LEONA QUARRY HYDROLOGIC REVIEW
PHASE TWO

Prepared for

City of Oakland
Public Works Agency

Prepared by

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PWA REF. # 1674

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1. INTRODUCTION

The City of Oakland (City) retained Philip Williams & Associates (PWA) in Fall 2002 to provide support services related to hydrology and drainage issues for the proposed Leona Quarry development project (Project). PWA's scope of work included reviewing the hydrologic analysis performed on behalf of the DeSilva Group (Project developer) by Balance Hydrologics (BH), coordinating with Alameda County Flood Control and Water Conservation District (ACFCWCD or County), and providing input to the City as requested. The review performed by BH and by PWA evaluated the ability of the Leona Quarry drainage system to accommodate not only the storm flows caused by development of the Leona Quarry Project, but also a portion of the storm flows from the existing Ridgemont subdivision.

In November 2002, PWA published a report on an initial review (*Leona Quarry Hydrologic Review*; Phase 1) of the BH hydrologic analysis. Since the time of that report, changes have been made to the Project's proposed stormwater management system and BH has revised their calculations and hydrologic analysis to address CEQA requirements and to comply with the Project's conditions of approval. This report (Phase 2) provides an updated summary of our review of the revised BH calculations and hydrologic analysis, including a description of how the analysis has changed since the time of our first report in November 2002. In PWA's opinion, the BH proposed stormwater management system complies with the requirement that the project not worsen existing conditions; that is, post-Project peak flows from the site are equal to or less than existing peak flows for the chosen simulated storm events (24-hour, 2-, 5-, 10-, 25-, and 100-year events).

PWA has reviewed several versions of the stormwater system for Leona Quarry. Typical engineering practice is to develop a preliminary design for a stormwater management system, then refine the fundamental design and add details as the project progresses. Final design details of a stormwater management system typically are not developed before the final stages of project implementation, because it would be impractical to refine the details of a design before the preliminary, fundamental aspects of the design have been evaluated and certified as complying with applicable stormwater management criteria. PWA has reviewed the designs presented by BH as of Spring 2003. These designs are more detailed than are typically seen at this stage of project development, and include some details that are typically not seen until the time of final design. PWA has determined the adequacy of these designs to meet the criteria specified in this report. As is typical, the designs will be further refined and details will be added as the project progresses. In accordance with standard engineering practices, standard practices of the City of Oakland, and as typically required by conditions of approval, such as the previously adopted Condition of Approval 23, once the final details of the stormwater management system design are determined, they should be reviewed to confirm that the Project will not worsen existing conditions (as related to CEQA requirements) and will meet conditions of approval. PWA recommends that the City of Oakland coordinate with ACFCWCD to determine if the City should require specific criteria beyond the CEQA requirement that the Project not worsen existing conditions.

2. BACKGROUND

2.1 INITIAL REVIEW

PWA's initial work included review of hydrologic issues based on the BH report entitled *Analysis of Hydrologic Opportunities and Constraints at Leona Quarry*, *City of Oakland*, *California* (July 2001) and coordination with ACFCWCD. PWA conducted on-site reconnaissance (August 29, 2002) and reviewed the 2001 BH report as well as the calculations and hydrologic model on which the report was based. BH provided the calculations and hydrologic model to PWA at the request of City staff. The purpose of our review was to assess the appropriateness of the BH approach, analysis and conclusions to evaluate if the BH approach differed from the approach recommended by ACFCWCD and to determine whether the BH approach met professional standards for civil engineering and hydrology. PWA used the document entitled *Hydrology and Hydraulics Criteria Summary for Western Alameda County* (ACFCWCD, 1989) as the primary source for determining ACFCWCD hydrologic modeling standards. Other sources noted at the end of this report were used for other aspects of our review.

The preliminary results of the above review were conveyed to the City staff in early October 2002. In addition to the above in-house review, PWA recommended that the City work with ACFCWCD to clarify the hydrologic performance criteria that the Project should meet. ACFCWCD staff agreed to assist the City by providing technical input on the description of existing and future site hydrologic conditions and, in particular, the hydrologic parameters to be included in the computer simulation model. Hydrologic simulation results were assessed based on conformance with the CEQA criteria that the Project not worsen existing drainage problems, and the additional standard that redirected runoff from the existing Ridgemont subdivision not worsen existing drainage problems ("Not Worsen Criteria").

In October 2002, ACFCWCD, BH, PWA, and the City of Oakland sought agreement on the following modeling parameters:

- Subwatershed boundaries
- SCS Curve Numbers
- Time lag estimates
- Existing on-site detention storage

This latter point (amount and functioning of existing site detention storage) was one of the primary concerns initially expressed by ACFCWCD staff. ACFCWCD staff indicated concern whether the prior BH study accurately reflected existing site conditions with respect to existing on-site detention storage. In general, they noted the difference in total potential detention storage between existing conditions (approximately 21.8 acre-feet) and the Fall 2002 proposed post-Project preliminary detention estimate (approximately 18.2 acre-feet), although it was recognized that the proposed ponds would be designed to function much more efficiently and safely than the existing ponds (which had been constructed to facilitate sediment trapping desired for quarry operations and not as a flood detention system). Other concerns related to the method in which existing detention was characterized in the hydrologic model.

The intent of this analysis was to accurately and conservatively characterize the effectiveness of the existing site detention storage in reducing peak flood flows from the site.

BH agreed to run their model using the more conservative parameters recommended by ACFCWCD staff. BH ran their model using approximately 21.8 acre feet of existing on-site detention storage in four non-engineered ponds, with the outflow for each pond modeled as appropriate based on the outlet structure or weir dimensions.

In response to the process described above, BH indicated how use of the more conservative parameters would affect their prior hydrologic analysis and design proposal, and PWA reviewed the revised analysis in a November 2002 technical memo (PWA: *Leona Quarry Hydrologic Review*). In that memo, PWA reported that the hydrologic modeling methodology used by BH was generally consistent with published standards. PWA clarified which elements of the analysis represented consensus on methodology between the developer's proposal and the ACFCWCD published guidelines, and which elements were not in agreement. PWA noted some differences between modeling inputs used by BH and those suggested by ACFCWCD staff. PWA performed sensitivity analyses on these differences and identified three areas where the input parameter selection made a noticeable difference in the model results (detention pond initial storage volume, the watershed boundary determination near Mountain Boulevard, and the runoff curve number selection). These areas are discussed in detail in Chapter 3 of this report.

During fall 2002, ACFCWCD indicated that they would not formally comment further regarding the Project, since the City was the lead agency. ACFCWCD did, however provide a copy of their preliminary hydrologic model of the site to PWA for purposes of comparison with the BH modeling approach.

2.2 RECOMMENDATIONS

In the November 2002 review, PWA made recommendations to refine the analysis. PWA recommended that specific, more conservative parameters be used in the hydrologic analysis of both existing and post Project conditions. PWA noted however, that if the recommended parameters were used, the stormwater management system as it was then designed would not meet the Not Worsen Criteria requirement. PWA added that, in order to meet the Not Worsen Criteria, revisions to the proposed detention pond design and/or changes to the proposed detention pond outlet design may be required. PWA recommended that the City impose conditions of approval on the Project that require drainage design refinements during subsequent project review and approval in order to assure that the Project meets the Not Worsen Criteria and ACFCWCD guidelines.

During the Spring 2003, BH proposed additional refinements to the design (adding approximately 2.6 acre-feet of storage to the proposed detention basin) and analysis. PWA coordinated with BH on additional analysis issues and reviewed the spring 2003 BH proposal. Chapter 3 of this document summarizes our review of the Spring 2003 BH Leona Quarry hydrologic analysis and proposed stormwater system.

3. PWA REVIEW

The following sections summarize PWA's review of the revised BH modeling methodology as of September 2003. BH used ACFCWCD published hydrology guidelines (*Hydrology and Hydraulics Criteria Summary*, 1989) to model existing and post-Project peak flows. The basic tool used by BH to assess the hydrology of the Leona Quarry site was the HEC-1 computer model, developed by the US Army Corps of Engineers. This model is the standard computer model used for estimating storm runoff and evaluating flow management facilities such as detention ponds. HEC-1 is specifically approved by ACFCWCD for this type of analysis. In addition, it is the standard rainfall-runoff model approved for use by the federal government in conducting FEMA (Federal Emergency Management Agency) flood studies. HEC-1 estimates storm runoff from a design rainstorm using a variety of input parameters that characterize the site hydrology. Based on the estimated runoff, the model can also be used to estimate storage volume requirements for detention facilities. PWA applied HEC-1 to review the BH parameters used as input to the model for conformance with the ACFCWCD procedures and reviewed the model results.

The input parameters to the HEC-1 model are listed below, followed by a description of the review performed by PWA and our conclusions.

3.1 DESIGN STORM

The Project site watershed area is approximately 250 acres, and as such, the ACFCWCD Hydrology manual classifies the site as a "Primary Facility," having a "drainage area between fifty acres and ten square miles" (1989). The Project site is located in Western Alameda County Flood Control Zone 12. The ACFCWCD Hydrology manual recommends a 25-year design storm for Primary Facilities in Zone 12 (1989). The ACFCWCD Hydrology manual (1989) provides methodology for developing the recommended 25-year storm with duration of 24-hours in order to use the storm as input to the HEC-1 model. A 24-hour, 25-year storm is a storm that produces an amount of rainfall over 24 hours that occurs, on average, once every 25 years.

The 24-hour, 25- and 100-year design storms were selected for the analysis of existing and proposed site conditions, based on concurrence between ACFCWCD, BH, and PWA as described in the November 2002, PWA report. The 2-, 5-, and 10-year events were also included to assess the impacts of the proposed development given a wider range of storm events and to demonstrate compliance with the Not Worsen Criteria of no peak flow increase.

