CITY OF BRENTWOOD

# Water Master Plan

Adopted March 14, 2006

# City of Brentwood

## Water Master Plan
## Model Update,Water System Analysis, and
## Water System Facilities

FINAL

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# Model Update, Water System Analysis, and Water System Facilities

## Purpose of Technical Memorandum

This technical memorandum (tech memo) presents the findings of CDM's work on Master Plan Tasks MP1, MP2, MP3, and MP4. These tasks covered: updating the previous model, updating the model demand allocations, establishing the analysis criteria, conducting model runs to review previous findings on water system improvements, preparing updated information on water system facilities, and preparing and revising this tech memo to incorporate City comments. The starting point for CDM's work was the previous model and findings developed for the City's June 2003 draft water master plan prepared by Water Resource Associates.

The following topics are discussed in this tech memo:

- Summary of Updated System-wide Water Demand
- Model Update
- Planning Criteria
- Water System Analysis
- Water System Facilities (Existing and Future Facilities)

## Summary of Updated System-Wide Water Demand

CDM updated the City's system-wide water demands, as described in the March 23, 2004 Technical Memorandum titled "Review of Water Demand Projections". The work described herein was based on the recommended demand projections described in the March 2004 Technical Memorandum.

A total system-wide maximum day buildout demand of 41 mgd is used for this master plan update. The buildout demand is based on the City's adopted 2001 General Plan land uses and the unit demand factors shown in Table 1.

The maximum day peaking factor is 2.1 times the average day demand. The peak hour factor is 4.0 times the average day demand.

| ***Table 1*** ***Master Plan Unit Demand Factors*** | |
|---|---|
| ***Land Use*** | ***Unit Demand*** |
| **RESIDENTIAL** (1) | **(GPD/unit)** |
| Ranchette Estate (RE) | 1,000 |
| Very Low Density (VL) | 710 |
| Low Density (L) | 535 |
| Medium Density (M) | 410 |
| High Density (H) | 260 |
| **NON-RESIDENTIAL** | **(GPD/acre)** |
| General & Regional Commercial (GC,RC) | 1785 |
| Office (O) | 1785 |
| Mixed Use Business Park (BP) | 1785 |
| Industrial (I) | 1785 |
| Public (PF) (2) | 895 |
| Semi Public (SP) (2) | 895 |
| Schools (S) (2) | 895 |
| Parks (P) (2) | 895 |
| Urban Reserve (undefined uses) | 0 |
| Open Space, Agriculture | 0 |

(1) No Very High Density residential land uses are identified in the 2001 General Plan.
(2) Unit demands assume that most large turf areas, such as playing fields in parks and schools, will be irrigated with non-potable water.

# Model Update

CDM reviewed and updated the Watercad hydraulic computer model of the water system provided by the City. The City's Watercad model was developed by the City's consultant who prepared the June 2003 draft water master plan. CDM revised the model to reflect recently constructed facilities and changes to planned future locations, as well as revising the model demands to reflect the updated system demand. The updated model is shown on water system maps (two maps – one showing north part of system, and the other showing the south part).

CDM reviewed the future facility locations with the City. City staff spent considerable time during preparation of the June 2003 draft plan to identify proposed future pipeline alignments to avoid construction impacts to existing streets, and other facility locations based on ground elevations and available land. The future gravity reservoir sites already identified by the City are the only elevated sites that are available. All other future sites will be non-gravity reservoirs, which will require pumping out of the reservoirs into the distribution system.

Previously, there was a separate model for each pressure zone. The three zones were not connected together in a single model. CDM combined the separate zone models into one system model. This allows for a system-wide evaluation of transmission and pumping facilities that transfer water between zones, and also for better preparation of a comprehensive map showing existing and future facilities.

Pump stations and wells were modeled based on available pump and discharge data, rather than as constant rate injection points as previously done. This allows for more accurate representation of system operations.

Ground elevations at nodes were already in the City's model. CDM reviewed and adjusted ground elevations at locations where adjustments were made to future facility locations. The adjustments were made using elevation data from USGS quad maps.

The demands allocated to the model nodes were revised to reflect the updated demands. The re-allocation of node demands was based on matching the updated system-wide demand, and general checks of the nodal demand allocations using available information.

CDM reviewed information provided by the City on the changes between the 1993 and 2001 General Plan land uses. This information included a copy of the wall map showing the changes between the 1993 and 2001 General Plans and an accompanying roster (listing) of the changes shown on the map.

The distribution of future demands among the model nodes was reviewed with respect to the previous and updated General Plan land uses. CDM identified if any areas required major re-allocations due to major changes in anticipated future land uses. For areas with major changes in land uses, nodal demands were re-calculated using the 2001 General Plan land uses and the master plan unit demand factors.

On a system-wide basis, after making necessary adjustments in specific areas, the model demands were adjusted by the ratio of the updated system demand to the previous system demand, in order to achieve an overall system-wide demand consistent with the updated system-wide demand.

Except as noted above for specific areas with major land use changes, CDM did not conduct a detailed check of the 2001 General Plan land uses with the node demands from the previous model. No back-up information was available to conduct a detailed check of the previous allocations.

If the City desires a more comprehensive check on the distribution of nodal demands in future updates, land use maps could be prepared in GIS based on the 2001 General Plan, and nodal demands developed using the master plan unit demand factors.

