County of Marin

Santa Venetia Storm Drain Hydraulic Study

Final Report

January 2015

Executive summary

GHD was retained by the County of Marin through the Gallinas Watershed Program to develop a SWMM-type urban drainage model for the watersheds contributing to Flood Control Zone No. 7 of the Marin County Flood Control & Water Conservation District in Santa Venetia. The goal is to evaluate the current level of flood protection, to identify vulnerabilities in the current system, and to develop a range of practical and self-mitigating modifications to the drainage and pumping flood management system in order to improve the level of flood protection and increase the cost-effectiveness of the system. The model will continue to serve as a dynamic tool that can be used not only to help the County and District to design and prioritize future flood control and road improvement projects, but could be adapted by other agencies in the area. For example, the Las Gallinas Valley Sanitary District could use the model to evaluate pipe inflow and infiltration.

The identified projects were established by modeling specific improvements utilizing software (PC SWMM version 5.0.013 – 2.0.002) capable of coupling dynamic one-dimensional channel/storm drain hydraulic modeling with dynamic two-dimensional floodplain/street flooding hydraulic modeling. Flooding in this study is defined as overland free flowing and/or ponded storm water. It should be noted that this current study analyses flood protection improvements related only to the internal drainage features and does not address flood protection concerns related to the levee.

Sub-watershed boundaries were delineated, respective flows estimated within each sub-watershed, and flows routed into the existing storm drain networks and associated pump stations. Three separate design storm conditions were modeled: the 10-year, 50-year, and 100-year return interval rainfall events.

The modelling results indicated the following:

- Number of homes likely impacted by flooding:
  - 10-year flood - 15 homes
  - 50-year flood - 23 homes
  - 100-year flood – 27 homes
- Problems identified included:
  - Limited capacity in storm drain pipes
  - Sedimentation in gravity interceptors
  - Pump station limitations
- Some of the pump stations are not operating efficiently or do not have the necessary capacity to handle larger events. Pump Stations 2 and 3 are receiving less flow than previous analysis indicated. Previous analysis indicated significant upsizing needed for Pump Station 2. The current analysis indicates that the pump station capacity is likely adequate but it is recommended that improvements be made to improve reliability
- Both the pump stations and the gravity interceptors will continue to function with little change in provided level of flood protection assuming a sea level rise of 24-inches

Eight areas were identified for drainage improvements with costs per area ranging from $507,000 to $1,303,000. If all 8 improvements are constructed, the number of homes likely impacted during a 100-year event was reduced from 27 to 2. A detailed description of the improvements is located in Section 5 – Alternatives Evaluation.

Table ES-1 Santa Venetia – Potential Improvements

Area	Est. Cost	Description	Existing Flooding Problem	Proposed Improvements
1	$1,351K	Adrian Way, Palmera Way and Pump Station ("PS") 4 improvements. Drains to PS 1 & 4.	Flooding along Adrian, Palmera and LaPlaya Way up to 3 ft deep and covering an area of 120,000 sq ft. Both the pump station and storm drains lack the capacity to prevent flooding under 10-yr and 100-yr storms.	Replace 2 pumps at PS 4; Upsize storm drains on Adrian and Palmera Way (Figure 10)
2	$942K	Labrea Way, Rosal Way, and Galerita Way improvements. Drains to PS 1 & 5.	Flooding along LaBrea, Rosal and Galerita Way up to 1.2 ft deep and covering an area of 50,000 sq ft. Storm drains have a maximum flow capacity of the 2-year storm event.	Upsize storm drains on Labrea Way; Rosal Way; Galerita Way (Figure 11)
3	$1,303K	PS 5 WS: Estancia Ditch; Descanso Way; Hacienda Way; Rincon Way. Drains to PS 5.	Estancia Ditch has a flat, inconsistent slope; storm drains on Descanso, Hacienda and Rincon streets have a maximum flow capacity of up to the 10-year storm event.	Remove gravity bypass at PS 5; Regrade Estancia Ditch and install permeable bottom pavers; upsize storm drains (Figure 12)
4+6	$908K	Vendola Dr and Mabry Way improvements. Drains to PS 1, 2 & 3.	Flooding in street and yards along Vendola Dr near Santa Margarita island up to 2.25 ft deep for the 100-yr event and covering an area of 210,000 sq ft. Convoluted flow routing to PS 2 rather than nearby PS3 passing up to the 10-year storm event.	Upsize storm drains; construct cross-connection on Mabry. (Figure 13)
5	$761K	Vendola Dr, La Pasada Way and Galerita Way Improvements Drains to PS 1 & 2.	PS 2 is old and in need of significant rehabilitation currently passing up to the 10-year storm event	Upsize storm drains; construct cross-connect on Vendola between PS 1 & 2 (Figure 14)
6	See Area 4	See Area 4	See Area 4	Area 6 was addressed with Area 4 improvements (Figure 13)
7	$1,175K	Meadow Dr Interceptor Improvements Watershed does not drain to pump stations.	Interceptor capacity reduced by sediment from Las Gallinas Creek that is costly to remove; results in overland flooding across N San Pedro Rd (NSPR). Interceptor with assumed sedimentation passes up to the 2-year storm event.	Construct outlet riser box at Las Gallinas Creek and install catch basin gallery and 48" pipe on NSPR from N San Pedro Ct to Meadow Dr. Interceptor (Figure 15)
8	$507K	Cross connection on Vendola: PS 1, PS 3 with PS 2 off-line	PS 2 is old and in need of significant rehabilitation	Modify settings to rely less on PS2; add new 16" diameter/50hP pump at PS 3 and larger intake pipe (Figure 16)

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1. SUMMARY OF EXISTING STORM DRAIN SYSTEM

1.1 Introduction

Flood Control Zone No. 7 (Zone No. 7) of the Marin County Flood Control & Water Conservation District covers much of the unincorporated community of Santa Venetia, east of Hwy 101 along N. San Pedro Road (NSPR). The community was one of the first subdivisions in Marin County (the County) to be constructed on fill over bay mud and was built in an era before the County had the authority to regulate or control development. Due to the low initial elevation of the fill and the compressible nature of the underlying bay mud, the area has subsided and is now below the high tide level. To protect themselves from tidal flooding of Las Gallinas Creek, the residents of Santa Venetia formed Zone No. 7 in 1962. The annual maintenance program for facilities includes pump stations and levees, as well as other drainage facilities in the Zone. Though the Santa Venetia neighborhood did not experience significant flooding during the 2006 New Year's Eve storm, sea level rise, land subsidence, aging infrastructure and the costs of maintaining the stormwater pumping system remain key flood protection challenges for Zone No. 7.

GHD was retained by the County through the Gallinas Watershed Program to develop a SWMM-type urban drainage model for the entire watershed contributing to Zone No. 7. The first goal was to evaluate the current level of flood protection and identify areas of flow capacity limitations in the current system. The second goal was to develop a range of practical and self-mitigating modifications to the drainage and pumping flood management system with the potential to increase the level of flood protection and/or increase the cost-effectiveness of the system.

