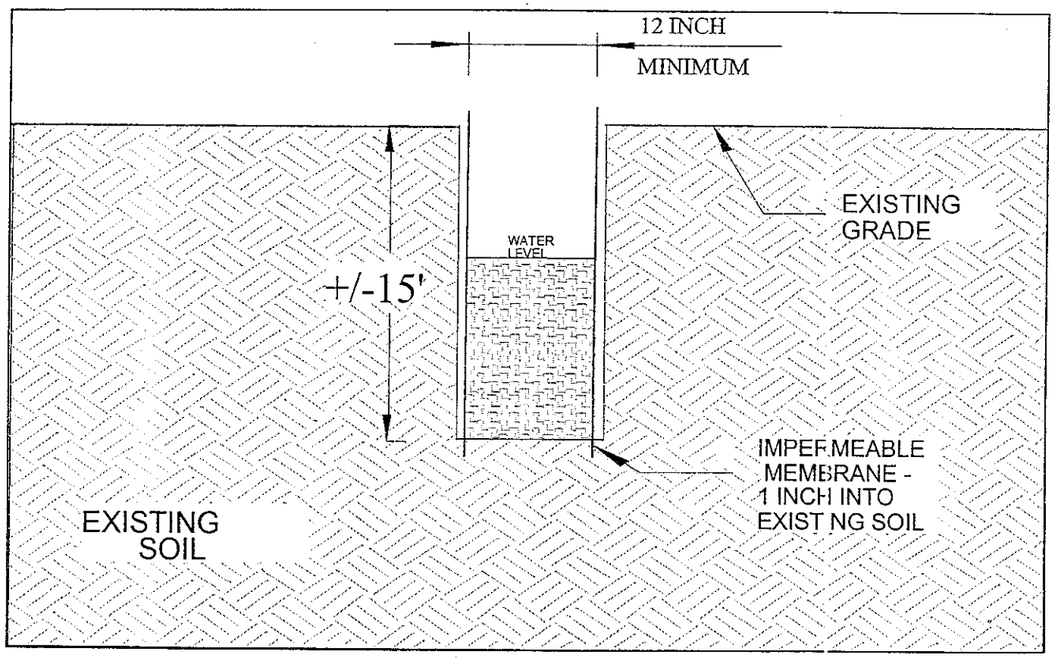
AVERAGE PERCOLATION RATE FOR LAST THREE READINGS **0.13** $FT^3/HOUR/FT^2$

Ninyo & Moore

SUMMARY OF PERCOLATION TEST RESULTS

PROJECT: Chandler Heights Detention Basin PROJECT NO.: 600198001

TECHNICIAN: MDE DATE: 07/19/01 LOCATION: PT-10 (Near CH-26)



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*
19:10	19:27	0:17	4.13	4.14	0.01	0.04
19:27	19:36	0:09	4.14	4.16	0.02	0.13
19:36	19:47	0:11	4.16	4.18	0.02	0.11
19:47	20:00	0:13	4.18	4.20	0.02	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	**0.07**	$FT^3/HOUR/FT^2$

APPENDIX D

AGRONOMIC TESTS RESULTS

FRUIT GROWERS LABORATORY, INC.

ANALYTICAL CHEMISTS

August 21, 2001

Lab #: SP 107342-03

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

Recommendations for Chandler Heights 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.
C. Select only those plants that have a slight or greater tolerance to Salinity for planting at this site. A review of the plants growing in the immediate area of the site to be landscaped will provide some additional guidelines as to the proper plant selection.

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)	0.00 lbs.
b. Phosphorus (P_2O_5)	4.50 lbs.
c. Potassium (K_2O)	2.80 lbs.
d. Magnesium (Mg)	0.00 lbs.
e. Zinc (Zn)	1.30 lbs.
f. Manganese (Mn)	0.00 lbs.
g. Iron (Fe)	0.80 lbs.
h. Copper (Cu)	.025 lbs.
i. Boron (B)	.000 lbs.

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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

It is recommended that this soil be thoroughly leached to lower the Chloride, and the total Soil Salinity prior to preplant planting. This leaching operation should be made after the application of any recommended soil amendments, but prior to applying any of the recommended preplant fertilizers. The leaching operation should consist of three applications of irrigation water with enough water being applied at each irrigation to thoroughly wet this soil to a depth of twenty-four inches with the water being applied at a rate slow enough to prevent any runoff. A two to three day waiting period between applications of water should occur to allow for internal soil drainage.

Chloride, and the total Soil Salinity(ECe) levels should be rechecked after the above leaching operation is completed to determine the degree of improvement. These new levels will allow for the selection of plants having the appropriate salt tolerances.

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

Nitrogen	3/4 lb.
Phosphorus	1/3 lb.
Potassium	1/3 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

Lab ID : SP 107342-03
Customer ID: 2-18569

Sampled On : July 13, 2001
Sampled By : Ninyo & Moore
Received On: August 15, 2001
Depth : 12-15'
Meth. Irrg. : S.S. Sprinklers

Description : CH-13
Project : Chandler Heights Basin

LANDSCAPE SOIL ANALYSIS

Test Description	Result		Optimum Range
Primary Nutrients			
Nitrate-Nitrogen	68.7	PPM	10 - 70
Phosphorus	ND	PPM	12 - 60
Potassium (Exch)	130	PPM	81 - 500
Potassium (Sol)	0.31	meq/L	0.25 - 1.0
Secondary Nutrients			
Calcium (Exch)	3900	PPM	---
Calcium (Sol)	18.2	meq/L	2.0 - 50
Magnesium (Exch)	240	PPM	---
Magnesium (Sol)	4.2	meq/L	1.5 - 60
Sodium (Exch)	130	PPM	---
Sodium (Sol)	10.9	meq/L	See SAR
Sulfate	5.5	meq/L	0.6 - 20
Micro Nutrients			
Zinc	0.2	PPM	0.7 - 50
Manganese	3.2	PPM	1.4 - 50
Iron	4.4	PPM	8.0 - 100
Copper	0.6	PPM	0.2 - 15
Boron	0.77	PPM	0.3 - 2.1
Chloride	7.91	meq/L	0.1 - 4.0
CEC	22.3	meq/100g	Variable
% Base Saturation			
CEC - Calcium	87.0	%	60 - 80
CEC - Magnesium	9.0	%	10 - 20
CEC - Potassium	1.5	%	2 - 5
CEC - Sodium	2.5	%	0 - 5
CEC - Hydrogen	0.0	%	0 - 3
pH	7.7	---	5.8 - 8.2

Graphical Results Presentation: Very Low | Moderately Low | Optimum | Moderately High | Very High

pH scale: Strongly Acidic | Moderately Acidic | Near Neutral | Moderately Alkaline | Strongly Alkaline

Good Problem

Table continued next page...

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FAX: 209/942-0423

Field Office
Visalia, CA
TEL: 559/734-9473
FAX: 559/734-8435
Mobile: 559/737-2399

August 21, 2001

Ninyo & Moore

Lab ID : SP 107342-03
Customer ID: 2-18569
Description : CH-13

LANDSCAPE SOIL ANALYSIS

Test Description	Result		Optimum Range
Others			
Soil Salinity	2.94	mmhos/cm	0.5 - 2.0
SAR	3.3		0.1 - 6
Limestone	1.1	%	0 - 0.1
Lime Requirement	0.0	Tons/AF	---
Moisture	3.1	%	1/2 Satn. %
Saturation	29.2	%	20 - 60

Graphical Results Presentation: Satisfactory | Possible Problem | Moderate Problem | Increasing Problem

0 | 1 | 2 | 3 | 4 | 5 | 6

Very Low | Moderately Low | Optimum | Moderately High | Very High

Loamy Sand | Sandy Loam | Loam | Silt Loam | Clay Loam | Clay | Organic

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 FISSURE EVALUATION

EAST MARICOPA FLOODWAY CHANDLER HEIGHTS 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 27, 2001.

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Appendix	Title
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GROUND SUBSIDENCE & EARTH FISSURING EVALUATION

EAST MARICOPA FLOODWAY CHANDLER HEIGHTS 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 of the proposed Chandler Heights Basin that is part of the East Maricopa Floodway Project. The Chandler Heights 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 basin.

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 Chandler Heights Basin.

- Recent aerial photography (provided by Kirkham Michael) 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 Chandler Heights Basin site, the following opinions are provided.

- The proposed basin is within a known ground subsidence area. The data indicates ground subsidence and related earth fissuring has been ongoing for more than 50 years.
- Groundwater level declines on the order of 150 to 300 feet (Schumann, 1986) within the study area triggered the primary subsidence. Residual ground subsidence, which is continuing ground subsidence without measured water table decline, 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, since the 1970s, groundwater level measurements show rising water table conditions have continued probably due to increased recharge inflow to the aquifer and a reduction in groundwater use. The rise of the groundwater table in the Chandler Heights Basin area through 1999 ranged from about 80 feet to almost 230 feet.
- 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 Chandler Heights basin area. Likewise, no earth fissures were observed outside the basin perimeter that project toward the basin.

  The closest known earth fissures to the basin are about 1¾-miles southeast. None of these fissures appear to trend toward the project area.

2.2 The amount of total subsidence to date that has occurred in the Chandler Heights 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 east of the Chandler Heights 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 Chandler Heights Basin area. Therefore, in our opinion, it is conceivable that both areas could have experienced similar ground subsidence during the same period.

A rough comparison of elevation data depicted on the 1956 Chandler Heights topographic quadrangle with recent survey data obtained by Kirkham Michael (2001) suggest that as much as 6.6 feet of ground subsidence may have occurred since 1955 in the Chandler Heights Basin area. The annual rate of subsidence ranged from about 0.10 feet per year to 0.14 feet per year.

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, because the Chandler Heights Basin is near the basin margin, it is possible that 2 to 4 feet of differential ground subsidence could occur during the next 20 to 30 years.

2.4 The Chandler Heights Basin is located near the southern boundary of the Chandler Basin. This area is also represents the boundary of the regional bowl of depression caused by ground subsidence. In this area of the subsidence bowl, some differential ground subsidence is expected to cause not only a vertical lowering of the ground surface but also a land surface tilt toward the north from the margins of the subsiding basin. Therefore, differential ground subsidence across the Chandler Heights Basin site could adversely impact Basin design.

The amount of subsidence should be expected to increase northward, away from the basin perimeter because the alluvial filled basin is deeper beyond the pediment edge. Ongoing residual subsidence may cause differential lowering of the ground elevation at this site. The magnitude of this differential subsidence will likely be greater at the north end of the basin

due to the basin fill sediment as compared to the south half of the basin, which is closer the bedrock high at the shallow end.

Ongoing ground subsidence within the Chandler Basin could result in a change of slope gradient and loss of embankment dam freeboard. Although the residual rate of ground subsidence in the project area is expected to be less that the previously documented active subsidence rate of about 0.12 feet per year, we recommend either a ground subsidence monitoring program be implemented to determine if a change in land slope gradient and loss of freeboard occurs during the life of the facility or the design freeboard for the basin embankment structures should be increased by a factor of 2.

If the recommended ground subsidence monitoring program option is selected in lieu of design modifications, 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 Chandler Heights 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) (Appendix A).

If the results of the monitoring program indicate a change in land slope gradient and loss of freeboard is occurring due to ground subsidence, safety modifications to retrofit the embankments and inlet/outlet structures could be designed and implemented. The modification could include raising the dam crest elevation and resizing inlet/outlet structures. The subsidence monitoring program should be tied into the Arizona Department of Water Resources (ADWR) or the Maricopa County networks of benchmarks used to monitor ground subsidence within the East Salt River Valley. Water level data should be obtained from wells in the project vicinity and the data could be integrated into the ADWR index well data base.

2.5 The proposed Chandler Heights basin is within an area of known earth fissures. There are several previously mapped earth fissures (Jennings, 1977; Schumann, 1974) located about 1¾ miles south of the proposed basin in the vicinity of Round Top (Figure 4).

Residual subsidence is expected to continue in the basin area which will result in the application of additional tensile stress to the basin sediments. This stress could cause renewed activity on existing dormant fissures and possibly the formation of new earth fissures.

3.0 GEOGRAPHIC SETTING

3.1 Location

The proposed Chandler Heights Basin is located in the eastern portions of Maricopa County (Figure 2). The proposed basin area is south of the town of Gilbert, extending across portions of Section 10, 15, and 22 in located in Township 2 South, Range 6 East (Figure 2). The Chandler Heights basin is accessible via Higley Road at the intersection of Queen Creek Road.

3.2 Physical Features

Regionally, the project area is situated within and near the south-central 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 from the site area flows to the south and southwest via Queen Creek Floodway, which in turn flows southwest to the Gila River. The dominant topographic feature in the vicinity is the Santan Mountains, which is south of the site and rises to an elevation of over 3,000 feet. Goldmine Mountain, located southeast of the proposed development rises to an elevation of 2,374 feet. Round Top, an inselberg, or prominent isolated bedrock hill, is an extension of the Santan Mountains to the north. Round Top is about 2 miles south of the Chandler Heights Basin (Figure 4).

The Roosevelt canal parallels the west boundary of the Chandler Heights basin. A short segment of Higley Road and the Queen Creek floodway channel bound the eastern property boundary. Agricultural lands presently surround the entire basin.

3.3 Climate and Vegetation

The climate of the area is arid to semiarid. Average annual temperature ranges from about 72° to 74° Fahrenheit (F) with summer maximums reaching more than 100° F and winter minimums below freezing (32° 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 6 to 8 inches. Although natural

desert vegetation, dominated by creosotebush, mesquite, paloverde, annual grasses, cacti (Adams, 1974), 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 proposed Chandler Heights Basin area, 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 is locally overlain by Quaternary age (younger than 1.6 mya) alluvium. The basin fill in the valley commonly makes up the principle groundwater aquifer of the region.

Structurally, the region has 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 deep intermontane basins that were subsequently filled with sediment. North of the Santan Mountains, a buried bedrock rock surface extends from the mountain front beneath the basin fill to the buried pediment edge (Figure 3). At the pediment edge, the bedrock surface drops off steeply to about 800 to 1000 feet below a ground surface and the basin fill sediments thicken. The pediment edge probably represents the location of an ancient basin-bounding fault that separates the Santan Mountain up-thrown bedrock block from the Chandler Basin down-thrown graben block. Known earth fissures are present within a zone centered along the buried pediment edge.

4.1.1 Basin Stratigraphy

The study area is situated near the south-central border 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. The alluvial deposits are expected to be a few feet to a few hundreds of feet thick near the

mountains due to the presence of a shallow buried bedrock pediment. Beyond the pediment edge (to the north), the basin fill thickens rapidly to several 1,000's of feet reaching to more than 10,000 feet southeast of Gilbert (ADWR, VII, 1994).

The basin filled strata can be grossly subdivided into three zones: upper sand and gravel unit that ranges in thickness from nil to more than 300 feet: middle silt and clay unit that ranges in thickness from less than 100 feet to over 1,800 feet, and a lower conglomerate unit that ranges in thickness from less than 100 feet to over 9,000 feet (ADWR VII, 1994).

Interpretation of ground surface contours using the U.S. Geological Survey Chandler Heights and Higley 7.5-minute topographic quadrangle maps show the project area is located in the central portion of a large alluvial fan deposit. The apex of the alluvial fan originates from the north side of the Santan Mountains south of the site. That coalesces with alluvial fans emanating from the western flanks of the Mineral Mountain area to the east. Earth fissures are present north of the Santan Mountains, north of Round Top that indicates the probable extension of a buried bedrock high in the area contributing to earth fissure development (Figure 4). If the interpretation of the site subsurface stratigraphy is valid, there seems to be a possibility that earth fissures could develop close to the Chandler Heights basin.

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 with a reasonable history of water level measurements (Figure 5) indicates water levels dropped from about 200 to 325 feet from the late 1930s through the mid 1970's (Figure 6). Water levels have increased from as much as 230 in the last 20 to 30 years. The net water level decline for the project area is unknown, however, it is expected to be about 200 feet in the proposed Chandler Heights 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 sediments 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 re-leveling of first-order survey lines by the Coast and Geodetic Survey (now the National Geodetic Survey (NGS)). Subsequent leveling 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, over-sized 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 1 to 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. Additional subsidence has likely occurred since that time over the entire study area.

The amount of subsidence at the Chandler basin is difficult to estimate since no definitive subsidence data is available after 1981 in this area. However, the map prepared by Schumann (1974) indicates that the subsidence for the Chandler basin area should be about the same for as other areas (near Queen Creek) where more subsidence data are available. Therefore, the estimated total subsidence for the Chandler Heights Basin area is about 5 to 6 feet through 2001.

An indication of the reasonableness of this estimate may be established by comparing recent topographic elevation data obtained by Kirkham Michael (2001) with topographic elevation information provided on the United States Geological Survey (USGS) 7½-minute Chandler Heights Topographic Quadrangle map (USGS, 1956). The comparison of the elevation data assumes the later 2001 elevations were measured at the same locations where the 1956 data were measured. Also, the 2001 elevation data had to be rectified from the North American Vertical Datum (NAVD) 88 to the National Geodetic Vertical Datum (NGVD) 29 so the elevation data could be compared using the same reference datum. The results of the comparison are summarized in Table 1.

Table 1
Comparison of 1955 vs. 2001 Elevation Survey Data
Chandler Heights Basin, East Maricopa Floodway

Benchmark Location	Topo Elevation 1955 (ft)	Topo Elevation 2001 (ft)	Elevation Δ 1955 to 2001 (ft)	Annual Rate (ft/yr)
Santan Rd @ Roosevelt Canal	1317	1312.4	4.6	0.10
Santan Rd @ Higley Rd	1323	1317.2	5.8	0.13
Rittenhouse Rd @ Higley Rd	1322	1315.4	6.6	0.14

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 7). 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 moister along the earth fissure. Other physical features associated with fissures are slump-related escarpments from one inch to a few inches in height, as well as a drainage pattern associated with the fissure that does not conform to the areas local drainage pattern.

Field evidence indicates fissures propagate upward and are exposed after overlying sediments are eroded by surface water runoff from rainfall or irrigation (Péwé, 1982). The surface expressions of

the fissures are exaggerated because the initial hairline crack is attacked by water to create wide (10 to 20 feet) and deep (more than 15 feet) erosional gullies that often have vegetation growing in them. During the early stage of fissure development at the surface, some erosion fissure gullies may reach depths of 40- feet or more (Euge, 2001). The fissures are commonly perpendicular to natural drainage channels. The length of the fissure at the ground surface varies, usually less than one mile but one fissure near Picacho is more than 9 miles long. These features are easily recognizable on aerial photographs and in the field except where the land surface is modified by agricultural activities or urban development.

Other indirect investigation techniques, such as gravity surveys, can be used to interpret subsurface conditions that can influence earth fissure development. Gravity studies can be an important tool in predicting the location of potential earth fissure zones. One common use of gravity studies is to indirectly determine the depth to bedrock for an area. This is useful since isolated buried bedrock highs as well as sudden bedrock drop-offs (faults) or near-vertical stratigraphic changes of soil types over relatively short distances make for favorable conditions for earth fissure development when combined with ground subsidence. A gravity survey south of Casa Grande, Arizona confirmed that earth fissure development in that area occurred directly over areas mapped as having buried bedrock highs.

The estimated the depth to bedrock (Oppenheimer et al, 1980) 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 beneath the Chandler Heights Basin ranges from about 2,400 feet to 6,400 feet below ground surface. The closest exposed bedrock to the Chandler Heights Basin is Round Top, which is about 2 miles south of the project area. The presence of Round Top, off the north flank of the Santan Mountains may be an indication of a buried bedrock high that could extend northwest toward the Chandler Heights Basin. Evidence supporting this interpretation includes the earth fissure north of Round Top that may indicate a northwest extension of the Round Top bedrock high (Figure 4). The Oppenheimer (1980) depth to bedrock map, although interpreted from regional gravity survey data, did not indicate the presence of any major isolated buried bedrock highs in the vicinity of, or directly beneath the Chandler Heights Basin.

5.4.1 Earth Fissures near Chandler Heights Basin

Numerous earth fissures are mapped in areas south of the proposed basin. Approximately 1¾-miles south of the study area are a series of earth fissures located north of the Round Top Hill (Figure 4). These earth fissures were mapped in the 1970's (Jennings, 1977). No evidence of the earth fissures located west of Higley Road could be found on our field reconnaissance. The southwest corner of Higley Road and Santan Boulevard has since been residentially developed, obscuring the surface expression of these earth fissures. Southeast of that intersection, an old, mature, inactive (or dormant)earth fissure was observed. None of these earth fissures appear to directly trend toward the Chandler Heights basin. Comparisons made between a 1977 map showing the earth fissure (Jennings, 1977) and a more recent map of the earth fissures (Harris, 1994) indicates that the earth fissures have not lengthened in 20 years. Maps of these fissures (Jennings, 1977) and (Harris, 1994) do not show any of the earth fissures trending toward the basin.

Aerial photography provided by Kirkham Michael could not discern any earth fissures or earth fissure-like features within or adjacent to the proposed basin. This was also substantiated during the field reconnaissance made by Geological Consultants Inc.

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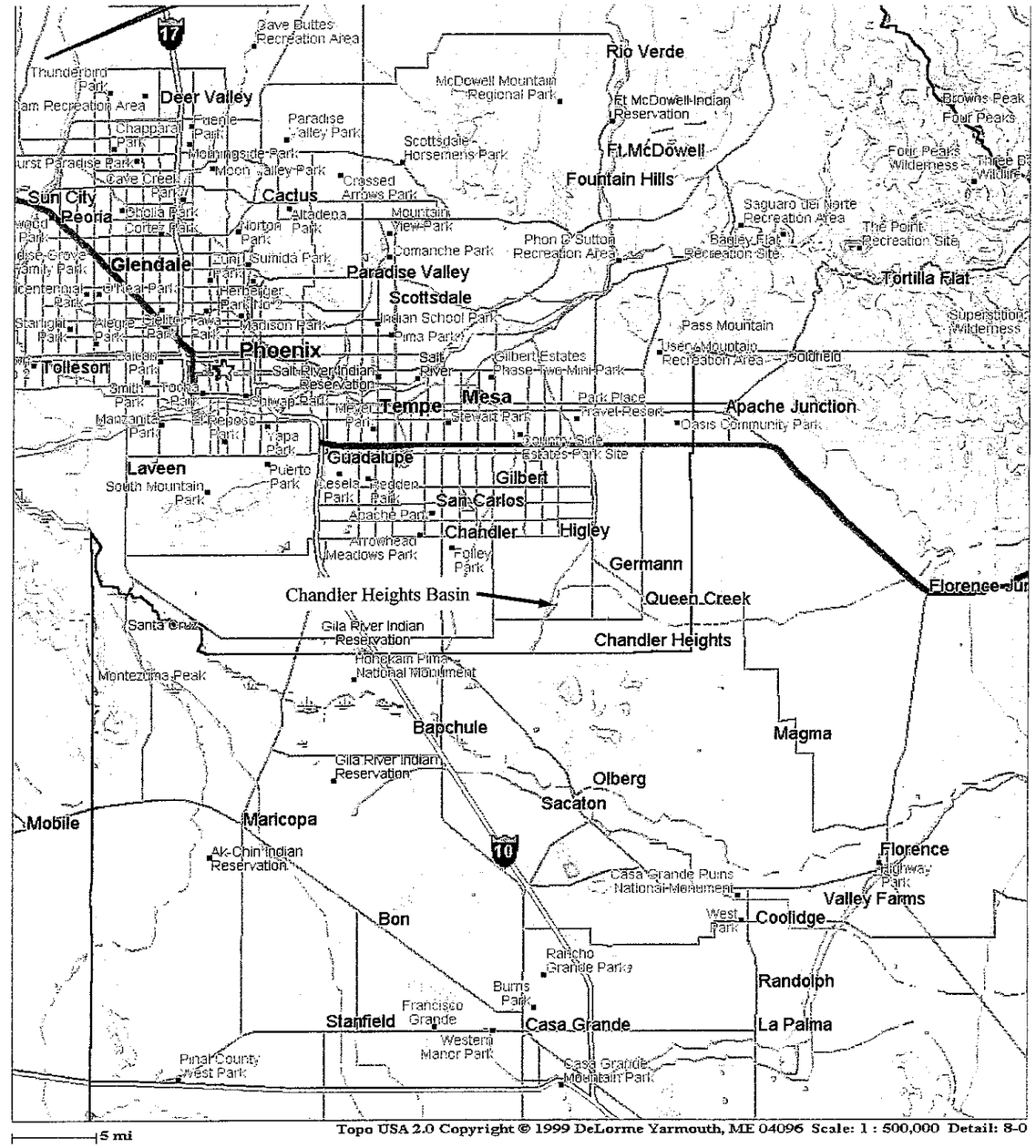
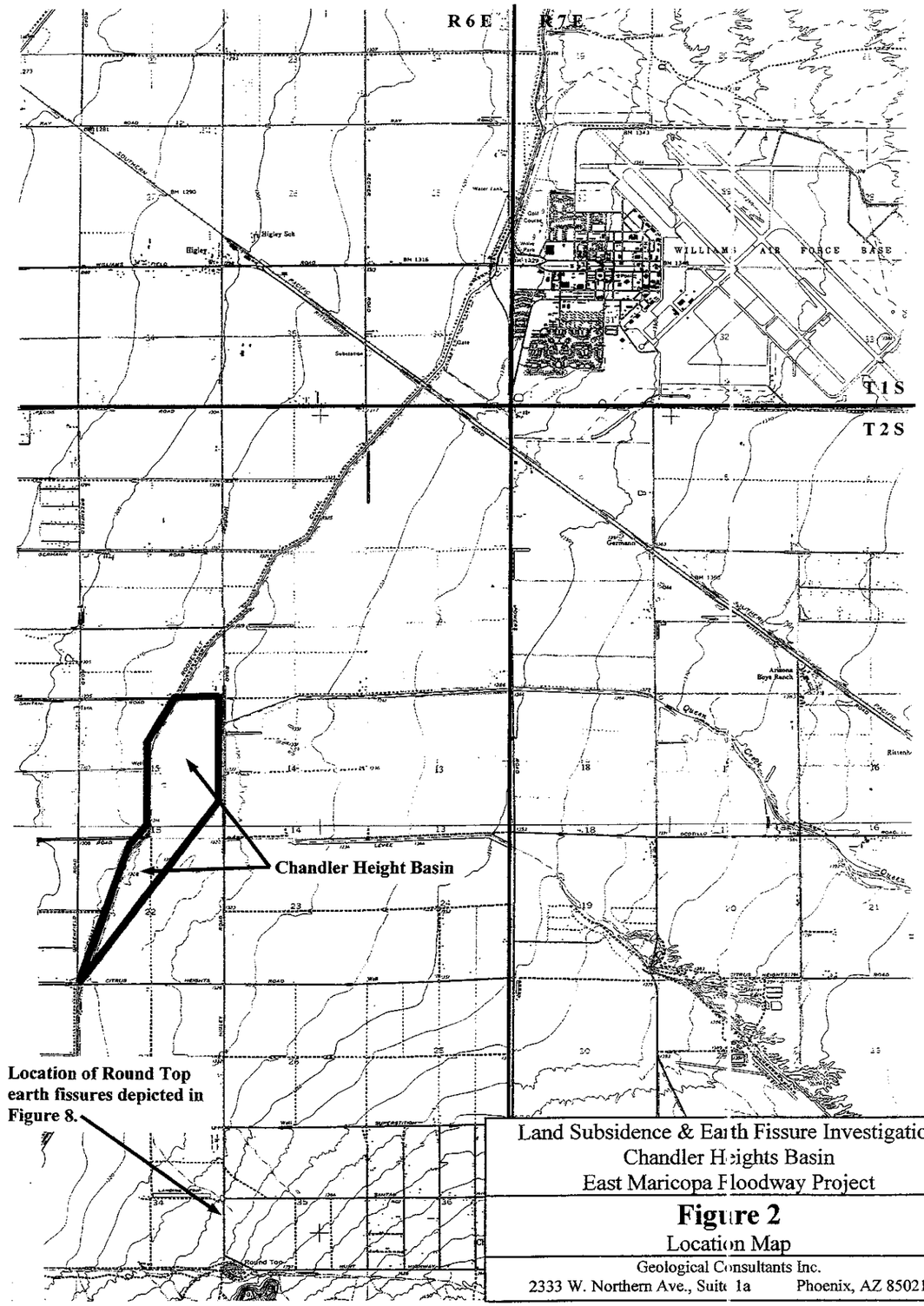