## Planning Criteria

Table 2 summarizes the key planning criteria for the water system analysis.

| ***Table 2*** <br> ***Summary of Key Criteria for System Analysis*** | |
|---|---|
| ***Element*** | ***Description*** |
| System Pressures | • Minimum:<br>– 40 psi during maximum day, peak hour<br>– 20 psi during maximum day plus fire flow<br>• Maximum: 110 psi |
| Fire Flow Requirements | • These requirements are for water system master planning purposes. Fire Department sets the specific requirements for individual subdivisions.<br>• One fire at a time in zone (no simultaneous fires in the same pressure zone).<br>• Flow rates and durations:<br>– Single family and multiple family residential – 2000 gpm for 2 hours<br>– High density residential and schools – 3000 gpm for 3 hours<br>– Office, commercial and industrial – 4000 gpm for 4 hours. |
| Pipeline Sizing | • Hazen Williams (Friction) Coefficients<br>– Existing 8" and smaller pipes – 100-120; future 8" pipes - 130<br>– 10" pipes – 125<br>– 12" and larger pipes - 130<br>• Maximum velocity: 5 fps or less during maximum day, peak hour; 10 fps or less during maximum day plus fire flow<br>• Maximum headlosses: 10 feet per 1000 feet or less during maximum day, peak hour<br>Note: The maximum velocity and headloss criteria are used for sizing new pipes. Existing pipes that provide adequate pressures in the system are not identified for improvement due to not meeting the maximum velocity and headloss criteria. |
| Storage Capacity | • Storage provides the following three functions:<br>– Operational (or balancing storage) to meet daily fluctuations in demand in excess of the water supply production capacity on the maximum day. For the City system, this component is estimated as 25 percent of the maximum day demand.<br>– Fire storage to provide a reserve for fire fighting. This is estimated as the most critical (highest) fire flow required in a zone times the required duration.<br>– Emergency storage to provide an emergency reserve in case of planned or unplanned outages of equipment or facilities, including power or supply outages. For the City system, this component is estimated as 50 percent of the maximum day demand.<br>• The total required storage is the sum of the above three components. |
| Pumping Capacity | • Pump stations pumping into zones with gravity reservoir storage are sized to have firm capacity equal to the maximum day zone demand (average rate over 24-hours). Fire flows are provided by gravity from the zone storage.<br>• Hydropneumatic pump stations are sized for firm capacity for domestic flows equal to the peak hour flow into the zone (or into the portion of the zone served by the pump station). Hydropneumatic stations must also have a fire pump to provide fire flows in the zone. Hydropneumatic stations must have back-up capabilities.<br>• For all zones, pump stations must also have the ability to pump any flow that must be lifted through to subsequent higher zones.<br>• Total pump station capacity equals the firm capacity plus a standby pump equal in size to the largest pump. For hydropneumatic stations, a standby domestic and standby fire pump are required. |
| Reservoir Levels | • Facilities are sized so that reservoirs drop no lower than 75 percent of full capacity on the maximum demand day, and storage can be re-filled over a 24-hour period on the maximum demand day. Hydraulic simulation was conducted for a 72-hour period to eliminate the effect of initial hydraulic conditions, i.e., after the system stabilized, the initial reservoir starting level had no effect on the subsequent 24-hour periods covered in the analysis. |

# Water System Analysis

CDM used the updated model to conduct water system analyses and revise the previous improvement recommendations in the June 2003 draft report.

The following model runs were conducted to determine the sizes of future facilities:

- Maximum day with fire flows at several critical fire flow locations in each zone. The locations were selected based on factors such as the most intensive land use, highest ground elevations, proximity to large diameter pipelines, and amount of looping in the vicinity.
- Peak hour on the maximum day; and
- Extended period simulation (EPS) on the maximum day to check reservoir re-filling. The diurnal curve already in the City's previous model was used for the EPS analysis. Figure 1 shows this diurnal curve.

The analysis was done only for buildout demands, and did not include any modeled phasing of required improvements between existing and buildout.

The findings from the water system analysis are described below.

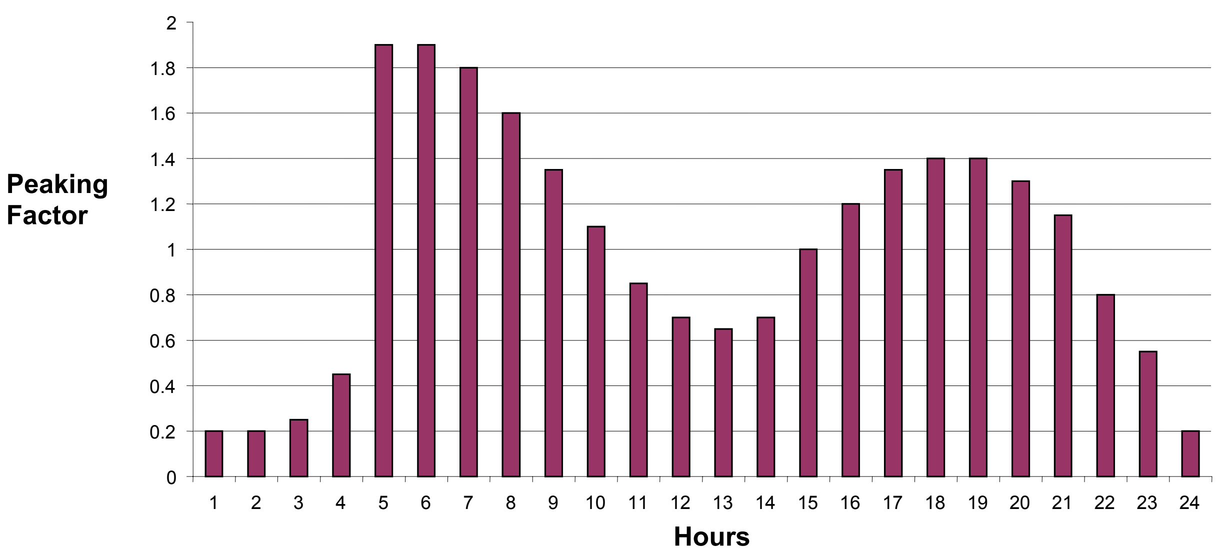


**Figure 1**
**Diurnal Curve**

# Water System Facilities

The water system maps (plan-size) show the existing and proposed water system facilities identified in the water system analysis. Facilities are sized to meet buildout demands. Key water system features are described below including:

- Pressure Zones
- Water Supply
- Storage Facilities
- Pumping Facilities
- Pipelines

## Pressure Zones

The City water system has three pressure zones: Pressure Zone 1, Pressure Zone 2 and Pressure Zone 3. Table 3 summarizes the service elevation ranges and demands for the three zones. Figure 2 shows the locations of the pressure zones.