The study entails three tasks: 1) an evaluation of existing data and development of a field collection plan to obtain additional data required for hydraulic modeling of the storm drain system; 2) development of a calibrated model for the existing system; and 3) development and evaluation of a suite of alternatives for improvements to the existing collection and pumping system to increase cost effectiveness over a range of flow conditions. As a part of the study the alternatives are developed to a preliminary level of design detail to allow for analysis of impacts, constructability and cost-benefit estimating and include both capital and O&M costs. Further feasibility analysis and design, and possibly right-of-way acquisition and environmental compliance would need to be completed prior to carrying out construction of any alternative.

1.2 Existing Storm Drain Network

The Santa Venetia area storm drain network consists of three gravity interceptors that route storm water from the hills east of North San Pedro Road directly to Las Gallinas Creek and a local gravity storm drain network that is collected by five separate storm water pump stations and four portable pump sites. Figure 1 provides an overview of the watershed and Figure 2 provides a detailed storm network layout.

2. DATA COLLECTION

Technical data relating to the Study for Santa Venetia, Zone No. 7 was collected and reviewed. This information was summarized in *Technical Memorandum #1: Data Gap Analysis*. A summary of findings is included below.

2.1 Construction Documents and Studies

Data gathered for this model consisted of paper and electronic construction documents, studies, and maps. These documents were used to gain an overall understanding of the operation of the storm drain and runoff system and assign elevations and alignments for specific structures. Review of the documents resulted in an understanding of data gaps which is discussed in more detail below. Many of the studies collected are old and outdated with little relevant data. During the initial discussions, one of the options considered was incorporating the USACE Las Gallinas Creek Study and HEC-HMS model for developing watersheds and rainfall information. Review of the documents suggests that the USACE study was a much broader watershed study of Las Gallinas Creek and does not necessarily focus in detail on the Santa Venetia watershed. It was therefore decided to utilize the SWMM model for developing the hydrology. The USACE study runoff flows were used for comparison of the SWMM model output.

There are five permanent pump stations and four portable pumps in the watershed. The County provided a summary table of information, including missing data which was included in the data gap analysis. The pump station locations are included in Figure 2. The portable pumps are identified below:

at Meadow Way: Portable diesel pump with trailer, 6” Discharge, Purchased 1999

at 866/870 Estancia Way: Portable diesel pump, 3” Discharge, purchased 2013

at Estancia Levee Toe Drain: Portable gasoline pump, 4” Discharge, purchased 2010

at end of La Playa (not District or County owned or maintained)

The pump station as-builts in some instances provided pump set points by elevation. However, the County believes that these set points are no longer valid as maintenance crews have adjusted them to optimize system operations. The County building maintenance division has recorded modified set points within each pump station, defined by distances from the wet well bottom.

2.2 Marin County GIS

GIS data sets for this project were provided by Marin County or were available from public data servers (NOAA, Bing maps). GIS data was used as the primary source for the storm drain system layout and physical parameters. The County GIS data consisted of:

- General – City boundaries, parcel map, roads, streams
- Santa Venetia Stormwater specific:
  - Catch Basins – grated inlets, curb inlets, inlets with manholes
  - Structures – connectors, outlets, inlets, concrete boxes, trash racks
  - Manholes
  - Pipes- material, shape, size
  - Pump Stations – type (permanent automated or portable manual on/off)
- LiDAR Surface – County of Marin digital topographic-bathymetric surface model (Revision 2013.11)

2.3 Data Gaps

Based on the collection and review of the above data, data gaps were identified and a process developed for collection of the missing data including field site visits and surveys. In some cases, the necessary model information was not available within existing data provided. This data was referred to as "missing" data and included for certain facilities: pipe size, material, invert elevation, channel geometry, channel roughness, sub-watershed boundary, and pump station information. The actions taken to gather the missing information can be categorized as:

1. **Field or as-built plan verification.** These parameters may be gathered by visual inspection of the item in the field or on plans. These parameters include: materials (RCP, CMP, etc.), item type (circular, elliptical, box, irregular channel, etc.) channel roughness (develop Manning's n), pump station info.
2. **Field measurement.** This included going into the field and physically measuring the parameter with a tape or survey equipment. Items to be measured include: pipe diameter, rim elevation, invert elevation, or surface elevation.
3. **Interpolated or approximated.** This included interpolating or estimating a parameter value because it is not possible to measure. This predominantly occurs with blind connectors where the invert elevation cannot be measured.

3. EXISTING MODEL DEVELOPMENT

3.1 Model Approach Overview

The Santa Venetia storm drain system was modeled using PC SWMM 2013 Professional 2D, Version 5.4.1528 (SWMM version 5.0.013 – 2.0.002). The PC SWMM modeling package combines stormwater hydrology, collection, and conveyance simulation with a GIS/CAD interface. The hydrology and hydraulics computational engine employs the standard US EPA SWMM5 which is a link-node model for the 1D modeling combined with a proprietary 2D modeling grid developed by CHI.

The PC SWMM model uses a GIS data structure for model construction and data handling which makes it the ideal choice for the Santa Venetia storm drain project given the availability of Marin County GIS data. The overall approach for model construction is to use County GIS storm drain system data, existing studies, as-built and design drawings, and County maintenance records to create a PC SWMM model of the storm drain system.

The downstream boundary conditions for the storm drain system are approximated by a Mean Higher High Water (MHHW) tide cycle. A conservative scenario was developed, where the peak tide is offset so that it coincides with the peak runoff time. The offset time was determined by first running the model with a fixed downstream outlet water elevation held steady at the MHHW level. The offset time for MHHW tide cycle is the time from the beginning of the simulation to the peak arrival time. The lower portion of the tidal cycle was cut off with a minimum downstream boundary condition set at 4 feet (NAVD88). This was done to represent storm runoff at bank full flow in lower Las Gallinas Creek.

The runoff hydrology was calculated using PC SWMM's infiltration and overland flow modules, watershed delineation and precipitation data from NOAA Atlas 14, Volume 6, Version 2. A NRCS (SCS) type 1A 24-Hour dimensionless rainfall distribution was assumed for the design storm rainfall.

3.2 Model Parameters

This section outlines the recommended parameters and elements used in developing both an existing conditions model as well as proposed improvement models. A schematic of the system is shown in Figures 3A through 3F.

3.2.1 Drainage System Components

The PC SWMM model input data includes individual components that comprise the drainage system. In PC SWMM these components are grouped into several categories. The main categories used in this model are: conduits, junctions, outfalls, pumps and sub-catchments. Conduits are linear features that convey flow; they include: pipes, ditches, and channels. Junctions include: manholes, drop inlets, curb inlets, concrete boxes (with and without grates), blind connectors, culvert inlets/outlets, and trash racks.

3.2.1.1. Pipes

In PC SWMM pipes are defined as conduits. Locations of pipe conduits were assigned by the GIS shape files. The model GIS layer identifies a total of 467 pipe sections. Some pipe reaches have multiple sections because of blind connectors. The pipe size, shape, and material were assigned based upon the attributes recorded in the GIS shape files.

The invert elevations of pipe sections are defined in the model by the upstream and downstream end elevations. The pipe invert elevations were not identified in the GIS shapefile attributes but the upstream and downstream “cut depths” were provided. The “cut depth” is the distance measured from the surface to the pipe invert. Elevations at the endpoints of the pipe are calculated by subtracting the “cut depth” from the surface elevation. Surface elevations at the end points were determined for the end points associated junction elevation. The associated end point junction for the pipes was not explicitly defined in the GIS shapefile attributes. Junction elevations were taken from the County surface model value at the location of the junction. The association was made with a spatial join with the County surface model using ArcGIS.