Land Subsidence & Earth Fissure Investigation
Chandler Heights Basin
East Maricopa Floodway Project

Figure 1
Site Vicinity Map

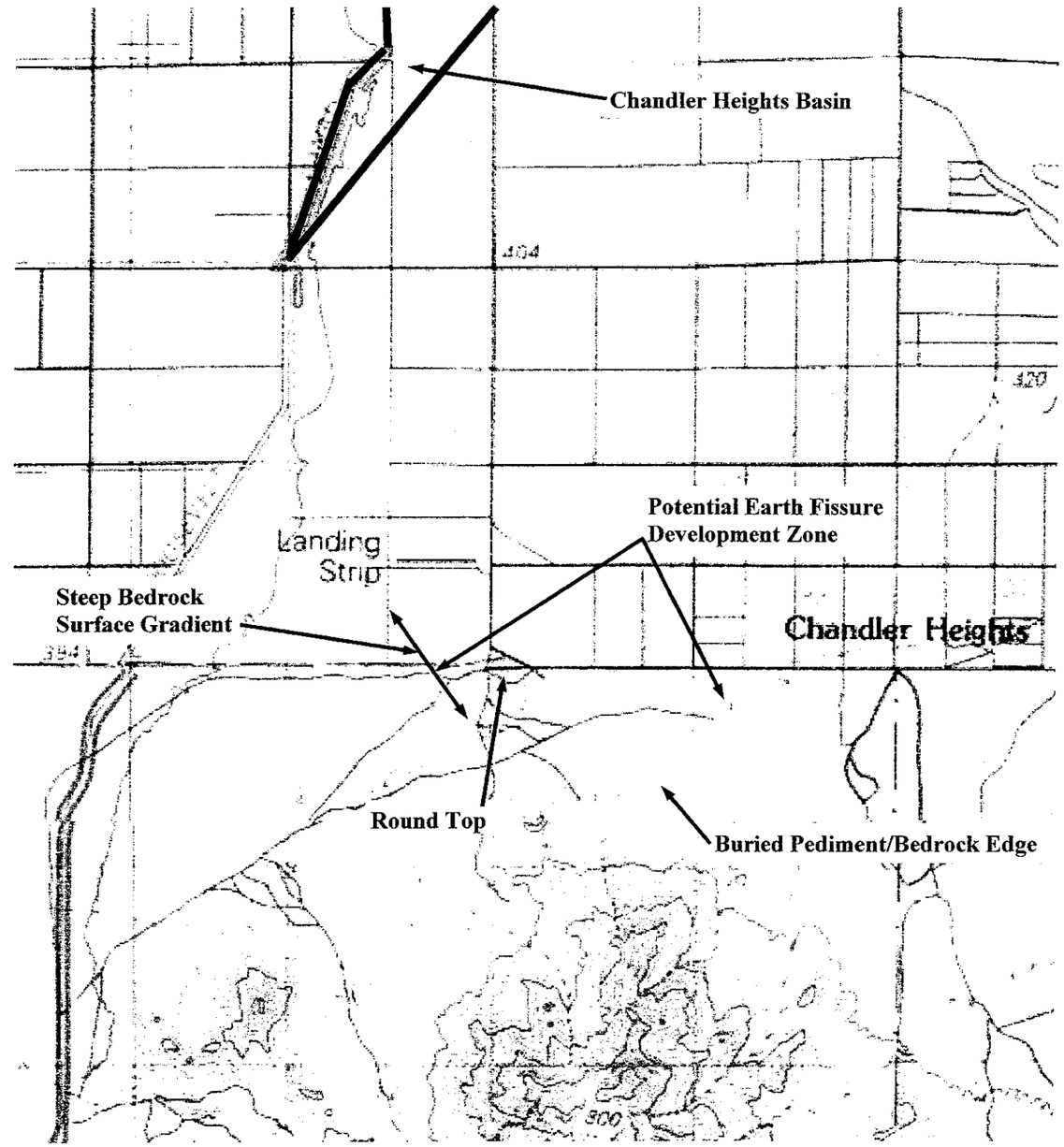
Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021



Land Subsidence & Earth Fissure Investigation
Chandler Heights Basin
East Maricopa Floodway Project

Figure 2
Location Map

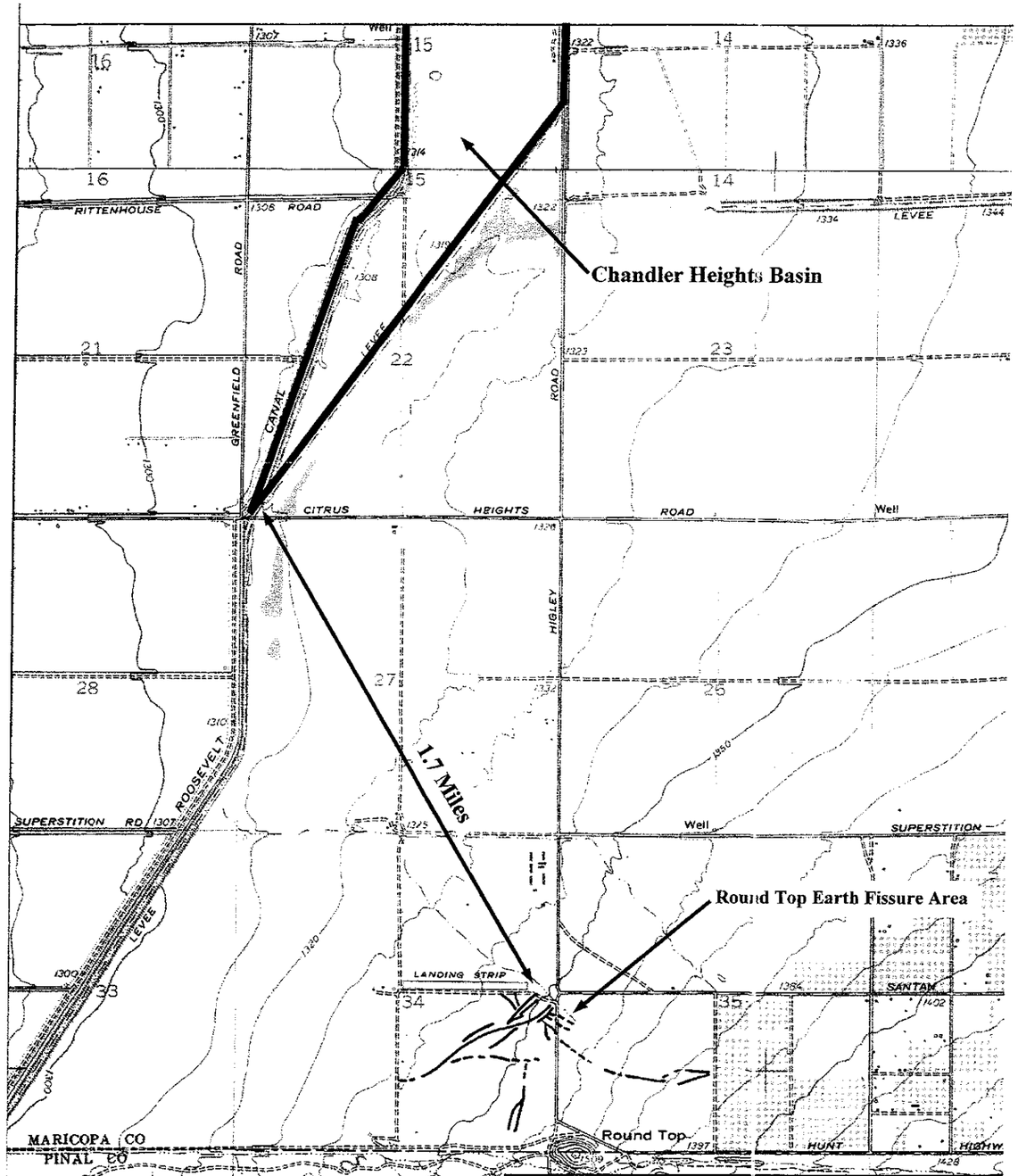
Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021



Land Subsidence & Earth Fissure Investigation
Chandler Heights Basin
East Maricopa Floodway Project

Figure 3
Buried Pediment / Earth Fissure Zone Location Map

Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021

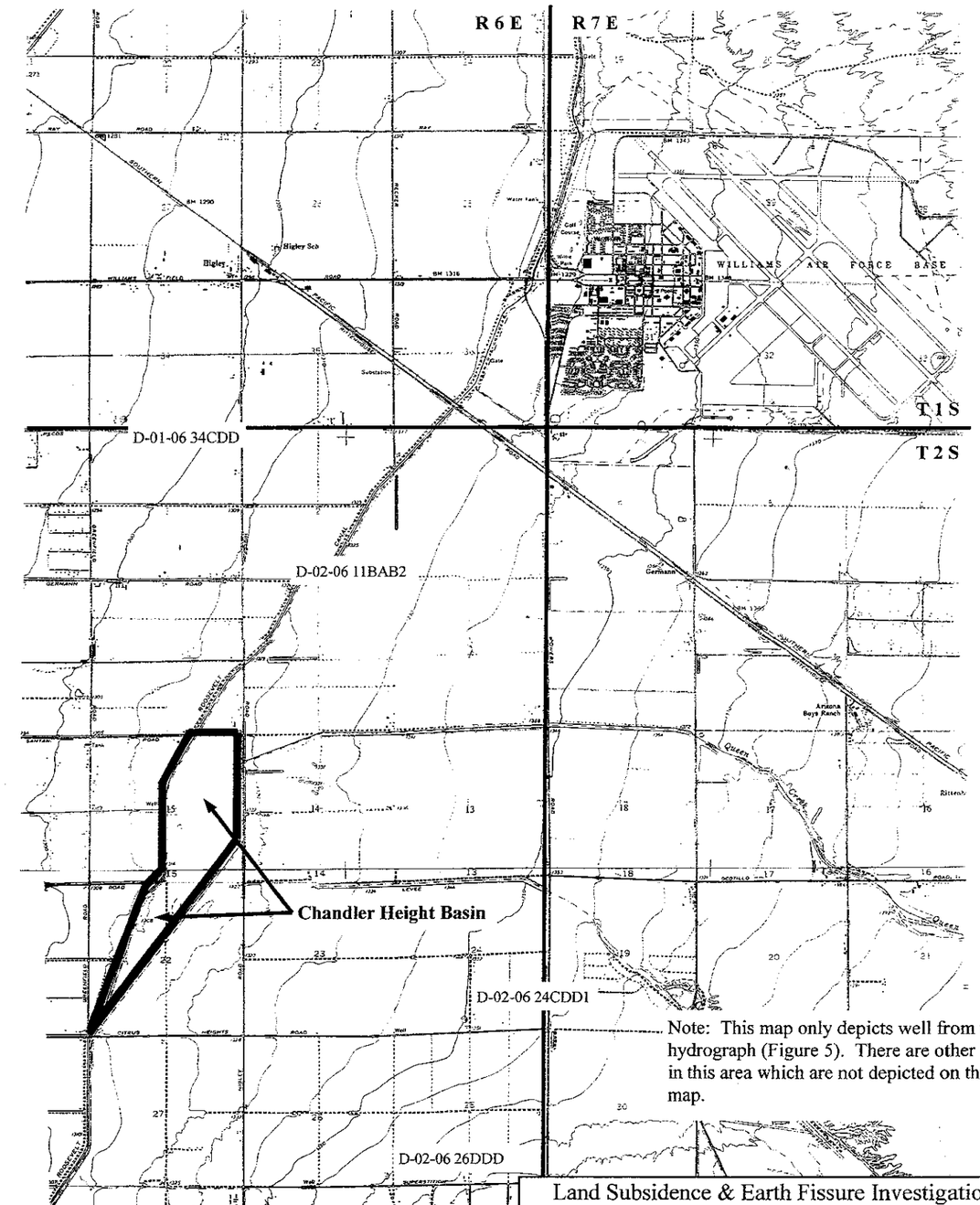


Land Subsidence & Earth Fissure Investigation
Chandler Heights Basin

Figure 4

Earth Fissure 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.

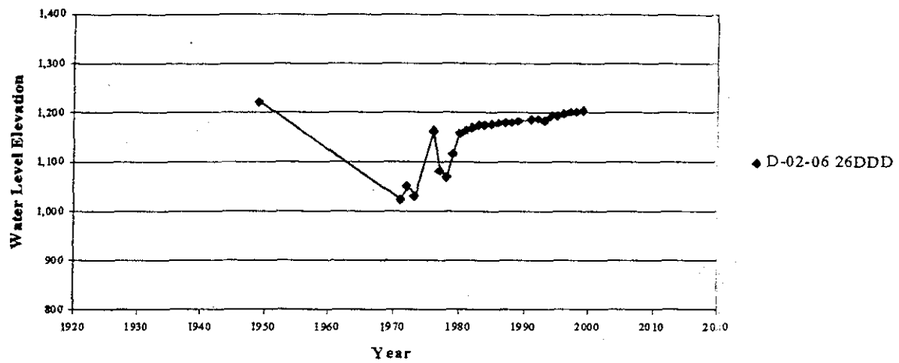
Land Subsidence & Earth Fissure Investigation
Chandler Heights Basin
East Maricopa Floodway Project

Figure 5

Well Location Map

Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021

D-02-06 26DDD



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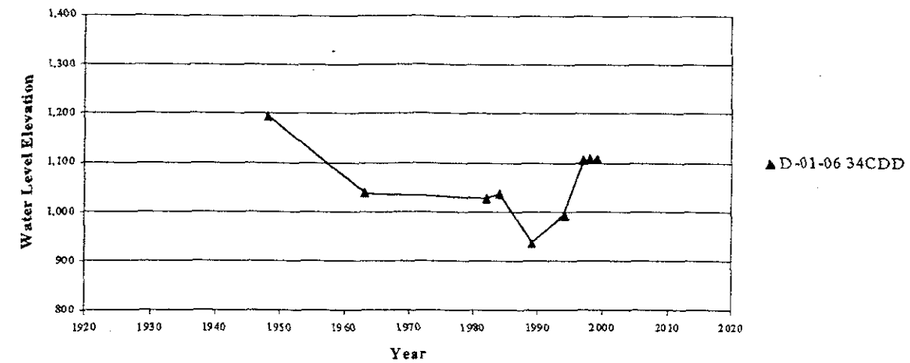

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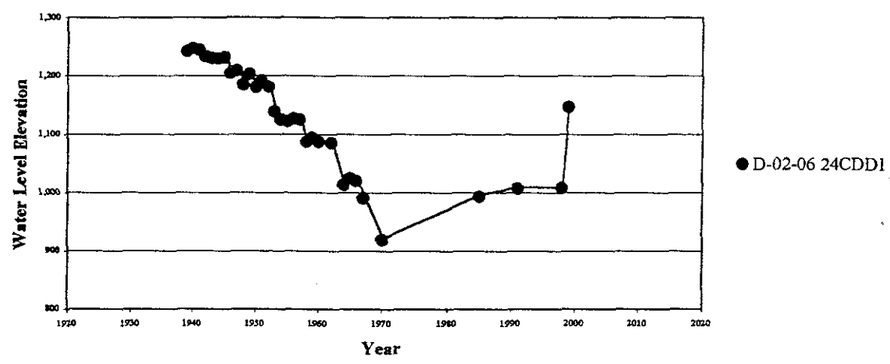

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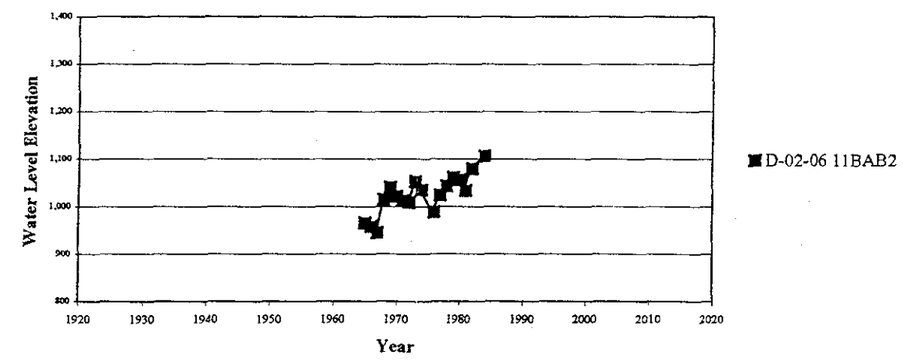
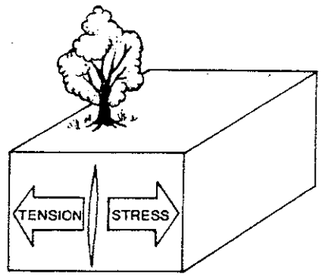


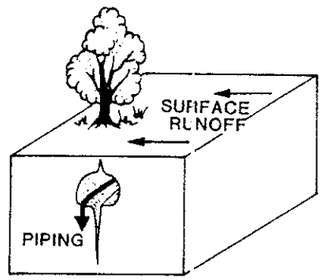
Land Subsidence & Earth Fissure Investigation
Chandler Heights Basin
East Maricopa Floodway Project

Figure 6
Well Hydrographs

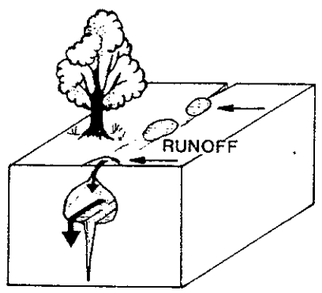
Geological Consultants Inc.
2333 W. Northern Ave., Suite 1a Phoenix, AZ 85021



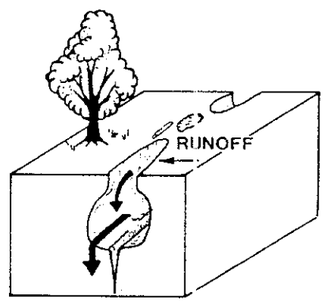
1. Lateral stresses induce tension cracking



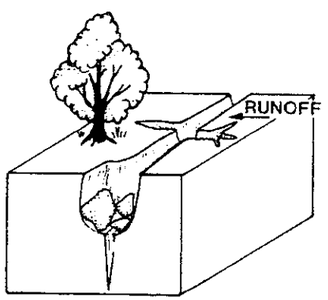
2. Surface runoff and infiltration enlarge crack through subsurface piping



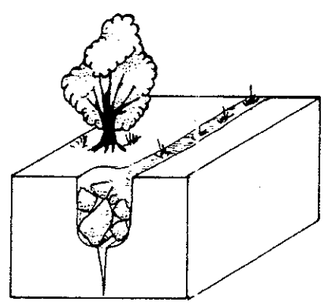
3. As piping continues, fissure begins to appear at surface as series of potholes and small cracks



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 7. Generalized states of fissure development (from Pewe, 1981).

APPENDIX A

General Guidelines for Recommended
Subsidence and Earth Fissure Monitoring
from Program and Policy, Structure Assessment Program Phase I
(Modified by Geological Consultants Inc., 2002)

General Guidelines for Recommended
Subsidence and Earth Fissure Monitoring (Kimley Horn, 2000)
(Modified by Geological Consultants Inc., 2002)

Many embankment flood control dams [and similar structures] under the jurisdiction of the District are located in areas of active ground subsidence and earth fissures. The prognosis for continued ground subsidence and earth fissure development is excellent for the foreseeable future. Therefore, the assessment of existing and future potential ground subsidence and earth fissures and their impact on the safety of existing District dams [or similar structures] is a critical element of the dam safety evaluation process. Guidelines for subsidence and earth fissure evaluations, as part of new design or repair investigations, have been developed by various agencies including the National Resources Conservation Service (NCRS) (1985), United States Bureau of Reclamation (BOR) (1971, 1977, 1981, 1985), Arizona Department of Water Resources (1999), and the Maricopa County Flood Control District, (1995). Studies by the BOR resulted in the design and implementation of mitigation measures to deal with subsidence and earth fissures affecting the Central Arizona Project (CAP) canal. The NCRS, in their National Engineering Manual sets forth criteria for subsidence evaluation for the design of earth dam embankments. Likewise, the ADWR Rules and Regulation for dam safety evaluation require subsidence and earth fissures to be addressed. The results of intermittent settlement monitoring of District structures dating back to the mid-1970's, the District staff prepared an in-house report (Staedicke, 1995) documenting subsidence and settlement effecting earthen dams. Recommendations for the implementation of a monitoring program were provided in the report; however, as of this date (2001) no formal program has been initiated.

We recommend the District Subsidence monitoring program outlined by Staedicke (1995) be adopted. It should also be refined and modified or amended where appropriate for application to District dams [or similar structures] and to satisfy other regulatory requirements. The following outline incorporates the salient items of the District program and lessons learn by the BOR, NCRS, and consultants with professional experience dealing with ground subsidence and earth fissures.

Ground subsidence due to groundwater withdrawal from deep aquifers is known to impact alluvium-filled basins in central Arizona including the District. The surface manifestations of ground subsidence include lowering of the ground surface over time and the development of earth fissures (or ground cracks) due to induced tensile stresses within the alluvium-filled basins. The initial

activity of the subsidence monitoring program will be an evaluation of known subsidence within the District. This evaluation will be performed to assess current ground subsidence conditions and characterize the earth fissures present. These results will help formulate the general parameters of the monitoring program and the specific details for monitoring at each of the District's embankment dams [or similar structures]. Where critical subsidence and each fissure conditions exist that might jeopardize dam safety, the monitoring program results could help to develop mitigation measures to reduce potential ground subsidence impacts caused by regional groundwater withdrawal.

The recommended scope of activities to accomplish the subsidence evaluation is separated into three tasks. Task 1 would be directed to an overview assessment of the District using available geological and hydrogeological data and geological interpretation of available aerial photography. Output from Task 1 would be a preliminary map of the District area identifying potential and known subsidence areas and earth fissures. This information would be used to target sites for direct field examination during Task 2 to verify the presence of the features close to District structures. Task 3 includes the preparation of comprehensive settlement/subsidence and earth fissure monitoring program tailored to each embankment dam [or similar] structure. The monitoring program would be design to incorporate trigger mechanisms that would kick in when excessive subsidence, or earth fissure emergency conditions, are identified.

Task 1: Compile Preliminary Subsidence/Earth Fissure Map:

- o Research and compile existing earth fissure and ground subsidence data pertaining to the District service area.
- o Assess future potential ground subsidence induced by groundwater withdrawal at the site and in the site vicinity. Data to complete this assessment will be obtained from the Arizona Department of Water Resources, U.S. Geological Survey Water Resources Branch and private sector hydrogeological consultants familiar with the area.
- o Acquire aerial photographs from available sources, such as Maricopa County, Arizona Department of Transportation, BOR, NCRS, and private sector companies. Aerial photograph interpretation would be used to identify suspect ground subsidence areas and earth fissures.

- o Compile and analyze the data gathered and prepare a preliminary subsidence/earth fissure map of the District area and target areas for the Task 2 field reconnaissance. Use available subsidence monitoring data to determine past subsidence and calculate future potential ground subsidence estimates.
- o Prepare summary report documenting the Task 1 study findings and conclusions.

Task 2: Subsidence/Earth Fissure Field Reconnaissance

- o Conduct a ground-truth field reconnaissance within a 5-mile radius of flood control embankment dams [or structures], identified in Task 1, that are in a subsidence area to:
  - (1) Verify, or refine, and update the earth fissure and ground subsidence data compiled during Task 1.
  - (2) Identify and map earth fissures or other related 'suspect' features that may be present and potentially affect District flood control dams [or structures].
  - (3) Determine the rate of earth fissure growth where feasible using Task 2 information and historical aerial photography or other documentation.
- o Stake and survey the location of the earth fissures and identify exploration sites.
- o Prepare a Task 2 summary report documenting the results of the field reconnaissance.

Task 3: Prepare Preliminary Subsidence and Earth Fissure Monitoring Program

- o Locate, relocate, or reestablish settlement/subsidence monitoring monuments on crest and downstream toe of embankment dams [or structures]. Establish new

monuments where deemed necessary. Relocated, reestablished, or new monuments should be constructed in accordance recognized plans and specifications (NRCS, BOR, ADOT, District). The number of survey monuments should be determined considering the future potential subsidence in the dam area, the structure hazard classification, and other factors that may be deemed critical based on discussion with District staff.

- Locate, relocate, or reestablish monitoring benchmarks in rock tied to an established survey network such as the National Geodetic Survey. All benchmarks should be thoroughly documented and witnessed.

- Identify and find wells near each embankment dam [or structure] that can be used to monitor changes in groundwater levels. This information would be used to refine estimates of future potential ground subsidence.

- Verify benchmark survey control and survey the elevation of all monitoring monuments. Using the new survey data, rectify all previously obtained subsidence monitoring data relative to an established survey datum.

- Based on the results of the new survey and the rectifying of past data, develop a resurvey schedule suited to each dam [or similar] structure. The surveys should be rerun at yearly intervals for two or three years to see if any trends exist. The monitoring intervals could be changed to range from one year to four or five years depending upon trends established or the calculated estimates of future potential subsidence. A suggested monitoring schedule is provided in Table A-1.

- Earth fissure monitoring should be conducted concurrently with the subsidence monitoring program. In areas of known active earth fissures, the monitoring intervals may need to be more closely spaced especially where an earth fissure is located within one mile of a District structure. Earth fissure monitoring could be conducted using (1) direct examination on the ground by geologists or geotechnical engineers or (2) low-sun-angle aerial photography. The earth fissure survey should also include measurement of its surface expression (length, width, depth, orientation, differential displacement, evidence of activity or inactivity).

Table A-1

Recommended Subsidence & Earth Fissure Monitoring Schedule

Dam Hazard Classification	**Monitoring Schedule[1]**			
	Ground Subsidence	**Earth Fissures**		
		≤ ¼ mile	**¼ mile < D ≤ 1 mile**	**1 mile < D ≤ 5 miles**
High	Annual	Annual	Annual	Biennial
Significant	Biennial	Annual	Biennial	Biennial
Low	Triennial	Triennial	Triennial	Pentad
Very Low	Pentad	Pentad	Pentad	Pentad

The monitoring schedule should be reevaluated on a triennial basis and revised if deemed necessary.

- Surveying of subsidence benchmark and structure monuments should be conducted using currently accepted surveying methods and standards of practice. Survey accuracy standards should be 0.05 feet (or about 2 centimeters).