***Table 3***
***Pressure Zones***

| ***Zone*** | ***Service Elevations (ft)*** | | ***Zone Demands (mgd)*** | | |
|---|---|---|---|---|---|
| | ***Lower*** | ***Upper*** | ***Average Day*** | ***Maximum Day (1)*** | ***Peak Hour (2)*** |
| 1 | 0 | 110 | 12.1 | 25.5 | 48.4 |
| 2 | 110 | 220 | 6.1 | 12.8 | 24.4 |
| 3 North | 220 | 330 | 0.05 | 0.1 | 0.2 |
| 3 Central | 220 | 330 | 0.95 | 2.0 | 3.8 |
| 3 South (Vineyards) | 220 | 330 | 0.3 | 0.6 | 1.2 |
| **Total** | | | **19.5** | **41.0** | **78.0** |

(1) Maximum day demand at 2.1 times average day demand.
(2) Peak hour demand on the maximum day at 4 times average day demand.

The Figure 3 profile schematic showing the service elevations of the City's three pressure zones, the elevations of the existing and future pump stations and reservoirs serving each zone. The profile schematically shows how the facilities are linked hydraulically. The reservoirs and pump stations are discussed later in this section.

Zone 1 is the largest and lowest zone and covers the east side of the City. All water supplied to the City goes through Zone 1 and is pumped up to the other zones. Zone 2 is located primarily on the west and south side of the City at higher elevations than Zone 1. Both Zones 1 and 2 have in-zone reservoir storage.

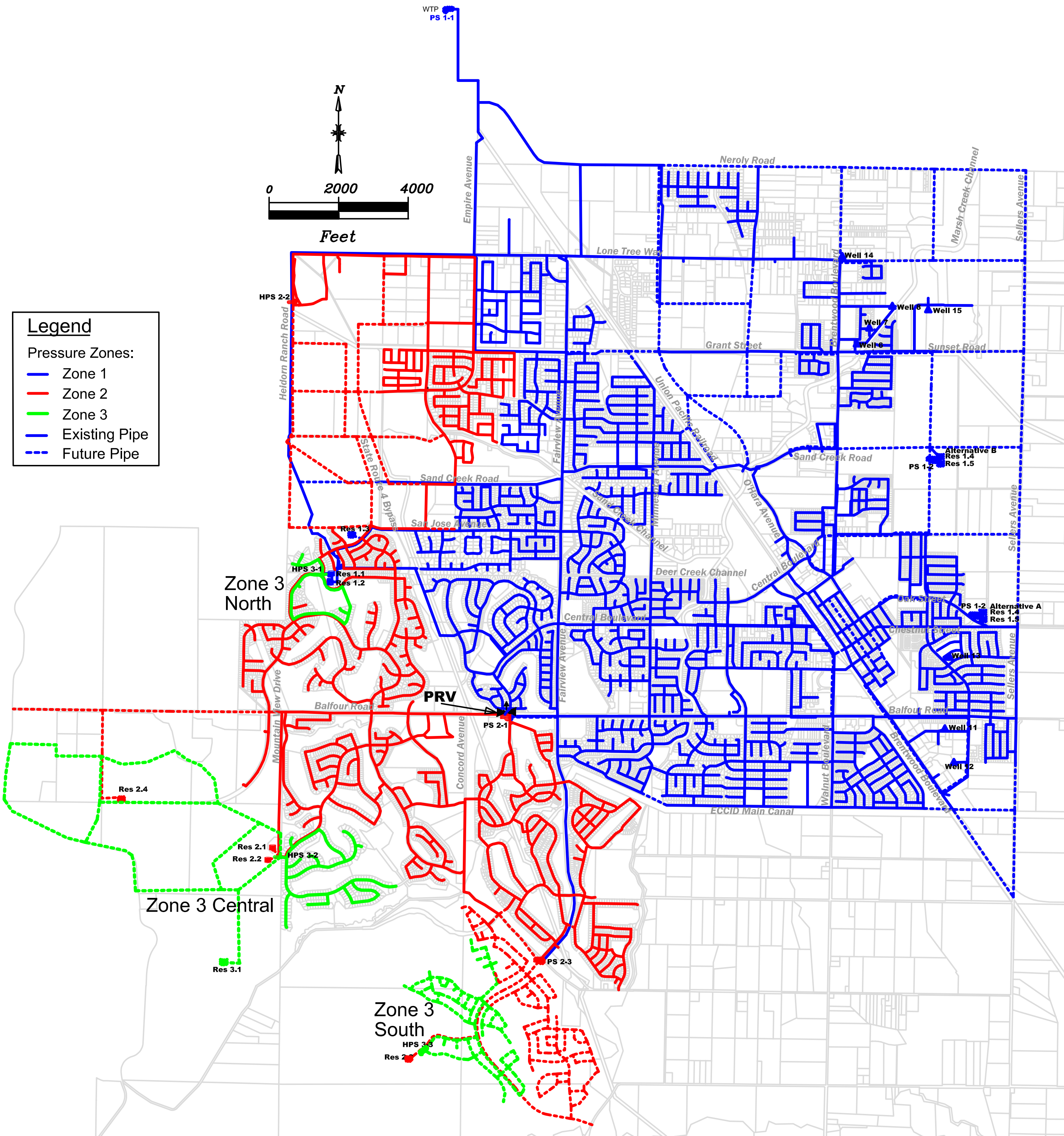


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**Figure 2**
**Pressure Zones**