3.2 PRECIPITATION

ACFCWCD (1989) provides a method for estimating the depth of rainfall for a range of design storms based on the mean annual precipitation for a given location. The method provides a factor that is applied to the annual precipitation depth to calculate the design storm depth. The estimated factors for various events are as follows:

Event:	2-year	5-year	10-year	25-year	100-year
Factor:	0.0950	0.1340	0.1608	0.1944	0.2411

(ACFCWCD (1989) provides values for the 10-, 25-, and 100-year events, as shown above. The 2- and 5-year event values can be estimated using ACFCWCD's "Unit Mass Precipitation Curves" figure found in the ACFCWCD hydrology manual (1989); the 2- and 5-year event values shown above are reasonable based on that figure.)

ACFCWCD (1989) provides an isohyetal map that shows mean annual precipitation for all of western Alameda County. For comparison with the ACFCWD isohyetal map, PWA also checked data from the Upper San Leandro Filter Plant (USLFP) rain gage, which has been operated continuously by the East Bay Municipal Utility District since 1948 and is located approximately 0.5 miles from the Leona Quarry site. The mean annual rainfall at the USLFP gage as reported by the Western Regional Climate Center (wrcc.dri.edu) is 25.4 inches. It is reasonable to assume that precipitation may be slightly higher in the portions of the Leona Quarry watershed area that are higher in elevation than the location of the USLFP rain gage. Therefore the 26-inch annual rainfall estimated from the ACFCWCD isohyetal map would provide a reasonable estimate of annual precipitation. BH used 26 inches for their current HEC-1 analysis.

Applying the ACFCWD factors (see above) to the 26-inch annual average rainfall yields 24-hour, design storm rainfall depths as follows:

Event:	2-year	5-year	10-year	25-year	100-year
Depth (inches):	2.47	3.48	4.18	5.05	6.27

These were the values used by BH in their Spring 2003 HEC-1 analysis.

3.3 DRAINAGE AREA

HEC-1 applies the design storm rainfall to the area of the watershed in order to estimate rainfall over the area of interest. The boundaries of the watershed area are determined by topography and drainage infrastructure contributing runoff to the area of interest. Watershed areas are often sub-divided into sub-watersheds in order to provide a more detailed reflection of site drainage patterns and/or to generate peak flow estimates at particular locations (such as drainage infrastructure or stream confluences).

Using proposed development plans supplied by City Staff, topographic data provided by BH (10/10/02), aerial photos from ACFCWCD, USGS topographic data, and information from site visits, PWA assessed the appropriateness of the watershed boundaries identified by BH for use in the HEC-1 model. Complete topographic data for the Ridgemont Road and Mountain Boulevard areas were not included on the AutoCAD map provided by BH. ACFCWCD staff used USGS digital topographic data for these areas (ACFCWCD, 10/10/02). PWA used the ACFCWCD map to assess watershed boundaries in areas not covered by the AutoCAD map.

Based on topography, the Leona watershed can be separated into four major sub-watersheds: Ridgemont Road (north of the proposed development area), Leona Quarry (the proposed development area), I-580 (southeast of the proposed development area), and Mountain Boulevard (south of the proposed development area). All four of these areas ultimately drain to a 39-inch culvert under I-580. Runoff from the Ridgemont Road sub-basin first passes through the Leona Quarry sub-basin before reaching the I-580 culvert. The four major sub-watersheds were divided into smaller subbasins with similar characteristics to allow more accurate modeling. BH separated the Ridgemont Road area into two sub-watersheds to reflect a difference in land use. BH separated the Quarry area into six sub-watersheds for the model of existing conditions to reflect existing drainage patterns and detention facilities on the site. For proposed conditions, the development area was modeled as a single sub-watershed draining to the proposed detention pond. To reflect the City's desire that the Project accommodate some stormflows from existing Ridgemont subdivision, BH also added approximately 4.5 acres of residential area to the Ridgemont Road sub-watershed. Under the proposal, runoff from this area would be re-directed to the Ridgemont Road sub-watershed from its current flow path. PWA believes that the selection of sub-watersheds is appropriate to reflect site topography, drainage patterns and facilities.

In general, as shown in Table 1, the BH Fall 2002 sub-watershed delineations are similar to ACFCWCD delineations except in the case of the Mountain Boulevard sub-watershed. In Fall 2002, PWA conducted sensitivity analyses on delineation differences between BH and ACFCWCD by altering the sub-watershed parameters of the BH HEC-1 model to reflect ACFCWCD sub-watershed acreages and then re-running the BH model. The ACFCWCD delineation of the Mountain Boulevard sub-watershed includes approximately 10 acres along the eastern edge of the study area that BH did not include as of Fall 2002. The inclusion of a larger area in the subwatershed results in a larger peak discharge estimated by the model for the Mountain Boulevard area in the ACFCWCD model.

In the November 2002 memo, PWA indicated agreement with the BH delineation in three of the four subwatersheds; however, the PWA report recommended that the more conservative Mountain Boulevard delineation be used. Following the November memo, PWA staff conducted site reconnaissance of the Mountain Boulevard sub-watershed. Based on that reconnaissance, PWA and BH reached consensus on the area estimate to use for this more conservative analysis (see Table 1). The Mountain Boulevard sub-watershed area change was incorporated into the hydrology model after ACFCWCD indicated that they would no longer formally comment on the Project (see above).

3.4 SCS CURVE NUMBER

The Curve Number is a parameter developed by the USDA Soil Conservation Service (SCS, now Natural Resource Conservation Service) to characterize infiltration characteristics of land areas. The SCS method is approved by ACFCWCD for hydrologic analysis. Based on aerial photographs, the proposed development plans, and site reconnaissance, PWA selected curve numbers to describe each of the existing and proposed land use types, and then weighted the curve numbers based on land use acreages in order to calculate an overall curve number for each sub-watershed. Tables 2, 3, and 4 show curve number estimates for existing and developed conditions.

In Fall 2002, PWA conducted sensitivity analyses on curve number differences between BH and ACFCWCD by altering the curve number parameters of the BH HEC-1 model to reflect ACFCWCD sub-watershed curve numbers and then re-running the BH model. There were no material differences in peak discharge for most of the sub-watersheds. However, a material increase to total watershed peak discharge occurred when the Mountain Boulevard sub-watershed curve number was changed from the curve number estimated by BH to that estimated by ACFCWCD. Because the selection of different curve numbers had a significant impact on model results, PWA used the USDA National Engineering Handbook (1985) (per ACFCWCD guidelines) to independently estimate a curve number for the Mountain Boulevard residential area, as documented in Tables 2 and 3. PWA calculated a composite curve number for the Mountain Boulevard sub-watershed of 80, which slightly increases the design storm total watershed peak discharge for both existing and proposed conditions over the BH estimate.

Subsequent to the PWA Fall 2002 memo, BH and PWA agreed to run the HEC-1 model using the more conservative ACFCWCD estimates of curve numbers. Due to the sensitivity analysis described above regarding the Mountain Boulevard sub-watershed curve number estimate, BH conducted a more detailed land use and soil type analysis thereby setting the curve number to 85.3 and 85.7 for existing and proposed conditions, respectively, as compared to the ACFCWCD estimate of 87 (see Table 4). This represents a minor difference, and PWA concurs that this adjustment represents a more refined assessment of actual site conditions for the Leona Quarry Project, based on the Alameda County Soil Survey (USDA, 1975) and PWA site reconnaissance following the Fall 2002 hydrologic review. However, model results do not change significantly when the refined curve number estimates (85.3 and 85.7) are used versus the ACFCWCD estimate of 87; therefore, all ACFCWCD curve number estimates, including the Mountain Blvd. sub-watershed estimate of 87, are used in the current HEC-1 model.

3.5 INITIAL LOSS

The initial loss parameter quantifies the amount of rainfall that is stored in small surface depressions, intercepted by vegetation etc. and therefore does not contribute to runoff for a given storm. BH uses the HEC-1 default method to calculate initial loss. This represents conformance with ACFCWCD methodology.

3.6 TIME LAG

The time lag calculation estimates the time required for runoff to reach the point where peak flows are being estimated. Inputs to the calculation include overland flow resistance, length of the runoff flow paths, and change in elevation. ACFCWCD recommends estimating a weighted overland flow resistance coefficient (or composite Manning's "n" value) based on resistance values assigned by ACFCWCD to various land categories (Saleh, 11/12/02). The composite n-value is then input to the ACFCWCD time lag equation, which also requires the total length of the sub-watershed flow path, the length from the bottom of the sub-watershed to the centroid, and the slope of the sub-watershed. BH and ACFCWCD time lag values are shown in Tables 5 and 6.

While significant differences appear to exist between BH and ACFCWCD time lag estimates, PWA found that those differences do not significantly impact the model results. In Fall 2002, PWA conducted sensitivity analyses on the time lag component of BH 24-hour, 25- and 100-year HEC-1 models for both existing and proposed conditions and found that time lag differences—in combination or singularly—did not materially change model results.

In April 2003, ACFCWCD staff confirmed that the time lag estimates BH had used in its 2002 analysis reflected current ACFCWCD guidelines. BH used ACFCWCD (10/12/02) input parameters for calculating the time lag estimates, with the exception of the Mountain Boulevard area, which was changed as discussed above. PWA concurs that the general methodology and the refinement of the time lag values are appropriate. Tables 5 and 6 show the time lag values used in the revised BH HEC-1 model (Spring 2003).

3.7 DETENTION PONDS

Our review of the site hydrologic functioning included review of the existing conditions and proposed detention facilities, site visits, and coordination with the ACFCWCD and BH to develop consensus on the methods for modeling the existing and future conditions.

Detention ponds can have the effect of reducing the rate of discharge from one point in the watershed to the next point further downstream. Detention pond size and outlet structure configuration, among other factors, control the discharge rate. Detention ponds can be used to offset the impact of flooding by reducing discharge rates to levels below that which may be damaging to downstream structures and/or facilities.

For detention pond analyses, HEC-1 calculates the volume of water stored in a detention pond of given dimensions based on the calculated storm runoff (inflow) in combination with a stage-discharge curve or rating curve (outflow) provided by the user. The rating curve indicates the amount of flow ("discharge") that will be leaving the pond when the water is at any given depth ("stage") in the pond. Stage-discharge relationships are calculated based on standard hydraulic equations for pipes, weirs and/or orifices, depending on the configuration of the pond outlet. PWA reviewed BH's analysis of storage and discharge for the five ponds: four existing ponds (ponds 1 – 4), and one proposed detention pond.

