RETURN

EVERGREEN • EAST HILLS VISION STRATEGY

SAN JOSÉ, CALIFORNIA

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EIR

APPENDIX

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HYDROLOGY &

WATER QUALITY REPORT

Evergreen Area Visioning Project Plan
(Evergreen Smart Growth)
Surface Water Hydrology
Flooding and Drainage Impact Analyses

Background and Issues

Background

The analyses completed for the flooding and drainage impacts for the Evergreen Area Visioning Plan (Evergreen Smart Growth – ESG) were coordinated with the City of San Jose and the Santa Clara Valley Water District through a series of meetings held alternatively at the City of San Jose offices and at the offices of the Santa Clara Valley Water District. Issues were discussed at these meetings, guidance for analyses was proposed and discussed, and intermediate results presented and discussed. These meetings formed a basis for the analyses presented in the impact report.

While some intermediate results were presented at those meetings, not all complete results were presented. Guidance as proposed and accepted was used in the analyses contained in this report.

The project alternatives and the number of various owners helped make this project unique. In all, six alternatives were established including the "do nothing" alternative. However, that particular alternative would allow the current zoning to remain in place for the properties. A seventh alternative was added. Called the "Existing Conditions" alternative, it was the alternative against which all impacts were measured.

The properties included in the ESG were: Pleasant Hills Golf Course; Evergreen Valley Community College; a 147-acre piece considered in the analysis as three separate pieces owned by Berg (called Berg South), IDS and Legacy (called Legacy North); a 92-acre site owned by Berg (called Berg North); Arcadia, and Legacy (called Legacy South).

These eight sites and seven alternatives make this project quite unique and potentially quite cumbersome to deal with. However, while rigorous analyses were completed for all sites for some of the land use alternatives that defined the upper and lower limits of urban density. Results for other intermediate land use densities were interpolated with impacts and mitigation measures believed to be reasonably accounted for.

Issues

There are three broad issues and one auxiliary issue that are covered by the various analyses contained in this report. The **first** issue is that of "Local Runoff Control." This is a rather new type of issue for the San Francisco Bay Area. The issue is the control of the peak discharge of runoff from newly developed sites as well as the control of the volume of runoff from those sites so that the receiving watercourses will have little impact to changes in sediment transport as well as to stream bed and stream bank erosion. Erosion materials in the runoff waters are believed to be a detriment to water quality and as such need to be controlled to protect the quality of runoff water flowing to San Francisco Bay.

This Hydromodification Management Plan (HMP) was analyzed based on the continuous simulation model developed for the *Hydromodification Management Plan Final Report* dated April 21, 2005 and prepared for the Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP). This plan proposes that post-development runoff mirror the "flow-duration" curve of existing conditions from 10 percent of the 2-year peak discharge up to the 10-year peak discharge. In addition, timely discharge of site drainage waters into receiving waters is necessary to minimize vector control issues in these detention basins.

The **second** issue to be addressed was change or changes to the downstream flood potential. This analysis was completed using a hydrologic provided by the Santa Clara Valley Water District according to guidance given at a joint City-District-Developer-EIR meeting. This Santa Clara Valley Water District hydrologic model represented existing land use plans and predicted 100-year design discharges on all watercourses in the Evergreen area. The model also predicted the 100-year design discharge and inflow hydrograph to Lake Cunningham under fully developed conditions under existing land use plans.

Lake Cunningham is a key element of the flood control system for Lower Silver Creek. The lake, a multi-purpose facility that incorporates recreation into the flood control function, is an off-channel storage device that "scalps" inflows and prevents downstream discharges from exceeding a limit that matches downstream design channel capacity. As the downstream Silver Creek channels are currently under construction, the existence of Lake Cunningham is a given, important portion of the overall flood control system for Lower Silver Creek.

The **third** issue to be addressed was the size and function of debris basins located at the upstream termini of the two flood control pipelines for Fowler Creek and the South Branch of Fowler Creek. Some type of debris basins have been shown on preliminary Santa Clara Valley Water District plans for over 30 years. The purpose of these facilities is to keep large debris and sediment from entering the pipelines. Such incursion into these lines has the potential to block these transmission facilities and/or reduce their capacities and thus create a flood potential for downstream residents.

The sizing criteria have been discussed with Santa Clara Valley Water District for many years. Final results are published in this report.

The one **auxiliary** issue discussed in this report is the size of the Fowler South Debris Basin and its fitness to act as a mitigation measure for the rupture of a water tank constructed in the watershed above the proposed basin.

Control of Local Runoff

The control theory behind the Hydromodification Management Plan is that downstream watercourses will not undergo any increased erosion potential if the "flow-duration" curve of a discharge point is identical to the curve under existing runoff conditions. This control theory assumes that something can be done with the additional runoff generated by the creation of impervious surfaces which are constructed as part of the development process of urbanization. The control theory as to be practiced in Santa Clara County as detailed in the April 21, 2005 *Hydromodification Management Plan* is that all future discharges between 10 percent of the 2-year peak discharge under existing conditions and the 10-year peak discharge under existing conditions must not occur any more often than they do at present. The findings of the April 21, 2005 report were that any discharges greater than 10 percent of the existing 2-year peak discharge would cause movement of sediment along the bed and along the lower portions of the banks of a natural water course. The more often such discharges would occur, the more the erosion of bed and banks creating down-cutting and/or bank erosion. Such eroded materials would be carried downstream in runoff waters thus adding sediment to those runoff waters and thus impairing the quality of that runoff.

There are two ways to match the "flow-duration" curve of existing conditions. The first is to hold the runoff in detention basins and release it at a rate that does not exceed the "flow-duration" curve of pre-project conditions above 10 percent of the 2-year existing peak discharge. The second is to allow percolation into the ground to allow the excess runoff to recharge the groundwater basin.

For the control strategies used in the Evergreen Smart Growth program the most often used control mechanism was the first with percolation limited to those sites where the existing soils conditions were of the B-type hydrologic group and thus quite permeable. In general the control mechanism tried to limit the runoff that was recharged so as to minimize any adverse impacts on the groundwater quality over the long term.

The procedures and parameters used to develop approximate sizing for the properties in the ESG are shown in the various documents in Appendix A. The basis for the computations was the HEC-HMS continuous simulation model. Parameters for infiltration and soil percolation were taken from existing GIS files for soils characteristics in Santa Clara County. These GIS data were presented by the Santa Clara Valley Water District.

Appendix A begins with Appendix A-1, a recap memorandum showing the results for each property for each land use scenario. For most of the land use scenarios, the HMP basin area is shown with the symbol "<" which means "less than." Sizes are given for current zoning (Scenario I) and for the maximum density land use scenario (usually Scenario VI).

Appendix A continues with a separate technical report for each property showing the parameters used in the continuous simulation analysis and showing the matching of the "flow-duration" curve with proper implementation of detention basin and outlet works.

Discharge by Percolation

For four properties – Arcadia, IDS, Evergreen Community College, and Berg North (Appendices A-2, A-3, A-6 and A-9) – the soil conditions at the proposed pond locations (which were generally situated at the downstream limits of the properties) were quite permeable, consisting of the B-type Hydrologic Group. (Except for Arcadia which was C-type Hydrologic Group.) To account for infiltration into the soil as part of the outlet capacity it was assumed that the average permeability of the entire local drainage basin applied to the HMP basin. This is generally an underestimate and generally speaking higher percolation capacity will cause the HMP basin size to decrease. It is recommended that site-specific percolation rates be determined for each HMP basin that is to incorporate percolation into its design outlet. In all cases the water table should be no closer than 10 feet from the bottom of the HMP basin that is to rely on percolation. (Soil borings on Berg North showed 50-foot deep borings without encountering groundwater.) Aside from these four properties, all other HMP basins were in much tighter soils and the model had no allowance for percolation into soils beneath the basin.

Time to Drain

Runoff could not be held in the detention basins for unlimited time. Vector control concerns required that stored water be discharged within a relatively short period of time. This parameter was tracked for each of the mitigation basins. It was found that in general the basins would drain within three days for most of the storms that would produce runoff over the simulated 53 years of rainfall data. Some of the properties, particularly Arcadia and Legacy South would take somewhat longer to drain. Legacy South (Appendix A-5) would take 7 days to drain but during the maximum event in 4 out of 10 years it would take approximately 6 days to drain. The reason for this longer drainage time is that the HMP basin drains a larger area than for existing conditions so that runoff could be placed into Evergreen Creek rather than flow into Yerba Buena Creek – a highly impacted, eroded channel. This diversion does make the HMP basin run longer because the new flows match the "flow-duration" curve of the Evergreen Creek drainage only. Arcadia would take 6 days to drain but would drain within 3 days in almost 80 percent of the years. The reason for this longer drainage time was the need for percolation through less permeable C-type soils in the local watershed.

In both of these cases the normal rainfall event that creates runoff would drain off in less than 5 days but in rare events the HMP pond could have water for a period of over five days. The water, however, would be moving as is would be draining and becoming shallower as time progressed. This is not considered a significant issue for these two HMP basins. All other HMP basins would drain well within a 5-day period.

Swapping HMP parameters

On the Evergreen Community College site (Appendix A—6) two HMP basins are proposed. One very small basin would be on the property proposed for commercial and high density residential development. The other HMP basin would convert the existing "pond" on the campus to an HMP basin. The report shown in Appendix A-6 shows that the total "flow-duration" curve for the entire site would be maintained if the basin configuration was implemented. As a benefit, the flow to Yerba Buena Creek would be reduced as would the flow in Thompson Creek below Yerba Buena Creek to the confluence with Evergreen Creek where the new development outfall would be located. The flows on Thompson Creek below Evergreen Creek would be unaffected by the mitigation provided by the two HMP basins.

Alternatives

Three alternatives were considered: combining Berg South, IDS and Legacy North into one drainage basin with one HMP basin that served the combined 147 acres; incorporating certain C.3 provisions the promote additional infiltration throughout the local drainage area; and using underground storage.

The HMP analysis for147-acre site (Berg South, IDS and Legacy North) is shown in Appendix A-11. There is, of course, some economy of scale when the drainage area increases. This is certainly an option that the three properties may wish to entertain.

The second alternative was to incorporate C.3 provision throughout the watershed rather than incorporating them all as part of the natural functioning of the HMP basins. An example is shown in Appendix A-10. The Berg North piece was assumed to have one-half of all impervious area in the watershed drain directly to pervious areas. This additional localized percolation dispersed throughout the watershed would have the effect of reducing the size of the HMP basin by approximately one third. This is certainly an alternative that would be available to each property requiring an HMP basin. Based on this result it is presumed that if all the impervious area could be drained to impervious areas the HMP basin size could be cut as much as two-thirds depending, of course, upon the ratio of the imperviousness in the watershed. The finding, however, is that localized percolation throughout the local watershed would reduce the size the required HMP basin.

The use of underground storage is certainly a possibility for all sites. This would reduce the surface area of any project given over to storage and/or percolation. With underground storage and/or percolation, use could be made of the overlying land use.

This may be of particular benefit on the IDS and Arcadia sites where the HMP basins taken up significant amount of the parcel size.

Conclusion

The conclusion of this continuous simulation analysis was that it was possible to construct HMP detention basins on each property and match the "flow-duration" curves of pre-project runoff. The most difficult properties on which to do this would be on the Arcadia parcel where more than 11.6 percent of the area is needed for the HMP basin and on the IDS parcel where 4.8 percent of the area is needed for he percolation basin. In these cases there could be the option of placing significant amounts of the needed storage in below-ground vaults or pipes. This would minimize the land given over to an open HMP basin but still retain the necessary storage.

Therefore, with the implementation of the HMP detention basins there are no downstream impacts to water quality in existing water courses due to increased erosion potential. These basins provide complete mitigation of any impacts as per the *Hydromodification Management Plan.* The HMP basins also completely mitigate all increases in discharge from the sites for all storms from 10-percent of the existing 2-year discharge to the 10-year discharge. Therefore, additional downstream mitigation is not required.

Downstream Flood Potential

A report on the flood hydrology for the Evergreen Area is shown in Appendix B. That report presents a detailed analysis of the flood control impacts of the maximum density development on the eight properties. The impacts and analysis were based on a hydrologic model developed by the Santa Clara Valley Water District for use in determining design discharges throughout the Evergreen Area downstream of Lake Cunningham.

Because the HMP procedures already discussed kept the post-development discharges equal to the pre-development discharges between 10 percent of the existing 2-year discharge and the 10-year discharge, it was considered proper to focus only on the 100-year flood when addressing the flood potential. With HMP in place it was evident that there would be no change in the hydrology up to and including a 10-year event. If there was little or no impact to the flood peaks or volumes at the 100-year event it would follow that there would be no intermediate impacts. If, however, there was a significant impact at the 100-year flood level then it would be appropriate to determine the frequency of induced downstream flooding and to propose mitigation measures to eliminate this impact.

After significant coordination with the Santa Clara Valley Water District the hydrologic model was prepared to simulate the 100-year flood for two conditions: existing conditions and developed conditions on the six sites according to the maximum density concept.

Tables 8 through 11 of Appendix B succinctly show the results for the 100-year flood. Table 8 shows that at all areas downstream of the six project sites the maximum increase in peak discharge would be 2 cfs out of 5,063 cfs under existing conditions at the total inflow to Lake Cunningham. This increase is a less than 0.1 percent increase. This insignificant increase in peak discharge would not create any noticeable changes downstream. It is also important to note that the 100-year peak discharges would not change the Santa Clara Valley Water District's original design discharges.

Table 9 shows the changes to the average 6-hour average discharge during a 100-year runoff event. Here the maximum change is 21 cfs a 0.5 percent increase in the existing 100-year value of 3,998 cfs. Again, this maximum increase is at a location that considers all the inflow to Lake Cunningham. Though unlikely, this increase may create a higher water surface level inside Lake Cunningham Park during a 100-year flood. Therefore, the impact cannot be determined just from the numbers themselves. The impacts on the Lake Cunningham water level and outflow will be discussed a little later in this report.

Tables 10 and 11 show the results for the 24-hour average flow and the 72-hour average flow. Here the maximum slight increases of a little more than 1 percent should not have any adverse impact as these flows generally stay within the Lower Silver Creek Channel.

The impacts to downstream flooding due to proposed maximum density development on the eight ESG sites are not significant.

The impact on the flood control function of Lake Cunningham will now be discussed to determine whether there is any significant increase in water level in the Lake and any significant increase to the outflow from the Lake.

Lake Cunningham is a flood control facility that is intended to "scalp" the peak discharge of the hydrograph coming into the Lake and put the "excess" flood waters into the volume provided by the Lake and the Park so that they may be "bled" out over a longer period of time. This off-channel flood control facility will allow the downstream channel to function as normal up until the capacity of that channel and then limit the amount of flow that can proceed downstream.

The Lake Cunningham flood control system was implemented in the mid to late 1970's and it is not clear that the facilities that are present today are the ones that will be present under ultimate conditions. Those facilities are: two inflow channels – one from Lower Silver Creek and one from Flint/Ruby Creek; a culvert under Cunningham Avenue that acts as a throttle to the flow and backs up the flood waters once culvert capacity is reached; weirs along the channels that send excess flow from the channels into the Lake and Park areas; storage in the Lake and Park areas; and outflow devices that move stored water from the Park and Lake areas back into Lower Silver Creek just upstream of the Cunningham Avenue culvert.

There are two relationships that need to be developed to route the flood through Lake Cunningham: storage and discharge. The two relationships are: storage as a function of elevation; and discharge as a function of elevation. In this special case a capacity parameter is also needed. This capacity figure is the maximum amount of flow allowed to pass through the culvert at Cunningham Avenue. This capacity value was estimated at either 2,000 cfs or 2,800 cfs. Both of these values can be inferred from Table 1 of the Flood Insurance Study report for the City of San Jose dated August 17, 1998. There the maximum discharge downstream of Lake Cunningham in Lower Silver Creek is listed as 2,580 cfs for the 100-year flood but slightly downstream at Ocala Avenue the discharge is listed as 2,000 cfs. It was clear from the Flood Insurance Study whether or not the 2,580 cfs was strictly confined to the flow through the culvert at Cunningham Avenue or whether it included the flow in the culvert and the flow leaving the Lake over the levees next to Cunningham Avenue. To make certain that the impacts were correctly accounted for on Lake Cunningham two different capacities were considered: the 2,000 cfs which is what the channel will hold at Ocala Avenue and 2,580 cfs which is the discharge listed by FEMA at Cunningham Avenue. The 2,000 cfs capacity is the more critical of the two for it would force more "scalped" flood waters into the lake proper and thus create a higher water elevation in the park.

The Lake Cunningham routing report is shown in Appendix C. In that appendix it is noted that the topography used to define the elevation-storage relationship was based on aerial photogrammetric topography dated June 2000. The elevation-discharge relationship was developed by assuming weir flow over the levees along Cunningham Avenue. There was no other outlet for the flood waters that spilled into Lake Cunningham Park.

Assuming the 2,800 cfs of flow in Lower Silver Creek before any flow is directed into Lake Cunningham Park, the existing and future land use conditions 100-year hydrographs only change the water surface elevation in Lake Cunningham Park by 0.1 feet from 129.8 feet (NAVD) to 129.9 feet (NAVD). The NAVD datum is 2.7 feet higher than the NAVD-29 datum used by FEMA for the Flood Insurance Study Rate Maps for the City of San Jose. These two elevations translate to 127.2 to 127.3 feet NGVD which are almost 4 feet lower than the elevation of 131 feet NGVD shown on the FEMA Flood Insurance Study Rate Maps. At those low levels no flow leaves Lake Cunningham Park by flowing over the levees along Cunningham Avenue.

Assuming the 2,000 cfs of flow in Lower Silver Creek before any flow is directed into Lake Cunningham Park, the maximum water levels in the park again change is less than 0.1 feet. It remains at 132.5 feet NAVD. Translated to the NGVD datum this elevation is 129.8 feet NGVD – lower than the FEMA published elevation but closer to it than with the 2,800 cfs flow in Lower Silver Creek.

In this second case of only 2,000 cfs of bypass down Lower Silver Creek, there would be 834 cfs of flow going over the levees onto Cunningham Avenue under either condition.

A second impact relating to Lake Cunningham is also under consideration. If the property denoted as Pleasant Hills Golf Course were to fill all their land that is below elevation 131.5 feet NGVD – that is, is believed to be in the existing FEMA floodplain – what would that do to the elevations in Lake Cunningham Park and what would it do to the amount of flood discharge leaving over the top of Cunningham Avenue.

The HEC-1 routing as detailed in the memo shown in Appendix D, indicates that the maximum water surface would change by 0.1 feet in the park itself and the overflow would increase from 834 to 835 cfs.

The conclusion reached is that there is no significant increase in downstream flooding due to the increased flows due to upstream urbanization or due to a combination of upstream urbanization coupled with filling the low-lying lands on the Pleasant Hills Golf Course.

Therefore, it is concluded that there are not unmitigated adverse impacts on downstream flooding due to changes in land use for the properties in the ESG.

Debris Basins

The upper limits of the proposed pipelines for Fowler Creek and the South Branch of Fowler Creek are expected to have debris basins constructed as part of the inlet works to these flood control pipelines. These debris basins have been shown in preliminary Santa Clara Valley Water District plans since the mid 1970's. These debris basins function to keep large objects out of the pipelines thus minimizing the potential for clogging. The sediments trapped in the debris basins would minimize the potential for siltation in the pipelines. The maintenance costs of clearing an open basin are less than those costs associated with clearing underground pipelines.

The size of these basins is an issue that was of concern to the Santa Clara Valley Water District as it was assumed that the operation and maintenance of these two debris collection facilities would reside with that agency.

Over the course of the past five years there have been many discussions with and reports submitted to the Santa Clara Valley Water District concerning the size of these debris basins. The final tentative agreement as to sizing was dependent upon the funding of annual maintenance/cleaning activities. The Santa Clara Valley Water District wishes to use a debris loading factor of 0.6 acre-feet/square mile of drainage area – year. Although this was shown to be too high based on data collected on nearby District debris facilities, the District wishes to use the 0.6 factor.

The District wished to have a size of basin based on 20 years of debris accumulation. Discussions with the staff of that agency resulted in a reduction to a 10-year accumulation factor if and only if the District was assured of adequate funding to clean the basins not less frequently than annually.

Appendix E contains some of the correspondence and reports on this issue that have occurred over the years. These reports show that there is a risk-based approach to the sizing of debris basins rather than assume each year will have the same amount of debris generated. The conclusion of these reports is that the proposed annual cleaning and the factor of 0.6 (acre-feet/square mile-year) combined with the10-year life will provide adequate protection to the downstream pipelines.

Auxiliary Issue

The Auxiliary Issue dealt with the use of the debris basin on the South Branch of Fowler Creek to contain a rupture of a 4 Million Gallon Water Tank situated in the small watershed above that debris basin. This was an issue as the debris basin now proposed is smaller than the one initially proposed due to the promise of the financial resources for annual removal of sediment accumulation.

Appendix F shows the routing of a tank rupture assumed to empty the tank in 30 minutes. This would be created by an instantaneously appearing 4-foot diameter hole in the bottom of the steel tank. (Ruptures of this type are virtually unknown in steel tanks.) However, after routing the flood through the debris basin which was assumed to be empty at the start of the rupture (annually cleared of debris and sediment) the outflow from the debris basin would be 382 cfs. The outlet pipe would account for slightly over 320 cfs. Therefore, approximately 50 cfs would flow over the spillway onto the streets of the propose development. At a 2 percent slope a standard 60-foot wide street would carry 50 cfs with the water approximately curb deep on each side of the street. Therefore, the overflow would stay in city right-of-way and would dissipate though the city's storm drainage system within a short distance from the debris basin.

The conclusion is that even with an unprecedented tank failure, the debris basin and downstream street pattern will completely mitigate this catastrophic event.

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Appendix A-1: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, PhD, PE	DATE:	Oct 13, 2005
FROM:	Stephanie Conran and Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	HMP Pond Sizes for *EIR* Alternatives		

Most of the Evergreen Visioning Project's scenarios detailed in the *EIR* entail significant changes to existing land uses, and these land use changes will affect changes in the volume and velocities of stormwater runoff from the project sites. Stormwater generated within a municipality and discharged to surface waters is regulated under the U.S. Environmental Protection Agency's NPDES permit program, which the California State and Regional Water Quality Control Boards administer in California. The San Francisco Bay Regional Water Quality Control Board (Regional Board) administers the NPDES permit under which the City of San José operates. The San José City Council recently adopted changes to its *Zoning Ordinance* and *Post-Construction Urban Runoff Management Policy* relevant to site development and its effects on stormwater runoff. These changes allow the City of San José to conform its policies to Provision C.3 of its NPDES permit.

In general, NPDES permits specify a permittee's allowed quality and quantity of stormwater discharge to surface waters. The intent of the NPDES regulation is to protect the Nation's surface waters from undue pollution. The Regional Board has furthermore utilized the NPDES program to reduce streambank erosion by requiring what are known as hydrograph modification – or hydromodification – management plans (HMPs) for new development and significant redevelopment projects. An HMP details how a project site will have its stormwater runoff mitigated so that its post-construction runoff equals in duration and flow the pre-project runoff rates and durations. Essentially, the volume and velocity of the runoff must stay the same within certain bounds. The hydrograph modification is achieved in practice mostly through the use of detention ponds on the project site. Stormwater collects in these "HMP ponds" before being discharged to the pre-project discharge point, usually a creek or river. The HMP ponds have outlets of varying sizes to regulate the volume and velocity of effluent depending on the height of the pond.

Provision C.3 and the City Council's policy changes require specifically that the developed site must have an equivalent or smaller runoff-duration curve as compared to the pre-project site for runoff falling between the pre-project's 10-year flow and 10% of the 2-year flow. It is thought that flows below 10% of the 2-year flow do not significantly erode the receiving streams or rivers, and the effects of stormwater runoff greater than the 10-year flow can be modeled by traditional flood analyses. Indeed, Schaaf & Wheeler has also analyzed the flood effects of the developments; the results are detailed in another section of the *EIR*.

The *EIR* development scenarios for each site entail different densities of development and, consequently, different amounts of impervious area. However, for each site, two bounding development scenarios – one with the least new development and one with the most - can be identified. To expedite the HMP analyses of the different scenarios, Schaaf & Wheeler has analyzed the bounding scenarios only and designed detention ponds for those scenarios. The HMP pond sizes for the scenarios between the bounding scenarios are assumed to be between the bounding scenarios' HMP pond sizes.

This memo details the HMP pond results for the various *EIR* alternatives. Also included is a brief note about each site's current condition, whose maintenance will be called the "status quo" scenario. For a couple of the sites, the *EIR*'s Scenario I corresponds to the status quo scenario, which – by definition – does not require an HMP pond. Our previous HMP pond calculations for some of the alternatives complement the analyses discussed herein. Indeed, for each site, it is assumed that either Scenario IV or V corresponds to the development plans used in our earlier calculations, depending on which one involves the densest development. The results from the highest density development scenario are emboldened in the table; this table gives a summary of all the results from the recent analysis and our other calculations. The summarized results are then discussed site by site. The memos detailing our overall HMP procedures and the results of our previous analyses are given in the appendices to this report. The only difference in the previous analyses and those for the remaining scenarios is the assumed percent impervious. This memo gives those numbers for the bounding analyses. Therefore, any of our analyses whose results are given below can be recreated by combining information from the appended memos and this one.

Note that the *EIR* divides the sites by property ownership and not by drainage groups as Schaaf & Wheeler has done. Therefore, it was assumed that the detailed development for the IDS, Berg, and Legacy properties was evenly spaced throughout the various drainage basins. This seems like a safe assumption given that these properties are developed similarly under each scenario.

The method used to change the HMP basin area was that of percent of impervious surface under the land use scenario. For Scenario V (IV for Arcadia) the HMP basins were subjected to a full-blown analysis using the continuous simulation based on 53 years of rainfall record at the San Jose rain gage. The other values shown in the table below were based on the difference of percent imperviousness compared to the percent imperviousness used in the detailed analysis.

Table of Maximum Pond Areas *EIR* Alternatives (Acres)

	Arcadia	Pleasant Hills	EVCC	IDS	Berg N.	Legacy S.	Legacy N.	Berg S.
0*	0	0	0	0	0	0	0	0
I	1.9	0	0	1.8	4.2	2.9	0.6	2.1
II	< 9.3	< 1.3	< 0.3	<1.2	< 2.8	< 1.9	<0.4	<1.4
III	< 9.3	< 1.3	< 0.3	<1.2	< 2.8	< 1.9	<0.4	<1.4
IV	**9.3**	< 1.3	< 0.3	<1.2	< 2.8	< 1.9	<0.4	<1.4
V	< 9.3	**1.3**	**0.3**	**1.2**	**2.8**	**1.9**	**0.4**	**1.4**
VI	< 9.3	<1.3	<0.3	<1.8	<4.2	<2.9	<0.6	<1.4

* - Scenario 0 is the status quo scenario, corresponding to the existing conditions, not the maximum possible build-out under current zoning.

Arcadia

Even though its existing zoning allows some development, the Arcadia site currently has no urban development. It is thus assumed to be completely pervious for the status quo scenario. Scenario I entails development to 17% imperviousness according to the existing zoning. The pond required for this scenario is 1.9 acres . Our original analysis of the Arcadia site assumed a high percent of imperviousness due to the commercial and high-density residential development. That analysis, as mentioned above, corresponds to the *EIR*'s Scenario IV. The HMP pond for this scenario is 9.3 acres , and is detailed in Appendix A-2. Scenarios II, III, V, and VI are similar to IV but with less dense housing development. Therefore, these scenarios each require a HMP basin less than 9.3 acres.

Pleasant Hills Golf Course

The Pleasant Hills Golf Course currently has some imperviousness due to its development as a golf course. Scenario I for the course is the status quo scenario; thus, no HMP pond would be required. Scenarios V and VI are identical and entail high-density residential development requiring a pond of 1.3 acres. Scenarios II through IV entail less dense residential development as compared to V and VI and so require ponds smaller than 1.3 acres.

Evergreen Valley Community College

Scenario I for the community college (EVCC) does not require a pond since it is the status quo scenario. Scenarios II through VI entail development of the site giving a maximum of about 80% imperviousness. To help keep the project site's pond small, an existing pond on another part of campus will be modified to mitigate most of the runoff from the other site. Such mitigation fits with the spirit of the HMP requirements since both sites runoff to the same creek, the more mitigated runoff is upstream of the project's runoff, and the combined mitigated flows still meet the HMP requirements as compared to the pre-project combined flows. Given the use of the other, non-project site, all of the project scenarios include a slight increase in height (and volume) to the existing 2.3-acre pond on the other site. The densest development of EVCC, under Scenarios V and VI, requires a pond of 0.3 acres on the project site, and is detailed in Appendix A-6. The other three project scenarios (II, III, IV) entail less dense development and therefore require slightly smaller ponds on the project site.

Berg South, Legacy North, and IDS)

Each of these sites have a status quo scenario that involves no urban development. Scenarios I and VI involve the existing zoning and commercial development of the site. Such development requires a 2.1 acre pond for Berg South, .6 for Legacy North, and 1.8 for IDS. For the other scenarios II through V, these sites would have fairly high-density residential development. Scenario V has the densest development and requires the 1.4 acre pond for Berg South, 0.4for Legacy North, and 1.2 for IDS, as detailed in Appendices A-8, A-4, and A-3 respectively. Scenarios II, III, and IV each require a bit smaller pond volume than Scenario V.

Berg North

The 92-Acre site is the northern part of the Berg property. The status quo scenario has no urban development. Scenarios I and VI, involving dense commercial development at 75 percent imperviousness, requires a pond of 5.6 acres. Scenario V requires a pond of 4.2 acres, as detailed in Appendix A-9. Scenarios II, III, and IV each require somewhat smaller ponds than that for Scenario V.

Legacy South

The status quo scenario for the southside of the Legacy property (Legacy South) entails no urban development. Currently, parts of Legacy South drain to two different creeks – Yerba Buena Creek and Evergreen Creek. After grading is done to develop the site, most of the post-construction drainage will flow into Evergreen Creek. The remaining flow into Yerba Buena will not require an HMP pond. For the Evergreen drainage, the commercial development of Legacy South under Scenario I and VI requires a pond of 2.9 acres. Scenario V requires a pond of 1.9 acres, as detailed in Appendix A-5. Scenarios II, III, and IV each require somewhat smaller ponds than that for Scenario V.

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Appendix A-2: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	Oct 13, 2005
FROM:	Stephanie Conran and Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	HMP Pond Design Specs for Arcadia Property		

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the Arcadia Property in San Jose. Our analysis was complicated by several factors. The existing land use is mostly open space, and the proposed land use is mostly commercial and thus mostly impervious. However, a satisfactory detention basin was designed and is detailed in this memo. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

RECOMMENDATIONS

A pond area of about 11.6% of the total project site will be needed to meet the HMP requirements. It was assumed that the basin floor will not be graded and that the side slopes will have a ratio of 2:1. The recommended pond depth is 1'10". The other specifics of the basin's design are described below in Table 1.

Table 1 – Detention Basin Details

Maximum Pond Area	9.3 acres (630' x 630' at bottom)
Maximum Pond Depth	1'10"
Time to Drain	5.99 days max
Weir #1 and Weir #2	Width = 8' Invert above pond bottom = 1'6"

Figure 1 shows the project site flow-frequency curves with the basin routing using MSExcel and HMS. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event.

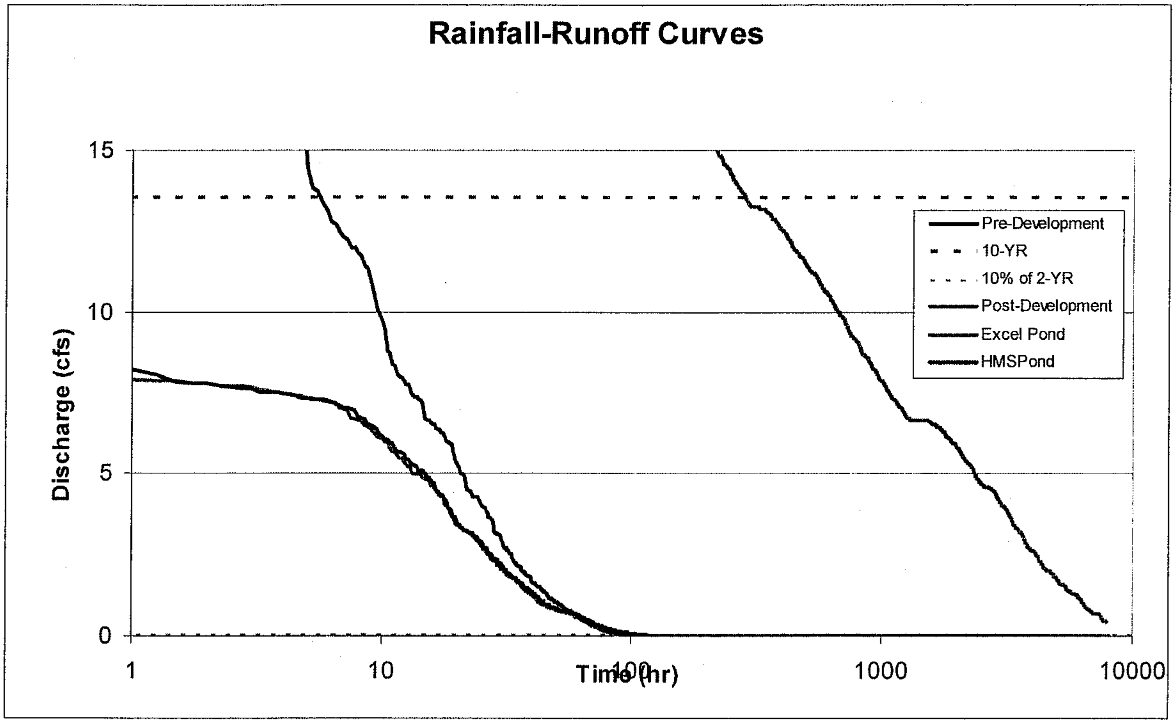

Figure 1. Hydrographs of Arcadia Site

SITE DETAILS

The Arcadia site currently has no urban development, so all of the pre-development area is assumed to be pervious. The site is underlain by Zamora-Pleasanton soil, which is a Group C soil of medium porosity. Ruth and Going, Inc. provided us with a drawing and calculations of the proposed land use. Combining those numbers with the impervious percentages from the Santa Clara Valley Water District's Hydrology Procedures manual, it was calculated that the developed area would be 76.7% impervious and 23.3% pervious.

MODELING OVERVIEW

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that will conform the post-development hydrograph to the pre-development hydrograph for the Arcadia site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve. Specifically between the flows of the 10-year flood and 10% of the 2-year flood, the post-development hydrograph must match or be less than the pre-development curve. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for each drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. Although on average the designed pond should drain within that time frame, there are times when it will take longer to drain. However, we assume that since the ponded water will be continuously flowing out of the pond, it will not really ever be standing water. Furthermore, we have presented at the end of this memo a graph and discussion explaining that the longer drain times occur only infrequently each year.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions. Existing conditions consist of a single basin. For the proposed conditions the model was broken into two basins, a pervious one and an impervious one. The sum of the hydrologic basin areas was set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 14.8 inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was adjusted from its MAP of 14 inches. Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows:

$$T_c\ (\text{hrs}) = K^*(n^*L)^{0.47*}S^{-0.235},$$

where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\ \right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\ \right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal to one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the Arcadia site is underlain by only Zamora-Pleasanton soil. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 2 summarizes the various parameters we input into the HMS model for the project site. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 2. Arcadia HMS Parameters.

		Pre	Post
Area (acres)		80.64	
% Pervious		100	23.3
C Soil (%)		100	
T_c (hours)	Pervious	0.586	
	Impervious	N/A	0.325
R (hours)	Pervious	0.746	
	Impervious	N/A	0.217
Canopy Storage Capacity (in)	Pervious	0.31	0.13
	Impervious	0	0
Surface Storage Capacity (in)	Pervious	0.375	
	Impervious	0.1875	
Soil Infiltration Max. Rate (in/hr)	Pervious	0.255	
	Impervious	0	
Soil Storage Capacity (in)		14.9	
Soil Tension Zone Capacity (in)		6.8	
Soil Percolation Max. Rate (in/hr)		0.170	
Groundwater Storage Capacity (in)		50	
Groundwater Percolation Max. Rate (in/hr)		0.10	
Groundwater Storage Coefficient (hr)		999	

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. The 10-year flow was calculated as 13.54 cfs, and the 2-year was calculated as .24 cfs, giving .02 cfs as 10% of the 2-year. Figure 2 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines.