- Data collected should be recorded in an easily used format such as Microsoft EXCEL. As a minimum, reporting should be done annually. The report should be distributed to other interested parties including BurRec, Corps, AGS, ADOT, ADWR, County highway departments, and local jurisdictions. Additional report could be necessary where rapidly occurring subsidence is documented or when earth fissure growth or development is observed.

References

Staedicke, J. M.;1995; Settlement Monitoring of Earthen Dams Operated by the Flood Control District of Maricopa County; unpublished report.

[1] Suggested monitoring schedule following initial (kick-off) subsidence and earth fissure survey.

SURVEY AND CONTROL

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(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.

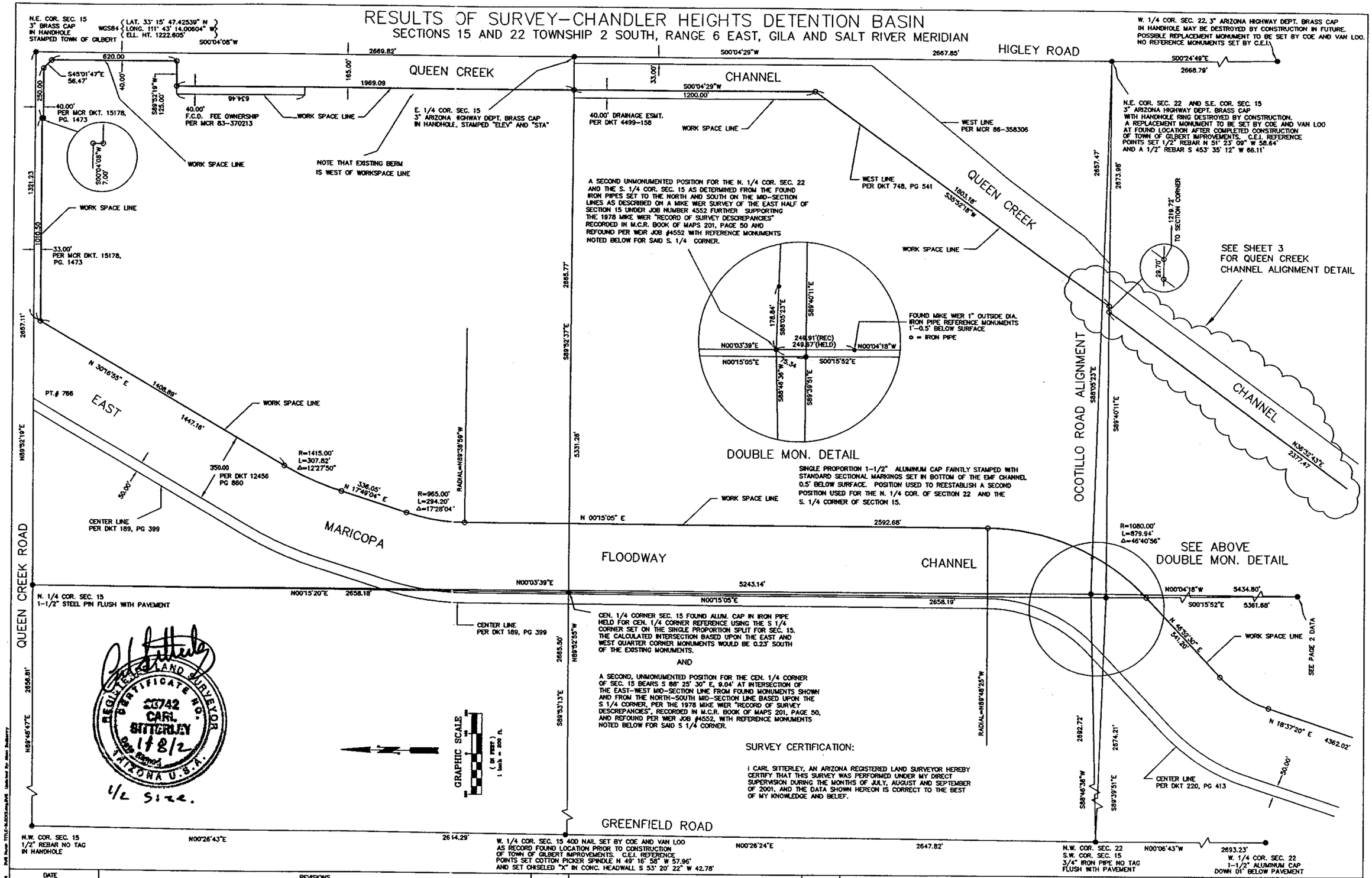
Boundary measurement data and researched record deeds, plats and maps were analyzed by an Arizona Registered Land Surveyor to identify boundary and easement issues affecting the development of each site. A results of survey and a results of survey control drawing is being produced for each site to provide 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 SURVEY–CHANDLER HEIGHTS DETENTION BASIN

SECTIONS 15 AND 22 TOWNSHIP 2 SOUTH, RANGE 6 EAST, GILA AND SALT RIVER MERIDIAN



DATE	REVISIONS

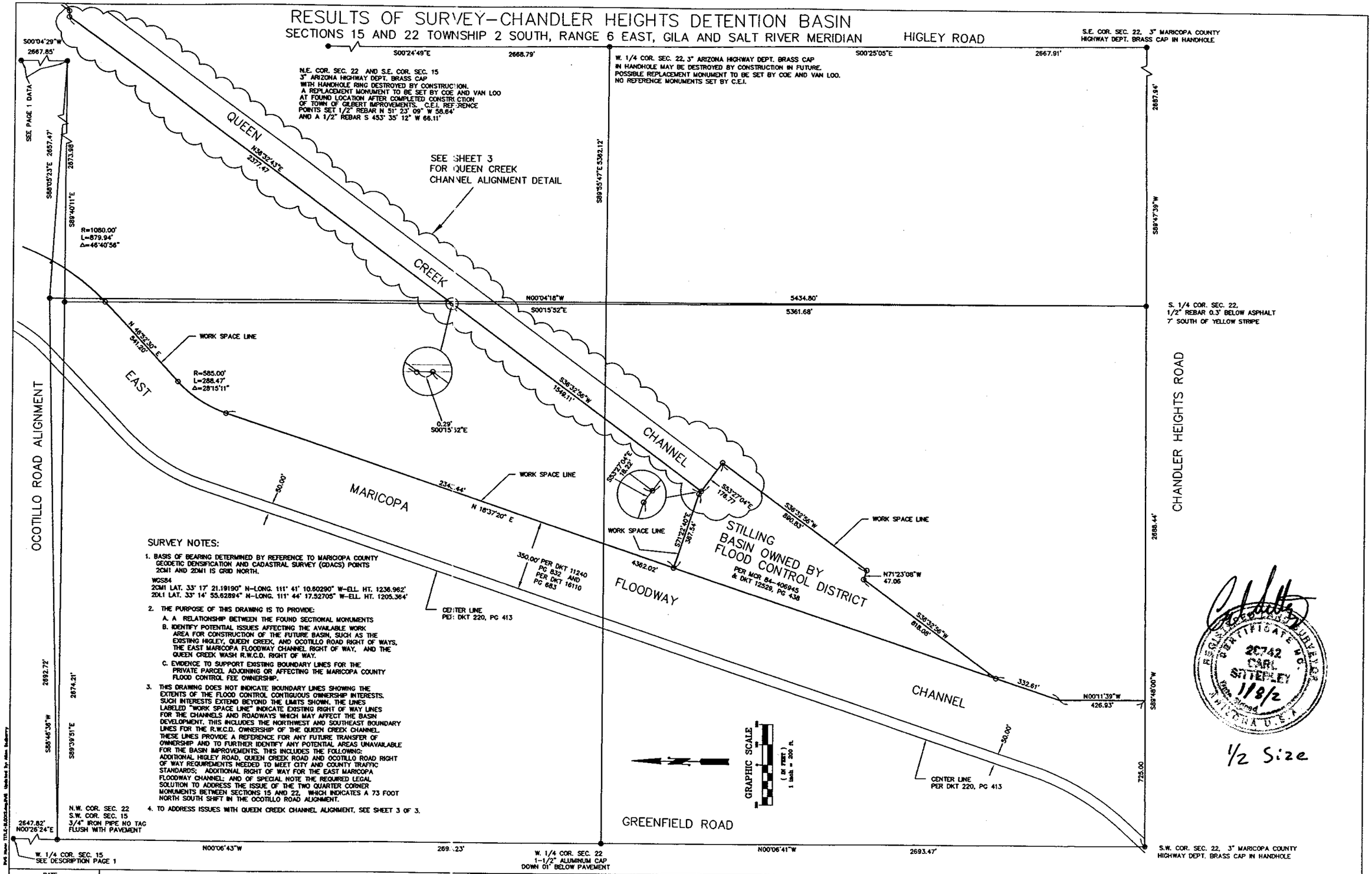
CEI
CONSULTANT ENGINEERING, INC.
3404 W. CHERYL DRIVE
PHOENIX, ARIZONA
602-866-5090

RESULTS OF SURVEY
FOR
MARICOPA COUNTY FLOOD CONTROL

DATE	01-08-02
SCALE	1" = 200'
DRAWN BY	CRS
APPROVED BY	CRS
SHEET	1 OF 3

RESULTS OF SURVEY–CHANDLER HEIGHTS DETENTION BASIN

SECTIONS 15 AND 22 TOWNSHIP 2 SOUTH, RANGE 6 EAST, GILA AND SALT RIVER MERIDIAN



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' 10.60290" 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 HIGLEY, QUEEN CREEK, AND OCOTILLO ROAD RIGHT OF WAYS, THE EAST MARICOPA FLOODWAY CHANNEL RIGHT OF WAY, AND THE QUEEN CREEK WASH R.W.C.D. RIGHT OF WAY.
   C. EVIDENCE TO SUPPORT EXISTING BOUNDARY LINES FOR THE PRIVATE PARCEL ADJOINING OR AFFECTING 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 ROADWAYS WHICH MAY AFFECT THE BASIN DEVELOPMENT. THIS INCLUDES THE NORTHWEST AND SOUTHEAST BOUNDARY LINES FOR THE R.W.C.D. OWNERSHIP OF THE QUEEN CREEK CHANNEL. 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 HIGLEY ROAD, QUEEN CREEK ROAD AND OCOTILLO ROAD RIGHT OF WAY REQUIREMENTS NEEDED TO MEET CITY AND COUNTY TRAFFIC STANDARDS; ADDITIONAL RIGHT OF WAY FOR THE EAST MARICOPA FLOODWAY CHANNEL; AND OF SPECIAL NOTE THE REQUIRED LEGAL SOLUTION TO ADDRESS THE ISSUE OF THE TWO QUARTER CORNER MONUMENTS BETWEEN SECTIONS 15 AND 22, WHICH INDICATES A 73 FOOT NORTH SOUTH SHIFT IN THE OCOTILLO ROAD ALIGNMENT.

4. TO ADDRESS ISSUES WITH QUEEN CREEK CHANNEL ALIGNMENT, SEE SHEET 3 OF 3.

REGISTERED LAND SURVEYOR
CERTIFICATE NO.
20742
CARL
SITTERLEY
Date Signed
1/8/2
ARIZONA U.S.A.

1/2 Size

DATE	REVISIONS

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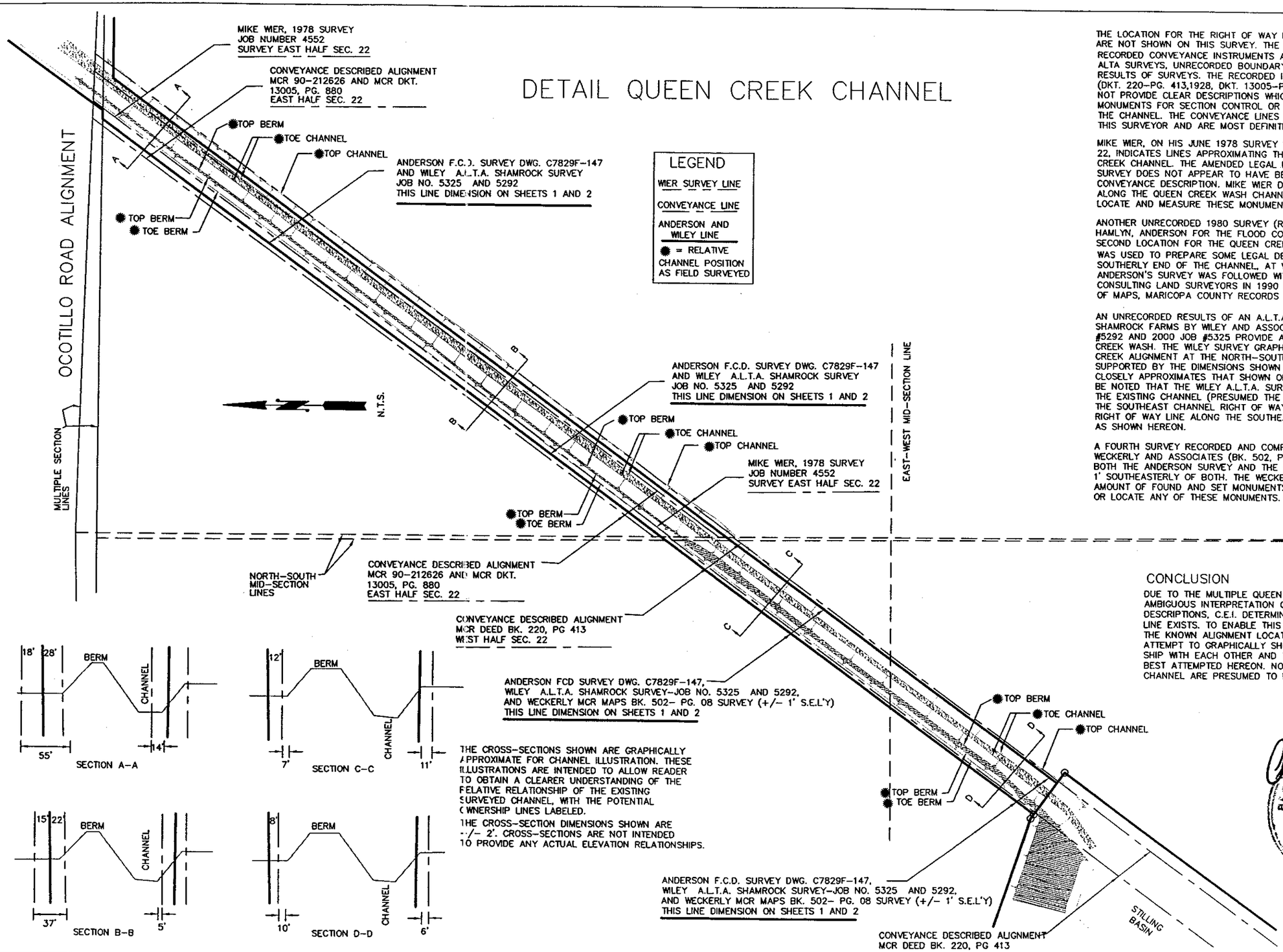
SHEET	2 OF 3

DETAIL QUEEN CREEK CHANNEL



THE CROSS-SECTIONS SHOWN ARE GRAPHICALLY APPROXIMATE FOR CHANNEL ILLUSTRATION. THESE ILLUSTRATIONS ARE INTENDED TO ALLOW READER TO OBTAIN A CLEARER UNDERSTANDING OF THE RELATIVE RELATIONSHIP OF THE EXISTING SURVEYED CHANNEL, WITH THE POTENTIAL OWNERSHIP LINES LABELED.

THE CROSS-SECTION DIMENSIONS SHOWN ARE +/- 2'. CROSS-SECTIONS ARE NOT INTENDED TO PROVIDE ANY ACTUAL ELEVATION RELATIONSHIPS.

THE LOCATION FOR THE RIGHT OF WAY LINES FOR QUEEN CREEK WASH ARE NOT SHOWN ON THIS SURVEY. THE LINES SHOWN ARE BASED UPON RECORDED CONVEYANCE INSTRUMENTS AS SHOWN, UPON UNRECORDED ALTA SURVEYS, UNRECORDED BOUNDARY SURVEYS, AND RECORDED RESULTS OF SURVEYS. THE RECORDED INSTRUMENTS OF CONVEYANCE (DKT. 220-PG. 413,1928, DKT. 13005-PG. 880, AND 90-212626) DO NOT PROVIDE CLEAR DESCRIPTIONS WHICH MATCH THE EXISTING SURVEY MONUMENTS FOR SECTION CONTROL OR THE EXISTING TOE AND TOP OF THE CHANNEL. THE CONVEYANCE LINES SHOWN ARE A BEST GUESS BY THIS SURVEYOR AND ARE MOST DEFINITELY OPEN TO INTERPRETATION.

MIKE WIER, ON HIS JUNE 1978 SURVEY FOR THE WEST HALF OF SECTION 22, INDICATES LINES APPROXIMATING THE TOP AND TOE OF THE QUEEN CREEK CHANNEL. THE AMENDED LEGAL DESCRIPTION ON THIS MIKE WIER SURVEY DOES NOT APPEAR TO HAVE BEEN USED FOR ANY LAND CONVEYANCE DESCRIPTION. MIKE WIER DID SET MONUMENTS IN THE FIELD ALONG THE QUEEN CREEK WASH CHANNEL. C.E.I. DID NOT ATTEMPT TO LOCATE AND MEASURE THESE MONUMENTS AS PART OF THIS SURVEY.

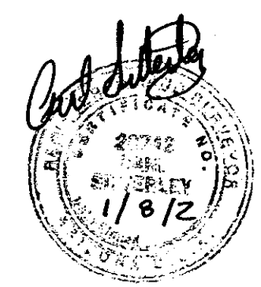
ANOTHER UNRECORDED 1980 SURVEY (REVISED UP TO 1984), BY ADAM, HAMLYN, ANDERSON FOR THE FLOOD CONTROL DISTRICT SHOWS A SECOND LOCATION FOR THE QUEEN CREEK CHANNEL. THE ANDERSON SURVEY WAS USED TO PREPARE SOME LEGAL DESCRIPTIONS FOR PARCELS AT THE SOUTHERLY END OF THE CHANNEL, AT WHAT IS NOW THE SEDIMENT BASIN. ANDERSON'S SURVEY WAS FOLLOWED WITH A RECORD OF SURVEY BY CONSULTING LAND SURVEYORS IN 1990 RECORDED IN BK. 340, PG. 03 OF MAPS, MARICOPA COUNTY RECORDS (MCR).

AN UNRECORDED RESULTS OF AN A.L.T.A. SURVEY WAS COMPLETED FOR SHAMROCK FARMS BY WILEY AND ASSOCIATES. WILEY'S 1999 SURVEY JOB #5292 AND 2000 JOB #5325 PROVIDE A THIRD LOCATION FOR THE QUEEN CREEK WASH. THE WILEY SURVEY GRAPHICALLY SHOWS A JOG IN THE QUEEN CREEK ALIGNMENT AT THE NORTH-SOUTH MID-SECTION LINE. THIS JOG IS NOT SUPPORTED BY THE DIMENSIONS SHOWN ON THE SURVEY, AND THE ALIGNMENT CLOSELY APPROXIMATES THAT SHOWN ON THE ANDERSON SURVEY. IT SHOULD BE NOTED THAT THE WILEY A.L.T.A. SURVEYS SHOW THE SOUTHEAST EDGE OF THE EXISTING CHANNEL (PRESUMED THE TOP) TO BE 30'-40' SOUTHEAST OF THE SOUTHEAST CHANNEL RIGHT OF WAY. THIS PLACES THE SOUTHEAST RIGHT OF WAY LINE ALONG THE SOUTHEAST TOE OF THE EXISTING CHANNEL, AS SHOWN HEREON.

A FOURTH SURVEY RECORDED AND COMPLETED IN 1998 AND 1999 BY WECKERLY AND ASSOCIATES (BK. 502, PG. 08, MCR MAPS) CLOSELY FOLLOWS BOTH THE ANDERSON SURVEY AND THE WILEY SURVEY AND IS APPROXIMATELY 1' SOUTHEASTERLY OF BOTH. THE WECKERLY SURVEY SHOWS A SIGNIFICANT AMOUNT OF FOUND AND SET MONUMENTS. C.E.I. DID NOT ATTEMPT TO FIND OR LOCATE ANY OF THESE MONUMENTS.

CONCLUSION

DUE TO THE MULTIPLE QUEEN CREEK LOCATION LINES AND THE AMBIGUOUS INTERPRETATION OF THE CONVEYANCE LEGAL DESCRIPTIONS, C.E.I. DETERMINED THAT NO CLEAR OWNERSHIP LINE EXISTS. TO ENABLE THIS SURVEY TO BE COMPLETED, ALL THE KNOWN ALIGNMENT LOCATIONS HAVE BEEN SHOWN AND AN ATTEMPT TO GRAPHICALLY SHOW AND DESCRIBE THIER RELATIONSHIP WITH EACH OTHER AND THE EXISTING CHANNEL LOCATION IS BEST ATTEMPTED HEREON. NO BOUNDARY LINES FOR QUEEN CREEK CHANNEL ARE PRESUMED TO BE INDICATED HEREON.



1/2 Size.

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DRAWN BY	CRS
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SHEET	3 OF 3

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

CHANDLER HEIGHTS BASIN – DRAFT REPORT.

Draft Landscaping and Multi-Use Report

I. *Existing Physical Characteristics*

A. Vegetation, terrain, existing features.

Existing vegetation is primarily composed of sporadic desertscrub and small undergrowth. The few trees that do exist on the site are not of high quality and opportunities for plant salvage are not feasible. As part of the proposed drainage improvements, the site will be cleared and grubbed of all existing vegetation.

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 is located around portions of the site's perimeter, predominately barbed wire. 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 eliminated by the basin construction.

The site is currently used by off-road enthusiasts, including four wheel drive trucks, motocross bikes and quads. These users have carved out several trails in and around the site through years of use. Due to the impending project, this user group will be displaced from the site.

B. Adjacent Land Uses and Prominent Features.

The existing land uses that surround the site include primarily agriculture and small pockets of residential development. There are few prominent features in the immediate area that are visible from the site. The East Maricopa Floodway (EMF) is a feature that is accessible from this site.

C. Views and Viewsheds.

Views of the EMF are visible from the site, otherwise on-site views are very limited by the sparcity of natural and man-made features. Significant views to off-site landforms include those looking southward to the San Tan Mountains, northeastly 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.).

D. Site Analysis.

In order to adequately understand the workings of this site a site analysis was prepared. This analysis identifies the physical and spatial considerations that affect how future users may experience this site. Existing conditions and future anticipated land uses are both considered. Within the analysis, major site connections to trails and off-site roadways are identified. The importance of such information is two fold: (1) so that the existing or proposed feature is identified; and (2) so that the spatial affect of this feature is noted. For example, a major road impacts a site on many levels, such as causing noise, disrupting views and vistas, providing access, and requiring illumination. Many of these features act as an activity edge. So, the affect on a site is not only physical, but experiential.

Besides roadways, the EMF and associated Marathon Trail, existing adjacent land uses are identified in the site analysis. The quality of views, in and around a site, is an important factor to any site analysis. By considering the proposed engineering configuration of the basin and its intended use as a recreation facility, broad patterns have been shown for the preferred location of facilities and uses. Areas requiring some type of screening are also called out on the site analysis.

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 TOG intended use of this site as a park facility. Adjacent land uses are predominantly agriculture and residential. The site is currently zoned as Open Space by the TOG.

B. Recreation/Multi-Use Opportunities.

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.

1. Facilities.

The TOG has proposed that the CHB be developed into a community park facility, managed by the GPRD (Gilbert Parks and Recreation Department). The GRPD has identified various recreational activities to be incorporated into this project. These activities range from highly active field sports to more passive, pedestrian orientated activities, such as bird watching. All of the activities require different support systems and infrastructure, and also have varying degrees of impact to the site. These facilities have been incorporated into the site plan to the extent possible as shown in the Conceptual Site Plan.

In addition to athletic and pedestrian facilities, the Conceptual Site Plan provides for the construction of an Activity Center. This center will be the largest structure on-site and a focal point for park activities. The center will house office space for on-site park superintendents and staff, in addition to equipment storage and possibly rental facilities. The structure will also allow for indoor activities such as table tennis, indoor volleyball/basketball and possibly racquetball. Restrooms and locker accommodations will also be included in this facility.

2. Parking.

Surface parking is a necessity at most recreation facilities. The careful design and layout of parking lots can insure proper circulation in and around the site by both vehicles and pedestrians. Parking lots can also enhance the overall design by regulating what visitors can access. It is anticipated that American With Disabilities (ADA) accommodations will be included in the final design of the parking lots by the GPRD.

"Drop-off" accommodations have been incorporated into the Conceptual Site Plan for buses delivering school children or vehicles dropping off and picking up visitors. Delivery vehicles could also utilize this feature. Temporary parking could also utilize this location. To conserve land, employee parking would be included in the main parking areas, rather than separating the uses as is common practice. Maintenance vehicle storage could be within the proposed Park Maintenance Yard.

3. Pedestrian Circulation and Amenities.
Pedestrian use is key to the success of any park. CHB visitors will be allowed the opportunity to participate in a variety of active and passive activities or to merely observe other park users. The Conceptual Site Plan indicates a series of paths and trails that afford pedestrians access to all activities and facilities. It is anticipated that the GPRD will install amenities and facilities solely for pedestrian use; those include benches, tables, drinking fountains, ramadas, trash receptacles, pedestrian-oriented signage and security lighting. These features will be located throughout the park and will occur at varying frequency. Where two or more major pedestrian paths converge, there will likely be several amenities, creating a node. Less definable areas will likely include benches and shade, and occasional receptacles and drinking fountains. It is expected that security lighting will occur at regular intervals along all pedestrian ways, as well as complying with applicable local guidelines for such lighting.

4. Plant Material.
The use of plant material in park settings insures a visually pleasing facility. As stated later in this section, the overall plant list includes predominantly desert-adapted plants. However, the use of turf and ornamental species will be an important component of the landscape plan and design. Even though many ornamental plant materials could be considered high water use plants, these plants are very affective at accenting entryways onto the site or for creating focal points. Plant material selection will vary depending upon the site location. For example, the area adjacent to basketball courts will likely be turf with no trees or shrubs. Along a multi-use path, there will likely be several trees installed to provide shade and increase wildlife observation. In addition to trees, there will be an understory consisting of shrubs, groundcovers and accent plant materials along trails and paths, creating a comfortable pedestrian zone.

We anticipate that turf will be installed not only for those activities that require it, such as soccer and softball. It could also be provided in large areas for more passive activities, such as frisbee throwing and kite flying. These large turf areas allow for many different activities that do not require facilities or permanent equipment. Large open lawns also allow for the gathering of large groups for demonstrations or other organized festivals or performances. Low lying areas, wetlands or swales will have a very different plant palette. These lowland areas will include plants that can tolerate periods of inundation, as well as times of minimal supplemental watering. In addition to a diverse plant palette, these areas may become habitat development zones for birds, reptiles and other desert wash or wetland species.

The landscaping associated with this facility will be in accordance to TOG standards. These standards would include the plant material selection, density and, to some extent, plant placement. The TOG's acceptable plant material is primarily desert-adapted trees and shrubs. The landscape design of this site will include plans for an automatic irrigation system, controlled by an electric or solar controller. This irrigation system will also be in accordance to local guidelines and regulations. The GPRD will provide maintenance of the landscape and irrigation system.

