CONCEPTUAL HYDROLOGIC AND HYDRAULIC PROJECT DESIGN AND FUNCTION: WETLAND/EXTENDED DETENTION ALTERNATIVE (ALTERNATIVE 2A) Cucamonga Creek Watershed Regional Water Quality Project[1]

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1 INTRODUCTION

The Cucamonga Creek Watershed Regional Water Quality Project (Project) proposes to create recreational and habitat opportunities while treating stormwater runoff from the Cucamonga Creek watershed. The multiple goals of the Project will be met by the creation of a series of hydraulically connected basins that incorporate wetland and riparian areas, recreational trails and educational kiosks, and water treatment components.

The purpose of this memorandum is to discuss the hydrologic and hydraulic design and function of the Project. The Project proposes to treat portions of both dry and wet weather flows in Cucamonga Channel that otherwise would continue flowing down to the Santa Ana River and Pacific Ocean untreated for stormwater pollutants such as bacteria and nutrients. The Project will achieve water quality treatment by diverting flows from Cucamonga Channel, routing diverted flows through a series of cascading ponds which combine constructed wetland and extended detention basin treatment features, and returning treated flows back to Mill Creek, 0.67 mile downstream of the diversion location on Cucamonga Channel[2].

The Project illustrates the leveraged benefit of regional treatment facilities. The Project will have the capacity to hold and treat 160 acre-feet of water at any given time and will provide treatment of 10%-18% of all wet-weather runoff from the Cucamonga Channel watershed (76.7 mi^2). In contrast for example, a single-function water quality project of the same volume capacity (160 AF) located to treat runoff from a developed area on-site could, for example, mitigate impacts from a 6.25 square mile urban area and treat approximately 70% of wet weather runoff, resulting in an effective capture volume of approximately 6 % of the total wet-weather runoff from the watershed (based on San Bernardino volume sizing requirements). The Project as designed

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[1] Funding for this project has been provided in full or in part through an agreement with the State Water Resources Control Board. The contents of this document do not necessarily reflect the views and policies of the State Water Resources Control Board, nor does mention of trade names or commercial products constitute endorsement or recommendation for use." (Gov. Code 7550, 40 CFR 31.20)

[2] The term "diversion" references the hydraulic diversion of flows from the channel that would eventually be returned to the same drainage system. It does not imply any diversion of flows from one watershed to another.

provides some flood control benefits, the extent of which varies with the magnitude of the flood flow rates in Cucamonga Channel.

The Project thus not only provides multiple benefits, it also increases effective volume treated by 40% to 200%, showing a significant increase in return on investment. This memorandum focuses on how the hydrology and hydraulics will be designed to optimize treatment potential and minimize project costs. Specific water quality benefits with respect to treatment are discussed in the Existing and Proposed Surface Water Quality memo (Geosyntec Consultants, November 2008).

2 PROJECT SETTING/SITE DESCRIPTION

2.1 Project Location

The Project is located in the City of Chino in San Bernardino County along Mill Creek/Cucamonga Channel upstream of the Prado Dam in the Santa Ana River Basin. The Project can roughly be delimited by Cucamonga Channel/Mill Creek to the southeast, Comet Road to the west and the Cucamonga Creek crossing of Hellman Avenue to the north. Figure 1 shows the location of the Project site and approximate Project limits.

The Project is adjacent to Cucamonga Channel near its terminus and transition to Mill Creek. The Project site is located on the northwest floodplain of the channel through this transition reach. The floodplain and Project site is bisected by Chino Corona Road, which runs in the east-west direction, and crosses the channel over a series of 10 culverts. Site slopes are generally mild from north to south, with somewhat steeper slopes in the creek corridor. Because the Project site is within the floodplain, it is subject to inundation under flood conditions when flows in the adjacent channel overtop the channel banks. The connection between the channel and floodplain varies along the length of the Project and therefore the Project site is inundated under varying conditions in different places.

2.2 Tributary Watershed

Cucamonga Channel, at its terminus, drains a watershed of approximately 76.7 mi^2. This watershed area incorporates the tributary areas of Deer Creek, Old Deer Creek and West Cucamonga Creek as well. The watershed is comprised of both low gradient urbanized valley areas as well as steeper mountain tributaries. The majority of the mountainous tributary area drains to more defined channels in the valley through debris basins. There are approximately ten debris basins which filter debris from mountain flows prior to discharge into the valley channels.

The mountainous areas are predominantly undeveloped open space with low imperviousness. The valley portion of the watershed, in contrast, is well developed and comprised of a mixture of