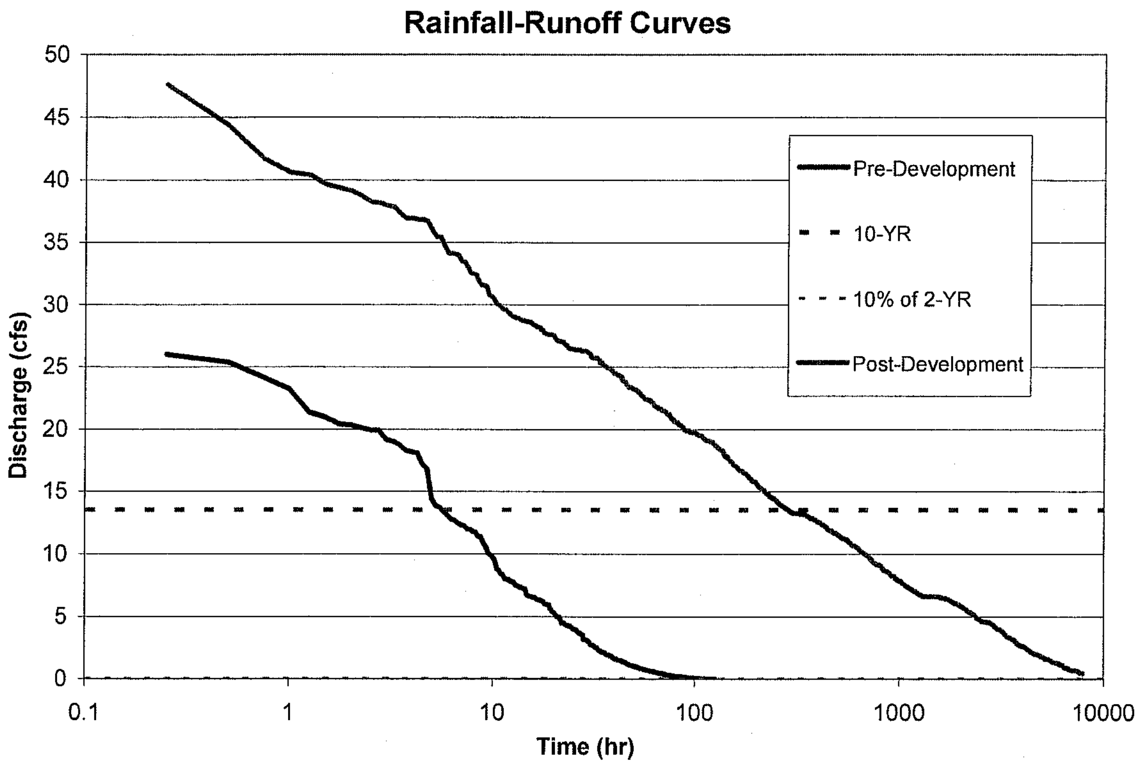

Figure 2. Hydrographs for Arcadia Site

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed.

A combination of artificial and natural outflows was modeled to drain the basin. Specifically, weirs and orifices with a small amount of percolation and evaporation were used. We used about 0.17 in/hr of natural percolation outflow from the basin bottom area of 8.4 acres and 0.00208 in/hr of evaporation. Also, this percolation rate was also used while calculating the time to drain for the basin. Weir outflow was based on the following equation:

$$Q = CLh^{3/2},$$

where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

Once a design met the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS model. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. Modifications to the pond design were made if needed after analyzing the pond outflows that HMS calculated. Specifically, we found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probability that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The graph shows that this pond should never take more than 5.99 days to drain, but that is for the largest storms. Although we are reporting the overall time to drain from the highest calculated depth of the pond, most of the time, the pond is calculated to be below that level. In other words, what at first seems like a long time to drain is tempered by the fact that the pond only reaches that level infrequently. Also, the pond will be continually draining, so the water should never be standing.

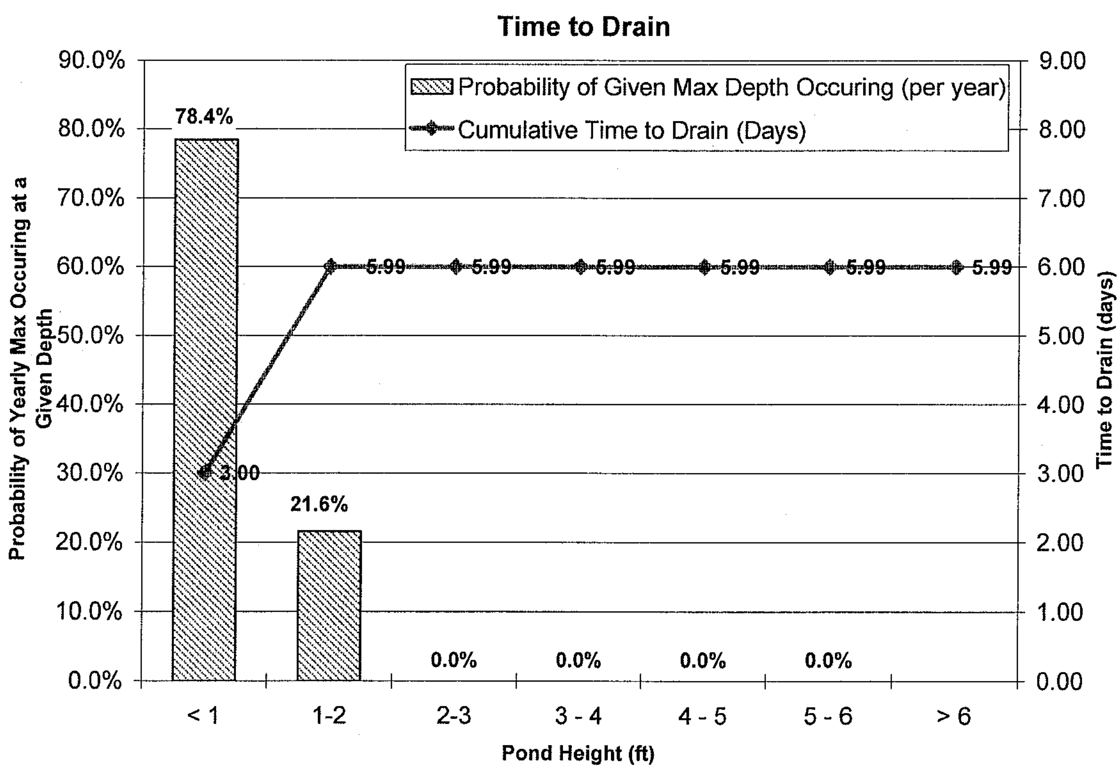

Figure 3. Time to Drain Graph

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Appendix A-3: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	Oct 11, 2005
FROM:	Stephanie Conran and Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	HMP Pond Design Specs for IDS Property		

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the IDS Property which had originally been included in the 147–acre property that combined the Berg and IDS properties and parts of the Legacy property in San Jose. The existing land use is mostly open space, and the proposed land use is mostly residential and thus more impervious. The analysis was complicated by the low values of the flow constraints, as described in the flow-frequency section below, and the relatively high infiltration rate of the B-type soil. A satisfactory detention basin was designed and is detailed in this memo. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

RECOMMENDATIONS

A pond area of about 4.8% of the total project site will be needed to meet the HMP requirements. It was assumed that the basin floor will not be graded and that the side slopes will have a ratio of 2:1. The recommended pond depth is 2'8" total. The bottom 2.25 feet are assumed to percolate. The other specifics of the basin's design are described below in Table 1.

Table 1 – Detention Basin Details

Maximum Pond Area	1.18 acres (216' x 216' floor)
Maximum Pond Depth	2'8"
Time to Drain	2.71 days max
Weir #1 and Weir #2	Width = 6' Invert above pond bottom = 2'3"

Figure 1 shows the project site flow-frequency curves with the basin routing using both MSExcel and HEC-HMS. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event.

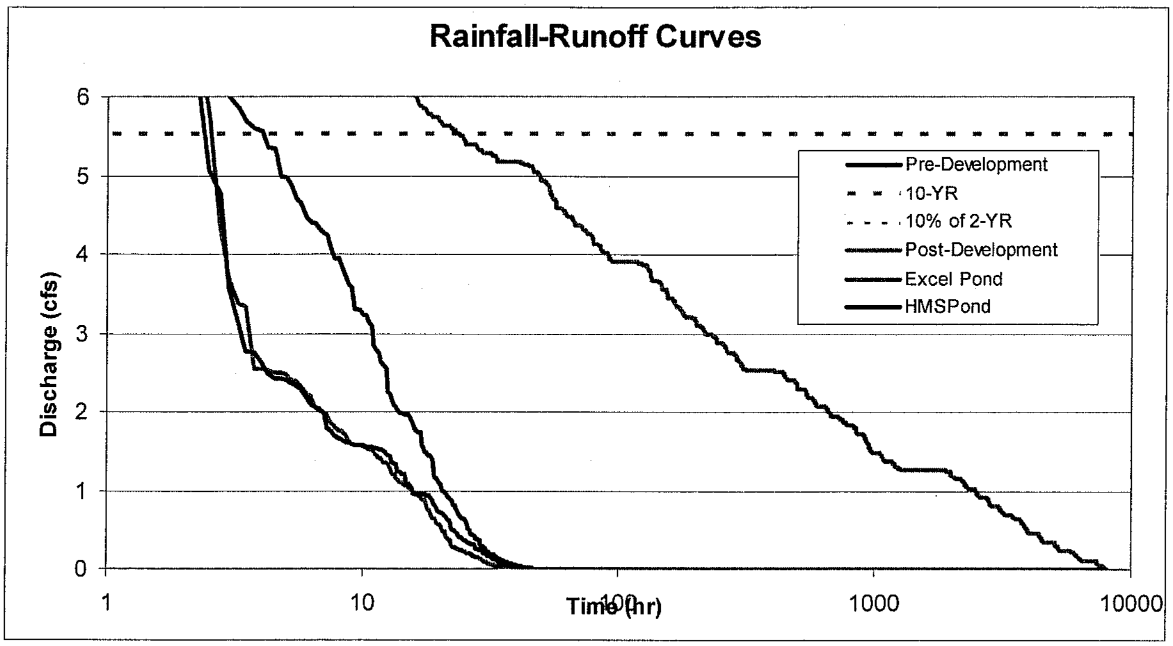

Figure 1. Hydrographs of IDS Site

Since this site is small with porous soil, in its predevelopment condition, there was almost no runoff. Because of this, the pond had to be designed large and shallow so the additional runoff from the development could be almost entirely dissipated through percolation.

SITE DETAILS

The IDS site currently has no urban development, so all of the pre-development area is assumed to be pervious. The site is underlain by Altamont-Azule soil, which is a Group D soil of low porosity, and Arbuckle-Pleasanton, which is a group B soil of high porosity. The D soil makes up about 56.4% of the underlying soil, with the B soil about 43.6%. Ruth and Going, Inc. provided us with a drawing and calculations of the proposed land use. Combining those numbers with the impervious percentages from the Santa Clara Valley Water District's Hydrology Procedures manual, it was calculated that the developed area would be 47% impervious and 53% pervious.

MODELING OVERVIEW

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that conforms the post-development hydrograph to the pre-development hydrograph for the site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve, specifically between the flows of the 10-year flood and 10% of the 2-year flood. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for each drainage area was found using the Partial-

Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. This criterion is addressed at the end of this memo.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions. Existing conditions consist of a single basin. For the proposed conditions the model was broken into two basins, a pervious one and an impervious one. The sum of the hydrologic basin areas was set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 16 inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was adjusted from its MAP of 14 inches. Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows:

$$T_c\ (\text{hrs}) = K*(n*L)^{0.47}*S^{-0.235},$$

where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal to one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the IDS site is underlain by both Altamont-Azule and Arbuckle-Pleasanton soils. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 2 summarizes the various parameters we input into the HMS model for the project site. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 2. IDS HMS Parameters.

		Pre	Post
Area (acres)		24.45	
% Pervious		100.0%	53.0%
B Soil (%)		43.6%	
D Soil (%)		56.4%	
Tc (hours)	Pervious	0.257	
	Impervious	N/A	0.198
R (hours)	Pervious	0.327	
	Impervious	N/A	0.132
Canopy Storage Capacity (in)	Pervious	0.31	0.13
	Impervious	0	0
Surface Storage Capacity (in)	Pervious	0.375	
	Impervious	0.1875	
Soil Infiltration Max. Rate (in/hr)	Pervious	0.296	
	Impervious	0	
Soil Storage Capacity (in)		15.24	
Soil Tension Zone Capacity (in)		5.73	
Soil Percolation Max. Rate (in/hr)		0.197	
Groundwater Storage Capacity (in)		50	
Groundwater Percolation Max. Rate (in/hr)		0.1	
Groundwater Storage Coefficient (hr)		999	

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. The 10-year flow was calculated as 5.52 cfs, and the 2-year was calculated as 0 cfs, giving 0 cfs as 10% of the 2-year. Figure 2 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines.

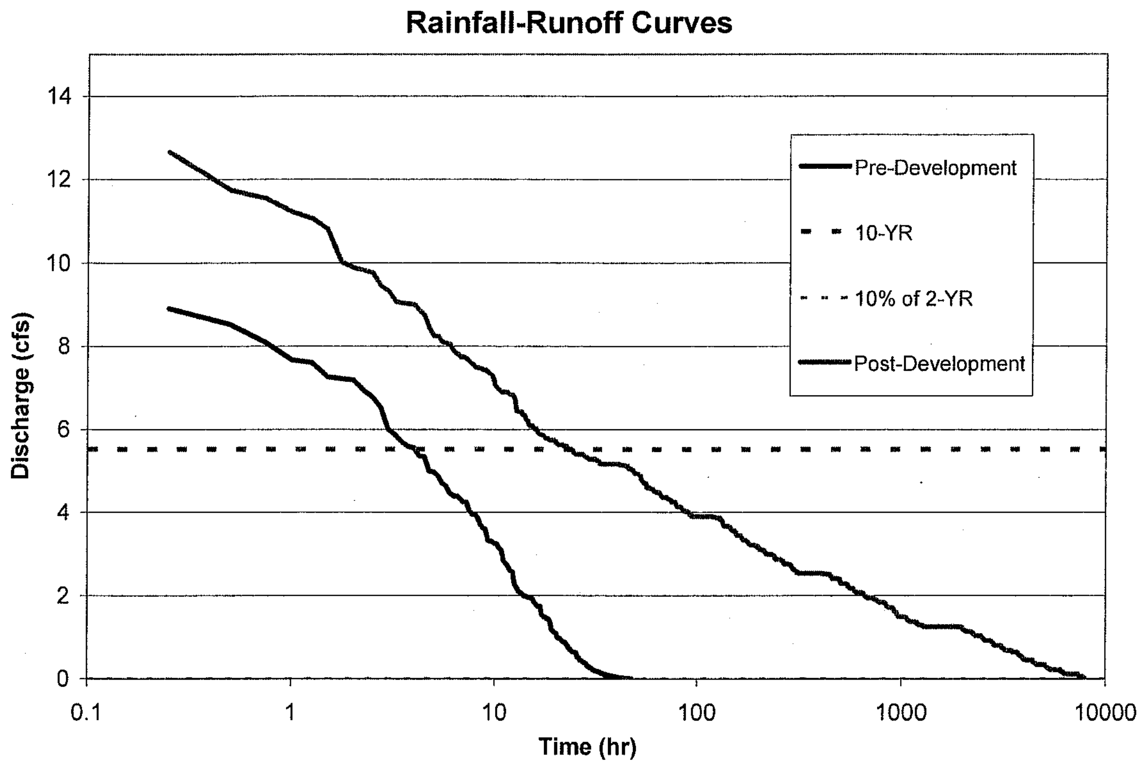

Figure 2. Hydrographs for IDS Site

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed.

A combination of artificial and natural outflows was modeled to drain the basin. Specifically, a weir with a small amount of percolation and evaporation were used. We used about 0.39 in/hr of natural percolation outflow from the basin bottom area of 1.07 acres and 0.00208 in/hr of evaporation. Also, this percolation rate was also used while calculating the time to drain for the basin.

Weir outflow was based on the equation $Q = CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

Once a design met the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS model. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. Modifications to the pond design were made if needed after analyzing the pond outflows that HMS calculated. Specifically, we found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probability that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The graph shows that this pond should never take more than 2.71 days to drain.

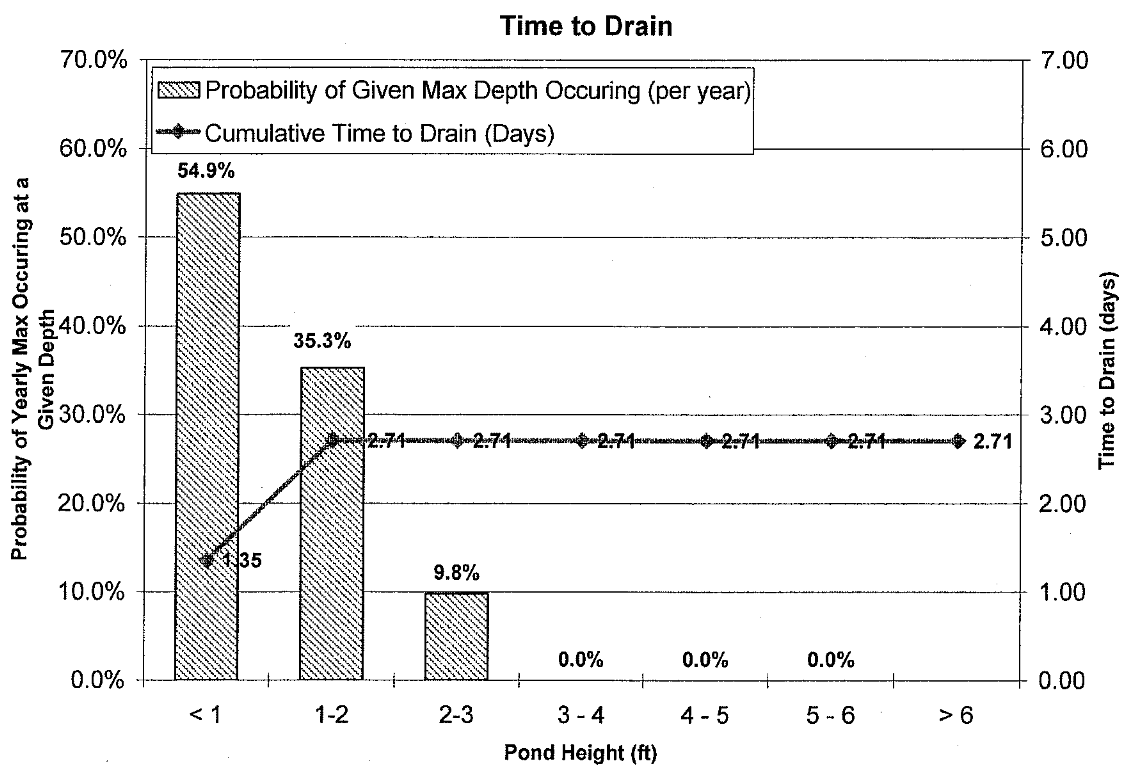

Figure 3. Time to Drain Graph

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Appendix A-4: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	Sept 26, 2005
FROM:	Stephanie Conran and Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	HMP Pond Design Specs for Legacy North Property		

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the Legacy North Property which had originally been included in the 147–acre property that combined the Berg and IDS properties and parts of the Legacy property in San Jose. The existing land use is mostly open space, and the proposed land use is mostly residential and thus more impervious. A satisfactory detention basin was designed and is detailed in this memo. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

RECOMMENDATIONS

A pond area of about .96% of the total project site will be needed to meet the HMP requirements. It was assumed that the basin floor will not be graded and that the side slopes will have a ratio of 2:1. The recommended pond depth is 6'2" total. The other specifics of the basin's design are described below in Table 1.

Table 1 – Detention Basin Details

Maximum Pond Area	.382 acres (105' x 105' floor)
Maximum Pond Depth	6'2"
Time to Drain	1.32 days max
Orifice #1	Diameter = 4"
	Invert above pond bottom = 0'0"
Weir #1	Width = 2'6"
	Invert above pond bottom = 4'

Figure 1 shows the project site flow-frequency curves with the basin routing using both MSExcel and HEC-HMS. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event. HMP allows the pre-development conditions to be exceeded by a maximum of 10% for no more than 10% of the length of the curve. The Excel curve has a maximum exceedance of 9.39% and the entire curve is exceeded for only 3.03 % of its length. The HMS curve does not exceed.

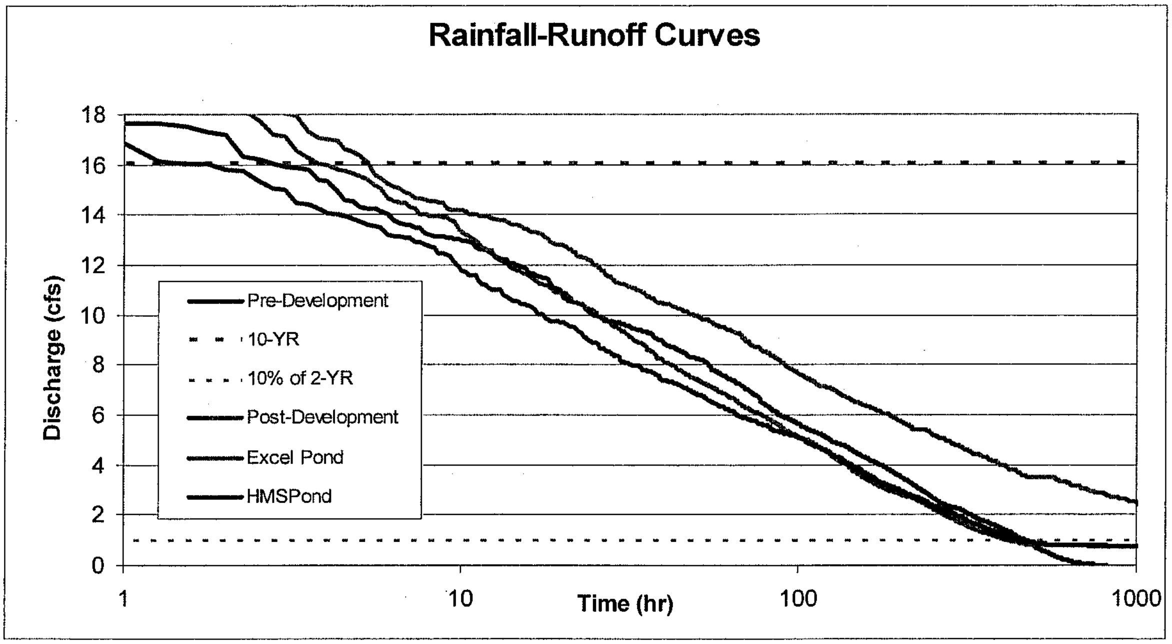

Figure 1. Hydrographs of Legacy North Site

Site Details

The Legacy North site currently has no urban development, so all of the pre-development area is assumed to be pervious. The site is underlain completely by Altamont-Azule soil, which is a Group D soil of low porosity. Ruth and Going, Inc. provided us with a drawing and calculations of the proposed land use. Combining those numbers with the impervious percentages from the Santa Clara Valley Water District's Hydrology Procedures manual, it was calculated that the developed area would be 48% impervious and 52% pervious.

Modeling Overview

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that conforms the post-development hydrograph to the pre-development hydrograph for the site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve, specifically between the flows of the 10-year flood and 10% of the 2-year flood, though HMP requirements allow for 10% exceedence over 10% of the length of the curve within this range. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation.

The flood-frequency curve to get these parameters for each drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. This criterion is addressed at the end of this memo.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions. Existing conditions consist of a single hydrologic basin. For the proposed conditions the model was broken into two basins, a pervious one and an impervious one. The sum of the hydrologic basin areas was set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 16 inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was adjusted from its MAP of 14 inches. Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows:

$$T_c\,(\text{hrs}) = K*(n*L)^{0.47}*S^{-0.235},$$

where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal to one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the Legacy North site is underlain by Altamont-Azule soil. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 2 summarizes the various parameters we input into the HMS model for the project site. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 2. Legacy North HMS Parameters.

		Pre	Post
Area (acres)		39.98	
% Pervious		100.0%	52.0%
B Soil (%)		0.0%	
D Soil (%)		100.0%	
Tc (hours)	Pervious	0.241	
	Impervious	N/A	0.186
R (hours)	Pervious	0.307	
	Impervious	N/A	0.124
Canopy Storage Capacity (in)	Pervious	0.31	0.13
	Impervious	0	0
Surface Storage Capacity (in)	Pervious	0.375	
	Impervious	0.1875	
Soil Infiltration Max. Rate (in/hr)	Pervious	0.072	
	Impervious	0	
Soil Storage Capacity (in)		13.5	
Soil Tension Zone Capacity (in)		5.7	
Soil Percolation Max. Rate (in/hr)		0.048	
Groundwater Storage Capacity (in)		50	
Groundwater Percolation Max. Rate (in/hr)		0.1	
Groundwater Storage Coefficient (hr)		999	

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. The 10-year flow was calculated as 16.06 cfs, and the 2-year was calculated as 9.93 cfs, giving .993 cfs as 10% of the 2-year. Figure 2 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines.

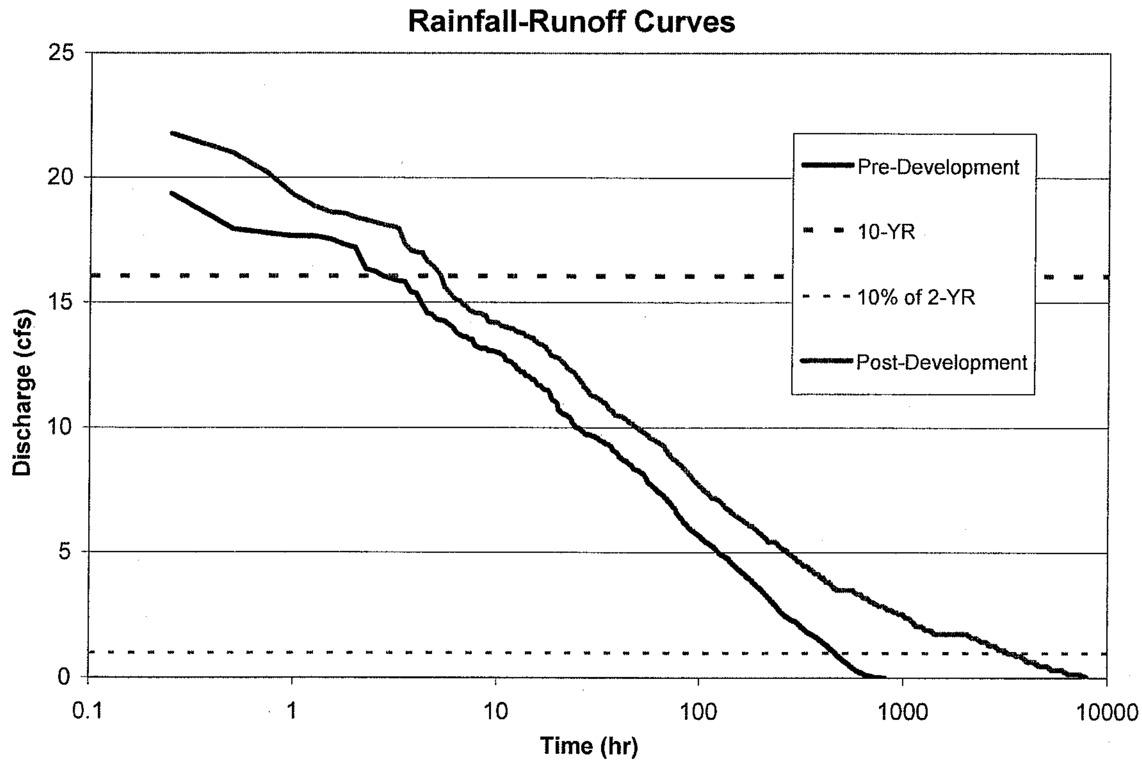

Figure 2. Hydrographs for Legacy North Site

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed.

A combination of artificial and natural outflows was modeled to drain the basin. Specifically, weirs and orifices with a small amount of percolation and evaporation were used. We used about 0.048 in/hr of natural percolation outflow from the basin bottom area of .25 acres and 0.00208 in/hr of evaporation. Also, this percolation rate was also used while calculating the time to drain for the basin.

Weir outflow was based on the equation $Q = CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

The orifice flow was based on two equations: one for open channel flow conditions and one for orifice flow conditions. When the pond level was below the top of the orifice opening (non-pressure flow) Manning's Equation was used. When the pond level was above the top of the orifice the orifice equation, $Q = CA\sqrt{2gh}$, where C is the orifice coefficient (0.6 used), A is the area of the orifice (in feet), g is the gravitational constant and h is the distance from the pond level to the midpoint of the orifice (in feet).

Once a design met the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS model. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. Modifications to the pond design were made if needed after analyzing the pond outflows that HMS calculated. Specifically, we found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probability that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The graph shows that this pond should never take more than 1.32 days to drain.

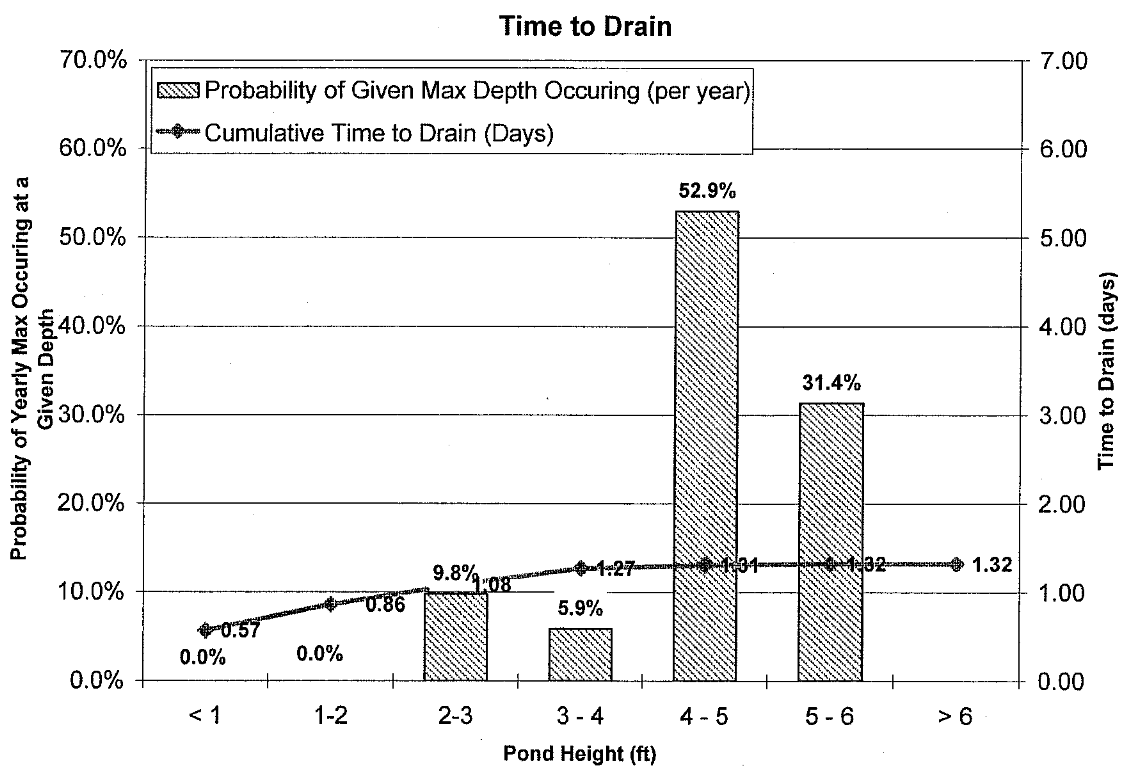

Figure 3. Time to Drain Graph

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Appendix A-5: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	Sept 30, 2005
FROM:	Stephanie Conran and Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	Pond Design Specs for Southside of Legacy Property		

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the southside of the Legacy Property in San Jose, called "Legacy South" in the rest of this memo. The fact that the topography of the existing (i.e., pre-developed) Legacy South site indicates two distinct drainage areas complicated our analysis. Various permutations of routing the post-development runoff were analyzed to find the most effective hydromodification solution. The first two alternatives tried were draining all of the post-development runoff into the Yerba Buena Creek and draining all of the post-development runoff into the Evergreen Creek. Both of these scenarios proved infeasible, but it was possible to route most of the drainage into Evergreen via a 1.16 acre, 15.6 foot pond. Only the roughly four acres of proposed baseball fields will be left to drain into Yerba Buena, and, assuming this area is graded to runoff into Yerba Buena, its drainage will not even require a detention pond; the post-development curve meets the hydrograph modification criterion without detention. This memo therefore summarizes the recommended hydraulic mitigations for the runoff draining into Evergreen alone. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

RECOMMENDATIONS

A pond area of about 2.5% of the total project site will be needed to meet the HMP requirements. It was assumed that the basin floor will not be graded and that the side slopes will have a ratio of 2:1. The recommended pond depth is 6'. The other specifics of the basin's design are described below in Table 1.

Table 1 – Detention Basin Details

Maximum Pond Area	1.94 acres (267' x 267' bottom)
Maximum Pond Depth	6'
Time to Drain	6.99 days max
Orifice #1	Diameter = 4" Invert above pond bottom = 0'
Weir #1	Width = 3' Invert above pond bottom = 4'3"
Weir #2	Width = 6' Invert above pond bottom = 5'

Figure 1 shows the project site flow-frequency curves with the basin routing using MSExcel and HMS. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event. HMP allows the pre-development conditions to be exceeded by a maximum of 10% for no more than 10% of the length of the curve. The Excel curve has a maximum exceedance of 9.52% and the entire curve is exceeded for only 6.9 % of its length. The HMS curve exceeds a maximum 4.14% for 7.69% of the length.

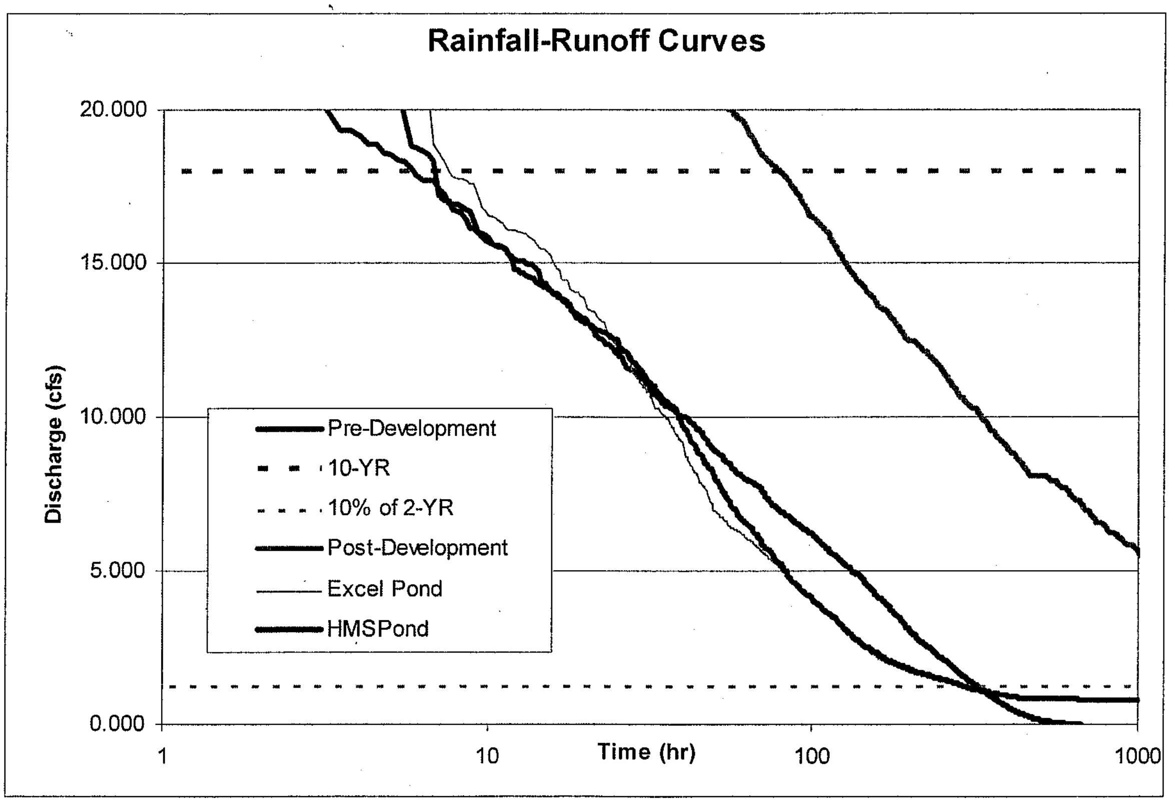

Figure 1. Hydrographs of Legacy South Draining to Evergreen Creek

Figure 2 indicates that the remaining drainage area of Legacy South will meet the pre-development hydrograph without a detention pond. All of the hydrograph of the developed area is below the existing area's hydrograph.