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
Blue Palo Verde	Cercidium floridum
Native Mesquite	Prosopis velutina
Texas Honey Mesquite	Prosopis glandulosa
Fan-Tex Ash	Fraxinus velutina
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

5. Interpretive Opportunities.
This proposed recreation facility would provide numerous interpretive opportunities. Because of these opportunities, visitors will not only have the chance to participate in numerous activities, but also gain knowledge on a number of possible topics. These interpretive opportunities could include a vast number of topics, some of which include wetland restoration/creation, wildlife habitat development, flood control, groundwater re-charge, plant identification and environmental issues pertaining to central Arizona. The interpretive exchange can be achieved through the use of signage, pamphlets and booklets. Interpretive opportunities are also viable for school children and self-guided tours. Some of the interpretive opportunities or themes include:

a. Wetland Restoration/Creation. The interpretive opportunities associated with a functioning wetland numerous. How a wetlands is formed, how it sustains itself and what role it plays in the environment are a few topics that could be addressed for this opportunity.
b. Wildlife Habitat Development. The development of habitat areas can include several different types and species. Habitats can range from desert-dwelling reptiles to bird habitats to aquatic environments. Local bird watcher clubs and similar groups could play a role in the development, maintenance and associated costs of developing such habitats. The interpretive opportunities can be achieved through a series of all-weather signs, brochures and pamphlets describing the creatures and habitats that can be

found at the site. Nature-walks along and through the Queen Creek conveyance channel on the Eastern edge of the basin, can bring visitors into these created environments and offer educational information at nodes along such trails. The possibility does exist for the development of a "Nature Center" within this park site. This facility could provide classroom space for lectures and classes and also provide space for displays and exhibits. School children may attend field trips to this location, and view presentations and demonstrations regarding the local wildlife and its habitat.

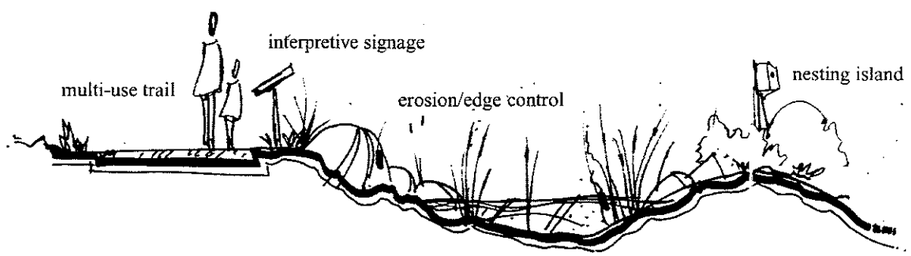
c. Flood Control/Recharge. The opportunity to educate the public about the intent and design of this site, and the role it plays in flood control and recharge, if the latter is included in the basin function, is a viable interpretive feature. As with habitat development, this can be achieved through the use of signage, pamphlets and brochures. Lectures and presentations may also be provided.

d. Plant Identification. The use of small placards to identify certain plant species has been used successfully throughout botanical gardens, as well as in public parks. These signs would likely include a photo or drawing of the specific plant, common and botanical names and a brief introduction to the plant and its identifiable characteristics.

e. Environmental Issues. This opportunity includes such issues as recycling, alternate power sources, urban sprawl and similar items. These elements would likely be issues covered by sponsored lectures and presentations on site, rather than randomly placed informative signs.

6. Civic Involvement.

Such park facilities often allow local clubs and organizations the opportunity to exercise their area of expertise within its boundaries. Such groups may include garden clubs, rose clubs, radio-controlled vehicle groups and bird watcher groups. These groups often provide improvements to the park, such as the planting of rose beds and enhanced bird habitats, at no actual cost to the community. The Conceptual Site Plan does not identify areas explicitly for such uses.



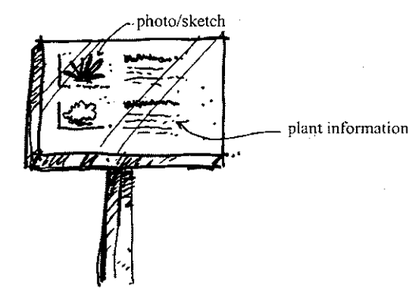
WETLAND/HABITAT DEVELOPMENT

7. Security.

We anticipate that GPRD, Park Rangers and Town of Gilbert Police Department will provide the security of the park. Park Rangers will likely patrol the park during hours of operation, and provide assistance to visitors. Police Officers will also patrol the park during hours of operation, but will also have access to the park after hours for general patrolling and to investigate possible disturbances. Park Rangers and Police officers could patrol the park by means of several different modes of transportation. Those means could include patrolling by foot, bicycle, horseback, ATV's and standard vehicles or cruisers. During times of basin inundation, water transportation maybe needed and should be addressed further by the involved departments. Because of this security presence, there may be a need for a TOG substation or staging area. In addition, FCD personnel will have unobstructed access to the basin and EMF for routine site visits and inspections.

B. Site Access.

This project site will primarily be accessed by motor vehicle by means of Queen Creek Road. There is also the possibility of a secondary, or maintenance access point along Higley Road or Ocotillo Road. The primary access, at Queen Creek Road, will be identified by signage, provided by the TOG, and will be consistent with GPRD park signage. In addition to signage, the entry to



PLANT IDENTIFICATION AND SIGNAGE

the site will be lighted by means of overhead and accent lighting. The use of plant material, grading and possibly water features will aid in the identification and accentuation of this access point.

Bicyclists, equestrians, rollerbladers and pedestrians can also access this facility; these visitors can enter the facility at various pedestrian entries around the site, including from the EMF. These entries will probably have park signage, identifying the park hours, rules and regulations. In addition, it is expected that lighting and landscaping will be included at the pedestrian locations. Other site amenities that may also be incorporated include seating, trash receptacles and drinking fountains.

Maintenance vehicle access to the site will be separate from the primary entry. Maintenance access will be signed to allow only authorized vehicles to proceed. These entries will likely be minimally address and include only security lighting and minimal landscaping or features. Security fencing will be a part of this location.

We anticipate the pedestrian trails within the site will include security lighting and frequent rest stops that will include seating, shade and a drinking water source. These trails are anticipated to be at least 10' wide and comprised of a hard surface material which meets ADA requirements. Grades and side slopes will be in accordance to ADA standards and requirements of TOG. It is expected that these trails will also be constructed as to accommodate use by emergency vehicles and general maintenance equipment. Pedestrian-oriented signage will probably be installed along these paths and will be in the form of post and pedestal-mounted signs and kiosks at path intersections.

There may be times when access to the site will not be allowed, for example; after operational hours, during times of basin inundation and when extensive maintenance is being conducted. During these instances, access to the site will be controlled by a combination of gates, fences and signage installed by the TOG. The role of park rangers and police officers, in regards to safety and controlled access to the site, has been described above.

C. Landform/Grading.

Overall grading of the site will be designed to encourage positive drainage into the EMF through the provided outlet facilities. Accent grading and berming by the GPRD may impede the overall drainage of the site; the GRPD's consulting engineer will address any issues related to the hydraulics of the site after installation of the recreation features. The proposed park facility will include several athletic courts and fields having specific drainage requirements to allow for their proper function. It is assumed that the GPRD will investigate whether these grading drainage requirements will reduce the effectiveness of the overall drainage patterns established for the basin during its final design.

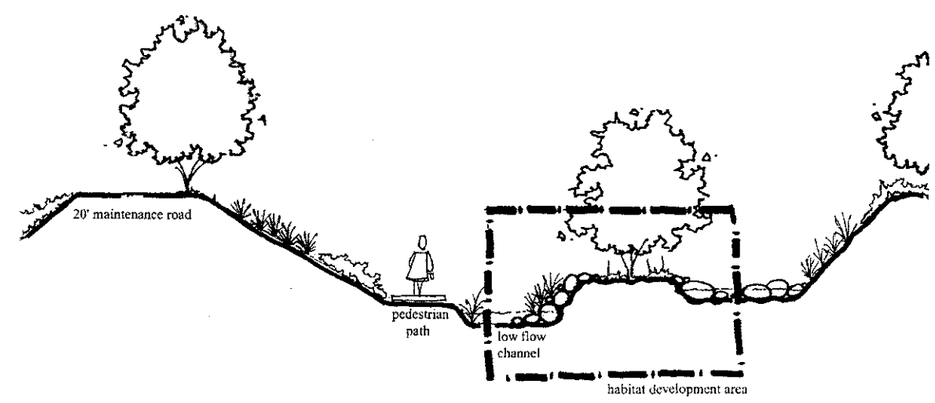
Within this project, there are proposed features and/or facilities that will be damaged or otherwise destroyed should they become inundated during storage of flows. These facilities include the activity center, BMX track, skate park, restrooms and concessions area, equestrian center, maintenance yard and a possible caretakers residence. As a result, these features will be located on a 20-acre platform, set at or above the highest possible water level that may be attained in the basin. As a result, these structures and facilities will not be exposed to inundation. Some proposed elements can be flooded for periods of time and can be designed to insure minimal damage or maintenance as a result of inundation. These elements include basketball courts, athletic fields (soccer, football, softball), volleyball courts and pedestrian use areas, such as paths, trails and rest areas.

It is known that some level of maintenance and general cleanup operations will be required after any significant storage of storm flows in the basin. Maintenance may include the removal of debris carried by floodwater, including uprooted vegetation, sediments and man-made refuse. Additionally, certain facilities may require repairs after basin activity, such as rust proofing, insulation of electrical equipment, painting and possible re-grading of softball/baseball infields. Pedestrian paths and trails will also likely require cleaning and removal of debris after such events. Through the use of area-specific grading, and the creation of swales adjacent to high use areas, a faster "dry-time" can be achieved as water recedes and zones begin to dry.

The use of grading and landforms to create both wetland (where possible) and habitat areas are recommended. Grading solutions can aid in the creation of wetland areas conveying rainfall runoff to pre-determined locations, and subsequently allowing for certain amounts of water to remain in these locations. This may create a self-sustaining wetland. Habitat development can also be enhanced by the manipulation of grades and landforms. These modifications can create nesting islands within wetland areas or the channels that may occur on this site. In addition, landforms can be designed so as to make human access a challenge, thus allowing for a habitat-dedicated zone or area that will not be infringed upon by visitors.

D. Conceptual/Illustrative Site Plan

The Conceptual Site Plan includes several elements and features identified as being integral to the success of this park by the TOG. Some of those features include athletic courts, BMX and skateboard facility, equestrian facility, soccer complex, softball fields, activity center, dog-park, parking, amphitheatre and access roads. Other activities and elements will be incorporated into the plan, where possible, as the design process continues. Some activities have not been shown on the Conceptual Site Plan: they include space for local "soaring" clubs, archery club space and playground areas. The plan is conceptual and subject to refinement as the final design for the basin is completed.



HABITAT DEVELOPMENT

IV. *Proposed Chandler Heights 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 minor locations. Twenty (20) acres of land (not all in a single location) will be retained at approximately the existing ground elevation so that a "dry" area for built facilities and parking can be provided during times of basin inundation. These recreation facilities would be designed and built by the TOG meet all municipal codes, regulations and standards. No "aesthetic grading" will be undertaken by FCD for these facilities. The floor of the CHB will be gently sloped to the outlet structures that discharge into the EMF to create positive drainage within the basin. The basin bottom grade will be so slight as to appear flat.

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 1,000' long concrete side-weir along the Queen Creek conveyance channel at the east edge 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. In addition, two outlets, with concrete headwalls will be provided to drain the basin into the EMF.

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:

- 1. Side-weir and apron. The main purpose of this device is to set a predetermined elevation at which flows in the conveyance channel will spill over into the basin via the side-weir, thus alleviating some demands on the channel. The side-weir is approximately 1,000' in length, and will have an access/maintenance road atop it, with a width of about 20'. The weir steps into the basin and terminates into riprap, that will function as an 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 basin engineer; however, the appearance may be modified to produce a more aesthetically pleasing feature.
- 2. Riprap 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 FCD standards, although the general shape and possibly the size of the cobbles can be influenced to create interesting elements, as well as maintaining its intended purpose.
- 3. Flood gates. The planned outlets from the basin, which 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. 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 TOG plant palette and within the parameters of the FCD's Aesthetic Guidelines. The plants will be primarily desert adapted, low water use trees, with additional use of shrubs, groundcovers and accent material. The plant selection will be coordinated with the GPRD representatives. The ground plane will be seeded with native plant species, and turf will be installed in certain locations. The plantings will be watered via an automatic, underground drip system.

D. Erosion Control/Protection.

Soil erosion can occur at two distinct phases of development and operation of the CHB. 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 through the pipe outlet or when water spills back over the EMF 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.

Rainfall and associated sheet-flow are common causes of slope erosion. Because the proposed recreation facility is within a flood control basin, the entire perimeter is comprised of man-made side slopes. The erosion damage located on such side slopes usually includes rivulet and gully formation, all resulting in destabilized slopes and a loss of soil and surface treatment material, such as decomposed granite or seed. Mitigation of such erosion includes the installation of vegetation on the slopes, implementation of swales, terraces and channels to move the runoff perpendicular to the slopes, and eventually down the slope in a controlled manor at a pre-determined location.

E. Section 404.

An Individual Permit will be used to authorize construction of the Chandler Heights 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.

GROUNDWATER RECHARGE CAPABILITIES

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Preliminary Estimation of Recharge Capabilities in the Proposed Chandler Heights Basin

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Appendix A. Ground Water Site Inventory Database Listing
Appendix B. ADWR Drillers' Well Logs

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1.0 Introduction

The purpose of this report is to document the subsurface recharge potential of the Chandler Heights Basin project proposed by the Maricopa County Flood Control District. Excess water is to be collected from major flood events into the basin, where it will be recharged into the soils and aquifer below. Ideally, an investigation such as this involves field work that may include the drilling of soil borings and perhaps surface geophysical surveys to develop a consistent evaluation of the soil characteristics. However, this study was dependent on existing information, including driller's reports, shallow percolation tests, Arizona Department of Water Resources (ADWR) well databases, and previous hydrologic investigations. The difficulty in relying on such sources is their inconsistencies and the limitation to others interpretation of raw field data, as well as the lack of site-specific field data. To make the most of the available information, the interpretations presented in this report are greatly simplified and must be treated as preliminary.

The location of the proposed Chandler Heights Basin is Township 2 South, Range 6 East, including parts of sections 10, 15, and 22. The west side of the basin is bound by the Roosevelt Canal and the southeast side of the basin is bound by the Queen Creek channel, both of which constitute the East Maricopa Floodway.

2.0 Background Hydrogeologic Conditions

The study area for this report encompasses the proposed Chandler Heights Basin and the area approximately one square mile beyond the basin borders. Well information was obtained for this area using the Arizona Department of Water Resources (ADWR) groundwater site inventory (GWSI) and 55 files databases (ADWR, 2000a) and driller's logs (*Appendix A and B*). A well location map is provided in *Figure 1*.

2.1 Geology

The subsurface geologic features that can most hinder a recharge project are shallow restricting layers above the water table. In the Salt River Valley, restricting layers are primarily fine-grained silt or clay layers and caliche. Stream channel deposits are generally coarse-grained and permeable, except locally where overbank deposits with intermixed silt and clay are more prevalent. In general, deposits outside of the flood plain are coarse-grained, highly permeable stream channel deposits covered with a thin veneer of finer-grained deposits (Ken D. Schmidt and Associates, 1995).

Three major subsurface alluvial units have been described in ADWR groundwater modeling studies of the Salt River Valley (Corkhill et al., 1993). The deposits of the Upper Alluvial Unit (UAU) are generally coarse-grained, unconsolidated, and average approximately 200 feet in thickness in the study area (Ken D. Schmidt and Associates, 1995). They include the alluvial fan deposits of Queen Creek and farther west, stream channel deposits of the ancestral Salt River. Deposits of the UAU are primarily above the water table. The Middle Alluvial Unit (MAU) is the next underlying unit with finer grained deposits than the UAU. These fine-grained deposits of the MAU typically act as a confining bed between groundwater in the Upper and Lower Units. However, in the area of the proposed Chandler Heights Basin, some of the MAU is coarser grained and not considered to be a confining bed (Ken D. Schmidt and Associates, 1995). The thickness of the MAU ranges from approximately 100 feet to more than 600 feet beneath the study area, thinning to the south. Most of the production wells in the area are screened entirely or primarily within

the MAU. The Lower Alluvial Unit (LAU), present beneath the MAU, consists of coarse-grained deposits that are often consolidated, but can be highly productive.

2.2 Groundwater Levels

The rate of seepage from a recharge basin will not be impacted by groundwater if the water table is below the basin bottom (Bouwer, 1978). The ADWR year 2000 databases were accessed for the latest groundwater level information. In the winter of 1997-1998 several wells in the area were measured for groundwater level, as shown in *Figure 2*. A couple of wells in the area indicated significant differences in water level at the same location and time. For instance, two wells located at D-02-06 21CAA have water levels differing by over 100 feet. Such differences may be explained by different screen intervals, but may also be explained by one well pumping and the other not. To minimize water level differences caused by differences in screen interval and pumping, the data presented in *Figure 2* represent non-pumping wells that are primarily screened or penetrating the MAU and only minor portions, if any, of the UAU. Under these constraints, the depth to water ranges from 92 feet below ground surface (bgs) on the west side of Chandler Heights Basin to depths greater than 220 feet bgs on the east side. The actual water levels beneath the basin area are probably from depths of 130 to 160 feet bgs or elevations of 1,180 to 1,150 feet above mean sea level (amsl). Based on these water levels, the general direction of groundwater flow is west to east across the study area. These water level data do not indicate the presence of a perched water table within the study area.

Historical water level data from the study area indicate a steady rise in water levels since the 1970s and 1980s, with at least one well indicating recovery to predevelopment levels. Groundwater level hydrographs of select wells are shown in *Figure 3*. *Figure 3.A.* is a hydrograph of a well located at D-02-06 22BDD, just southeast of the Queen Creek channel boundary of the Chandler Heights Basin. This well, which is screened in the MAU and LAU, shows an average water level rise of almost 6 feet per year since November 1982. *Figure 3.B.* is a hydrograph of a well located at D-02-06 9ADD, approximately three quarters of a mile northwest of the Roosevelt Canal that borders Chandler Heights Basin. This well, which is screened in the MAU, shows a water level decline of 170 feet from 1948 until 1975,

and then a steady recovery to the pre-development 1948 water level by 1997. Several wells in the study area were also measured for water levels in November 1991. In comparing these 1991 water levels with the winter of 1997-1998 water levels, there was an average water level rise of 47.5 feet over the 6-year period, or nearly 8 feet of rise per year.

2.3 Groundwater Quality

Groundwater data have been examined to understand the ambient groundwater quality beneath the proposed Chandler Heights Basin area. It is not anticipated that there will be any adverse chemical reactions to recharging floodwaters in the basin since the source water will be composed of precipitation water that is already naturally recharging the area. However, physically adding larger volumes of this water to the subsurface may increase the migration of existing chemical constituents from the soil to the groundwater; therefore, it is important to know the condition of the groundwater quality prior to artificial recharge.

In January 1995, water samples were collected from seven private domestic wells and three stock wells in the vicinity of the proposed Chandler Heights Basin (Ken D. Schmidt and Associates, 1995). These samples were collected from wells with known screened intervals. *Figure 4* shows the well locations, screened interval, and chemical concentrations of selected inorganic constituents from this sampling.

Total dissolved solids (TDS) concentrations in these samples ranged from approximately 400 to 3,000 mg/L. The lowest and highest TDS concentrations were found in wells south of the proposed Chandler Heights Basin. There appears to be little correlation between the screened interval or well depth and TDS concentration, but the wells having the highest TDS concentrations tended to have shallower perforations (<500 feet). Chloride is a predominant ion in these groundwater samples and trends in chloride concentrations were similar to those for TDS. The occurrence of high TDS and chloride concentrations appears to coincide with areas that have been irrigated with Salt River Water (Ken D. Schmidt and Associates, 1995). This increase in TDS may be attributed to the leaching of salts from the soil during irrigation.

Three well samples near the southern tip of the proposed Chandler Heights Basin exceeded the maximum contaminant level (MCL) of 45 mg/L for nitrate concentrations (Ken D. Schmidt and Associates, 1995). All arsenic concentrations measured were below the current MCL of 0.05 mg/L.

Based on the groundwater quality conditions near the proposed Chandler Heights Basin, it is likely that additional recharge with floodwaters could help to dilute some of the groundwater having high TDS concentrations. Alternatively, some of the irrigated soils in the area are probably concentrated with salts that could be mobilized by the additional recharge and contribute to the dissolved solids load of the groundwater.

3.0 Soil Characteristics

Soils located in and around the area of the proposed Chandler Heights Basin have been evaluated for their recharge potential. Data for characterizing the upper 25 feet of soils were obtained from soil boring logs and percolation testing performed by Ninyo and Moore (2001). A broader evaluation of the soil properties down to 500 feet was completed using driller well logs acquired from ADWR.

3.1 Soil Borings and Percolation Testing

Ninyo and Moore (2001) performed shallow soil borings and percolation tests in the area of the proposed Chandler Heights Basin in July of 2001. Six soil borings were completed with an 8-inch diameter hollow-stem auger to depths ranging from 19 to 26.5 feet. The locations of the soil borings are provided in *Figure 1*.

The predominant material identified in most of the soil boring samples, especially in the upper 15 feet, was clay. Layers dominated by sand and gravel were identified below 15 feet in four of the six soil borings (*Table 1*). This is more relevant than the upper 15 feet because the Chandler Heights Basin will be excavated to depths near 15 to 20 feet from the current land surface.

Percolation tests were performed adjacent to the shallow soil borings according to standard method C-109 of the City of Chandler. The tests were performed at or near the proposed elevation of the bottom of the Chandler Heights Basin, at 15 to 20 feet below the current land surface. The diameters of the percolation test holes were at least 12 inches and the sides of the test hole were cased with an impermeable membrane to a depth of one inch below the bottom of the hole. Each test hole was prewetted for 24 hours, or until a stabilized percolation rate had been achieved, maintaining a free water surface during the entire prewetting period.

The results of the percolation test varied from a maximum of 21.8 feet per day to a minimum of 1.7 feet per day. *Table 1* lists the percolation test results along with soil characteristics determined from drilling the shallow soil borings.

Table 1. Shallow Soil Boring and Percolation Test Results

Test Hole Name	CH21	CH22	CH23	CH24	CH25	CH26
Percolation Test Depth (feet bgs)	±20	±20	±15	±20	±20	±15
Drilling Penetration Rate at Test Depth (blows/foot)	34	89	91	37	83	84
Dominant Soil Type at Test Depth	Sand	Clay	Clay	Sand	Gravel	Sand/Gravel*
Infiltration Rate (feet/day)	21.8	7.9	2.4	7.2	3.1	1.7

*Interface between sand and gravel units.

3.2 Drillers' Well Logs

The map in *Figure 1* shows the location of all the wells having drillers' logs available for this investigation. Only the upper 500 feet of each log was evaluated because the focus of this study is the unsaturated soils, above the water table. In addition, because the drillers' logs are so varied according to time, quality, and who logged them, interpretations had to be greatly simplified in order to make them more consistent. Each reported soil type was classified as either coarse, if it was dominated by sands and gravels, or fines, if it was dominated by silt and clay. Thin layers of coarse or fine materials were sometimes omitted if the remaining portion of the log was dominated by another contrasting soil type.

The results of this effort are shown in a three-dimensional diagram, identified as *Figure 5*. Based on the coloring of the diagram, it is apparent that the surface and deeper soils in the area of the proposed Chandler Heights Basin are dominated by fines. However, the unsaturated zone soils do indicate lenses of coarser material near the surface at the

northern tip of the basin and near the surface in the southern part of the basin, south of Ocotillo Road. There also appears to be a thicker section of coarser soils to the southeast of the proposed basin location.

It is important to emphasize that these interpretations of the soil, using drillers' well logs, are based on large simplifications of suspect data. Therefore, the interpretations made here should only be viewed as preliminary, until additional site-specific field data can be acquired.

4.0 Estimation of Recharge Capability

The measured short-term infiltration rates from the percolation testing have been assigned to soil types by relating the test results to the borehole soil samples from similar depths and locations (*Table 1*). The slowest infiltration rates for each soil type were chosen as representative values because over the long-term, slower infiltration rates are generally more realistic (Martin and Swieczkowski, 1999). Using these methods, the following short-term infiltration rates were assigned: clays and other fines = 2.4 feet/day; sand = 7.2 feet/day; and gravel = 3.1 feet/day.

Long-term measured infiltration rates have been evaluated for a number of recharge sites in Arizona and related to their previously estimated recharge rates (Martin and Swieczkowski, 1999). The result is that estimated infiltration rates determined from short-term testing vary significantly from actual long-term infiltration rates. Much of the variation is caused by site-specific factors that cannot be measured during short-term tests, including site variability, facility use and maintenance practices, and subsurface conditions. All of the long-term measured infiltration rates in near-by recharge facilities suggest that the infiltration rates obtained from percolation testing in the Chandler Heights Basin area are overestimates. For instance, the GRUSP site, located in coarse riverbed materials, has one of the highest average long-term infiltration rates of 3.4 feet/day, but more typical long-term infiltration rates of less than a foot per day are found at the Mesa NWWR (0.3 feet/day), Gilbert (0.3 feet/day), Sweetwater (0.8 feet/day), and Vidler MBT Ranch (0.7 feet/day) recharge facilities (Martin and Swieczkowski, 1999).

Applying short-term infiltration rates from percolation testing and long-term infiltration rates from other facilities to the soils information shown in *Figure 5* suggests that infiltration rates within the Chandler Heights Basin, north of Ocotillo Road, will range from less than 1 foot/day up to approximately 2 feet/day. Infiltration rates within the basin, south of Ocotillo Road, will likely be slightly higher, ranging from 2 to 4 feet/day. These estimations suggest possible recharge rates of up to 370 acre-feet/day in the northern part of the basin and approximately 90 to 180 acre-feet/day in the southern part of the basin.

Clogging is generally a long-term problem in artificial recharge with basins. Deposition of fine-grained materials (biological and detrital) is likely to clog the upper soils over time and reduce infiltration rates. Clogging can reduce infiltration rates to only a fraction (sometimes less than one-tenth) of the original infiltration rate (Bouwer, 1978). A sedimentation basin is proposed in the design of this project, which will help reduce clogging problems. Another way that clogging is prevented is by periodically drying the basin so that the clogging layer can dry, crack, curl up, and the organic materials can decompose. However, maintenance issues such as this may not necessarily apply to the Chandler Heights Basin, being a proposed multi-use facility. Landscaping and recreational use of the site are likely to affect the surface soils in other ways, including the occurrence of some natural compaction.

5.0 Conclusions

Hydrogeologic conditions in the area of the proposed Chandler Heights Basin appear to be favorable for periodic recharge of excess storm water. The following points summarize the findings of this investigation.

- Groundwater levels are sufficiently deep at 130 to 160 feet bgs to not impede recharge rates.
- Groundwater quality beneath the Chandler Heights Basin is quite variable from well to well, but some wells show a small indication that shallower groundwater may be higher in TDS and chloride. It is unclear whether additional recharge from the Chandler Heights Basin will dilute existing groundwater quality or increase the dissolved solids by leaching salts from the soil along the way.
- Unsaturated soils in the northern half of the Chandler Heights Basin are dominated by fine-grained materials including silt and clay. Unsaturated soils in the southern half of the basin show higher amounts of coarse-grained materials such as sand and gravel.
- Preliminary estimates of short-term infiltration rates, suggested by discrete percolation tests within the basin area, are as high as 2 to 5 feet/day. Longer-term infiltration rates, indicated by other Arizona recharge facilities are generally lower, suggesting rates of less than one foot/day in fine-grained soils up to 4 feet/day in coarser soils.