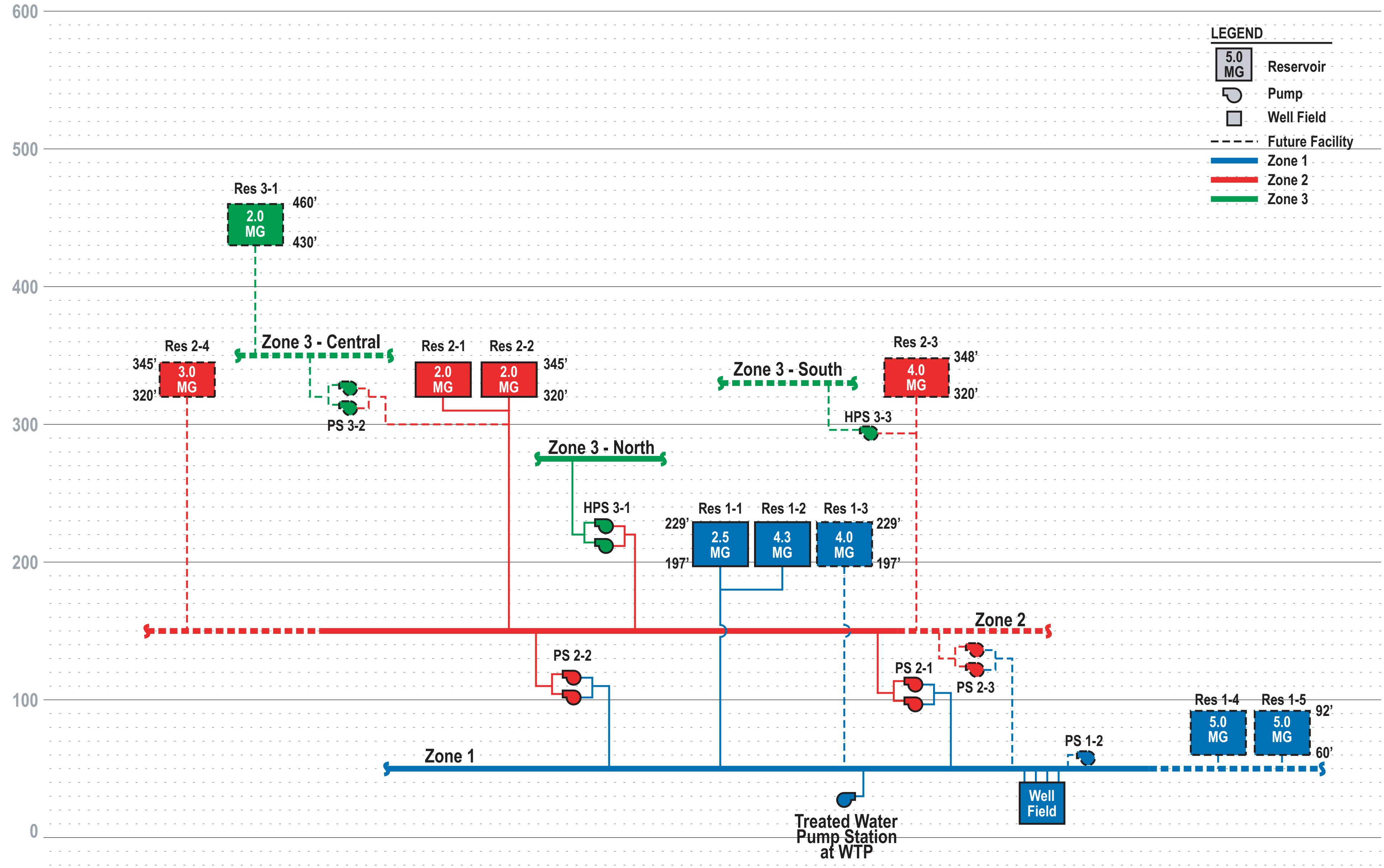


**Figure 3**
System Profile Schematic

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Zone 3 is comprised of three isolated islands that operate as independent subzones to serve the highest elevation areas in the City. The three Zone 3 subzones are not connected to each other, and are located generally in the north, central and south part of the western side of the City.

All Zone 3 subzones are currently hydropneumatic zones with no reservoir storage located within the subzones. In the future, the central subzone (Zone 3 Central) is expected to have in-zone reservoir storage. The north and south subzones (Zone 3 North and Zone 3 South) will remain hydropneumatic zones.

## Water Supply

For the buildout analysis, the total maximum day demand is 41 mgd. At buildout, the City will have a total maximum day supply of 42 mgd from the following sources:

- 30 mgd from a future surface water treatment plant (WTP) located at Laurel Road and Empire Avenue adjacent to the Randall Bold WTP (RBWTP);
- 6 mgd from the RBWTP; and
- 6 mgd from the City's groundwater wells.

The City is contracting with the Contra Costa Water District (CCWD) for the new surface water treatment plant that will be located north of Brentwood. The plant will treat surface water supply purchased by the City from the East Contra Costa Irrigation District under an existing agreement that covers the City though buildout. Until the new WTP is completed, treated surface water will be supplied from the existing Randall-Bold WTP. Completion of the first stage of the new WTP is anticipated by early 2008. The plant will be expanded in the future when needed to meet increased demands.

Treated surface water will be pumped into the City's Zone 1 via a treated water pump station located at the WTP and a large diameter transmission pipeline. This pump station and pipeline are under construction and anticipated to be operational in early 2006. Until the new facilities are completed, the City has an interim treated water pump station located near the crossing of the Southern Pacific Railroad tracks and Empire Avenue that supplies water to the system.