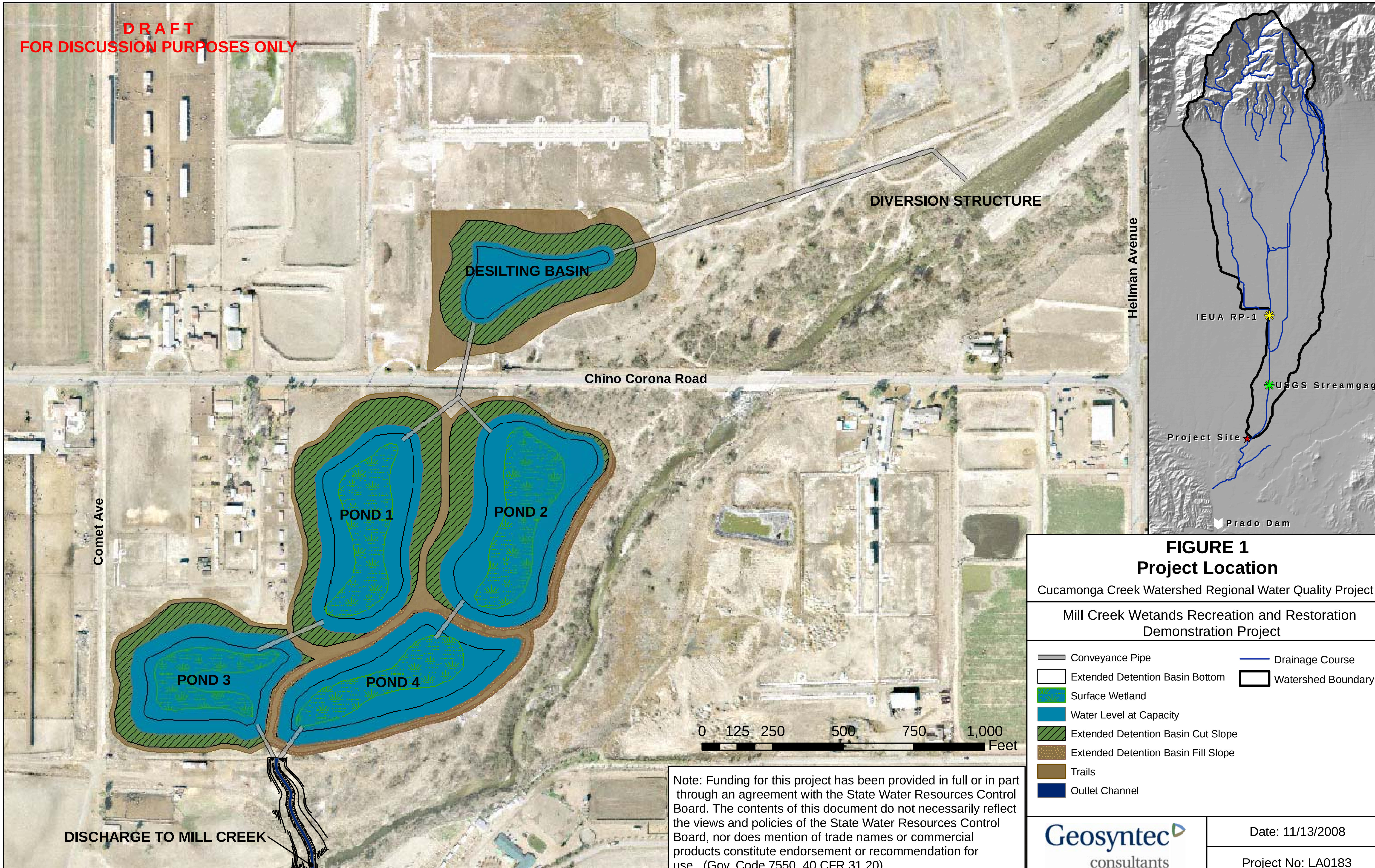

FIGURE 1
Project Location

Cucamonga Creek Watershed Regional Water Quality Project

Mill Creek Wetands Recreation and Restoration Demonstration Project

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Geosyntec consultants

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industrial, commercial, agricultural and residential land uses. Historically, the valley portion of watershed consisted of predominantly agricultural and dairy land uses. Today, the agricultural and dairy farms are being replaced by residential developments. The watershed, as a whole, is approximately 25% impervious based on land use data from 2005 (SCAG, 2005).

2.3 Cucamonga Channel

Cucamonga Channel is a concrete flood control channel that was constructed in the 1970’s by the United States Army Corps of Engineers (USACE). The upstream end of the concrete channel coincides with the outlet of the Cucamonga Creek Debris Basin in the foothills of the San Bernardino Mountains. At its terminus downstream of Hellman Avenue and upstream of Chino Corona Road, the concrete flood control channel transitions to a grouted rip-rap channel and then finally discharges to Mill Creek. At this point, Cucamonga Channel is a trapezoidal channel with a bottom width of 138 ft and a design depth of 21 ft.

2.4 Mill Creek

Mill Creek is not a concrete flood control channel but is comprised of natural soils and vegetation. The current connection of Mill Creek with Cucamonga Channel however, is not necessarily natural. As recent as 1928, Mill Creek terminated prior to connection with Cucamonga Creek.

During dry weather, flows in Cucamonga Channel generally vary between 30 and 60 cfs, primarily consisting of effluent from the Inland Empire Utilities Agency (IEUA) RP-1 water reclamation plant that discharges to the channel upstream of the diversion. Cucamonga Channel also receives dry weather flows from activities in the tributary watershed associated with dairy farming, agriculture, residential communities, industry and commercial activities, and potential groundwater sources.

During wet weather, the channel conveys stormwater runoff from the entire 76.7 mi^2 watershed, with peak flows observed as high as 17,300 (USGS, October 2004) in 20 years of stream gage data (USGS). The estimated 100-year flow event is 32,000 cfs (USACE, 1973) and the Probable Maximum Flood (PMF) is 52,000 cfs (USACE, 1973). Under the 100-year and PMF events, Mill Creek overtops its channel banks along the entire reach adjacent to the Project and inundates portions of the floodplain and Project site.

The 10 culverts which convey flows in Mill Creek under Chino Corona Road have modified the hydrologic regime within the vicinity of the Project. As sized, the culverts only effectively convey low flows and result in backwater effects up Mill Creek and Cucamonga Channel under medium to high flow conditions. Under high flow conditions, the culvert crossing acts as an Arizona Crossing and flows overtop the road and are conveyed back into the channel on the

downstream end of the culverts. Chino Corona Road, because it is higher than the adjacent floodplain, acts as a partial dam on the floodplain and prevents flood waters up to a certain water surface elevation from draining downstream along the floodplain. These conditions have facilitated overtopping of the channel banks and sediment deposition in the reach immediately upstream of the culverts.

Downstream of the culverts, the Mill Creek channel is degraded, likely as a result of the impacts of the culverts on sediment transport processes within the reach and upstream increases in hydrologic source loading. Because of the backwater effects and reduced velocities upstream of the culverts, sediments in suspension may be deposited under a range of flows upstream of the culverts. The flows through the culverts, therefore, lose a portion of their sediment load and are discharged to the downstream reach deficient in sediment. To account for sediment losses upstream, flows downstream of the culverts erode the channel bed and banks. The channel, in this reach, is therefore deeply incised with steep channel banks. Because of this channel geometry, the channel has lost its natural connection with the floodplain and the frequency of floodplain inundation in this reach is less than that upstream of the Chino Corona Road culverts. It is also noted that the artificial connection of Cucamonga Channel to Mill Creek constructed in first half of the century likely had dramatic effects on the hydrologic regime in the channel at that time and the channel may still be adjusting today.

2.5 Prado Dam

Prado Dam is approximately 5 miles downstream of the Project location. Prado Dam was constructed for flood control purposes in 1941. The Project location is within the area that could potentially be inundated by water retained behind the dam at its capacity. The spillway on the dam is set at 543 ft MSL, however the crest of the embankments have been set at 566 ft MSL. The Project site ranges in elevation between 519 ft MSL and 563 ft MSL and therefore, the entire site would be inundated if the dam were filled to capacity. Because the 100-year flow event in Cucamonga Channel does not necessarily coincide with the water surface elevation at Prado Dam capacity, these circumstances were not used as boundary conditions in analyses discussed herein.

3 PROPOSED PROJECT CONDITIONS

The Project proposes to divert flow from Cucamonga Channel in the vicinity of Hellman Avenue and route it through a series of wetlands/extended detention basins before discharging back to Mill Creek approximately 0.67 mile downstream of the diversion point. The Project will also receive dry weather and low storm flows from a portion of an adjacent development known as The Chino Preserve. The diversion, flow routing between the basins, and discharge back to Mill

Creek will be driven by gravity flow. A schematic representation of the proposed project is provided in Figure 2.