3.2.1.2. Open Channel

In PC SWMM open flow channels are a type of conduit that is modeled similarly to pipes with irregular bottom geometries. The flow characteristics of channels are modeled using the slope of the channel, Manning’s roughness, and channel geometry. The channel alignments were based upon the GIS shapefiles. Channel slope is based upon the inlet and outlet junction elevations and is assumed to be constant between the junctions.

There are 86 channel segments represented in the GIS model layer. In some cases the channel geometry was included in the feature GIS attribute table; however most channel geometries were not defined and were either assumed based upon aerial images (or Google Street view) or field measured as part of the identified data gap efforts.

3.2.1.3. Junctions

In PC SWMM, junctions are nodes that connect conduits. Junctions are assigned from the County GIS features for storm drain: catch basin, manhole, and structures. Junctions include: catch basins, manholes, inlets, trash racks, culvert inlets/outlets, blind junctions, outfalls, and other. Outfalls are a special type of junction where the outflow from the junction is defined by an associated boundary condition. The model has 466 junctions. The elevations of junctions were not explicitly defined in the GIS shapefile attributes for most features. The association was made with a spatial join with the County surface model using ArcGIS.

Junctions have an inflow and an outflow. The inflow can come from a conduit (pipe or channel) or from overland flow from a sub catchment. Specific loss coefficient “K” values were assigned to junctions based on type: 0.5 for inlets and 1.0 for outlets.

3.2.1.4. Pumps

In PC SWMM pumps are a combined junction and conduit that is used to lift water to a higher elevation or hydraulic grade. The operation of the pumps was represented in the model as a Type 3 In-line pump where flow varies continuously with head difference between inlet and outlet (flow vs. head).

The County stormwater GIS features included nine pump station locations in the study area; five automated, and four portable. All nine pump stations are represented in the current model as Type 3 pumps because of the transient nature of the downstream boundary condition.

The five permanent pump stations were modeled based on a head/discharge curve based on current pump set points and pump curves. Pumping rules were incorporated into the model to start and stop pumping based upon pump on-off settings.

Only one of the four portable pumps, the portable pump located on Meadow Drive, was assumed to be operational in the existing conditions model as this provides a conservative estimate of flooding.

3.2.1.5. Sub-watershed ("subcatchment" in PC SWMM)

Sub-watersheds (or "subcatchments" in PC SWMM) are a key feature for developing the inflow to the storm drain system. The sub-watershed is an area that receives rainfall and concentrates the overland runoff flow into a point of concentration where the stormwater runoff enters the storm drain system.

The sub-watersheds are delineated using the ArcHydro tools in ArcGIS. The tool uses the ground surface model (LiDAR surface) and the junctions capable of receiving runoff storm water (catch basins, inlets, open or grated boxes, culvert inlets, and trash racks) to delineate sub-watersheds drainage areas. There are a total of 328 sub-watersheds in the model.

The automated ArcHydro sub-watershed delineation works well on the regions of the watershed with mild to steep terrain; however, it has difficulty delineating in the flat portions of watershed. Sub-watersheds in the flat portions were initially delineated using the ArcHydro tools but were adjusted based on evaluating the topography, aerial image, and Google Street View. Some basins required field verification and county review in order to finalize watershed delineation.

When delineating the sub-watersheds the longest flow path is also estimated. Again, the automated GIS ArcHydro tools work well on the regions of the watershed with mild to steep terrain and poorly in the flat regions. The longest flow paths in the flat region of the watershed were estimated as twice the distance from the sub-watershed pour point, usually the catch basin, to the sub-watershed centroid. The distance was checked and adjusted based on evaluating the topography, aerial image, and Google Street View.

The length of the longest flow path is used to estimate the effective flow width of the sub-watershed. The effective flow, defined by the length and width, is a key parameter used by PC SWMM to calculate the runoff of the sub-watershed. The estimate of flow width is calculated by dividing the sub-watershed area by the length of the longest flow path.

The slope of each sub-watershed is used by PC SWMM in the runoff calculation. The slope for each individual sub-watershed was assigned based upon the region of the site in which the sub-watershed resides. The flat regions of the site were assigned a slope of 0.0025. The slope for the steeper regions of the site varied by location and ranged from 0.35 to 0.02.

Groundwater interaction and evaporation loss calculation features associated with sub-watersheds were not included in the analyses. These effects were assumed to be negligible during major storm events.

3.2.2 Hydrologic/Hydraulic Components

PC SWMM calculates rainfall runoff by applying the rainfall to watersheds and routing the flow through the collection system. In the process, the model accounts for water infiltration and the overland flow timing. The following sections describe the methodology used for rainfall, overland flow routing, and infiltration.

3.2.2.1. Rainfall

In PC SWMM, rainfall is the primary source of water entering the system. The rainfall is represented as a hyetograph, which is referred to as “Rain Gages”. “Rain Gages” are assigned to each watershed and can vary from sub-watershed to sub-watershed.

For the Existing Conditions Model and Future Conditions Models the “Rain Gages” were populated with a SCS Type 1A 24-hour storm which is a predetermined rainfall distribution. The scale of the design storm is based upon the NOAA Atlas 14, Volume 6, Version 2, 24-hour storm depths that are fit to the Type IA storm. Design storms analyzed include a 10-year, 50-year and 100-year (all 24-hour storm durations). The NOAA Atlas 14 was used to create isohyetal regions at the site. The overall watershed was subdivided into three isohyetal regions. “Rain Gages” were created for various return periods for each of the isohyetal regions. The isohyetal regions are shown in Figure 4. Table 1 provides 24-hour rainfall depths used in development of the model. The rainfall drives the runoff for the project.

Table 1 24-Hour Rainfall Totals from NOAA Atlas 14

Region	10-yr 24-Hr Total (in)	50-yr 24-Hr Total (in)	100-yr 24-Hr Total (in)
Region 1	5.13	7.10	7.98
Region 2	5.29	7.31	8.23
Region 3	5.59	7.66	8.62

3.2.2.2. Routing

The routing of stormwater runoff starts with overland flow to the point of concentration where it enters the collection and drain system. In PC SWMM overland flow may be modeled as steady, kinematic wave, or dynamic wave. The dynamic wave method is used in this model. The parameters used in the dynamic wave are not explicitly declared, they are derived from the parameters defined in the sub-watershed. The modeling of overland flow uses Manning’s equation. The Manning’s roughness coefficient is specified for the pervious and impervious areas and is stored in the sub-watersheds attributes. Routing utilizes the effective flow width as described above in section 3.2.1.5.

3.2.2.3. Infiltration

In PC SWMM the stormwater runoff accounts for the loss of runoff flow due to soil infiltration. The model provides three options for estimating the portion of rainfall that is infiltrated: Horton, Green

Ampt, and SCS Curve Number. Each of the methods uses some or all of the following parameters: sub-watershed area, sub-watershed width, percent impervious, roughness coefficient for impervious area, roughness coefficient for pervious area, depth of storage of impervious area, and depth of storage pervious area. These parameters are stored in the in the sub-watersheds attributes.