The water system analysis was based on the proposed pumps at the new treated water pump station to be located at the future Water Treatment Plant. The proposed buildout pumps include 2 pumps at 6 mgd (includes one standby) and 3 pumps at 15 mgd (includes one standby). The analysis assumed the pumps were set to operate on level control at the existing Zone 1 reservoirs that operate by gravity flow. The analysis assumed that pumping occurs at an average hourly rate of 36 mgd over a 24-hour period on the maximum day at buildout.

There are several groundwater wells located in the eastern part of the City. These wells will continue to provide a portion of the City's normal supply. As an alternate supply to surface water, the wells provide additional reliability for the City's system.

Table 4 summarizes the six City wells that were assumed in the analysis to be active wells. The City also has four other wells that are rarely used due to poor water quality, although would be available if needed as back-up supply in emergencies. It is expected that the City will maintain a minimum firm groundwater pumping capacity of at least 5 mgd to meet buildout maximum day demands. This minimum firm capacity allows for one of the six active wells in Table 4 to be out-of-service.

***Table 4***
***Modeled Groundwater Wells***

| ***Well Number*** (1) (2) | ***Well Capacity*** | | ***Well Pump HP*** | ***Type of Pump*** | ***Modeled Ground Elevation (ft)*** | ***Modeled Buildout Discharge HGL Range*** (3) ***(ft)*** |
|---|---|---|---|---|---|---|
| | ***(gpm)*** | ***(mgd)*** | | | | |
| 6 | 800 | 1.15 | 100 | Vertical Turbine | 64 | 195-270 |
| 7 | 700 | 1.01 | 100 | Submersible | 57 | 196-272 |
| 8 | 1000 | 1.44 | 100 | Vertical Turbine | 55 | 200-275 |
| 12 | 400 | 0.58 | 50 | Submersible | 66 | 190-232 |
| 13 | 300 | 0.43 | 50 | Submersible | 60 | 190-230 |
| 14 | 1000 | 1.44 | 150 | Submersible | 67 | 191-275 |
| 15 | 450 | 0.65 | 60 | Vertical Turbine | 53 | 200-275 |
| *Total for All* | *4,250* | *6.7* | | | | |

(1) Wells 9, 10A, and 11 are not listed and were not modeled. Well 11 was considered to be out of service for the analysis, since it sometimes experiences water quality issues; although most times of the year the quality is good. Due to their poor water quality, Wells 9 and 10A were considered to be out of service for the analysis. They are considered to be backup wells.

(2) Wells are typically pumped in the following order: first - Well 13, second - Well 14, third – Well 15, and then Wells 6, 7, 8 and 12 in no regular order.

(3) The discharge HGL is the low and high range for the maximum day at buildout. The discharge HGL will vary depending on hourly demand and reservoir re-filling conditions.

For buildout, the existing well pumps will need to be replaced with higher head (pressure) pumps, since it will be necessary to operate at a higher discharge head to overcome higher system headlosses due to higher demands conveyed by the system at buildout. Table 4 shows the low and high range for the discharge hydraulic gradeline (HGL) on the maximum day at buildout. The discharge HGL will vary according to the hourly demand and reservoir re-filling conditions.

## Storage Facilities

Table 5 summarizes the storage capacity requirements at buildout of the City's three zones. It shows the required storage capacity per the City's planning criteria, and identifies the additional storage volume needed in each zone.

**Table 5**
**Buildout Storage Requirements**

| Zone | Maximum Day Demand (mgd) | Maximum Fire Flow Requirement (gpm) | Required Buildout Storage Volume (MG) | | |
|---|---|---|---|---|---|
| | | | Operational and Emergency (at 75% of maximum day demand) | Fire Reserve | Total Storage Needed |
| 1 | 25.5 | 4000 | 19.1 | 0.96 | 20.1 |
| 2 (includes 3 North and 3 South) | 13.5 | 4000 | 10.1 | 0.96 | 11.1 |
| 3 Central | 2.0 | 2000 | 1.5 | 0.36 | 1.9 |
| **TOTAL** | **41.0** | | **30.8** | **2.3** | **33.0** |

Table 6 shows the existing and proposed storage reservoirs to meet buildout storage requirements. The locations of existing and future reservoirs are shown on the water system map. All future storage locations are conceptual. Future storage could be located in the same general vicinity that provided that same hydraulic characteristics.

**Table 6**
**Existing and Future Storage Reservoirs**

| Zone | Reservoir Name | Location | Current Status | Type | Reservoir Elevation (ft) [2] | | Volume (MG) | |
|---|---|---|---|---|---|---|---|---|
| | | | | | Bottom | Overflow | Existing | Future |
| 1 | Res 1.1 | South of San Jose Ave and west of SR 4 Bypass | Existing | Gravity | 197 | 229 | 2.5 | |
| | Res 1.2 | | Existing | Gravity | 197 | 229 | 4.3 | |
| | Res 1.3 | Off San Jose Ave near Res 1.1 and 1.2 | Existing | Gravity | 197 | 229 | ----- | 4.0 |
| | Res 1.4 | East side of Zone 1 (1) | Future | Pumped - Not Gravity | 60 | 92 | ----- | 5.0 |
| | Res 1.5 | East side of Zone 1 (1) | Future | Pumped - Not Gravity | 60 | 92 | ----- | 5.0 |
| *Subtotal - Zone 1* | | | | | | | *6.7* | |
| 2 | Res 2.1 | Mountain View Dr south of Balfour Rd | Existing | Gravity | 320 | 348 | 2.0 | |
| | Res 2.2 | | Existing | Gravity | 320 | 348 | 2.0 | |
| | Res 2.3 | Vineyards | Under Construction | Gravity | 320 | 348 | ----- | 4.0 |
| | Res 2.4 | Future west side site, west of Mountain View Dr | Future | Gravity | 320 | 348 | ----- | 3.0 |
| *Subtotal - Zone 2* | | | | | | | *4.0* | |
| 3 | Res 3.1 | Future site in southwest off Mountain View Dr | Future | Gravity | 430 | 460 | ----- | 2.0 |
| ***Breakdown of Existing and Future Storage Volumes*** | | | | | | | ***10.8*** | ***23.0*** |
| **TOTAL ULTIMATE SYSTEM-WIDE STORAGE VOLUME** | | | | | | | **33.8** | |

(1) In Zone 1, there are two alternative locations for Reservoirs 1.4 and 1.5, as shown on the water system map. Both reservoirs would be constructed at the same location and served by the same pump station, PS 1.2.
(2) Elevations for future gravity reservoirs are consistent with existing gravity reservoirs. Elevations for pumped (non-gravity) reservoirs can be adjusted as needed for site conditions.