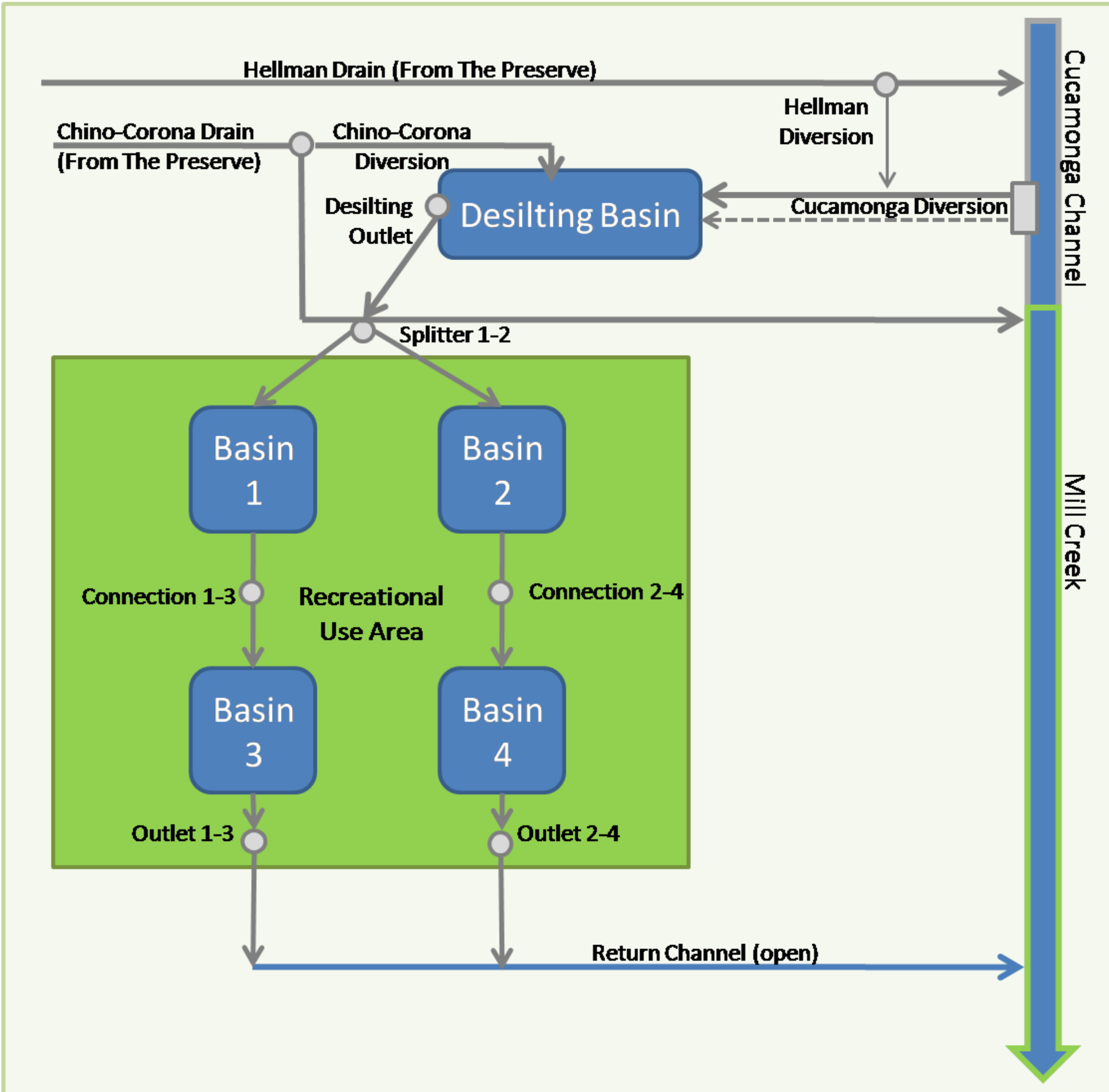

Figure 2: Plan view schematic of processes of Cucamonga Creek Watershed Regional Water Quality Project

3.1 Project Operational Regimes and Design Flows

The Project will operate differently during dry and wet weather regimes as described below and shown graphically in Figure 3. The wet-weather regime is further divided into filling, overflow, and drawdown. Design flows under each regime are discussed.

3.1.1 Dry Weather

During dry weather, a portion of the flow in Cucamonga Channel will be diverted and routed through the Desilting Basin into constructed wetlands located in the bottoms of Ponds 1-4. Minimal dry-weather runoff is expected from The Chino Preserve. Under the dry-weather flow regime, water surface elevations in the basin system will remain relatively constant, with water surface elevations maintained by internal hydraulic controls at the outlet of each Basin. The design dry weather flowrate, Q_{dw}, will be set to maintain the integrity of the constructed wetlands, and to balance the treatment of additional flows with the preservation of required in-channel flows. It is anticipated that the Q_{dw} will be on the order of 2.5 to 15.0 cfs; actual design criteria are presented in Section 3.2.1 and is driven by the in-creek flow required to avoid environmental impacts.

3.1.2 Wet Weather

Filling. During wet weather, water levels in Cucamonga Channel and The Chino Preserve drains will rise, resulting in much higher flows through the diversions to the basins. Diverted flowrates that exceed rates of discharge from the system will cause the Basins to fill and water surface elevations to rise. Hydraulic controls (pipes and/or orifices sized based on maintenance of head) between Basins 1 and 3, and 2 and 4, will serve to maintain different water surface levels between Basins 1 and 2, and 3 and 4 during normal operation. The ability to maintain the 2 feet of head between Basins 1 and 2, and 3 and 4 maximizes basin storage capacity. This operational strategy will result in overflow between ponds during peak flow events, as peak flows will not be permitted to equalize sufficiently through the normal flow orifice.

Design flowrates will be based on the performance of the system over the entire range of expected conditions in Cucamonga Channel. Where appropriate, flow frequency and duration relationships will be developed for design purposes. For the purpose of the discussion below, Q_{ssmax} is defined as the maximum steady-state flowrate through the system without overflow. It would be expected to occur when the basin chain is just below overflow stage.

Drawdown. Outlet control structures at the end of the basin chain will control the rate of release of stored water, providing a specified drawdown time for surcharged volume following a storm event.

Discharge during the drawdown phase will be governed by an orifice-type control intended to provide the established drawdown time of the basin chain while continuing to process incoming dry weather flows. Currently, the drawdown time is set at 48 hours, sufficient to achieve water quality improvement.

Overflow. Extended periods of wet weather diversion greater than Q_{ssmax} will cause the basins to fill completely and spill over overflow structures (overflow riser or spillway) into subsequent downstream ponds or into the outlet channel and back to Mill Creek. Under normal operations, when there are not extended periods of high surface water elevations in Cucamonga Channel, these overflow structures will not be active. If overflow structures are designed as spillways (the specifics of which are not critical for this analysis), recreational trails that line the perimeters of the ponds will have to be closed during periods of active flow over the overflow structures for safety purposes. The structure diverting wet weather flows from Cucamonga Channel, as discussed herein, is sized to minimize potential for flow over the overflow structures while maintaining treatment objectives. If further modifications are necessary for the diversion during design, this will remain a consideration.

The design maximum flowrate through the basin chain will be determined through hydraulic analysis of proposed diversion works under the 100-year runoff conditions in Cucamonga Channel and The Chino Preserve storm drains. For the purpose of the discussion below, this flowrate is referred to as Q_{max}. Based on the diversion works being considered at this time for design, the Q_{max} will be approximately 384 to 404 cfs.

3.2 Project Design Features and Design Criteria

Proposed Project Design Features (PDFs) are described in the sections below. Source and form of design criteria are also discussed.