The Curve Number method is used in this model. It is a widely used method that relates directly to known, readily available parameters including soil type and land use. It should be noted that the model input asks for pervious and impervious percentages. Thus in defining a representative curve number, CN, a value should be selected for both the pervious and impervious portions of the sub-watershed. In PC SWMM each subcatchment is treated as a reservoir with nonlinear runoff properties. In each sub catchment there is a portion of the “reservoir” that is filled prior to any runoff and is referred to as the depression storage. The value of the storage depth varies by land type: impervious, lawns, pastures, forest, etc. The impervious and pervious percentages were estimated for each sub-catchment.

3.2.3 Downstream Boundary Conditions

Outfalls are the locations where the modeled flow terminates and leaves the model. The flow conditions at the boundary are defined by the tidally influenced water level of Las Gallinas Creek, near San Pablo Bay. The tidal cycle time series boundary condition was applied to both Las Gallinas Creek and the marsh/wetland outfalls. This assumes that the tidal elevations in both are the same. If the water levels in the marsh system vary significantly from the Las Gallinas Creek – San Pablo Bay tidal time series a separate marsh/wetland boundary condition time series would need to be developed. Figure 5 shows a typical tidal cycle. The defined boundary condition in the model was based upon this tidal cycle with all the lower levels truncated at elevation 4 ft. This was done to conservatively represent the creek flow conditions expected during a typical rainfall event.

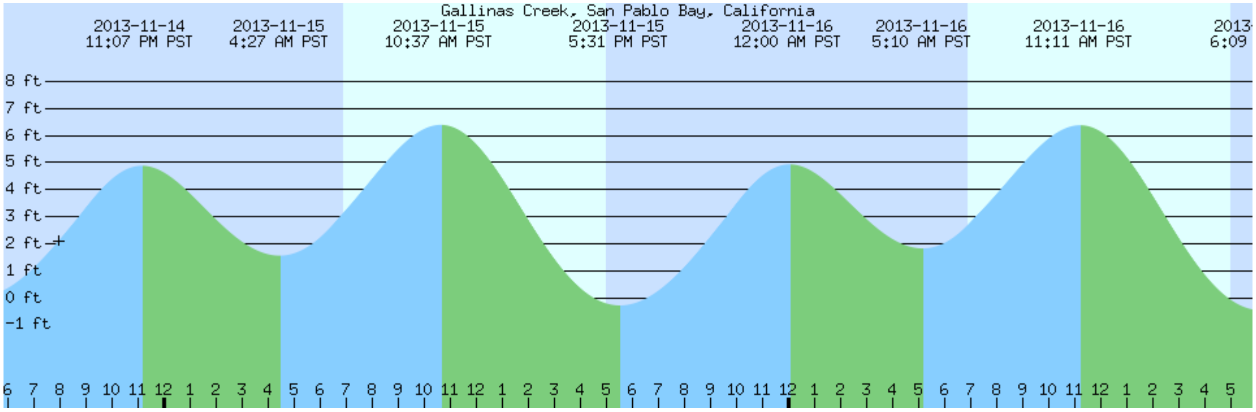

Figure 5 - Typical Tidal Cycle

3.2.4 Groundwater

Groundwater is generally assumed to have minimal impact in terms of added flow when compared to runoff from significant storm flows. For future reference, this was investigated as part of the sensitivity analysis.

Although ground water isn't assumed to directly contribute flows to the storm drain network, in areas of high groundwater, the watershed infiltration can be impacted which can contribute to higher runoff. The soil was modeled as saturated and this effect on runoff was accounted for by setting an

Antecedent Moisture Condition III for the 100-year storm event (highest saturation assumption), resulting in higher curve numbers and associated increased downstream runoff.

3.3 Sensitivity Analysis

Section 3.2 summarizes the PC SWMM model parameters being used in developing the Santa Venetia watershed model. There are several key parameters that are based on literature values that can have a legitimate range depending on conditions evident on site.

This section evaluates the various parameters to assess which parameters have the most impact on the system performance and the modeling results. The sensitivity analysis provides the modeler information on which parameters to modify should the modeling results not match field observations, allowing for some calibration of the model. A sensitivity analysis was conducted by selecting several key model parameters and modifying their values both higher and lower to see what impact changes in parameters have on both peak flows and volume. This analysis was completed for the 10-year storm routed through the 1D model. The overall watershed was subdivided into watersheds contributing to each outlet, either gravity or pump station. Refer to Figure 1 Watershed Map. The peak flow and volume as well as the differences based on parameter changes were developed at each downstream node. The following parameters were analyzed:

- Curve Number (CN)
- Overland roughness coefficients
- Flow length/width
- Storage depth (depression storage)
- Watershed slope

Of the parameters analyzed, the SCS CN value had the most significant impact on model results while modest changes to other parameters examined had relatively insignificant impacts. This indicates that the model is sensitive to the selection of soil type and perhaps antecedent moisture conditions since both of these factors are incorporated into the selection of the curve number. The detailed sensitivity analysis tables are located in *Technical Memorandum # 2: Watershed Model Parameter Development*.

3.4 Antecedent Soil Moisture Condition

The model allows selection of soil drying time after a storm, with a typical range of 2 days to 14 days. 14 days has been selected as a good representation of the hydrologic soil types on site which tend towards clays with poor infiltration capacity (hydrologic soil classification C/D). This would only impact continuous simulation model runs which currently are not being considered.

Another significant impact on runoff is antecedent moisture condition (AMC) which represents soil moisture at the time of the event. There are three classifications: AMC I, II and III. AMC I represent soils that are dry, AMC II represents "average" soil conditions and AMC III represents soils that have received relatively heavy rainfall and are saturated. Based on discussions with county staff, AMC II was assumed for the 10-year and 50-year storm and AMC III was assumed for the 100-year storm. This is because, historically, 100-year runoff often occurs during long periods of rain that saturate the ground. Increases in the curve number (CN) were input into the model to reflect AMC III conditions. Model results indicate an average increase in runoff for all the watersheds of approximately 29% by assuming AMC III conditions.

3.5 Sedimentation Evaluation

The Santa Venetia storm drain system tends to accumulate sediment in the direct gravity outfalls to the creek. There are three gravity interceptors; Meadow Drive, La Pasada and Sunny Oaks. The La Pasada interceptor was recently improved by slip-lining the last 260 feet of 48-ich storm drain with a 27-inch plastic pipe which was extended an additional 10 feet into the channel and equipped with a tide gate. The Sunny Oaks interceptor is also equipped with a tide gate. Thus only the Meadow Drive interceptor was analyzed for sediment impacts.

The Meadow Drive interceptor consists of a 7' x 4' Reinforced Box Culvert (RCB). Two and a half feet depth of sediment was added to the first 700 feet of the RCB in the model based on estimated condition from observations at the outfall.

3.6 Calibration

Except for local residents and maintenance crew accounts of flooding, there was no available data to perform a calibration of the model. However, model results appear to bear out accounts on where flooding has occurred.