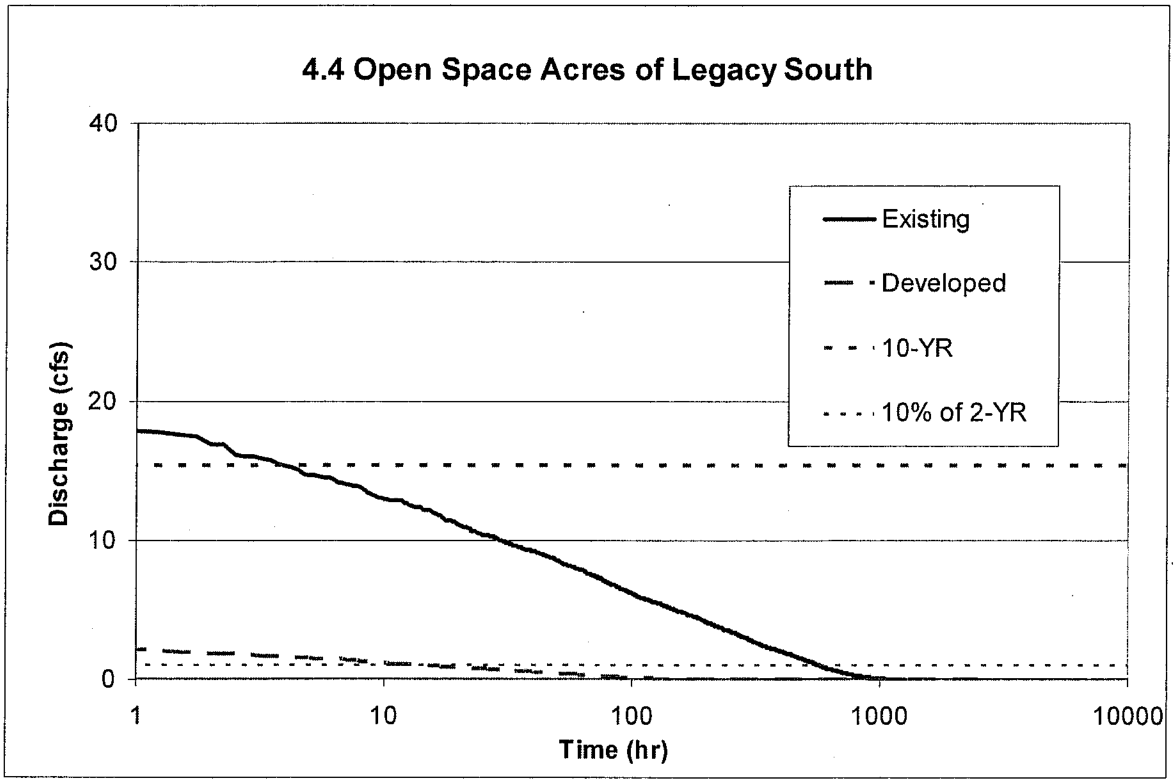

Figure 2. Hydrographs of Legacy South Draining to Yerba Buena Creek

SITE DETAILS

The Legacy South site has currently no urban development, so all of the pre-development area is assumed to be pervious. An analysis of the existing topography indicates that the site currently drains to two different creeks, as seen in Figure 3 below. About two-thirds of the site currently drains to the Evergreen Creek, whereas the other third currently drains to the Yerba Buena Creek. Legacy South is underlain by three types of soil, Arbuckle-Pleasanton, a Group B soil, Los Gatos-Gaviota-Vallecitos, which is a Group C soil, and Altamont-Azule, which is a Group D soil. Specific soil characteristics for each drainage area are detailed below in the discussion of setting up the HEC-HMS model. Using an AutoCAD drawing of the proposed land use and impervious percentages from the Santa Clara Valley Water District's Hydrology Procedures manual, it was calculated that the developed area would be 50% impervious and 50% pervious.

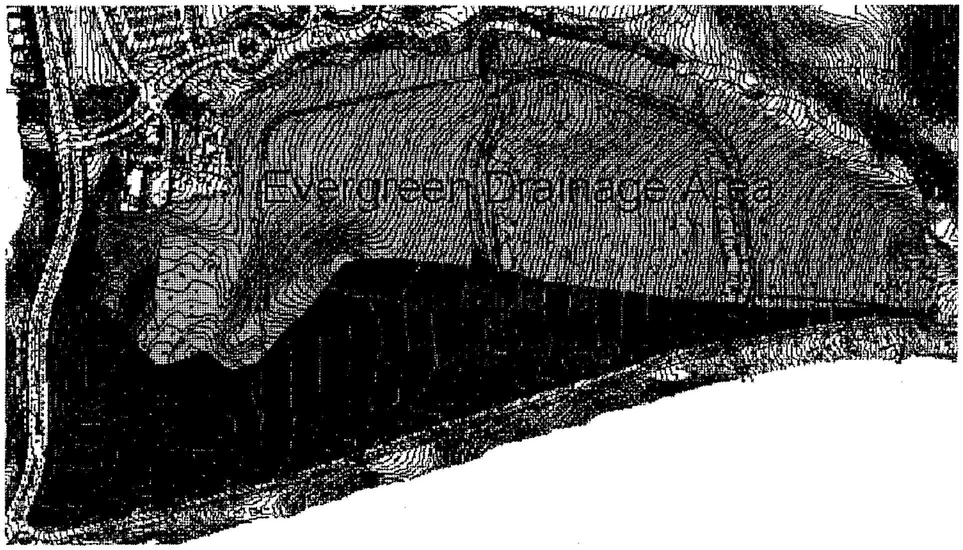

Figure 3. Drainage Areas Based on Topography of Existing Legacy South Site

Modeling Overview

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that will conform the post-development hydrograph to the pre-development hydrograph for the Legacy South site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve. Specifically between the flows of the 10-year flood and 10% of the 2-year flood, the post-development hydrograph must match or be less than the pre-development curve. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for each drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. This criterion is addressed at the end of this memo.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both of the drainage areas. Existing conditions consist of a single basin. For the proposed conditions each drainage area was broken into two basins, a pervious one and an impervious one. The hydrologic basin areas were set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 16 inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was adjusted from its MAP of 14 inches. Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows:

$$T_c \text{ (hrs)} = K*(n*L)^{0.47}*S^{-0.235},$$

where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development runs, and the same Clark values used for existing (pervious) conditions were also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the Legacy South area contains a mixture of Altamont-Azule, Arbuckle-Pleasanton, and Los Gatos-Gaviota-Vallecitos soils. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas are set equal to a weighted sum of the soil type coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 2 summarizes the various parameters we input into the HMS model for both drainage areas of Legacy South. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 2. Legacy South HMS Parameters.

		Evergreen Drainage		Yerba Buena Drainage	
		Pre	Post	Pre	Post
Area (acres)		49.0	72.7	28.1	4.4
% Pervious		100	50	100	100
B Soil (%)		5.6	0	7.5	50.0
C Soil (%)		1.2	1.6	2.0	0
D Soil (%)		93.4	98.4	90.4	50.0
T_c (hours)	Pervious	0.3984	0.3984	0.3984	0.3984
	Impervious	N/A	0.221	N/A	N/A
R (hours)	Pervious	0.507	0.507	0.507	0.507
	Impervious	N/A	0.147	N/A	N/A
Canopy Storage Capacity (in)	Pervious	0.31	0.13	0.31	0.13
	Impervious	0	0	N/A	N/A
Surface Storage Capacity (in)	Pervious	0.375	0.375	0.375	0.375
	Impervious	0.1875	0.1875	0.1875	0.1875
Soil Infiltration Max. Rate (in/hr)	Pervious	0.1032	0.0749	0.1145	0.329
	Impervious	0	0	N/A	N/A
Soil Storage Capacity (in)		13.67	13.40	13.67	15.5
Soil Tension Zone Capacity (in)		5.69	5.65	5.66	5.85
Soil Percolation Max. Rate (in/hr)		0.0688	0.0499	0.0763	0.219
Groundwater Storage Capacity (in)		50	50	50	50
Groundwater Percolation Max. Rate (in/hr)		0.10	0.10	0.10	0.10
Groundwater Storage Coefficient (hr)		999	999	999	999

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. Figures 1 and 2 above show the flow-frequency curves of the existing and post-development conditions for each of the two drainage areas. The flow constraints are indicated as horizontal lines.

HYDRAULIC MODELING – DETENTION BASIN DESIGN

The initial idea for the hydromodification of Legacy South was to have all of the developed area ultimately drain into the Yerba Buena Creek through a one acre detention basin. However, it was determined to be physically infeasible to modify the hydrograph for the whole post-development site to conform to the hydrograph of the pre-development Yerba Buena drainage. We did determine, however, that the hydrograph for 72.7 acres of the post-development site can be modified to conform to the hydrograph of the pre-development Evergreen runoff. This condition assumes that the 4.4 acres of the three proposed baseball fields will be graded to runoff to Yerba Buena creek.

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadhseet. Numerous basin sizes and outlet structures were analyzed.

A combination of artificial and natural outflows was modeled to drain the basin. Specifically, weirs and orifices with a small amount of percolation and evaporation were used. We used about 0.048 in/hr of natural percolation outflow from the basin bottom area of 1.64 acres and 0.00208 in/hr of evaporation. Also, this percolation rate was also used while calculating the time to drain for the basin. Weir outflow was based on the following equation:

$$Q = CLh^{3/2},$$

where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

The orifice flow was based on two equations: one for open channel flow conditions and one for orifice flow conditions. When the pond level was below the top of the orifice opening (non-pressure flow) Manning's Equation was used. When the pond level was above the top of the orifice the following orifice equation was applied:

$$Q = CA\sqrt{2gh},$$

where C is the orifice coefficient (0.6 used), A is the area of the orifice (in feet), g is the gravitational constant and h is the distance from the pond level to the midpoint of the orifice (in feet).

Once a design meets the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS models. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. If needed after analyzing the pond outflows that HMS calculated, we made modifications to the pond design, found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probability that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The graph shows that this pond should never take more than 6.99 days to drain, but that is for the largest storms. Although we are reporting the overall time to drain from the highest calculated depth of the pond, most of the time, the pond is calculated to be below that level. In other words, what at first seems like a long time to drain is tempered by the fact that the pond only reaches that level infrequently. Also, the pond will be continually draining, so the water should never be standing.

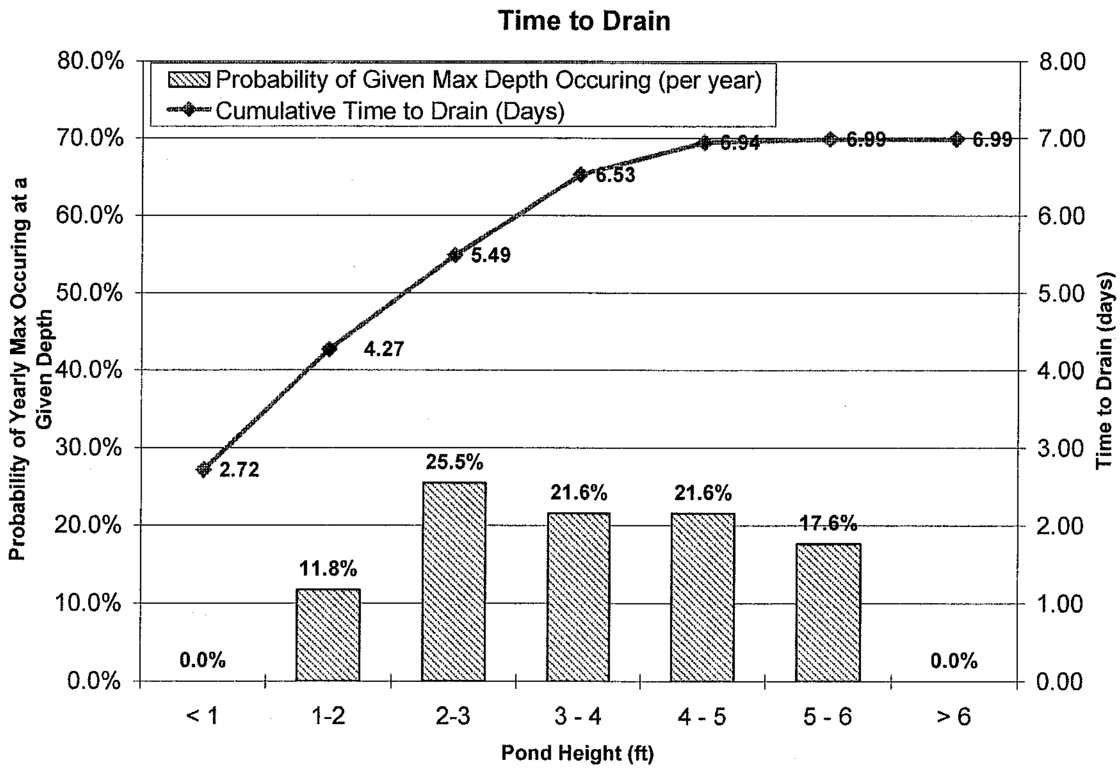

Figure 4. Time to Drain Graph

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Appendix A-6: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	June 24, 2005
FROM:	Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	HMP Ponds Design Specs for EVCC Property		

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the Evergreen Valley Community College (EVCC) property in eastern San Jose. It was hoped that by modifying the flow on another part of campus (Site 2 in Figure 1 below) than that being commercially developed (Site 1 in Figure 1), no detention basin would be needed on Site 1. It was reasoned that since Site 1's runoff would combine downstream with the modified runoff from Site 2, the combined runoff is an adequate variable to modify. However, as detailed below, even completely reducing Site 2's runoff would not adequately mitigate the total runoff. Consequently, two detention basins have been designed and are detailed in this memo – one small basin on Site 1 and a larger basin on Site 2. Note that the larger basin already exists as a pond on campus – indeed, Site 2 was delineated as the area that drains into the existing pond – and we are proposing modifying only how the pond's level is maintained. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

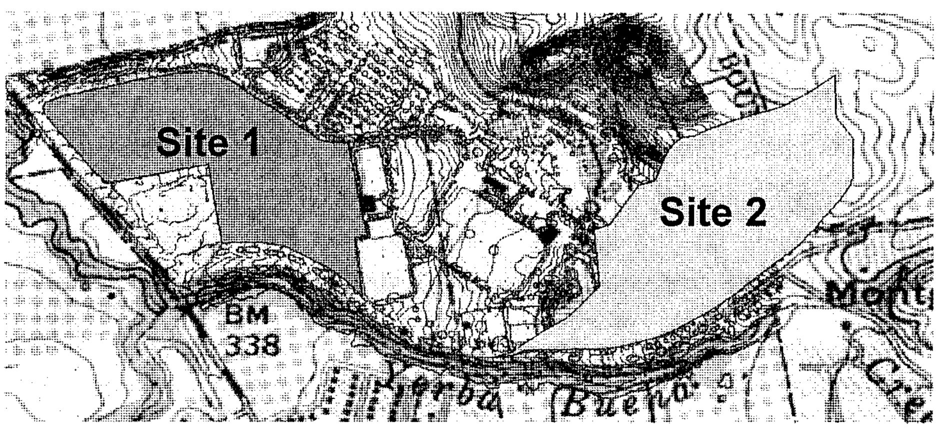

Figure 1 - Plan view of Evergreen Valley Community College campus

Recommendations

A new quarter-acre pond (about 0.8% of Site 1's area) with a depth of 7.7 feet will be needed to meet the HMP requirements. Since the other site (Site 2) already has a relatively large pond on it, it was assumed that the existing pond could be slightly modified for HMP purposes. Therefore, a depth of less than two extra feet will be needed for the existing pond. It was assumed for the new pond that the basin is graded at one percent up to about six inches and much steeper after that up to the top. The model actually assumes straight walls after the 1% grade. It was assumed for the existing pond that it has straight walls for the entire added depth. The other specifics of the two basins' designs are described below in Tables 1 through 4.

Table 1 - New Detention Basin Overview

Maximum Pond Area	0.25 acres (105' by 105')
Maximum Pond Depth	8'1.2"
Top of 1% Grading	6.3"
Percolation Rate	0.195 in/hr
Time to Drain	3.43 days max

Table 2 - New Detention Basin Outlet Works

Outlet Description	Diameter or Width	Invert above Pond Bottom
Orifice #1	3"	6"
Orifice #2	4"	1'
Orifice #3	6"	2'
Orifice #4	6"	3'6"
Orifice #5	12"	5'
Weir	2'	6'6"

Table 3 - Existing Detention Basin Overview

Maximum Pond Area	2.30 acres (500' by 200')
Maximum Pond Added Depth (above normal pond depth)	1'10"
Percolation Rate	0 in/hr
Time to Drain	9.67 days max

Table 4 - Existing Detention Basin Outlet Works

Outlet Description	Diameter or Width	Height above Normal Pond Level
Orifice #1	3"	0"
Orifice #2	4"	6"
Orifice #3	6"	12"
Weir	10'	1'9"

Figure 2 shows the project site flow-frequency curves with the results from the basin routing using MSExcel and HEC-HMS. It can be seen that the designed basins adequately modify the total post-development hydrograph to match the total hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event.

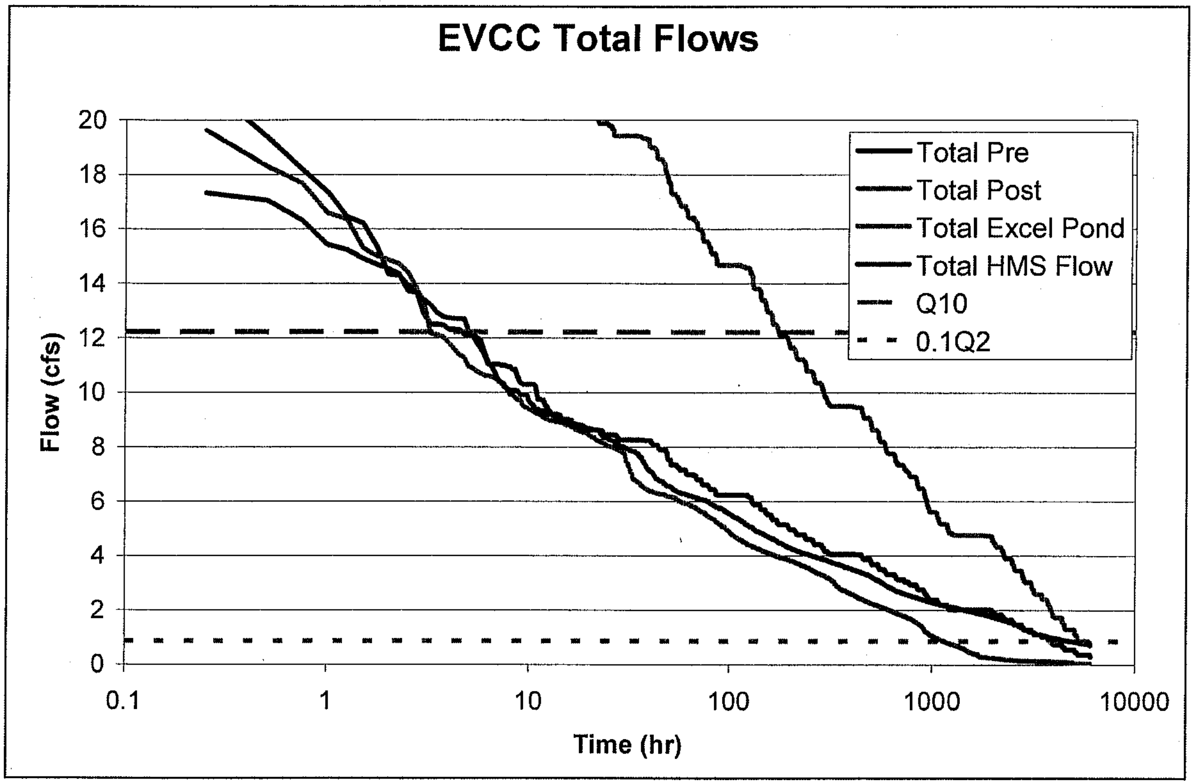

Figure 2 - Hydrographs of Combined Flow from Site 1 and Site 2

Site Details

Site 1 currently has little urban development, so most of the pre-development area (84%) is assumed to be pervious. Some of Site 2 is already developed, but it is still 68% pervious. Site 1 is underlain by two types of soil, Zamora-Pleasanton, which is a Group C soil, and Arbuckle-Pleasanton, which is a Group B soil. The Group B Soil has a relatively high porosity and infiltration rate, resulting in much smaller flows for the existing condition relative to the developed condition, as Figure 2 makes clear. Site 2 is underlain by Arbuckle-Pleasanton, as well, and Altamont-Azule, a group D soil of low porosity. For Site 1, the developed area was calculated to be 80% impervious; for site 2, the area will remain 32% impervious.

Modeling Overview

The US Army Corps of Engineer's HEC-HMS software was used to simulate 52-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet model to design two detention ponds that will conform the post-development hydrograph to the pre-development hydrograph for the total of the two EVCC sites. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve. Specifically between the flows of the 10-year flood and 10% of the 2-year flood, the post-development hydrograph must match or be less than the pre-development curve. The area below the 10-percent of the 2-year flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for the total drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the pre-development duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. Although on average both of the designed ponds should drain within that time frame, there are times when it will take longer for the existing pond to drain back to its normal level. However, since the existing basin is currently a pond, there is already standing water in the basin and simply adding more standing water should not be a problem.

It was desired to modify the runoff from Site 2 such that no modification of the post-development runoff from Site 1 would be necessary considering the combined runoff of the two sites. However, even by containing *all* of the flow from Site 2, the post-development runoff from Site 1 alone would exceed that for the combined pre-development flow. This is understandable given two factors: (1) the drastic change in permeability of the Site 1, from a relatively pervious site to one with a high degree of imperviousness and (2) the large amount of underlying B-type soil, which is highly permeable and leads to little runoff in the pre-development condition. Therefore, some modification of the Site 1 runoff was necessary in addition to a significant modification of Site 2's runoff. Details of the analysis leading to this conclusion can be provided if desired. In the end, developing Site 1 will increase the number and intensity of runoff flows into the receiving waters, and the designed detention basin only slightly modifies those flows. However, by modifying the flows from Site 2 to somewhat below the existing amounts, the combined runoff flows has been calculated to give a hydrograph meeting the aforementioned HMP criterion.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions for Sites 1 and 2. Existing conditions consist of two basins for each site, a pervious one and an impervious one. The basins for each site were joined at junctions, and then the two junctions were combined for the total pre-development flow. For the proposed conditions each site again had two basins, with changes only to the square mileage of the basins and, for Site 1, to the the canopy storage capacity value. Actually, for Site 2, the basins for pre-development and post-development are identical since the site will remain unchanged except for the routing of stormwater into the existing pond. In the post-development model, reservoirs were added at the junction of each site's basin to simulate the detention basins. The output from the reservoirs was then combined into a final junction from which the total post-development flow could be found. In inputting the areas for each basin, it was assumed that the sites were hydrologically contained – i.e., no water enters either site from upstream. In the model this translates to the total drainage area being equal to the total area of Sites 1 and 2. This is a reasonable assumption for Site 1 since it is bordered on the north by a creek and elsewhere by developed areas having their own storm drain systems. It is reasonable for Site 2 since the boundaries for the site are based on the topography of the area that would drain into the existing pond if the storm drains were not there.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 16.5-inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was increased by 27%

(16.5-inches divided by 13-inches). Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows: $T_c \text{ (hrs)} = K^*(n^*L)^{0.47} * S^{-0.235}$, where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage for each site was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation for Site 1 is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for these sites as the ones we have used for other nearby sites.

Surface Storage Capacity values for each site were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate for each site was set equal to one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate for each site was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values for each site are based on soil classification. According the SCVWD GIS, Sites 1 and 2 are underlain by mostly Arbuckle-Pleasanton soil with some Zamora-Pleasanton soil under Site 1 and some Altamont-Azule under Site 2. We assumed the

GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Soil Tension Zone Capacity for each site was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 5 summarizes the various parameters we input into the HMS model for the two EVCC sites. The "Pre" columns for each site give values for the existing conditions, and the "Post" columns for each site give values for the post-development conditions. Columns have been merged where the values are the same for pre and post conditions or between the sites.

Table 5 - EVCC HMS Parameters

<table>
<tr><th colspan="2" rowspan="2"></th><th colspan="2">Site 1</th><th colspan="2">Site 2</th></tr>
<tr><th>Pre</th><th>Post</th><th>Pre</th><th>Post</th></tr>
<tr><td colspan="2">Area (acres)</td><td colspan="2">33.3</td><td colspan="2">40.4</td></tr>
<tr><td colspan="2">% Pervious</td><td>83.7</td><td>20.0</td><td colspan="2">68.1</td></tr>
<tr><td colspan="2">B Soil (%)</td><td colspan="2">70.6</td><td colspan="2">92.0</td></tr>
<tr><td colspan="2">C Soil (%)</td><td colspan="2">29.4</td><td colspan="2">0</td></tr>
<tr><td colspan="2">D Soil (%)</td><td colspan="2">0</td><td colspan="2">8.0</td></tr>
<tr><td rowspan="2">T_c (min)</td><td>Pervious</td><td colspan="2">21.4</td><td colspan="2">20.0</td></tr>
<tr><td>Impervious</td><td colspan="2">11.9</td><td colspan="2">11.1</td></tr>
<tr><td rowspan="2">R (min)</td><td>Pervious</td><td colspan="2">27.2</td><td colspan="2">25.5</td></tr>
<tr><td>Impervious</td><td colspan="2">7.9</td><td colspan="2">7.4</td></tr>
<tr><td rowspan="2">Canopy Storage Capacity (in)</td><td>Pervious</td><td>0.31</td><td>0.13</td><td colspan="2">0.13</td></tr>
<tr><td>Impervious</td><td colspan="4">0</td></tr>
<tr><td rowspan="2">Surface Storage Capacity (in)</td><td>Pervious</td><td colspan="4">0.375</td></tr>
<tr><td>Impervious</td><td colspan="4">0.1875</td></tr>
<tr><td rowspan="2">Soil Infiltration Max. Rate (in/hr)</td><td>Pervious</td><td colspan="2">0.488</td><td colspan="2">0.544</td></tr>
<tr><td>Impervious</td><td colspan="4">0</td></tr>
<tr><td colspan="2">Soil Percolation Max. Rate (in/hr)</td><td colspan="2">0.325</td><td colspan="2">0.363</td></tr>
<tr><td colspan="2">Soil Storage Capacity (in)</td><td colspan="2">16.7</td><td colspan="2">17.2</td></tr>
<tr><td colspan="2">Soil Tension Zone Capacity (in)</td><td colspan="2">6.24</td><td colspan="2">5.98</td></tr>
<tr><td colspan="2">Groundwater Storage Capacity (in)</td><td colspan="4">50</td></tr>
<tr><td colspan="2">Groundwater Percolation Max. Rate (in/hr)</td><td colspan="4">0.10</td></tr>
<tr><td colspan="2">Groundwater Storage Coefficient (hr)</td><td colspan="4">999</td></tr>
</table>

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run and the output flows extracted into an Excel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the combined pond output. To determine the flow constraints of the pre-development flows for the 10-year and 10% of the 2-year storm, the combined pre-development flow peaks were ranked. A peak flow was defined as a flow for which the two previous and two following times-steps have less flow. The 10-year flow was found to be 12.22 cfs, and the 2-year was 8.64 cfs, giving 0.864 cfs as 10% of the 2-year. Figure 3 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines. Although it can be seen that there are several values of much larger magnitude than the 10-year storm, all of these values occurred during only one storm event in the recorded data.

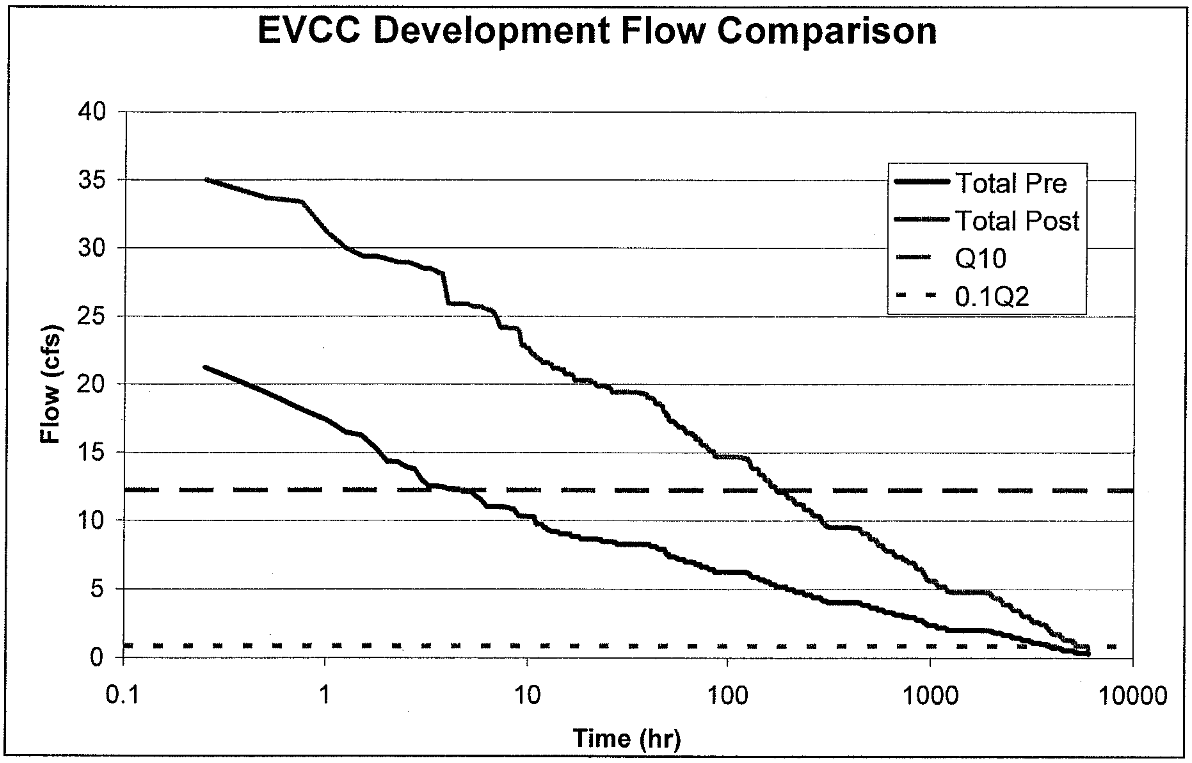

Figure 3 - Hydrographs for EVCC

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was initially performed using an Excel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed to design the new detention basin on Site 1. For Site 2, the existing pond size (2.3 acres) was taken as the size of the detention basin, and various outlet structures were analyzed to modify the outflow.

We used a combination of artificial and natural outflows to drain the basins. Specifically, we simulated weirs, orifices, and some percolation. For both sites, to avoid overstating the basins' natural outflows, we used only half of the B soil percolation rate, assuming that the basin would be placed in B-type soil. We used 0.195 in/hr of natural outflow from the bottom of both basins. Also for calculating the time to drain for each basin, 0.195 in/hr was used.

Weir outflow was based on the following equation $Q=CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet). The orifice flow was based on two equations: one for open channel flow conditions and one for orifice flow conditions. When the pond level was below the top of the orifice opening (non-pressure flow) Manning's Equation was used. When the pond level was above the top of the orifice the following orifice equation was applied: $Q=CA\sqrt{2gh}$, where C is the orifice coefficient (0.6 used), A is the area of the orifice (in feet), g is the gravitational constant and h is the distance from the pond level to the midpoint of the orifice (in feet).

Since the result of interest for this project was the combined outflow of the two detention basins, the results from each basin's Excel spreadsheet were combined and those results compared to the pre-development curve found with HMS. Once the design for both ponds in Excel gave a result which met the HMP requirements, the basin designs were entered into the HMS model. Specifically, for each pond design, a rating curve of outflow versus height of the ponded water was input into the respective Pond element in the HMS model. The HMS results are a final test of the basin designs since HMS has a more sophisticated routing procedure and includes some data values left out of the Excel model for ease of use. We then modified the Excel pond designs if necessary after analyzing the pond outflows calculated by HMS, and the process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probability that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The new pond on Site 1 will take 3.43 days to drain at its maximum height. The existing pond on Site 2 will take a maximum of 9.67 days to reach its normal, "non-stormwater" level. Figure 4 presents the results for the new pond, and Figure 5 for the existing pond. The Site 1 pond is within the vector control criteria. Since the existing pond on Site 2 already has standing water, its drain time is irrelevant. Thus, the time-to-drain graph for the existing pond is presented here for informative purposes only. More details can be supplied if desired.

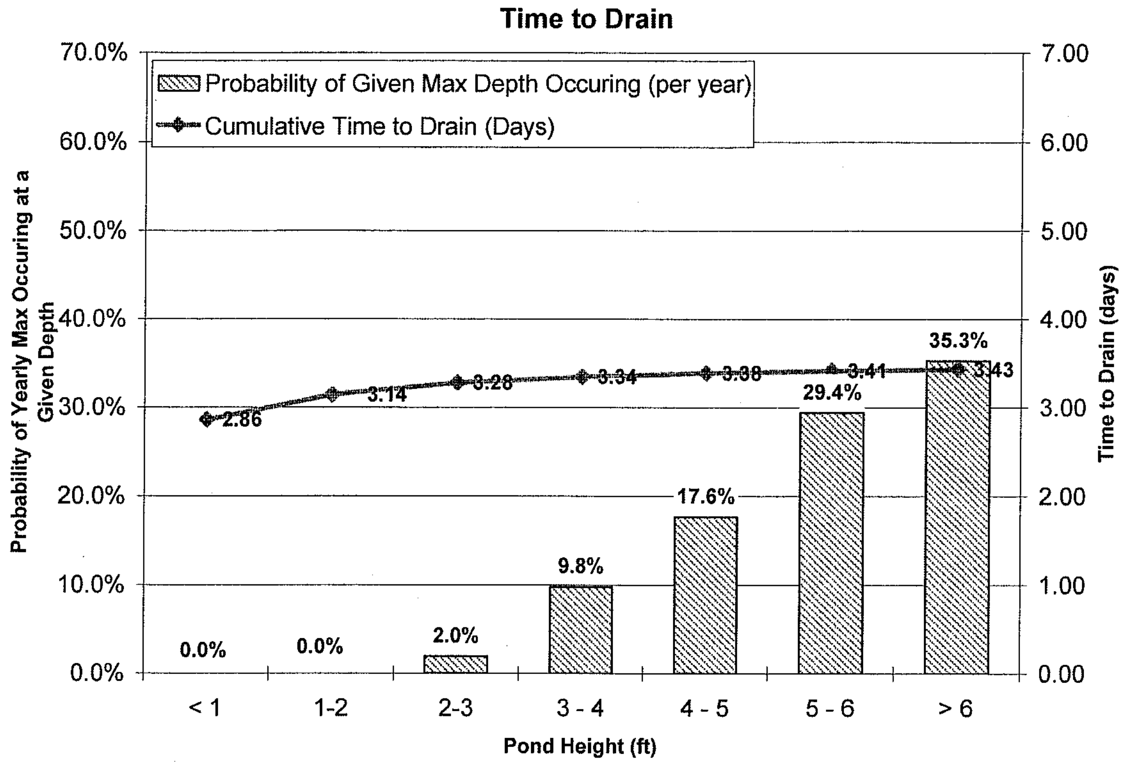

Figure 4 Time to Drain graph for New Pond

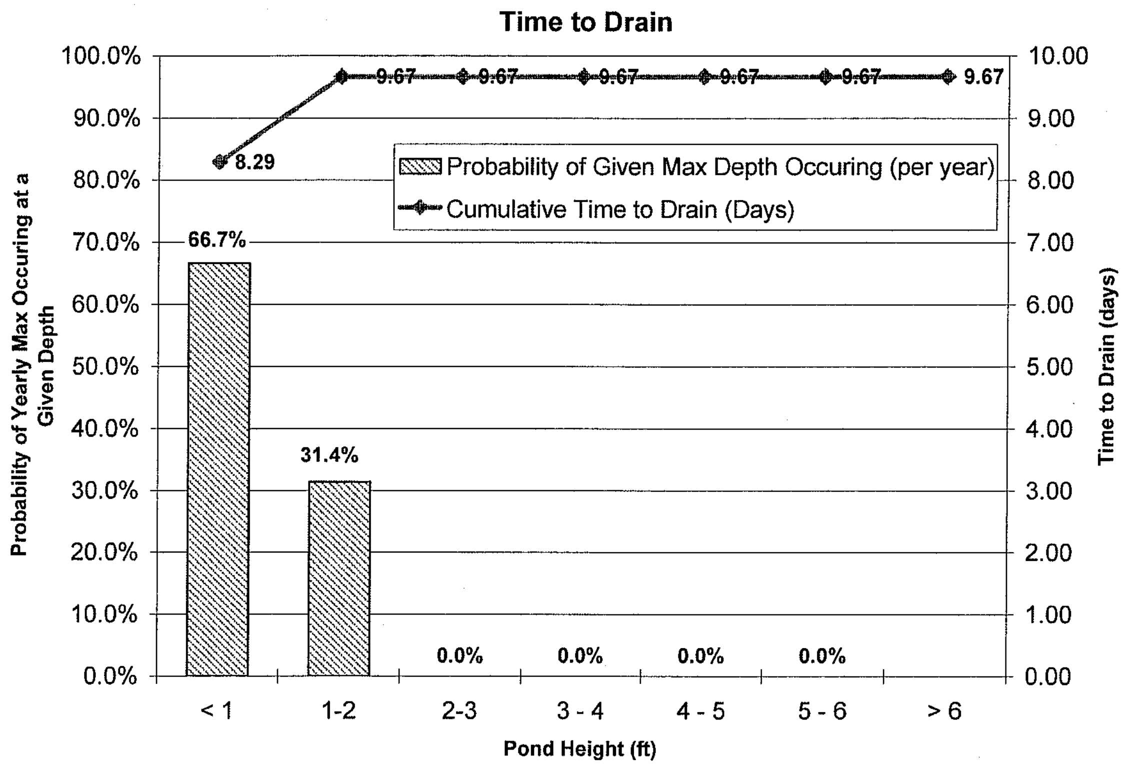

Figure 5 Time to Drain graph for Existing Pond

Schaaf & Wheeler
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Appendix A-7: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	Oct 12, 2005
FROM:	Stephanie Conran and Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	HMP Pond Design Specs for Pleasant Hills Golf Club Property		

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the Pleasant Hills Golf Course, excepting the five acres that is to be developed as a municipal fire station. As the aerial photograph in Figure 1 below shows, the existing land use is mostly open space as a public golf course. The proposed land use is mostly residential, so the post-project land will have more imperviousness than the pre-project land. A satisfactory detention basin was designed and is detailed in this memo. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

Figure 6 - Pleasant Hills Golf Course Site

RECOMMENDATIONS

A pond area of 1.2% of the 108-acre project site will be needed to meet the HMP requirements. It was assumed that the basin floor will not be graded and that the side slopes will have a ratio of 2:1. The recommended pond depth is 5'11". The other specifics of the basin's design are described below in Tables 1 and 2.