3.2.1 Cucamonga Channel Diversion Structure and Piping

Description A diversion structure and conduit from Cucamonga Channel will divert dry- and wet-weather flows to the basin system. It is anticipated that a cutoff-trench and low-flow orifice or equivalent design will be used to control the desired dry weather flowrate to the basin chain. Depending on design constraints, this flow may be routed in a separate conduit or be conveyed in the wet-weather diversion line described below. The maximum flow through this line, Q_{dw}, is assumed to be small compared to the wet-weather diversion rates and therefore represents a minor design component for hydraulic capacity limitations.

It is anticipated that the wet weather diversion structure will consist of a side-flow weir or equivalent, spilling to a side-flow channel, connected to a large gravity diversion line between the diversion point and the desilting basin. Hydraulic limitations may result from weir losses at the diversion, capacity limitations in the diversion pipeline, tailwater effects from the basin chain, or a combination of the three.

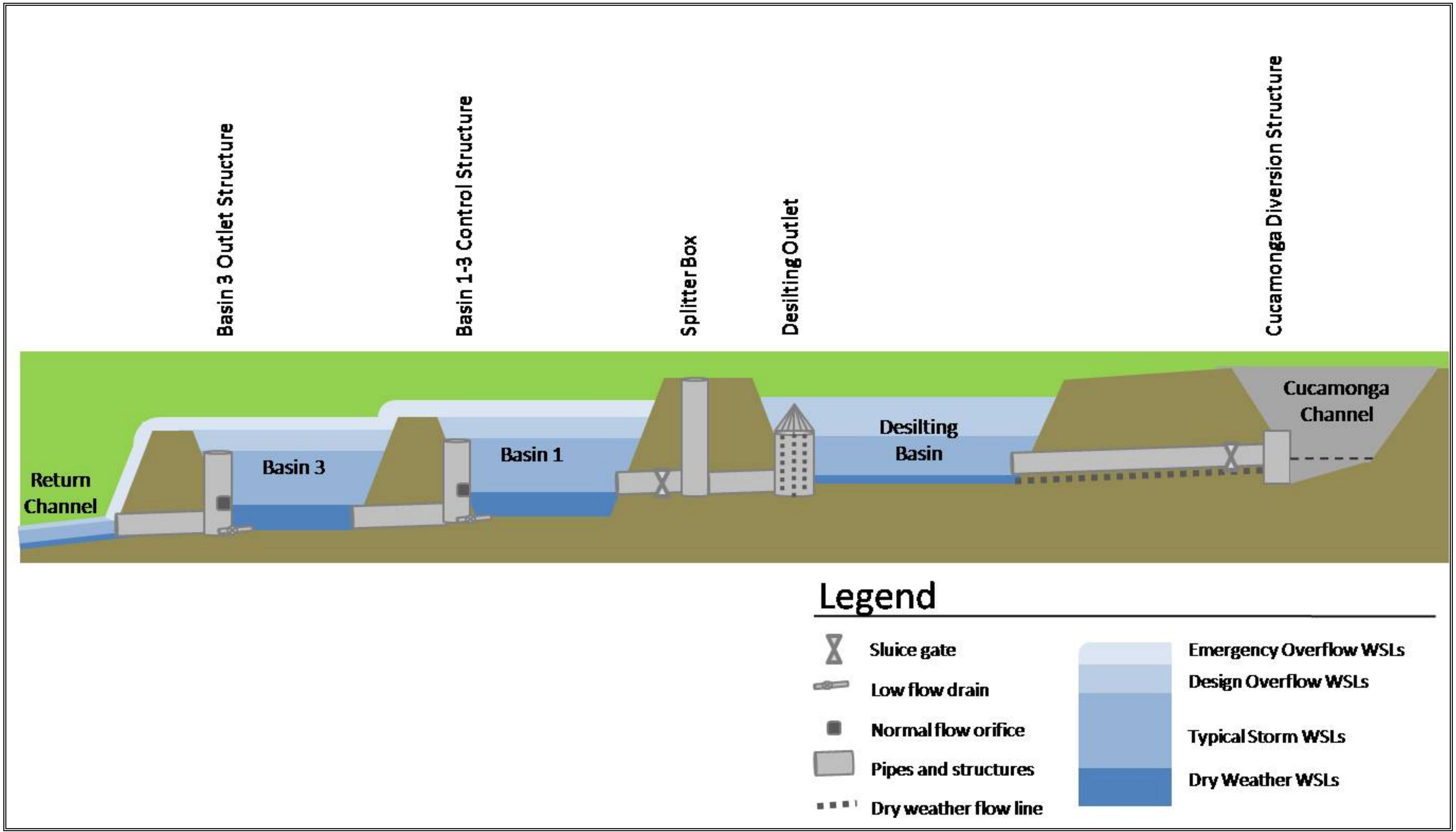

Figure 3: Illustration of operational regimes

Table 1 summarizes the diversion structure geometry currently under evaluation for design and the maximum diverted flows from Cucamonga Channel under such conditions.

Table 1. Preliminary Diversion Dimensions and Maximum Diverted Flowrate from Cucamonga Channel

Design Parameter		**Diverted Q (cfs)**
Weir width	(ft)	45
Weir Crest Height (above channel invert)	(in)	3.1
Conduit Width	(ft)	7.5
Conduit Height	(ft)	3
Low Flow Orifice Diameter	(in)	15
Q diverted from Cucamonga at 2-yr WSL (542.5ft)[1] (steady-state with basins full)	***(cfs)***	***105***
Q diverted from Cucamonga at 5-yr WSL (544.6 ft)[1] (steady-state with basins full)	***(cfs)***	***145***
Q diverted from Cucamonga at 100-yr WSL (552 ft)[1] (steady-state with basins full)	***(cfs)***	***249***
Q diverted from Cucamonga at 100-yr WSL (552 ft)[1] (with basins empty)	***(cfs)***	***269***

Criteria. The low flow diversion orifice will be sized to divert Q_{dw} under typical dry weather head conditions in the channel. Because dry weather head conditions in the channel fluctuate (see Existing and Proposed Surface Water Quality memo, Geosyntec 2008), the dry weather flowrate diverted to the system will range between 2.5 and 15 cfs. The design will be flexible to allow modifications to orifice plate dimensions allowing adjustment to operation in the future.

The wet-weather diversion system will be sized to divert an optimal flow regime to the basin chain such that the overall volume of wet-weather flow treated in the basin is maximized without resulting in frequent overflow events or the potential for very high flows through the basin. Conceptually, this involves the diversion of large fractions of small to medium storm flows, while minimizing the portion of large storms diverted. Because of the inter-related nature of the diversion weir, diversion line, and transient storage conditions in the basin chain, individual elements of the diversion design cannot be considered individually in attempting to meet this design goal. Rather, design requirements will be established iteratively through simulation of the integrated system with EPA SWMM making use of measured streamflow records, historic

rainfall records, and rainfall-runoff simulation. EPA SWMM is described in Section 4 and model files are attached.