A stage-storage curve or table for a pond shows the volume of water stored in that pond that corresponds to any given pond depth. To assess the stage–storage relationship of the existing ponds, PWA compared surveyed data of each pond to the volume estimations assumed by BH. BH's pond volume and stage-storage relationship estimations for the existing ponds appear reasonable based on topographic data received from BH.

3.7.1 Ponds 1 and 2: Lower Ponds

Ponds 1 and 2 are existing ponds located in the lower, southwestern portion of the proposed development site. Ponds 1 and 2 would be removed under the proposed Project design. BH and ACFCWCD stage–storage relationships for Ponds 1 and 2 are almost identical (Figures 1 and 2). Per BH survey data, Pond 1 has a maximum volume of 3.35 acre-feet, and Pond 2 has a maximum volume of 4.38 acre-feet. Neither ACFCWCD nor BH found existing outlet structures at either pond. According to BH survey data, Pond 1 outflows via a non-engineered spillway, below which the pond retains 0.95 acre-feet of storage. Similarly, Pond 2 outflows via spillway, below which the pond retains 2.35 acre-feet of storage. BH and ACFCWCD used slightly different methods to estimate a stage-discharge curve based on assumptions regarding flow over the spillway, resulting in slightly different curves (Figure 3 and 4). Both methods appear reasonable. As the results are essentially the same, PWA does not recommend any changes to BH's rating curves. Neither pond has a low elevation outlet to drain the pond quickly following a storm event. Water below the spillway elevation exits the ponds via seepage and evaporation.

At the time of the November 2002 PWA memo, there was one significant difference between the BH and ACFCWCD approaches for modeling Ponds 1 and 2. In the existing conditions HEC-1 model, BH assumed that Ponds 1 and 2 were initially full of water up to the spillway crest elevations, thereby retaining 0.95 acre-feet and 2.35 acre-feet, respectively. The reasoning was that the design rainstorm would likely not be the first storm of the wet season, and the ponds would already be filled by prior minor storms to the spillway elevation at the beginning of the design storm. ACFCWCD recommended that the ponds be considered initially empty, so that the full pond storage volume below the spillway is available to store water from the design storm. Assuming that the ponds are initially empty results in a more conservative characterization of existing site conditions. In the November 2002 memo, PWA concurred with this more conservative assumption, recommending that Ponds 1 and 2 should be considered initially empty. The spring 2003 BH model reflects that recommendation.

3.7.2 Pond 3: Large Upper Pond

Pond 3 is the existing pond located in the east-central portion of the proposed development site. Pond 3 would be removed under the proposed Project design. Figure 5 shows the stage–storage relationship of the proposed pond. BH estimates that Pond 3 has a detention volume of 14.27 acre-feet. The exact details of the existing outlet structure are not known because much of the structure is covered by rock. BH and ACFCWCD assume that the pond outlet structure can be modeled as an 18-inch riser with a 12-inch circular orifice, whereby flow is limited to 22 cubic-feet per second by the 18-inch corrugated metal outlet pipe. The limiting flow parameter (22 cfs) appears reasonable based on the assumed outlet dimensions. Based on the above assumptions for Pond 3, PWA calculated a rating curve, which roughly agrees with BH's curve (Figure 6). The assumptions for Pond 3 and the related calculations appear reasonable.

3.7.3 Pond 4: Small Upper Detention Pond

Pond 4 is the existing pond located at the west-central corner of the proposed development site. BH's estimates of the storage volume and associated stage-storage relationship of Pond 4 (Figure 7) appear reasonable based on the footprint of the pond. BH estimates that Pond 4, under existing conditions, has a detention volume of 3.07 acre-feet. PWA generated a stage-discharge relationship using standard equations for pipe hydraulics based on the existing 30-inch corrugated metal pipe outlet and assuming a 15-foot weir spillway if the pond is overtopped. The rating curve generated by PWA approximately agrees with that used by BH (Figure 8).

Under the current development proposal, Pond 4 would remain and the Pond 4 outlet works would be improved. BH estimates that Pond 4 improvements would result in a slight detention storage increase, from 3.07 acre-feet to 3.15 acre-feet. BH models the outflow under proposed conditions assuming a single drop box (the perimeter would be comparable to that of a 36-inch riser) with one rectangular orifice (2.75 ft x 2.00 ft). BH's model assumes that all flow up to and including the 100-year flow would pass through the new drop box. Given the outlet dimensions specified by BH for future conditions, PWA verified the Pond 4 rating curve (without spillway) for the developed scenario (Figure 9). It should be recognized that the existing 30-inch outlet pipe would not have the capacity to handle the flow from the proposed 36-inch riser/orifice structure. BH reports that the applicant plans to replace this pipe with a 42-inch pipe, which appears adequate. PWA assumes that the proposed riser/orifice structure would dictate the limit of outflow and, as such, existing Pond 4 outlet structures, including the existing 30-inch outlet pipe, would also be replaced as necessary. This was required by Condition of Approval 23, and is consistent with standard engineering practice, and would be expected to be imposed as a condition of project approval. PWA assumes that the 42-inch outlet pipe would discharge directly to the proposed detention pond (see below), which would further control stormwater flows from Pond 4.

The Quarry operators constructed Pond 4 at some time in the past. We are not aware of any engineering design plans, construction plans, or specifications to document the construction practices. A geotechnical engineer should therefore inspect the pond levees and proposed Project changes to verify structural integrity for future, ongoing use. This is required by Condition of Approval 23h and is consistent with standard engineering practice.

3.7.4 Proposed Detention Pond

In assessing the proposed detention pond, it is PWA's understanding that BH sought to develop a detention pond design that would contain the design storm (24-hour, 25-year event) such that all storm runoff from the Project site, including the 4.5 acres of the Ridgemont subdivision and the Mountain Boulevard sub-watershed described above, would drain through the existing 39-inch culvert that passes under the I-580 freeway, and would satisfy the Not Worsen Criteria for the chosen range of storms (24-hour, 2-, 5-, 10-, 25-, and 100-year storms).

In November 2002, BH proposed a pond detention storage capacity of 13.2 acre-feet, maintaining one foot of freeboard (15.0 acre-feet total detention capacity when full). The plan has been refined to provide a proposed detention storage capacity of 15.6 acre-feet of water volume, maintaining one foot of freeboard (resulting in total capacity greater than 17 acre-feet). The BH model assumes that the proposed detention basin will maintain 3:1 internal side slopes (same as November 2002) and a four-foot northern (uphill side, adjacent to proposed homes) retaining wall of approximately 700 linear feet (this was added to the design subsequent to the November 2002 review). As of November 2002, BH projected the bottom and top proposed pond elevations to be 298.5 and 314.0 feet, respectively, and assumed a single outlet box with a perimeter comparable to that of a 48-inch riser at an elevation of 312.5 feet and two rectangular orifices (lower orifice 2 ft by 2 ft with flowline elevation at 298.5 ft, and upper orifice 2.25 ft by 2 ft with flowline elevation at 304.0 ft). As of Spring 2003, BH models the proposed pond outlet assuming bottom and top elevations at 296.0 and 315.5 feet, respectively, and assuming a single outlet box with a perimeter comparable to that of a 42-inch riser at an elevation of 313.5 feet and two rectangular orifices (lower orifice 2 ft by 2 ft with flowline elevation at 299.0 ft, and upper orifice 1.75 ft by 2 ft with flowline elevation at 307.0 ft). Currently, BH assumes that the lowermost three feet of the proposed pond would be reserved for water quality improvement, and the 15.6 acre-foot detention capacity of the pond does not include these bottom three feet.

Figure 10 shows the stage–storage relationship of the proposed pond. As is commonly the case, the precise stage–storage relationship and rating curve for the proposed detention pond cannot be known until the final design of the pond outlet is complete. The outlet structure included in the plans BH submitted in Spring 2003 is designed to limit peak discharge for the 25-year design storm to within the estimated capacity of the 39-inch pipe (see below). Based on the design assumptions described above, PWA verified the BH rating curve (Figure 11). The HEC-1 model confirms that the proposed detention pond would return to its original water surface elevation within 24 hours after cessation of the 100-year, 24-hour rain storm, as required by ACFCWCD. The HEC-1 model also indicates that the 100-year, 24-hour hour rain storm would yield a peak stage of approximately 314.5 feet in the proposed detention basin (corresponding to the proposed volume of 15.6 acre-feet, while maintaining one foot of freeboard). PWA recommends that the proposed detention basin be designed so as not to structurally fail in the event of a 24-hour, 100-year storm event; therefore, PWA recommends that appropriate engineers (for example, geotechnical engineer and/or civil engineer) review the final design of the proposed detention basin as a Project condition of approval.

The capacity of the storm drain system downstream of the Project site, including the I-580 39-inch culvert, has been estimated "on the order of 180 cfs" by ACFCWCD staff (Balance Hydrologics, 2001). BH estimated that the I-580 culvert has a maximum non-pressurized flow capacity of 172 cfs based on a Manning's pipe flow analysis. It is standard engineering practice to determine precisely how the proposed Project stormwater system (including the detention pond, the proposed pond outlet structure, the junction box between the proposed pond and the I-580 culvert, and all connections to and from the junction box) will accommodate the constraints of downstream facilities during the final design stages. The final design details of the stormwater system should be reviewed to confirm the conclusions of this report; the review should include a detailed hydraulic evaluation of this junction point in the context of upstream and downstream facilities and conditions. This analysis would be used to determine the flow characteristics under which post-project flows will be released. The flow characteristics are controlled by the final design of the outlet structure, its configuration and its connection to the junction box. This analysis would therefore be used to confirm that the Project (as discussed herein) would not worsen the peak flow rates and that applicable ACFCWCD and City of Oakland stormwater management requirements would be met.

