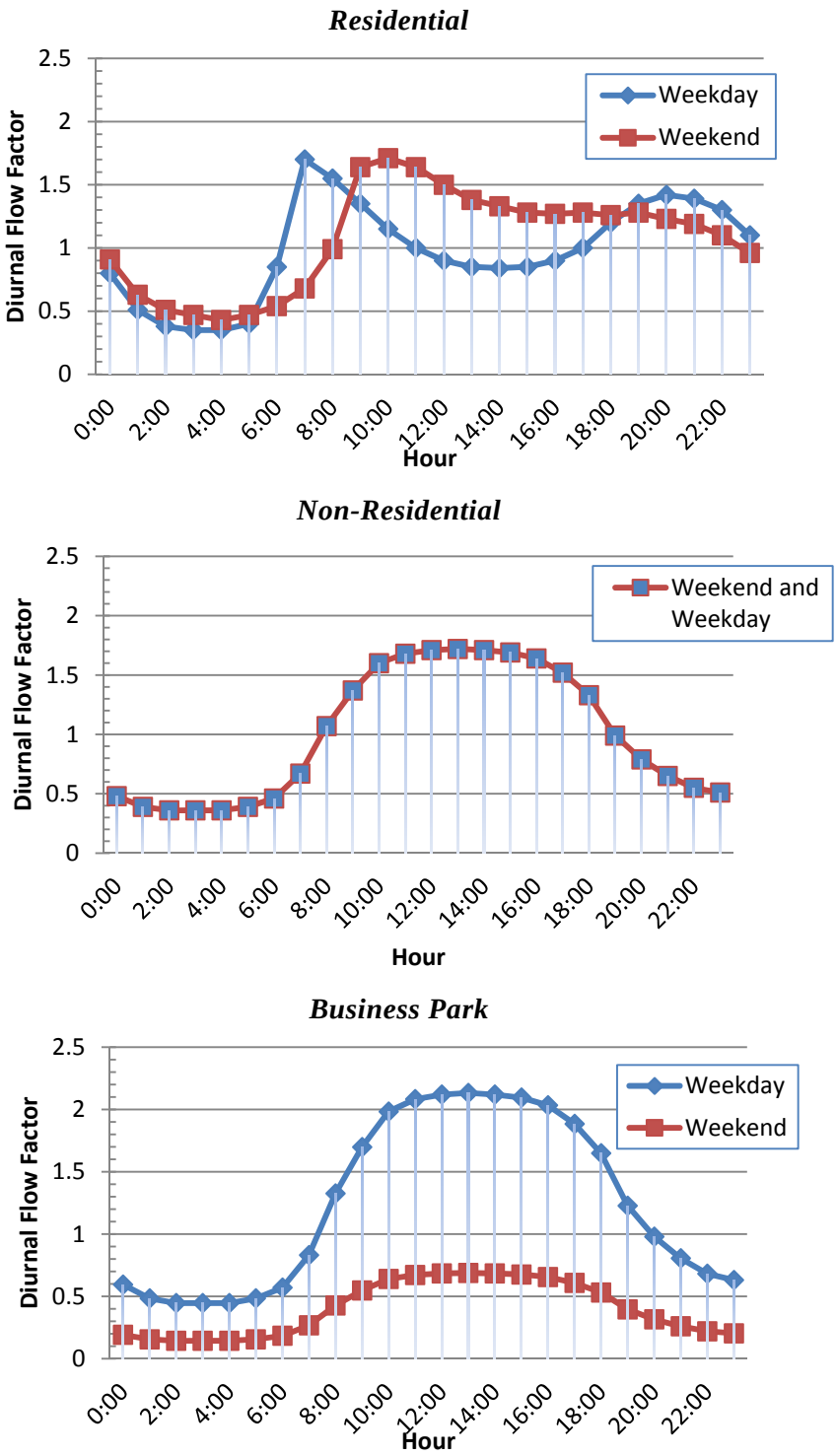
Table 3-3: Potential Future Developments