Recommendations based on the findings of this report are listed below:

- The recharge estimates provided in this report are only preliminary and further site-specific field investigations are recommended for a more accurate assessment of the recharge conditions of the site.
- Areas with greater concentrations of fine-grained surface soils might offer good locations for sedimentation basins, which leaves the coarser grained soils for recharge areas.

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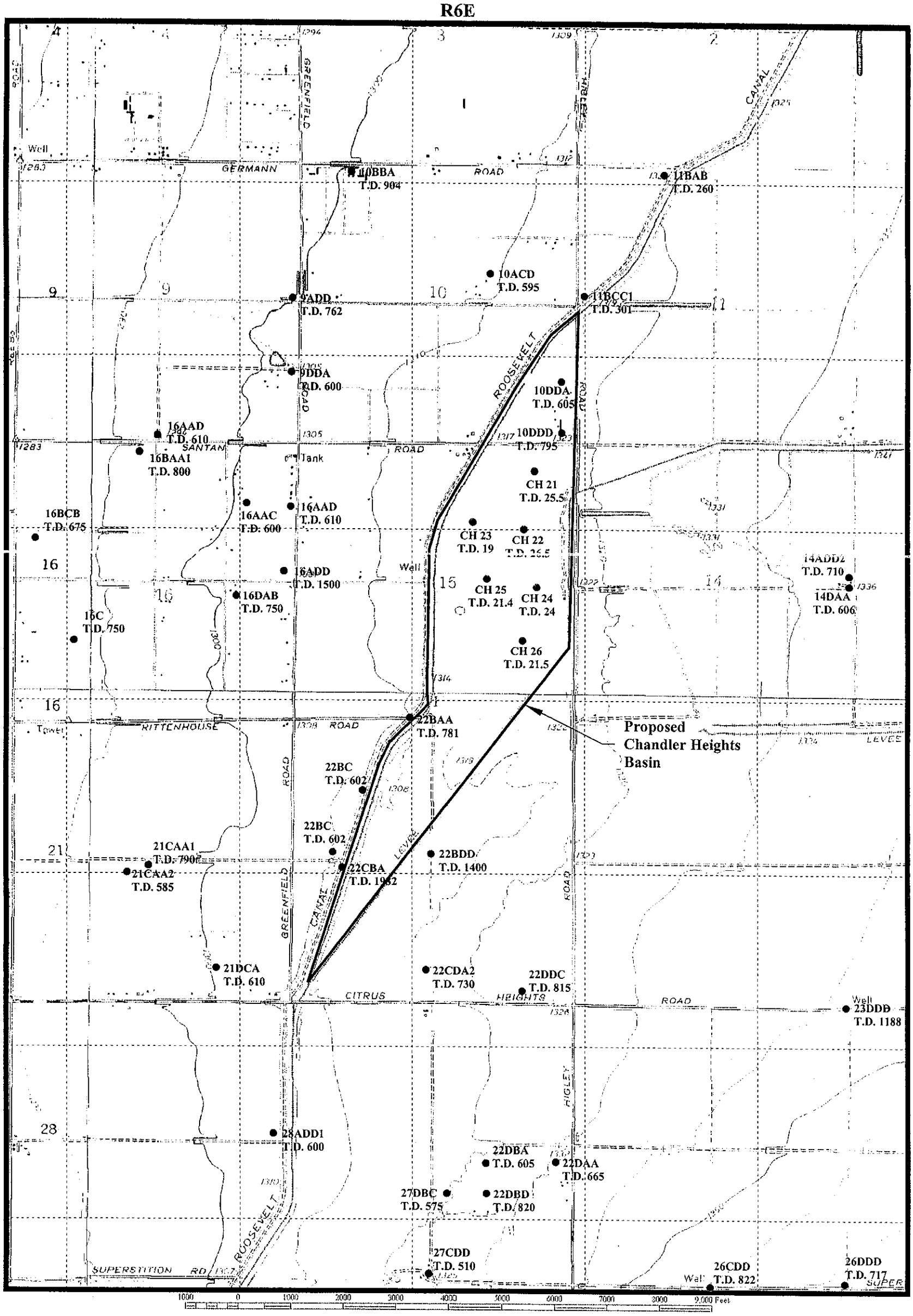

Explanation

● CH 26	Soil Boring Identification	**Sources:** Ninyo & Moore, 2001
T.D. 21.5	Total Depth (feet)	
● 27CDD	Well Identification	ADWR, 2000b
T.D. 510	Total Depth (feet)	

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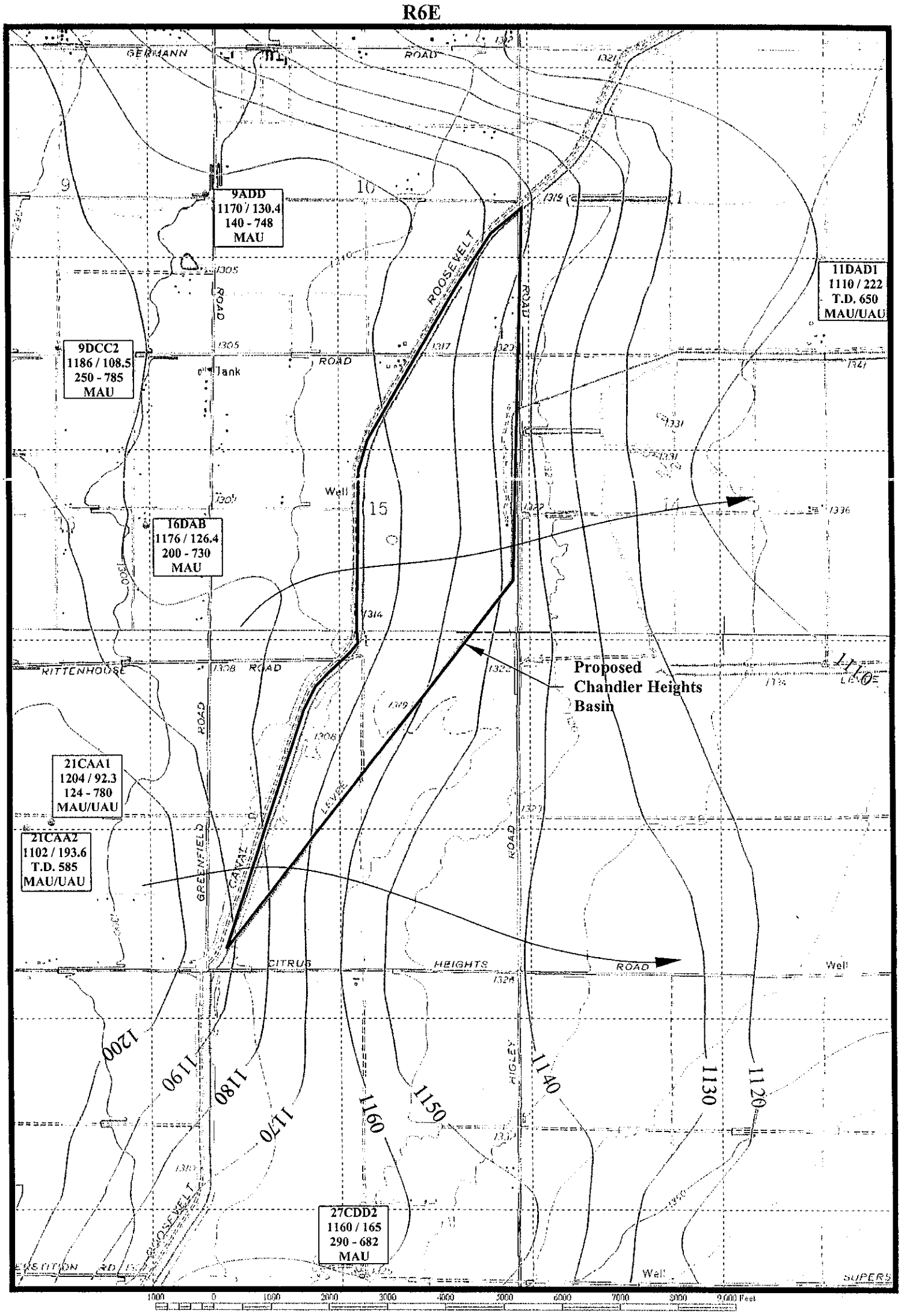
Well & Soil Boring Location Map

Figure 1



Explanation

27CDD2	Well Identification
1160 / 165	Water Level Elevation / Depth to Water (feet Above Mean sea level) (feet Below Surface)
290 - 682	Perforated Interval (feet) or Total Depth (T.D.)
MAU	Alluvial Unit (MAU = Middle, UAU = Upper)

Contour of Water Level Elevation (feet Above Mean Sea Level)

Direction of Groundwater Flow

Source:
ADWR, 2000a

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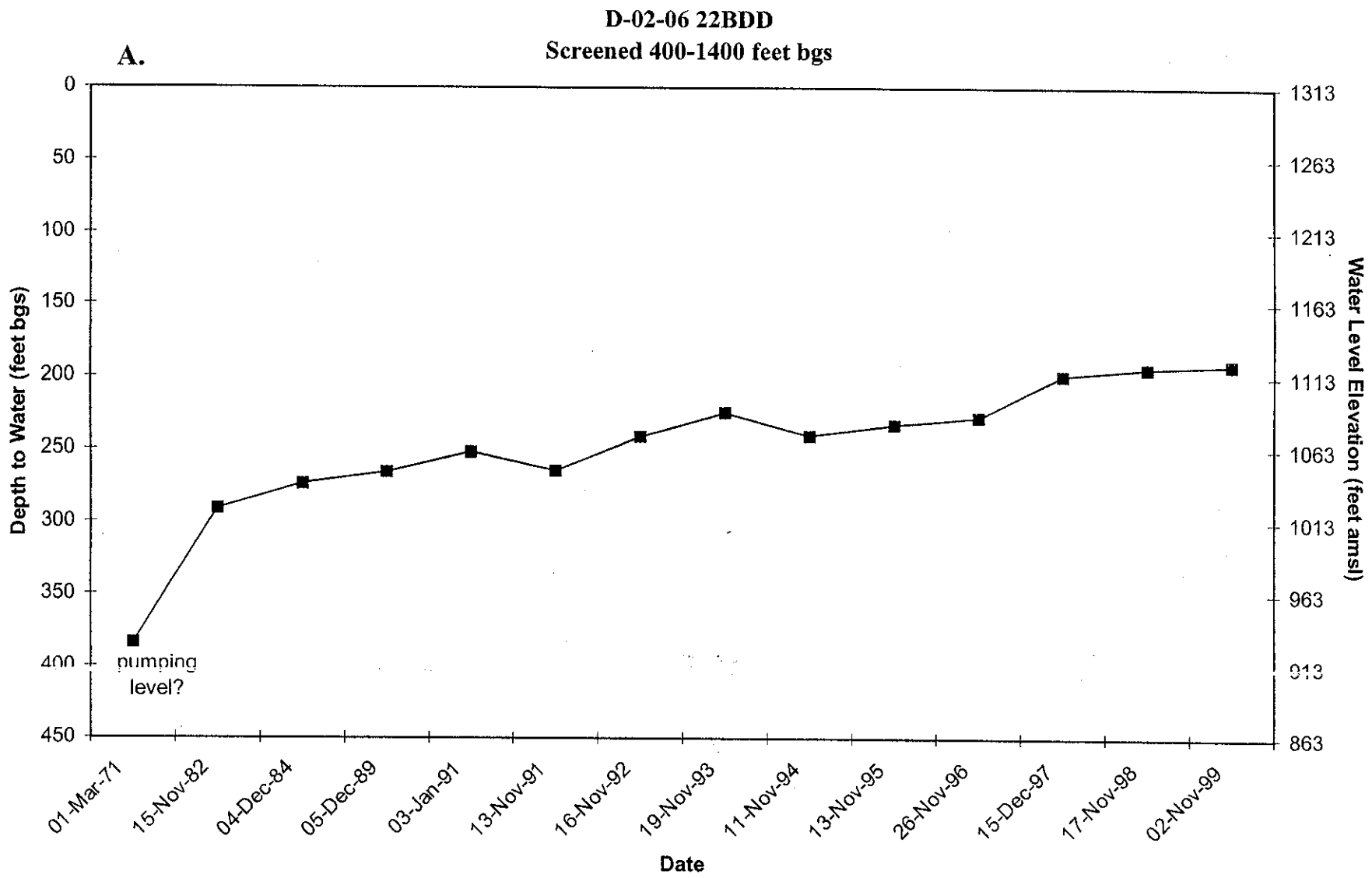
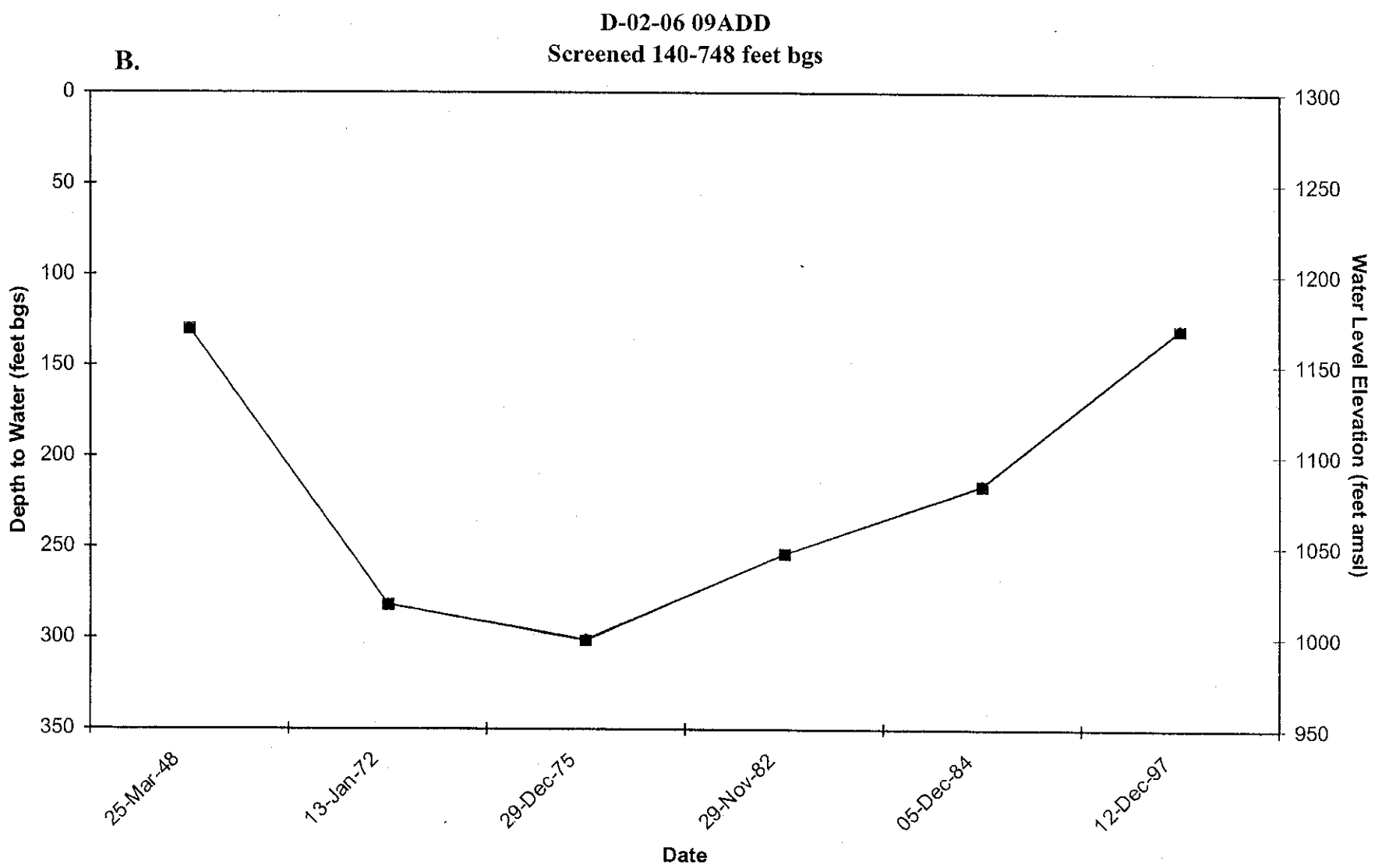
Groundwater Elevations for Winter 1997 - 1998

Figure 2





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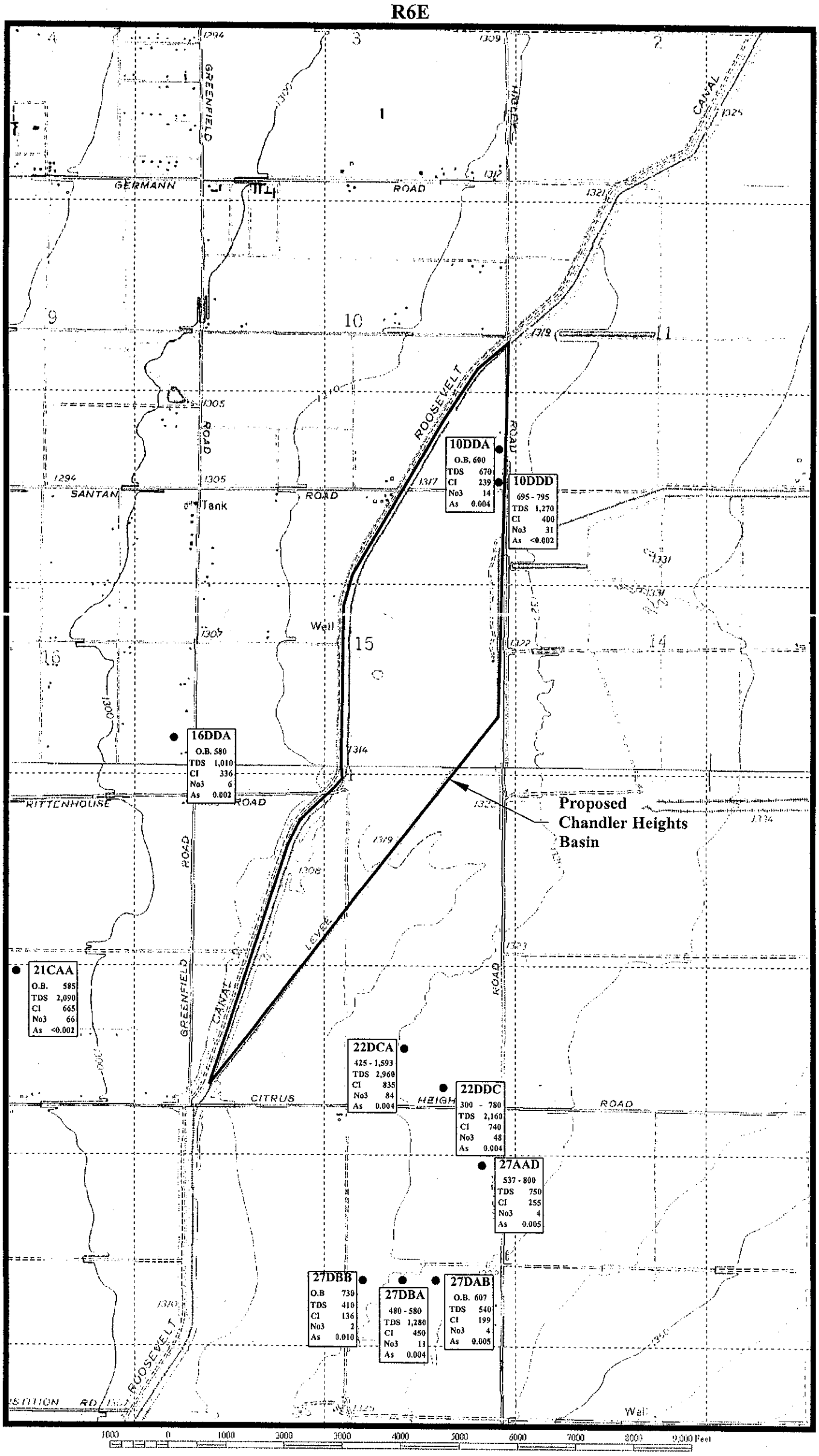
Groundwater Level Hydrographs

Figure 3

R6E



T2S

Explanation

22DCA	Well and Identification
425 - 1,593	Perforated Interval (feet) or Total Depth (T.D.) or Open Bottom (O.B.)
TDS 2,160	Total Dissolved Solids (mg/L)
Cl 835	Chloride (mg/L)
No3 84	Nitrate (mg/L)
As 0.004	Arsenic (mg/L)

Source:
Modified from Schmidt and Associates, 1995

HydroSystems Inc.
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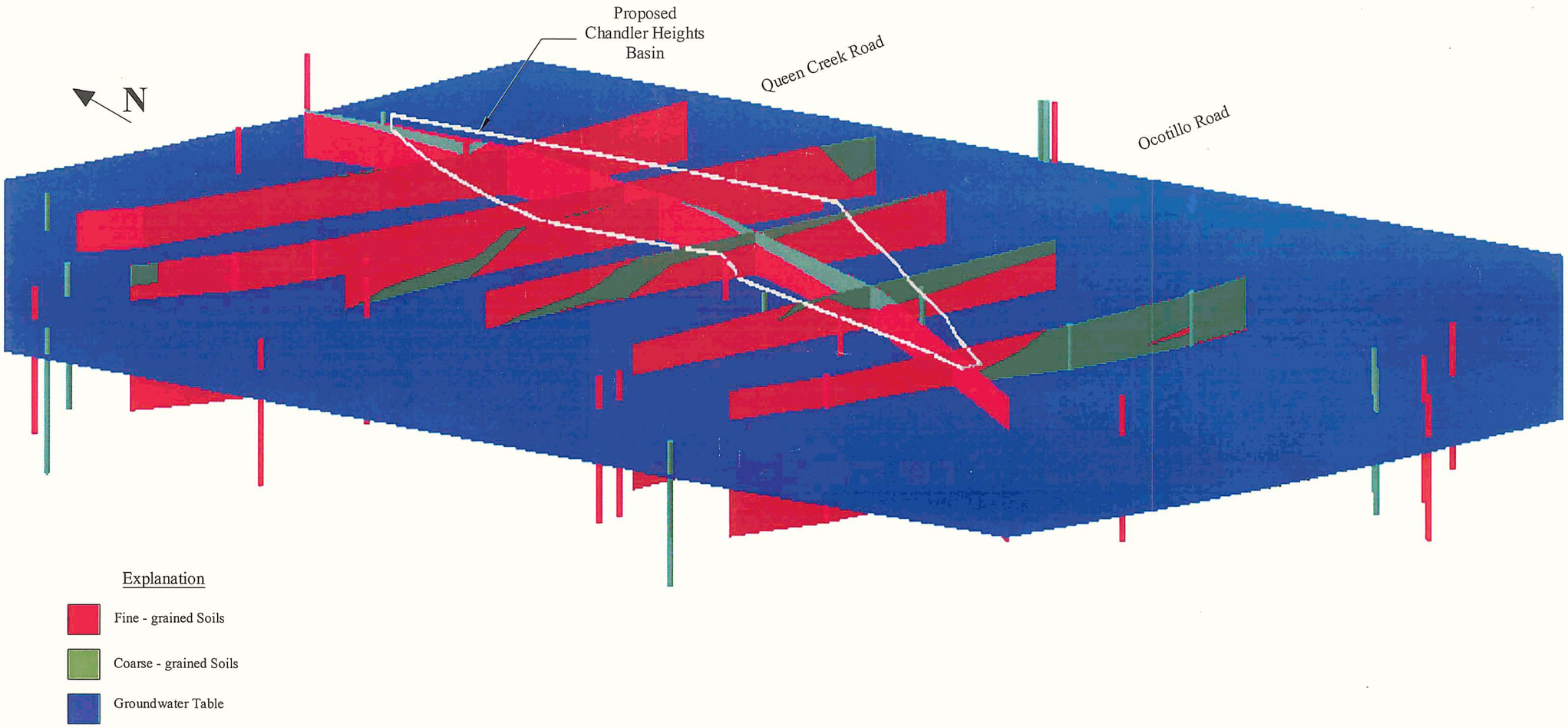
Concentration of Selected Inorganic Chemical Constituents in Groundwater

Figure 4



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Fence Diagram of Chandler Heights Basin Area
Based on Drillers Logs of Existing Wells

Figure 5
Source: ADWR, 2000b

Appendix A

GWSI
General Well Information
T2S, R6E, Sections 9-11, 14-16, 21-23, and 26-28

Local ID	Regulatory Number	Latitude Decimal	Longitude Decimal	Well Altitude	Site Use	Water Use	Well Depth
D-02-06 09ABB1		33.277222	111.745278	1290	U	U	195
D-02-06 09ABB2		33.276944	111.745278	1290	U	U	
D-02-06 09ABB3		33.276944	111.745	1290	W	H	
D-02-06 09ADD		33.270556	111.7375	1300	W	I	762
D-02-06 09CCC		33.263333	111.754167	1284	U	U	255
D-02-06 09DCC1		33.263333	111.745556	1292	Z	U	350
D-02-06 09DCC2		33.263333	111.745833	1294	U	U	800
D-02-06 09DDA		33.266667	111.7375	1290	W	I	600
D-02-06 10ACD		33.271944	111.725278	1310	W	H	595
D-02-06 10BBA		33.277222	111.733889	1300			904
D-02-06 10DCC		33.264167	111.7275	1312	U	U	
D-02-06 10DCD1		33.263611	111.725833	1315	W	I	
D-02-06 10DCD2	620531	33.263889	111.725556	1315	W	I	2730
D-02-06 11BAB1		33.277222	111.714444	1317	U	U	600
D-02-06 11BAB2		33.277222	111.714722	1321	U	U	900
D-02-06 11BCC1		33.270833	111.719444	1318	Z	U	301
D-02-06 11BCC2	620529	33.270278	111.719722	1318	W	I,P	1015
D-02-06 11DAD1		33.267778	111.702778	1332	U	U	650
D-02-06 11DAD2	618554	33.267778	111.7025	1332	U	U	
D-02-06 11DDD	618553	33.263611	111.702778	1333	U	U	400
D-02-06 14ADD1		33.256111	111.702778	1336	Z	U	354
D-02-06 14ADD2	612273	33.256389	111.702778	1336	W	I	1710
D-02-06 14BDD		33.256667	111.7125	1325			420
D-02-06 14DAA		33.255833	111.702778	1337	W	I	606
D-02-06 14DDA	612274	33.251389	111.7025	1335	W	I	1303
D-02-06 15BDD1	620533	33.256389	111.728889	1312	W	I	1000
D-02-06 15BDD2		33.256111	111.729167	1312	W	H	
D-02-06 16AAA		33.261944	111.738056	1304	U	U	
D-02-06 16AAD		33.259722	111.7375	1304	W	P	610
D-02-06 16ACC		33.256667	111.745	1298	U	U	
D-02-06 16BAA1	086951	33.2625	111.746944	1291	W	C	800
D-02-06 16BAA2		33.261667	111.746944	1292	W	N	
D-02-06 16DAA		33.254722	111.738333	1306	Z	U	
D-02-06 16DAB	620534	33.255	111.740833	1302	U	U	750
D-02-06 21BBB		33.247778	111.754167	1288	W	H	
D-02-06 21CAA1	610424	33.240833	111.746111	1296	U	U	790
D-02-06 21CAA2	800253	33.240556	111.7475	1296	W	H	585
D-02-06 21DCA		33.235556	111.741944	1300	W	P	610
D-02-06 22BAA	623411	33.248611	111.73	1310	W	I	781
D-02-06 22BCD1		33.241667	111.734722	1310	W	I	900
D-02-06 22BCD2		33.241667	111.733333	1312	*	*	1252
D-02-06 22BDD	609478	33.241667	111.728611	1313	W	I	1400
D-02-06 22CBA		33.240833	111.734167	1310	U	U	1970
D-02-06 22CCC		33.234444	111.735833	0	*	*	2060
D-02-06 22CDA1		33.238333	111.728611	1317	Z	U	400
D-02-06 22CDA2	609477	33.235556	111.728889	1317	U	U	730
D-02-06 23DDD	647066	33.233889	111.702778	1342	W	I	1138
D-02-06 26CDD	647146	33.219167	111.711111	1360	U	U	1197
D-02-06 26DDD		33.219444	111.702778	1372	U	U	717
D-02-06 27CDD1		33.219444	111.728611	1325	U	U	350
D-02-06 27CDD2	647145	33.219444	111.728611	1325	U	U	682