4. MODEL RESULTS/CONCLUSIONS

As of June 2003, PWA had reviewed the refined plans proposed by BH (Spring 2003), including the input parameters and assumptions to be used in the Project analysis. In PWA's opinion, all parameters and assumptions used to design the stormwater management system (as indicated by BH), including the 15.6 acre-foot proposed detention pond capacity, comply with standard engineering practice. Table 7 summarizes peak stormwater flows before and after Project development as shown in current PWA Leona watershed HEC-1 peak flow model results. These are based on input parameters recommended by PWA, including the parameter that characterizes existing conditions of Ponds 1 and 2 being initially empty.

The existing Project site detention capacity, including ponds 1 – 4, is approximately 21.8 acre-feet. However, existing Project site detention facilities were not engineered to handle stormwater flow from a housing development. As of November 2002, the proposed Project site detention, including Pond 4 (after proposed improvements) and the proposed detention basin, was approximately 18.2 acre-feet. As of Spring 2003, the proposed Project site detention was approximately 20.8 acre-feet.

The model results indicate that the refined stormwater management system, with the 15.6 acre-foot proposed detention pond capacity (total capacity greater than 17 acre-feet due to freeboard requirements), complies with the Not Worsen Criteria. However, PWA recommends that a detailed hydraulic analysis, which would take into account hydraulic conditions (head and tailwater conditions) both upstream and downstream of the I-580 culvert as described above, be conducted as the proposed stormwater management system is refined and finalized during the design phase. Based on the current analysis, it should be noted that the 39-inch pipe is not adequate to handle the estimated 100-year discharge under either existing conditions or post-development conditions. The model results also indicate that the refined stormwater management system, with the 15.6 acre-foot proposed detention pond capacity, will maintain post-Project, 100-year, 24-hour peak flow from the site at a rate equal to or less than the existing peak flow from the 100-year, 24-hour design storm.

5. ADDITIONAL SITE OPPORTUNITIES

Given the current status of the Project review and design process, there is benefit in maintaining some flexibility regarding the exact volume and design specifications of the proposed detention pond. It is recognized that the area available for this pond is constrained. The proposed detention pond footprint can be maintained while altering various pond design specifications to vary detention capacity, such as varying the internal side slopes or making the pond deeper. While a detailed analysis of the exact stage-storage-discharge relationship would be necessary in order to provide precise design specifications, and the feasibility of increasing the proposed detention pond size, both hydraulically and geotechnically, would require detailed analysis before further consideration, our preliminary estimate is that the design of the proposed detention pond could be increased without disrupting the Project site plan.

6. SOURCES

Alameda County Flood Control and Water Conservation District (ACFCWCD), *Hydrology and Hydraulics Criteria Summary*, Alameda County Public Works Agency August 7, 1989.

Alameda County Flood Control and Water Conservation District (ACFCWCD), Watershed delineation map, received at meeting (10/14/02, ACFCWCD offices), map dated October 10, 2002.

Alameda County Flood Control and Water Conservation District (ACFCWCD), Preliminary HEC-1 model of Leona Quarry and surrounding sub-watersheds, received from ACFCWCD via e-mail on October 13, 2002.

Balance Hydrologics, Inc. (BH), *Analysis of Hydrologic Opportunities and Constraints at Leona Quarry City of Oakland, California*, July 2001.

Balance Hydrologics, Inc. (BH), Topographic map of study area and watershed delineations, AutoCAD format, posted to Balance Hydrologics, Inc. FTP site: ftp://ftp.balancehydrologics.com/, October 10, 2002.

Balance Hydrologics, Inc. (BH), HEC-1 model of Leona Quarry and surrounding sub-watersheds and supporting calculations, received via e-mail on October 14, 2002.

McCuen, Richard, *Hydrologic Analysis and Design*, Prentice Hall, New Jersey, 1989.
USDA, Soil Conservation Service, *National Engineering Handbook*, 210-VI-NEH-4, March, 1985.

Philip Williams & Associates, Ltd. (PWA), *Leona Quarry Hydrologic Review*, November 20, 2002.

Saleh, Rohin, Alameda County Flood Control and Water Conservation District (ACFCWCD), Personal communication via e-mail, regarding ACFCWCD guidelines for estimating time lag, November 12, 2002.

United States Department of Agriculture Soil Conservation Service (USDA SCS), *Soil Survey of Alameda County, California, Western Part*, (1975 soil conditions), 1981.

United States Department of Agriculture Soil Conservation Service (USDA SCS), *National Engineering Handbook, Hydrology*, 210-VI-NEH, March, 1985.

Western Regional Climate Center, USLFP gage data, http://www.wrcc.dri.edu/, November 11, 2002.

Table 1. Sub-watershed Delineation

	Sub-watershed	Area, square miles BH, 10/14/02	 County, 10/13/02	 BH, July, 2003
Existing Conditions	Ridgemont1	0.078	0.087	0.078
	Ridgemont2	0.041	0.043	0.041
	Upper Quarry	0.078	0.078	0.078
	Pond1 Basin	0.006	0.006	0.006
	Pond2 Basin	0.019	0.019	0.019
	Lower Quarry - West	0.031	0.030	0.031
	Lower Quarry	0.010	0.010	0.010
	Lower Quarry - East	0.009	0.009	0.009
	I-580	0.011	0.014	0.011
	Mountain Blvd	0.075	0.093	0.080
	Total	**0.358**	**0.389**	**0.363**
Developed Conditions	Ridgemont1	0.085	0.087	0.085
	Ridgemont2	0.041	0.043	0.041
	Development	0.152	0.159	0.152
	I-580	0.011	0.010	0.011
	Mountain Blvd	0.078	0.093	0.083
	Total	**0.367**	**0.392**	**0.372**

Table 2. Curve Number Estimations, Existing Conditions

	Curve Number	Ridge-mont1, %	Ridge-mont2, %	Upper Quarry, %	Pond1 Basin, %	Pond2 Basin, %	Lower Quarry West, %	Lower Quarry, %	Lower Quarry East, %	Mtn Blvd, %	I-580, %
County, 10/13/02											
Wooded, fair	79.0	40.0	100.0				15.0		90.0	20.0	55.0
Residential	89.0	45.0								80.0	
Dirt cover	89.0			100.0	100.0	100.0	78.0	98.0	10.0		
Hard surface	92.0	15.0					7.0	2.0			45.0
Total %		100	100	100	100	100	100	100	100	100	100
Weighted Curve Number		**85.5**	**79.0**	**89.0**	**89.0**	**89.0**	**87.7**	**89.1**	**80.0**	**87.0**	**84.9**
BH, 10/14/02[1]											
Wooded, fair	79.0	20.6	98.9				31.5		89.3	49.4	60.3
Residential, 1/4 acre lots	87.0	79.4					2.0				
Residential, 1/3 acre lots[2]	72.0									47.9	
Detention basin	98.0		1.1	0.4	7.3	3.3					
Dirt cover	89.0			99.6	92.7	96.7	41.1	39.1			
Gravel road	91.0						18.3	45.3			
Paved surfaces	98.0						7.1	15.6	10.7	2.7	39.7
Total %		100	100	100	100	100	100	100	100	100	100
Weighted Curve Number		**85.4**	**79.2**	**89.0**	**89.7**	**89.3**	**86.8**	**91.3**	**81.0**	**76.2**	**86.5**
PWA, November 2002											
Wooded, fair	79.0	35.0	99.0				20.0		85.0	30.0	45.0
Residential (D soil type)	89.0	60.0					2.0			20.0	
Residential (B soil type)	77.0									50.0	
Detention basin	98.0		1.0	1.0	6.0	3.0					
Dirt cover	89.0			99.0	94.0	97.0	60.0	75.0	10.0		10.0
Hard surface	92.0	5.0					18.0	25.0	5.0		45.0
Total %		100	100	100	100	100	100	100	100	100	100
Weighted Curve Number		**85.7**	**79.2**	**89.1**	**89.5**	**89.3**	**87.5**	**89.8**	**80.7**	**80.0**	**85.9**

[1]BH chose to use County (10/12/02) curve numbers for the current model (as of July, 2003).

[2]BH (10/14/02) selected a curve number for Mtn. Blvd. 1/3-acre lots based on group B soils.

Table 3. Curve Number Estimations, Proposed Developed Conditions

	Curve Number	Ridge-mont1, %	Ridge-mont2, %	Quarry, %	Mtn Blvd, %	I-580, %
County, 10/13/02						
Wooded, fair	79.0	40.0	100.0		20.0	55.0
Residential	89.0	45.0			80.0	
Dirt cover	89.0					
Newly graded open space	89.0			80.0		
Hard surface	92.0	15.0				45.0
Impervious area	98.0			20.0		
Total %		100	100	100	100	100
Weighted Curve Number		**85.5**	**79.0**	**90.8**	**87.0**	**84.9**
BH, 10/14/02[1]						
Wooded, fair	79.0	18.9	98.9	5.5	47.5	62.5
Residential, rowhouses	92.0			40.9		
Residential, 1/4 acre lots	87.0	81.1		2.6		
Residential, 1/3 acre lots[2]	72.0				46.3	
Detention basin	98.0		1.1	1.1		
Dirt cover	89.0			11.6		
Gravel road	91.0					
Paved surfaces	98.0				2.6	37.5
Restored slope areas	89.0			38.2		
Commercial	95.0				3.8	
Total %		100	100	100	100	100
Weighted Curve Number		**85.5**	**79.2**	**89.7**	**76.9**	**86.1**
PWA, November 2002						
Wooded, fair	79.0	35.0	99.0		30.0	45.0
Residential (D soil type)	89.0	60.0		55.0	20.0	
Residential (B soil type)	77.0				50.0	
Graded/Restored slopes	92.0			35.0		
Detention basin	98.0		1.0	1.0		
Dirt cover	89.0			9.0		10.0
Hard surface	92.0	5.0				45.0
Total %		100	100	100	100	100
Weighted Curve Number		**85.7**	**79.2**	**90.1**	**80.0**	**85.9**

[1]BH chose to use County curve numbers (10/13/02) for the current model (as of July, 2003).
[2]BH (10/14/02) selected a curve number for Mtn. Blvd. 1/3-acre lots based on group B soils.