4. EXISTING CONDITIONS MODEL RESULTS

The existing conditions model was run for three separate flood recurrence intervals: a 10-year interval (at an antecedent moisture condition [AMC] of II), a 50-year interval (AMCII) and a 100-year interval (AMCIII). Figures 6 through 8 below highlight all junctions where flooding is indicated under the 1D model run and Figures 6A through 8A show overland flooding results from the 2-D model run. The figures identify homes most vulnerable to flood damage based on available door sill elevation surveys supplied by the County. Note that these elevations for damage assessments are based on informal survey elevations of front doors collected by County non-surveying staff in 2008 of many, but not all, houses in the project area where access was provided. This assessment was for general modeling and alternatives development and is not intended to be a definitive survey of each property. Therefore, regardless of the model results, there may still be damage to structures at elevations below the critical elevation used for this model due to a variety of factors including different storm event patterns and intensities, survey and data

4.1 10-year Storm

Figure 6 is an overall site map showing maximum flooding based on the 1D model results for the 24-hour, 10-year return frequency flood event. Figure 6A is an overall site map showing maximum flooding based on the 2D model results for the 24-hour, 10-year return frequency flood event. The 10-year flood results indicate that there are 15 likely structures to be impacted by flooding.

4.2 50-year Storm

Figure 7 is an overall site map showing maximum flooding based on the 1D model results for the 24-hour, 50-year return frequency flood event. Figure 7A is an overall site map showing maximum flooding based on the 2D model results for the 24-hour, 50-year return frequency flood event. The 50-year flood results indicate that there are 23 likely structures to be impacted by flooding.

4.3 100-year Storm

Figure 8 is an overall site map showing maximum flooding based on the 1D model results for the 24-hour, 100-year return frequency flood event. Figure 8A is an overall site map showing maximum flooding based on the 2D model results for the 24-hour, 100-year return frequency flood event. The 100-year flood results indicate that there are 27 likely structures to be impacted by flooding.

4.4 Identified Problem Areas – Storm Drains and Channels

Based on review of the flooding indicated in the maps referenced above and review of model output, certain problem areas were identified. The red stars on figures 6 through 8 show where at least some water is simulated as flowing out of structures and obvious clusters were reviewed during the alternatives investigation. During a simulated 2-year event, the flooding elevation does not exceed estimated sill elevations in any structures. However, 15 structures have sill elevations below simulated adjacent water surface elevations in a 10-year event. This is also the case for 23 homes in a 50-year event and 27 homes flood in a 100-year event.

All three interceptor drains show the hydraulic grade line (HGL) [which simulates the modelled water surface elevation] exceeding rim elevations at certain junctions. This is expected because they were designed to flow under pressure. Flooding does not occur at these junctions as long as the manhole lids are properly bolted down. However, this causes a significant rise in the HGL that has some impact on storage upstream in the system.

Addressing sediment in the Meadow Drive interceptor would greatly reduce any flooding threat related to the main. With its manholes bolted, the flooding related to this spill may not be an issue but there does appear to be an impact to upstream detention and allowable flows to the system.

The HGL in the slip-lined La Pasada interceptor closely matches that of the pre-existing 48-inch pipe if 2.5-feet of sediment were assumed to impact the 48-inch storm drain - a condition to which in 2011 the pipe returned within 6 months of sediment removal in the lower reaches of the stormdrain. The slip-lined drain is expected to experience less sedimentation.

Flooding is simulated to occur on Estancia Way and is impacted by backwater in the levee perimeter channel that routes to Pump Station 5. This has been reviewed with some preliminary solutions in a previous study (Flood Control Zone 7 Flood Improvement Project; Marin County – Estancia Way; Winzler & Kelly; March 5, 2008). The backwater does not appear to be directly related to Pump Station 5 capacity and is more related to perimeter ditch capacity easterly of the Pump Station and likely capacity in the street drain and gutter flow-line.

4.5 Identified Problem Areas – Pump Stations

Several of the pump stations (PS) are not operating at optimal efficiency. Table 2 provides information on all pumps including pump flow and volume, number of starts and operation time for the 10-year flood event. At higher storm events the pumps operated with less cycling due to increased runoff flow volume.

Table 2 10 Year Storm Pump Station Operations

Pump Name	Utilized %	# of Starts	Average Flow (cfs)	Max Flow (cfs)	Total Flow (MG)
	29.47	28	1.58	1.58	0.125
SVPS1-1	43.8	19	23.6	25.56	2.783
SVPS1-2	5.77	3	22.96	24.66	0.357
SVPS1-3	0	0	0	0	0
SVPS1-4	0	0	0	0	0
SVPS2-1	74.96	3	4.94	5.05	0.996
SVPS2-2	25.22	2	14.55	15.1	0.988
SVPS2-3	3.43	1	22.91	24.55	0.212
SVPS3-1	38.55	23	13.28	14.12	1.379
SVPS3-2	1.2	4	27.77	29.2	0.09
SVPS4-1	34.51	18	4.9	5.11	0.455
SVPS4-2	4.32	3	4.84	4.94	0.056
SVPS5-1	26.55	1	15.47	16.53	1.106
SVPS5-2	0	0	0	0	0
SVPS5-3	0	0	0	0	0

4.6 Pump Station Results

Pump Station 1 has three equally sized pumps, pumping to a common manifold plus a smaller sump pump. Table 2 shows that pump 1 is cycling much more than is acceptable and one pump does not even start during a 10-year event. An existing small sump pump was added in the latest modeling results and this did reduce the pump starts but they are still high. Possible solutions may be to replace one of these main pumps with a second smaller pump or modify the existing sump pump to accommodate a wider range of flows.

Based upon the simulated flooding and operational run times of the pumps, Pump Station 2 has adequate capacity for at least 10-year storm events. Prior studies indicate a capacity of less than a 10-year event based on an assumed a larger contributing watershed and a recommendation for upsizing of the pump station. Based on the current available topographic data, the contributing watershed is smaller than originally estimated. The pump station could still benefit by upsizing pumping capacity to handle extreme storm events and there is other significant work required to bring it up to current standards. One pump was recently replaced, but the pump station discharge pipe and generator are quite old and much of the electrical system requires upgrading.

Pump Station 3 is modeled with approximate curves representing two different sized pumps pumping to a common manifold. The results of the modeling (Table 2) shows that the smaller pump is turning on and off (cycling) excessively while the larger pump does not come on at all. This situation was evaluated during the alternatives analysis described in Section 5.

Pump Station 4 is again represented by approximate pump curves for two pumps of equal size pumping to a common standpipe/gravity system. The actual pump curves for this pump station were not available. Approximate pump curves were provided by the pump manufacturer (Meyers Pump) for a similar model pump. The first pump appears to be over-cycling while the second pump only operates for a short period of time

Pump Station 5 consists of three equally sized pumps and does not appear to be utilized efficiently as only one pump comes on and only for a short duration. The pump on/off settings are set high allowing only minimal storage in the wet well, and should be examined to see if this is a contributing factor Pump Station and to determine if lowering wet well pump on/off settings could possibly decrease the potential flooding in the area.

Preliminary review of the pump results for the simulated 100-year flood event show a lessening of cycling which might be expected as the pumps are running for a much longer period of time to keep up with the higher flows. However, even for the larger event, Pump Station 3 is still showing significant cycling from both pumps.