GWSI
General Well Information
T2S, R6E, Sections 9-11, 14-16, 21-23, and 26-28

Local ID	Regulatory Number	Latitude Decimal	Longitude Decimal	Well Altitude	Site Use	Water Use	Well Depth
D-02-06 27CDD3		33.219722	111.728611	1325	U	U	500
D-02-06 27DBC		33.223889	111.7275	1320			575
D-02-06 28AAA1		33.233333	111.738333	1310	U	U	606
D-02-06 28AAA2	622207	33.233333	111.737778	1309	U	U	
D-02-06 28ADD1		33.226944	111.738333	1305	Z	U	600
D-02-06 28ADD2	620540	33.226944	111.7375	1305	W	I	1675
D-02-06 28BBB		33.233056	111.753611	1290	Z	U	606

GWSI
Water Level Information
T2S, R6E, Sections 9-11, 14-16, 21-23, and 26-28

Local ID	Water Level Date	Water Level Depth	Water Level Elevation
D-02-06 09ADD	25-Mar-48	130	1170
D-02-06 09ADD	13-Jan-72	282	1018
D-02-06 09ADD	29-Dec-75	301.5	999
D-02-06 09ADD	29-Nov-82	254	1046
D-02-06 09ADD	05-Dec-84	216.5	1084
D-02-06 09ADD	12-Dec-97	130.4	1170
D-02-06 09CCC	07-Oct-39	90.5	1194
D-02-06 09CCC	09-Feb-45	106.38	1178
D-02-06 09CCC	24-Aug-46	112.18	1172
D-02-06 09DCC1	01-Oct-39	96	1196
D-02-06 09DCC1	19-Nov-82		
D-02-06 09DCC2	21-Nov-84	174.3	1120
D-02-06 09DCC2	12-Dec-97	108.5	1186
D-02-06 09DDA	17-Apr-48	130	1160
D-02-06 10DCC	01-Jan-40	110	1202
D-02-06 11BAB1	17-Apr-30	87	1230
D-02-06 11BAB1	13-Aug-31	87.75	1229
D-02-06 11BAB1	23-Feb-34	81.4	1236
D-02-06 11BAB1	01-Apr-36	86.5	1231
D-02-06 11BAB1	01-Dec-48	142	1175
D-02-06 11BAB2	15-Apr-65	354.6	966
D-02-06 11BAB2	23-Jul-65	386	935
D-02-06 11BAB2	20-Sep-65	394.7	926
D-02-06 11BAB2	20-Jan-66	363.2	958
D-02-06 11BAB2	20-Apr-66	363.4	958
D-02-06 11BAB2	20-May-66	374.1	947
D-02-06 11BAB2	20-Jun-66	384.5	937
D-02-06 11BAB2	18-Aug-66	389.9	931
D-02-06 11BAB2	21-Sep-66	358.2	963
D-02-06 11BAB2	21-Oct-66	325.36	996
D-02-06 11BAB2	22-Nov-66	297.53	1023
D-02-06 11BAB2	21-Dec-66	294.8	1026
D-02-06 11BAB2	20-Jan-67	296.16	1025
D-02-06 11BAB2	21-Feb-67	327.2	994
D-02-06 11BAB2	20-Mar-67	372.67	948
D-02-06 11BAB2	23-Mar-67	376.55	944
D-02-06 11BAB2	21-Apr-67	368.77	952
D-02-06 11BAB2	23-May-67	374.9	946
D-02-06 11BAB2	23-Jun-67	380.07	941
D-02-06 11BAB2	25-Jul-67	377.59	943
D-02-06 11BAB2	28-Aug-67	379.8	941
D-02-06 11BAB2	26-Sep-67	369.12	952
D-02-06 11BAB2	25-Oct-67	332.62	988
D-02-06 11BAB2	27-Nov-67	311.63	1009
D-02-06 11BAB2	26-Dec-67	281.3	1040
D-02-06 11BAB2	26-Jan-68	282.05	1039
D-02-06 11BAB2	26-Mar-68	305.55	1015
D-02-06 11BAB2	25-Apr-68	330.54	990
D-02-06 11BAB2	28-May-68	355.28	966
D-02-06 11BAB2	28-Jun-68	377.45	944
D-02-06 11BAB2	26-Jul-68	385.5	936
D-02-06 11BAB2	26-Aug-68	384.45	937

GWSI
Water Level Information
T2S, R6E, Sections 9-11, 14-16, 21-23, and 26-28

Local ID	Water Level Date	Water Level Depth	Water Level Elevation
D-02-06 11BAB2	25-Sep-68	377.5	944
D-02-06 11BAB2	25-Oct-68	331	990
D-02-06 11BAB2	27-Nov-68	292.98	1028
D-02-06 11BAB2	26-Dec-68	282.9	1038
D-02-06 11BAB2	29-Jan-69	264.65	1056
D-02-06 11BAB2	26-Feb-69	281.08	1040
D-02-06 11BAB2	25-Mar-69	337.45	984
D-02-06 11BAB2	28-Apr-69	365.08	956
D-02-06 11BAB2	26-May-69	369.9	951
D-02-06 11BAB2	26-Jun-69	375.95	945
D-02-06 11BAB2	25-Jul-69	370.92	950
D-02-06 11BAB2	26-Aug-69	381.25	940
D-02-06 11BAB2	26-Sep-69	363.52	957
D-02-06 11BAB2	24-Oct-69	345.99	975
D-02-06 11BAB2	26-Nov-69	298.4	1023
D-02-06 11BAB2	24-Dec-69	299.86	1021
D-02-06 11BAB2	26-Jan-70	298.9	1022
D-02-06 11BAB2	25-Feb-70	337.7	983
D-02-06 11BAB2	27-Mar-70	349.2	972
D-02-06 11BAB2	24-Apr-70	367.88	953
D-02-06 11BAB2	22-May-70	381.3	940
D-02-06 11BAB2	23-Jun-70	384.13	937
D-02-06 11BAB2	22-Jul-70	389.05	932
D-02-06 11BAB2	25-Aug-70	389.25	932
D-02-06 11BAB2	25-Sep-70	361.35	960
D-02-06 11BAB2	26-Oct-70	331	990
D-02-06 11BAB2	23-Nov-70	306.9	1014
D-02-06 11BAB2	21-Dec-70	302.49	1019
D-02-06 11BAB2	22-Jan-71	308.4	1013
D-02-06 11BAB2	23-Feb-71	331.15	990
D-02-06 11BAB2	23-Mar-71	363.1	958
D-02-06 11BAB2	23-Apr-71	365.47	956
D-02-06 11BAB2	24-May-71	380.43	941
D-02-06 11BAB2	29-Jun-71	389.21	932
D-02-06 11BAB2	26-Jul-71	385.9	935
D-02-06 11BAB2	03-Sep-71	390	931
D-02-06 11BAB2	29-Sep-71	350.05	971
D-02-06 11BAB2	15-Oct-71	330.9	990
D-02-06 11BAB2	16-Nov-71	319.3	1002
D-02-06 11BAB2	23-Dec-71	308.4	1013
D-02-06 11BAB2	20-Jan-72	312.5	1009
D-02-06 11BAB2	16-Feb-72	326.1	995
D-02-06 11BAB2	15-Mar-72	363.8	957
D-02-06 11BAB2	17-Apr-72	362.4	959
D-02-06 11BAB2	15-May-72	374	947
D-02-06 11BAB2	12-Jun-72	373	948
D-02-06 11BAB2	13-Jul-72	377.7	943
D-02-06 11BAB2	18-Aug-72	374.3	947
D-02-06 11BAB2	20-Sep-72	361.8	959
D-02-06 11BAB2	30-Oct-72	335.6	985
D-02-06 11BAB2	17-Nov-72	309.5	1012
D-02-06 11BAB2	23-Jan-73	285.2	1036

GWSI
Water Level Information
T2S, R6E, Sections 9-11, 14-16, 21-23, and 26-28

Local ID	Water Level Date	Water Level Depth	Water Level Elevation
D-02-06 11BAB2	27-Feb-73	290.6	1030
D-02-06 11BAB2	20-Mar-73	268.3	1053
D-02-06 11BAB2	20-Apr-73	309.1	1012
D-02-06 11BAB2	14-May-73	322.5	999
D-02-06 11BAB2	21-Jun-73	344.7	976
D-02-06 11BAB2	18-Jul-73	360.8	960
D-02-06 11BAB2	21-Aug-73	371.7	949
D-02-06 11BAB2	19-Sep-73	404.9	916
D-02-06 11BAB2	23-Oct-73	340	981
D-02-06 11BAB2	20-Nov-73	317.4	1004
D-02-06 11BAB2	15-Dec-73	306	1015
D-02-06 11BAB2	17-Jan-74	284.9	1036
D-02-06 11BAB2	23-Dec-74	305.3	1016
D-02-06 11BAB2	07-Jan-76	332.2	989
D-02-06 11BAB2	29-Dec-76	308.2	1013
D-02-06 11BAB2	27-Dec-77	297.2	1024
D-02-06 11BAB2	26-Jan-78	278.3	1043
D-02-06 11BAB2	24-Jan-79	260.4	1061
D-02-06 11BAB2	23-Jan-80	265.4	1056
D-02-06 11BAB2	25-Jan-81	287.1	1034
D-02-06 11BAB2	13-Jan-82	242.2	1079
D-02-06 11BAB2	05-Dec-84	214.7	1106
D-02-06 11BCC2	02-Dec-82	277	1041
D-02-06 11BCC2	20-Nov-84	274.6	1043
D-02-06 11BCC2	14-Jun-89	409.1	909
D-02-06 11BCC2	18-Dec-89	280.1	1038
D-02-06 11BCC2	13-Nov-91	262.9	1055
D-02-06 11BCC2	15-Aug-94	338.1	980
D-02-06 11BCC2	11-Dec-97	200.7	1117
D-02-06 11DAD1	07-Nov-91	258.6	1073
D-02-06 11DAD1	08-Jan-98	222	1110
D-02-06 11DAD2	07-Nov-91	260.3	1072
D-02-06 11DAD2	08-Jan-98	226.4	1106
D-02-06 11DDD	13-Jan-72	323.5	1010
D-02-06 11DDD	07-Jan-76	356.5	977
D-02-06 11DDD	29-Nov-82	293.9	1039
D-02-06 11DDD	05-Dec-84	277.2	1056
D-02-06 11DDD	05-Apr-88	344.2	989
D-02-06 11DDD	29-Nov-88	260.6	1072
D-02-06 11DDD	08-Jun-95	322	1011
D-02-06 14ADD1	01-Jan-27	80	1256
D-02-06 14DAA	01-Nov-39	118.82	1218
D-02-06 14DDA	01-Feb-64	295	1040
D-02-06 14DDA	01-Dec-82	308.8	1026
D-02-06 14DDA	05-Dec-84	283.3	1052
D-02-06 15BDD1	18-Dec-97	156.9	1155
D-02-06 15BDD2	12-Dec-97	156.1	1156
D-02-06 16AAA	16-Oct-39	101.73	1202
D-02-06 16AAA	02-Nov-39	101.03	1203
D-02-06 16AAA	20-Dec-39	100.07	1204
D-02-06 16ACC	16-Oct-39	96.1	1202
D-02-06 16ACC	02-Nov-39	95.93	1202

GWSI
Water Level Information
T2S, R6E, Sections 9-11, 14-16, 21-23, and 26-28

Local ID	Water Level Date	Water Level Depth	Water Level Elevation
D-02-06 16ACC	21-Dec-39	96	1202
D-02-06 16ACC	15-Jan-40	95.86	1202
D-02-06 16ACC	22-Feb-40	95.94	1202
D-02-06 16ACC	03-Apr-40	97.53	1200
D-02-06 16DAA	16-Oct-39	100.5	1206
D-02-06 16DAA	02-Nov-39	100.5	1206
D-02-06 16DAA	21-Dec-39	100.75	1205
D-02-06 16DAA	25-Mar-76		
D-02-06 16DAB	02-Dec-82	201.3	1101
D-02-06 16DAB	16-Jun-89	475.6	826
D-02-06 16DAB	06-Dec-89	217.8	1084
D-02-06 16DAB	18-Dec-89	174	1128
D-02-06 16DAB	03-Jan-91	168.1	1134
D-02-06 16DAB	13-Nov-91	173.3	1129
D-02-06 16DAB	17-Nov-92	161.2	1141
D-02-06 16DAB	30-Nov-93	149.86	1152
D-02-06 16DAB	17-Nov-94	146.8	1155
D-02-06 16DAB	13-Nov-95	141.3	1161
D-02-06 16DAB	26-Nov-96	139.9	1162
D-02-06 16DAB	15-Dec-97	126.4	1176
D-02-06 16DAB	17-Nov-98	127.3	1175
D-02-06 16DAB	02-Nov-99		
D-02-06 21CAA1	01-Jan-51	124	1172
D-02-06 21CAA1	02-Dec-82	209.1	1087
D-02-06 21CAA1	15-Dec-97	92.3	1204
D-02-06 21CAA2	05-Dec-89	251.3	1045
D-02-06 21CAA2	03-Jan-91	238.4	1058
D-02-06 21CAA2	08-Nov-91	254	1042
D-02-06 21CAA2	16-Nov-92	227.4	1069
D-02-06 21CAA2	19-Nov-93	212.1	1084
D-02-06 21CAA2	11-Nov-94	228	1068
D-02-06 21CAA2	13-Nov-95	224.6	1071
D-02-06 21CAA2	26-Nov-96	218.2	1078
D-02-06 21CAA2	15-Dec-97	194.2	1102
D-02-06 21CAA2	17-Nov-98	193.6	1102
D-02-06 21CAA2	02-Nov-99	191.7	1104
D-02-06 21DCA	06-Apr-71	190	1110
D-02-06 22BCD1	20-Oct-53	249	1061
D-02-06 22BDD	01-Mar-71	384	929
D-02-06 22BDD	15-Nov-82	291.1	1022
D-02-06 22BDD	04-Dec-84	274.1	1039
D-02-06 22BDD	05-Dec-89	266.2	1047
D-02-06 22BDD	03-Jan-91	252.4	1061
D-02-06 22BDD	13-Nov-91	265.1	1048
D-02-06 22BDD	16-Nov-92	241.7	1071
D-02-06 22BDD	19-Nov-93	224.9	1088
D-02-06 22BDD	11-Nov-94	241.1	1072
D-02-06 22BDD	13-Nov-95	233.3	1080
D-02-06 22BDD	26-Nov-96	228.1	1085
D-02-06 22BDD	15-Dec-97	199.4	1114
D-02-06 22BDD	17-Nov-98	194.4	1119
D-02-06 22BDD	02-Nov-99	191.6	1121

GWSI
Water Level Information
T2S, R6E, Sections 9-11, 14-16, 21-23, and 26-28

Local ID	Water Level Date	Water Level Depth	Water Level Elevation
D-02-06 22CDA2	02-Dec-82	254.5	1063
D-02-06 22CDA2	04-Dec-84	227.1	1090
D-02-06 23DDD	09-Jan-98	284.7	1057
D-02-06 26CDD	07-Oct-84	637.8	722
D-02-06 26CDD	03-Dec-84	369.1	991
D-02-06 26CDD	14-Nov-91	227.2	1133
D-02-06 26CDD	09-Jan-98	198.6	1161
D-02-06 26DDD	07-Jan-49	151.1	1221
D-02-06 26DDD	30-Dec-71	350.1	1022
D-02-06 26DDD	29-Jan-73	322.5	1050
D-02-06 26DDD	09-Jan-74	343	1029
D-02-06 26DDD	26-Dec-74	362	1010
D-02-06 26DDD	06-Jan-76	210	1162
D-02-06 26DDD	03-Jan-77	290.6	1081
D-02-06 26DDD	05-Jan-78	304.1	1068
D-02-06 26DDD	11-Jan-79	255.7	1116
D-02-06 26DDD	27-Jan-80	215.5	1157
D-02-06 26DDD	04-Feb-81	207.8	1164
D-02-06 26DDD	19-May-81	208.5	1164
D-02-06 26DDD	13-Jan-82	203.7	1168
D-02-06 26DDD	01-Dec-82	203.1	1169
D-02-06 26DDD	07-Dec-83	197.4	1175
D-02-06 26DDD	03-Dec-84	197.3	1175
D-02-06 26DDD	27-Jun-85	197.9	1174
D-02-06 26DDD	13-Dec-85	196.2	1176
D-02-06 26DDD	27-May-86	194.1	1178
D-02-06 26DDD	01-Dec-86	193	1179
D-02-06 26DDD	04-Jun-87	193.5	1179
D-02-06 26DDD	14-Dec-87	192.5	1180
D-02-06 26DDD	10-Jun-88	192.7	1179
D-02-06 26DDD	12-Dec-88	191.7	1180
D-02-06 26DDD	05-Dec-89	190.3	1182
D-02-06 26DDD	03-Jan-91	186.5	1186
D-02-06 26DDD	14-Nov-91	187.6	1184
D-02-06 26DDD	17-Nov-92	185.2	1187
D-02-06 26DDD	19-Nov-93	182.6	1189
D-02-06 26DDD	01-Dec-94	180	1192
D-02-06 26DDD	13-Nov-95	178.7	1193
D-02-06 26DDD	26-Nov-96	174.7	1197
D-02-06 26DDD	15-Dec-97	171.1	1201
D-02-06 26DDD	17-Nov-98	170.8	1201
D-02-06 26DDD	01-Nov-99	168	1204
D-02-06 27CDD1	10-Oct-39	93.6	1231
D-02-06 27CDD1	18-Dec-39	88.5	1237
D-02-06 27CDD1	23-Apr-43	108.83	1216
D-02-06 27CDD1	03-Dec-48	137.7	1187
D-02-06 27CDD1	08-Feb-85	155.6	1169
D-02-06 27CDD1	14-Nov-91	148.6	1176
D-02-06 27CDD1	09-Jan-98	129.5	1196
D-02-06 27CDD2	08-Feb-85	269.6	1055
D-02-06 27CDD2	14-Nov-91	257.5	1068
D-02-06 27CDD2	09-Jan-98	165	1160

GWSI
Water Level Information
T2S, R6E, Sections 9-11, 14-16, 21-23, and 26-28

Local ID	Water Level Date	Water Level Depth	Water Level Elevation
D-02-06 27CDD3	06-Jan-76	349.2	976
D-02-06 28AAA1	01-Feb-53	133	1177
D-02-06 28AAA1	18-Feb-55	132.84	1177
D-02-06 28AAA1	09-Jan-59	130.63	1179
D-02-06 28AAA1	17-Feb-60	129.81	1180
D-02-06 28AAA1	22-Jan-63	128.15	1182
D-02-06 28AAA1	09-Jan-64	245.72	1064
D-02-06 28AAA1	09-Feb-65	232.95	1077
D-02-06 28AAA2	18-Jan-67	323.2	986
D-02-06 28AAA2	09-Feb-70	329.3	980
D-02-06 28AAA2	22-Jan-71	316.9	992
D-02-06 28AAA2	30-Dec-71	203.7	1105
D-02-06 28AAA2	29-Jan-73	180.6	1128
D-02-06 28AAA2	09-Jan-74	173	1136
D-02-06 28AAA2	06-Jan-76	243.6	1065
D-02-06 28AAA2	30-Jan-77	229.4	1080
D-02-06 28AAA2	20-Feb-85	182.7	1126
D-02-06 28AAA2	14-Nov-91	160.1	1149
D-02-06 28AAA2	12-Dec-97	115.7	1193
D-02-06 28ADD1	13-Dec-63	241.1	1064
D-02-06 28ADD2	13-Nov-91	420.6	884
D-02-06 28ADD2	15-Aug-94	321.8	983
D-02-06 28ADD2	12-Dec-97	281.7	1023

Appendix B

FORM W-3 2M 12-47 JAHN-TYLER

LAND DEPARTMENT
WATER DIVISION
STATE OF ARIZONA

REPORT OF WELL DRILLER

Report of Well Driller is required to be made and filed with the State Land Commissioner as required by Section 7, Chapter 12, Senate Bill No. 3, Seventeenth Legislature, First Special Session, 1945. A separate report shall be made for each well and filed within 30 days after completion of the well.

1. Owner D. L. Hadley
Name
Chandler ~~Higley~~, Arizona.
Address

RECEIVED APR 2 1948 State Land Dept.

2. Lessee or Operator
Name
Address

3. Driller Roscoe Moss Company (Driller J. E. Cunningham)
Name
4360 Worth Street., Los Angeles, California.
Address

4. Location of well: Twp. 2 South Rge. 6 East Section 9 Maricipa County, Arizona.
SE¼ SE¼ NE¼
10-acre subdivision

DESCRIPTION OF WELL

5. Total depth of hole 762 ft.
6. Type of casing Hard Red Steel
7. Diameter and length of casing 20 in. from 0 to 762 in. from to in. from to
8. Method of sealing at reduction points Not Reduced.
9. Perforated from 140 to 748 from to from to from to
10. Size of cuts 1/2 x 4½ Number cuts per foot 8 per 12 inches
11. If screen was installed: Length ft. Diam. in. Type None.
12. Method of construction Drilled California Type Cable Tool.
drilled, dug, driven, bored, jetted, etc.
13. Date completed March 25, 1948
Month Year
14. Depth to water 130 ft.
If flowing well, so state.
15. Describe point from which depth measurements were made, and give sea-level elevation if available Ground Surface.
16. If flowing well, state method of flow regulation Not flowing.

DISCHARGE DATA

Not Tested.

17. Well discharge
gal. per min. or cu. ft. per sec. or miner's inches.
18. Method of discharge measurement Weir
weir, orifice, current meter, etc.
19. Drawdown ft.
20. Purpose of use irrigation purposes
21. Place of use: Twp. 2 Rge. 7 Section 9 Acres 480
(See 22) Legal subdivision
22. Purpose of use
Twp. Rge. Section Acres
Legal subdivision
22. If well is part of irrigation system of Irrigation District, Association or Company, omit 23 and give name of project.
private pump in R.W.C.D
Name of Project

(D-2-6)9 add

EQUIPMENT DATA

Johnson pump Turbine
23. Kind of pump 18E 7 Stage 700 GPM 1160 RPM
turbine, centrifugal, etc.
24. Kind of power Natural Gas
electric, natural gas, etc.
25. Horsepower rating of motor 280 Continuous

DO NOT WRITE IN THIS SPACE
OFFICE RECORD
Received 7-9-49 by ld
Filed 7-12-49 by ld
File No. (D-2-6)9 add
Cross-referenced (Name) by
Cross-referenced (Basin) by
Cross-referenced by

(See Other Side)

FORM W-3 2M 2-47 JAHN-TYLER Appl. S-164 (D-2-6)9dcc Permit # S-155

Land Department
Ground Water Division
State of Arizona

REPORT OF WELL DRILLED IN CRITICAL AREA

Report of Well Drilled in Critical Area is required to be made and filed with the State Land Commissioner upon completion of the construction of such well, pursuant to Section 10, Chapter 5, House Bill No. 2, Eighteenth Legislature, Sixth Special Session, 1948.

1. Owner ROOSEVELT WATER CONSERVATION DISTRICT
Name
Higley, Arizona.
Address
2. Lessee or Operator
Name
Address
3. Driller ?
Name
Address
4. Location of Well: Twp. 2 S Rge. 6 E Sec. 9 Legal Subdivision SW¼ SW¼ SE¼
10 acre subdivision
It is yards direction from the nearest irrigation well.
If less than ¼ mile
5. Purpose of use IRRIGATION
6. Place of use: Twp. Rge. Section(s) Legal Subdivision Acres
7. If well is part of Irrigation District, Association, or Company, omit 6 and give name of project:
ROOSEVELT WATER CONSERVATION DISTRICT LANDS

DESCRIPTION OF WELL

8. Total depth of hole 800 ft. 9. Type of Casing Hard red steel
10. Diameter and length of casing: 20 in. from 0 ft. to 800 ft; in. from ft. to ft;
in. from ft. to ft; in. from ft. to ft.
11. Method of sealing at reduction points
12. Perforated from 250 ft. to 785 ft; from ft. to ft; from ft. to ft.
13. Size of cuts 5/8 x 6-1/2 Number of cuts per foot 10 per 12 inches
14. If screen was installed: Length ft. Diameter in. Type
15. Method of drilling Drilled California Type Cable Tool
drilled, dug, driven, bored, etc.
16. Date completed April 16, 1953. 17. Depth to water 160 ft.
Month Year (If flowing well, so state)
18. Describe points from which depth measurements were made Ground Surface
Sea-level elevation 1292.5
(If available)
19. Method of flow regulation if flowing well

REPORT OF PUMP INSTALLATION AND TEST

20. Tested well capacity 2531 Method of measurement Weir
gallons per minute weir, orifice, meter, etc.
21. Depth immediately prior to capacity test, from land surface to static water level 162 ft.
22. Non-flowing well: Drawdown 162 ft. measured after 48 hours of continuous operation, and while pump is still operating.
(at least 4)
23. Flowing well: Shut-in pressure ft. above the land surface, or pounds per square inch at the land surface.
24. Kind of Pump Turbine
turbine, centrifugal, etc.
25. Kind of Power Electric
electric, natural gas, etc.
26. Horsepower Rating of Motor 300
27. Permittee Oakley Jones
Signature

DO NOT WRITE IN THIS SPACE
OFFICE RECORD
Received 6-3-53 by en
Filed 6-3-53 by en
File No. (D-2-6) 9 dcc
Cross-referenced (Name) by
Cross-referenced (Basin) by
Cross-referenced by

Fee of $3.00 must be paid with filing of application

Applicant Secretary
Higley, Arizona
(Post Office Address)

FORM W-2 3M 12-47 JAHN-TYLER

LAND DEPARTMENT
WATER DIVISION
STATE OF ARIZONA

REPORT OF WELL DRILLER

Report of Well Driller is required to be made and filed with the State Land Commissioner as required by Section 7, Chapter 12, Senate Bill No. 1, Seventeenth Legislature, First Special Session, 1945. A separate report shall be made for each well and filed within 30 days after completion of the well.