No.	Name	Description
1	Coast Guard Housing	This parcel is the site of the former Coast Guard housing development. It is proposed for redevelopment to single-family residential. The size of the development area is approximately 36 acres and zoned for a maximum of 435 DUs.
2	Alameda Landing Phase II	Located north of Mitchell Avenue. Proposed redevelopment to approximately 400,000 SF of office space and 20,000 SF of commercial uses
3	Alameda Landing Resid. Phase II	Located north of Mitchell Avenue and currently vacant. Proposed for development of 56 DUs to be completed in 2016
4	Alameda Landing Resid. Phase I	Located west of Fifth St. and north of Willie Strargell Ave. 220 residential DUs (partially completed).
5	Shipways	Located at 1200 Marina Village Parkway, east of Invincible. Planned for residential development up to 30 DUs/acre, which may yield up to 146 DUs.
6	Del Monte	Planned for 380 residential DUs and 30,000 SF of commercial uses.
7	Encinal Terminal	Located north of Entrance Rd. Currently low intensity industrial uses. Potential redevelopment as mixed use with 234 residential DUs (likely multi-family) and 50,000 SF of non-residential uses.
8	Marina Cove II	Approximately 89 residential DUs (in construction)
9	City Maintenance Yard	Planned for redevelopment with 41 residential DUs.
10	Pennzoil Site	Located at Fortman Way and Grand St. Currently used as low intensity industrial uses. Size of redevelopment area is approximately 4.1 acres. Planned for future residential development at 21 DU/acre, yielding 78 units.
11	Alameda Marina	1801 Clement. Planned for 156 DUs on 8.66 acres.
12	Alameda Marina	2033 Clement. Planned for 240 DUs on 13.34 acres.
13		2100 Clement. Planned for 52 single family attached townhouse units.
14	Boatworks	Located north of Clement between Park and Blanding. Currently used as low intensity industrial uses. Proposed redevelopment as residential with 182 single-family DUs by 2017.
15	Former Ron Goode Toyota	1801 and 1825 Park St. Total of 20 residential units.
16	Housing Authority	2437 Eagle Ave. Affordable housing project with 22 single family attached units.
17	Mapes Ranch	11 single family DUs.
18	Esplanade	Located in Harbor Bay Business Park. Approximately 110,000 SF of business park office development on 9.2 acres.
19	VF Outdoor	55,288 SF service facility on 2.8 acres
20	Marriott Hotel	100-room hotel
21	Hampton Inn	New 72-room hotel expansion in detached building.
22	Potential hotel	Located West of Harbor Bay between Maitland and Garden. Currently vacant, approximately 5 acres. Potential hotel development (about 200 rooms) by 2020.

Note: Table does not include development on Alameda Point.

Figure 3-6: Diurnal Profiles



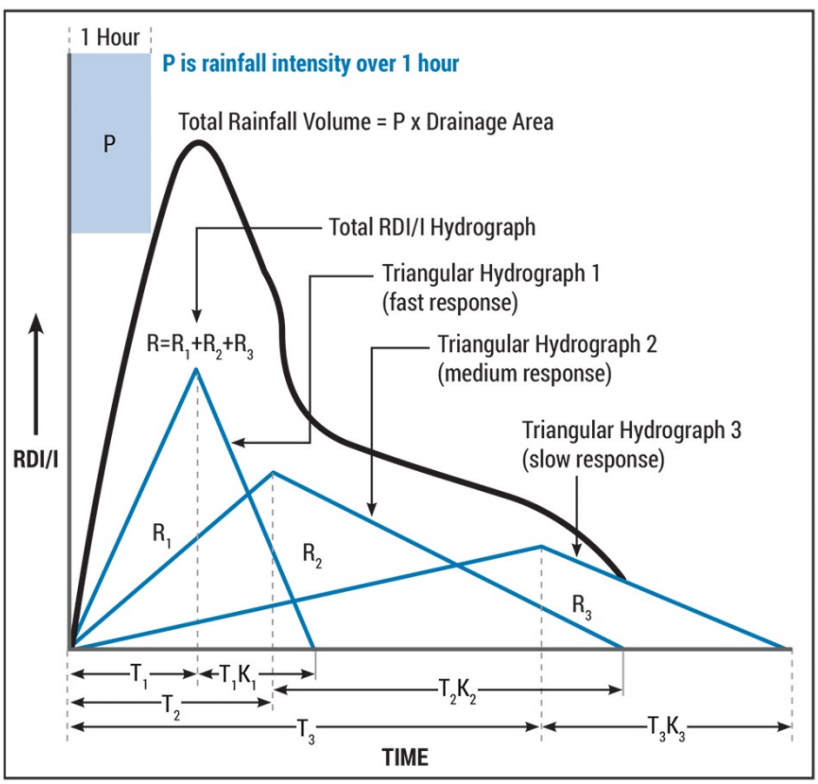
3.4.3 Groundwater Infiltration

GWI is typically applied in the model as a constant load in addition to the BWF. The amount of GWI in any particular area is determined during model calibration by comparing the modeled flows to actual observed non-rainfall period flows at points in the system where flow meter data are available. Where modeled is less than monitored flow, the difference may be due to GWI. The GWI determined at the monitoring location is then distributed to the meter tributary area on a per-acre basis. Note that because GWI is seasonal in nature, the modeled GWI represents an increase in GWI rate during the wet weather season rather than a dry season (summertime) GWI. As noted previously, the BWF unit rates used in the model are assumed to also include some baseline GWI.