5. ALTERNATIVES EVALUATION

5.1 Identified Areas for Analysis

The results of the existing conditions modelling were used to identify potential areas of flooding for further investigation during the alternatives development phase of the study (Figure 9). Eight areas were identified based on estimated flood risk. In general, the areas are listed based on the number of structures simulated as likely vulnerable to flood damage, with the areas with most vulnerable structures listed first. This does not mean that that the County has performed any ranking of alternatives and does not make any commitment to implementing alternatives according to this listing.

Potential improvements that call for upsizing the existing storm drain system should include replacing the catch basins associated with the improved pipe network. The County is currently replacing some of the old catch basins in the area and the need to replace any catch basins as part of the overall improvement project should be explored in more detail during the future design phases of the project. Figures providing more detail on the potential improvements are referenced below.

5.1.1 Area 1

5.1.1.1 Nature of Flooding

Area 1 is located along Vendola Drive and Adrian Way, extending south along Adrian Way to the intersection with Estancia Way. It is part of the Pump Station 4 watershed (i.e. its flows drain to Pump Station 4) although the simulated flooding in the area appears to be due to limited capacity in the storm drainage network that routes both to Pump Station 4 and Pump Station 1. The modelling shows this site as having the most number of homes potentially impacted by flooding based on existing estimates of flooding and structure door sill elevations. The increased risk of flooding in this project area appears to be due to the fact that the end of La Playa and Adrian Way are in depressed (subsided) areas where ground elevations are lower and form ponds during storm events that cannot be drained without the addition of new drainage inlets which may be best sited on private property.

The pumps at Pump Station 4 are not able to keep up with storm flows routed to them during greater than a 10 year recurrence interval events. In addition, the pump on/off settings are set relatively high, exacerbating backwater into the street drains which may cause overtopping in the streets. The upper limits of the storm drain network that routes to Pump Station 1 are not sized to carry larger peak storm events and also overtop in the model, causing overland flow to collect in the low lying areas.

5.1.1.2 Alternatives Investigated

The alternatives investigated include:

- upsizing pipes,
- cross connecting to Pump Station 1 watershed,
- new storm drain along the levee to pick up overland flow;
- modifications to Pump Station 4.

Routing flows through a pipeline along the levee is considered infeasible due to anticipated impacts to the levee and immediately adjacent private properties.

5.1.1.3 Most Effective Alternative

The most effective alternative, as shown on Figure 10, includes several combined improvements:

- as an interim measure, it is recommended that the pump start settings be adjusted to a lower starting wet well depth in order to turn on sooner as this will help to alleviate flooding. At least one pump could be set for minimum recommended surcharge elevation. This will need to be done on a "try and see" basis because the impacts of ground water intrusion at these lower settings are unclear.
- install two larger pumps capable of pumping approximately 2200 gpm, with lower pump on settings. This likely will require upgrades to the power service and wet well,
- upsize upper pipe network to Pump Station 4,
- upsize upper pipe network located southerly on Adrian that routes to Pump Station 1.

5.1.2 Area 2

5.1.2.1 Nature of Flooding

Area 2 is located on Rosal Way south of the intersection with Labrea Way and is part of the Pump Station 5 watershed. This area also includes Galerita Way which routes to Pump Station 1. Flooding at this site may threaten several homes and appears to be related to limited capacity in storm drains that route to both Pump Station 5 and Pump Station 1. Flooding in this area can be mitigated by system improvements to the storm drain conveyance system to bring more flows to Pump Station 1 and Pump Station 5 and by improvements to Pump Station 5.

The storm drain system on Labrea and Rosal that routes via Vendola Drive to Pump Station 5 has limited capacity. The existing storm drain connection from Labrea to Vendola is via an 18-inch pipe that has reverse grade through two catch basins.

The pump on/off settings at Pump Station 5 are high, causing an upstream backwater effect.

The pump station operates year-round due to drainage of groundwater through Estancia ditch and through a tide gate that doesn’t seal properly in a gravity bypass line adjacent to Pump Station 5.

The storm drain system on Galerita Way that routes south to Pump Station 1 has a limited flow capacity which could impact flooding of homes on Galerita Way during high recurrence interval storm events (greater than the 10 year event).

5.1.2.2 Alternatives Investigated

- upsizing pipes,

- adding a new pipe section (Labrea Way and Vendola),
- additional cross-connection to Pump Station 1 watershed,
- modifications to Pump Station 5.

5.1.2.3 Most Effective Alternative

The most effective alternative, shown on Figure 11, includes several improvements that together reduce the flooding threat:

- a more direct 24-inch connection to the storm drain on Vendola (potential utility conflicts should be explored before any final alternative is carried forward),
- abandon gravity system adjacent to Pump Station 5,
- upsize pipes on Labrea Way, Rosal Way, and Galerita Way,
- lower the pump on settings. Note: groundwater contributions to drainage need to be assessed prior to determining new pump settings.

5.1.3 Area 3

5.1.3.1 Nature of Flooding

Area 3 includes portions of Estancia Way, Rosal Way, Descanso Way and Hacienda Way; all of which drain to the levee toe drain (Estancia Ditch) that drains to Pump Station 5. Simulated flooding is caused by a combination of low gradients, low surface elevations, limited pipe capacity, and limited capacity in Estancia Ditch due to flat grades and sedimentation which, over time, have reduced the ditch cross sectional area.

5.1.3.2 Alternatives Investigated

- installing larger and or new storm drains,
- re-grading of the levee channel perhaps with a hard flow-line surface allowing for maintenance of grade,
- lowering the channel invert at Pump Station 5 allowing for a steeper channel slope upstream,
- modify Pump Station 5 pump settings.

5.1.3.3 Most Effective Alternative

The most effective alternative, as shown on Figure 12, includes:

- upsize and re-grade the Estancia ditch and line the ditch with a hardscape (possibly permeable if groundwater interaction allows). It is not recommended to lower the elevation of the Estancia Ditch due to possibility of groundwater interactions and concerns over levee stability. A previous study recommended protection to the levee toe adjacent to Estancia Ditch and seeded geo-fabric, or stronger method of stabilization, and this should be part of any ditch improvements.
- upsize pipes on Descanso Way, Hacienda Way and Rincon Way.

Pump Station 5 settings should not be modified without further study of groundwater interactions.

5.1.4 Area 4

5.1.4.1 Nature of Flooding

Area 4 includes simulated flooding along Vendola Drive northwest of Meadow Drive. This site drains to Pump Station 3 but the storm drain connection for flow to the pump station is inefficient and may exacerbate flooding. The existing drainage connection is a very flat and circuitous route. The model indicates that Pump Station 3 is not fully utilized as much of the flows within the historic watershed boundaries are not reaching the pump station. The hydraulic conveyance capacity of the storm drain system in the area is also somewhat limited and can be improved as described below.

5.1.4.2 Alternatives Investigated

- installing a new storm drain connection west on Vendola, tying to the Pump Station 3 trunk main,
- rerouting some of the flows south to Meadow Drive to the existing portable pump site. This option may require reanalysis of the portable pump capacity and potentially installing additional storage/wet-well,
- cross-connect between the proposed County storm drain on Mabry Way (flowing to Pump Station 3) to existing storm drain pipe at Mabry and Rafael (flowing south to Pump Station 2). (Refer to Area 6 below).