Table 4. Balance Hydrologics Mountain Blvd. Curve Number Estimations

Existing Conditions:	Area, mi^2	Soil Group	CN
Grassland, range, poor	0.0156	D	89
Residential at top	0.0039	D	89
Wooded, fair	0.0156	D	79
Residential Mayhem	0.0156	D	89
Residential Xerorthents	0.0191	B	81
Residential Millsholm/Mayhem	0.0068	D	89
Residential Los Gatos	0.0014	C	87
I-580 I	0.0014	n.a.	98
I-580 II	0.0006	n.a.	98
Total	0.0801		
Weighted Curve Number			85.3

Proposed Developed Conditions:	Area, mi^2	Soil Group	CN
Grassland, range, poor	0.0156	D	89
Residential at top	0.0039	D	89
Wooded, fair	0.0156	D	79
Residential Mayhem	0.0156	D	89
Residential Xerorthents	0.0191	B	81
Residential Millsholm/Mayhem	0.0068	D	89
Residential Los Gatos	0.0014	C	87
I-580 I	0.0014	n.a.	98
I-580 II	0.0006	n.a.	98
Leona Gateway, commercial	0.0029	D	95
Total	0.0829		
Weighted Curve Number			85.7

BH chose to use County curve numbers (10/13/02) for the current model (as of July, 2003).

Table 5. Time Lag Estimations, Existing Conditions

SubBasin	Basin "N"	Length, L (ft)	Length, L (mi)	Length to Centroid, Lc (ft)	Length to Centroid, Lc (mi)	Slope, S (ft/mi)	K-factor[1]	Time Lag[2] (hr)	Time Lag[2] (min)
County, 10/13/02									
Ridgemont1	0.024	3953	0.749	1451	0.275	245	28.4	0.132	7.9
Ridgemont2	0.060	1739	0.329	877	0.166	1686	42.3	0.205	12.3
Upper Quarry	0.045	2138	0.405	900	0.170	1315	37.6	0.156	9.4
Pond1 Basin	0.045	528	0.100	363	0.069	1307	45.0	0.078	4.7
Pond2 Basin	0.045	1701	0.322	659	0.125	955	42.9	0.155	9.3
Lower Quarry - West	0.052	1885	0.357	1086	0.206	857	40.4	0.216	12.9
Lower Quarry	0.045	740	0.140	423	0.080	264	45.0	0.128	7.7
Lower Quarry - East	0.058	1531	0.290	804	0.152	1357	45.0	0.203	12.2
I-580	0.040	1935	0.366	1183	0.224	409	39.7	0.196	11.8
Mountain Blvd	0.046	3272	0.620	1990	0.377	682	30.6	0.234	14.1
BH, 10/14/02									
Ridgemont1	0.023	3872	0.733	2216	0.420	817	28.7	0.118	7.1
Ridgemont2	0.060	1649	0.312	716	0.136	1825	43.7	0.190	11.4
Upper Quarry	0.045	1985	0.376	858	0.163	1402	39.2	0.154	9.2
Pond1 Basin	0.045	503	0.095	327	0.062	1214	45.0	0.075	4.5
Pond2 Basin	0.045	1485	0.281	460	0.087	1126	45.0	0.130	7.8
Lower Quarry - West	0.045	1890	0.358	1047	0.198	869	40.3	0.183	11.0
Lower Quarry	0.041	746	0.141	449	0.085	228	45.0	0.123	7.4
Lower Quarry - East	0.057	1474	0.279	508	0.096	1419	45.0	0.163	9.8
I-580	0.037	1466	0.278	938	0.178	540	47.1	0.168	10.1
Mountain Blvd	0.038	4160	0.788	2925	0.554	529	28.0	0.235	14.1
BH, Spring 2003									
Ridgemont1	0.024	3953	0.749	1451	0.275	245	28.5	0.132	7.9
Ridgemont2	0.060	1739	0.329	877	0.166	1686	30.0	0.145	8.7
Upper Quarry	0.045	2138	0.405	900	0.170	1315	30.0	0.125	7.5
Pond1 Basin	0.045	528	0.100	363	0.069	1307	30.0	0.052	3.1
Pond2 Basin	0.045	1701	0.322	659	0.125	955	30.0	0.108	6.5
Lower Quarry - West	0.052	1885	0.357	1086	0.206	857	30.0	0.160	9.6
Lower Quarry	0.045	740	0.140	423	0.080	264	30.0	0.085	5.1
Lower Quarry - East	0.058	1531	0.290	804	0.152	1357	30.0	0.135	8.1
I-580	0.040	1935	0.366	1183	0.224	409	30.0	0.148	8.9
Mountain Blvd	0.046	3272	0.620	1990	0.377	682	30.0	0.230	13.8

[1] Fall 2002 ACFCWCD K-factor = 15.22 + 2.1464 * L + (8.6981 / L); Spring ACFCWCD 2003 K-factor = 15.22 + 2.15 * (8.7 / L)

[2] ACFCWCD Time Lag = K-factor * N * [(L * Lc) / (S ^ 0.5)] ^ 0.38

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Table 6. Time Lag Estimations, Proposed Developed Conditions

SubBasin	Basin "N"	Length, L (ft)	Length, L (mi)	Length to Centroid, Lc (ft)	Length to Centroid, Lc (mi)	Slope, S (ft/mi)	K-factor[1]	Time Lag[2] (hr)	Time Lag[2] (min)
County, 10/13/02									
Ridgemont1	0.024	3953	0.749	1451	0.275	245	28.4	0.132	7.9
Ridgemont2	0.060	1739	0.329	877	0.166	1686	42.3	0.205	12.3
Development	0.028	4556	0.863	1522	0.288	727	27.2	0.128	7.7
I-580	0.040	1935	0.366	1183	0.224	409	39.7	0.196	11.8
Mountain Blvd	0.046	3272	0.620	1990	0.377	682	30.6	0.234	14.1
BH, 10/14/02									
Ridgemont1	0.023	3872	0.733	2216	0.420	817	28.7	0.118	7.1
Ridgemont2	0.060	1649	0.312	716	0.136	1825	43.7	0.190	11.4
Development	0.023	4581	0.868	2975	0.563	680	27.1	0.138	8.3
I-580	0.037	1466	0.278	938	0.178	540	45.0	0.161	9.6
Mountain Blvd	0.038	4160	0.788	2925	0.554	529	28.0	0.235	14.1
BH, Spring 2003									
Ridgemont1	0.024	3953	0.749	1451	0.275	245	28	0.132	7.9
Ridgemont2	0.060	1739	0.329	877	0.166	1686	30	0.145	8.7
Development	0.028	4556	0.863	1522	0.288	727	27	0.128	7.7
I-580	0.040	1935	0.366	1183	0.224	409	30	0.148	8.9
Mountain Blvd	0.046	3272	0.620	1990	0.377	682	30	0.230	13.8

[1] Fall 2002 K-factor = 15.22 + 2.1464 * L + (8.6981 / L); Spring 2003 K-factor = 15.22 + 2.15 * (8.7 / L)

[2] Time Lag = K-factor * N * [(L * Lc) / (S ^ 0.5)] ^ 0.38

PWA

Table 7. HEC-1 Results

Conditions		Storm	BH July, 2001	County 10/13/02	BH 10/14/02	PWA November 20, 2002, recommended changes to BH (10/14/02) model: Mtn. Blvd. CN = 80; & Ponds 1&2, no initial vol.	BH Spring 2003
Existing Conditions	Discharge, cfs	24-hr, 2-yr					71
		24-hr, 5-yr					112
		24-hr, 10-yr					139
		24-hr, 25-yr	276	181	167	158	168
		24-hr, 100-yr	368	233	214	212	224
Developed Conditions	Discharge, cfs (additional Ridgemont, 4.5ac)	24-hr, 2-yr					70
		24-hr, 5-yr					112
		24-hr, 10-yr					137
		24-hr, 25-yr	173	191	163	167	163
		24-hr, 100-yr	335	246	207	210	224
	Discharge, cfs (NO additional Ridgemont)	24-hr, 25-yr			162		
		24-hr, 100-yr			201		

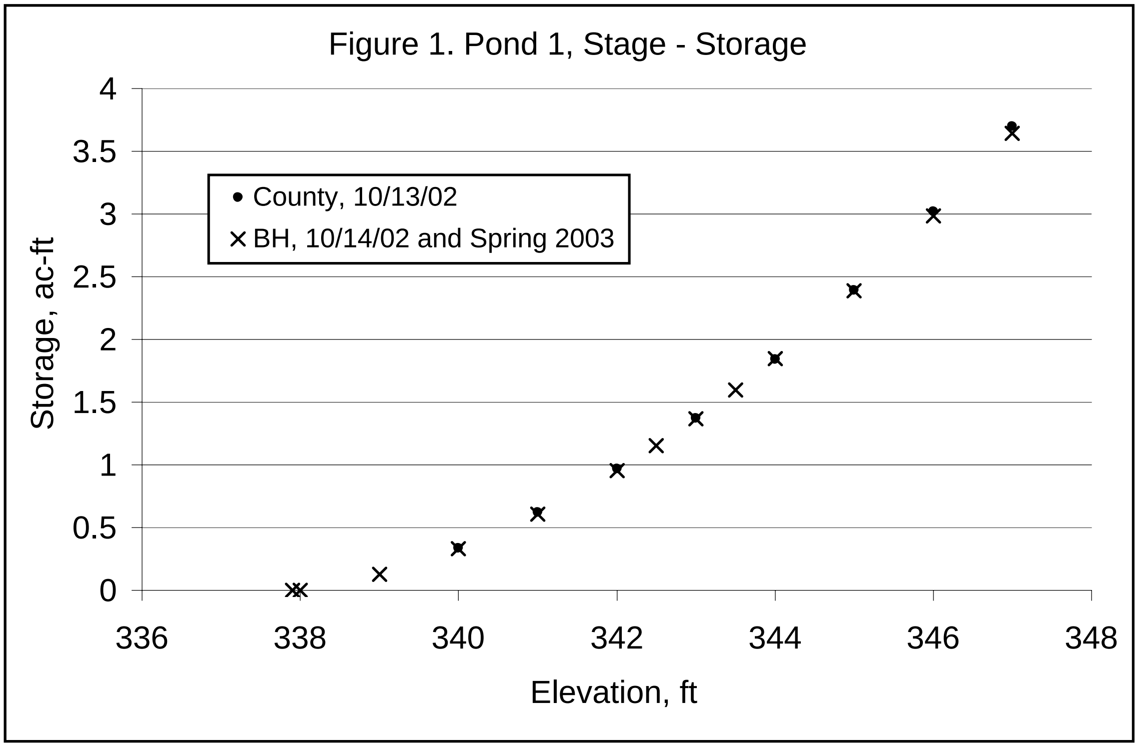


Figure 1. Pond 1, Stage - Storage
Storage, ac-ft
Elevation, ft
County, 10/13/02
BH, 10/14/02 and Spring 2003
0
0.5
1
1.5
2
2.5
3
3.5
4
336
338
340
342
344
346
348