The wet –weather diversion will allow adjustment to the rating curve in response to potential changes in channel flow-depth characteristics resulting from channel modifications. For example, diversion designs currently under evaluation have been sized to achieve objectives under existing conditions in Mill Creek and Cucamonga Channel. However, if the Chino Corona Road culverts were to be replaced in the future with a bridge, effective flow through the road crossing will likely alter the rating curve at the diversion. In this case, the invert elevations of the diversion structures would have to be modified to achieve capture objectives under the modified rating curve. It is anticipated that this will be accomplished through the provision of interchangeable weir boards or equivalent. This offers design flexibility to ensure that future projects influencing the rating curve in Cucamonga Creek at the diversion will not impact the Project's ability to meet objectives. Likewise, the Project will not hinder the success of such future Projects, if they should be planned. Interchangeable weir boards, if necessary will have to be manually installed and adjusted to accommodate any changes to channel flow conditions.

3.2.2 Minor Diversions from The Chino Preserve

Description. Stormwater runoff from The Chino Preserve will be diverted to the basin chain at locations along two storm drains planned to convey runoff from The Chino Preserve to Cucamonga Channel and Mill Creek. The first storm drain is referred to as the Hellman Road storm drain and will likely convey flows from The Chino Preserve out to Cucamonga Channel along Hellman Avenue. Water quality flows diverted from this storm drain will be routed through the Cucamonga diversion pipes into the desilting basin. Bypassed flows will continue out the planned storm drain along Hellman Avenue to Cucamonga Channel. The second storm drain conveying runoff from The Chino Preserve will likely discharge to Mill Creek at Chino Corona Road and is therefore referred to herein as the Chino Corona Road storm drain. Flows will be diverted from this storm drain directly into the desilting basin. Bypassed flows will continue out to Mill Creek.

These diversions are not design features of the Project, however their contribution to basin hydraulics and storage conditions must be considered in the analysis and design of the facility. The Chino Preserve diversion structures will be designed to divert all flows up to a specific cutoff flowrate. Once flows in the pipe reach the cutoff flowrate, a portion of the flow in the pipe will bypass the diversion. This is to ensure that only water quality sized storms will be diverted into the ponds. Flowrates to the Project from these diversion structures are expected to exceed the established cutoff flowrate as a result of high head conditions in the main pipes exerting greater head on the diversion line. The diverted flowrate with the mainline flowing full is referred to as the peak diversion flowrate.

Because designs are still preliminary for both of these diversions and their respective tributary watersheds, simplified representations were used in the SWMM simulation.

Criteria. Cutoff flowrates and peak flowrates have been established for these diversions using simplified methods (Table 2). These criteria will be incorporated into the SWMM simulation. It is expected that The Chino Preserve designers will eventually develop diversion designs to meet these criteria.

Table 2: Design criteria assumed for The Chino Preserve diversion structures

Diversion Line	Cutoff Flowrate, cfs	Peak Flowrate, cfs
Hellman Avenue	22	50
Chino-Corona Road	37	85

Diversion flowrates from The Chino Preserve diversion structures will be added to peak flows from Cucamonga Channel to calculate the design Q_{max} through the basin chain per Table 3, below. Peak flows with basins full are most appropriate for design of intra-basin structures that will only operate under these conditions. By contrast, peak flows with basins empty are most appropriate for design of the desilting basin.

Table 3: Total Design Peak Flows Through Basin Chain (Q_{max})

Basin Status	Total Q_{max}, cfs[1]
Full	384
Empty	404

1- Total Q_{max} = sum of Cucamonga and Preserve diversion flowrates; neglects backwater effects on Preserve diverted flowrates.

3.2.3 Desilting Basin and Desilting Outlet

Description. The desilting basin will receive flows from the Cucamonga diversion, the Hellman Avenue diversion, and the Chino-Corona Road diversion. The basin will have a gradually-sloping bottom and high-flow desilting outlet similar to a debris basin outlet. The intent of the desilting basin is to remove debris and coarse sediments from diverted flows while providing minimal hydraulic restriction to water entering the downstream basin chain. The desilting basin will drain completely following storm events.

Criteria. The shape of the desilting basin will be designed to prevent short-circuiting of flows and to maintain non-scouring velocities under normal peak flows up to Q_{max}.
The desilting outlet will be designed to reduce velocities under normal operating conditions to settle out gross solids but also to convey the Q_{max} under overflow conditions, and may include gross solids removal devices or trash capture systems.

3.2.4 Flow Splitter

Description. A flow-splitting structure (or equivalent parallel piping system) is proposed to split flows between the two basin chains downstream of the desilting basin. It is anticipated that this can be adequately achieved through a symmetrical structure. Small asymmetries in piping from the structure to the respective basins will be negligible in low flows (no capacity restriction), and the basins will be self-equalizing in high-flows.

Criteria. The flow-splitting structure will effectively split and equalize flow between the two halves of the basin. The structure and connecting pipes will be sized to pass appropriately split low flows, wet-weather treatment flows, and flood-condition flows.

3.2.5 Basins

Description. The Project includes four combined constructed wetland/extended detention basins (ponds) (see Figure 1) which, with the Desilting Basin provide a total system volume capacity of approximately 160 acre-feet.

The treatment wetlands will be constructed within the bottom of each extended detention basin. The constructed wetlands will be designed to look natural and meander through the base of each extended detention basin. As shown in Figure 1, they will not cover the entire bottom of each basin.

The extended detention basins will be graded to look natural and side slopes will undulate to simulate natural topography. Freeboard will be provided in all basins to protect against overtopping of embankments during emergency conditions. Freeboard is measured from the maximum water surface level under overflow conditions to the lowest part of the embankment, assuming horizontal water surface.

Basins 1 and 2 will have volume capacities of 41 and 45 acre-feet, respectively. Due to a sloping groundwater table, the basin design will include a grade transition to accommodate a 3 foot drop in invert within the basin. Wetland inverts will be 530 ft MSL and 527 ft MSL for the upper and lower portion of each basin respectively. Extended detention basin inverts will be 534 ft and 531 ft for the upper and lower portions respectively.

Basins 3 and 4 will have volume capacities of 34 and 32 acre-feet, respectively. The groundwater table in this area of the Project site slopes much more gradually and therefore, the invert of each basin is level at 529 ft MSL. Wetland inverts are 4 ft below the extended detention basin inverts at 525 ft MSL.

Criteria. The combined extended detention/constructed wetlands will be designed to detain at least 160 acre-feet of wet-weather runoff. Basin inverts have been set to allow for gravity flow through the system while maintaining a 3-5 foot separation between existing groundwater elevations. Design water depth for all extended detention basins is 9 feet (6 feet for higher portion of Basins 1 and 2) and for all wetlands is 4 ft. The invert of the constructed wetlands will be inset 4 ft below the invert of each extended detention basin. When at system capacity, the total depth in the deepest part of each basin will be 13 ft.

Minimum freeboard of 1 foot will be maintained under overflow conditions.