3.4.4 Rainfall-Dependent I/I

RDI/I flows result from rainfall events that produce infiltration and inflow of storm water runoff into the sewer system. RDI/I flows are defined by the magnitude, shape, and timing of the RDI/I response. RDI/I varies depending on many factors, including the magnitude and intensity of the storm event, area topography, type of soil, and the condition of the sewers, manholes, and sewer service laterals. In a dynamic model, RDI/I is typically computed as a percentage of the rainfall (sometimes referred to as the "R value") falling on the contributing area of a subcatchment for each of three or more hydrograph components, representing different response times to rainfall, e.g., fast, medium, and slow, as illustrated in **Figure 3-7.** (The contributing area is assumed to be the sum of the area of all developed parcels, except for large open areas such as parks.) Summing all of the component hydrographs for the entire duration of the rainfall event results in the total RDI/I hydrograph for the event for that subcatchment. Note that although the "slow" RDI/I component can contribute significantly to the total RDI/I volume, the "fast" component has the biggest impact on the magnitude of the peak wet weather flow.

Figure 3-7: RDI/I Hydrograph Components



The model parameters defining the RDI/I flows to the system within a given meter area are determined by comparing modeled wastewater flow at the meter location to the measured wastewater flow during one or more rainfall events, as discussed in the model calibration section later in this chapter. The same calibrated parameters are generally applied to all subcatchments within each meter area.

3.5 Model Calibration

Model calibration is the process of comparing model-computed flows to observed (monitored) flows and adjusting various model parameters until the model is accurately simulating flows in the sewer system. The model was calibrated for both dry and wet weather conditions.

3.5.1 Dry Weather Calibration

Non-rainfall periods during the flow monitoring periods were used for comparing flow data to the model results. Periods that were not impacted by previous rainfall and in which a majority of the meters showed consistent readings were selected for dry weather calibration.

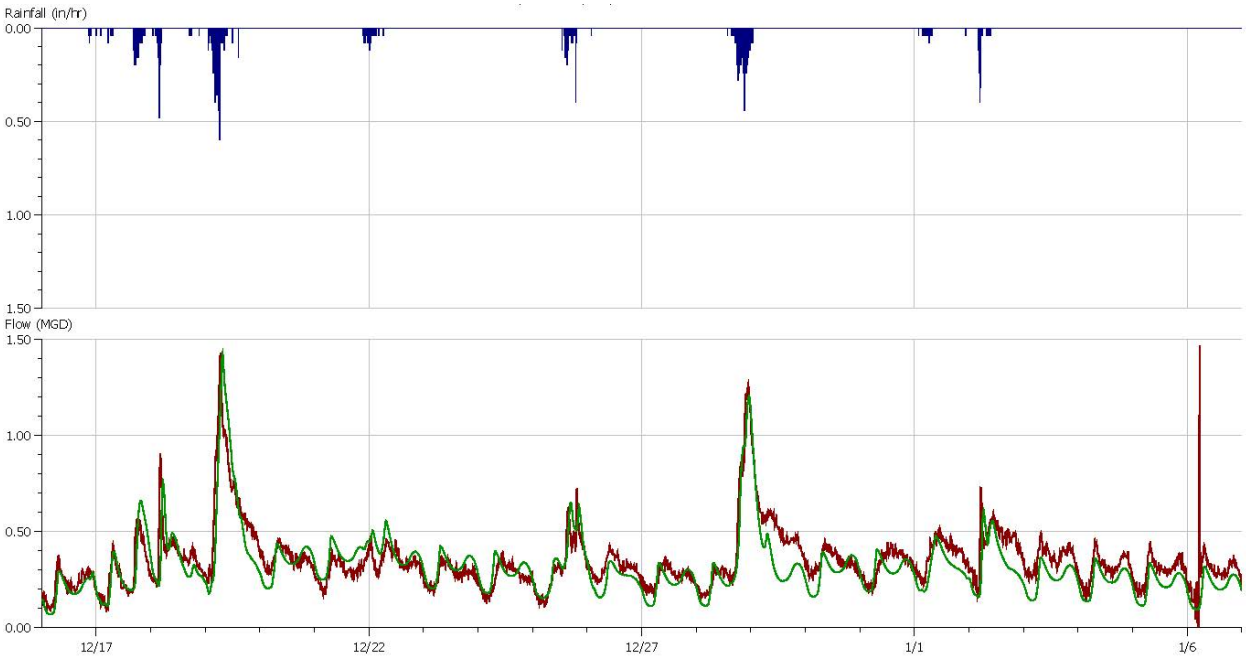
The primary focus of the dry weather calibration was to confirm that the calculated BWF based on the unit flow factors and diurnal curves applied for this study was generally consistent with the measured flows at the meter locations, and to identify which areas of the system appeared to have additional GWI above that reflected in the baseline BWF rates. Based on the comparison of metered to model-simulated flows over seven-day dry periods in January and early March 2011, GWI was added to the model for Basins 90B (except Alameda Landing area where the sewer infrastructure has been completely replaced), 96C1, 96C2, and 97 to achieve a reasonable match to the flow monitoring data overall. One exception was Meter 90-3 (Marina Village Lift Station influent), in which the modeled flows were significantly higher than monitored flows from 2011. It is likely that considerable development may have occurred in this area since the flow monitoring was conducted, resulting in the differences in flows at this location.