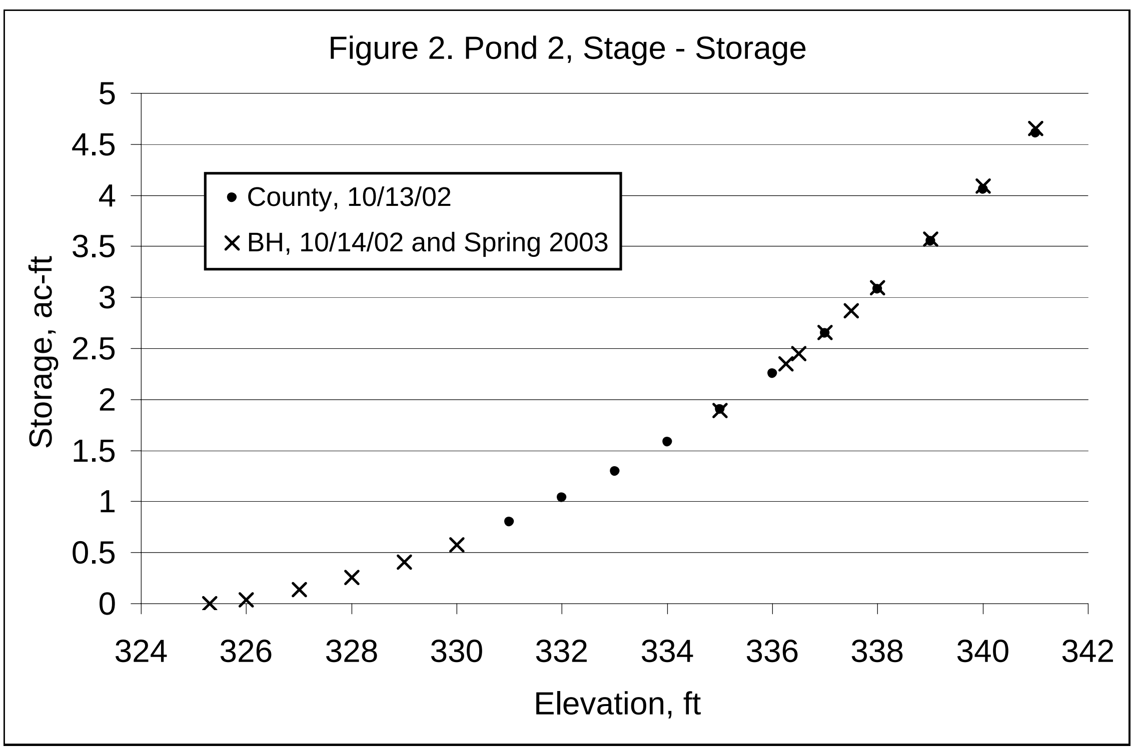


Figure 2. Pond 2, Stage - Storage
Storage, ac-ft
Elevation, ft
County, 10/13/02
BH, 10/14/02 and Spring 2003
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
324
326
328
330
332
334
336
338
340
342

PWA

Figure 3. Pond 1, Rating Curve

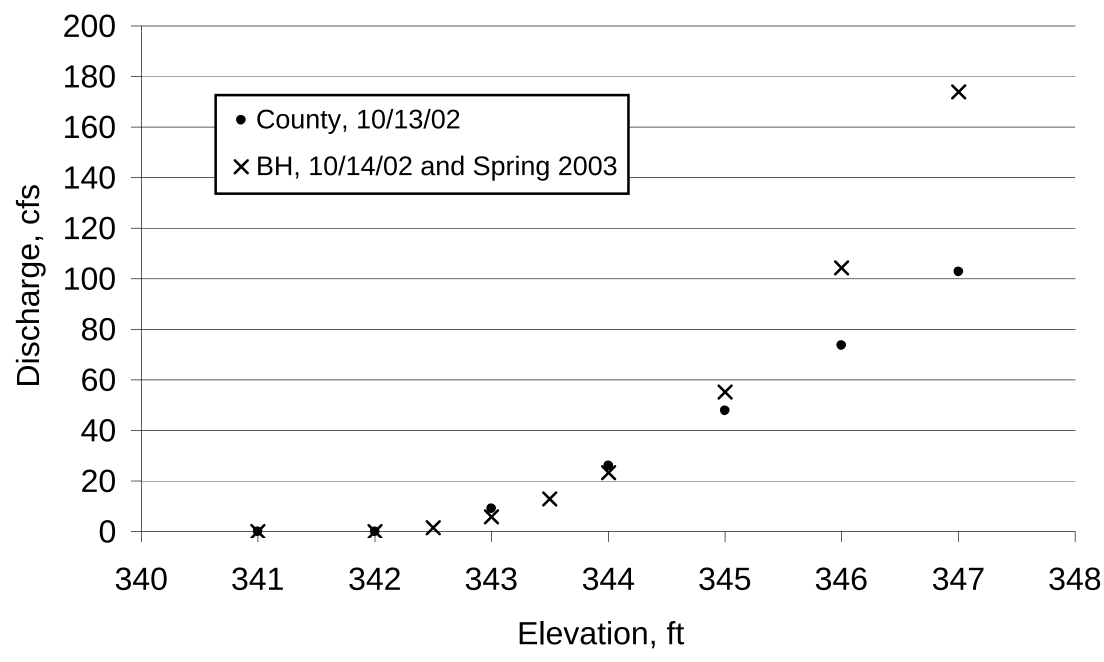
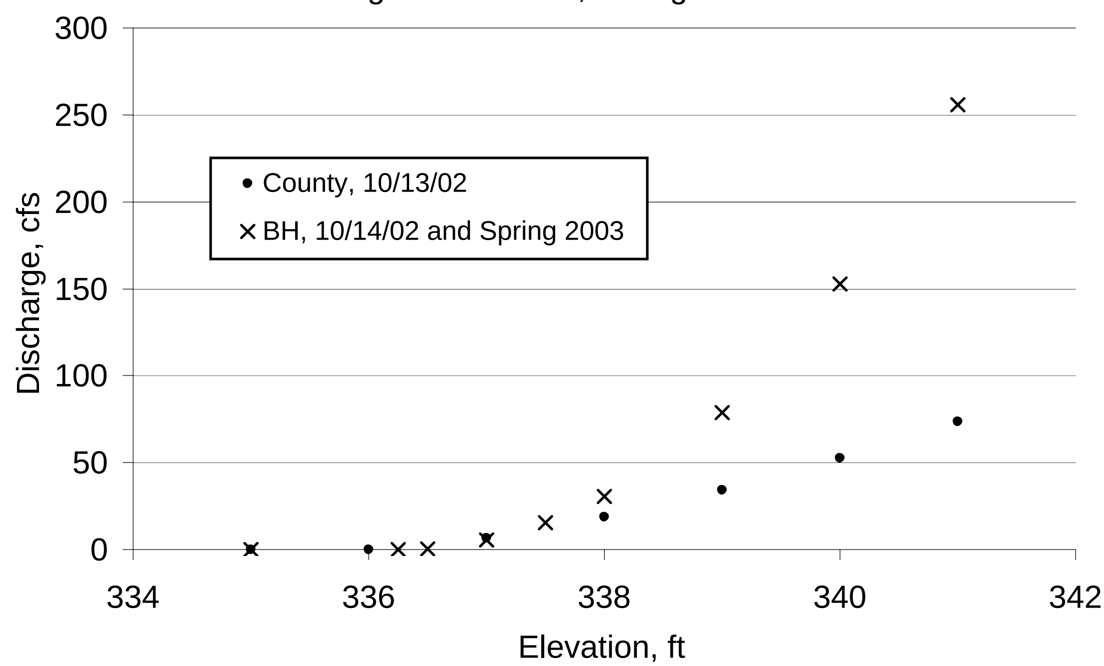