3.2.6 Perimeter Levees

Description. The pond system will be surrounded along the eastern edge by a perimeter levee that will range between 2 and 6 feet in elevation above existing grade at the exterior levee toe.

Criteria. The levee is intended to maintain design water surface elevations inside the ponds and to provide at least 2 feet of freeboard under normal operations and 1 foot of freeboard under overflow conditions. Design water surface elevations have been set at 540 ft MSL for Ponds 1 and 2 and 538 ft MSL for Ponds 3 and 4 in order to maintain gravity flow through the system while achieving volume sizing objectives.

The levee will be designed to prevent failure due to piping, blowout, levee-toe scour, erosion due to levee overtopping and scour of the creekside levee face. Related analyses will be conducted to determine levee design parameters. External levee protection will be provided using materials such as turf reinforcing mats or rip rap according to the permissible velocities for different materials and the conditions at the levee. Permissible velocities and shear stresses for various protection materials are discussed in the Analysis of Existing and Proposed Scour Potential and Protection Recommendations memo (Geosyntec, 2008).

3.2.7 Inter-basin Control Structures and Piping

Description. Key hydraulic control will be provided at two inter-basin control structures located between Basin 1 and 3 and Basins 2 and 4. Figure 4 illustrates the inter-basin control structures. These will include three main components:

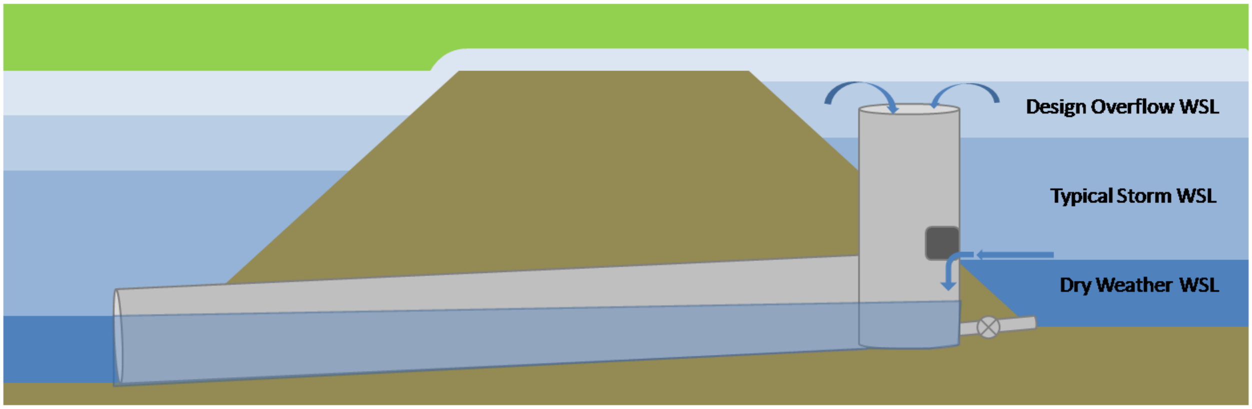

Figure 4: Schematic of inter-basin control structures under various conditions

Low-flow (Maintenance) Drains: A drain at the invert of the wetland permanent pool will allow the basins to be completely dewatered if necessary for maintenance purposes.

Normal Flow Orifices: An orifice/box weir will govern the dry weather water surface and restrict flow between basins during wet weather operations. The elevation of this orifice may be adjustable to provide flexibility in operation of the constructed wetlands. For example, it may be necessary to adjust weir elevations to maintain lower operating water levels during vegetation initiation. The incorporation of the ability to alter operating water levels in the constructed wetlands will facilitate proper functioning throughout all stages of wetland development. Under normal flow conditions, the orifice will restrict flow to promote the specified residence time. During normal storm flow, it is the intent of the design to maintain a drop in water surface elevation between the upstream and downstream basins, thereby maximizing active storage volume of the system.

Overflow Structures and Spillways: There will be inter-basin overflow structures and spillways in both Basins 1 and 2. Inter-basin overflow structures and spillways will connect Basin 1 to Basin 3 and Basin 2 to Basin 4. When incoming flows are greater than the flowrate through the normal flow orifices in Basins 1 and 2 or when the normal flow orifices are clogged, the water surface will rise in the upstream basins until reaching the crest elevation of an overflow riser or spillway or combination of the two. Above this elevation, the overflow structure and/or spillway will convey flows from Basin 1 to Basin 3 or Basin 2 to Basin 4. If a riser is designed, it can be constructed to have a variety of geometries with equivalent hydraulic performance.

At this time it is anticipated that peak flows will be conveyed between ponds with a combination of risers and spillway. Risers will handle more frequent overflow events to reduce the frequency of spillway discharge.

Criteria. Table 4 summarizes the design criteria for the inter-basin control structures. Dry-weather low flow drains will be designed to draw down the entire wetland volume in 1 day

assuming the downstream wetland is dry at the start of drawdown. Normal flow orifices will be designed to pass the Q_{dw} in less than 6 inches over the orifice invert. The orifice will likely behave as a weir in this range of depths.

Table 4. Inter-basin Control Structure Design Criteria

Component	Design Flowrate	Design Criteria
Low Flow Drains	$Vol_{wetland}/86400$	Draw down entire wetland volume in 1 day
Normal Flow Orifices	$0.5*Q_{ssmax}$	Differential head of 2 ft when water surface just below overflow stage; Q_{dw} in less than 6 inches over invert
Overflow Risers	Selected fraction of Q_{max} less than $0.5*Q_{max}$	Less than 1 foot of head over riser crest
Spillways	$0.5*Q_{max}$	Less than 1 foot of head over spillway crest (assumed normal flow orifice and riser are out of service)

Normal flow orifices will be designed to maintain approximately 2 feet of differential head between upstream and downstream basins when basins are just below the overflow stage. At this point, the flow through each half of the basin chain will equal half of the maximum drawdown flowrate, or $0.5*Q_{ssmax}$.

Overflow risers will be designed to pass a selected fraction of Q_{max} less than half of Q_{max} in less than 1 foot of head over the riser crest. The fraction will be selected to balance infrastructure requirements with the frequency of spillway discharge. Each overflow spillway (located in Basins 1 and 3) will be designed to pass half of Q_{max} in less than 1 foot of head over the spillway crest. This assumes that the normal flow orifice and overflow risers are not operable.

3.2.8 Outlet Structures

Description. Outlet structures in Basins 3 and 4 will provide the final hydraulic control in the basin chains. Each Basin will include three main components:

Low-flow (Maintenance) Drains: A drain at the invert of the wetland permanent pool will allow the basins to be completely dewatered if necessary for maintenance purposes.

Normal Flow Orifices: An orifice/box weir will govern the dry weather water surface and restrict discharge from the basins during wet weather operations. The elevation of this orifice may be adjustable to provide flexibility in operation of the constructed wetlands. For example, it may be necessary to adjust weir elevations to maintain lower operating water levels during vegetation initiation. The incorporation of the ability to alter operating water levels in the constructed wetlands will facilitate proper functioning throughout all stages of wetland development. Under normal flow conditions, the orifice will restrict flow to promote the specified residence time.