Table 1 – Detention Basin Overview

Maximum Pond Area	1.26 acres
Maximum Pond Depth	5'11"
Time to Drain	1.58 days max

Table 2 – Detention Basin Outlet Works

Outlet Description	Diameter or Width	Invert above Pond Bottom
Orifice #1	8"	0'
Weir #1	8'	3'10"

Figure 2 shows the project site flow-frequency curves with the basin routing using both MSExcel HEC-HMS. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event. HMP allows the pre-development conditions to be exceeded by a maximum of 10% for no more than 10% of the length of the curve. The Excel curve has a maximum exceedance of .15% and the entire curve is exceeded for only .05% of its length and the HMS curve does not exceed. Both lie within the HMP requirements.

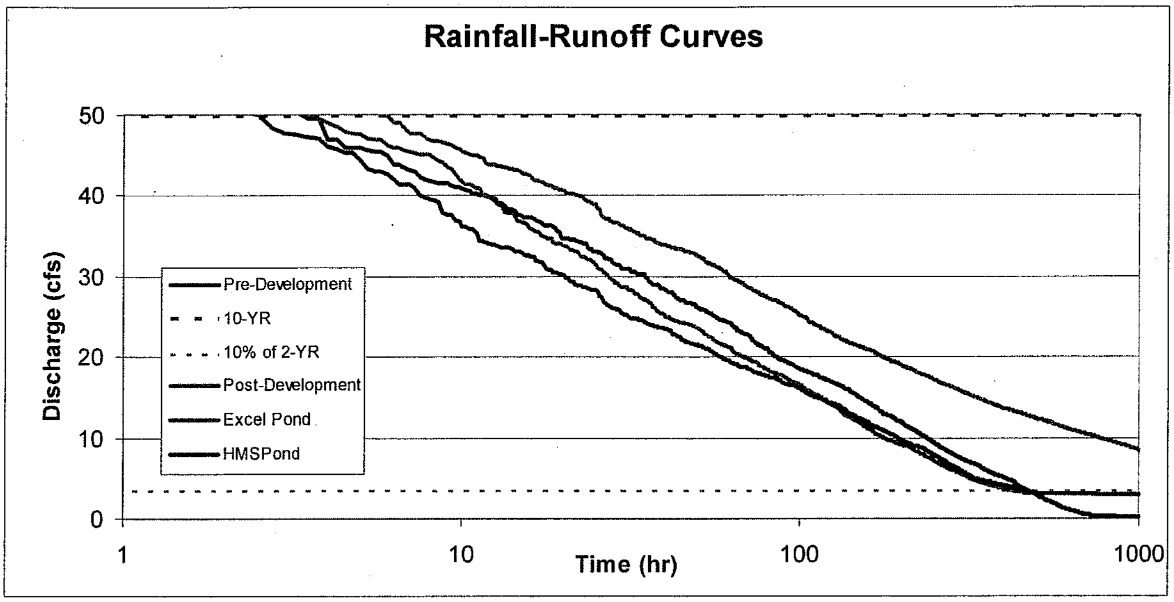

Figure 2 - Hydrographs of Pleasant Hills 108-Acre Site

SITE DETAILS

The Pleasant Hills Golf Course site is currently developed as a golf course, so most of the pre-development area is assumed to be pervious. The site is underlain by Cropley-Rincon soil, which is a Group D soil of low porosity, and Zamora-Pleasanton, which is a group C soil of medium porosity. The D soil makes up about 76.5% of the underlying soil, with the C soil about 23.5%. Combining the land use values with the impervious percentages from the Santa Clara Valley Water District's Hydrology Procedures manual, it was calculated that the developed area would be 56% impervious and 44% pervious.

MODELING OVERVIEW

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that conforms the post-development hydrograph to the pre-development hydrograph for the site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve. Specifically between the flows of the 10-year flood and 10% of the 2-year flood, the post-development hydrograph must match or be less than the pre-development curve. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for each drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. The designed pond should drain within that time frame.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions. Existing conditions consist of a single basin. For the proposed conditions the model was broken into two basins, a pervious one and an impervious one. The sum of the hydrologic basin areas was set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 15.5 inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was adjusted from its MAP of 14 inches. Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation: $T_c\ (hrs) = K*(n*L)^{0.47}*S^{-0.235}$, in which K is a constant equal to 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\ \right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\ \right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal to the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was also set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the 108-acre golf course site is underlain by both Cropley-Rincon and Zamora-Pleasanton soils. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 3 summarizes the various parameters we input into the HMS model for the project site. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 3. 108-Acre Pleasant Hills Golf Course HMS Parameters.

		Pre	Post
Area (acres)		108.4	
% Pervious		99.0	44.0
C Soil (%)		23.5	
D Soil (%)		76.5	
T_c (hours)	Pervious	0.44	
	Impervious	N/A	0.44
R (hours)	Pervious	0.30	
	Impervious	N/A	0.30
Canopy Storage Capacity (in)	Pervious	0.31	0.13
	Impervious	0	0
Surface Storage Capacity (in)	Pervious	0.375	
	Impervious	0.1875	
Soil Infiltration Max. Rate (in/hr)	Pervious	0.0767	
	Impervious	0	
Soil Storage Capacity (in)		21.4	
Soil Tension Zone Capacity (in)		8.64	
Soil Percolation Max. Rate (in/hr)		0.0767	
Groundwater Storage Capacity (in)		50	
Groundwater Percolation Max. Rate (in/hr)		0.10	
Groundwater Storage Coefficient (hr)		999	

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. The 10-year flow was calculated as 49.76 cfs, and the 2-year was calculated as 34.21 cfs, giving 3.42 cfs as 10% of the 2-year. Figure 3 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines.

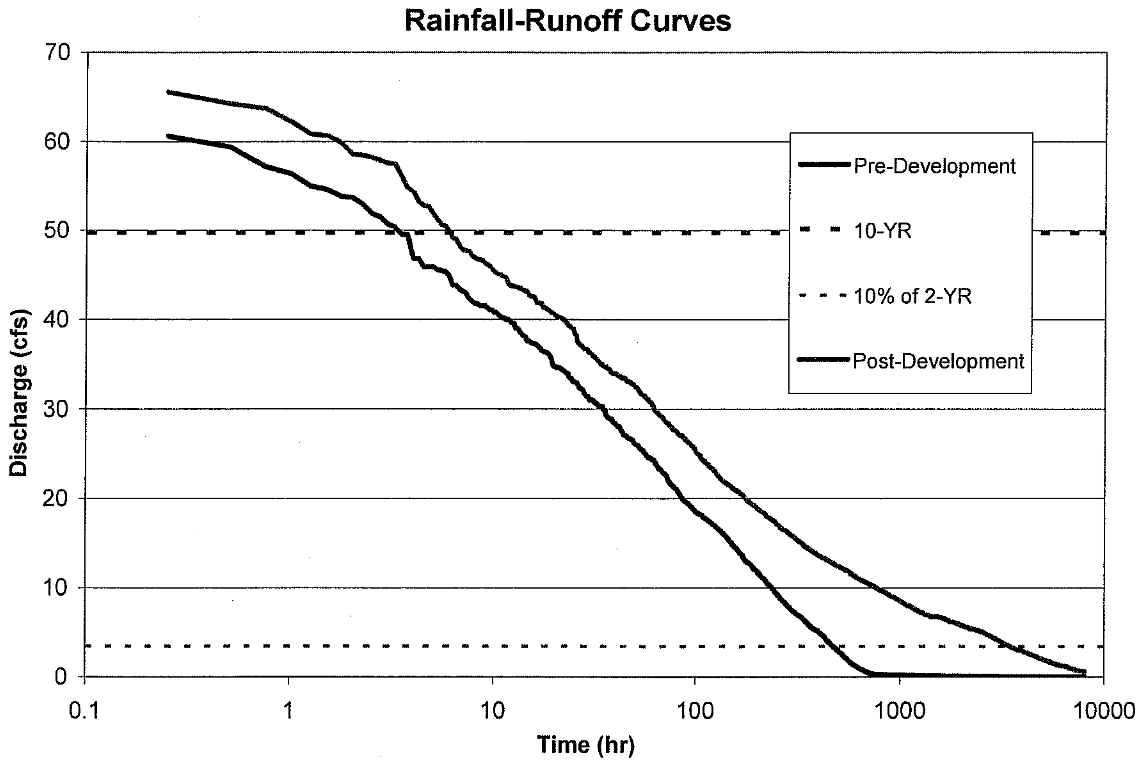

Figure 3 – Pre- and Post-Project Hydrographs of Pleasant Hills Golf Course Site

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed.

A combination of artificial and natural outflows was modeled to drain the basin. Specifically, weirs and orifices with a small amount of percolation and evaporation were used. It is assumed that the basin will be placed in an area of C soil, so we used about 0.17 in/hr of natural percolation outflow from the basin bottom area of 1.01 acres and 0.00208 in/hr of evaporation. Also, this percolation rate was also used while calculating the time to drain for the basin. Weir outflow was based on the equation $Q=CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

Weir outflow was based on the equation $Q=CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet). The orifice flow was based on two equations: one for open channel flow conditions and one for orifice flow conditions. When the pond level was below the top of the orifice opening (non-pressure flow) Manning's Equation was used. When the pond level was above the top of the orifice the orifice equation, $Q=CA\sqrt{2gh}$, where C is the orifice coefficient (0.6 used), A is the area of the orifice (in feet), g is the gravitational constant and h is the distance from the pond level to the midpoint of the orifice (in feet).

Once a design met the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS model. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. Modifications to the pond design were made if needed after analyzing the pond outflows that HMS calculated. Specifically, we found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probability that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The graph shows that this pond should never take more than 1.58 days to drain.

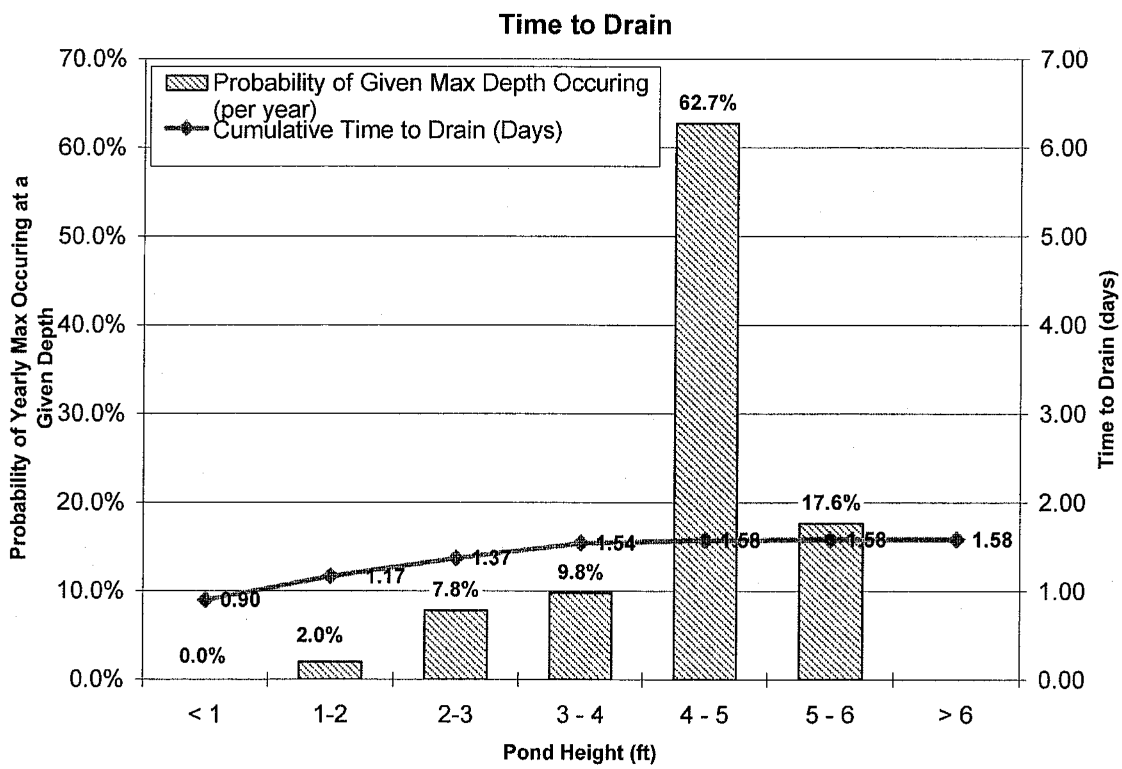

Figure 4 – Time to Drain for Pleasant Hills Golf Course Pond

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Appendix A-8: TECHNICAL MEMORANDUM

TO: Jim Schaaf, P.E., Ph.D.

DATE: Oct 11, 2005

FROM: Stephanie Conran and Charles Hardy

JOB #: HMHI.17.04

SUBJECT: HMP Pond Design Specs for Berg South Property

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the Berg South Property which had originally been included in the 147–acre property that combined the Berg and IDS properties and parts of the Legacy property in San Jose. The existing land use is mostly open space, and the proposed land use is mostly residential and thus more impervious. A satisfactory detention basin was designed and is detailed in this memo. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

RECOMMENDATIONS

A pond area of about 1.6% of the total project site will be needed to meet the HMP requirements. It was assumed that the basin floor will not be graded and that the side slopes will have a ratio of 2:1. The recommended pond depth is 6'1" total. The other specifics of the basin's design are described below in Table 1.

Table 1 – Detention Basin Details

Maximum Pond Area	1.42 acres (230' x 230' floor)
Maximum Pond Depth	6'1"
Time to Drain	2.5 days max
Orifice #1	Diameter = 5" Invert above pond bottom = 0'
Weir #1	Width = 6' Invert above pond bottom = 4'5"
Weir #2	Width = 6' Invert above pond bottom = 5'3"

Figure 1 shows the project site flow-frequency curves with the basin routing using both MSExcel and HEC-HMS. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event. HMP allows the pre-development conditions to be exceeded by a maximum of 10% for no more than 10% of the length of the curve. The Excel curve has a maximum exceedance of 5.62% and the entire curve is exceeded for only 1.04% of its length and the HMS curve exceeds by a maximum 1.01% over 1.04%, both of which lie within the HMP requirements.

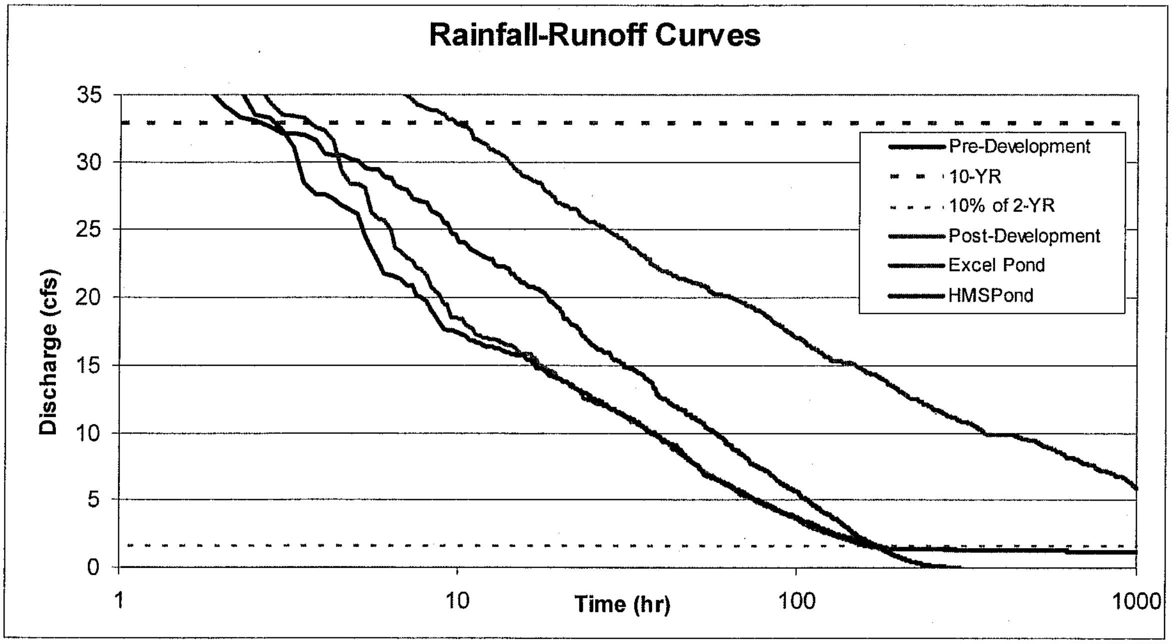

Figure 1. Hydrographs of Berg South Site

Site Details

The Berg South site currently has no urban development, so all of the pre-development area is assumed to be pervious. The site is underlain by Altamont-Azule soil, which is a Group D soil of low porosity, and Arbuckle-Pleasanton, which is a group B soil of high porosity. The D soil makes up about 83.4% of the underlying soil, with the B soil about 16.6%. Ruth and Going, Inc. provided us with a drawing and calculations of the proposed land use. Combining those numbers with the impervious percentages from the Santa Clara Valley Water District's Hydrology Procedures manual, it was calculated that the developed area would be 49.7% impervious and 50.3% pervious.

Modeling Overview

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that conforms the post-development hydrograph to the pre-development hydrograph for the site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve, specifically between the flows of the 10-year flood and 10% of the 2-

year flood, though HMP requirements allow for 10% exceedence over 10% of the length of the curve within this range. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for each drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. This criterion is addressed at the end of this memo.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions. Existing conditions consist of a single basin. For the proposed conditions the model was broken into two basins, a pervious one and an impervious one. The sum of the hydrologic basin areas was set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 16-inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was adjusted from its MAP of 14 inches. Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows:

$$T_c\ (\text{hrs}) = K^*(n^*L)^{0.47}{}^*S^{-0.235},$$

where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\ \right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\ \right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal to one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the Berg South site is underlain by both Altamont-Azule and Arbuckle-Pleasanton soils. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 2 summarizes the various parameters we input into the HMS model for the project site. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 2. Berg South HMS Parameters.

		Pre	Post
Area (acres)		89.39	
% Pervious		100.0%	50.3%
B Soil (%)		16.6%	
D Soil (%)		83.4%	
Tc (hours)	Pervious	0.323	
	Impervious	N/A	0.249
R (hours)	Pervious	0.411	
	Impervious	N/A	0.166
Canopy Storage Capacity (in)	Pervious	0.31	0.13
	Impervious	0	0
Surface Storage Capacity (in)	Pervious	0.375	
	Impervious	0.1875	
Soil Infiltration Max. Rate (in/hr)	Pervious	0.157	
	Impervious	0	
Soil Storage Capacity (in)		14.16	
Soil Tension Zone Capacity (in)		5.75	
Soil Percolation Max. Rate (in/hr)		0.105	
Groundwater Storage Capacity (in)		50	
Groundwater Percolation Max. Rate (in/hr)		0.1	
Groundwater Storage Coefficient (hr)		999	

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. The 10-year flow was calculated as 32.86 cfs, and the 2-year was calculated as 16.31 cfs, giving 1.631 cfs as 10% of the 2-year. Figure 2 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines.

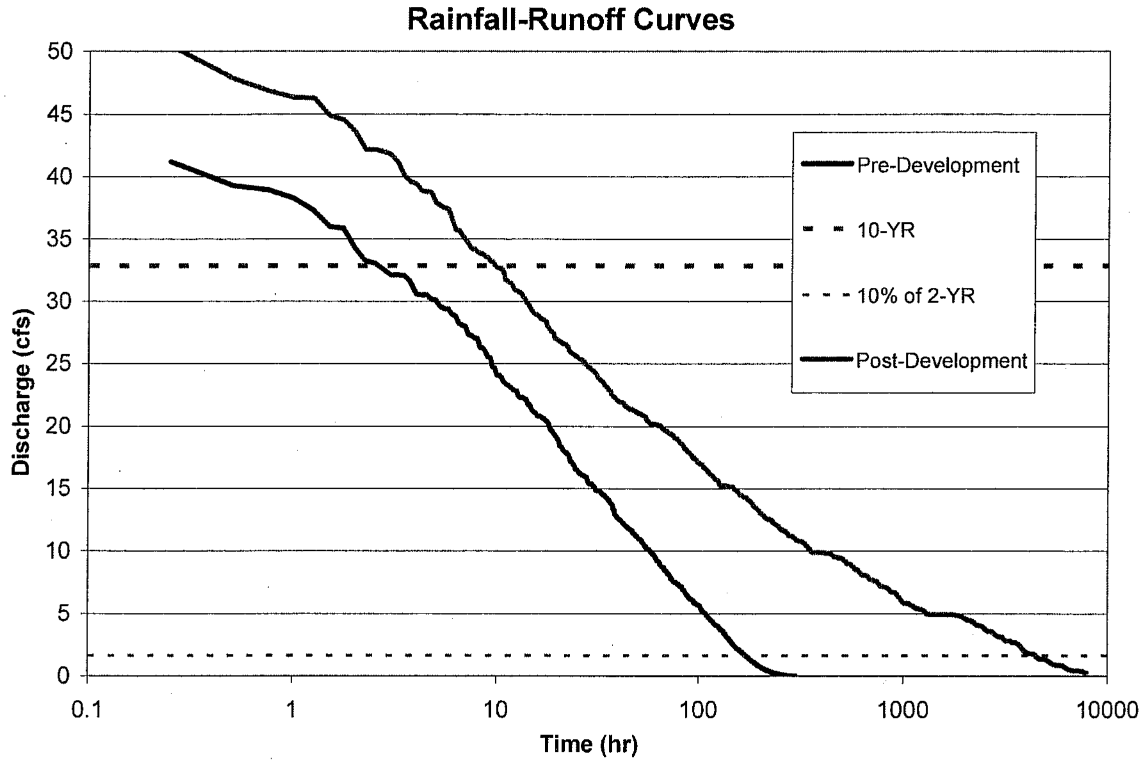

Figure 2. Hydrographs for Berg South Site

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed.

A combination of artificial and natural outflows was modeled to drain the basin. Specifically, weirs and orifices with a small amount of percolation and evaporation were used. We used about 0.39 in/hr of natural percolation outflow from the basin bottom area of 1.21 acres and 0.00208 in/hr of evaporation. Also, this percolation rate was also used while calculating the time to drain for the basin. Weir outflow was based on the equation $Q=CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

The orifice flow was based on two equations: one for open channel flow conditions and one for orifice flow conditions. When the pond level was below the top of the orifice opening (non-pressure flow) Manning's Equation was used. When the pond level was above the top of the orifice the orifice equation, $Q=CA\sqrt{2gh}$, where C is the orifice coefficient (0.6 used), A is the area of the orifice (in feet), g is the gravitational constant and h is the distance from the pond level to the midpoint of the orifice (in feet).

Once a design met the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS model. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. Modifications to the pond design were made if needed after analyzing the pond outflows that HMS calculated. Specifically, we found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probability that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The graph shows that this pond should never take more than 2.5 days to drain.

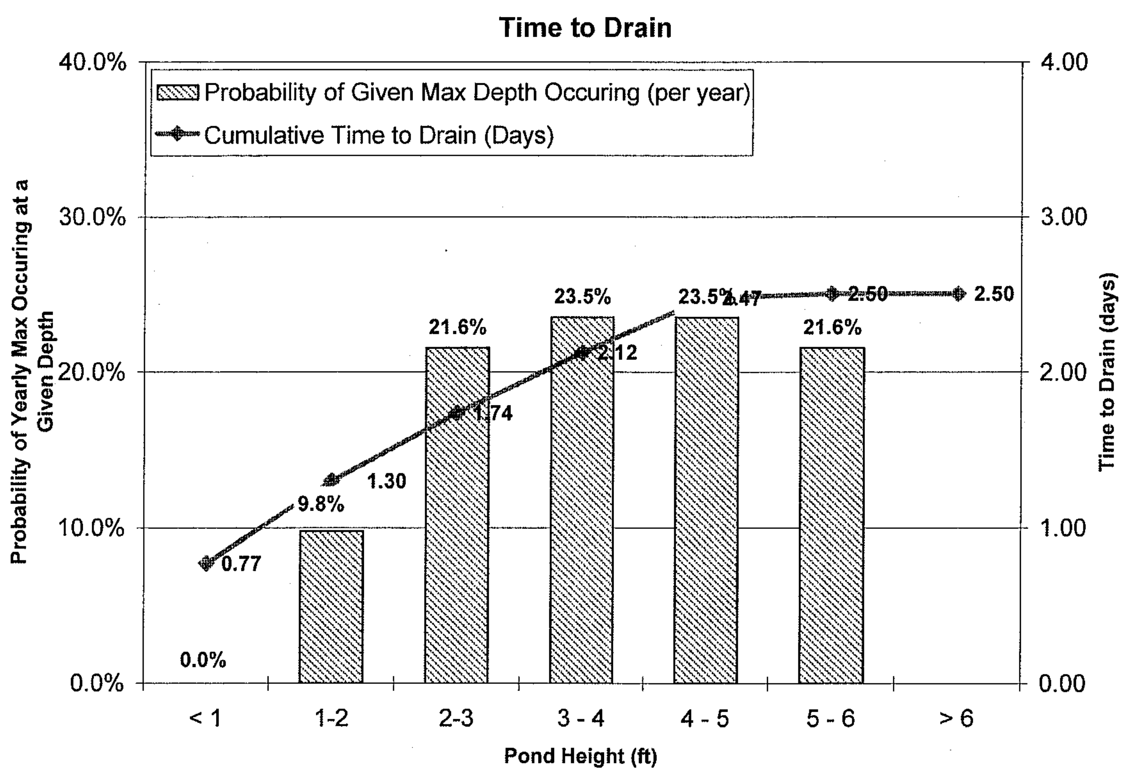

Figure 3. Time to Drain Graph

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Appendix A-9: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	Sept 28, 2005
FROM:	Stephanie Conran and Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	HMP Pond Design Specs for 92 Acre Property (Berg North)		

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the 92-Acre property north of the 147-Acre property. The analysis was complicated by the low values of the flow constraints, as described in the flow-frequency section below, and the relatively high infiltration rate of the B-type soil. To help reduce the artificial flow from the pond, we assumed a small amount of natural percolation through the basin bottom, as discussed below. Overall, a satisfactory detention basin was designed and is detailed in this memo. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

RECOMMENDATIONS

A pond area of about 3.0% of the total project site will be needed to meet the HMP requirements. It was assumed that the basin floor will not be graded and that the side slopes will have a ratio of 2:1. The recommended pond depth is 5.9 ft total, 4.9 ft above the lowest orifice. The bottom foot is assumed is percolate. The other specifics of the basin's design are described below in Tables 1 and 2.

Table 1 – Detention Basin Overview

Maximum Pond Area	2.76 acres (323' x 323' floor)
Maximum Pond Depth	5'11"
Time to Drain	4.48 days max

Table 2 – Detention Basin Outlet Works

Outlet Description	Diameter or Width	Invert above Pond Bottom
Orifice #1	5"	1'
Orifice #2	4"	3'
Weir #1	3'	5'5"

Figure 1 shows the project site flow-frequency curves with the basin routing using MSExcel and HMS. HMP allows the pre-development conditions to be exceeded by a maximum of 10% for no more than 10% of the length of the curve. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event.

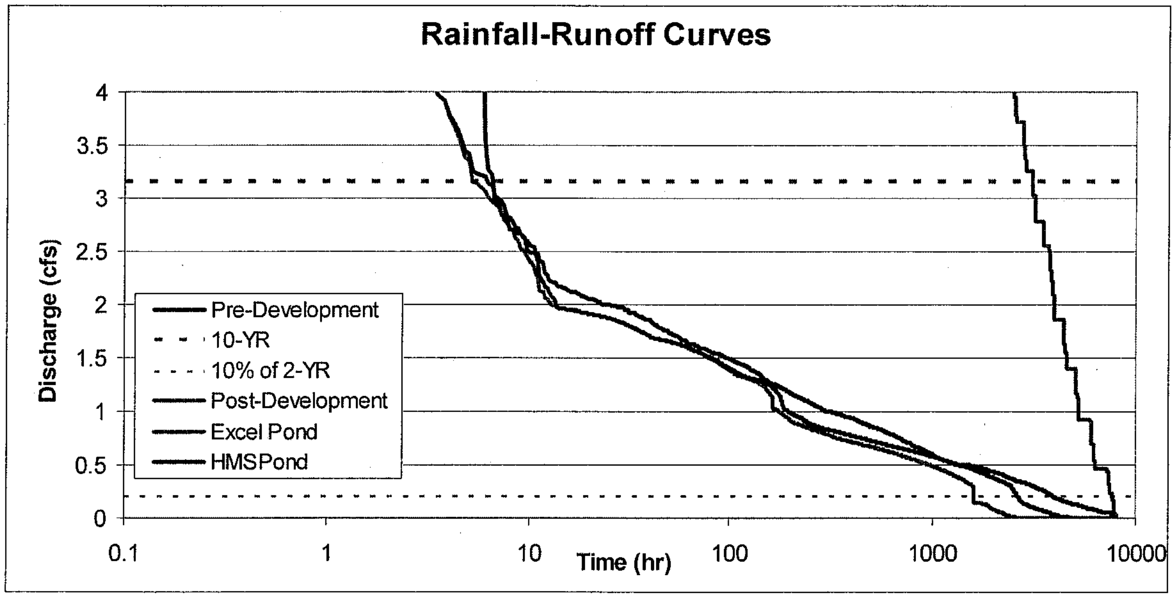

Figure 1. Hydrographs of 92-Acre Site

Site Details

This site currently has very little urban development, so almost all (95%) of the pre-development area is assumed to be pervious. The site is underlain by three types of soil, Altamont-Azule and Hillgate-Positas, which are both Group D soils, and Arbuckle-Pleasanton soil, which is a Group B soil. The Group B Soil has a relatively high porosity and infiltration rate, resulting in many small flows for the existing condition relative to the developed condition, as Figures 1 and 2 make clear. It was calculated that the developed area would be 50% impervious and 50% pervious.

Modeling Overview

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that will conform the post-development hydrograph to the pre-development hydrograph for the 92-Acre site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve, specifically between the flows of the 10-year flood and 10% of the 2-year flood, though HMP requirements allow for 10% exceedence over 10% of the length of the curve within this range. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for each drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. This criterion is addressed at the end of this memo.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions. Existing conditions consist of a single hydrologic basin. For the proposed conditions the model was broken into two basins, a pervious one and an impervious one. The sum of the hydrologic basin areas was set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 16-inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was adjusted from its MAP of 14 inches. Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows: T_c (hrs) = $K*(n*L)^{0.47}*S^{-0.235}$, where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal to one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the 92-Acre site is underlain by mostly Arbuckle-Pleasanton soil with some Altamont-Azule and Hillgate-Positas soils, as well. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 3 summarizes the various parameters we input into the HMS model for the project site. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 3. 92-Acre HMS Parameters.

Parameter		Pre	Post
Area (acres)		92.16	
% Pervious		95.1	50.0
B Soil (%)		72.3	
D Soil (%)		27.7	
T_c (hours)	Pervious	0.221	
	Impervious	N/A	0.160
R (hours)	Pervious	0.281	
	Impervious	N/A	0.107
Canopy Storage Capacity (in)	Pervious	0.25	0.10
	Impervious	0	0
Surface Storage Capacity (in)	Pervious	0.375	
	Impervious	0.1875	
Soil Infiltration Max. Rate (in/hr)	Pervious	0.443	
	Impervious	0	
Soil Storage Capacity (in)		16.19	
Soil Tension Zone Capacity (in)		5.83	
Soil Percolation Max. Rate (in/hr)		0.295	
Groundwater Storage Capacity (in)		50	
Groundwater Percolation Max. Rate (in/hr)		0.10	
Groundwater Storage Coefficient (hr)		999	

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. The 10-year flow was calculated as 3.156 cfs, and the 2-year was calculated as 2.049 cfs, giving 0.205 cfs as 10% of the 2-year. Figure 2 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines. Although it can be seen that there are several values of much larger magnitude than the 10-year storm, most of these values occurred during only two storms in the recorded data.

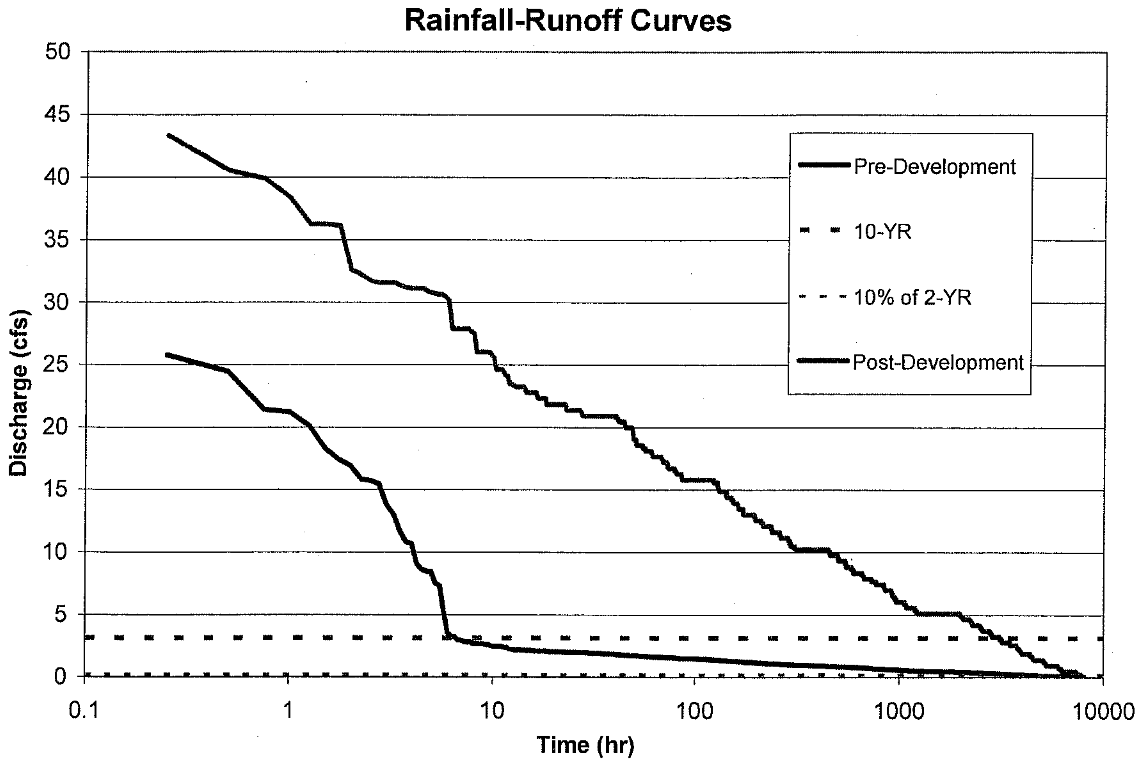

Figure 2. Hydrographs for 92-Acre Site

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed.