1. Owner: Wiley Herman
Name
Higley Arizona
Address

2. Lessee or Operator
Name
Address

3. Driller: J. C. Click
Name
P.O. Box 736 Mesa, Arizona
Address

4. Location of well: Twp. 2 S Rge. 6 E Section 9 NW ¼ SE ¼ ¼
One mile west & three miles south of
10-acre subdivision

DESCRIPTION OF WELL Higley, Arizona

5. Total depth of hole 600 ft.

6. Type of casing Stove-pipe casing

7. Diameter and length of casing 20 in. from to in. from to in. from to

8. Method of sealing at reduction points

9. Perforated from 580 to 208 ft. from to from to from to

10. Size of cuts ⅛" x 4½" Number cuts per foot

11. If screen was installed: Length ft. Diam. in. Type

12. Method of construction Drilled
drilled, dug, driven, bored, jetted, etc.

13. Date completed April 1948
Month Year

14. Depth to water 188 ft.
If flowing well, so state.

15. Describe point from which depth measurements were made, and give sea-level elevation if available Top of casing

16. If flowing well, state method of flow regulation

DISCHARGE DATA

17. Well discharge
gal. per min. or cu. ft. per sec. or miner's inches.

18. Method of discharge measurement
weir, orifice, current meter, etc.

19. Drawdown ft.

20. Purpose of use Irrigation

21. Place of use: Twp. Rge. Section Acres
(See 22) Legal subdivision

22. Purpose of use
Twp. Rge. Section Acres
Legal subdivision

22. If well is part of irrigation system of Irrigation District, Association or Company, omit 23 and give name of project.

Name of Project

EQUIPMENT DATA

23. Kind of pump
turbine, centrifugal, etc.

24. Kind of power
electric, natural gas, etc.

25. Horsepower rating of motor

DO NOT WRITE IN THIS SPACE
OFFICE RECORD

Received 4-19-49 by ld
Filed 5-7-49 by ld
File No. (D-2-6)9 dda
Cross-referenced (Name) by
Cross-referenced (Basin) by
Cross-referenced by

(See Other Side)

LAND DEPARTMENT
WATER DIVISION
STATE OF ARIZONA

REPORT OF WELL DRILLER

This report should be prepared by the driller in all detail and filed with the State Land Commissioner following completion of the well.

1. OWNER RONALD J LINES
Name
PO BOX 172, HIGLEY AZ 85236
Address

2. Lessee or Operator
Name
Address

3. DRILLER D & R Drilling
Name
PO BOX 828 QUEEN CREEK AZ 85242
Address

4. Location of well: Twp. 2 South Rge. 6 East Section 10 SE ¼ SW ¼ NE ¼
10-acre subdivision

5. Intention to Drill File No. D(2-6) 10 acd Permit No.

DESCRIPTION OF WELL

6. Total depth of hole 595 ft.

7. Type of casing 8" 2.50 Wall (0'to 586'Casing)

8. Diameter and length of casing 250 in. from 0 to 586 in. from to in. from to

9. Method of sealing at reduction points

10. Perforated from to from to from to from to

11. Size of cuts Number of cuts per foot

12. If screen was installed: Length ft. Diam. in. Type

13. Method of construction Drilled
drilled, dug, driven, bored, jetted, etc.

14. Date started Sep 28,1973
Month Day Year

15. Date completed Oct 18,1973
Month Day Year

16. Depth of water XXXXX 200 ft.
If flowing well, so state.

17. Describe point from which depth measurements were made, and give sea-level elevation if available
Ground level

18. If flowing well, state method of flow regulation No.

19. REMARKS:

DO NOT WRITE IN THIS SPACE
OFFICE RECORD

Received 12-10-73 by fmr
Filed by
File No. D(2-6)10 acd

(Well Log to Appear on Reverse Side)

WD FORM 6-2001
REV. 4-27-63

D(2-6)10bba

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL		
0	3	Top soil 1		
3	50	Sandy clay 3		
50	65	Sand trace of clay 1	3	
65	125	Sandy clay, layers hard shell 3		
125	155	Sand & gravel trace of clay 1		
155	165	Sandy clay 3		
165	221	Sand 1	1	
221	226	Sandy clay 3		
226	260	Sand & gravel cmented layers 2	2	
260	272	Gravel cemented layers 2		
272	343	Sandy clay cmented layers 3		
343	441	Sandy clay gravel cmented layers 3		
441	452	Sandy clay 3		3
452	469	Sand gravel boulders trace of clay 1		
469	430	Sandy clay 3	3	
480	502	Sand trace of clay 1		
502	604	Sandy clay 3		
604	607	Cemented gravel 2		
607	624	Sand & gravel 1	1	
624	665	Sand trace of clay 1		
665	677	Clay & sand layers 2		
677	694	Clay with gravel 2	2	
694	710	Sand & gravel 1		
710	728	Sticky clay 3		
728	740	Clay trace of gravel 3	3	
740	785	Red clay 3		
785	830	Sand gravel & clay 2	2	
830	835	Sandy clay 3		
835	860	Clay trace of gravel 3		
860	876	Blue clay 3	3	
876	880	Sand gravel & clay 2		
880	904	Sandy clay & hard shale 3		

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Bert E. Perry
Name

6336E Apache Dr. Mesa
Address

Date Aug 1, 1972

RECEIVED AUG 21 1972 STATE LAND DEPT.

960

LOG OF WELL D(2-6)10dda

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is sian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL		
0	18	Silt		
18	60	Sand & Gravel 1	1	
60	240	Silty clay & sand streaks 3		
240	245	Quick sand, ~~XXXXXXXXXXXXXXX~~ first water 1		
245	290	Silty sand 1	1	
290	295	Gravel 1		
295	450	Silty sand & clay streaks 2	2	
450	459	Cemented sand 2		
459	485	Clay 3		
485	530	Clay with cemented sand streaks 3		3
530	555	Clay with cemented sand streaks 3	3	
555	595	Clay 3		
595	602	Sand & gravel 1		
602	605	Clay 3		

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Frank L. Moore
Name

FL MOORE DRILLING CO
814 East Oakland
Chandler Az 85224
Address

Date 24 July 1975

2-73

RECEIVED JUL 28 1975 STATE LAND DEPT.

(D-2-6) 10 dad

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL		
0	10	Sand - Top Soil	1	
10	60	Sandy Clay	3	
60	180	Sandy Clay	3	
180	185	Sandy gravel	1	3
185	230	Sandy Clay	3	
230	245	Silky Sand - Top water	1	
245	280	Sandy Clay	3	
280	290	Sand with gravel	1	
290	300	gravel	1	1
300	315	Clay + gravel	2	
315	400	Clay	3	
400	430	Clay	3	
430	490	Sandy clay	3	
490	510	Silky clay	3	3
510	525	Clay with gravel	3	
525	550	Clay	3	
550	*560	Sandy Clay	3	
560	590	Sandy Clay	3	
590	600	Sandy Clay	3	
600	640	Clay	3	
640	650	Silky Clay	3	
650	690	Clay	3	
690	780	Clay	3	
780	790	Gravel	1	
790	795	Clay	3	

I hereby certify that I have read the foregoing statements, and that each and all of the items therein contained are true to the best of my knowledge and belief.

Bill Fenton
Owner, Operator or Driller

7230 E. McDowell Mesa
Address

Date 7-30-1967

RECEIVED AUG 2 1967 STATE LAND DEPT.

11361

LOG OF WELL

(D-2-6) 11 bab

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encounterd, and depth to which it rose in well.

From (feet)	To (feet)	Description of formation material	
0	6	Sandy Top Soil	
6	87	Clay	3
87	170	Sandy Clay	
170	260	Clay w/ Layers of Broken Rock	2

I hereby certify that this well was drilled by me (or under my supervision), and that each and all statements herein contained are true to the best of my knowledge and belief.

Driller Name: Max Blair

Box 4858
Street

Huachuca City Az 85616
City State Zip

Date 11/3/92

Well No. (D-2-6) 11bcc

Latitude-longitude 33 d, 16 m, 15 s (N) (S) 111 d, 43 m, 10 s

HYDROGEOLOGIC CARD

SAME AS ON MASTER CARD (1–19) Physiographic Province: BASIN & RANGE (20–21) 22 Section: SONORAN

DESERT (22) B Drainage Basin: (23–25) Subbasin: (26)

Topo of well site: (D) depression, (C) stream channel, (E) dunes, (F) flat, (H) hilltop, (K) sink, (L) swamp, (B) offshore, (P) pediment, (S) hillside, (T) terrace, (U) undulating, (V) valley flat (27)

MAJOR AQUIFER: system, series (28–29) aquifer, formation, group (30–31)

Lithology: (32–33) Origin: (34) Aquifer Thickness: ft

(35–37) Length of well open to: ft (38–40) Depth to top of: ft (41–43)

MINOR AQUIFER: system, series (44–45) aquifer, formation, group (46–47)

Lithology: (48–49) Origin: (50) Aquifer Thickness: ft

(51–53) Length of well open to: ft (54–56) Depth to top of: ft (57–59)

Intervals Screened:

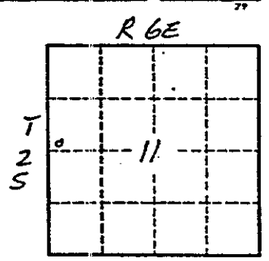
Depth to consolidated rock: ft (60–63) Source of data: (64)

Depth to basement: ft (65–68) Source of data: (69)

Surficial material: (70–71) Infiltration characteristics: (72)

Coefficient Trans: gpd/ft (73–75) Coefficient Storage: (76–78)

Coefficient Perm: gpd/ft^2; Spec cap: gpm/ft; Number of geologic cards: (79)



Well No. (D-2-6) 11bcc

GPO 869-201

Roosevelt Water Conservation District
(D-2-6) 11bcc Higley, Ariz.

0 – 7	Soil
7 – 17	Hardpan with strata of caliche
17 – 36	Dry sand and gravel
36 – 60	Caliche
60 – 74	Dry sand and gravel
74 – 100	Clay and caliche with strata of sand and gravel
100 – 107	Sand and gravel
107 – 136	Clay and caliche
136 – 143	Sand and gravel
143 – 149	Clay
149 – 160	Sand
160 – 162	Hard cement
162 – 206	Clay and caliche
206 – 220	Hard caliche
220 – 301	Clay and caliche
301 – 354	Clay
356 – 363	Sand
363 – 377	Clay
377 – 386	Gravel
386 – 465	Clay
465 – 475	Gravel
475 – 490	Clay
490 – 515	Coarse sand
515 – 700	Clay

"20" Casing 0 – 301
16" Casing 301 – 535
Open hole 535 – 700

(D-2-6)14 add

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL
0	30	surface silt and sand 1
30	261	pea gravel and clay 2
261	567	clay with streaks of fine sand and gravel 3
567	780	clay, gravel, streaks of sand 3
780	820	streaks of boulders with good gravel, small streaks of clay 1
820	940	clay with streaks of sand 3
940	960	clay 3
960	1320	clay with streaks of sand 3
1320	1360	gravel with streaks of clay 1
1360	1400	clay 3
1400	1510	clay with streaks of sand and good gravel 3
1510	1610	clay with streaks of sand 3
1610	1710	mostly clay 3

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller CAMERON & CLAYTON
Name

P.O. Box 746, Buckeye, Ariz.
Address

Date April 26, 1968

U.S. Geological Survey Water Resources Division

WELL-CUTTINGS ANALYSIS FORM

Page 1 of 5

Location (D-2-6) 14

LAB # 2898

Owner: KENNETH T. EARL Driller: CAMERON Depth: 1710

Method drilled: R. ROTARY

Depth	Pan No.	Munsell color Dry	Munsell color Wet	%Fines	%Sand	% Gravel	% Mud-stone	HCL reaction*	Max. size	Remarks
100	B 5	7.5YR 7/2 (LGRAY)	7.5YR 5/4 (BROWN)	4(24)	16	80(61)	20	M	3/4	100'-260' SAND COLOR: LIGHT BROWN
160	8	"	"	4(19)	7	89(74)	15	M	7/8	MUCH GRAVEL
200	10	"	"	9(24)	14	77(62)	15	M	7/8	50% QUARTZITE 50% VOLCANICS
220	11	"	"	10(20)	22	68(58)	10	M	3/4	SOME MUDSTONE, FREE QUARTZ
240	12	"	"	7(22)	10	83(68)	15	M	1	ANGULAR TO ROUNDED
260	13	"	"	12(22)	16	72(52)	10	M	3/4	SAND: FINE TO COARSE
280	14	"	"	25(30)	31	44(39)	5	M	3/4	BREAK 280'-1020'
300	15	"	"	23(33)	28	49(39)	10	M	3/4	SAND COLOR: PINKISH BROWN
320	16	"	"	15(20)	19	66(61)	5	M	3/4	SOME GRAVEL
340	17	"	"	19(24)	29	52(47)	5	M	7/8	60% VOLCANICS
360	18	"	"	20(30)	33	47(37)	10	M	5/8	30% QUARTZITE
380	19	"	"	19(29)	35	46(36)	10	M	3/4	10% FREE QUARTZ
400	20	"	"	31(51)	29	40(20)	20	M	3/4	MUCH MUDSTONE
420	21	"	"	39(44)	30	31(21)	10	M	5/8	ANGULARITY: SUB-ANGULAR
440	22	"	"	33(38)	36	37(32)	5	M	1/2	TO SUB-ROUNDED
460	23	"	"	48(44)	32	28(23)	5	M	5/8	SIZE: FINE TO MEDIUM
480	24	"	"	33(38)	37	30(25)	5	M	3/8	
500	25	"	"	27(32)	44	29(21)	5	M	1/2	
520	26	"	"	44(119)	36	20(15)	5	M	1/2	

*S-strong; M-moderate; W-weak; O-none

ANAL. BY J.L. DATE 7-12-71

Page 3 of 5

U.S. Geological Survey Water Resources Division

WELL-CUTTINGS ANALYSIS FORM

Location

LAB # 2898

Owner: Driller: Depth: Method drilled:

Depth	Pan No.	Munsell color Dry	Munsell color Wet	%Fines	%Sand	% Gravel	% Mud-stone	HCL reaction*	Max. size	Remarks
920	B46	"	"	1 55(57)	37	8(6)	2	M	1/2	2
940	47	"	"	1 36(39)	47	17(14)	3	M	1/2	2
960	48	"	"	1 37(41)	44	19(15)	4	M	1/2	2
980	49	"	"	1 23(27)	41	36(32)	4	M	1/2	1
1000	50	"	"	1 29(31)	60	11(9)	2	M	1/2	1
1020	51	"	"	1 34(37)	52	14(11)	3	M	3/8	1
1040	52	"	"	1 38(46)	52	10(8)	2	M	3/8	BREAK → 1040'-1280' 2
1060	53	"	"	1 47(48)	49	4(3)	1	M	1/4	SAND COLOR: GRAY BROWN
1080	54	"	"	1 57(59)	36	7(5)	2	M	1/2	LITTLE GRAVEL
1100	55	"	"	1 62(64)	32	6(4)	2	M	3/8	75% QUARTZITE
1120	56	"	"	1 51(56)	37	12(7)	5	M	1/4	25% VOLCANICS
1140	57	"	"	1 67(68)	30	3(2)	1	M	1/2	SOME MUDSTONE
1160	58	"	"	1 67(68)	30	3(2)	1	M	3/8	SOME FREE QUARTZ
1180	59	"	"	1 77(78)	21	2(1)	1	M	1/4	ANGULARITY: SUB-
1200	60	"	"	1 54(60)	34	12(6)	6	M	3/8	ANGULAR TO SUB-ROUNDED
1220	61	"	"	1 63(64)	33	4(3)	1	M	3/4	SAND: FINE TO MEDIUM
1240	62	"	"	1 72(73)	25	3(2)	1	M	1/4	
1260	63	"	"	1 61(62)	35	4(3)	1	M	1/4	
1280	64	"	"	1 67(68)	31	2(1)	1	M	3/8	

ANAL. BY DATE

*S-strong; M-moderate; W-weak; O-none

BREAK →

971 in 14414 1010'

Page 5 of 5

U.S. Geological Survey Water Resources Division

WELL-CUTTINGS ANALYSIS FORM

Location (D-26) 14

LAB # 2898

Owner: Driller: Depth: Method drilled:

Depth	Pan No.	Munsell color Dry	Munsell color Wet	%Fines	%Sand	% Gravel	% Mud-stone	HCL reaction*	Max. size	Remarks
1680	B84	"	"	1 52(53)	39	9(8)	1	M	1/2	
1700	85	"	"	1 53(54)	38	9(8)	1	M	3/8	

ANAL. BY DATE

*S-strong; M-moderate; W-weak; O-none

(D-2-6)14daa

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

From (feet)	To (feet)	Description of formation material		
0	11	Silt	3	
11	70	Clay and caliche	3	3
70	145	Cemented clay and gravel	3	
145	157	Gravel 2"	1	
157	170	Sandy clay	3	2
170	195	Clay and gravel 1"	2	
195	214	Sandy clay	3	
214	230	Caliche and clay	3	3
230	304	Clay and gravel 2"	2	
304	340	Sandy clay	3	
340	415	Clay and caliche	3	
415	503	Sandy clay	3	
503	514	Clay and gravel to 1"	2	3
514	526	Hard clay	3	
526	540	Clay and gravel 2"	2	
540	570	Hard clay	3	
570	606	Sticky clay	3	

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Roscoe Moss Company
Name

4360 Worth Street, Los Angeles, Calif
Address

Date March 5, 1948

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

(D-2-6) 16aac

From (feet)	To (feet)	Description of formation material		
0	5	TOP SOIL	1	
5	30	SILTY CLAY	3	
30	35	gravel	1	4
35	80	CLAY	3	
80	88	GRAVEL	1	
88	135	CLAY	3	
135	140	GRAVEL	1	
140	160	CLAY	3	3
160	165	SAND, GRAVEL AND FIRST WATER	1	
165	485	CLAY AND GRAVEL STREAKS	3	3
485	525½	CLAY	3	
525	530	SAND	1	
530	574	CLAY AND GRAVEL STREAKS	3	
574	577	BENTONITE	3	2
577	585	CLAY	3	
585	590	GRAVEL AND SAND	1	
590	600	GRAVEL	1	
		HAD TO WAIT ON POWER TO TEST PUMP. FINAL HOOKUP AND SET UP		
		WATER TANKS WAS ON AUGUST 23, 1980.		

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller F.L. MOORE DRILLING CO.
Name

17140 E. MEWS RD.
Address

CHANDLER, ARIZONA 85240
City State Zip

Date SEPTEMBER 10, 1980

Well No. (D-2-6) 16aad

Latitude-longitude 33 15 35 (N) (S) 111 44 15
d m s d m s

HYDROGEOLOGIC CARD

SAME AS ON MASTER CARD

Physiographic Province: BASIN & RANGE 22

Section: SONORAN

DESERT B

Drainage Basin:

Subbasin:

Topo of well site: (D) depression, (C) stream channel, (E) dunes, (F) flat, (H) hilltop, (K) sink, (L) swamp, (O) offshore, (P) pediment, (S) hillside, (T) terrace, (U) undulating, (V) valley flat

MAJOR AQUIFER: system, series, aquifer, formation, group

Lithology: Origin: Aquifer Thickness: ft

Length of well open to: ft Depth to top of: ft

MINOR AQUIFER: system, series, aquifer, formation, group

Lithology: Origin: Aquifer Thickness: ft

Length of well open to: ft Depth to top of: ft

Intervals Screened:

Depth to consolidated rock: ft Source of data:

Depth to basement: ft Source of data:

Surficial material: Infiltration characteristics:

Coefficient Trans: gpd/ft Coefficient Storage:

Coefficient Perm: gpd/ft^2; Spec cap: gpm/ft; Number of geologic cards:

PERFORATED 200 -590'

R 6E

T 2N

16

Well No. (D-2-6) 16 aad

GPO 869-201

(D-2-6)16aad

Vernon Shipp
Higley, Ariz.

0'3 to	424	Sandy clay stks clechies gravel 3
424	429	Packed pea gravel in clechie 2
429	438	Sticky clay stks shale rock 3
438	452	Cannon clay very tough & clechie 3
452	461	Sand and gravel size 3/4 in 1
461	464	Packed stks of clay 3
464	478	Sand and gravel size 1/8 inch 1
478	501	Clechie in clay 3
501	512	Sand and gravel stks cement 2
512	531	Clechie in red clay 3
531	533	Packed sand and grave very hard. 2
533	556	Course sand and gravel size 8" good 2
556	563	Hard brown clay and clechie 3
563	577	Conglomerate 2
577	587	Hard red clay stksclechieredclay 3
587	589	Course sand 1
589	596	Silty clay stksclechiered clay 3
596	610	Hard brown clay 3

3

LOG OF WELL

(D-2-6) 16 a d d

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL		
0	100	Surface silt and sand	2	3
100	300	Fine sand streaks of gravel	1	
300	428	Clay with streaks of sand	3	
428	700	Clay broken sandy clay	3	3
700	780	Clay	3	
	920	Gravel	1	1
	1030	Sandy Clay	3	
	1280	Clay with streaks of gravel	3	
	1300	Clay--Brown	3	3
	1375	Sandy Clay	3	
	1385	Clay with streaks of good gravel	3	
	1425	Clay	3	
	1500	Clay with streaks of sand and gravel	3	

I hereby certify that I have read the foregoing statements, and that each and all of the items therein contained are true to the best of my knowledge and belief.

Cameron & Clayton
Owner, Operator or Driller

Buckeye, Arizona
Address

Date April 10, 1962

16923

RECEIVED MAY 3 1966 STATE LAND DEPT.

(D-2-6) 16bab

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL		
0	55	Sandy clay	3	
55	280	Sand thin clay	1	
280	462	Sand layers & clay	2	3
462	505	Clay Gravel layers	2	
505	511	Sand & Gravel	2	
511	532	Sandy clay & Gravel	1	
532	536	Sand & Gravel	1	
536	570	Sandy clay	3	
570	574	Sand & Gravel	1	
574	590	Sandy clay	3	
590	593	Sand & Gravel	1	
593	612	Sandy clay	3	
612	615	Sand & Gravel	1	
615	634	Sandy clay	3	
634	637	Sand & Gravel	1	
637	657	Sandy clay	3	
657	661	Sandy clay Coarse Sand	2	
661	665	Sand & Gravel	1	
665	675	Sandy clay	3	
665	800	Sandy clay thin layers Sand & Gravel	3	

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

DEPARTMENT OF WATER RESOURCES

Driller Bert E Perry
Name

5338 F Apache Dr
Phoenix Ariz Address 85205

Date 5-31-81

2-73

LOG OF WELL (D-2-6)16bcb

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

From (feet)	To (feet)	Description of formation material
0	27	Sandy topsoil 1
27	48	Sand & Gravel 1
48	143	Sandy clay 3
143	295	Muddy sand & gravel (1st water 160') 2
295	340	Gravelly clay 3
340	495	Sandy clay (thin gravel strata's) 3
495	500	Red clay 3
500	535	Gravelly clay 3
535	555	Sticky red clay 3
555	607	~~Clay~~ Sand 1
607	627	Clay 3
627	645	Sand & Gravel 1 2 3
645	655	Sticky red clay 3
655	678	Coarse sand & gravel 1

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Leon Noel
Name

P.O. Box 1042 Chandler, Az.
Address

City State 85244 Zip

Date June 16, 1987

LOG OF WELL (D-2-6) 16c

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

From (feet)	To (feet)	Description of formation material
0	4	Sandy silt 3
4	18	Sandy clay 3
18	23	Sand and gravel 1
23	47	Sandy clay 3
47	53	Sand and gravel 1
53	72	Coarse sand and stks of clay 1
72	83	Red clay 3
83	94	Sandy clay 3
94	107	Coarse sand and pea gravel 1
107	119	Clechie in clay 3
119	130	Coarse sand and some gravel 1
130	151	Clay and stks of clechie 3
151	157	Sand and gravel ~~1~~ 1
157	169	Tight coarse sand and clay 3
169	177	Clechie in clay 3
177	182	Cemented gravel and stks of brown rock 2
182	208	Sand and gravel loose ~~1~~ water 1
208	258	Sandy clay and stks cement in clechie 3
258	286	Clay and clechie and stks of gravel 3
286	312	Sticky clay and stks of gravel 3
312	322	Tight sand and clay 2
322	329	Brown clay and stks 3
329	332	Solid cemented gravel 2
332	337	Sandy silt 3
337	340	Solid cement 3
340	346	Sandy Sandy and gravel ~~1~~" 1
346	354	Sandy clay 3
354	362	Gravel in clay, tight 2
362	363	Loose gravel 1" 1
363	367	Sandy clay 3
367	379	Sandy clay 3
379	402	Clechie in clay and stks of shale 3
402	407	Sand and gravel 3/4 good water. 1

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Roscoe Moss Company
Name

4360 Worth St, Los Angeles, California.
Address

Date

(D-2-6)
16 dab-

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

From (feet)	To (feet)	Description of formation material
0	4	Sandy Silt 3
4	18	Sandy Clay
18	23	Sand & Gravel 1
23	47	Sandy Clay 3
47	53	Sand & Gravel 1
53	72	Coarse sand, stks of clay 2
72	83	Red Clay 3
83	94	Sandy Clay 3
94	107	Coarse sand, pea gravel 1
107	119	Caliche in clay 3
119	130	Coarse sand, some gravel 1
130	151	Clay, stks of caliche 3
151	157	sand, gravel ½ 1
157	169	Tight coarse sand and clay 2
169	177	Caliche in clay 3
177	182	Cemented gravel, stks of rock 2
182	208	Sand, Gravel loose ½, water 1
208	258	Sandy clay, stks cement caliche 3
258	286	Clay, caliche, stks of gravel 3
286	312	Sticky clay, stks of gravel 3
312	322	solid cemented gravel
322	329	Brown clay, stks of shale rock 3
329	332	Solid cemented gravel
332	337	Sandy silt 3
337	340	Solid cement 3
340	346	Sandy gravel ½ 1
346	354	Sandy Clay 3
354	362	Gravel in clay tight 2
362	363	Loose gravel 1" 1
363	367	Caliche 3
367	379	Sandy clay 3
379	402	Caliche in clay, stks of shale 3
402	407	Sand & Gravel 3/4"
407	418	Hard red clay and gravel stks
418	427	Coarse sand, gravel stks shale rock 1
427	432	Hard Red Clay 3
432	437	Gray sandstone 1
437	441	Tight gravel in clay 2
441	447	Good gravel ½ water 1
447	451	Coarse sand in caliche, tight 2
451	473	Tough red clay 3
473	484	Caliche & Clay 3
484	489	Tight coarse sand 1
489	502	Clay in stks, caliche 3
502	517	Hard red clay
517	521	Sand, Gravel ½
521	552	Brown clay, stks shale rock
552	559	Sand, gravel ½
559	576	Brown rock, stks of clay
576	589	Sand, gravel ½
589	602	Blue sticky clay stks, caliche
602	605	Tight sand and clay
605	633	Coarse sand, gravel 2"
633	638	Brown clay
638	646	Red clay, stks. shale rock
646	656	Tight sand (no good)
656	672	Hard red clay, brown rock
672	679	Caliche in clay, hard
679	719	Red Clay, hard
719	723	Coarse sand in clay (not good)
723	725	Stks of gray rock, hard
725	731	Gray sandstone, soft
731	750	Red clay, stks. shale rock, hard

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

ROSCOE MOSS COMPANY

Driller: A. A. Barney Mgr.