3.5.2 Wet Weather Calibration

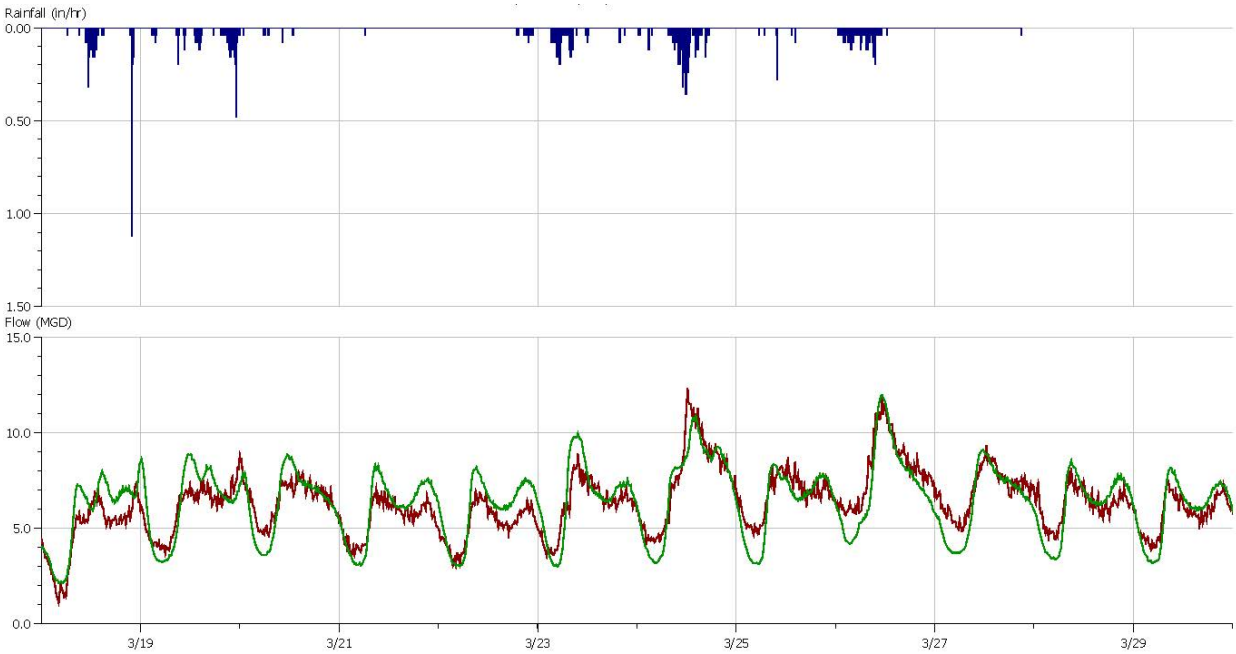
During wet weather calibration, parameters are adjusted to simulate the volume and timing of RDI/I for monitored storm events. Wet weather calibration focused on the December 2010 and March 2011 storm periods, when several significant storms occurred. The wet weather calibration resulted in a reasonably good match between modeled and metered flows for both peak and volume. **Figure 3-8** shows two examples of wet weather calibration graphs. The red line represent the observed (monitored) flow, and the green line represents the model-simulated flow. Rainfall is shown as the dark blue bars across the top of the plots.