A combination of artificial and natural outflows was modeled to drain the basin. Specifically, weirs and orifices with a small amount of percolation and evaporation were used. We used about 0.39 in/hr of natural percolation outflow from the basin bottom area of 2.4 acres and 0.00208 in/hr of evaporation. Also, this percolation rate was also used while calculating the time to drain for the basin.

Weir outflow was based on the following equation $Q=CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

The orifice flow was based on two equations: one for open channel flow conditions and one for orifice flow conditions. When the pond level was below the top of the orifice opening (non-pressure flow) Manning's Equation was used. When the pond level was above the top of the orifice the following orifice equation was applied: $Q=CA\sqrt{2gh}$, where C is the orifice coefficient (0.6 used), A is the area of the orifice (in feet), g is the gravitational constant and h is the distance from the pond level to the midpoint of the orifice (in feet).

Once a design met the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS model. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. Modifications to the pond design were made if needed after analyzing the pond outflows that HMS calculated. Specifically, we found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probably that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The graph shows that this pond should never take more than 4.48 days to drain.

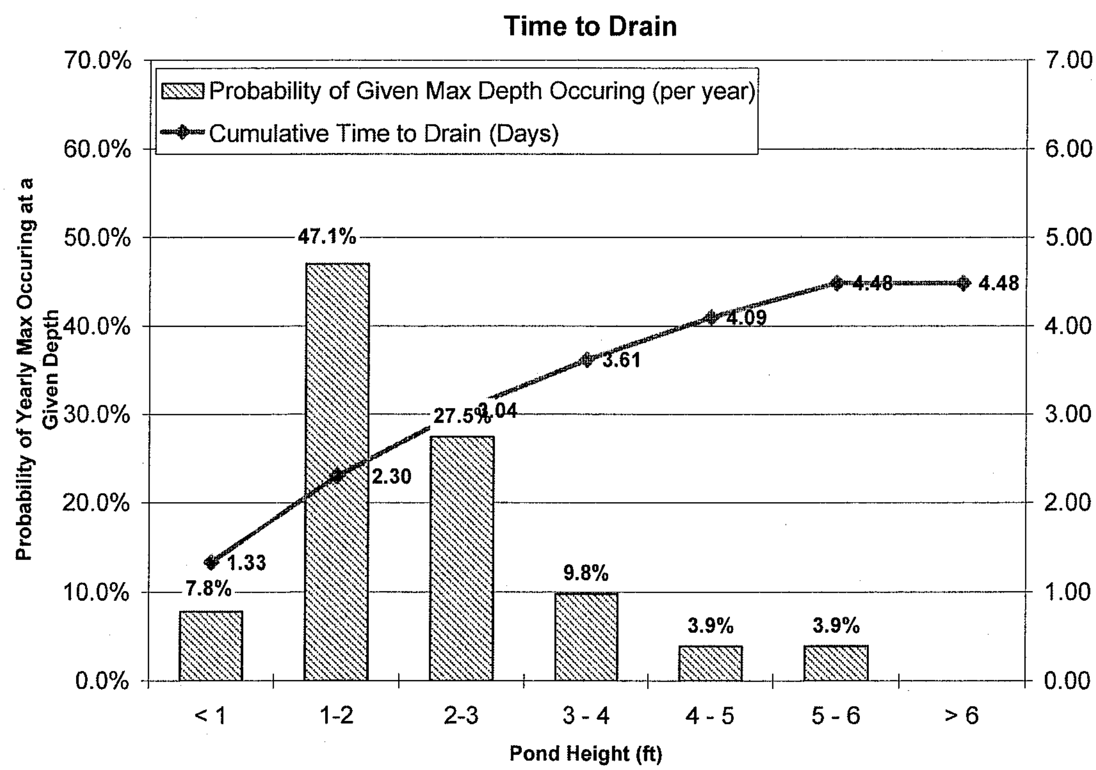

Figure 3. Time to Drain Graph

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Appendix A-10: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	Sept 28, 2005
FROM:	Stephanie Conran and Charles Hardy	JOB #:	HMHI.17.04
SUBJECT:	Alternative HMP Pond Design Specs for 92 Acre Property (Berg North) using precipitation adjustments		

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the 92-Acre property north of the 147-Acre property. The analysis was complicated by the low values of the flow constraints, as described in the flow-frequency section below, and the relatively high infiltration rate of the B-type soil. To help reduce the artificial flow from the pond, we assumed a small amount of natural percolation through the basin bottom, as discussed below. Overall, a satisfactory detention basin was designed and is detailed in this memo. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

RECOMMENDATIONS

A pond area of about 2.0% of the total project site will be needed to meet the HMP requirements. It was assumed that the basin floor will not be graded and that the side slopes will have a ratio of 2:1. The recommended pond depth is 6.3 ft total, 5.3 ft above the lowest orifice. The bottom foot is assumed is percolate The other specifics of the basin's design are described below in Tables 1 and 2.

Table 1 – Detention Basin Overview

Maximum Pond Area	1.85 acres (260' x 260' floor)
Maximum Pond Depth	6'3"
Time to Drain	3.83 days max

Table 2 – Detention Basin Outlet Works

Outlet Description	Diameter or Width	Invert above Pond Bottom
Orifice #1	5"	1'
Orifice #2	3"	1'8"
Orifice #3	3"	4'6"
2 Weirs	8	5'5"

Figure 1 shows the project site flow-frequency curves with the basin routing using MSExcel and HMS. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event. HMP allows the pre-development conditions to be exceeded by a maximum of 10% for no more than 10% of the length of the curve. The HEC-HMS curve has a maximum exceedance of 4.16% and entire curve is exceeded for only 1.90% of its length. The Excel curve exceeds by a maximum of 8.78% and is exceeded for 3.37% of its length. Both curves meet the requirements.

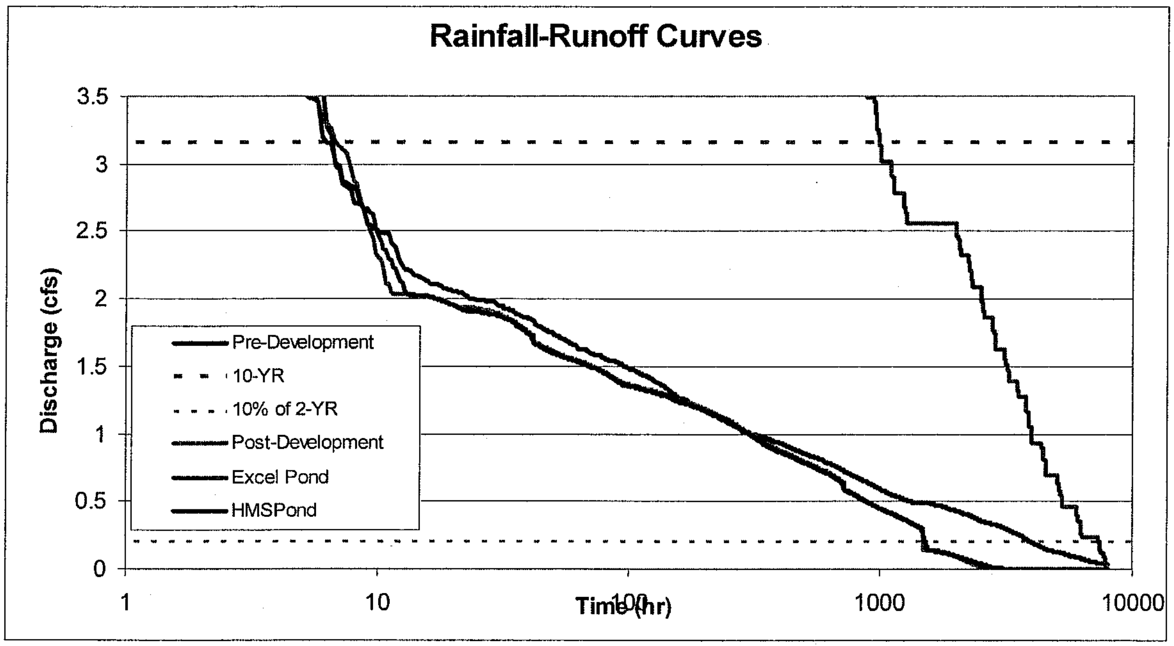

Figure 1. Hydrographs of 92-Acre Site

SITE DETAILS

This site currently has very little urban development, so almost all (95%) of the pre-development area is assumed to be pervious. The site is underlain by three types of soil, Altamont-Azule and Hillgate-Positas, which are both Group D soils, and Arbuckle-Pleasanton soil, which is a Group B soil. The Group B Soil has a relatively high porosity and infiltration rate, resulting in many small flows for the existing condition relative to the developed condition, as Figures 1 and 2 make clear. It was calculated that the developed area would be 50% impervious and 50% pervious.

MODELING OVERVIEW

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that will conform the post-development hydrograph to the pre-development hydrograph for the 92-Acre site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve, specifically between the flows of the 10-year flood and 10% of the 2-year flood, though HMP requirements allow for 10% exceedence over 10% of the length of the curve within this range. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for each drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. This criterion is addressed at the end of this memo.

Hydrologic Modeling – Rainfall-Runoff Simulations

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions. Existing conditions consist of a single hydrologic basin. For the proposed conditions the model was broken into two basins, a pervious one and an impervious one. The sum of the hydrologic basin areas was set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 16-inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was adjusted from its MAP of 14 inches. Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

Also, to adapt the rainfall data to more accurately model the basin, half of the precipitation that would naturally fall on the impervious area was applied the pervious area instead. This is a fairly safe assumption since much of the precipitation that falls on sidewalks, roofs, and roads immediately runs off onto nearby lawns and other pervious areas instead of running directly into the storm sewer system.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows: T_c (hrs) = $K*(n*L)^{0.47}*S^{-0.235}$, where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\frac{R}{R+T_c} = 0.56 \Big\} \text{existing conditions} \qquad \frac{R}{R+T_c} = 0.4 \Big\} \text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal to one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the 92-Acre site is underlain by mostly Arbuckle-Pleasanton soil with some Altamont-Azule and Hillgate-Positas soils, as well. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 3 summarizes the various parameters we input into the HMS model for the project site. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 3. 92-Acre HMS Parameters.

<table>
<tr><th></th><th></th><th>Pre</th><th>Post</th></tr>
<tr><td colspan="2">Area (acres)</td><td colspan="2">92.16</td></tr>
<tr><td colspan="2">% Pervious</td><td>95.1</td><td>50.0</td></tr>
<tr><td colspan="2">B Soil (%)</td><td colspan="2">72.3</td></tr>
<tr><td colspan="2">D Soil (%)</td><td colspan="2">27.7</td></tr>
<tr><td rowspan="2">T_c (hours)</td><td>Pervious</td><td colspan="2">0.221</td></tr>
<tr><td>Impervious</td><td>N/A</td><td>0.160</td></tr>
<tr><td rowspan="2">R (hours)</td><td>Pervious</td><td colspan="2">0.281</td></tr>
<tr><td>Impervious</td><td>N/A</td><td>0.107</td></tr>
<tr><td rowspan="2">Canopy Storage Capacity (in)</td><td>Pervious</td><td>0.25</td><td>0.10</td></tr>
<tr><td>Impervious</td><td>0</td><td>0</td></tr>
<tr><td rowspan="2">Surface Storage Capacity (in)</td><td>Pervious</td><td colspan="2">0.375</td></tr>
<tr><td>Impervious</td><td colspan="2">0.1875</td></tr>
<tr><td rowspan="2">Soil Infiltration Max. Rate (in/hr)</td><td>Pervious</td><td colspan="2">0.443</td></tr>
<tr><td>Impervious</td><td colspan="2">0</td></tr>
<tr><td colspan="2">Soil Storage Capacity (in)</td><td colspan="2">16.19</td></tr>
<tr><td colspan="2">Soil Tension Zone Capacity (in)</td><td colspan="2">5.83</td></tr>
<tr><td colspan="2">Soil Percolation Max. Rate (in/hr)</td><td colspan="2">0.295</td></tr>
<tr><td colspan="2">Groundwater Storage Capacity (in)</td><td colspan="2">50</td></tr>
<tr><td colspan="2">Groundwater Percolation Max. Rate (in/hr)</td><td colspan="2">0.10</td></tr>
<tr><td colspan="2">Groundwater Storage Coefficient (hr)</td><td colspan="2">999</td></tr>
</table>

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. The 10-year flow was calculated as 3.156 cfs, and the 2-year was calculated as 2.049 cfs, giving 0.205 cfs as 10% of the 2-year. Figure 2 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines. Although it can be seen that there are several values of much larger magnitude than the 10-year storm, most of these values occurred during only two storms in the recorded data.

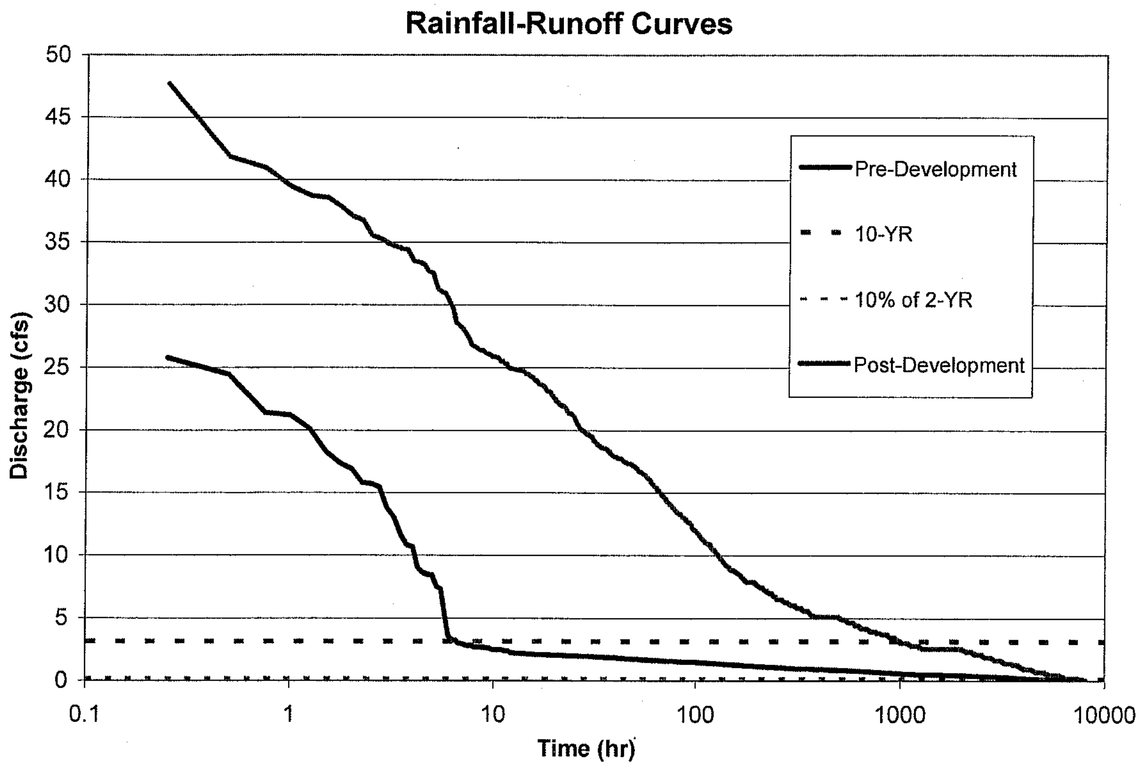

Figure 2. Hydrographs for 92-Acre Site

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed.

A combination of artificial and natural outflows was modeled to drain the basin. Specifically, weirs and orifices with a small amount of percolation and evaporation were used. We used about 0.39 in/hr of natural percolation outflow from the basin bottom area of 1.55 acres and 0.00208 in/hr of evaporation. Also, this percolation rate was also used while calculating the time to drain for the basin.

Weir outflow was based on the following equation $Q=CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

The orifice flow was based on two equations: one for open channel flow conditions and one for orifice flow conditions. When the pond level was below the top of the orifice opening (non-pressure flow) Manning's Equation was used. When the pond level was above the top of the orifice the following orifice equation was applied: $Q=CA\sqrt{2gh}$, where C is the orifice coefficient (0.6 used), A is the area of the orifice (in feet), g is the gravitational constant and h is the distance from the pond level to the midpoint of the orifice (in feet).

Once a design met the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS model. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. Modifications to the pond design were made if needed after analyzing the pond outflows that HMS calculated. Specifically, we found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various yearly maximum pond heights and the probably that that maximum height will occur in a given year. For vector control, the desired maximum time to drain for standing water is three to five days. The graph shows that this pond should never take more than 3.83 days to drain.

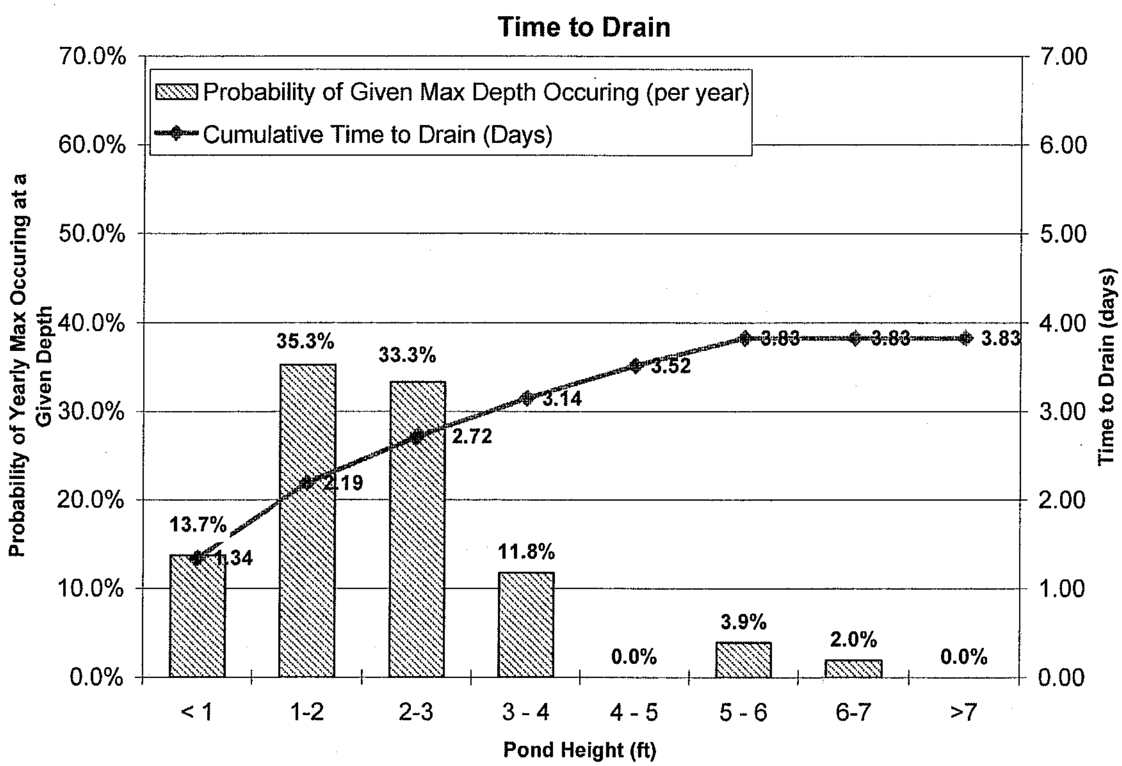

Figure 3. Time to Drain Graph

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Appendix A-11: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, P.E., Ph.D.	DATE:	May 2, 2005
FROM:	Charles Hardy	JOB #:	HMHI.17.04

SUBJECT: Alternative HMP Pond Design Specs for 147-Acre Property (ballpark pond for Berg South, IDS, and Legacy North combined)

Schaaf & Wheeler has created a Hydromodification Management Plan (HMP) for the 147-Acre Property that combines the Berg and IDS properties and parts of the Legacy property in San Jose. The existing land use is mostly open space, and the proposed land use is mostly residential and thus more impervious. A satisfactory detention basin was designed and is detailed in this memo. The results are first presented, followed by summaries of the procedures and parameters used in our hydrologic and hydraulic modeling.

RECOMMENDATIONS

Since the area of the ballfield, where we were told the detention basin would be placed, was already set, the main basin variable that we had to work with was the pond height. It is assumed at the limits of the ballfield area that a berm exists up to the needed pond depth. Our hydraulic model assumes that the berm has straight walls. The recommended pond depth is eight feet, which requires a berm height of about four feet given the current grading plan of the ballfields. The other specifics of the basin's design are described below in Table 1.

Table 1 – Detention Basin Details

Maximum Pond Area	2.79 acres
Maximum Pond Depth	8'
Top Elevation of Ballfield Grading	144'2.5" (4'2.5" above bottom)
Time to Drain	5.22 days max (1.4 days average)
Orifice #1	Diameter = 4" Invert above pond bottom = 0'
Orifice #2	Diameter = 4" Invert above pond bottom = 6"
Orifice #3	Diameter = 4" Invert above pond bottom = 1'
Orifice #4	Diameter = 9" Invert above pond bottom = 6'
Weir #1	Width = 9' Invert above pond bottom = 5'5"
Weir #2	Width = 11' Invert above pond bottom = 7.5'
Weir #3	Width = 12'6" Invert above pond bottom = 8'

Figure 1 shows the project site flow-frequency curves with the basin routing using HEC-HMS. It can be seen that the designed basin adequately modifies the post-development hydrograph to match the hydrograph of the existing conditions within the required parameters of the 10-year flood event and 10% of the 2-year flood event. Indeed, the HMS Pond curve is well under the existing (PRE) curve except for at the low flows, where it is about equal.

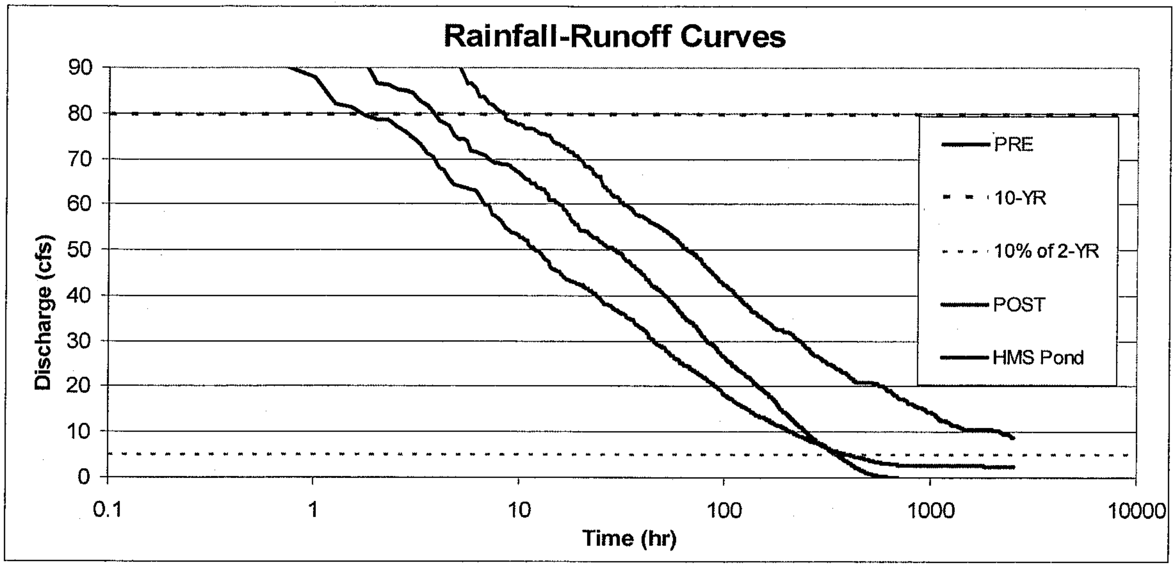

Figure 1. Hydrographs of 147-Acre Site

SITE DETAILS

The 147-acre site currently has no urban development, so all of the pre-development area is assumed to be pervious. The site is underlain by Altamont-Azule soil, which is a Group D soil of low porosity, and Arbuckle-Pleasanton, which is a group B soil of high porosity. The D soil makes up about 82.7% of the underlying soil, with the B soil about 17.3%. Ruth and Going, Inc. provided us with a drawing and calculations of the proposed land use. Combining those numbers with the impervious percentages from the Santa Clara Valley Water District's Hydrology Procedures manual, it was calculated that the developed area would be 50% impervious and 50% pervious.

MODELING OVERVIEW

The US Army Corps of Engineer's HEC-HMS software was used to simulate 53-years of rainfall-runoff at the site. With the results from the HEC-HMS analysis, we then used an MSExcel spreadsheet to design a detention pond that conforms the post-development hydrograph to the pre-development hydrograph for the site. The goal of the basin routing is to modify the post-development flows so that they produce a flow-frequency curve equal to or less than the existing conditions curve. Specifically between the flows of the 10-year flood and 10% of the 2-year flood, the post-development hydrograph must match or be less than the pre-development curve. The area below the 10-percent of the 2-yr flow rate was not matched since it has been suggested that flows below this rate are inconsequential to stream degradation. The flood-frequency curve to get these parameters for each drainage area was found using the Partial-Duration Method and data output from HMS, as described in the next section. The 50-percent (2-year) and 10-percent (10-year) flow rates were extracted from the peaks of the duration curves. The last criterion for the ponds to meet was that they should drain completely after three to five days for mosquito control. Although on average the designed pond should drain within that time frame, there are times when it will take longer to drain. These longer drain times occur, of course, when the pond's depth is near its highest point. However, we assume that since the ponded water will be continuously flowing out of the pond, it will not really ever be standing water. Furthermore, we have presented at the end of this memo a graph and discussion explaining that the longer drain times occur only infrequently each year.

HYDROLOGIC MODELING – RAINFALL-RUNOFF SIMULATIONS

Setting up the HEC-HMS Models

HEC-HMS models were set up for both the pre- and post-development conditions. Existing conditions consist of a single basin. For the proposed conditions the model was broken into two basins, a pervious one and an impervious one. The sum of the hydrologic basin areas was set equal to the development area; this assumes no water enters the site from upstream.

Rainfall was based on the City of San Jose precipitation gage and was adjusted directly proportionate to Mean Annual Precipitation (MAP). The study site has a MAP of 16.5-inches based on the SCVWD GIS shapefile for MAP. The CSJ precipitation data was increased by 27% (16.5-inches divided by 13-inches). Though the CSJ gage data is available in 15-minute intervals we used the 1-hour rainfall data used in the 3-acre Babb Creek sample HMP project. While the rainfall was in one-hour increments, the computations were done on a fifteen-minute basis by assuming four equal amounts of rainfall for each hour.

The Transform method we used for the HMS modeling was the Clark Method, which requires a calculation of the Time of Concentration (T_c) and Storage Coefficient (R). To calculate T_c, we used the Kirby-Hathaway equation, as follows:

$$T_c\,(\text{hrs}) = K*(n*L)^{0.47}*S^{-0.235},$$

where K is a constant = 0.01377, n is the roughness value of the flowpath, L is the approximate overland flow length, and S is the average slope of the land. The length was set as the longest flow path. The Storage Coefficients were then calculated from the Time of Concentration values using the following equations:

$$\left.\frac{R}{R+T_c} = 0.56\right\}\text{existing conditions} \qquad \left.\frac{R}{R+T_c} = 0.4\right\}\text{developed conditions}$$

The Clark values (T_c and R) for post-development conditions were used for the impervious area of the post-development run, and the same Clark values used for the existing (pervious) condition was also used for the pervious post-development condition.

The Soil Moisture Accounting (SMA) method was used as the Loss Rate method in HMS to determine runoff. Several parameters needed to be calculated and inserted into the HMS model, including canopy storage, surface storage capacity, maximum infiltration rate, maximum percolation rate, soil profile storage capacity, tension zone capacity, and characteristics of the groundwater flow.

Canopy Storage was based on existing and proposed land vegetation. Values used for each vegetation type in the SMA are directly from Table C-4 in the HMP Report. The existing land use vegetation is assumed 50-percent orchard and 50-percent alfalfa. The developed pervious areas are assumed to be 60-percent lawn and 40-percent trees. An earlier sensitivity analysis of various variables indicated that the Canopy Storage values do not significantly affect the model conclusions, so we have used the same canopy storage values for this site as the ones we have used for other nearby sites.

Surface Storage Capacity values were based on Maximum Surface Depression Storage values from Table C-4 in the HMP Report. These values match those published in *Open Channel Hydraulics (Chow, 1958)* for medium sloped areas.

Maximum Infiltration Rate was set equal to one-and-a-half times the hydraulic conductivity of the soil (K_{sat}). The K_{sat} values where taken from Table C-3 in the HMP Report. These values only vary by the soil's Hydrologic Group (A, B, C, or D).

Maximum Percolation Rate was set equal to $K_{sat.}$ This matches the method used in the three-acre Babb Creek example.

Soil Profile Storage Capacity values are based on soil classification. According the SCVWD GIS, the 147-acre site is underlain by both Altamont-Azule and Arbuckle-Pleasanton soils. We assumed the GIS soils classification and the Soil Profile Depth from Table C-2 in the HMP Report were appropriate. SMA coefficients for these the drainage areas were set equal to the Group C coefficients.

Tension Zone Capacity was set equal to the Available Water holding Capacity (AWC) from the soil survey. Because the soil classifications in the GIS are not in the soils survey, we assumed the values in Table C-2 of the HMP Report are adequate.

Base flow was not included in the model for these areas; the computer model had this function set to "off" the same as it was in the Babb Creek example.

Ground Water parameters used in the Babb Creek example were used on all HEC-HMS models applied to the Evergreen Area sites. The Storage Capacity was 50 inches; the Percolation Rate was 0.1 inches per hour; and, the Storage Coefficient in hours was 999.

Table 2 summarizes the various parameters we input into the HMS model for the project site. The "Pre" columns give values for the existing conditions, and the "Post" columns give values for the post-development conditions.

Table 2. 147-Acre HMS Parameters.

		Pre	Post
Area (acres)		147.2	
% Pervious		100	50
B Soil (%)		17.3	
D Soil (%)		82.7	
T_c (hours)	Pervious	0.287	
	Impervious	N/A	0.207
R (hours)	Pervious	0.365	
	Impervious	N/A	0.138
Canopy Storage Capacity (in)	Pervious	0.31	0.13
	Impervious	0	0
Surface Storage Capacity (in)	Pervious	0.375	
	Impervious	0.1875	
Soil Infiltration Max. Rate (in/hr)	Pervious	0.1605	
	Impervious	0	
Soil Storage Capacity (in)		14.2	
Soil Tension Zone Capacity (in)		5.75	
Soil Percolation Max. Rate (in/hr)		0.107	
Groundwater Storage Capacity (in)		50	
Groundwater Percolation Max. Rate (in/hr)		0.10	
Groundwater Storage Coefficient (hr)		999	

Determining the Flow-Frequency Curves and Flow Constraints

After inserting the various coefficients into HMS, the basin models were run, and the output flows were extracted into an MSExcel worksheet. Pre-development and post-development flows from HMS were then ranked and plotted. The pre-development curve was used as the matching point for the pond output described in the next section. To determine the flow constraints of the pre-development 10-year and 10% of the 2-year storm flow, the peaks from the pre-development flow were ranked. A peak flow was defined as when the two previous and two following times-steps have less flow. The 10-year flow was calculated as 79.66 cfs, and the 2-year was calculated as 51.60 cfs, giving 5.16 cfs as 10% of the 2-year. Figure 2 below shows the flow-frequency curves of the existing and post-development conditions. The flow constraints are indicated as horizontal lines.

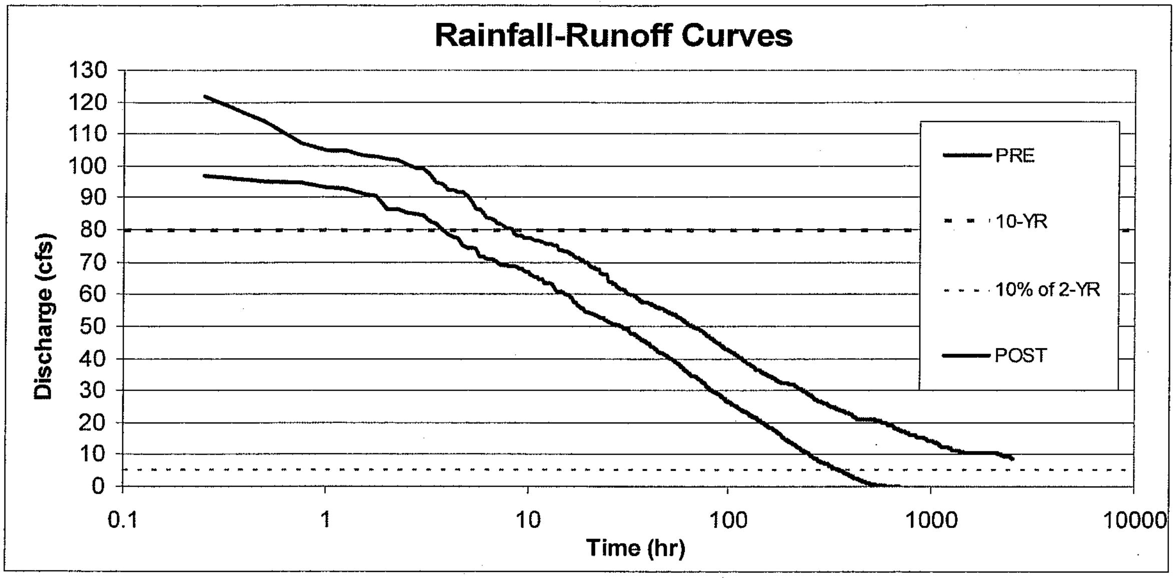

Figure 2. Hydrographs for 147-Acre Site

HYDRAULIC MODELING – DETENTION BASIN DESIGN

Basin routing was performed using an MSExcel spreadsheet, modified from GeoSyntec's spreadsheet. Numerous basin sizes and outlet structures were analyzed.

Weirs and orifices were used to drain the proposed detention basins. For running the overall basin calculations, it was assumed that the basin has no percolation. However, for calculating the time to drain, a percolation of half of the underlying soil's infiltration rate was used. Weir outflow was based on the equation $Q=CLh^{3/2}$, where C is the weir coefficient (3.0 used), L is the length of the weir (in feet) and h is the head above the weir (in feet).

The orifice flow was based on two equations: one for open channel flow conditions and one for orifice flow conditions. When the pond level was below the top of the orifice opening (non-pressure flow) Manning's Equation was used. When the pond level was above the top of the orifice the orifice equation, $Q=CA\sqrt{2gh}$, where C is the orifice coefficient (0.6 used), A is the area of the orifice (in feet), g is the gravitational constant and h is the distance from the pond level to the midpoint of the orifice (in feet).

Once a design met the HMP requirements with the MSExcel routing, the basin design was entered into the HEC-HMS model. Specifically, a rating curve of outflow versus height of the ponded water was input into the Pond element in the HMS model. HMS uses a more sophisticated routing method than the Excel method. Routing with HMS assures the design works properly. Modifications to the pond design were made if needed after analyzing the pond outflows that HMS calculated. Specifically, we found a new rating curve and reran the HMS model. This process was repeated until the HMS output met the hydromodification requirements.