Overflow Structures and Spillways: When incoming flows are greater than the flowrate through the normal flow orifices or when the normal flow orifices are clogged, the water surface will rise in the upstream basins (Basins 3 and 4) until reaching the crest elevation of an overflow riser or spillway or a combination of the two. Above this elevation, the overflow structures and/or spillways will convey flows from Basins 2 and 4 to the outlet channel. If a riser is designed, it can be constructed to have a variety of geometries with equivalent hydraulic performance.

At this time it is anticipated that peak flows will be conveyed out of the basin system with a combination of risers and a spillway. Risers will handle more frequency overflow events to reduce the frequency of spillway discharge. In this case, recreational trails that line the perimeters of the ponds will have to be closed during periods of active flow over the spillway for safety purposes.

Criteria. Table 5 summarizes the design criteria for each outlet structure. Normal flow orifices will be designed to pass half of Q_{dw} in less than 6 inches over the orifice invert. The orifice will likely behave as a weir in this range of depths. Low flow drains will be designed to draw down the entire wetland volume in 1 day assuming the upstream wetland does not contribute during the drawdown period.

Normal flow orifices will be designed to ensure drawdown of the basin chain in approximately 48 hours assuming only design dry-weather inflow following a storm event. Table 6 tabulates the approximate design rating curve for each normal flow orifice.

Overflow risers will be designed to pass a selected fraction of Q_{max} less than half of Q_{max} in less than 1 foot of head over the riser crest. The fraction will be selected to balance infrastructure requirements with the frequency of spillway discharge. Each overflow spillway will be designed to pass half of Q_{max} in less than 1 foot of head over the spillway crest. This assumes that the normal flow orifice and overflow risers are not operable.

Table 5. Outlet Structure Design Criteria

Component	Design Flowrate	Design Criteria
Low Flow (Maintenance) Drains	$0.5*(Vol_{wetland}/86400)$	Draw down tributary wetland volume in 1 day when needed for maintenance
Normal Flow Orifices	See Table 6	Draw down of basin chain in 48 hours (assuming dry-weather inflow only after storm event)
Overflow Risers	Selected fraction of Q_{max} less than $0.5*Q_{max}$	Less than 1 foot of head over riser crest
Spillways	$0.5*Q_{max}$	Less than 1 foot of head over spillway crest (assumed normal flow orifice and riser are out of service)

Table 6. Normal Flow Orifice Rating Curve

Water Surface Elevation, ft MSL	Flow, cfs	Water Surface Elevation, ft MSL	Flow, cfs
529	0.0	534	16.6
529.5	1.4	534.5	17.6
530	4.1	535	18.5
530.5	7.2	535.5	19.3
531	9.2	536	20.2
531.5	10.8	536.5	20.9
532	12.2	537	21.7
532.5	13.4	537.5	22.4
533	14.6	538	23.1
533.5	15.7		

3.2.9 Outlet Channel

Description. The outlet (return) channel will convey flows from the outlet structures back to Mill Creek. A naturalized open channel with grade control and/or bank/bed stabilization is proposed for the return channel.

Criteria. The channel will be designed to safely pass Q_{max}.

The channel will be designed to be actively stable under the flow-duration regime expected from the facility. Flow-duration statistics will be generated from model output for use in design and evaluation of this feature.

The connection of the return channel to Mill Creek will be designed to minimize potential for adverse impacts. Protection of the channel at the confluence will be provided according to the permissible conditions for various materials summarized in the Analysis of Existing and Proposed Scour Potential and Protection Recommendations (Geosyntec, 2008). Channel protection will be designed to protect the channel from scour under the conditions which create the highest potential for scour due to the addition of outflows from the pond system to flows in Mill Creek. The HEC-RAS memorandum, by DMJM Harris (2008) provides more detailed information on the impacts to flow velocities in Mill Creek resulting from discharges from the pond system.

3.2.10 Sluice Gates

Description. Sluice gates or equivalent will be used to restrict flow and isolate facility components to support operational flexibility, emergency controls and maintenance activities.

As shown in Figure 5, proposed sluice gates will be located at the diversion from Cucamonga Channel, at the diversion from Hellman Avenue, at the diversion from Chino Corona Road, and at the inlets to Basins 1 and 2. Sluice gates at the diversion structures are provided primarily to allow maintenance of the entire basin system all at once. They may also be used to shut off or reduce inflows under emergency conditions where longer duration storm or runoff events fill the basins and continue to push high flowrates through the structures into the system, creating sustained overtopping conditions. Sluice gates at the diversion structures will be operated manually.

Sluice gates at the inlets to Basins 1 and 2 will be provided for maintenance purposes. The gates will allow for maintenance crews to shut off and dewater one side of the basin system while keeping the other side (the side not undergoing maintenance) online. Once finished with maintenance of one side, the sluice gate will be opened to bring both basins online again and

then the sluice gate at the inlet to the other set of basins will be closed to allow for dewatering and maintenance. The sluice gates at the Basin 1 and 2 inlets will be operated manually as well.

Criteria. Sluice gates will be designed with the ability to close completely and prevent water from passing.