Figure 3-8: Example Wet Weather Flow Calibration Graphs

Meter AL-1, December 2010



Meter 94-1, March 2011



3.6 Design Flow and Performance Criteria

Sewer system capacity is assessed with respect to the system's performance under a design flow condition. The subsections below define the design flow criteria used for the capacity assessment and the criteria for assessing system performance and identifying system capacity deficiencies.

3.6.1 Design Storm Condition

The use of wet weather design events as the basis for sewer capacity evaluation is a well-accepted practice. The approach is to first calibrate a hydraulic model of the system to match wet weather flows from observed storm(s), and then apply the calibrated model to a design rainfall event to identify capacity deficiencies and size needed improvement projects. The design event may be synthesized from rainfall statistics, or may be an actual historical rainfall event of appropriate duration and intensity. Other considerations for the design event include the spatial variation of the rainfall and the timing of the storm relative to the diurnal BWF pattern.

Selection of a design rainfall event is typically based on an allowable probability or frequency of occurrence (i.e., risk tolerance), often expressed as the return period. It is recognized that while wet weather overflows are highly undesirable, it is not cost-effective to provide capacity for the largest possible storm event. Regulatory agencies have not adopted standard criteria for return periods, so each agency must choose a target return period based on desired level of service, potential impacts of overflows, and cost.

EBMUD and the Satellites have historically used a design rainfall event that was selected during the 1980s wet weather I/I studies based on historical rainfall frequency analysis. The "EBMUD design event" is a historical storm that occurred on December 5, 1952, considered to be a 5-year return period rainfall event for durations of 1 to 5 hours. These durations are representative of RDI/I response and travel times in small to medium size collection system drainage areas, therefore considered to be appropriate for capacity analysis of the EBMUD satellite systems. The December 5, 1952 rainfall recorded at the Oakland Airport (1.57 inches over 7 hours with a peak hour intensity of 0.64 inches) is adjusted for other locations within the EBMUD service area based on mean annual precipitation. For Alameda, the adjustment factor is approximately 0.97, resulting in a total storm rainfall of 1.52 inches and a peak hour intensity of 0.62 inches.

Figure 3-9 shows a histogram of the design event rainfall for Alameda on an hourly basis. Note that the actual rainfall is modeled in 15-minute increments rather than the hourly increments that were used for the 1980s analyses. The peak hour rainfall was distributed into 15-minute intervals based on the 15-, 30-, and 60-minute return period statistics presented in the Alameda County Hydrology Manual. As such, the peak 15-minute rainfall for Alameda has a peak *intensity* of about 1.35 inches per hour, as shown in **Figure 3-10**.