After achieving a satisfactory basin design, we used a small Excel model to calculate the time to drain from various pond heights and the average number of days per year that each height was reached. Although we are reporting the overall time to drain from the highest calculated depth of the pond, most of the time, the pond is calculated to be below that level. In other words, what at first seems like a long time to drain is tempered by the fact that the pond only reaches that level infrequently. Figure 3 helps clarify this with two sets of data. The bars represent the number of distinct times (in average days per year) that there is standing water in the pond at a given height, as given on the abscissa. The other curve represents the time to drain the pond completely from that height. For instance, the largest number of days of standing water occurs when the pond is between one and two feet deep, and at that depth it takes less than a day to drain. This graph shows that most of the time there is standing water in the pond, it will drain in less than five days (120 hours). The average time to drain, weighted by the number of days that standing water is at a specific height, is about 1.4 days (34 hours).

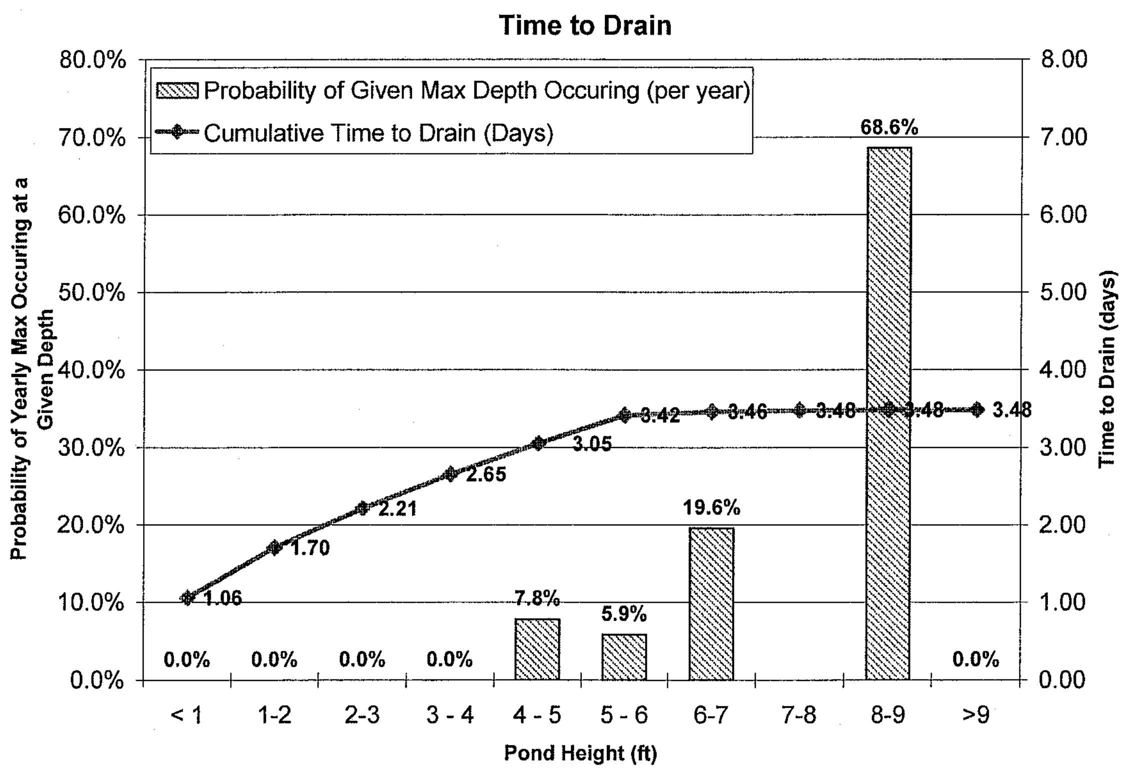

Figure 3. Time to Drain Graph

Appendix B-1: Lower Silver and Thompson Creeks Memo

Introduction

Schaaf & Wheeler conducted a hydrologic analysis using the Santa Clara Valley Water District's HEC-1 model for Lower Silver and Thompson Creeks (Silver-Thompson HEC-1) to evaluate the impacts of the proposed developments of six (6) parcels located within the combined Lower Silver Creek watershed upstream of Lake Cunningham in the Evergreen area of San Jose. The hydrologic analysis evaluated the impacts of the proposed development not only on the individual creeks that the projects drain to (Evergreen Cr., Fowler Creek, Quimby Creek, etc.) but also the more downstream impacts on Fowler Creek and Lower Silver Creek upstream of Lake Cunningham.

The analysis consisted of using the Original District Model (cun100.dat) provided to Schaaf & Wheeler by the District and making revisions to that model to create a Revised District Model, Existing Condition Model, and Project Condition Model as discussed later in this report. The Revised District Model is essentially the Original District Model with minor changes to the split between the pervious and impervious urban areas. The Existing Condition Model was created by removing the 6 proposed parcels (currently undeveloped) from the urban component of each of their respective sub-basins and placing them in the rural component. Finally, the Project Condition Model was created by returning those same portions of land back to the urban component of their respective sub-basins based on the proposed land uses shown on the latest development plans for the 6 parcels.

The hydrologic analysis was completed for the 100-year storm event only. The extent of the analysis was from Thompson Creek upstream of Yerba Buena Creek to Lower Silver Creek upstream of Lake Cunningham.

Parcels

A description of the six parcels, including their size, location and current land use designation is included below:

1. Pleasant Hills Golf Course (PHGC)

The approximately 114-acre existing Pleasant Hills Golf Course (PHGC) site is primarily designated as park in the District's Silver-Thompson HEC-1 model with a small portion of medium-density residential. The parcel is located within sub-basins B30 and B32 with the majority (approx. 85%) in B30. The parcel is underlain by a mixture of C- and D-type soils. The District's HEC-1 model assumes that the urban portions of sub-basins B30 and B32 are 45% impervious. Sub-basin B32 also has a rural component in the District's HEC-1 model.

2. Arcadia

The approximately 80.6-acre Arcadia site is designated as having multiple land uses in the District's Silver-Thompson HEC-1 model, including public/quasi-public, park, medium density residential, commercial and industrial. The parcel is located within sub-basins B20 and B28 with the majority (approx. 74%) in B20. The parcel is underlain entirely by C-type soils. The District's HEC-1 model assumes that the urban portions of sub-basins B20 and B28 are 65% and 80% impervious, respectively. Neither sub-basin has a rural component in the District's HEC-1 model.

3. Evergreen Community College

The approximately 33.6-acre Evergreen Community College (EVCC) site is designated as having public/quasi-public and commercial land uses in the District's Silver-Thompson HEC-1 model. The parcel is located within sub-basins B09, B10 and B12. The parcel is underlain by B- and C-type soils. The District's HEC-1 model assumes that the urban portions of sub-basins B09, B10 and B12 are 50%, 55% and 70% impervious, respectively. Sub-basin B09 also has a rural component in the District's HEC-1 model.

4. Berg South (including IDS and Legacy North)

The approximately 147-acre Berg South site, including IDS and Legacy North, is designated as having medium density residential and industrial land uses in the District's Silver-Thompson HEC-1 model. The parcel is located within sub-basins B12, B13 and B16. The parcel is underlain by B- and D-type soils. The District's HEC-1 model assumes that the urban portions of sub-basins B12, B13 and B16 are 70%, 45% and 65% impervious, respectively. None of the sub-basins have a rural component in the District's HEC-1 model.

5. Berg North

The approximately 92-acre Berg North site is designated as having multiple land uses in the District's Silver-Thompson HEC-1 model, including rural/estate residential, park, medium and high density residential, and industrial. The parcel is located within sub-basins B16 and B19. The parcel is underlain entirely by B- and D-type soils. The District's HEC-1 model assumes that the urban portions of sub-basins B16 and B19 are 65% and 60% impervious, respectively. Neither sub-basin has a rural component in the District's HEC-1 model.

6. Legacy South

The approximately 77.1-acre Legacy South site is designated as entirely industrial in the District's Silver-Thompson HEC-1 model. The parcel is located within sub-basins B09 and B12 with the majority (approx. 70%) in B12. The parcel is underlain by a combination of B-, C- and D-type soils, with the majority being the higher runoff potential D-type soils. The District's HEC-1 model assumes that the urban portions of sub-basins B09 and B12 are 50% and 70% impervious, respectively. Sub-basin B09 also has a rural component in the District's HEC-1 model.

Original District Model

The Original District Model (cun100.dat) was provided to Schaaf & Wheeler by the District. In addition, the District provided Schaaf & Wheeler with the Geographic Information System (GIS) shape files (.shp) for the watershed boundaries, creeks, land use, soils, etc. that were used to create their HEC-1 model. Schaaf & Wheeler was able to use the shape files to confirm the hydrologic parameter values used in the District's model. Finally, the District also provided Schaaf & Wheeler with their assumed values for Antecedent Moisture Condition (AMC), percent impervious tables for various land uses, etc. that they used to calculate the Curve Numbers (CN) and pervious/impervious land area splits for the urban components of each of the sub-basins.

The Original District Model for Thompson Creek and Lower Silver Creek is split into several sub-basins for the various tributary creeks (Evergreen Cr., Fowler Cr., Quimby Cr., Norwood Cr., etc.) as well as tributary areas draining directly to Thompson Creek and Lower Silver Creek. Thompson Creek joins together with Lower Silver Creek downstream of Norwood Creek to become Lower Silver Creek, eventually draining into Lake Cunningham.

The sub-basins are broken into urban and rural components with the urban component being further broken down into pervious and impervious components. The urban/rural split is primarily based on the City of San Jose's Urban Service Boundary and the urban pervious/impervious split is based on the assumed percent impervious for the various land uses within the sub-basin. The land use assumptions are based on the Land Use/Transportation Diagram contained in the City of San Jose's 2020 General Plan.

Revised District Model

The Water District's Silver-Thompson HEC-1 model used a Geographic Information System (GIS) overlay of the hydrologic sub-basins with the Land Use/Transportation Diagram contained in the City of San Jose's 2020 General Plan to estimate the percent imperviousness of the urban portion of each sub-basin. The calculated percent imperviousness was rounded up to nearest 5% which makes sense for computing design flows for the various channels; however, it makes it difficult to assess the impacts of minor changes in land use(s) within the sub-basins. Therefore, Schaaf & Wheeler used the District's own GIS shape files and the same land use vs. percent impervious tables used by the District to calculate the actual percent imperviousness for each sub-basin. These values were used to revise the split between pervious and impervious area (BA card in HEC-1 model) for the urban components of each of the sub-basins contained in the District's Silver-Thompson HEC-1 model. The resulting model (revcun.dat) is herein referred to as the "Revised District Model." Table 1 shows a comparison of the percentage impervious of each of the sub-basins with an urban component for the Original District Model and the Revised District Model.

Table 1 – Percentage Impervious for Original and Revised District Models

Sub-Basin	Percentage Impervious	
	Original District Model	Revised District Model
B02	30.0	30.0
B03	30.0	31.25
B04	29.2	30.0
B05	22.6	25.0
B06	46.0	50.0
B07	31.0	35.0
B09	46.7	50.0
B10	52.6	55.0
B12	69.6	70.0
B13	44.9	45.0
B16	64.2	65.0
B17	56.2	60.0
B18	15.0	20.0
B19	57.6	60.0
B20	64.1	65.0
B223	40.3	45.0
B24	67.1	70.0
B25	31.1	65.0
B26	52.6	55.0
B27	64.6	65.0
B28	76.9	80.0
B30	44.6	45.0
B32	44.1	45.0
B33	40.5	45.0

Table 2 provides a breakdown of the area, in square miles, of the area designated as urban pervious, urban impervious and rural in the Revised District Model for each of the sub-basins included in this analysis.

Table 2 – Urban and Rural Areas in Revised District Model

Sub-Basin	Urban		Rural (mi^2)
	Pervious (mi^2)	Impervious (mi^2)	
B09	0.473	0.415	0.149
B10	0.143	0.159	---
B12	0.101	0.232	---
B13	0.704	0.572	---
B16	0.162	0.289	---
B19	0.404	0.549	---
B20	0.313	0.559	---
B28	0.061	0.201	---
B30	0.586	0.471	---
B32	0.193	0.152	0.336

No changes were made to any of the other hydrologic parameters, including Curve Numbers, times of concentration, initial abstraction, Clark's storage coefficient, etc. to create the Revised District Model.

The Revised District Model is reflective of the condition in which the entire Lower Silver Creek and Thompson Creek watersheds are developed according to the current land use zoning shown in the City's 2020 General Plan.

Existing Condition Model

The District's Silver-Thompson HEC-1 model is based on the Land Use/Transportation Diagram contained in the City of San Jose's 2020 General Plan. Not all of the parcels of land located within the Lower Silver and Thompson Creek watersheds have been fully developed, including the six parcels included in this analysis; therefore an Existing Condition Model (excun.dat) was created that assumes all of the parcels are developed with the exception of the 6 parcels included in this analysis. To create this model, the undeveloped portions of these 6 parcels were taken out of the urban component of the sub-basin and placed in a rural component and the size of the pervious and impervious portions of the urban component were adjusted accordingly. No revisions were made for the Pleasant Hills Golf Course site since it is already developed as shown on the land use map. Similarly, approximately 5.4 acres of the Evergreen Community College site has already been developed as public/quasi-public as shown on the land use map. This 5.4 acre portion is located within sub-basins B10 and B12 with approximately half of its land area located in each sub-basin.

Table 3 provides a summary of the acreage of each type of land use that was removed from the urban component of each sub-basin in the Revised District Model and placed in the rural component. It is important to note that not all of the sub-basins had a rural component; therefore a rural component had to be added for several of the sub-basins. Please see the discussion below regarding the hydrologic parameters for the rural component.

Table 3 – Acreage Transferred from Urban to Rural in Existing Condition Model

Sub-Basin	Parcel	Area (acres) Transferred to Rural by Land Use							
		Rural/Estate Residential	Low Density Residential	Medium Density Residential	High Density Residential	Commercial	Industrial	Public/Quasi Public	Parks
B09	EVCC							5.82	
	Legacy South						23.16		
B10	EVCC					1.1		14.7	
B12	EVCC							6.39	
	Legacy South						54.12		
	Berg South						21.14		
B13	Berg South	1.86		1.65	1.30		99.47		
B16	Berg North			0.39			45.15		0.18
	Berg South			0.40			17.86		
B20	Arcadia			26.5		27.8	1.1	4.2	
B28	Arcadia			6.2		4.8		8.9	0.7
B30	PHGC	No Change – Golf Course Already Developed							
B32	PHGC	No Change – Golf Course Already Developed							

Table 4 provides a breakdown of the area, in square miles, of the area designated as urban pervious, urban impervious and rural in the Existing Condition Model for each of the sub-basins included in this analysis.

Table 4 – Urban and Rural Areas in Existing Condition Model

Sub-Basin	Urban		Rural (mi^2)
	Pervious (mi^2)	Impervious (mi^2)	
B09	0.469	0.374	0.045
B10	0.141	0.137	0.025
B12	0.088	0.117	0.128
B13	0.684	0.429	0.163
B16	0.151	0.200	0.100
B19	0.393	0.502	0.058
B20	0.293	0.486	0.093
B28	0.054	0.176	0.032
B30	0.465	0.441	0.152
B32	0.172	0.147	0.027

Revising the amount of urban area in the Existing Condition Model required that the storage-discharge relationships be modified for the storm drain routing step for the urban component of each of the sub-basins. The relationships were revised using the appropriate Generalized Unitized Storage-Discharge Rating Curve shown on Figure 7 of the District's Hydrology Procedures.

The Curve Numbers (CN) for the newly created rural components for each of sub-basis were calculated based on GIS overlays of the sub-basins and the soils shape files provided by the District. For the two sub-basins that already had a rural component in the Original District Model (B09 and B32), the CN values were revised to reflect the additional area added to the rural component according to the soil type(s) of the additional area. An Antecedent Moisture Condition (AMC) of II ¼ was used to calculate the CN values. The CN values for the rural component of each of the sub-basins included in this analysis are shown on Table 5. The time of concentration (T_c) for the newly created rural components for each sub-basin were assumed to be 0.5 hours (30 minutes) and the Clark's Storage Coefficient, R, was calculated based on the assumed relationship, $R/(T_c + R) = 0.75$, thus $R = 1.5$. The initial abstraction values, I_a, were calculated based on the relationship, $I_a = 0.2 * (1000/CN - 10)$.

Table 5 – CN Values for Rural Component in Existing Condition Model

Sub-Basin	CN
B09	79
B10	65
B12	79
B13	76
B16	68
B19	66
B20	74
B28	74
B30	N/A
B32	N/A

The Existing Condition Model is reflective of the condition in which the entire Lower Silver Creek and Thompson Creek watersheds are developed according to the current land use zoning shown in the City's 2020 General Plan with the exception of the 6 parcels which would remain undeveloped as they are currently.

Project Condition Model

Several site development scenarios exist for each of the six parcels included in this analysis. The "worst case" scenario (that resulting in the greatest amount of impervious surfaces for each of the parcels) was used to develop a Project Condition Model (prjcun.dat) that essentially took the acreage that was placed in the rural component of the Existing Condition Model in the previous step and reverted it back to the urban component. In this step, additional area was added to the urban pervious and urban impervious components based on the calculated percent imperviousness associated with the "worst case" scenario for each of the parcels. The calculations to estimate the percent imperviousness for each of the parcels were done independently by Charles Hardy of Schaaf & Wheeler and the results are shown on Table 6.

Table 6 – Percent Imperviousness for Development Parcels

Parcel	Area (acres)	Percent Impervious (%)
PHGC	114	56
Arcadia	80.6	77
EVCC	33.6	90
Berg South	147.2	50
Berg North	92.2	50
Legacy South	77.1	50

Table 7 provides a breakdown of the area, in square miles, of the area designated as urban pervious, urban impervious and rural in the Existing Condition Model for each of the sub-basins included in this analysis.

Table 7 – Urban and Rural Areas in Project Condition Model

Sub-Basin	Urban		Rural (mi^2)
	Pervious (mi^2)	Impervious (mi^2)	
B09	0.488	0.400	0.149
B10	0.143	0.159	---
B12	0.148	0.185	---
B13	0.766	0.510	---
B16	0.201	0.250	---
B19	0.422	0.531	---
B20	0.315	0.557	---
B28	0.061	0.201	---
B30	0.531	0.526	---
B32	0.183	0.162	0.336

The Project Condition Model is reflective of the condition in which the entire Lower Silver Creek and Thompson Creek watersheds are developed according to the current land use zoning shown in the City's 2020 General Plan with the exception of the 6 parcels which would be developed according their proposed development plans. It should be noted that the proposed development will result in less impervious land area than that permitted by the land use zoning shown in the City's 2020 General Plan.

Results

Tables 8 through 11 on the following pages show the peak, 6-, 24-, and 72-hour average 100-year discharge estimates for Yerba Buena, Evergreen, Fowler, Quimby and Norwood Creeks as well as Thompson Creek upstream and downstream of each of those creeks. In addition, the tables show the discharges in Thompson Creek upstream of Lower Silver Creek, Lower Silver Creek upstream and downstream of Thompson Creek and Lower Silver Creek upstream of Lake

Cunningham. The discharge estimates are shown for the Original District Model, Revised District Model, Existing Condition Model and Project Condition Model. Finally, the percent difference between the existing and project conditions was calculated to show the impact on the proposed development of the six parcels on the discharges within the various creeks as compared to the present condition.

Conclusion

The results of the hydrologic analysis, as shown on Tables 8 through 11 clearly demonstrate that the proposed development of the 6 parcels would have a negligible impact on the 100-year discharges in Evergreen, Fowler, Quimby, Norwood, Flint, Ruby, Thompson and Lower Silver Creeks. The 100-year peak discharge in Lower Silver Creek upstream of Lake Cunningham would remain virtually unchanged. Any increases predicted for the tributary streams (Evergreen Cr., Fowler Cr., Quimby Cr.) would also be mitigated by on-site detention, which was not included in this analysis. In short, the proposed development for the 6 parcels would have a negligible impact on runoff to the various creeks as compared to the existing runoff and the proposed development results in lower runoff than that predicted by the currently zoned land uses for the 6 parcels.

Table 8 - Peak Discharge (cfs)

Location	Original District HEC-1 Model	Revised District HEC-1 Model	Existing Condition HEC-1 Model	Project Condition HEC-1 Model	% Change
Thompson Cr. Upstream of Yerba Buena Cr.	1307	1304	1304	1304	0.00%
Yerba Buena Cr. Upstream of Thompson Cr.	517	515	515	514	-0.19%
Thompson Cr. Downstream of Yerba Buena Cr.	1816	1810	1810	1809	-0.06%
Thompson Cr. Upstream of Evergreen Cr.	1814	1808	1807	1807	0.00%
Evergreen Cr. Upstream of Thompson Cr.	365	365	380	363	-4.47%
Thompson Cr. Downstream of Evergreen Cr.	2206	2201	2192	2198	0.27%
Thompson Cr. Upstream of Fowler Cr.	2205	2200	2190	2197	0.32%
Fowler Cr. Upstream of Thompson Cr.	510	510	500	508	1.60%
Thompson Cr. Downstream of Fowler Cr.	2885	2879	2860	2871	0.38%
Thompson Cr. Upstream of Quimby Cr.	2884	2878	2860	2870	0.35%
Quimby Cr. Upstream of Thompson Cr.	434	432	428	431	0.70%
Thompson Cr. Downstream of Quimby Cr.	3376	3368	3345	3358	0.39%
Thompson Cr. Upstream of Norwood Cr.	3375	3367	3344	3358	0.42%
Norwood Cr. Upstream of Thompson Cr.	606	601	601	601	0.00%
Thompson Cr. Downstream of Norwood Cr.	4020	4008	3999	3998	-0.03%
Thompson Cr. Upstream of Lower Silver Cr.	**4020**	**4008**	**3999**	**3998**	**-0.03%**
Lower Silver Cr. Upstream of Thompson Cr.	300	298	296	298	0.68%
Lower Silver Cr. Downstream of Thompson Cr.	**4319**	**4306**	**4294**	**4296**	**0.05%**
Ruby Cr. Upstream of Flint Cr.	294	294	294	297	1.02%
Flint Cr. Upstream of Ruby Cr.	525	525	525	525	0.00%
Lower Silver Cr. Upstream of Lake Cunningham	**5088**	**5074**	**5063**	**5065**	**0.04%**

Table 9 - 6-Hour Average Discharge (cfs)

Location	Original District HEC-1 Model	Revised District HEC-1 Model	Existing Condition HEC-1 Model	Project Condition HEC-1 Model	% Change
Thompson Cr. Upstream of Yerba Buena Cr.	987	985	985	985	0.00%
Yerba Buena Cr. Upstream of Thompson Cr.	396	395	394	395	0.25%
Thompson Cr. Downstream of Yerba Buena Cr.	1383	1381	1379	1380	0.07%
Thompson Cr. Upstream of Evergreen Cr.	1383	1380	1379	1380	0.07%
Evergreen Cr. Upstream of Thompson Cr.	230	230	223	228	2.24%
Thompson Cr. Downstream of Evergreen Cr.	1654	1651	1641	1648	0.43%
Thompson Cr. Upstream of Fowler Cr.	1654	1651	1641	1649	0.49%
Fowler Cr. Upstream of Thompson Cr.	366	365	357	364	1.96%
Thompson Cr. Downstream of Fowler Cr.	2201	2197	2174	2191	0.78%
Thompson Cr. Upstream of Quimby Cr.	2201	2197	2174	2191	0.78%
Quimby Cr. Upstream of Thompson Cr.	339	338	334	338	1.20%
Thompson Cr. Downstream of Quimby Cr.	2593	2589	2561	2582	0.82%
Thompson Cr. Upstream of Norwood Cr.	2593	2589	2561	2582	0.82%
Norwood Cr. Upstream of Thompson Cr.	458	456	456	456	0.00%
Thompson Cr. Downstream of Norwood Cr.	3129	3123	3097	3116	0.61%
Thompson Cr. Upstream of Lower Silver Cr.	**3129**	**3123**	**3097**	**3116**	**0.61%**
Lower Silver Cr. Upstream of Thompson Cr.	249	248	246	248	0.81%
Lower Silver Cr. Downstream of Thompson Cr.	**3378**	**3370**	**3343**	**3363**	**0.60%**
Ruby Cr. Upstream of Flint Cr.	227	227	227	228	0.44%
Flint Cr. Upstream of Ruby Cr.	350	350	350	350	0.00%
Lower Silver Cr. Upstream of Lake Cunningham	**4034**	**4025**	**3998**	**4019**	**0.53%**

Table 10 - 24-Hour Average Discharge (cfs)

Location	Original District HEC-1 Model	Revised District HEC-1 Model	Existing Condition HEC-1 Model	Project Condition HEC-1 Model	% Change
Thompson Cr. Upstream of Yerba Buena Cr.	506	504	504	504	0.00%
Yerba Buena Cr. Upstream of Thompson Cr.	210	209	206	208	0.97%
Thompson Cr. Downstream of Yerba Buena Cr.	716	713	710	712	0.28%
Thompson Cr. Upstream of Evergreen Cr.	716	713	710	712	0.28%
Evergreen Cr. Upstream of Thompson Cr.	114	114	108	112	3.70%
Thompson Cr. Downstream of Evergreen Cr.	856	853	843	850	0.83%
Thompson Cr. Upstream of Fowler Cr.	856	852	843	850	0.83%
Fowler Cr. Upstream of Thompson Cr.	184	184	177	182	2.82%
Thompson Cr. Downstream of Fowler Cr.	1153	1149	1124	1142	1.60%
Thompson Cr. Upstream of Quimby Cr.	1153	1149	1124	1142	1.60%
Quimby Cr. Upstream of Thompson Cr.	190	189	185	188	1.62%
Thompson Cr. Downstream of Quimby Cr.	1373	1368	1340	1360	1.49%
Thompson Cr. Upstream of Norwood Cr.	1373	1368	1340	1360	1.49%
Norwood Cr. Upstream of Thompson Cr.	283	261	261	261	0.00%
Thompson Cr. Downstream of Norwood Cr.	1709	1701	1670	1693	1.38%
Thompson Cr. Upstream of Lower Silver Cr.	**1709**	**1701**	**1670**	**1693**	**1.38%**
Lower Silver Cr. Upstream of Thompson Cr.	154	152	151	152	0.66%
Lower Silver Cr. Downstream of Thompson Cr.	**1862**	**1853**	**1820**	**1845**	**1.37%**
Ruby Cr. Upstream of Flint Cr.	134	134	134	136	1.49%
Flint Cr. Upstream of Ruby Cr.	174	174	174	175	0.57%
Lower Silver Cr. Upstream of Lake Cunningham	**2222**	**2212**	**2178**	**2205**	**1.24%**

Table 11 - 72-Hour Average Discharge (cfs)

Location	Original District HEC-1 Model	Revised District HEC-1 Model	Existing Condition HEC-1 Model	Project Condition HEC-1 Model	% Change
Thompson Cr. Upstream of Yerba Buena Cr.	173	172	172	172	0.00%
Yerba Buena Cr. Upstream of Thompson Cr.	72	71	70	71	1.43%
Thompson Cr. Downstream of Yerba Buena Cr.	245	243	242	243	0.41%
Thompson Cr. Upstream of Evergreen Cr.	245	244	243	243	0.00%
Evergreen Cr. Upstream of Thompson Cr.	38	38	36	38	5.56%
Thompson Cr. Downstream of Evergreen Cr.	293	291	288	290	0.69%
Thompson Cr. Upstream of Fowler Cr.	283	282	288	291	1.04%
Fowler Cr. Upstream of Thompson Cr.	62	62	60	62	3.33%
Thompson Cr. Downstream of Fowler Cr.	395	393	385	391	1.56%
Thompson Cr. Upstream of Quimby Cr.	395	394	385	391	1.56%
Quimby Cr. Upstream of Thompson Cr.	65	65	63	64	1.59%
Thompson Cr. Downstream of Quimby Cr.	471	469	459	466	1.53%
Thompson Cr. Upstream of Norwood Cr.	472	470	460	467	1.52%
Norwood Cr. Upstream of Thompson Cr.	90	89	89	89	0.00%
Thompson Cr. Downstream of Norwood Cr.	592	589	577	586	1.56%
Thompson Cr. Upstream of Lower Silver Cr.	**592**	**589**	**577**	**586**	**1.56%**
Lower Silver Cr. Upstream of Thompson Cr.	53	53	52	53	1.92%
Lower Silver Cr. Downstream of Thompson Cr.	**645**	**641**	**629**	**638**	**1.43%**
Ruby Cr. Upstream of Flint Cr.	46	46	46	46	0.00%
Flint Cr. Upstream of Ruby Cr.	59	59	59	59	0.00%
Lower Silver Cr. Upstream of Lake Cunningham	**769**	**766**	**753**	**763**	**1.33%**

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Appendix C: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, Ph.D., P.E.	DATE:	August 2, 2005
FROM:	Charles Hardy	JOB #:	HMHI.17.04

SUBJECT: Routing of Lower Silver Creek flooding through Lake Cunningham

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As Lower Silver Creek (LSC) converges with other creeks before Cunningham Avenue in San José, flood waters can spill into Lake Cunningham (Lake C) and its adjoining park and low-lying areas. In other words, Lake C Park adds storage to the flood system. Since the site currently known as Pleasant Hills Golf Course (PHGC) is hydraulically linked to the Lake C Park through culverts, altering the grading of the golf course site may possibly affect the flooding of the park. The EIR for the Evergreen Specific Plan includes environmental effects of altering the PHGC site, so this memo details our work to calculate the nature and magnitude of such effects.

Several pieces of information were used for the analysis of the park's 100-year flooding under existing and project conditions. First, a HEC-1 model of LSC originally created by the Santa Clara Valley Water District in February of 2001 was used as the baseline for our own HEC-1 model of flooding in the park. Jim Gessford had already recently modified this HEC-1 model to represent both existing conditions in Lake C Park and its vicinity as well as proposed conditions under the EIR scenario having the most substantial development. Therefore, his models for existing and project conditions were used with slight modifications to allow routing through the park. Using AutoCAD, topographic information for both the park area and the PHGC site was also used for calculating the weir values described below and the elevation-storage curves input into HEC-1. The PHGC topography was provided by HMHI Engineers, dated June 1989, and the Lake C Park topography is from the Water District and CH2MHill, dated June 2000. ArcView GIS software with its CAD Reader, Spatial Analyst, and 3-D Analyst extensions was also used to calculate elevation-storage curves from the topographic information.

If the flooding in the park is great enough, floodwaters can overtop and spill out of the Lake C Park. Therefore, some overflow has been modeled as staged weirs, using the following standard weir equation, $Q = CLh^{1.5}$, where Q is the flow over the weir in cfs, C is the weir coefficient, L is the length of the weir in feet, and h is the depth of the flow going over the weir in feet. The value for C is based on the nature of the weir; we have used 2.6. The lowest point on the park's outskirts where flow can overtop these "weirs" is at elevation 131' (in NAVD); at this elevation the weir length is 21'. At elevation 132', the weir is 116' long; at 133', 717' long; and at 134', 2159' long. A weir length was not estimated above 134' since there are several more places where the flow would start flowing out, and much more substantial losses of the flood waters would occur. Furthermore, after running the HEC-1 simulations, it was found that the flood stage does not reach that elevation.

Using the HEC-1 modeling software, we calculated the stage of the Lake C flooding due to the overflow from LSC. Added to Jim Gessford's modified models were the following items:

- a diversion of flow out of the park – representing the flood flow that continues downstream past the park,
- an overall elevation-storage curve to represent storage in the Lake C Park and PHGC site, and
- an elevation-discharge curve representing weir flows out of the park due to flooding.

Other changes included combining a few hydrographs at different points than the Water District and Gessford's models to allow for the insertion and analysis of the Lake C Park storage. Details of the changes can be seen in the HEC-1 data files.

The diversion of flow out of the park represents the downstream flow after the confluence of LSC and Ruby and Flint Creeks. Hydraulic limitations on the downstream creek allow only a certain amount of flow past this confluence point, and the balance of the flow floods the Lake C Park. We ran the model with both 2,850 cfs and 2,000 cfs as the downstream flows, with 2,000 cfs being the allowed flow in the creek a few blocks past Cunningham Avenue at Ocala Avenue. With the higher amount of diverted flow, the park flooded to about 129.8' (NAVD) for existing conditions and 129.9' for project conditions; thus, no weir outflow occurred since the peak flooding stage was below the lowest weir. Under the second scenario with less diverted flow – and, consequently, more flood flow entering the park – the peak flood stage was found to be 132.5' for existing conditions and 132.6' for project conditions. The existing conditions with 2,000 cfs diverted would have 834 cfs peak overflow out of the park; the project conditions would have 835 cfs overflowing. The differences between the peak elevations and peak weir flows for both conditions are insignificant. Therefore, the project will have insignificant environmental effects on the flooding and storage along this portion of Lower Silver Creek.

The existing volume and area of the PHGC site at or below 132.5' are 20.9 acre-feet and 15.9 acres, respectively. Assuming the PHGC site would flood to the same level as the Lake C Park, re-grading the PHGC site would remove that much volume and surface area at the highest calculated flood elevation for existing conditions. This amounts to 1.9% of the overall storage of the site and park being removed, which is an insignificant amount, especially since the weir outflow from the park due to the lost storage is negligibly changed by the removal.

It should be noted that the calculated water surface elevation in the flooded park is lower than the FEMA value of 131' (NGVD), which is about 133.9' in NAVD, the datum by which our values are reported. For the scenario with 2,000 cfs diverted flow, we have calculated almost a foot-and-a-half less for the peak flood elevation. Through comparison of our model with the original model, it seems that the original model somehow underestimated the amount of storage in the park, although we do not have more details on how the storage for the original model was calculated. Given the higher storage in our model, the flood waters do not reach as high an elevation. Since we have used more recent topography, it is reasonable to assume our values of storage are accurate.

Routing of Lower Silver Creek flooding through Lake Cunningham

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Appendix D: TECHNICAL MEMORANDUM

TO:	Jim Schaaf, Ph.D., P.E.	DATE:	August 29, 2005
FROM:	Stephanie Conran	JOB #:	HMHI-17-04 003

SUBJECT: Effects of Land Use and Grading Changes on the Water Surface Elevation of Lake Cunningham

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The site currently known as Pleasant Hills Golf Course (PHGC) is hydraulically linked to Lake Cunningham Park through culverts. Altering the grading and the land use of the golf course site may affect the water surface elevation of Lake Cunningham. The development of PHGC will change both the land use as well as the topography, so this memo details our work to calculate the nature and magnitude of such effects through the use of HEC-1.

Under existing conditions, with 2000 cfs being diverted, the maximum water surface elevation is 132.5 ft NAVD. With the existing topographical conditions and future land use changes, the maximum flood stage was also found to be 132.5 ft NAVD. Under project conditions, with both land use and topography changes, the peak flood stage was found to be 132.6 ft NAVD.

Changing only the land use variable results is no change in water surface elevation. With both land use and grading changes incorporated, the change is within a tenth of a foot.

Appendix E-1: Original Debris Basins Report for EIR Prepared for David J. Powers

December 3, 1998

Jill Zachary
David J. Powers and Associates
1885 The Alameda
Suite 204
San Jose, CA 95126

Dear Jill:

Schaaf & Wheeler is pleased to submit this report on the flood control and drainage impacts for the proposed campus industrial development on the Pacific Rim Financial Corporation site located between Evergreen and Fowler Creeks in the Evergreen area of San Jose. Three issues relating to on-site drainage, completion of the Fowler Creek pipeline, and sizing of a debris basin on the South Branch of Fowler Creek were addressed and are presented in the report.

The results of this Initial Project level study can be used be used as a basis for future EIR path decisions. This report should also be forwarded to the Santa Clara Valley Water District for review before EIR path decisions are made.

We welcome your comment on our findings and are prepared to address any concerns. Please do not hesitate to contact Jim Gessford or myself if we can be of further service.

Very truly yours,
SCHAAF & WHEELER

James R. Schaaf, PhD, PE
President

Enclosure

Introduction

The purpose of this report is to define the flood control and drainage impacts and mitigation measures for the proposed industrial/office building project located between Evergreen and Fowler Creeks in the Evergreen area of the City of San Jose. There are three issues that were investigated as potential impacts of the project that would need to be addressed at the Initial Project level before EIR path decisions are made.

The first issue that was addressed was the proposed storm drain system on site and the potential impacts of the runoff from that system on the downstream storm drainage systems that are already in place. In addition, the potential impact on the level of flood protection along Evergreen Creek, as well as the downstream effects on Thompson Creek and Silver Creek, were investigated.