The first two alternatives identified above were looked at and considered more costly with likely serious existing utility conflicts when compared to the most effective project alternative. The existing portable pump at Meadow Drive would have needed to be upsized and impacts to existing utilities and properties would likely have been significant as well as increasing operations and maintenance costs for a new, larger pump station.

5.1.4.3 Most Effective Alternative

The most effective alternative is shown in Figure 13 and includes the following improvements:

- 15-inch cross-connect from the proposed County storm drain on Mabry Way to the intersection with Rafael Way where an existing 21-inch pipeline flows south towards Adrian Way. (Note: this cross-connect was constructed during the course of this study).
- increase the pipe size along Vendola Drive

5.1.5 Area 5

5.1.5.1 Nature of Flooding

Area 5 encompasses La Pasada Way from the intersection with Vendola Drive south to Galerita Way located within both the Pump Station 1 and Pump Station 2 watersheds. Flooding in this reach is mainly due to limited capacity in the storm drains and an effective solution could be to upsize portions of the existing pipe network. It should be noted that the simulated flooding in this area appears to mainly stay in streets and yards, though it could impact homes.

5.1.5.2 Alternatives Investigated

- examining the flow characteristics of the drainage system to Pump Station 1 and Pump Station 2 to identify potential benefits of pipe upsizing,
- pump station modifications,

- cross-connect along Vendola between Pump Station 1 and Pump Station 2 systems,
- Area (Alternative) 8, consolidation of pump stations to eliminate Pump Station 2, also impacts this site.

5.1.5.3 Most Effective Alternative

The most effective alternative includes several improvements that together could improve drainage. These improvements are shown on Figure 14 and include:

- upsizing the pipes on Vendola both east and west of La Pasada Way,
- installing a cross-connect on Vendola at La Pasada Way between Pump Station 1 and Pump Station 2 and upsizing storm drains on La Pasada Way that tie to Vendola. There is potential for utility conflicts that should be explored before any final alternative is carried forward.

In the existing conditions model, the upper catch basins on the storm drain system on La Pasada Way that routes south and then west on Adrian Way show backwater flooding into the streets potentially contributing overland flows off-site. There is no apparent cost-effective solution to fully address this flooding threat.

5.1.6 Area 6

5.1.6.1 Nature of Flooding

Area 6 was identified to examine extensive simulated overland flow and ponding in the street and yards on Mabry Way, part of Pump Station 3 watershed. At the time that the initial modelling for this study took place, there was no storm drain inlet in this street to collect runoff. The simulated flooding appears to be mainly nuisance flooding as the water depth did not appear to be excessive relative to home elevations; however street access could be severely hindered during high recurrence interval events.

5.1.6.2 Alternatives Investigated

The improvements investigated to address flooding on Mabry Way are discussed above under site 4 in section 5.4.4.

5.1.6.3 Most Effective Alternative

The most effective improvements are shown on Figure 13. Since the analysis took place, the County installed a 15-inch storm drain on Mabry Way connecting to the storm drain on Vendola Drive that routes flow to Pump Station 3. As recommended, they also installed a cross-connection pipe from this new storm drain to the storm drain routed southerly on Mabry Way. Prior to this construction, the flow path routed flows south on Mabry via an inefficient, circuitous pathway to Pump Station 2. This new stormdrain system will allow flow to travel both north and south on Mabry with much of the flow routing to Pump Station 3 where there is extra capacity and hence a more cost-effective use of the existing pumping system. Construction was performed as part of a road paving project covering much of the Zone. This new storm drain was modeled in PC SWMM during this study.

5.1.7 Area 7

5.1.7.1 Nature of Flooding

Area 7 includes the Meadow Drive Interceptor. Simulated flooding occurs at the upstream end of the Meadow Drive interceptor at the intersection of Meadow Drive and North San Pedro Road. The model reveals two potential reasons for flooding at this location. The flows coming off the steep hillside west of North San Pedro Road exceed the drainage facility capacity and flow as overland flow to the inboard easterly channel along North San Pedro Road. In addition, due to the modeled sediment build up in the Meadow Drive Interceptor and the fact that the manholes for this system are bolted; the hydraulic grade line exceeds the ground elevation at North San Pedro so some flows during high recurrence interval storms cannot enter the storm drain and flow instead into the street.

5.1.7.2 Alternatives Investigated

- new pump station,
- dredging outfall of Interceptor and clearing drain,
- new outfall riser,
- upsizing storm drains near N. San Pedro Rd.

5.1.7.3 Most Effective Alternative

The most effective improvement, as shown on Figure 15, includes:

- construction of a new outfall riser ("bubble-up") at the outlet of the Meadow Drive Interceptor built to a higher elevation to limit sediment intrusion from Las Gallinas Creek from entering and clogging the pipe. This will create an elevated downstream boundary condition because water remains in the riser, but will inhibit sediment clogging of the pipe which requires expensive maintenance. This outlet structure would have a grated access cover to allow removal of any accumulated sediment as well as to allow dewatering of the system with a pump.
- install a series of catch basins at the cul-de-sac on N San Pedro Court,
- new 48-inch storm drain on N. San Pedro Road that routes back to the Meadow Drive Interceptor. It may be possible to instead tie directly to the existing 48-inch storm drain that crosses N San Pedro Court just south of the cul-de-sac that ties to the Meadow Drive Interceptor west of North San Pedro. In order to determine if this is feasible, the County would need to verify the existing 48-inch pipe location and depth.

5.1.8 Area 8

5.1.8.1 Nature of Flooding

The County has requested as part of the alternatives analysis to look at opportunities for pump station consolidation. From past studies, it is evident that Pump Station 2 needs significant upgrades due to the age of the system and worker safety concerns. A design was completed for a new pump station, but due to funding shortages it was never constructed. The results of this study have shown that the contributing watershed to Pump Station 2 is somewhat smaller than previous studies had indicated and therefore the required flood flow is less than previously estimated and the pump station does not need upsizing. However, there is currently a cross connection between Pump Station 2 and Pump Station 3 which is a relatively new pump station that appears to have excess capacity and therefore, this may provide an opportunity to consolidate these pump stations.

5.1.8.2 Alternatives Investigated

- take Pump Station 2 offline
- install new pump at Pump Station 3
- install cross-connect between Pump Stations 2 and 1 along Vendola Dr. across La Pasada Way
- upsizing storm drains on Vendola Dr. that connect Pump Stations 2 and 3

5.1.8.3 Most Effective Alternative

The Marin County Flood Control & Water Conservation District should not take Pump Station 2 offline, as shown in Figure 8, unless all of the above investigated alternatives are implemented. Before selecting this option utility conflicts must be investigated and costs of alternatives evaluated (such as improvements to pump station 2). It may be more cost-effective to leave the pump station operational with improvements to the generator, electrical components and outfall pipe.

In the current alternatives model, Pump Station 2 was taken offline and a third pump was added at Pump Station 3 where there is an existing space for this future pump, in order to compensate for this loss of pumping capacity at PS #2. In addition, parallel 36-inch pipes were modeled adjacent to the 36-inch connector to Pump Station 3 for a distance of approximately 400-feet upstream where the Castro Ditch connects to the Vendola pipe. The Area 5 analysis also included a short cross connect between Pump Station 1 and Pump Station 2 which is an essential component to this alternative. While this scenario does not stop flooding, the majority of flooding is considered nuisance flooding (i.e. flooding in streets and yards but likely not in buildings). To reduce flooding further and maintain redundancy, it may be advantageous to keep at least one operational pump at Pump Station 2. It may be possible to modify the pump settings so they only come on during imminent threat of flooding allowing the majority of storm flows to be handled by Pump Station 1 and Pump Station 3.