Figure 3-9: Historical Design Rainfall Event for Alameda

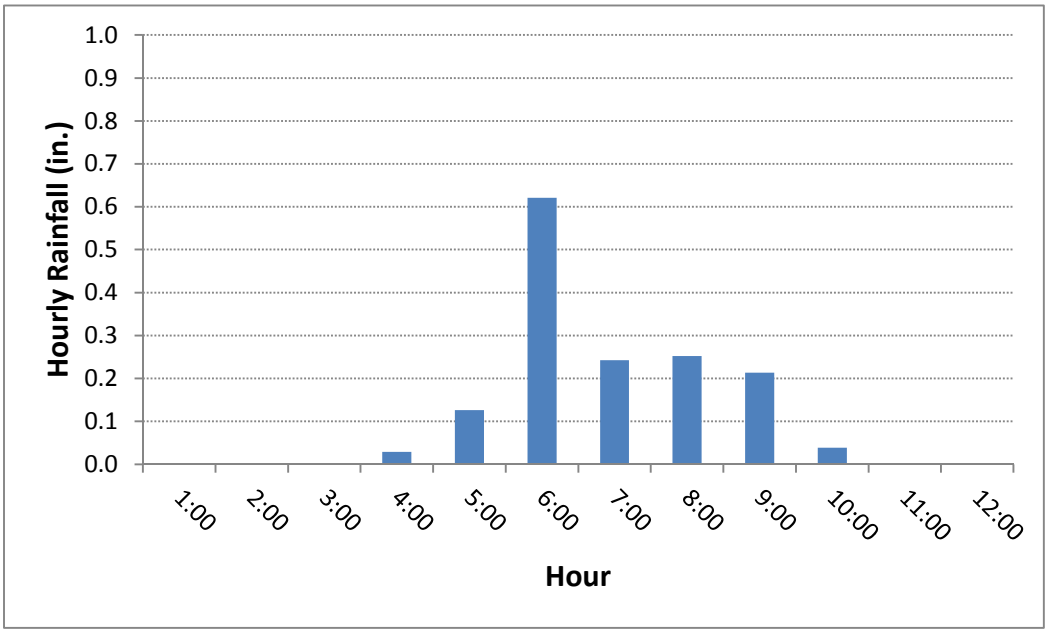
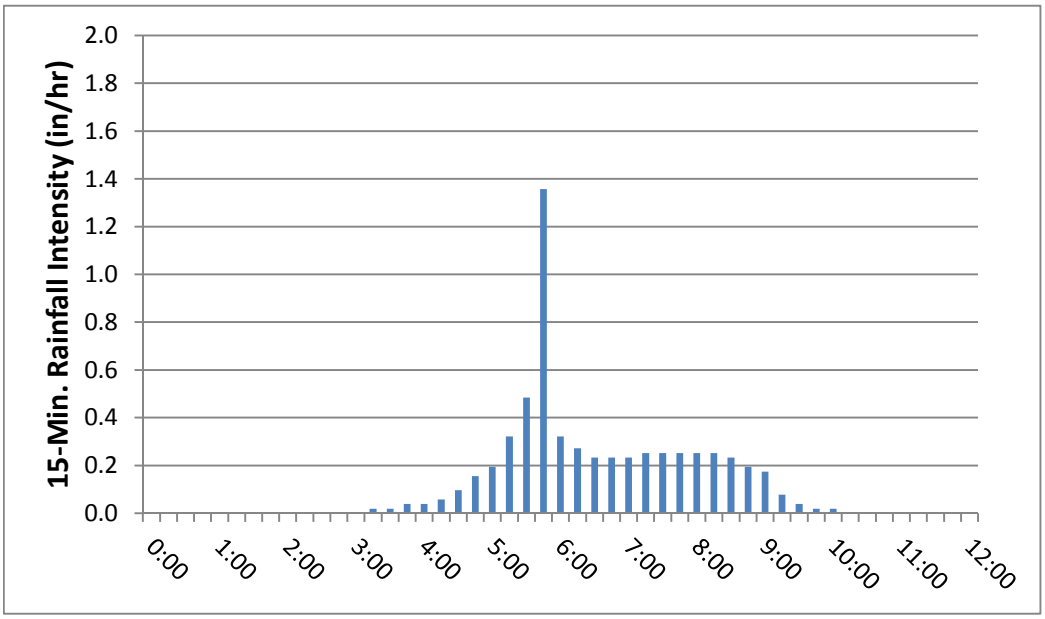


Figure 3-10: Design Event Rainfall Intensity



The timing of the design storm also affects the resultant peak wet weather flows. If the design storm is timed such that the peak RDI/I occurs at roughly the same time as the peak BWF ("peak-on-peak"), the total PWWF will be higher than if the design storm occurs under average or lower-than-average BWF conditions. For Alameda, the EBMUD design event is timed such that the peak rainfall occurs at approximately 8 a.m. on a weekend day (to reflect the slightly higher weekend morning peak in primarily residential areas), resulting in peak RDI/I occurring at or near the time of peak BWF flow (which typically occurs around 10 a.m. on a weekend) for most areas of the system. For business park areas on Harbor Bay Isle, the peak rainfall intensity of the design storm is timed to occur at about 11 a.m. on a weekday to reflect the higher mid-day base wastewater flows in that area.

The design event is also assumed to occur under saturated soil conditions, yielding maximum I/I response. This assumption, together with the timing of the storm to produce peak-on-peak results, is generally thought to create a return period of the peak wastewater *flow* that is greater than the return period of the design rainfall event.

3.6.2 Capacity Deficiency Criteria

Capacity deficiency or performance criteria are used to determine when the capacity of a sewer pipeline is exceeded to the extent that a capacity improvement project (e.g., a relief sewer or larger replacement sewer) is required. Capacity deficiency criteria are sometimes called "trigger" criteria in that they trigger the need for a capacity improvement project. These criteria may differ from "design criteria" that are applied to determine the size of a new facility, which may be more conservative than the performance criteria.

It is important that the capacity deficiency criteria be coordinated with the peak design flow criteria. For example, if the peak design flow considers only peak dry weather flow and little or no I/I, the deficiency criteria should be conservative (e.g., require pipes to flow less than full during dry weather flow to allow capacity for I/I that may increase the flow under a wet weather condition). On the other hand, if the peak design flow includes I/I from a large, relatively infrequent design storm event, it is appropriate to allow the sewers to flow full or even surcharged to some extent, since the peak flows will be infrequent and brief in duration.