The second issue that was addressed was the proposed pipeline for the South Branch of Fowler Creek and the pipeline for Fowler Creek from the confluence with the South Branch all the way downstream to the existing upstream terminus of the Fowler Creek pipeline. Included in this analysis was a determination of whether any of the proposed project will drain to Fowler Creek. Finally, the 100-year flood peak discharge was calculated to determine the appropriate pipe size and slope for this portion of the overall project on Fowler Creek. The Santa Clara Valley Water District's regional regression equations were used to estimate the 100-year flood and these values have been sent to the District for concurrence.

The final issue that was addressed was the proposed siltation/debris basin on the south branch of Fowler Creek. This basin was sized using a debris load estimate, in yd^3/mi^2, that was similar to the estimates made for debris basins in neighboring watersheds. The basin size and preliminary design have been sent to the District for approval.

Drainage

The proposed on-site storm drain system for the project will connect to the 48 inch RCP storm drain at the intersection of Murillo Avenue and Altia Avenue. Based on Rational Method calculations, the 48 inch RCP is appropriately sized to drain the entire 110 acre campus industrial site assuming a runoff coefficient of 0.85.

The 24 inch and 36 inch RCPs that run down Murillo Avenue were sized assuming a runoff coefficient of 0.65, a value that is too low considering the proposed campus industrial development. Again, a value of 0.85 to 0.9 is more likely for this type of development. At the time these pipes were sized, the proposed development site was orchards and open fields and an assumption was made as to its future use. Two options exist to overcome these inadequate pipe capacities. The first option is to replace the 24 inch and 36 inch RCPs along Murillo Avenue and allow the on-site storm drain system to connect to the existing system at the four manholes on Murrillo Avenue. In addition, the three 15 inch RCPs that stub to the site would also have to be replaced due to insufficient capacity. The 42 inch RCP stub that extends into the site at the intersection of Murrillo Avenue and Altia Avenue has sufficient capacity to remain in place. The second option is to design an on-site drainage system that discharges directly to the 48 inch R.C.P. at the intersection of Murillo Avenue and Altia Avenue. Since the 48 inch R.C.P. has sufficient capacity to carry the total runoff from the site, this option appears to be the most reasonable approach.

The runoff from the proposed campus industrial site will discharge into a storm drain system that eventually discharges to Evergreen Creek. As part of the Evergreen Specific Plan, any development that increases the 100-year flood peak discharge in Evergreen Creek needs to provide on-site detention to mitigate these effects. This criteria also applies to downstream increases in Thompson Creek and Silver Creek. An analysis was made using the U.S. Army Corps of Engineers HEC-1 model that was developed by the Santa Clara Valley Water District in 1976 and revised in 1986 as part of the Evergreen Specific Plan.

The 1986 revision to the District's HEC-1 model incorporated the District's new 10-year storm drain capacity requirements and extended the Urban Service Limit of Fowler, Quimby and Evergreen Creeks, adding area to the Urban Service portion of the model and removing an equal portion from Urban Reserve. Although the proposed Campus Industrial site is outside of the Evergreen Specific Plan area limit, it was included in the model as additional Urban Service area. The Urban Service portions of the watershed were assumed to be 60 percent impervious and 40 percent pervious in the HEC-1 model. These percentages correspond to medium residential with a density of 5 to 8 dwelling units per acre. Obviously, the impervious area for a Campus Industrial site is greater than that of medium residential. According to the General Plan for the Evergreen site, the Campus Industrial Land Use Designation is 30 percent building area, 50 percent parking, and 20 percent landscaping. This designation corresponds to an 80 percent impervious portion of the watershed. For the 110 acre site, the difference in impervious area between using 80 percent and 60 percent impervious is 0.03 mi^2. This 0.03 mi^2 was added to the impervious portion of the Urban Service portion of the Evergreen Creek watershed and subtracted from the pervious portion. No other changes were made to the 1986 HEC-1 model.

The result of the HEC-1 analysis is that the Campus Industrial site increases the 100-year discharge in Evergreen Creek by 1 cfs, from 527 cfs to 528 cfs. This 1 cfs increase is carried downstream to Thompson Creek. By the time Thompson Creek reaches Quimby Creek, there is no increase in the 100-year discharge. The 1 cfs increase in Evergreen Creek represents a 0.2 percent increase in discharge on Evergreen Creek, a 0.04 percent increase in Thompson Creek upstream of Fowler Creek and a 0.03 percent increase in Thompson Creek downstream of Fowler Creek. By the time the 100-year discharge reaches Quimby Creek, there is no increase in discharge. These 100-year discharge increases of 0.03 to 0.2 percent are insignificant and should not require on-site detention as a mitigation effort. Detailed HEC-1 input and output are shown in Appendix A.

Fowler Creek Pipeline

As part of the Evergreen Specific Plan, the last upstream development on Fowler Creek is responsible for completing the installation of the Fowler Creek pipeline. In the case of the Pacific Rim Financial Corporation's Campus Industrial site, this includes extending the Fowler Creek pipeline from its current upstream terminus to its confluence with the South Branch of Fowler Creek. In addition, the pipeline must be installed for the South Branch of Fowler Creek from its confluence with Fowler Creek to its upstream terminus. A debris basin will be constructed at the upstream terminus of the South Branch of Fowler Creek pipeline to prevent silt and debris from being introduced in to the pipeline. This debris basin will be located at the northeast corner of the proposed development where the creek emerges from the steep foothills.

As previously discussed, the proposed Campus Industrial site will drain entirely to Evergreen Creek. Since none of the site drains to the South Branch of Fowler Creek, it is assumed that the proposed pipeline for the South Branch of Fowler Creek will need to have a capacity equal to the 100-year design flood. This design value was obtained from the Santa Clara Valley Water District Regional Regression Equations, assuming a drainage basin area of 460 acres and a mean annual precipitation of 16 inches, with the Los Animas correction factor applied to the calculated 100-year discharge value. The required slope and pipe size for the South Fork of Fowler Creek was calculated using a Q_{100} of 280 cfs. Additional consideration was given to keeping the velocities in the pipe to approximately 15 feet per second. Utilizing a 60 inch RCP at a 1.5 percent slope would achieve velocities in the desired 14 to 15 feet per second range. Drop manholes would need to be installed whenever the top of the pipe comes within 5 feet of the natural ground to ensure minimum cover.

The portion of the Fowler Creek pipeline between its current upstream terminus and its confluence with the South Branch was evaluated using the 100-year discharge value obtained from the 1986 Evergreen Specific Plan HEC-1 model. The resulting 100-year discharge is approximately 670 cfs. This value is approximately the same value used by the District in their design calculations. At the existing upstream terminus of Fowler Creek there is a 66 inch RCP, which limits the size of the pipe that can be continued upstream to 66 inches. Holding the velocities to the desired 15 feet per second is not possible in a 66 inch RCP pipeline with the given flow rate and required slope placement. Based on an average existing ground slope of .047, a 66 inch RCP placed parallel to the ground slope will carry the design flows with velocities of 26 to 27 feet per second. It should be noted that the portion of the pipe previously installed based on the District's plans also has velocities in this range.

Upon completion of the South Branch pipeline and the Fowler Creek pipeline from its current upstream terminus to the confluence with the South Branch, the flow that once spread out over a broad area will be concentrated in Fowler Creek. This increased flow in Fowler Creek is not a concern because the pipeline is designed to carry the 100-year discharge and the open channel portion of Fowler Creek, as well as Thompson Creek and Silver Creek have a 100-year capacity all the way down to Lake Cunningham.

Debris Basin on South Branch of Fowler Creek

A debris basin will be constructed at the northwest corner of the proposed Campus Industrial site to intercept silt and debris as the South Branch of Fowler Creek emerges from the steep foothills. The debris basin will drain to the proposed South Branch Fowler Creek pipeline. Interception of the silt and debris will prevent erosion and siltation problems that could damage the integrity of the pipeline or reduce its discharge capacity.

To size the debris basin and perform a preliminary design, an estimate was needed as to the average yearly debris that could be expected to be trapped by the basin. Based on a conversation with Sue Tippets of the Santa Clara Valley Water District, an estimate of 0.4 acre-ft/mi^2/yr was used. This value is representative of actual annual debris loads in debris basins located in neighboring watersheds that the District is responsible for maintaining. It was made clear by Ms. Tippets that since the District would be responsible for maintaining the basin, they required that the basin not need to be cleaned out more frequently than every ten years, preferably every 25 years. The preliminary sizing and design of the debris basin is based on an annual debris load of 0.4 acre-ft/mi^2 for 25 years. The portion of the South Branch of Fowler Creek that will contribute to the debris basin has a drainage area of approximately 400 acres. Applying the estimated annual debris load to this area for a 25 year period results in an estimated debris volume of 6.25 acre-feet. Alternative basin sizings were performed for basins that would need to be cleaned out every ten years and every 50 years. These basins would require an estimated volume of 2.5 acre-feet and 12.5 acre-feet, respectively.

Two options were considered for the design of the debris basin. A plan view showing the proposed location of the two options is included in Figure 2. The first option involved building a dam perpendicular to the flow path of the South Branch of Fowler Creek at the bottom of the steep foothills. The dam would be a straight vertical wall that would connect to the existing banks of the steep channel as it emerges from the foothills. For this option, the portion of the South Branch below the dam would need to intercepted and mitigated to prevent increasing discharge to Evergreen Creek.

The second option is to construct a curved berm further downstream on the existing alluvial floodplain. This berm would curve around and connect with the existing topography and would serve to dam the flow. This option would require a much longer constructed fill, however, its decreased height would prevent it from being classified as a dam by the Bureau of Dam Safety. In addition, its more downstream location would serve to decrease the length of the South Branch pipeline. The average end-area method was used to perform a preliminary design of the basins utilizing available topographic information in the area. Based on this analysis, the first option would require a 26 foot high dam with a length of 205 feet to provide 6.25 acre-feet of debris storage. If it were desired to only clean the basin out every 50 years, the dam would increase to 34.5 feet in height and 285 feet in length to provide 12.5 acre-feet of debris storage. These values do not include freeboard. The California Bureau of Safety of Dams criteria states that any berm higher than 24 feet shall be classified and operated as a dam. For this project, constructing and maintaining a dam is not an available option. The design calculations for the dam are included in Table 1.

The second option would require a curved berm, 14 feet in height and 485 feet in length to provide 6.25 acre-feet of debris storage. For a 50-year cleaning interval, those values increase to 19 feet high by 535 feet long in order to provide 12.5 acre-feet of storage. Again, these values do not include freeboard. The design calculations for the curved berm are included in Table 2.

A cross section of the proposed curved berm debris basin is shown in Figure 1. The berm will be placed at an elevation of 568 feet. Its upstream face will intersect the existing ground at a 3:1 slope and its downstream face would intersect existing ground at a 2.5:1 slope. The top of the berm will be approximately 12 feet wide to provide vehicle access. The berm will have a 50-foot wide trapezoidal emergency spillway designed to pass the District's 80% Confidence Flood of 400 cfs. The bottom of the spillway will have an elevation of 583 feet, allowing for the required 14 feet of debris storage, with an additional foot of storage to make up for lost storage as a result of the 3:1 upstream face of the berm. The spillway was sized to pass the 80% Confidence Flood at a discharge head of 2 feet. An additional 2 feet of freeboard was provided above the discharge water surface elevation, for a final crest elevation of 587 ft. The figures and tables provided are initial estimates of the length, height and location of the proposed berm. Alternative alignments are available and should be considered to maximize storage and minimize fill prior to construction of the debris basin.

Conclusion

This report should serve as an Initial Project Level study on which to base EIR decisions for the proposed Evergreen Campus Industrial Park. An effort was made to define all of the pertinent issues relating to the development and to clarify what has been constructed to date with regard to downstream drainage. As far as increased discharge to Evergreen Creek from the proposed project, no impact is expected. The runoff from the site can be accommodated by the existing storm drain system to Evergreen Creek if a parallel storm drain system is constructed on-site. The Fowler Creek pipeline from its current upstream terminus to its confluence with the South Branch of Fowler Creek, as well as the South Branch pipeline was sized to pass the 100-year discharge. Finally, the debris basin has been sized and a preliminary design and location has been provided for use in developing an EIR for the project site.

Appendix E-2: Addendum to EIR, Including Revisions to Debris Loading

July 8, 1999

Mike Sheehy
Ruth & Going
2216 The Alameda
Santa Clara, CA 95050

Dear Mike:

Schaaf & Wheeler is pleased to submit this addendum to our December 3, 1998 flood control and drainage report prepared for the Pacific Rim Financial Corporation site located in the Evergreen area of San Jose. The changes to our original report contained herein were intended to address the concerns of the Santa Clara Valley Water District and are based on our last meeting with Sue Tippetts on June 23, 1999.

The revisions included in this report relate to on-site drainage and the need for on-site detention to prevent increases in downstream discharges and to the latest revisions to the criteria used to size the debris basin for the South Branch of Fowler Creek. Our analysis indicates that no on-site detention is necessary, however, the size of the debris basin has been increased based on the new criteria.

Please see to it that this addendum gets included with our original report as the revisions included here supersede all previous efforts to address the flood control and drainage issues related to developing this site as Campus Industrial. A copy of this addendum will also be forwarded to David Powers and Associates and to Sue Tippetts at the Water District. We will await your comments before forwarding this information to either of the parties mentioned above.

We welcome your comments and/or concerns related to our latest revisions to this study. Please do not hesitate to contact me or Jim Gessford if we can be of further service.

Very truly yours,
SCHAAF & WHEELER

James R. Schaaf, Ph.D., P.E.

cc: Ms. Sue Tippetts, SCVWD; Ms. Michelle Yesney, David J. Powers & Associates.

Introduction

The purpose of this addendum to the December 3, 1998 report on flood control and drainage impacts for the proposed Pacific Rim Financial Corporation (PRFC) Evergreen Campus Industrial site is to revise earlier efforts performed by Schaaf & Wheeler to address the comments and concerns expressed by the Santa Clara Valley Water District (SCVWD). These revisions are an effort to address concerns about increases in downstream discharges due to development of the site and to the initial criteria used to size the debris basin to be located on the South Branch of Fowler Creek.

The first issue that was addressed was the impact of development on downstream discharges in Evergreen Creek, as well as the further downstream effects on Thompson Creek and Silver Creek. In addition, the impact of similar development in the Evergreen Creek, Fowler Creek and Quimby Creek on downstream discharges was also analyzed in order to address the bigger picture of complete development of the remaining portions of land within the urban service limit identified in the Evergreen Specific Plan.

The second issue that was addressed was the criteria used to size the debris basin to be located on the South Branch of Fowler Creek. Revisions were made to our earlier estimates based on additional information made available to the Water District and due to our analysis of the Flint Creek and Norwood Creek debris basins. These revisions are intended to address both the concerns of the Water District related to maintenance and the concerns of the property owner related to size and cost of the basin.

Drainage

Evergreen Specific Plan

The runoff from the proposed campus industrial site will discharge into a storm drain system that eventually discharges to Evergreen Creek. As part of the Evergreen Specific Plan, any development that increases the 100-year flood peak discharge in Evergreen Creek needs to provide on-site detention as mitigation. This criteria also applies to downstream increases in Thompson Creek and Silver Creek. An analysis was made using the U.S. Army Corps of Engineers' HEC-1 model that was developed by the Santa Clara Valley Water District in 1976 and revised by Schaaf & Wheeler in 1986 as part of the Evergreen Specific Plan.

The Water District's 1976 model broke each of the watersheds into two sub-basins, urban reserve and urban service. The urban reserve portion included those areas not subject to development, primarily portions of the watershed in the steep foothills. The remaining urban service area consisted of those areas within the urban service limits as designated by the 1976 Urban Service Boundary (USB) shown in Figure 1. Because the District's 1976 limit as shown in Figure 1 was used on the District's watershed maps and used in the hydrologic model, it was assumed that the 1976 hydrologic model represents existing design conditions. The urban service area is sub-divided into pervious and impervious portions. The impervious portion includes those parts of the watershed that are covered by buildings, sidewalks, and parking lots. The pervious portion includes those areas dedicated to landscape cover and open space set-aside. In the 1976 Water District Model the urban service area is assumed to be 60 percent impervious.

As part of the Evergreen Specific Plan, one change was made to the District's model before it was used to represent existing design conditions. In the 1976 model the District assumed that flows from the urban areas which were in excess of the storm drain capacity would pond and enter the storm drain at basically the capacity of that system. Field reconnaissance of the area, however, revealed that it was highly unlikely that flows in excess of the storm drain capacity could pond on the steep streets in the area. It appeared more likely that flows in excess of the storm drain capacity would flow down the street pattern parallel to the flood control facilities and enter Thompson Creek directly or in the case of Fowler Creek enter the open channel portion of that flood control facility. All other hydrologic model features for the entire Silver Creek and Thompson Creek watersheds were unchanged from the District's original 1976 model.

Schaaf & Wheeler revised the District's model to include development in the Evergreen Area as part of the Evergreen Specific Plan performed for the City of San Jose. This 1986 revision to the District's HEC-1 hydrologic model incorporated the City of San Jose's new 10-year storm drain capacity requirements and extended the urban service limits of Fowler, Quimby, and Evergreen Creeks. This extension of the urban service limits added area to the Urban Service portions of the three watersheds and removed an equal amount of area from the Urban Reserve portions. Two additional properties were located outside the Evergreen Specific Plan boundaries but inside the 1986 urban service limits. To make the revisions to the District's model as comprehensive as possible, these properties, planned for campus industrial uses, were therefore included in the analysis. The Pacific Rim Financial Corporation Evergreen Campus Industrial site was included in the hydrologic model that was revised to reflect the 1986 urban service limits.

As previously mentioned, the District's 1976 hydrologic model assumed that the entire urban service area was 60 percent impervious. The revisions performed by Schaaf & Wheeler based on the 1986 Urban Service Boundary also included this assumption. Table 1 shows the portions of the Evergreen, Fowler, and Quimby Creek watersheds modeled as urban reserve and urban service (pervious and impervious) for both the existing 1976 District model and the revised (1986 USB) model.

TABLE 1

<table>
<tr><th rowspan="2">Basin</th><th colspan="4">1976 SCVWD Model</th><th colspan="4">Revised Model (1986 USB)</th></tr>
<tr><th>Urban Reserve</th><th>Urban Service*</th><th>Perv.</th><th>Imp.</th><th>Urban Reserve</th><th>Urban Service*</th><th>Perv.</th><th>Imp.</th></tr>
<tr><td>**Evergreen**</td><td>**1.61**</td><td>**0.37**</td><td>**0.15**</td><td>**0.22**</td><td>**1.15**</td><td>**0.83**</td><td>**0.33**</td><td>**0.5**</td></tr>
<tr><td>**Fowler**</td><td>2.26</td><td>0.52</td><td>0.21</td><td>0.31</td><td>2.04</td><td>0.74</td><td>0.296</td><td>0.444</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

Quimby	1.31	0.86	0.34	0.52	1.04	1.13	0.452	0.678

* 60% Impervious Urban Service Area

Table 2 shows the 100-year discharges in Evergreen, Fowler, and Quimby Creeks upstream of their confluences with Thompson Creek, as well as the 100-year discharge in Thompson Creek downstream of its confluence with each of the creeks for both the 1976 District model and the revised (1986 USB) model. In addition, the 100-year discharge in Silver Creek downstream of Thompson Creek is shown.

TABLE 2

Location	'76 SCVWD Model 100-Year Design Discharge (cfs)*	Revised Model ('86 USB) 100-Year Design Discharge (cfs) *
Evergreen Cr. U/S of Thompson	**539**	**527**
Thompson Cr. D/S of Evergreen	**2539**	**2530**
Fowler Cr. U/S of Thompson	656	668
Thompson Cr. D/S of Fowler	3209	3213
Quimby Cr. U/S of Thompson	502	517
Thompson Cr. D/S of Quimby	3746	3768
Silver Cr. D/S of Thompson	4341	4365

* 60% Impervious Urban Service Area

The results of the hydrologic modeling shown in Table 2 indicate that extending the urban service limits actually decreases the 100-year discharge in Evergreen Creek, however, development in the Fowler Creek and Quimby Creek watersheds lead to an increase in the 100-year discharge in their respective creeks and downstream in Thompson and Silver Creeks. The decrease in 100-year discharge in Evergreen Creek can be attributed to a difference in timing of the urban service discharges and urban reserve discharges. The Evergreen Specific Plan proposed constructing water features within the Fowler Creek and Quimby Creek watersheds to accommodate flows in excess of the storm drain capacity and grading the streets to allow the excess flow to travel down the street pattern and into the water features. Thus, they would provide detention storage only when the storm drainage system was overtaxed and water was flowing down the street. No water features were proposed for Evergreen Creek.

PRFC Campus Industrial Development

It is important to note that the original 1976 Water District, as well as the model revisions performed by Schaaf & Wheeler to reflect the 1986 urban service limits assumed that development within the Urban Service Area would be 60 percent impervious. Current

City of San Jose standards call for a maximum of 30 percent building cover and a minimum of 25 percent landscape cover. In the absence of a defined development plan for the PRFC Campus Industrial Site, Schaaf & Wheeler assumed that the development would adhere to the City's standards, resulting in a site plan that would be approximately 75 percent impervious. In addition, Schaaf & Wheeler's knowledge of similar development adjacent to the site led them to believe that perhaps the entire urban service area should be modeled as 75 percent impervious. The increase in the 100-year discharge over the 1976 District model discharges in each of the watersheds could then be attributed to each individual property owner based on their percentage of property within the urban service area of each of the watersheds. Alternatively, only the additional area added as urban service between the 1976 model and the extended urban service model could be modeled as 75 percent impervious and the increase in discharge attributed to each land owner based on their percentage of land within the newly annexed portion. The difference being, for one scenario, the land owner would be responsible for a small percentage of a bigger increase in discharge, and for the other, a big percentage of a smaller increase in discharge. Schaaf & Wheeler's analysis chose to model the entire urban service area of each of the watersheds as 75 percent impervious.

Table 3 shows the portions of the Evergreen, Fowler, and Quimby Creek watersheds modeled as urban reserve and urban service (pervious and impervious) for both the existing 1976 District model and the 75 percent impervious urban service area model revised by Schaaf & Wheeler.

TABLE 3

Basin	1976 SCVWD Model				1999 S&W Revised Model			
	Urban Reserve	Urban Service*	Perv.	Imp.	Urban Reserve	Urban Service**	Perv.	Imp.
Evergreen	1.61	0.37	0.15	0.22	1.15	0.83	0.207	0.623
Fowler	2.26	0.52	0.21	0.31	2.04	0.74	0.185	0.555
Quimby	1.31	0.86	0.34	0.52	1.04	1.13	0.282	0.848

* 60% Impervious Urban Service Area ** 75% Impervious Urban Service Area

Table 4 shows the 100-year discharges in Evergreen, Fowler, and Quimby Creeks upstream of their confluences with Thompson Creek, as well as the 100-year discharge in Thompson Creek downstream of its confluence with each of the creeks for both the 1976 District model and the 1999 Schaaf & Wheeler revisions to reflect 75% imperviousness of the urban service areas of each of the watersheds. In addition, the 100-year discharge in Silver Creek downstream of Thompson Creek is shown.

TABLE 4

Location	'76 SCVWD Model 100-Year Design Discharge (cfs)*	1999 S&W Revised Model 100-Year Design Discharge (cfs) **
Evergreen Cr. U/S of Thompson	539	529
Thompson Cr. D/S of Evergreen	2539	2533
Fowler Cr. U/S of Thompson	656	670
Thompson Cr. D/S of Fowler	3209	3219
Quimby Cr. U/S of Thompson	502	519
Thompson Cr. D/S of Quimby	3746	3775
Silver Cr. D/S of Thompson	4341	4372

* 60% Impervious Urban Service Area ** 75% Impervious Urban Service Area

Again, it can be seen that based on the Water District's HEC-1 hydrologic model, extending the urban service limits of the Evergreen Creek watershed leads to a decrease in the 100-year discharge in Evergreen Creek, even with the increase in imperviousness of the urban service area. This can be attributed to a change in the timing of the urban service and urban reserve discharges. These results indicate that no on-site detention needs to be provided as mitigation for development in the Evergreen watershed. However, development in the upper portions of the Fowler and Quimby Creek watersheds does lead to an increase in discharge to their respective creeks and to downstream discharges in Thompson and Silver Creeks. Besides the water features proposed in the Evergreen Specific Plan to accommodate storm water discharges in excess of the storm drain capacity, on-site storage could be provided in parking lots by under sizing the inlets or storm drains in the parking lots.

As previously mentioned, the requirement for on-site detention as mitigation for the increase in 100-year discharge in the Fowler Creek and Quimby Creek watersheds could be apportioned based on the individual land owner's percentage of land within the urban service portion of the watershed. This way, each land owner could determine how to accommodate on-site detention on their property and would only be on the hook for a small increase. This is a more conservative approach than trying to adjust the Water District's model to reflect only the increase resulting from each individual development, whereby each increase gets written off as insignificant, when as a whole there is a significant increase.

Finally, it bears repeating that the Schaaf & Wheeler analysis is merely a modification to the Water District's 1976 model to reflect increased urban service area in the Evergreen, Fowler, and Quimby Creek watersheds. The latest revisions also reflect the changes attributed to the greater imperviousness associated with the proposed developments. The results of our analysis conclude that although on-site detention needs to be provided to mitigate for development in the Fowler and Quimby Creek watersheds, no on-site detention is necessary to mitigate for development in the Evergreen Creek watershed.

The HEC-1 output for the 1976 Water District model, the extended urban service limit model performed by Schaaf & Wheeler for the Evergreen Specific Plan, and the 75% impervious urban service area model performed by Schaaf & Wheeler as part of this study are included in Appendix A.

S. Branch Fowler Creek Debris Basin

Based on several discussions with the Water District and Schaaf & Wheeler's analysis of the Norwood and Flint Creek debris basins, new criteria have been developed to size the debris basin to be placed on the South Branch of Fowler Creek. These new criteria are the result of on-going discussions between Ruth& Going, Schaaf & Wheeler, and the Water District and are intended to satisfy all parties involved. The new debris loading factor is 0.6 acre-ft/sq. mi./yr. The desired design life is 20 years, after which time the Water District can expect to need to clean out the basin. The original debris loading factor was 0.4 acre-ft/sq. mi./yr., with a design life of no less than 10 years, preferably 25. It was shown by Schaaf & Wheeler that several years of debris loading can be expected to occur in only a few years of heavy rainfall since the rate of debris loading is a function of the stream discharge cubed. During years of heavy rainfall the debris basin may experience a greater percentage of its total loading than during years of light rainfall. Thus, 20 years should be an acceptable design life.

The new debris loading factor estimate and revised design life lead to a 7.5 acre-ft of storage requirement for the debris basin to be located on the South Branch of Fowler Creek. Based on Schaaf & Wheeler's earlier efforts, this storage requirement could be met by a curved berm approximately 15 ft. in height and 500 ft. long, not including freeboard or spillway headwater. A cross section of the proposed curved berm debris basin was provided in the December 3, 1998 Schaaf & Wheeler report for which this addendum is to be included. A revision to that proposed cross section can be performed once Ruth & Going and Schaaf & Wheeler proceed toward preliminary design of the basin.

Conclusion

This addendum to the December 3, 1998 report on the flood control and drainage aspects of the Pacific Rim Financial Corporation Evergreen Campus Industrial Site was intended to address the comments and concerns of the Santa Clara Valley Water District related to increases in 100-year discharges in Evergreen, Thompson, and Silver Creek, as well as to the debris loading estimates used to size the S. Branch of Fowler Creek debris basin. As part of our response, Schaaf & Wheeler has included analysis previously performed as part of the Evergreen Specific Plan to address the Water District's concerns related to increases in downstream discharges. Our analysis indicates that no on-site detention needs to be provided for development in the Evergreen Creek watershed, for which the PRFC Campus Industrial Site is a part of. Finally, the Water District's final demands for a debris loading factor and design life of a debris basin led to a modification and slight increase in the required debris storage for the S. Branch of Fowler Creek debris basin.

Appendix E-3: Memo to Ruth & Going regarding our plans to survey Norwood Creek

Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS
100 N. Winchester, Suite 200
Santa Clara, CA 95050
P: 408-246-4848
F: 408-246-5624

MEMO

TO: Steve Sherman
Ruth & Going

Date: May 3, 1999

FROM: Jim Gessford

Job No.: PRFC.01.99

SUBJECT: Pacific Rim Financial Campus Industrial

Steve:

Per the meeting with Ruth & Going, Schaaf & Wheeler, and the Santa Clara Valley Water District, a few issues need to be revisited as part of out hydrology and hydraulics study for the Pacific Rim Campus Industrial Site. The issues include determining a better estimate on the rate of debris loading used as the basis of design for the S. Branch Fowler Creek debris basin. In addition, we need to convince the Water District that the work Schaaf & Wheeler previously performed for the City of San Jose as part of the Evergreen Specific Plan adequately reflects the effects of further development in the Evergreen area.

I spoke with Sue Tippets today about obtaining the design plans for the Norwood Creek and Ruby Creek debris basins and was told I can come make copies of them. We may then be able to survey the existing debris line and estimate a debris loading rate from a comparison of the plans and the existing debris load. This work will be performed in the coming two weeks, and may require the assistance of a Ruth and Going employee in the field, if one is available. We will also revisit our hydrologic analysis and compare the proposed development condition to the original 1976 Water District HEC-1 model. The results of this analysis will determine whether on-site detention is necessary. If detention is required, we can probably accommodate it by providing storage in the parking lots.

It is essential that we finish this work and receive comment before proceeding with the Annie Chan/ Evershine Group site. We believe that by proceeding in this fashion, we can prevent having to duplicate corrections on both projects as a result of changing District criteria and response. Again, we anticipate making substantial progress on these issues in the next two weeks. Please contact me to discuss any concerns.

cc: Mike Sheehy (Ruth & Going)

Appendix E-4: Letter to Sue Tippets showing the results of Norwood Creek survey and suggesting new debris loading rate

May 18, 1999

Sue Tippets
Community Projects Review Unit
Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118

RE: Pacific Rim Financial Corp. Evergreen Campus Industrial Site

Dear Ms. Tippets:

As a follow-up to our April, 1999 meeting we obtained as-built plans for the Norwood Creek debris basin from your agency. On May 13, 1999 we performed a field survey of the basin in order to estimate the amount of debris that has accumulated in the 23 water years it has been in service. The results of our survey and subsequent analysis are shown below:

Average Depth of Debris	=	5.5 ft.
Surface Area of Debris Basin	=	1.9 acres
Drainage Area	=	702 acres (1.1 mi^2)
Years of Service	=	23 years

Total Debris Loading	=	(5.5 ft.) * (1.9 Acres) =	**10.45 acre-ft**
Total Debris Yield	=	(10.45 ac-ft)/(1.1 mi^2) =	**9.5 ac-ft/mi^2**
Annual Debris Yield	=	(9.5 ac-ft/mi^2)/(23 Years) =	**0.4 ac-ft/mi^2/yr**

The results of our survey and subsequent analysis are consistent with our initial estimate based on conversations with you at the onset of this project. The Norwood Creek data verified the debris loading factor and the District's concerns should now be minimized. The design of the debris basin on the South Branch of Fowler Creek should now be able to proceed. Please feel free to contact me to discuss this matter.

Very truly yours,
SCHAAF & WHEELER

James R. Schaaf, PH.D., P.E.

Appendix E-5: Letter to Ruth & Going Confirming our "Compromise" with the District and our Concerns About Them Trying to Make the Basin Too Small.

7/2/2001

Mike Sheehy
Ruth & Going
2216 The Alameda
Santa Clara, CA 95050

RE: Fowler Creek Debris Basins

Dear Mike:

Per our meeting with Steve Sherman at our office on March 28, 2001, we are providing this letter to document our response to the proposed changes of the size of the debris basins to be located on the north and south branches of Fowler Creek. Although we have not yet seen the revised plans for the basins, Mr. Sherman has indicated that alternatives are being sized for one and three-year design lives.

In a meeting with Ms. Sue Tippetts of the Santa Clara Valley Water District on June 23, 1999, a debris-loading factor of 0.6 acre-feet per square mile per year and a design life of 20 years were agreed upon by Schaaf & Wheeler and the District. Ruth and Going was also present at that meeting representing the interests of Chester Wang and Annie Chan. In this meeting Schaaf & Wheeler demonstrated to the District that their proposed debris-loading factor of 1.0 was too high and proposed a factor of 0.4.

The District proposed the 1.0 factor based on already having to clean out a couple of nearby basins on Flint Creek and Quimby Creek that had only been in service for a few years. In the meeting, Schaaf & Wheeler demonstrated that since the rate of debris production is proportional to the flow rate cubed, having a few years of above average rainfall could result in higher than expected debris accumulation during the years that the basins were in service. Over the long run, the annual rate of debris accumulation could be much smaller that that observed over those years of heavy rainfall.

In this same meeting, Schaaf & Wheeler proposed a debris-loading factor of 0.4 based on our recent survey of nearby Norwood Creek. The results of the survey were compared to the design plans for the basin to estimate that amount of debris accumulation that had occurred in the 23 water years that the basin had been in service. Based on an average debris depth of 5.5 ft across the 1.9-acre basin and a contributing drainage area of approximately 1.1 square miles, the following results were presented to the District:

Total Debris Loading	= (5.5 ft.) * (1.9 acres)	=	10.45 acre-ft
Total Debris Yield	= (10.45 ac-ft)/(1.1 mi^2)	=	9.5 ac-ft/mi^2
Annual Debris Yield	= (9.5 ac-ft/mi^2)/(23 Years)	=	0.4 ac-ft/mi^2/yr

In addition, Schaaf & Wheeler recommended that the basins be cleaned out as frequently as possible to avoid the emergence of wetlands vegetation that might make it impossible to clean them out. Schaaf & Wheeler's recommendation was to clean the basins out at least every ten years or whenever the basins became half full by elevation. At the time of the meeting the District was requesting a 25-year design life. As previously noted, all parties in attendance at the meeting agreed in principle to a debris loading factor of 0.6 acre-ft per square mile per year and a design life of 20 years. The provision to clean out the basins whenever they became half full by elevation was also agreed upon in the meeting provided a staff gage would be installed in the basins.

The recently proposed changes to the sizes of the debris basins call for making them only large enough to contain either one or three years of debris accumulation based on a debris loading factor of 0.6 acre-ft per square mile per year. Although the proposed changes include a provision for the owner of the property to clean out the basins on an annual basis, Schaaf & Wheeler's previous analysis of Upper Penitencia Creek has shown that five times the average amount of debris accumulation can occur in one year of heavy rainfall. Other stream gages in the region show slightly upwards of ten times the average in very wet years. Therefore, based on our previous analysis, Schaaf & Wheeler does not recommend decreasing the size of the debris basins.

Please do not hesitate to contact either Jim Gessford or myself if you have any questions or concerns related to our efforts.