Zone 1 is the largest zone, so requires the most storage. Gravity storage is located on the west side of Zone 1. Future non-gravity storage will be located on the east side of Zone 1. The non-gravity reservoirs may be at grade or buried. A pump station will be required at the site to pump the stored water out of the non-gravity reservoir into the system.

All the future Zone 1 east-side non-gravity storage is shown on the water system map at the same site. Since this storage must be pumped into the system, it would be more efficient to locate it at the same site and served by one pump station. Having multiple locations with multiple pump stations close together would make operations more difficult.

Implementation of the future east-side storage would be phased as two 5 MG tanks, called Reservoir 1.4 and Reservoir 1.5. It could also be phased in smaller increments if desired, and space is available. One pump station PS 1-2 would serve both tanks, and should be designed to easily add future pumping capacity when needed.

There are many alternative locations for the future Zone 1 east-side non-gravity pumped storage. The following two representative alternative locations for Reservoirs 1.4 and 1.5 are shown on the water system map:

- Alternative A location near existing waterlines at the intersection of Oak Street and Chestnut Street. The benefit of this location is that a storage tank could be constructed at any time to discharge into the existing waterlines, regardless of the timeframe for construction of future east-side waterlines.

- Alternative B location adjacent to future waterlines approximately one mile southeast of the intersection of Sunset Road and Brentwood Boulevard. The timing for construction of this storage would be contingent on construction of the future waterlines.

With the proposed pipeline network, the future site for Reservoirs 1.4 and 1.5 could essentially be located anywhere in the Alternative A vicinity so long as it discharges into the existing 20-inch main in Oak Street or Sellers Avenue, or anywhere in the Alternative B vicinity along the future 24-inch transmission main south of Sunset Road between Brentwood Boulevard and Sellers Avenue. In essence, this could be anywhere in the area shown on Figure 4 that is bounded by Sunset Road on the north, Brentwood Boulevard on the west, Sellers Avenue on the east, and Oak Street on the south. If the reservoir location is modified, then the future pipeline diameters should be modified to be consistent with the requirement that the pump station discharge into the future 24-inch transmission main.

The future Zone 2 storage requirements include all of Zone 2 plus the required storage for the two Zone 3 hydropneumatic subzones (Zone 3 North and Zone 3 South). Since the two Zone 3 hydropneumatic subzones do not have any in-zone storage, the storage capacity is provided in Zone 2. The location for future Reservoir