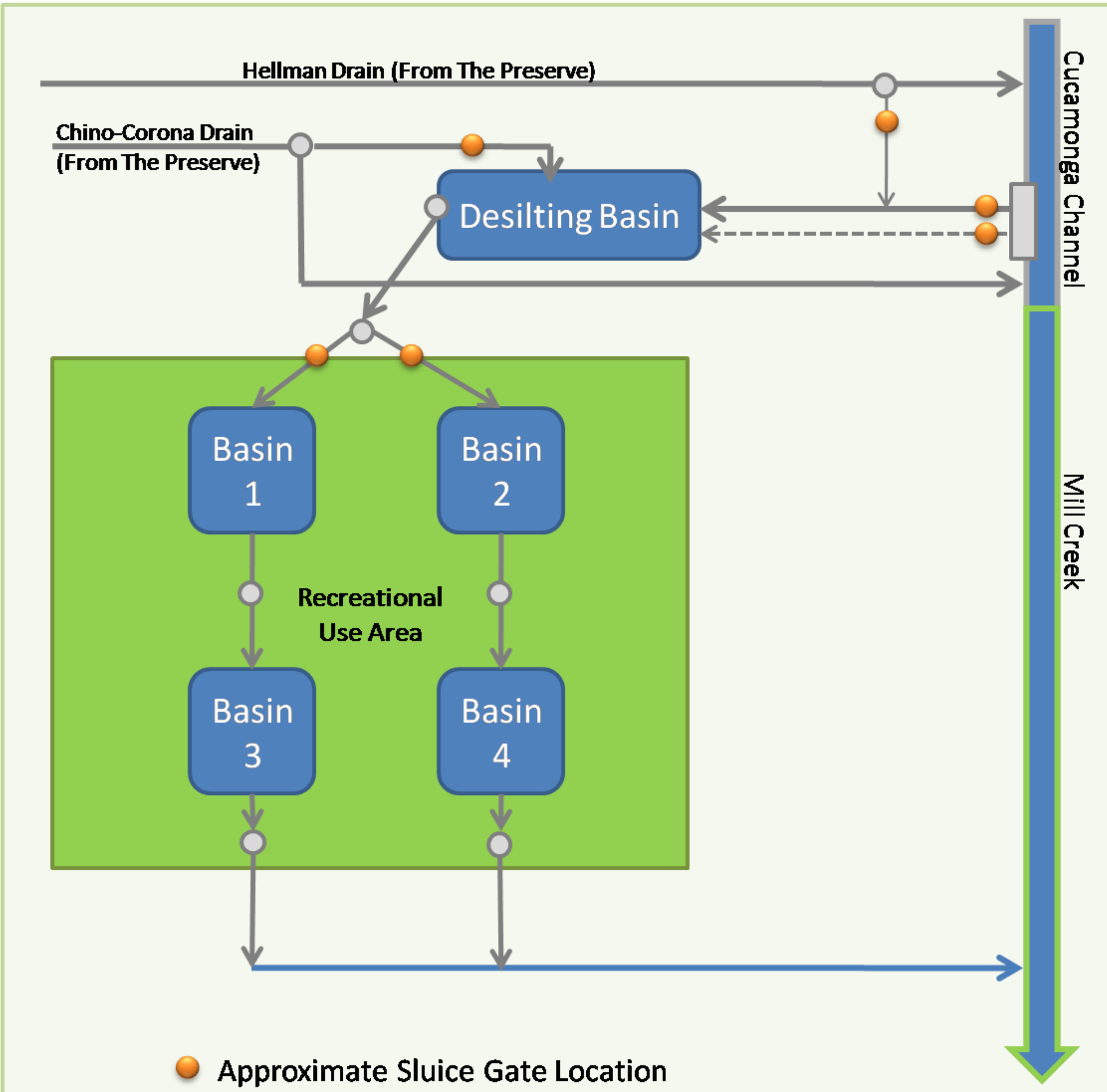

Figure 5: Tentative location of sluice gates

Sluice gates on diversion lines will be designed with variable settings to allow specified fractions of peak flows to be permitted into the facility. For example, partially closing the diversion sluice gates to reduce Q_{max} by 50 percent would allow one half of the basin chain to be shut down

temporarily without potentially exceeding the overflow capacity of the active half. Sluice gate settings meeting partial-flow requirements will be established through hydraulic analysis at a later stage in the design process.

3.2.11 Inlet and Outlet Protection

Description. Protection will be provided to prevent scour around inlets and outlets and where flow velocities may contribute to scour.

Criteria. Protection will be designed commensurate with the range of flows and conditions expected in the system, based on a frequency distribution of potentially-erosive flows generated from model output, and/or Q_{max}. Protective measures will be provided based on the permissible velocities and shear stresses of various materials discussed in Analysis of Existing and Proposed Scour Potential and Protection Recommendations (Geosyntec, 2008).

4 PROJECT PERFORMANCE

The design criteria discussed above have been determined based on preliminary basin optimization evaluations. Through such evaluations, modeled measures such as 1) percentage of watershed runoff treated and 2) percentage of diverted flows bypassed without treatment are used to refine design criteria and evaluate design performance. Basin performance is optimized in terms of treatment and design when a coupling of percent treated and percent bypassed is found that best reflects design and cost constraints. Based on the above design criteria, the Project will achieve the treatment of between 10% and 18% of all watershed wet-weather runoff while receiving a maximum flowrate of between 384 and 404 cfs and will bypass approximately 2% of flows diverted.

Estimates of basin performance and were obtained through the use of the Environmental Protection Agency (EPA) Stormwater Management Model (SWMM) Version 5. SWMM is a dynamic rainfall-runoff-routing simulation model used for single event or continuous simulation of runoff from primarily urban areas. In addition to runoff generation, the model contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage/treatment units and diversion structures (USEPA, 2008). SWMM was selected because of its proven capabilities in simulation of urban hydrology and facility hydraulics, and its flexibility in representing the proposed systems.

5 PRE-VS. POST PROJECT CONDITIONS

The existing Project site is an active floodplain with inundation regimes that vary with the different degrees of connectivity between the channel and floodplain. The Mill Creek Wetlands

HEC-RAS memorandum (DMJM Harris, 2008) discusses the inundation regimes of the existing and proposed floodplain in more detail. The Project will alter the hydrology of the site by creating off-line basins in the floodplain that receive diverted dry-weather flow consistently and wet-weather flow under storm runoff conditions in the channel. The Project will be designed based on the hydrologic and hydraulic criteria discussed above to take advantage of natural processes while minimizing operational and maintenance requirements and preventing potential for system failure.

Under dry-weather conditions, the flows along Cucamonga Channel and Mill Creek between the diversion location and the discharge location of the outlet channel will be reduced by between 2.5 to 15 cfs. The potential implications of this flow modification are discussed in the Mill Creek HEC-RAS memorandum (DMJM Harris, 2008). After system start-up, flows in Mill Creek downstream of the outlet channel will be minimally affected by losses in the system due to evapotranspiration. It is expected that even with the minor losses, flows in the Mill Creek downstream of the outlet channel will remain essentially unchanged.

Under wet-weather conditions, the in-channel flows adjacent to the Project will be reduced by up to 249 to 269 cfs. To evaluate potential changes to the flows downstream of the outlet due to diversion and 48-hour detention of wet-weather flows, a coupled time series of flows immediately downstream of the outlet were compared for existing and proposed project conditions. Measured flows from the USGS stream gage for 1998 (wet year) were run through the existing and proposed conditions SWMM models to obtain the coupled time series of flows. Out of 15 wet-weather events between 1000 and 9000 cfs, peak flows at the outlet were increased for 3 events by between 2% and 4%. For the remaining 12 wet-weather events greater than 1000 cfs in the channel, peak flows at the outlet were reduced by between 1% and 12%. For higher frequency in-channel flows, preliminary analyses indicate that durations of flows within the critical range for sediment transport will remain essentially unchanged. A more detailed discussion of potential for changes in flows due to the project to impact sediment transport processes is discussed in the Analysis of Existing and Proposed Scour Potential and Protection Recommendations (Geosyntec, 2008).

6 LIMITATIONS

The information presented in this document is conceptual and intended to identify key design features, describe the benefits and hydraulic performance of these features, and present information that can be used for environmental clearance efforts.

The information herein does not take the place of a Final Engineering Study, which will be required to confirm the validity of these conceptual analyses for final design and construction.

The design criteria cited within this document is not of enough detail/accuracy for final design or construction.

The conclusions contained in this investigation are based on preliminary design concepts and project objectives and developed with the Project team. No warranty, expressed or implied, is made regarding the professional opinions expressed in this report or concerning the completeness of the data presented to us.

Geosyntec is not liable for any use of the information contained in this report by persons other than for the stated project purpose.

7 REFERENCES

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