Name: P. O. Box 64, Station A, Los Angeles, Calif.

Address

Date: June 25, 1948.

(D-2-6) 2nd

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL
0	30	Calichie 3
30	52	Sand and Clay 2
52	124	Clay 3
124	156	Water at 124 feet 1st, sand and Gravel 1
156	210	Sand and Gravel 1
210	265	Sand 1
265	268	Clay 3
268	379	Sandy Clay 3
379	396	Pack Sand 2
396	410	Sandy Clay 3
410	418	Sand Stone 2
418	430	Clay 3
430	435	Sand Stone 2
435	460	Clay 3
460	465	Sand Stone 2
465	495	Hard Clay 3
495	508	Sand Stone 2
508	525	Clay 3
525	543	Clay 3
543	550	Sand Stone 2
550	552	Clay 3
552	560	Sand Stone 2
560	570	Sand Stone 2
570	572	Clay 3
572	615	Sand Stone 2
615	675	Clay - Hard 3
675	710	Hard Clay 3
710	725	Calichie Sand Stone 2
725	732	Gray Sand Stone conglomerate 2
732	750	Calichie 3
750	765	Blue Shale 3
765	767	Sand 1
767	790	Blue Shale 3

I hereby certify that I have read the foregoing statements, and that each and all of the items therein contained are true to the best of my knowledge and belief.

Robison & Mason Drilling Company
Owner, Operator or Driller

P. O. Box 1991, Phoenix, Arizona
Address

Date 3-12-1951

ERR.

RECEIVED
MAY 2 1951
STATE LAND DEPT.
OF ARIZONA

LAND DEPARTMENT
WATER DIVISION
STATE OF ARIZONA

REPORT OF WELL DRILLER

This report should be prepared by the driller in all detail and filed with the State Land Commissioner following completion of the well.

1. OWNER Larry McKay (Name)
22803 So Val Vista, Chandler, Arizona 85224 (Address)

2. Lessee or Operator (Name)
(Address)

3. DRILLER D and R Drilling (Name)
(Address)

4. Location of well: Twp. 2S Rge. 6E Section 21 NE ¼ NE ¼ SW ¼ (10-acre subdivision)

5. Intention to Drill File No. _____ Permit No. _____

DESCRIPTION OF WELL

6. Total depth of hole 585 ft.

7. Type of casing Steel

8. Diameter and length of casing 8 in. from 0 to 585 in. from ___ to ___, ___ in. from ___ to ___

9. Method of sealing at reduction points

10. Perforated from ___ to ___, from ___ to ___, from ___ to ___, from ___ to ___

11. Size of cuts _____ Number of cuts per foot

12. If screen was installed: Length ___ ft. Diam ___ in. Type

13. Method of construction Drilled. (drilled, dug, driven, bored, jetted, etc.)

14. Date started April 7, 1980 (Month Day Year)

15. Date completed June 6, 1980 (Month Day Year)

16. Depth of water 300 ft. (If flowing well, so state.)

17. Describe point from which depth measurements were made, and give sea-level elevation if available

18. If flowing well, state method of flow regulation

19. REMARKS: Gravel pack from 585 up to 565

DO NOT WRITE IN THIS SPACE
OFFICE RECORD

Received _____ by _____
Filed 12-4-80 by Pat
File No. D(2-6)21ccc 8403

(Well Log to Appear on Reverse Side)

WQ FORM 6-201
REV. 4-27-63

(D-2-6)21ddb

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL	
0	5	Clay 3	
5	16	Caliche 3	
16	33	Sand 1	
33	46	Sandy clay 3	
46	55	Sand 1	
55	70	Sandy clay 3	
70	80	Sand 1	
80	85	Sandy clay 3	
85	100	Sand 1	
100	110	Sandy clay 3	
110	220	Clay and sand streaks 3	
220	250	Clay 3	
250	260	Fine sand and clay 3	
260	297	Sand 1	
297	312	Gravel and sand in clay 2	
312	319	Sand 1	
319	438	Clay 3	
438	445	Sand 1	
445	455	Sandy Clay 3	
455	490	Clay 3	
490	505	Clay with sand streaks 3	
505	520	Clay 3	
520	528	Sand 1	
528	550	Clay 3	
550	554	Sand 1	
554	595	Clay 3	
595	610	Clay with sand streaks 3	
		First Water at 115	
		Second Water at 420	

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Cabl Gatlin (Name)
Rt 1 Box 517 Eunice Az 85334 (Address)
Date April 20 1971

RECEIVED MAY 19 1971 STATE LAND DEPT.

E+S

(D-2-5) 22bac

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is a
indicate depth at which encountered, and depth to which it rose in well

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL	
0'	4'	Top soil 1	2
4'	12'	Clay 3	
12'	52'	Sand 1	1
52'	68'	Sand and gravel 1	
68'	128'	Sandy clay 3	3
128'	152'	Hard coletchy clay 3	
152'	158'	Sand and gravel	
158'	212'	Clay 3	
212'	232'	Cemented sand 2	
232'	248'	Sticky clay 3	
248'	256'	Sand and gravel 1	
256'	348'	Clay 3	
348'	368'	Cemented sand 2	2
368'	376'	Clay 3	
376'	384'	Sandy clay 3	
384'	412'	Hard clay 3	
412'	460'	Cemented sand 2	
460'	480'	Clay 3	
480'	492'	Cemented sand 2	
492'	500'	Clay 3	
500'	520'	Sticky clay 3	
520'	524'	Sand and gravel 1	
524'	624'	Sticky clay 3	
624'	640'	Cemented sand 2	
640'	668'	Clay 3	
668'	688'	Sticky clay 3	
688'	692'	Sand and gravel 1	
692'	708'	Cemented sand 2	
708'	728'	Clay 3	
728'	781'	Cemented sand 2	

I hereby certify that this well was drilled by me (or under my supervision , and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Vaughn Mosley
Name

Chandler, Arizona
Address

Date July 5, 1951

(D-3-6) 22 ac

ROOSEVELT WATER CONSERVATION DISTRICT

WELL NO. 16 1/2 - 3/4 W.

0 - 10	Soil Silt	1		1
10 - 30	Sand	1		
30 - 40	Clay	3		
40 - 55	Sand & Gravel	1		
55 - 75	Caliche Clay	3		
75 - 88	Sand & Gravel	1		
88 - 120	Clay	3		
120 - 230	Cemented Gravel	2		
230 - 246	Hard Clay	3		
246 - 252	Fine Sand	1		
252 - 275	Hard Clay & Gravel	2		3
275 - 290	Cemented Gravel	2		
290 - 326	Clay & Gravel	2		
326 - 344	Water Gravel, 3"	1		
344 - 415	Hard Clay & Gravel	2		
415 - 465	Cemented Gravel	2		
465 - 475	Hard Clay	3		
475 - 505	Cement or Sandstone	2		
505 - 550	Sandstone	2		
550 - 570	Hard Clay	3		
570 - 590	Sandstone	2		
590 - 602	Hard Clay	3		

Drilled: January 28, 1942 to February 18, 1942
602' of 20" 10 Ga. Casing.
Perforated 114' to 588' 10 Holes per 12"
Diameter of Perforations - 1/2"
Length of Perforations - 5"
Depth at which water first found - 120'
Standing Level before perforating - 95'
Standing Level after Perforating - 95'

ROOSEVELT WATER CONSERVATION DISTRICT

(D-2-6)22bdd

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL
0	20	Surface sand 1
20	60	Gravel and sand 1
60	100	Sand 1
100	320	Gravel and sand with very thin streaks of clay 2
320	380	Clay and gravel 2
380	600	Sand and gravel with clay streaks 2
600	1000	Clay with thin gravel streaks 3
1000	1350	Clay and gravel 2
1350	1400	Sand with clay and gravel 2

3

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller F. K. Hedler
Name

503 Erson Ave Buckeye
Address

Date Sept. 24. 1970

RECEIVED OCT 1 1970 STATE LAND DEPT.

1701

(D-2-6)22cba

Water level when first started Test ____ ft.

Draw down from standing level ____ "

No. of gallons per minute pumped when Test first started ____

No. of gallons per minute pumped when Test completed ____

Draw down at completion of Test ____ ft.

Hours Testing Well ____

Formation: Mention size of water gravel—

0	ft. to	12	ft.	Clay and caliche 3
12	" "	29	"	Coarse sand and gravel 1
29	" "	76	"	Clay, sand and gravel 2
76	" "	87	"	Sand, gravel and boulders 1
87	" "	113	"	Clay and fine sand 2
113	" "	176	"	Sandy clay, sand and gravel 3
176	" "	185	"	Sand and gravel 1
185	" "	195	"	Sandy clay 3
195	" "	260	"	Sandy clay, sand and gravel 3
260	" "	289	"	Gravel, some clay 2
289	" "	300	"	Clay, some sand 3
300	" "	320	"	Sand and gravel 1
320	" "	890	"	Sandy clay, sand and gravel 3
890	" "	1110	"	Sandy clay 3
1110	" "	1150	"	Sandy clay, sand and gravel 3
1150	" "	1505	"	Clay 3
1505	" "	1810	"	Clay, sand and gravel streaks 3
1810	" "	1840	"	Sand and gravel 1
1840	" "	1900	"	Clay, sand and gravel 2
1900	" "	1920	"	Sand and gravel 1
1920	" "	1960	"	Clay and hard sand 3
1960	" "	1982	"	Cemented conglomerate 2

3

If reducing strings of casing were cut off, state how cut ____

Depth from surface cut ____ ft.

Size of casing cut ____ in.

Lap in larger casing ____ ft.

Was adapter or cement used? ____

If casing was swedged or repaired, state depth, describe repairs [illegible]

and condition in which casing was left and probable [illegible]

(1)-2-6) 22 edd

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water-bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (feet)	TO (feet)	DESCRIPTION OF FORMATION MATERIAL		
		~~Top Soil~~	1	
0	55	Sand	1	
55	226			
226	270	Clay	3	
270	290	Sand	1	
290	376	Sand, Clay, and Gravel	2	3
376	482	Clay and Gravel	2	
482	506	Gravel	1	
506	576	Sandy Clay	3	
576	590	Sandy Clay	3	
590	595	Hard Clay	3	
595	730	Sandy Clay	3	

RECEIVED AUG 1 1 1952 STATE LAND DEPT.

RECEIVED AUG 28 1952 STATE LAND DEPT.

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller G. R. Maison
Name

1761 N. Country Club Dr., Mesa, Ariz.
Address

Date Aug. 12, 1952

NOTE: Report and Log are to be completed by the permittee and driller, and mailed by the permittee to the Ground Water Division of the State Land Department immediately upon completion of the well and pumping test.

LOG OF WELL (0-2-6) 22 ddc

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

From (feet)	To (feet)	Description of formation material		
				1
40	120	SAND & GRAVEL, SMALL ROCKS	1	
120	140	TAN COLORED SANDY CLAY	3	3
140	180	SAND & GRAVEL	1	1
180	310	SANDY CLAY	3	
310	400	CLAY WITH TRACES OF GRAVEL	3	
400	550	CLAY WITH SAND AND GRAVEL	3	3
550	570	SOFT BALLED CLAY	3	
570	670	SAND AND CLAY	2	
670	810	SOFT CLAY AND BALLED CLAY	3	

I hereby certify that this well was drilled by me (or under my supervision), and that each and all statements herein contained are true to the best of my knowledge and belief.

Driller LAYNE WESTERN CO., INC.
Name

12030 E. RIGGS RD
Address

CHANDLER AZ 85249
City State Zip

Date 11/15/90

(D-2-6)23ddd

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL	
		~~No log.~~	
		Land to be irrigated.	
		SE 4 NE 4 SW 4 480 acres	
		Were irrigated from wells on sec. 24, T2S, R5E	
		4-2-51.	
0	55	Sand - Started Drilling 4:00PM-4-3-55 / 17½" Bit	1
55	100	Sand	1
100	180	Sand & gravel	1
180	250	" "	1
250	270	clay	3
270	285	gravel	1
285	455	Clay w/ Sand streaks	3
455	600	Clay & gravel	2
600	700	clay	3
765	950	clay	3
950	955	Sand Drilled to 771 w/ 17½" Bit	1
		Come out of Hole put on 15" Bit	
955	1020	Clay & gravel	2
1020	1080	Sand & gravel	1
1080	1100	Clay & gravel	2
1100	1138	gravel (very Hard)	1
		Come out & Put on 26" Reamer	
		Depth - 1138"	
		Reamed & set 1097' of Pipe.	
		521' - 16" perf 3/8" up & Down Cuts	
		220' 20" perf. pipe cut same	
		356' of 20" Blank Pipe	
		2' 20" Pipe on top of ground.	

3

I hereby certify that I have read the foregoing statements, and that each and all of the items therein contained are true to the best of my knowledge and belief.

Owner, Operator or Driller

Address

Date

(D-2-6)26cdd

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL	
0	20	soil sand	1
20	40	soil, sand & gravel	1
40	60	" " "	1
60	80	" " "	1
80	100	" " " some clay	2
100	120	" " " "	2
120	140	" " " "	2
140	200	" " " "	2
200	220	clay sand	3
220	240	" "	3
240	280	sand & gravel	1
280	300	" "	1
300	320	clay	3
320	370	clay sand showing	3
370	400	" " "	3
400	450	" " "	3
450	500	" " "	3
500	550	" " "	3
550	600	caliche - clay	3
600	650	clay	3
650	700	clay	3
7000	720	clay	3
720	740	caliche - clay	3
740	822	sand & gravel	1
		Water to be used Twp 2S, Rge 6E, Sec. 26, SW 1/4, 160 acres	
		May 1, 1964 Had picture Via Level 2-1-61 Underground	
		Survey - No perforations in 14" Casing obstruction	
		at 471' - 12" slow went to 704' - Weber drilling	
		perforated from 970' up to 605' in 14" Casing 5 holes	
		per round 1½" to 18" apart - In 18" casing perforated 230' up to	
		196' 6 cuts per round 1½" X 2½" - 12" apart - Top of 14"	
		Casing 471' - total depth 1197 - Well Crooked below 1000'	

2

I hereby certify that I have read the foregoing statements, and that each and all of the items therein contained are true to the best of my knowledge and belief.

Owner, Operator or Driller

Address

Date

(D-2-6) 26 ddd

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL	
0	71	Sand 1	1
71	88	sand & clay 2	3
88	145	sand & boulders 1	1
145	290	sand, clay & boulders 2	
290	360	clay 3	
360	495	clay, sand & boulders 3	5
495	580	sand, clay & gravel 2	
580	611	gravel & clay 2	
611	668	gravel 1	
668	698	gravel & boulders 1	
698	717	clay & some boulders 3	
		total depth 717'	

I hereby certify that I have read the foregoing statements, and that each and all of the items therein contained are true to the best of my knowledge and belief.

Owner, Operator or Driller

Address

Date

(D-2-6) 27 cdd

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL	
0	4	Top Soil 1	
4	50	Hard clay 3	
50	64	Sand 1	
64	144	Clay 3	
144	148	Sand & gravel 1	
148	176	Clay 3	
176	180	Sand Gravel 1	
180	184	Stickey Clay 3	
184	192	Sandy Clay 3	3
192	212	Hard Clay 3	
212	236	Sticky Clay 3	
236	252	Sandy clay 3	
252	256	Sticky Clay 3	
256	264	Cemented Sand 2	
264	298	Clay 3	
298	303	Sandy Clay 3	
303	338	Clay 3	
338	340	Sand & gravel 1	
340	384	Clay with cemented streaks 3	
384	392	Sandy clay 3	
392	414	Sticky clay 3	
414	468	Sandy clay with cemented streaks 3	
468	500	Clay 3	
		(Perforated from 490' up to 155', 8 holes per foot.	
		Water level 148'.)	
		Driller, Vaughn Mosley	

RECEIVED AUG 14 1958 STATE LAND DEPT.

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Travis Mosley
Name

734 No. Colorado
Address

Date

(D-2-C) 27 dad

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL		
0	4	Top Soil	1	
4	32	Cobble w/ Stks. Gravel + Clay	2	3
32	71	Clay w/ Stks. Gravel	3	
71	153	Gravel + Boldors	1	1
153	205	Clay w/ Stks. Gravel	3	3
205	217	Boldors	1	
217	340	Clay w/ Stks. Gravel	3	
340	375	Boldors in Clay	2	
375	417	Clay w/ Stks Gravel	3	
417	429	Boldors + Sand	1	3
429	524	Clay w/ Stks. Cong.	3	
524	532	Cong. Cemented Hard.	2	
532	606	Clay w/ Stks. Cong.	3	
606	611	Sand + Gravel	1	
611	665	Clay w/ Stks. Cong.	3	

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller George L. Pfahl
Name

P.O. Box 277 Chandler Hts., Arizona 85227
Address

Date 3/3/1974

RECEIVED MAR 21 1974 STATE LAND DEPT.

2-73

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

From (feet)	To (feet)	Description of formation material			
0	55	Sand	1	540 - 580 Gravel embedded clay	3
55	70	Gravel	1	580 - 605 Sandy clay	3
70	140	Sand	1		
140	150	Water Sand	1		
150	155	Sandy Clay	3		
155	170	Sand & Gravel	1		
170	185	Sand	1		
185	205	Clay	3		
205	215	Gravel - (water)	1		
215	230	Conglomerate	2		
230	245	Sand	1		
245	250	Clay	3		
250	260	Sand	1		
260	280	Clay	3	3	
280	300	Clay	3		
300	310	Gravel (Water)	1		
310	330	Clay	3		
330	380	Sandy Clay	3		
380	390	Gravel	1		
390	410	Clay	3		
410	450	Gravel embedded clay	3		
450	475	Sandy Clay	3		
475	510	Gravel embedded clay	3		
510	530	Gravel embedded clay	3		
530	540	Gravel & Sand (Water)	1		

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller Kermit Hoover
Name

Box 487
Address

Stanfield, Az. 85272
City State Zip

Date 6-30-86

(D-2-D) 27 dbb

(0-2-0) 27 Abc

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

FROM (FEET)	TO (FEET)	DESCRIPTION OF FORMATION MATERIAL		
0	2	top soil	1	
2	180	sand & gravel w/ streaks of cleache	2	1
180	210	sand & gravel	1	
210	565	clay w/ streaks of sand	3	3
565	575	conglomerate (water formation)	2	

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller White Drilling CO.
Name

P.O. Box 558 Casa Grande, Arizona
Address

Date 1/20/ 1970

Geo. T. White

RECEIVED FEB 10 1970 STATE LAND DEPT.

LOG OF WELL

Indicate depth at which water was first encountered, and the depth and thickness of water bearing beds. If water is artesian, indicate depth at which encountered, and depth to which it rose in well.

From (feet)	To (feet)	Description of formation material		
0	7	Top Soil	1	
7	78	Shale w/ stk. Sand + Gravel	3	
78	88	Boulders	1	
88	360	Shale w/ stks. Sand + Gravel	3	
360	382	Sand + Gravel	1	
382	460	Shale w/ stks Sand + Gravel	3	3
460	478	Shale	3	
478	495	Sand + Gravel	1	
495	582	Shale w/ stks. Sand	3	
582	590	Conq.	2	
590	602	Shale	3	
602	610	Sand	1	
610	820	Shale w/ stks. Sand	3	

I hereby certify that this well was drilled by me (or under my supervision), and that each and all of the statements herein contained are true to the best of my knowledge and belief.

Driller DRILL X INC George T. White
Name

P.O. Box 277
Address

CHANDLER AZ. 85227
City State Zip

Date 12-22-1986

27 Abc (9-2-0)

(D-2-6)28 add

ROOSEVELT WATER CONSERVATION DISTRICT

WELL NO. 17 1/2 - 1 W

0	-	6'	Soil	1
6	-	20	Caliche	3
20	-	38	Sand & Gravel	1
38	-	45	Clay	3
45	-	55	Hard Sandy Clay	3
55	-	80	Sand & Gravel	1
80	-	95	Soft Sandy Clay	3
95	-	102	Caliche Clay	3
102	-	115	Sand & Gravel	1
115	-	150	Caliche Clay & Gravel	3
150	-	192	Caliche Clay	3
192	-	206	Brown Clay	3
206	-	214	Caliche	3
214	-	258	Cemented Gravel	2
258	-	286	Hard Clay & Gravel	3
286	-	336	Caliche Clay & Gravel	3
336	-	346	Cemented Gravel	2
346	-	372	Hard Caliche Clay	3
372	-	378	Soft Clay	3
378	-	386	Cemented Sand	2
386	-	394	Sand & Gravel	1
394	-	420	Sticky Clay	3
420	-	466	Hard Clay & Gravel	3
466	-	482	Sandy Clay	3
482	-	510	Hard Sandy Clay	3
510	-	520	Sand	1
520	-	550	Hard Clay	3
550	-	562	Gravel	1
562	-	600	Hard Clay	3

3

Drilled: Jan. 4, 1942 to Jan. 25, 1942.
600' of 20", 10 Ga. Casing
Perforated 90' to 585', 10 Holes per 12"
Diameter of Perforations - 5/8"
Length of Perforations - 5"
Depth at which water first found - 78'

24. If well is part of irrigation system of Irrigation District, Association or Company, omit 23 and give name of project.

ROOSEVELT WATER CONSERVATION DISTRICT
Name of Project

(D-2-6)28 a.d

EQUIPMENT DATA

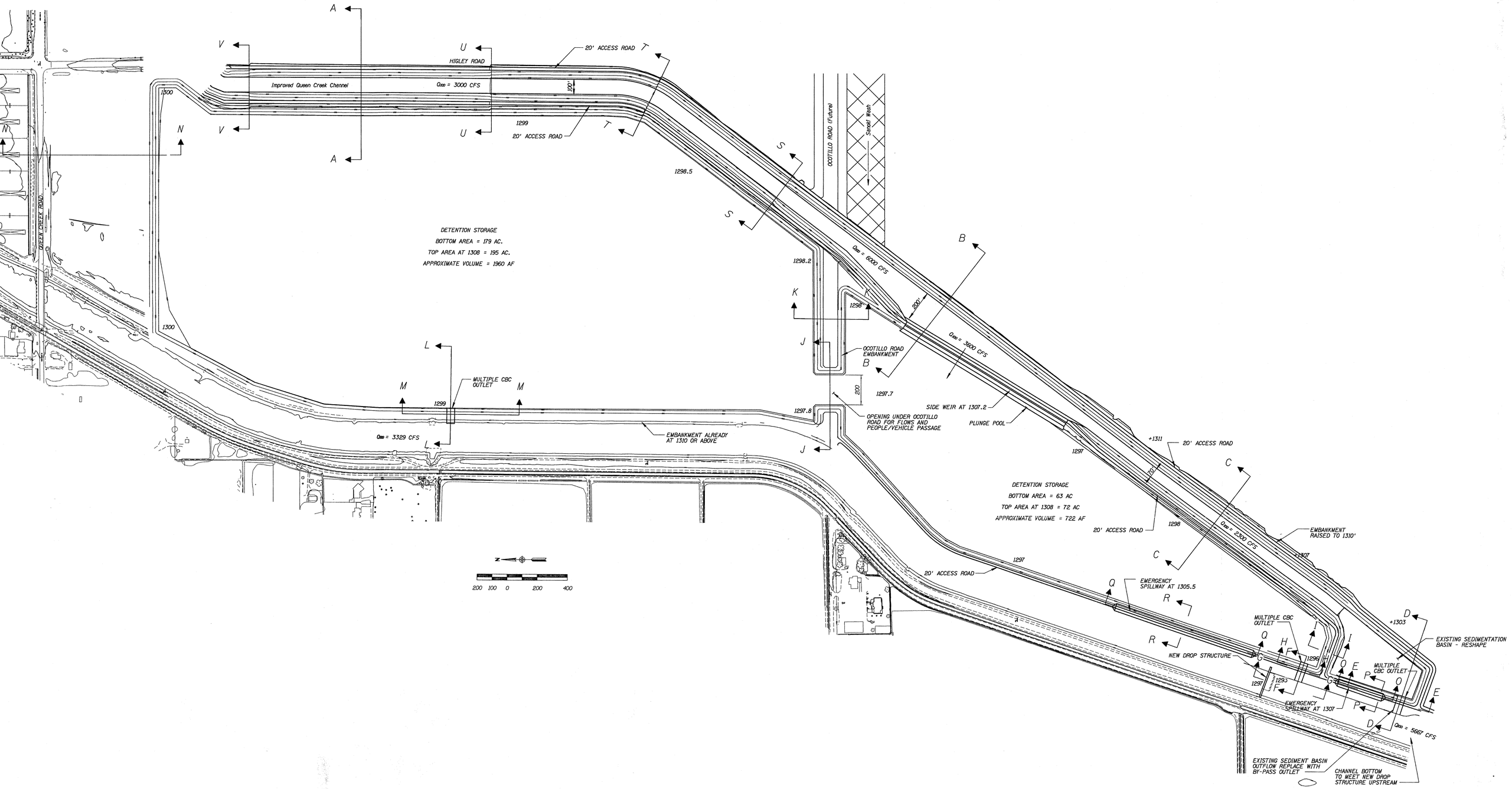
25. Kind of pump Turb.
turbine, centrifugal, etc.

26. Kind of power Elec.
electric, natural gas, etc.

27. Horsepower rating of motor 75

DO NOT WRITE IN THIS SPACE
OFFICE RECORD

Received 6-1-46 by IJ
Filed 6-17-46 by IJ
File No. (D-2-6)28 add
Cross-referenced (Name) _____ by _____
Cross-referenced (Basin) _____ by _____
Cross-referenced _____ by _____


HIGLEY ROAD
Improved Queen Creek Channel
Q100 = 3000 CFS
20' ACCESS ROAD
100'
1299
1300
1298.5
OCOTILLO ROAD (Future)
Sandy Wash
DETENTION STORAGE
BOTTOM AREA = 179 AC.
TOP AREA AT 1308 = 195 AC.
APPROXIMATE VOLUME = 1960 AF
QUEEN CREEK ROAD
Q100 = 6000 CFS
1298.2
1298
200'
Q100 = 3600 CFS
OCOTILLO ROAD EMBANKMENT
1297.7
200
1297.8
OPENING UNDER OCOTILLO ROAD FOR FLOWS AND PEOPLE/VEHICLE PASSAGE
SIDE WEIR AT 1307.2
PLUNGE POOL
MULTIPLE CBC OUTLET
Q100 = 3329 CFS
EMBANKMENT ALREADY AT 1310 OR ABOVE
+1311
20' ACCESS ROAD
1297
DETENTION STORAGE
BOTTOM AREA = 63 AC
TOP AREA AT 1308 = 72 AC
APPROXIMATE VOLUME = 722 AF
1298
Q100 = 2300 CFS
EMBANKMENT RAISED TO 1310'
1307
EMERGENCY SPILLWAY AT 1305.5
NEW DROP STRUCTURE
1296
1295
EMERGENCY SPILLWAY AT 1307
+1303
EXISTING SEDIMENTATION BASIN - RESHAPE
Q100 = 5667 CFS
EXISTING SEDIMENT BASIN OUTFLOW REPLACE WITH BY-PASS OUTLET
CHANNEL BOTTOM TO MEET NEW DROP STRUCTURE UPSTREAM
200 100 0 200 400