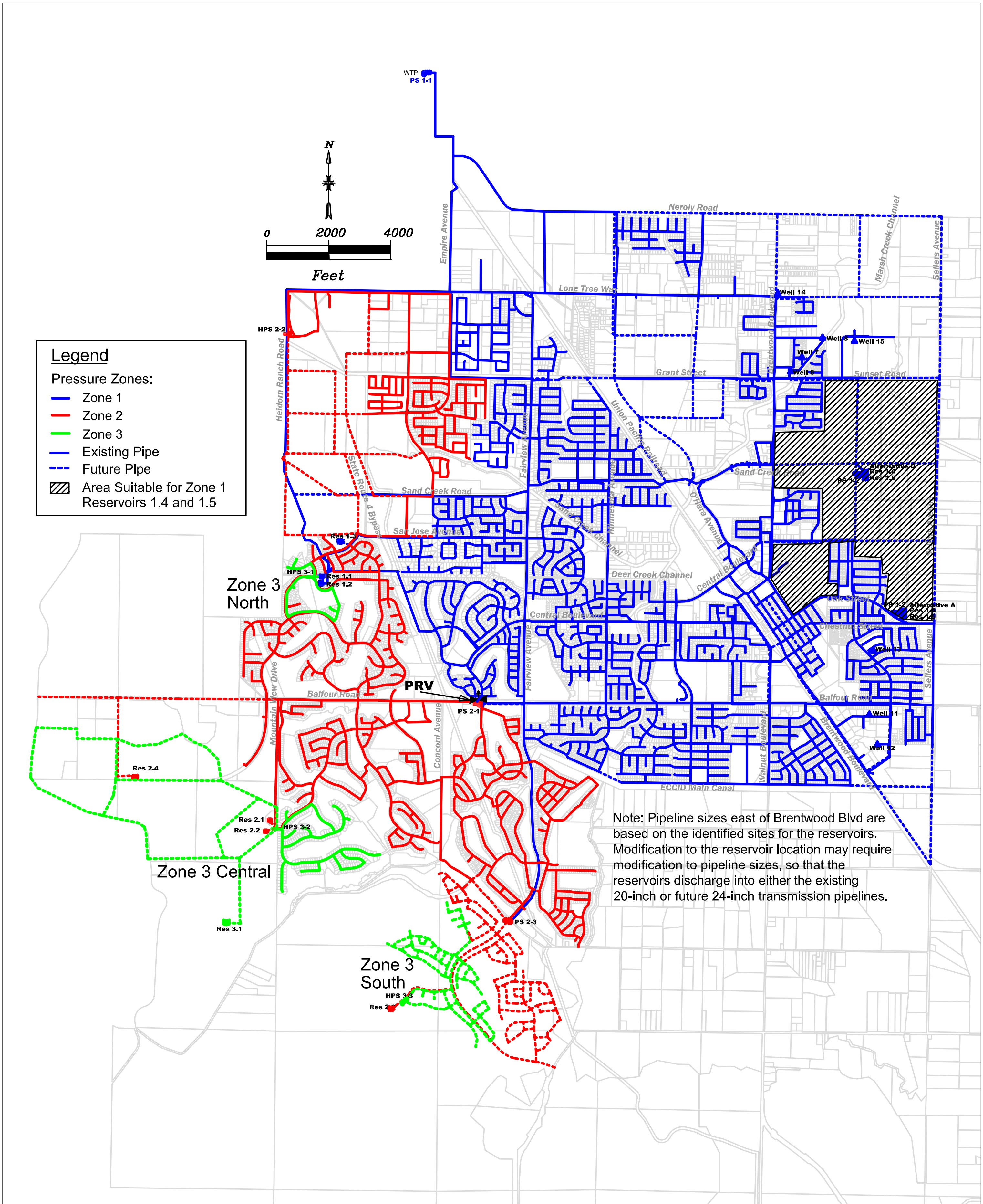



**Figure 4**
**Suitable Area for Locating Future Zone 1 Reservoirs 1.4 and 1.5**

2.3 in the Vineyards development is already identified. The only remaining future Zone 2 reservoir, Reservoir 2.4, will serve future development in the west side of Zone 2.

The future Reservoir 2.4 location is farther west than the existing Reservoir 2.1 and 2.2. The location is in the future development area on the west side of the City that would be served by the reservoir, and is a good location for distributing storage throughout the zone. Below are some considerations for future zone operations with Reservoir 2.4:

- Reservoir 2.4 should be located at the same elevation as Reservoir 2.1 and 2.2 (see Table 6 for bottom and overflow elevations):
  - If PS 2.1 is set to operate on level control at Reservoirs 2.1 and 2.2, Reservoir 2.4 which is the farthest in the zone from PS 2.1 will be cycling at lower levels, 5 feet lower than the other reservoirs due to system head loss. This will not effectively use the reservoir volume and is not recommended.
  - A better option would be to set PS 2.1 to operate based on level control at Reservoir 2.4. With this option, Reservoirs 2.1 and 2.2 will fill first. These two reservoirs must have altitude valves to prevent overfilling while the pumps continue to operate to finish filling Reservoir 2.4.
- Reservoir 2.4 should not be located at a lower elevation than Reservoirs 2.1 and 2.2. Otherwise, it will have a lower turnover rate which could lead to water quality problems.

Only Zone 3 Central will have future in-zone storage. Since this subzone operates independently from the other two Zone 3 subzones, the required storage is only for Zone 3 Central. The other two Zone 3 subzones, Zone 3 North and Zone 3 South, will continue to be hydropneumatic zones, and storage for these hydropneumatic zones is included as part of the Zone 2 storage. (Note: If Zone 3 Central remains a hydropneumatic zone in the future, then its required storage should be located in Zone 2.)

## Pumping Facilities

Table 7 summarizes key features of the existing and future pumping facilities for buildout of the City system. It identifies whether the pump stations are booster stations or hydropneumatic stations. The table also shows the low and high range for the discharge HGL on the maximum day at buildout. The discharge HGL will vary according to the hourly demand and reservoir re-filling conditions.

***Table 7***
***Existing and Future Pump Stations***

| Zone Pumping Into | Pump Station Name | Location | Status | Type | Purpose | Modeled Ground Elevation (ft) | Existing Facilities | | Buildout Pumping Requirements | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Pumps | Firm Capacity | Firm Capacity | Modeled Discharge HGL Range (ft) |
| 1 | Interim | Empire Ave and UPRR | Existing - temporary | Booster | Will be abandoned when PS 1.1 is in service. | 95 | 2 at 2,100 gpm each (150 HP each) | 3 mgd | To be abandoned in future. | |
| | PS 1.1 | New WTP | Under Construction | Booster | Supply treated surface water from new WTP. | 85 | None | None | 36 mgd | 204 - 303 |
| | PS 1.2 | At future Res 1.4 and 1.5 site | Future | Booster | Pump water out of ground-level tanks on east side of Zone 1. | 60 | None | None | 8.6 | 186 - 230 |
| 2 | PS 2.1 | South of Balfour Rd in Brentwood Country Club | Existing | Booster | Existing Zone 2 pumping capacity to re-fill existing Zone 2 reservoirs and pass water through to Zone 3. | 105 | 2 at 2,500 gpm each (125 HP each) | 3.6 mgd | 9.3 mgd (expand existing pump station to add 1 at 1500 gpm plus 1 standby at 2500 gpm) | 342 - 373 |
| | HPS 2.2 (Future PS 2.2) | Heidorn Ranch Rd near Lone Tree Way | Existing | Hydro-pneumatic now; convert to booster | Serves isolated northwest part of Zone 2. Will be converted to a booster station when area is connected with rest of Zone 2. | 140 | 2 at 700 gpm (40 HP) each; 1 at 3000 gpm (100 HP) | 2.0 mgd without fire pump | Replace pumps to provide 2.3 mgd (1600 gpm) total firm capacity plus standby pump to operate as booster station at TDH of 124 feet | 342 - 373 |
| | PS 2.3 | Vineyards at Fairview Ave | Future | Booster | Additional Zone 2 pumping capacity and re-fill Res 2.3. | 137 | None | None | 3.9 mgd (1 at 1700 gpm, 2 at 500 gpm, plus 1 standby at 1700 gpm) | 343 - 365 |
| 3 | HPS 3.1 | Near Res 1.1 and 1.2 (Brentwood Hills) | Existing | Hydro-pneumatic | Existing hydropneumatic station serving Zone 3 North. | 220 | 1 at 400 gpm and 1 at 3000 gpm | Station has adequate capacity for demand but no standby pumps. | Could consider adding standby pump. | 358 - 405 |
| | HPS 3.2 (Future PS 3.2) | Near Res 2.1 and 2.2 off Mountain View Rd | Existing | Hydro-pneumatic | Existing hydropneumatic station serving Zone 3 Central that would be converted to booster station when Res 3.1 constructed. | 320 | 2 at 640 gpm each and 1 at 3000 gpm | 1.84 mgd without fire pump | 2.0 mgd when converted to PS 3.2 Booster Station (2 at 700 gpm, plus a 700 gpm standby pump) | 452 - 461 |
| | HPS 3.3 | Vineyards | Future | Hydro-pneumatic | Future hydropneumatic station serving Zone 3 South. | 296 | None | None | 4.0 mgd total (1 at 200 gpm, 1 at 550 gpm, 1 at 2000 gpm, plus 550 gpm standby pump) | 400 - 426 |