Figure 4. Pond 2, Rating Curve



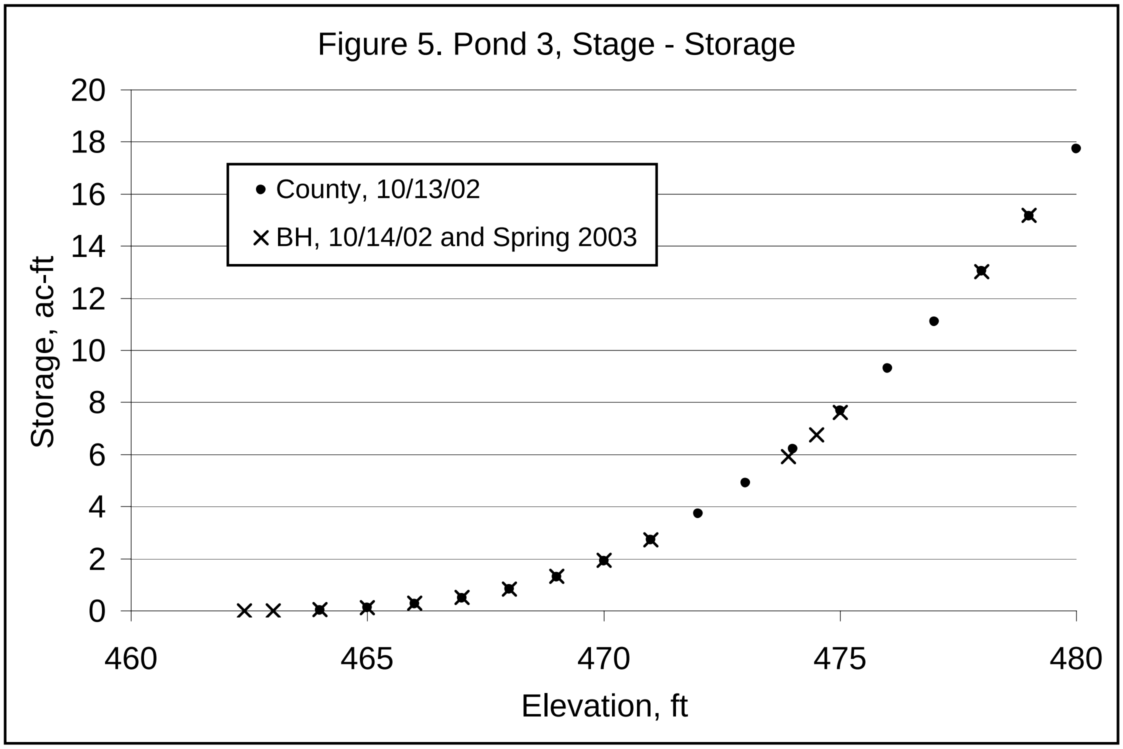


Figure 5. Pond 3, Stage - Storage
Storage, ac-ft
Elevation, ft
County, 10/13/02
BH, 10/14/02 and Spring 2003
0
2
4
6
8
10
12
14
16
18
20
460
465
470
475
480

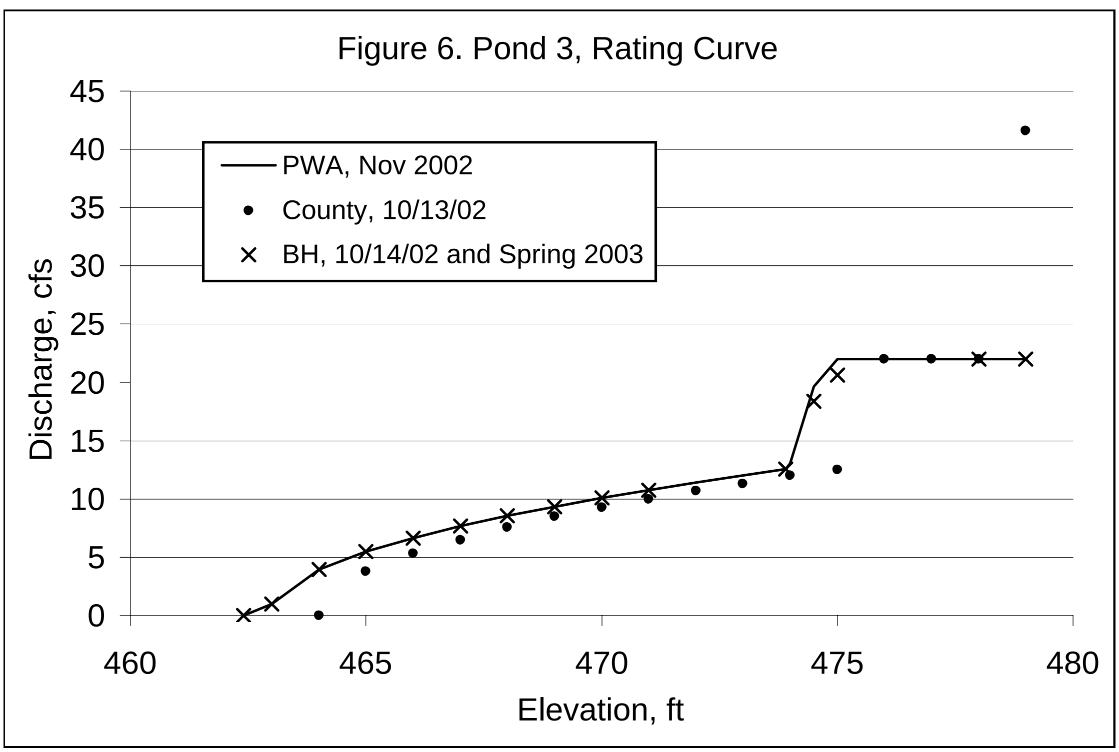


Figure 6. Pond 3, Rating Curve
Discharge, cfs
Elevation, ft
PWA, Nov 2002
County, 10/13/02
BH, 10/14/02 and Spring 2003
0
5
10
15
20
25
30
35
40
45
460
465
470
475
480

PWA

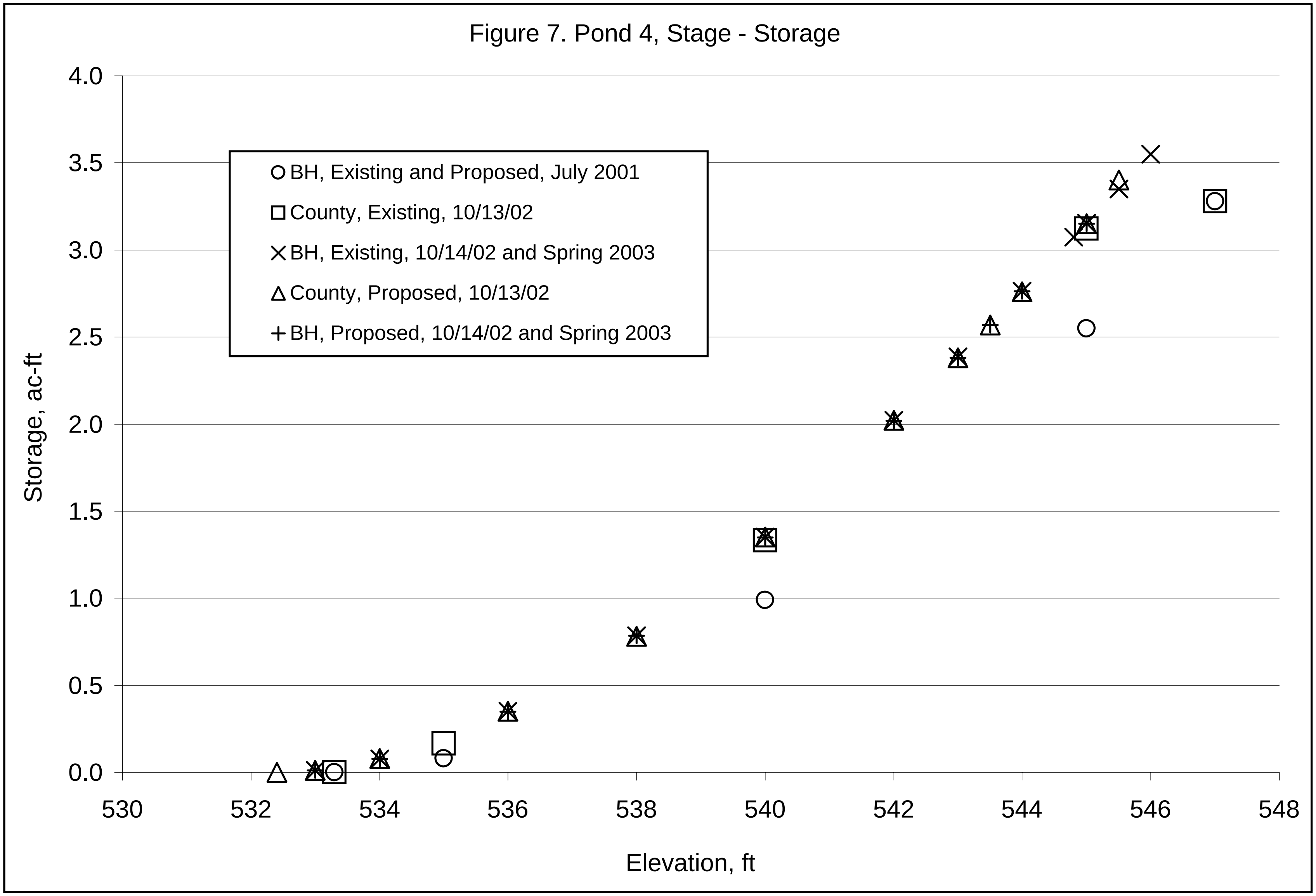


Figure 7. Pond 4, Stage - Storage
Storage, ac-ft
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
530
532
534
536
538
540
542
544
546
548
Elevation, ft
BH, Existing and Proposed, July 2001
County, Existing, 10/13/02
BH, Existing, 10/14/02 and Spring 2003
County, Proposed, 10/13/02
BH, Proposed, 10/14/02 and Spring 2003

PWA

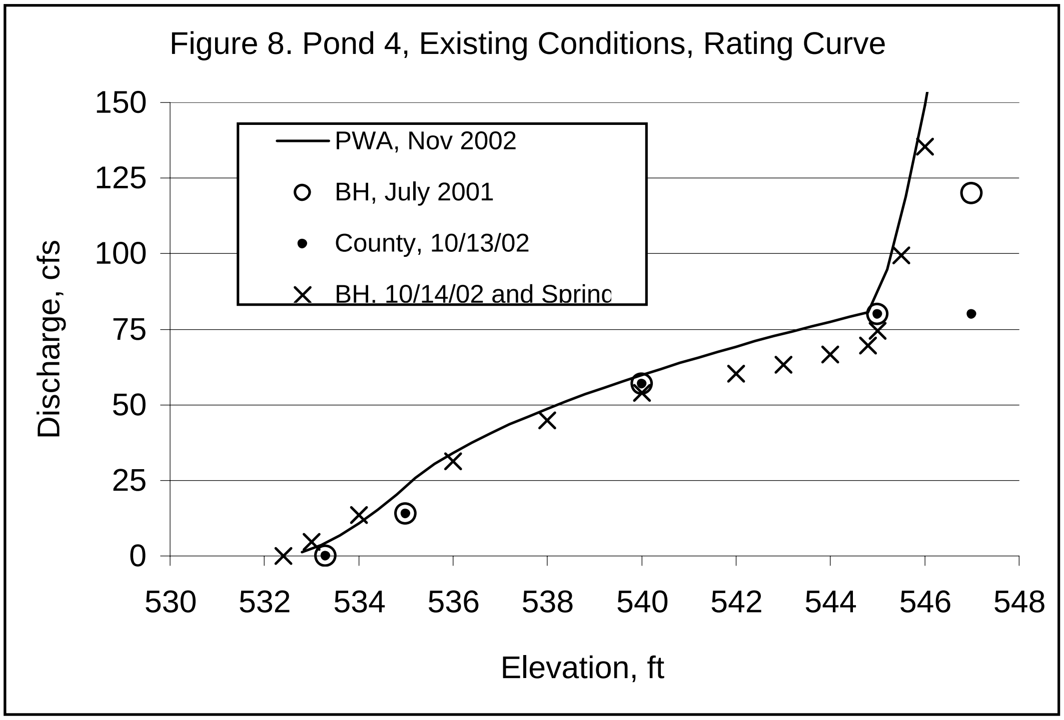


Figure 8. Pond 4, Existing Conditions, Rating Curve
PWA, Nov 2002
BH, July 2001
County, 10/13/02
BH, 10/14/02 and Spring
Discharge, cfs
Elevation, ft
150
125
100
75
50
25
0
530
532
534
536
538
540
542
544
546
548