5.2 Alternatives Analysis Results

Most effective alternatives have been identified in Section 5 and improvements are shown on Figures 10 through 16. This sets forth a draft capital improvement plan for consideration in Table 3 below. Projects worth pursuiing are listed in order of number of homes potentially affected. Figures 17 through 19 below highlight all junctions where flooding is indicated under the 1D model run for 10-year, 50-year, and 100-year storm events and Figures 17A through 19A show overland flooding results from the 2-D model run based on all the most effective improvements in place.

Costs were developed based on review of recent construction costs in both Marin and Sonoma Counties as well as RS Means Heavy Construction Cost Data, 2013. The costs assume a 30% contingency, 5% Pre-Design, 11% design (Contract documents), 4% Construction Management Administrative Support and 14% Construction Management Field Support.

Table 3 Santa Venetia – Most Effective Improvements

Area	Est. Cost	Description	Existing Flooding Problem	Proposed Improvements
1	$1,351K	Adrian Way, Palmera Way and Pump Station ("PS") 4 improvements. Drains to PS 1 & 4.	Flooding along Adrian, Palmera and LaPlaya Way up to 3 ft deep and covering an area of 120,000 sq ft. Both the pump station and storm drains lack the capacity to prevent flooding under 10-yr and 100-yr storms.	Replace 2 pumps at PS 4; Upsize storm drains on Adrian and Palmera Way (Figure 11)
2	$942K	Labrea Way, Rosal Way, and Galerita Way improvements. Drains to PS 1 & 5.	Flooding along LaBrea, Rosal and Galerita Way up to 1.2 ft deep and covering an area of 50,000 sq ft. Storm drains have a maximum flow capacity of the 2-year storm event.	Upsize storm drains on Labrea Way; Rosal Way; Galerita Way (Figure 11)
3	$1,303K	PS 5 WS: Estancia Ditch; Descanso Way; Hacienda Way; Rincon Way. Drains to PS 5.	Estancia Ditch has a flat, inconsistent slope; storm drains on Descanso, Hacienda and Rincon streets have a maximum flow capacity of up to the 10-year storm event.	Remove gravity bypass at PS 5; Regrade Estancia Ditch and install permeable bottom pavers; upsize storm drains (Figure 12)
4+6	$908K	Vendola Dr and Mabry Way improvements. Drains to PS 1, 2 & 3.	Flooding in street and yards along Vendola Dr near Santa Margarita island up to 2.25 ft deep for the 100-yr event and covering an area of 210,000 sq ft. Convoluted flow routing to PS 2 rather than nearby PS3 passing up to the 10-year storm event.	Upsize storm drains; construct cross-connection on Mabry. (Figure 13)
5	$761K	Vendola Dr, La Pasada Way and Galerita Way Improvements Drains to PS 1 & 2.	PS 2 is old and in need of significant rehabilitation currently passing up to the 10-year storm event	Upsize storm drains; construct cross-connect on Vendola between PS 1 & 2 (Figure 14)
6	See Area 4	See Area 4	See Area 4	Area 6 was addressed with Area 4 improvements (Figure 13)
7	$1,175K	Meadow Dr Interceptor Improvements Watershed does not drain to pump stations.	Interceptor capacity reduced by sediment from Las Gallinas Creek that is costly to remove; results in overland across N San Pedro Rd (NSPR). Interceptor with assumed sedimentation passes up to the 2-year storm event.	Construct outlet riser box and catch basin gallery and 48" pipe on NSPR from N San Pedro Ct to Meadow Dr. Interceptor (Figure 15)
8	$507K	Cross connection on Vendola: PS 1, PS 3 with PS 2 off-line	PS 2 is old and in need of significant rehabilitation	Modify settings to rely less on PS2; add new 16" diameter/50hP pump at PS 3 and larger intake pipe (Figure 16)

6. SEA LEVEL RISE

Sea level rise scenarios were examined to assess the impacts on drainage system operation. Based on review of literature, as well as discussion with County staff, a 12-inch sea level rise scenario is assumed for 2030 and a 24-inch rise is assumed by 2050. The downstream tidal boundary condition was modified in the 100-year flood scenario, assuming all identified improvements in place, by raising the tidal highs by 12-inches and 24-inches. The sea level rise scenarios focus on increases in the downstream boundary condition and does not explicitly account for storm surge, land subsidence, or levee overtopping from direct coastal flooding. The purpose of this analysis was not to design for future sea level rise conditions but to evaluate the impact of sea level rise on the system and proposed modifications. Also, this analysis did not address any direct coastal flooding due to sea level rise overtopping the levee system which may occur regularly with 24-inch sea level rise under otherwise existing conditions.

The flooding impacts of sea level rise of 12-in and 24-in are shown in Figures 17 and 18 for the 100-year flood event. These scenarios are simulated with all of the CIP projects represented in the model. There were no significant increases in local flooding under either scenario indicating that the pump stations were generally able to keep up with peak storm flows even at higher pumping heads and that the pressurized interceptors continue to carry upland flows without downstream flooding impacts.

The impact of sea level rise was observed in the performance and operation of the pumping stations and in the three gravity flow interceptors. All of the pump stations continued to function; however, pump run times increased and there was a decrease in the pumping capacity, as a result of higher pumping heads. The most significant impact was observed at Pump Station 4 where the pumps were operating at close to their maximum capacity. This indicates that under future sea level rise conditions, additional pump system modifications may be useful.

The gravity flow interceptors were operating at a higher downstream condition which reduced their flow capacity; however there was no observed increase in flooding as a result as the upstream segments of the interceptors were still not surcharging.

7. BASELINE MODEL RUNS

This study was initiated in August of 2013. During the progress of the study, the County conducted paving work in the Santa Venetia area that was completed during the 2014 summer construction season. Based on preliminary model results, a new storm drain connection on Mabry Way was identified as a recommended project (Area 4) and this storm drain was constructed as part of the work. An additional model run was made based on the drainage improvements constructed in 2014. Figure 22 is an overall site map showing maximum flooding based on the 1D model results for the 24-hour, 100-year return frequency flood event. Figure 22A is an overall site map showing maximum flooding based on the 2D model results for the 24-hour, 100-year return frequency flood event.

In addition, a model run was performed based on 2014 drainage improvements assuming a 24-inch sea level rise. Figure 23 is an overall site map showing maximum flooding based on the 1D model results for the 24-hour, 100-year return frequency flood event. Figure 23A is an overall site map showing maximum flooding based on the 2D model results for the 24-hour, 100-year return frequency flood event.

There are minor differences in depth of flooding with sea level rise, the most evident occurring in the lower portions of Meadow Drive and along Vendola Drive north of the intersection with Meadow Drive. The additional flooding indicated along Las Gallinas Creek at the oxbow around the island is actually outside of the protective levee and is not impacting residents.