For Alameda, since the design storm peak wet weather flow (PWWF) represents a relatively infrequent return period event, the City considers it acceptable to allow surcharging over the pipe crown, provided the hydraulic grade line (water level) remains at least five to six feet below the ground surface. During peak dry weather conditions, however, sewers should be able to convey the peak flow without surcharge.

3.7 Capacity Analysis Results

The hydraulic model was used to simulate flows for the design event and identify areas of the Alameda sewer system that fail to meet the specified performance criteria during predicted design event PWWF. No capacity deficiencies in the system were identified for dry weather conditions.

Under design PWWF conditions, the model shows several areas where backwater surcharge is predicted due to insufficient pump station capacity. The City has already planned for upgrades to these pump stations as part of its Pump Station Renovation program.

The only sewers with identified pipe capacity deficiencies are the 10-inch sewer in Harbor Bay Parkway upstream of the Harbor Bay Parkway I Pump Station, and the 12- and 15-inch sewers in Harbor Bay Parkway and Beach Road downstream of the pump station. The model indicates that the 10-inch sewer is predicted to be capacity deficient under existing design storm PWWF conditions; and the 12- and 15-inch sewers downstream of the pump station would be deficient in the future. These are the same areas identified as potential future capacity deficiencies in the 2010 study. Note that there are no predicted overflows in the model under either the existing or future PWWF scenarios.

"Solution model" runs were made with assumed improvements (pipe upsizing) to address the predicted pipe capacity deficiencies identified above. **Figure 3-11** shows the locations of these predicted pipe capacity deficiencies. **Appendix A** includes model plots of the hydraulic profile of the Harbor Bay Parkway and Beach Road sewer under future design event PWWF conditions with and without improvements.

For the "solution model" runs, pump stations were modeled without limiting capacity so as to be able to convey all flows reaching them to downstream pipes without backwater due to pump capacity limitations. **Table 3-4** summarizes PDWF and PWWF to each pump station compared to existing or planned firm and total capacity. A pump station should be able to convey the design peak flow with the largest pumping unit out of service (firm capacity). The table shows that all of the pump stations (other than those with only a single pump) have adequate existing or planned firm capacity. The five modeled pump stations that have only one pump, however, all have high level gravity bypass pipes which can prevent overflows if the pump is out of service or pump capacity is exceeded.

3.7.1 Capacity Deficiency LOF Scores

The results of the hydraulic modeling were used to assign Capacity Deficiency Likelihood of Failure (LOF) scores for modeled pipes. As shown in Table 2-1, scores were based on the magnitude of predicted sewer surcharge under design event PWWF conditions. The 10-inch sewer in Harbor Bay Parkway upstream of the Harbor Bay Parkway I Pump Station (between the two intersections with Loop Road) is predicted to surcharge to within 5 to 6 feet of the ground under existing PWWF conditions, therefore was assigned a capacity deficiency LOF score of 3. The pipe in Harbor Bay Parkway and Beach Road downstream of the pump station would only be surcharged under future flows, therefore was assigned a capacity deficiency LOF score of 2. All other pipes were assigned a score of 1 (no deficiency).

Legend

- Capacity Deficient Sewers
- Modeled Sewers
- Modeled Force Mains
- Unmodeled City Sewers
- EBMUD Interceptor
- Modeled City Pump Stations
- Unmodeled City Pump Stations
- EBMUD Pump Stations
- Alameda City Limit

City of Alameda
Sewer Master Plan

Figure 3-11
Location of Model-Predicted Pipe Capacity Deficiencies

Sources: ESRI Basemap

J:\Projects\0232 Alameda, City of\0232-013 Alameda Sewer MP Model Update (SJ)\G. GIS\MXDs\Fig3-11 Deficiencies.mxd