Very truly yours,
SCHAAF & WHEELER

James R. Schaaf, Ph.D., P.E.
Principal

R&GI.17.99 002

Appendix E-6: Q-cubed summary for Upper Penitencia Creek

Water Year	Annual Sum Daily Q's	Annual Sum Daily Q^3's	Above/Below Average Annual Q	Above/Below Average Annual Q^3	Percentage Q	Percentage Q^3	Annual Q^3 / Average Q^3	
1973	5207.61	67421929	Above	Above	6.656362677	4.71546797	**1.2**	**1.2**
1974	2826.98	27454614	Below	Above	3.613443434	1.92016685	0.5	
1975	2624.15	15668346	Below	Above	3.3541863	1.09583903	0.3	
1976	140.4	76.62757	Below	Below	0.179459161	5.3593E-06	0.0	
1977	84.62	21.58231	Below	Below	0.108161212	1.5095E-06	0.0	
1978	2711.76	26274663	Below	Below	3.466169328	1.83764148	0.5	
1979	819.48	655403.8	Below	Below	1.04745864	0.04583873	0.0	
1980	2872.92	2.02E+08	Below	Above	3.672163903	14.1567401	**3.5**	**3.5**
1981	925.69	3601664	Below	Below	1.183216171	0.25189921	0.1	
1982	5524.52	2.07E+08	Above	Above	7.06143677	14.4976712	**3.6**	
1983	10728.92	2.87E+08	Above	Above	13.71369643	20.0461628	**5.0**	**8.6**
1984	2829.32	7746725	Below	Below	3.61643442	0.54180343	0.1	
1985	930.91	418425.1	Below	Below	1.189888371	0.02926452	0.0	
1986	3507.33	1.42E+08	Above	Above	4.483066226	9.95452344	**2.5**	**2.5**
1987	326.6	60081.92	Below	Below	0.417459842	0.00420211	0.0	
1989	1320.47	40153.95	Below	Below	1.687823632	0.00280835	0.0	
1990	1244.1	35100.71	Below	Below	1.590207563	0.00245493	0.0	
1991	1562.53	4585066	Below	Below	1.997224518	0.32067807	0.1	
1992	1971.19	12379182	Below	Below	2.519573383	0.86579597	0.2	
1993	4098.28	23488133	Above	Below	5.238418014	1.64275245	0.4	
1994	1077.78	111934.3	Below	Below	1.37761748	0.00782865	0.0	
1995	6437.83	1.03E+08	Above	Above	8.228828836	7.196965	**1.8**	
1996	7702.8	69756926	Above	Above	9.845712415	4.8787769	**1.2**	
1997	6075.8	1.39E+08	Above	Above	7.766082398	9.70686023	**2.4**	
1998	4683.08	89760949	Above	Above	5.985908877	6.27785175	**1.6**	**7.0**

Total Years of Record 25

Total	78235.07	1.43E+09	100	100	25.0	**23**
Average	3129.403	57192143				
Minimum	84.62	21.58231				
Maximum	10728.92	2.87E+08				

1988 Data Was Not Available, therefore it was assumed to be an average of the '87& '89 Data

KEY RESULTS:

23 Years of Debris Total Occurs in 9 Years

16 of 23 Years Have Annual Debris less than 1/2 of 25-Yr. Average

Appendix F-1: Routing of Catastrophic Tank Failure

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1******************************************                    ****************************************
*                    *                                          *                                      *
* FLOOD HYDROGRAPH PACKAGE (HEC-1) *                            * U.S. ARMY CORPS OF ENGINEERS         *
*       JUN 1998                 *                             * HYDROLOGIC ENGINEERING CENTER        *
*       VERSION 4.1              *                             *   609 SECOND STREET                  *
*                                *                             * DAVIS, CALIFORNIA 95616              *
* RUN DATE 29AUG05 TIME 17:02:11 *                             *   (916) 756-1104                     *
*                                *                             *                                      *
******************************************                    ****************************************


              X     X  XXXXXXX  XXXXX        X
              X     X  X        X     X     XX
              X     X  X        X            X
              XXXXXXX  XXXX     X     XXXXX  X
              X     X  X        X            X
              X     X  X        X     X      X
              X     X  XXXXXXX  XXXXX       XXX


THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL  LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                        HEC-1 INPUT                        PAGE 1

  LINE      ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

           *DIAGRAM
    1       ID    EVERGREEN SMART GROWTH SPECIFIC PLAN - 4 MG STEEL TANK RESERVOIR     FILE=RUP
    2       ID    CATASTROPHIC FAILURE ANALYSIS
    3       ID    PERFORMED BY SCHAAF & WHEELER -- AUGUST 2005
    4       IT    1     0     0   200
    5       IO    0     0
           *

    6       KK    0
    7       KM    STEEL TANK FAILURE OUTFLOW HYDROGRAPH
           * Based on initial estimate of 30 minute failure
    8       BA  .0006
    9       QI   594   575   555   535   515   495   475   455   435   415
   10       QI   395   675   355   335   320   300   275   255   240   220
   11       QI   200   180   160   140   120   100    80    60    40    20
   12       QI     0
           *

   13       KK    4
   14       KM    ROUTE OUTFLOW FROM X-SEC 0 TO X-SEC 4
   15       RS    1  STOR   0
   16       RC  0.08  0.08  0.08   500   0.2
   17       RX     0     6    21    28    33    40    60    75
   18       RY   630   628   622   620   618   620   625   630
           *

   19       KK    5
   20       KM    ROUTE HYDROGRAPH THROUGH NEW DEBRIS BASIN
   21       RS    1  STOR   0
   22       SV  0.00   7.5   8.0   8.1   8.6   9.1
   23       SQ  0.00   0.1   260   280   301   340
   24       SE   568   584   585 585.2 585.7 586.2
           *
   25       ZZ
1
         SCHEMATIC DIAGRAM OF STREAM NETWORK
INPUT
LINE     (V) ROUTING        (--->) DIVERSION OR PUMP FLOW

NO.      (.) CONNECTOR      (<---) RETURN OF DIVERTED OR PUMPED FLOW

  6        0
           V
           V
 13        4
           V
```

```
     V
19      5

(***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*****************************************                  ****************************************
*                          *                    *           *
* FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                         * U.S. ARMY CORPS OF ENGINEERS   *
*           JUN  1998            *                * HYDROLOGIC ENGINEERING CENTER  *
*         VERSION 4.1               *                *     609 SECOND STREET      *
*                          *                    *   DAVIS, CALIFORNIA 95616    *
* RUN DATE  29AUG05  TIME  17:02:11  *                  *      (916) 756-1104          *
*                          *                    *           *
*****************************************                  ****************************************
```

```
              EVERGREEN SMART GROWTH SPECIFIC PLAN - 4 MG STEEL TANK RESERVOIR          FILE=RUP
              CATASTROPHIC FAILURE ANALYSIS
              PERFORMED BY SCHAAF & WHEELER -- AUGUST 2005

5 IO      OUTPUT CONTROL VARIABLES
            IPRNT        0  PRINT CONTROL
            IPLOT        0  PLOT CONTROL
            QSCAL        0.  HYDROGRAPH PLOT SCALE

IT        HYDROGRAPH TIME DATA
            NMIN         1  MINUTES IN COMPUTATION INTERVAL
            IDATE     1  0  STARTING DATE
            ITIME     0000  STARTING TIME
              NQ       200  NUMBER OF HYDROGRAPH ORDINATES
           NDDATE     1  0  ENDING DATE
           NDTIME     0319  ENDING TIME
            ICENT       19  CENTURY MARK

         COMPUTATION INTERVAL    .02 HOURS
             TOTAL TIME BASE   3.32 HOURS

     ENGLISH UNITS
         DRAINAGE AREA         SQUARE MILES
         PRECIPITATION DEPTH   INCHES
         LENGTH, ELEVATION     FEET
         FLOW                  CUBIC FEET PER SECOND
         STORAGE VOLUME        ACRE-FEET
         SURFACE AREA          ACRES
         TEMPERATURE           DEGREES FAHRENHEIT

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

        **************
        *            *
6 KK    *      0     *
        *            *
        **************
              STEEL TANK FAILURE OUTFLOW HYDROGRAPH

         SUBBASIN RUNOFF DATA

8 BA     SUBBASIN CHARACTERISTICS
           TAREA        .00  SUBBASIN AREA

                                 ***

***************************************************************************************************************

                          HYDROGRAPH AT STATION         0

***************************************************************************************************************
```

DA	MON	HRMN	ORD	FLOW	DA	MON	HRMN	ORD	FLOW	DA	MON	HRMN	ORD	FLOW	DA	MON	HRMN	ORD	FLOW
1		0000	1	594.	1		0050	51	0.	1		0140	101	0.	1		0230	151	0.
1		0001	2	575.	1		0051	52	0.	1		0141	102	0.	1		0231	152	0.
1		0002	3	555.	1		0052	53	0.	1		0142	103	0.	1		0232	153	0.
1		0003	4	535.	1		0053	54	0.	1		0143	104	0.	1		0233	154	0.
1		0004	5	515.	1		0054	55	0.	1		0144	105	0.	1		0234	155	0.
1		0005	6	495.	1		0055	56	0.	1		0145	106	0.	1		0235	156	0.
1		0006	7	475.	1		0056	57	0.	1		0146	107	0.	1		0236	157	0.
1		0007	8	455.	1		0057	58	0.	1		0147	108	0.	1		0237	158	0.
1		0008	9	435.	1		0058	59	0.	1		0148	109	0.	1		0238	159	0.
1		0009	10	415.	1		0059	60	0.	1		0149	110	0.	1		0239	160	0.
1		0010	11	395.	1		0100	61	0.	1		0150	111	0.	1		0240	161	0.
1		0011	12	675.	1		0101	62	0.	1		0151	112	0.	1		0241	162	0.
1		0012	13	355.	1		0102	63	0.	1		0152	113	0.	1		0242	163	0.
1		0013	14	335.	1		0103	64	0.	1		0153	114	0.	1		0243	164	0.
1		0014	15	320.	1		0104	65	0.	1		0154	115	0.	1		0244	165	0.
1		0015	16	300.	1		0105	66	0.	1		0155	116	0.	1		0245	166	0.
1		0016	17	275.	1		0106	67	0.	1		0156	117	0.	1		0246	167	0.
1		0017	18	255.	1		0107	68	0.	1		0157	118	0.	1		0247	168	0.

1	0018	19	240.	*	1	0108	69	0.	*	1	0158	119	0.	*	1	0248	169	0.
1	0019	20	220.	*	1	0109	70	0.	*	1	0159	120	0.	*	1	0249	170	0.
1	0020	21	200.	*	1	0110	71	0.	*	1	0200	121	0.	*	1	0250	171	0.
1	0021	22	180.	*	1	0111	72	0.	*	1	0201	122	0.	*	1	0251	172	0.
1	0022	23	160.	*	1	0112	73	0.	*	1	0202	123	0.	*	1	0252	173	0.
1	0023	24	140.	*	1	0113	74	0.	*	1	0203	124	0.	*	1	0253	174	0.
1	0024	25	120.	*	1	0114	75	0.	*	1	0204	125	0.	*	1	0254	175	0.
1	0025	26	100.	*	1	0115	76	0.	*	1	0205	126	0.	*	1	0255	176	0.
1	0026	27	80.	*	1	0116	77	0.	*	1	0206	127	0.	*	1	0256	177	0.
1	0027	28	60.	*	1	0117	78	0.	*	1	0207	128	0.	*	1	0257	178	0.
1	0028	29	40.	*	1	0118	79	0.	*	1	0208	129	0.	*	1	0258	179	0.
1	0029	30	20.	*	1	0119	80	0.	*	1	0209	130	0.	*	1	0259	180	0.
1	0030	31	0.	*	1	0120	81	0.	*	1	0210	131	0.	*	1	0300	181	0.
1	0031	32	0.	*	1	0121	82	0.	*	1	0211	132	0.	*	1	0301	182	0.
1	0032	33	0.	*	1	0122	83	0.	*	1	0212	133	0.	*	1	0302	183	0.
1	0033	34	0.	*	1	0123	84	0.	*	1	0213	134	0.	*	1	0303	184	0.
1	0034	35	0.	*	1	0124	85	0.	*	1	0214	135	0.	*	1	0304	185	0.
1	0035	36	0.	*	1	0125	86	0.	*	1	0215	136	0.	*	1	0305	186	0.
1	0036	37	0.	*	1	0126	87	0.	*	1	0216	137	0.	*	1	0306	187	0.
1	0037	38	0.	*	1	0127	88	0.	*	1	0217	138	0.	*	1	0307	188	0.
1	0038	39	0.	*	1	0128	89	0.	*	1	0218	139	0.	*	1	0308	189	0.
1	0039	40	0.	*	1	0129	90	0.	*	1	0219	140	0.	*	1	0309	190	0.
1	0040	41	0.	*	1	0130	91	0.	*	1	0220	141	0.	*	1	0310	191	0.
1	0041	42	0.	*	1	0131	92	0.	*	1	0221	142	0.	*	1	0311	192	0.
1	0042	43	0.	*	1	0132	93	0.	*	1	0222	143	0.	*	1	0312	193	0.
1	0043	44	0.	*	1	0133	94	0.	*	1	0223	144	0.	*	1	0313	194	0.
1	0044	45	0.	*	1	0134	95	0.	*	1	0224	145	0.	*	1	0314	195	0.
1	0045	46	0.	*	1	0135	96	0.	*	1	0225	146	0.	*	1	0315	196	0.
1	0046	47	0.	*	1	0136	97	0.	*	1	0226	147	0.	*	1	0316	197	0.
1	0047	48	0.	*	1	0137	98	0.	*	1	0227	148	0.	*	1	0317	198	0.
1	0048	49	0.	*	1	0138	99	0.	*	1	0228	149	0.	*	1	0318	199	0.
1	0049	50	0.	*	1	0139	100	0.	*	1	0229	150	0.	*	1	0319	200	0.
				*					*					*				

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************************************************************************************************************************

 PEAK FLOW    TIME                  MAXIMUM AVERAGE FLOW
                          6-HR     24-HR     72-HR    3.32-HR
+ (CFS)      (HR)
                (CFS)
+    675.     .18           46.       46.       46.       46.
              (INCHES)  396.953   396.953   396.953   396.953
               (AC-FT)      13.       13.       13.       13.

              CUMULATIVE AREA =    .00 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


       **************
       *            *
13 KK  *      4     *
       *            *
       **************
               ROUTE OUTFLOW FROM X-SEC 0 TO X-SEC 4

       HYDROGRAPH ROUTING DATA

15 RS      STORAGE ROUTING
            NSTPS          1  NUMBER OF SUBREACHES
            ITYP        STOR  TYPE OF INITIAL CONDITION
           RSVRIC        .00  INITIAL CONDITION
              X          .00  WORKING R AND D COEFFICIENT

16 RC      NORMAL DEPTH CHANNEL
             ANL       .080  LEFT OVERBANK N-VALUE
             ANCH      .080  MAIN CHANNEL N-VALUE
             ANR       .080  RIGHT OVERBANK N-VALUE
            RLNTH       500.  REACH LENGTH
             SEL      .2000  ENERGY SLOPE
            ELMAX         .0  MAX. ELEV. FOR STORAGE/OUTFLOW CALCULATION

                          CROSS-SECTION DATA
             --- LEFT OVERBANK --- + ------ MAIN CHANNEL ------- + --- RIGHT OVERBANK ---
18 RY     ELEVATION   630.00  628.00  622.00  620.00  618.00  620.00  625.00  630.00
17 RX     DISTANCE       .00    6.00   21.00   28.00   33.00   40.00   60.00   75.00

                                 ***

                     COMPUTED STORAGE-OUTFLOW-ELEVATION DATA

     STORAGE       .00      .01      .05      .12      .22      .35      .52      .72      .95     1.21
     OUTFLOW       .00     4.46    28.31    83.48   187.38   346.73   571.70   903.70  1338.57  1868.77
   ELEVATION    618.00   618.63   619.26   619.89   620.53   621.16   621.79   622.42   623.05   623.68

     STORAGE      1.50     1.82     2.17     2.54     2.94     3.36     3.81     4.29     4.79     5.32
     OUTFLOW   2501.07  3241.96  4109.68  5094.75  6200.16  7430.16  8787.53 10272.28 11897.69 13668.69
   ELEVATION    624.32   624.95   625.58   626.21   626.84   627.47   628.11   628.74   629.37   630.00

*** WARNING *** MODIFIED PULS ROUTING MAY BE NUMERICALLY UNSTABLE FOR OUTFLOWS BETWEEN    572. TO   13669.
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THE ROUTED HYDROGRAPH SHOULD BE EXAMINED FOR OSCILLATIONS OR OUTFLOWS GREATER THAN PEAK INFLOWS.
THIS CAN BE CORRECTED BY DECREASING THE TIME INTERVAL OR INCREASING STORAGE (USE A LONGER REACH.)

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HYDROGRAPH AT STATION 4

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DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE *	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE *	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE
1		0000	1	0.	.0	618.0 *	1		0107	68	0.	.0	618.0 *	1		0214	135	0.	.0	618.0
1		0001	2	494.	.5	621.6 *	1		0108	69	0.	.0	618.0 *	1		0215	136	0.	.0	618.0
1		0002	3	562.	.5	621.8 *	1		0109	70	0.	.0	618.0 *	1		0216	137	0.	.0	618.0
1		0003	4	546.	.5	621.7 *	1		0110	71	0.	.0	618.0 *	1		0217	138	0.	.0	618.0
1		0004	5	526.	.5	621.7 *	1		0111	72	0.	.0	618.0 *	1		0218	139	0.	.0	618.0
1		0005	6	506.	.5	621.6 *	1		0112	73	0.	.0	618.0 *	1		0219	140	0.	.0	618.0
1		0006	7	486.	.5	621.5 *	1		0113	74	0.	.0	618.0 *	1		0220	141	0.	.0	618.0
1		0007	8	466.	.4	621.5 *	1		0114	75	0.	.0	618.0 *	1		0221	142	0.	.0	618.0
1		0008	9	446.	.4	621.4 *	1		0115	76	0.	.0	618.0 *	1		0222	143	0.	.0	618.0
1		0009	10	426.	.4	621.4 *	1		0116	77	0.	.0	618.0 *	1		0223	144	0.	.0	618.0
1		0010	11	406.	.4	621.3 *	1		0117	78	0.	.0	618.0 *	1		0224	145	0.	.0	618.0
1		0011	12	530.	.5	621.7 *	1		0118	79	0.	.0	618.0 *	1		0225	146	0.	.0	618.0
1		0012	13	516.	.5	621.6 *	1		0119	80	0.	.0	618.0 *	1		0226	147	0.	.0	618.0
1		0013	14	351.	.4	621.2 *	1		0120	81	0.	.0	618.0 *	1		0227	148	0.	.0	618.0
1		0014	15	329.	.3	621.1 *	1		0121	82	0.	.0	618.0 *	1		0228	149	0.	.0	618.0
1		0015	16	312.	.3	621.0 *	1		0122	83	0.	.0	618.0 *	1		0229	150	0.	.0	618.0
1		0016	17	290.	.3	620.9 *	1		0123	84	0.	.0	618.0 *	1		0230	151	0.	.0	618.0
1		0017	18	267.	.3	620.8 *	1		0124	85	0.	.0	618.0 *	1		0231	152	0.	.0	618.0
1		0018	19	249.	.3	620.8 *	1		0125	86	0.	.0	618.0 *	1		0232	153	0.	.0	618.0
1		0019	20	232.	.3	620.7 *	1		0126	87	0.	.0	618.0 *	1		0233	154	0.	.0	618.0
1		0020	21	212.	.2	620.6 *	1		0127	88	0.	.0	618.0 *	1		0234	155	0.	.0	618.0
1		0021	22	192.	.2	620.5 *	1		0128	89	0.	.0	618.0 *	1		0235	156	0.	.0	618.0
1		0022	23	173.	.2	620.4 *	1		0129	90	0.	.0	618.0 *	1		0236	157	0.	.0	618.0
1		0023	24	154.	.2	620.3 *	1		0130	91	0.	.0	618.0 *	1		0237	158	0.	.0	618.0
1		0024	25	134.	.2	620.2 *	1		0131	92	0.	.0	618.0 *	1		0238	159	0.	.0	618.0
1		0025	26	114.	.2	620.1 *	1		0132	93	0.	.0	618.0 *	1		0239	160	0.	.0	618.0
1		0026	27	94.	.1	620.0 *	1		0133	94	0.	.0	618.0 *	1		0240	161	0.	.0	618.0
1		0027	28	75.	.1	619.8 *	1		0134	95	0.	.0	618.0 *	1		0241	162	0.	.0	618.0
1		0028	29	57.	.1	619.6 *	1		0135	96	0.	.0	618.0 *	1		0242	163	0.	.0	618.0
1		0029	30	38.	.1	619.4 *	1		0136	97	0.	.0	618.0 *	1		0243	164	0.	.0	618.0
1		0030	31	20.	.0	619.0 *	1		0137	98	0.	.0	618.0 *	1		0244	165	0.	.0	618.0
1		0031	32	9.	.0	618.7 *	1		0138	99	0.	.0	618.0 *	1		0245	166	0.	.0	618.0
1		0032	33	4.	.0	618.6 *	1		0139	100	0.	.0	618.0 *	1		0246	167	0.	.0	618.0
1		0033	34	3.	.0	618.4 *	1		0140	101	0.	.0	618.0 *	1		0247	168	0.	.0	618.0
1		0034	35	2.	.0	618.2 *	1		0141	102	0.	.0	618.0 *	1		0248	169	0.	.0	618.0
1		0035	36	1.	.0	618.1 *	1		0142	103	0.	.0	618.0 *	1		0249	170	0.	.0	618.0
1		0036	37	1.	.0	618.1 *	1		0143	104	0.	.0	618.0 *	1		0250	171	0.	.0	618.0
1		0037	38	0.	.0	618.1 *	1		0144	105	0.	.0	618.0 *	1		0251	172	0.	.0	618.0
1		0038	39	0.	.0	618.0 *	1		0145	106	0.	.0	618.0 *	1		0252	173	0.	.0	618.0
1		0039	40	0.	.0	618.0 *	1		0146	107	0.	.0	618.0 *	1		0253	174	0.	.0	618.0
1		0040	41	0.	.0	618.0 *	1		0147	108	0.	.0	618.0 *	1		0254	175	0.	.0	618.0
1		0041	42	0.	.0	618.0 *	1		0148	109	0.	.0	618.0 *	1		0255	176	0.	.0	618.0
1		0042	43	0.	.0	618.0 *	1		0149	110	0.	.0	618.0 *	1		0256	177	0.	.0	618.0
1		0043	44	0.	.0	618.0 *	1		0150	111	0.	.0	618.0 *	1		0257	178	0.	.0	618.0
1		0044	45	0.	.0	618.0 *	1		0151	112	0.	.0	618.0 *	1		0258	179	0.	.0	618.0
1		0045	46	0.	.0	618.0 *	1		0152	113	0.	.0	618.0 *	1		0259	180	0.	.0	618.0
1		0046	47	0.	.0	618.0 *	1		0153	114	0.	.0	618.0 *	1		0300	181	0.	.0	618.0
1		0047	48	0.	.0	618.0 *	1		0154	115	0.	.0	618.0 *	1		0301	182	0.	.0	618.0
1		0048	49	0.	.0	618.0 *	1		0155	116	0.	.0	618.0 *	1		0302	183	0.	.0	618.0
1		0049	50	0.	.0	618.0 *	1		0156	117	0.	.0	618.0 *	1		0303	184	0.	.0	618.0
1		0050	51	0.	.0	618.0 *	1		0157	118	0.	.0	618.0 *	1		0304	185	0.	.0	618.0
1		0051	52	0.	.0	618.0 *	1		0158	119	0.	.0	618.0 *	1		0305	186	0.	.0	618.0
1		0052	53	0.	.0	618.0 *	1		0159	120	0.	.0	618.0 *	1		0306	187	0.	.0	618.0
1		0053	54	0.	.0	618.0 *	1		0200	121	0.	.0	618.0 *	1		0307	188	0.	.0	618.0
1		0054	55	0.	.0	618.0 *	1		0201	122	0.	.0	618.0 *	1		0308	189	0.	.0	618.0
1		0055	56	0.	.0	618.0 *	1		0202	123	0.	.0	618.0 *	1		0309	190	0.	.0	618.0
1		0056	57	0.	.0	618.0 *	1		0203	124	0.	.0	618.0 *	1		0310	191	0.	.0	618.0
1		0057	58	0.	.0	618.0 *	1		0204	125	0.	.0	618.0 *	1		0311	192	0.	.0	618.0
1		0058	59	0.	.0	618.0 *	1		0205	126	0.	.0	618.0 *	1		0312	193	0.	.0	618.0
1		0059	60	0.	.0	618.0 *	1		0206	127	0.	.0	618.0 *	1		0313	194	0.	.0	618.0
1		0100	61	0.	.0	618.0 *	1		0207	128	0.	.0	618.0 *	1		0314	195	0.	.0	618.0
1		0101	62	0.	.0	618.0 *	1		0208	129	0.	.0	618.0 *	1		0315	196	0.	.0	618.0
1		0102	63	0.	.0	618.0 *	1		0209	130	0.	.0	618.0 *	1		0316	197	0.	.0	618.0
1		0103	64	0.	.0	618.0 *	1		0210	131	0.	.0	618.0 *	1		0317	198	0.	.0	618.0
1		0104	65	0.	.0	618.0 *	1		0211	132	0.	.0	618.0 *	1		0318	199	0.	.0	618.0
1		0105	66	0.	.0	618.0 *	1		0212	133	0.	.0	618.0 *	1		0319	200	0.	.0	618.0
1		0106	67	0.	.0	618.0 *	1		0213	134	0.	.0	618.0 *							

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	PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
				6-HR	24-HR	72-HR	3.32-HR
+	(CFS)	(HR)					
			(CFS)				
+	562.	.03		46.	46.	46.	46.
			(INCHES)	396.953	396.953	396.953	396.953
			(AC-FT)	13.	13.	13.	13.

	PEAK STORAGE	TIME	MAXIMUM AVERAGE STORAGE			
			6-HR	24-HR	72-HR	3.32-HR
+	(AC-FT)	(HR)				
	1.	.03	0.	0.	0.	0.

PEAK STAGE TIME MAXIMUM AVERAGE STAGE

+ (FEET)	(HR)	6-HR	24-HR	72-HR	3.32-HR
621.76	.03	618.44	618.44	618.44	618.44

CUMULATIVE AREA = .00 SQ MI

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*　　　*
19 KK * 5 *
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ROUTE HYDROGRAPH THROUGH NEW DEBRIS BASIN

HYDROGRAPH ROUTING DATA

21 RS STORAGE ROUTING
NSTPS 1 NUMBER OF SUBREACHES
ITYP STOR TYPE OF INITIAL CONDITION
RSVRIC .00 INITIAL CONDITION
X .00 WORKING R AND D COEFFICIENT

22 SV	STORAGE	.0	7.5	8.0	8.1	8.6	9.1
23 SQ	DISCHARGE	0.	0.	260.	280.	301.	340.
24 SE	ELEVATION	568.00	584.00	585.00	585.20	585.70	586.20

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HYDROGRAPH AT STATION 5

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DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE
1		0000	1	0.	.0	568.0	*	1		0107	68	0.	7.5	584.0	*	1		0214	135	0.	7.5	584.0
1		0001	2	0.	.3	568.7	*	1		0108	69	0.	7.5	584.0	*	1		0215	136	0.	7.5	584.0
1		0002	3	0.	1.1	570.3	*	1		0109	70	0.	7.5	584.0	*	1		0216	137	0.	7.5	584.0
1		0003	4	0.	1.8	571.9	*	1		0110	71	0.	7.5	584.0	*	1		0217	138	0.	7.5	584.0
1		0004	5	0.	2.6	573.5	*	1		0111	72	0.	7.5	584.0	*	1		0218	139	0.	7.5	584.0
1		0005	6	0.	3.3	575.0	*	1		0112	73	0.	7.5	584.0	*	1		0219	140	0.	7.5	584.0
1		0006	7	0.	4.0	576.5	*	1		0113	74	0.	7.5	584.0	*	1		0220	141	0.	7.5	584.0
1		0007	8	0.	4.6	577.9	*	1		0114	75	0.	7.5	584.0	*	1		0221	142	0.	7.5	584.0
1		0008	9	0.	5.2	579.2	*	1		0115	76	0.	7.5	584.0	*	1		0222	143	0.	7.5	584.0
1		0009	10	0.	5.8	580.5	*	1		0116	77	0.	7.5	584.0	*	1		0223	144	0.	7.5	584.0
1		0010	11	0.	6.4	581.7	*	1		0117	78	0.	7.5	584.0	*	1		0224	145	0.	7.5	584.0
1		0011	12	0.	7.1	583.1	*	1		0118	79	0.	7.5	584.0	*	1		0225	146	0.	7.5	584.0
1		0012	13	108.	7.7	584.4	*	1		0119	80	0.	7.5	584.0	*	1		0226	147	0.	7.5	584.0
1		0013	14	269.	8.0	585.1	*	1		0120	81	0.	7.5	584.0	*	1		0227	148	0.	7.5	584.0
1		0014	15	281.	8.1	585.2	*	1		0121	82	0.	7.5	584.0	*	1		0228	149	0.	7.5	584.0
1		0015	16	284.	8.2	585.3	*	1		0122	83	0.	7.5	584.0	*	1		0229	150	0.	7.5	584.0
1		0016	17	285.	8.2	585.3	*	1		0123	84	0.	7.5	584.0	*	1		0230	151	0.	7.5	584.0
1		0017	18	284.	8.2	585.3	*	1		0124	85	0.	7.5	584.0	*	1		0231	152	0.	7.5	584.0
1		0018	19	283.	8.2	585.3	*	1		0125	86	0.	7.5	584.0	*	1		0232	153	0.	7.5	584.0
1		0019	20	280.	8.1	585.2	*	1		0126	87	0.	7.5	584.0	*	1		0233	154	0.	7.5	584.0
1		0020	21	268.	8.0	585.1	*	1		0127	88	0.	7.5	584.0	*	1		0234	155	0.	7.5	584.0
1		0021	22	242.	8.0	584.9	*	1		0128	89	0.	7.5	584.0	*	1		0235	156	0.	7.5	584.0
1		0022	23	211.	7.9	584.8	*	1		0129	90	0.	7.5	584.0	*	1		0236	157	0.	7.5	584.0
1		0023	24	186.	7.9	584.7	*	1		0130	91	0.	7.5	584.0	*	1		0237	158	0.	7.5	584.0
1		0024	25	164.	7.8	584.6	*	1		0131	92	0.	7.5	584.0	*	1		0238	159	0.	7.5	584.0
1		0025	26	143.	7.8	584.5	*	1		0132	93	0.	7.5	584.0	*	1		0239	160	0.	7.5	584.0
1		0026	27	122.	7.7	584.5	*	1		0133	94	0.	7.5	584.0	*	1		0240	161	0.	7.5	584.0
1		0027	28	102.	7.7	584.4	*	1		0134	95	0.	7.5	584.0	*	1		0241	162	0.	7.5	584.0
1		0028	29	83.	7.7	584.3	*	1		0135	96	0.	7.5	584.0	*	1		0242	163	0.	7.5	584.0
1		0029	30	64.	7.6	584.2	*	1		0136	97	0.	7.5	584.0	*	1		0243	164	0.	7.5	584.0
1		0030	31	46.	7.6	584.2	*	1		0137	98	0.	7.5	584.0	*	1		0244	165	0.	7.5	584.0
1		0031	32	29.	7.6	584.1	*	1		0138	99	0.	7.5	584.0	*	1		0245	166	0.	7.5	584.0
1		0032	33	17.	7.5	584.1	*	1		0139	100	0.	7.5	584.0	*	1		0246	167	0.	7.5	584.0
1		0033	34	10.	7.5	584.0	*	1		0140	101	0.	7.5	584.0	*	1		0247	168	0.	7.5	584.0
1		0034	35	6.	7.5	584.0	*	1		0141	102	0.	7.5	584.0	*	1		0248	169	0.	7.5	584.0
1		0035	36	3.	7.5	584.0	*	1		0142	103	0.	7.5	584.0	*	1		0249	170	0.	7.5	584.0
1		0036	37	2.	7.5	584.0	*	1		0143	104	0.	7.5	584.0	*	1		0250	171	0.	7.5	584.0
1		0037	38	1.	7.5	584.0	*	1		0144	105	0.	7.5	584.0	*	1		0251	172	0.	7.5	584.0
1		0038	39	1.	7.5	584.0	*	1		0145	106	0.	7.5	584.0	*	1		0252	173	0.	7.5	584.0
1		0039	40	0.	7.5	584.0	*	1		0146	107	0.	7.5	584.0	*	1		0253	174	0.	7.5	584.0
1		0040	41	0.	7.5	584.0	*	1		0147	108	0.	7.5	584.0	*	1		0254	175	0.	7.5	584.0
1		0041	42	0.	7.5	584.0	*	1		0148	109	0.	7.5	584.0	*	1		0255	176	0.	7.5	584.0
1		0042	43	0.	7.5	584.0	*	1		0149	110	0.	7.5	584.0	*	1		0256	177	0.	7.5	584.0
1		0043	44	0.	7.5	584.0	*	1		0150	111	0.	7.5	584.0	*	1		0257	178	0.	7.5	584.0
1		0044	45	0.	7.5	584.0	*	1		0151	112	0.	7.5	584.0	*	1		0258	179	0.	7.5	584.0
1		0045	46	0.	7.5	584.0	*	1		0152	113	0.	7.5	584.0	*	1		0259	180	0.	7.5	584.0
1		0046	47	0.	7.5	584.0	*	1		0153	114	0.	7.5	584.0	*	1		0300	181	0.	7.5	584.0
1		0047	48	0.	7.5	584.0	*	1		0154	115	0.	7.5	584.0	*	1		0301	182	0.	7.5	584.0

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1   0048 49   0.   7.5  584.0 * 1   0155 116   0.   7.5  584.0 * 1   0302 183   0.   7.5  584.0
1   0049 50   0.   7.5  584.0 * 1   0156 117   0.   7.5  584.0 * 1   0303 184   0.   7.5  584.0
1   0050 51   0.   7.5  584.0 * 1   0157 118   0.   7.5  584.0 * 1   0304 185   0.   7.5  584.0
1   0051 52   0.   7.5  584.0 * 1   0158 119   0.   7.5  584.0 * 1   0305 186   0.   7.5  584.0
1   0052 53   0.   7.5  584.0 * 1   0159 120   0.   7.5  584.0 * 1   0306 187   0.   7.5  584.0
1   0053 54   0.   7.5  584.0 * 1   0200 121   0.   7.5  584.0 * 1   0307 188   0.   7.5  584.0
1   0054 55   0.   7.5  584.0 * 1   0201 122   0.   7.5  584.0 * 1   0308 189   0.   7.5  584.0
1   0055 56   0.   7.5  584.0 * 1   0202 123   0.   7.5  584.0 * 1   0309 190   0.   7.5  584.0
1   0056 57   0.   7.5  584.0 * 1   0203 124   0.   7.5  584.0 * 1   0310 191   0.   7.5  584.0
1   0057 58   0.   7.5  584.0 * 1   0204 125   0.   7.5  584.0 * 1   0311 192   0.   7.5  584.0
1   0058 59   0.   7.5  584.0 * 1   0205 126   0.   7.5  584.0 * 1   0312 193   0.   7.5  584.0
1   0059 60   0.   7.5  584.0 * 1   0206 127   0.   7.5  584.0 * 1   0313 194   0.   7.5  584.0
1   0100 61   0.   7.5  584.0 * 1   0207 128   0.   7.5  584.0 * 1   0314 195   0.   7.5  584.0
1   0101 62   0.   7.5  584.0 * 1   0208 129   0.   7.5  584.0 * 1   0315 196   0.   7.5  584.0
1   0102 63   0.   7.5  584.0 * 1   0209 130   0.   7.5  584.0 * 1   0316 197   0.   7.5  584.0
1   0103 64   0.   7.5  584.0 * 1   0210 131   0.   7.5  584.0 * 1   0317 198   0.   7.5  584.0
1   0104 65   0.   7.5  584.0 * 1   0211 132   0.   7.5  584.0 * 1   0318 199   0.   7.5  584.0
1   0105 66   0.   7.5  584.0 * 1   0212 133   0.   7.5  584.0 * 1   0319 200   0.   7.5  584.0
1   0106 67   0.   7.5  584.0 * 1   0213 134   0.   7.5  584.0 *
                    *                           *
*****************************************************************************************************************************

 PEAK FLOW    TIME              MAXIMUM AVERAGE FLOW
                   6-HR    24-HR    72-HR    3.32-HR
+ (CFS)     (HR)
               (CFS)
+  285.     .27       19.      19.      19.      19.
            (INCHES) 163.248  163.248  163.248  163.248
             (AC-FT)     5.       5.       5.       5.

 PEAK STORAGE  TIME             MAXIMUM AVERAGE STORAGE
                   6-HR    24-HR    72-HR    3.32-HR
+ (AC-FT)   (HR)
    8.   .27          7.       7.       7.       7.

 PEAK STAGE   TIME              MAXIMUM AVERAGE STAGE
                   6-HR    24-HR    72-HR    3.32-HR
+ (FEET)    (HR)
  585.31    .27        583.59   583.59   583.59   583.59

             CUMULATIVE AREA =    .00 SQ MI

1
                                RUNOFF SUMMARY
                          FLOW IN CUBIC FEET PER SECOND
                        TIME IN HOURS,  AREA IN SQUARE MILES

                   PEAK  TIME OF   AVERAGE FLOW FOR MAXIMUM PERIOD   BASIN   MAXIMUM    TIME OF
     OPERATION    STATION   FLOW   PEAK                               AREA    STAGE   MAX STAGE
+                                6-HOUR   24-HOUR   72-HOUR

     HYDROGRAPH AT
+                0    675.   .18      46.      46.      46.      .00

     ROUTED TO
+                4    562.   .03      46.      46.      46.      .00
+                                                          621.76      .03

     ROUTED TO
+                5    285.   .27      19.      19.      19.      .00
+                                                          585.31      .27


*** NORMAL END OF HEC-1 ***
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