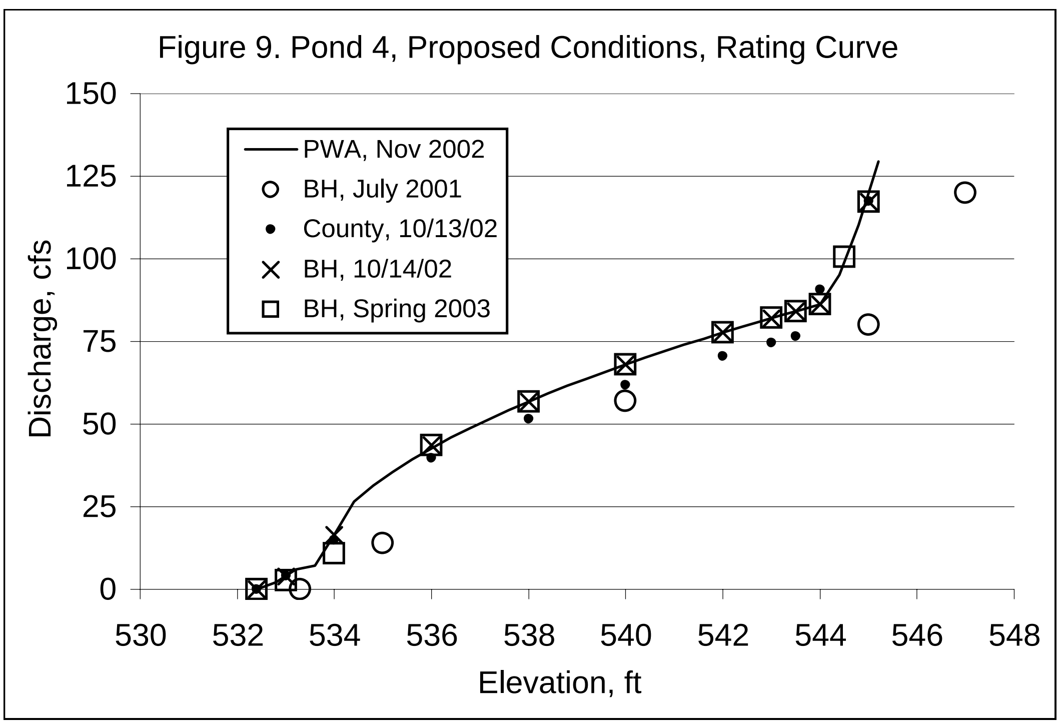


Figure 9. Pond 4, Proposed Conditions, Rating Curve
PWA, Nov 2002
BH, July 2001
County, 10/13/02
BH, 10/14/02
BH, Spring 2003
Discharge, cfs
Elevation, ft
150
125
100
75
50
25
0
530
532
534
536
538
540
542
544
546
548

PWA

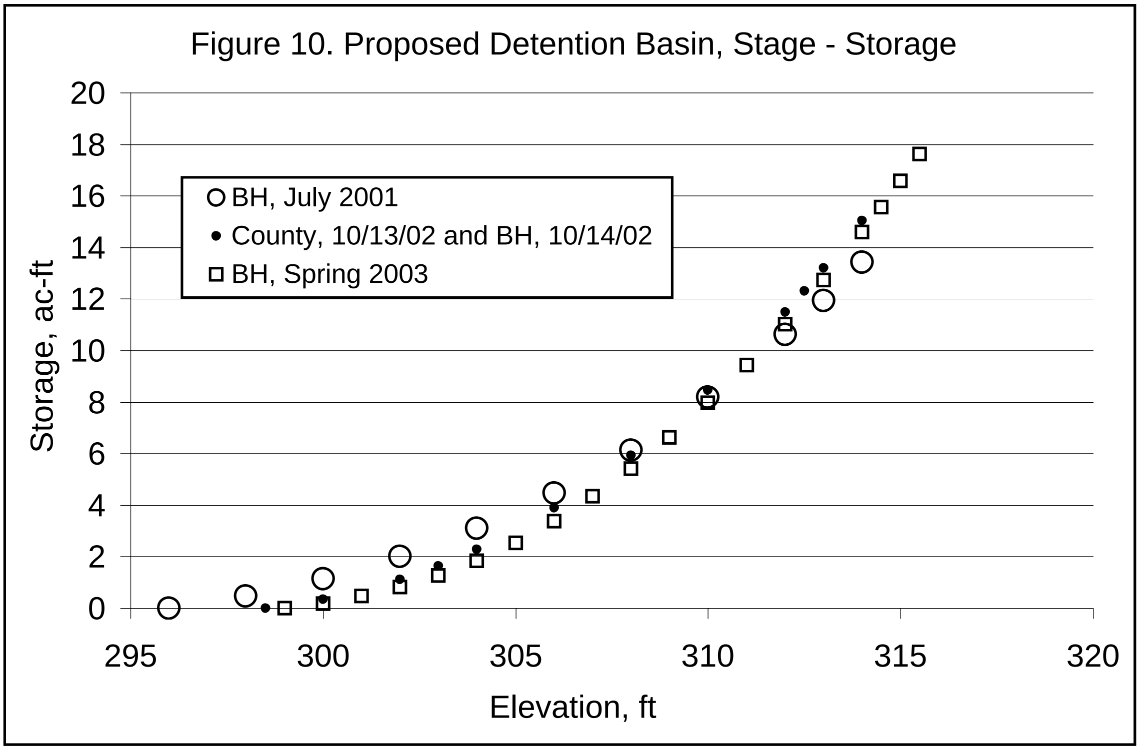


Figure 10. Proposed Detention Basin, Stage - Storage
BH, July 2001
County, 10/13/02 and BH, 10/14/02
BH, Spring 2003
Storage, ac-ft
Elevation, ft
0
2
4
6
8
10
12
14
16
18
20
295
300
305
310
315
320

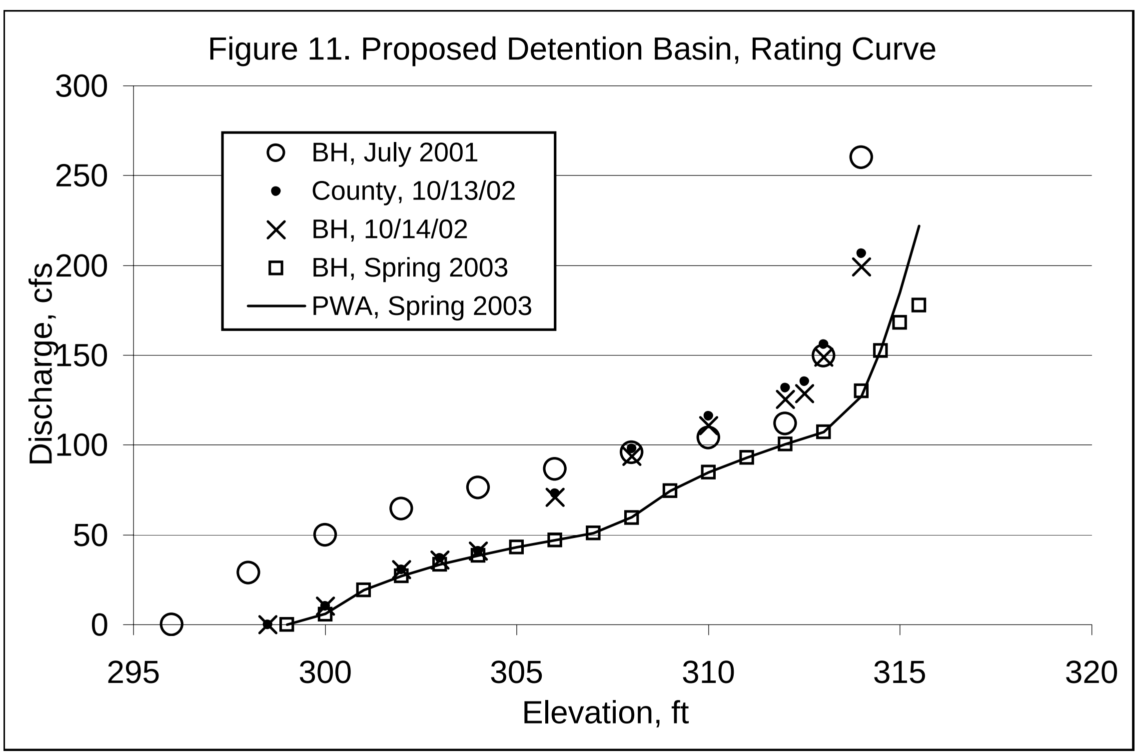


Figure 11. Proposed Detention Basin, Rating Curve
BH, July 2001
County, 10/13/02
BH, 10/14/02
BH, Spring 2003
PWA, Spring 2003
Discharge, cfs
Elevation, ft
0
50
100
150
200
250
300
295
300
305
310
315
320

PWA