Table 3-4: Pump Station Capacities and Design Flows

Pump Station	No. of Pumps		Firm Capacity (mgd)	Max. Capacity (mgd)	Future PDWF (mgd)	Future PWWF (mgd)
Adelphian	2	*	0.29	0.52	0.036	0.11
Aughinbaugh	2		0.90	1.09	0.16	0.35
Bayview	1		--	0.78	0.046	0.074
BFI	2		4.25	4.69	1.42	3.00
Catalina	2	*	1.15	1.73	0.39	0.79
Channing	1		0.21	0.27	0.075	0.18
Cola Ballena	2		0.19	0.30	0.046	0.066
Dublin	2	*	0.38	0.50	0.11	0.27
Eighth/Portola	2		1.87	3.17	0.81	1.31
Eighth/Taylor	2		0.56	0.89	0.26	0.39
Grand Street	2		0.83	1.33	0.019	0.042
Grand/Otis	2	*	1.07	1.66	0.43	0.71
Haile	1		--	0.11	0.037	0.042
Harbor Bay Parkway I	3	*	1.87	3.17	0.59	1.27
Harbor Bay Parkway II	2		0.36	0.48	0.12	0.22
LS 6	2		0.47	0.75	0.27	0.46
Marina Village	4		2.10	2.24	0.36	0.46
Park/Otis	2	*	1.37	1.58	0.64	0.97
Pond/Otis	1		0.42	0.69	0.11	0.17
Sand Beach	1		--	0.06	0.063	0.11
Seaview I	2		0.34	0.54	0.037	0.081
Seaview II	1		--	0.14	0.029	0.069
Sheffield/Cumberland	1		--	0.06	0.040	0.097
Tideway	2	*	0.47	0.68	0.13	0.17
Triumph/Independence	2		0.31	0.50	0.11	0.14
Verdemar	2	*	0.35	0.69	0.12	0.20
Willow/Whitehall	2	*	0.36	0.72	0.23	0.30

* Capacity based on planned pump upgrades.
Pump capacity for BFI Pump Station provided by Waterworks Engineers; all others provided by Schaaf & Wheeler Engineers.

3.8 Alameda Point

As part of the development plan for Alameda Point, the City prepared a *Master Infrastructure Plan*[6] (MIP) that describes the proposed backbone infrastructure to serve the area, including the sewer collection system. As part of this effort, flow estimates were developed for the development project site, and a hydraulic model of the proposed trunk sewer system was created and used to confirm the adequacy of the proposed sewer infrastructure. The model utilized flow parameters and hydraulic criteria consistent with the modeling for the rest of the City's sewer system that are described in this chapter of the report. The modeling methodology and results were summarized in a technical memorandum[7], which is also included as an appendix to the MIP.

The existing sewer system serving Alameda Point was largely constructed by the U.S. Navy over the past 70 years. The system includes about 14 miles of gravity sewers, 2 miles of force mains, and 15 sewer pump stations. All flow is conveyed to EBMUD's Pump Station R, located on the north side of Alameda Point adjacent to Navy Way, from where it is pumped through a 20-inch force main to the Alameda siphon inlet structure. Because most of the sewer infrastructure will be replaced as part of the Alameda Point development, and the sewer attribute data (rim and invert elevations, etc.) for the existing system were incomplete, only the proposed system and future scenarios (described below) were included in the modeling work.

The ultimate development of Alameda Point is expected to include over 1,400 residential dwelling units and 5.5 million square feet of commercial floor space. The development is expected to take place in stages, with the portion identified as the "Development Area", largely located on the eastern side of the site, being developed first with all new sewer infrastructure. Development of the remainder of the site, called the "Reuse Area," would proceed incrementally over time, initially making use of the existing infrastructure with some rehabilitation to address existing deficiencies and reduce I/I. Ultimately, new sewer infrastructure would also be constructed in the Reuse Areas as well.

Accordingly, three model scenarios were analyzed for Alameda Point

- **Scenario A –** Full development in the Development Area with new sewer infrastructure conveying flow to Pump Station R; existing uses in the Reuse Area utilizing existing sewer infrastructure but tying into major trunks constructed as part of the Development Area to convey flow to Pump Station R.
- **Scenario B –** Scenario A plus additional development in the Reuse Area, but still utilizing existing sewer infrastructure with some rehabilitation to address deficiencies and reduce I/I.
- **Scenario C -** Full development and all new sewer infrastructure in both the Development and Reuse Areas.

Figure 3-12 depicts the modeled sewer network and subcatchments. Based on the model analysis, the proposed backbone sewer network as developed in the MIP would be adequate to convey the Alameda Point flows to Pump Station R.

[6] Alameda Point Master Infrastructure Plan, Carlson, Barbee & Gibson, March 2014.
[7] Technical Memorandum - Draft, Alameda Point Sanitary Sewer Flow Estimates and Modeling, RMC Water and Environment, June 2013.

Legend

- E — EBMUD Pump Station R
- L — Lift Station
- Modeled Sewer Scenario A & B
- Additional Modeled Sewer Scenario C
- Development Area
- Reuse Area

City of Alameda
Sewer Master Plan

Figure 3-12
Alameda Point
Proposed Sewer System
Modeled Network

J:\Projects\0232 Alameda, City of\0232-013 Alameda Sewer MP Model Update (SJ)\G. GIS\MXDs\Fig3-12 APOverview.mxd

Sources: ESRI Basemap