As discussed in the Table 2 Planning Criteria, booster pump stations are sized for the maximum day demand (average flow on the maximum day) for the area that they are serving. Hydropneumatic stations must provide domestic pump capacity to meet domestic peak hour flows plus a standby pump, and a large fire pump and standby fire pump. Hydropneumatic stations also require backup power to ensure that pumps will be operational. The large fire pump and domestic pumps should be selected to provide the required flows under the same head conditions, i.e., that all the pumps can operate together to provide the maximum day demand plus fire flow. Booster pump stations that pump into zones with reservoir storage do not need fire pumps or backup power.

As discussed under Water Supply, a treated water pump station PS 1.1 is under construction at the future WTP site. This pump station will pump all treated surface water for the City into Zone 1, where it will be passed through as needed to other zones. The first stage of the pump station construction will consist of two 6 mgd pumps and one 15 mgd pump. Ultimately, the pump station will have a firm capacity of 36 mgd.

When the new treated water pump station is on-line, the existing interim pump station at Empire Avenue and the Union Pacific Railroad will no longer be needed for normal treated water supply. The City will have emergency intertie capabilities with Diablo Water District at this location; but the pumping facilities will be removed.

The only other pump station in Zone 1 will be PS 1.2 located at the future site for Reservoirs 1.4 and 1.5. This pump station will boost water from the ground-level tanks into the Zone 1 distribution system. Backup power must be provided at this location, since the storage must be available to meet peak demands and fire flows on the east side of Zone 1. Under the high demand conditions, there is not sufficient hydraulic capacity to move water all the way across Zone 1 from the gravity reservoirs located on the west side of the City. The pump station PS 1.2 must operate when hourly demand conditions exceed the flow provided by the water supply sources or during fire flows in the east side of Zone 1. It would also be operated as needed under lower demand conditions to turn over reservoir storage for water quality purposes.

The Zone 2 pump stations must provide maximum day capacity for all of Zone 2, as well as passing through the maximum day demand to Zone 3. Ultimately, a total of 15.5 mgd (10,765 gpm) firm pumping capacity is required in Zone 2, which is the sum of all the Zone 2 pump stations. This pumping capacity will be provided at several locations, as discussed below.

As part of the Vineyards development in the south part of the City, pumping facilities are being designed and constructed for Zone 2. The Zone 2 pump station serving the Vineyards, PS 2.3, will be a booster station pumping to the future Reservoir 2.3. The water system analysis assumed that PS 2.3 is set to operate on level control at Reservoir 2.3.

The best location hydraulically to provide the remaining pumping capacity needed for Zone 2 future needs is at PS 2.1. The existing PS 2.1 location is at a central location with large diameter pipelines extending to future development areas and future reservoir sites. The existing PS 2.1 structure should be expanded, or a parallel structure constructed in the vicinity that would operate as a single pump station.

In the near-term, Vineyard PS 2.3 will be used primarily to fill Reservoir 2.3. In the future with the new Reservoir 2.4 on the west side of Zone 2, there will be available capacity at the Vineyard PS 2.3 that can be used to help fill the other Zone 2 reservoirs including future Reservoir 2.4. In the future, due to proximity of PS 2.3 to the reservoir, Reservoir 2.3 will fill faster than the other reservoirs in Zone 2 and the pump will shut off. To maximize future use of the available capacity at PS 2.3, Reservoir 2.3 should be operated to prevent the reservoir from overfilling while PS 2.3 continues to operate to convey water into other parts of the zone to fill the other Zone 2 reservoirs. A general discussion of future operations at Reservoir 2.3 is provided below.

In the future, PS 2.3 would be set to operate on Reservoir 2.4 level control. The discharge head at PS 2.3 will be higher to overcome headlosses between it and Reservoir 2.4, since Reservoir 2.4 is located much farther away than Reservoir 2.3. In order to prevent Reservoir 2.3 from overfilling at the higher future HGL (higher than the overflow elevation of Reservoir 2.3), there will need to be some means of stopping flow into Reservoir 2.3 when it is full, but before Reservoir 2.4 is full. Two alternatives to control flow into the reservoir would be:

- Altitude valve that is hydraulically operated and pilot controlled. It is a non-throttling valve that closes at a high water level, and opens again when the pressure at the valve inlet is less than the reservoir pressure.

- Butterfly valve that is electronically controlled by telemetry to open/close automatically based on reservoir level sensors. This is an option if there is telemetry at the site.

Either of these valves would be located in the Reservoir 2.3 valve vault and would be automatically controlled. There should be provision for two-way flow at Reservoir 2.3 to allow for continuous flow out of Reservoir 2.3 into Zone 2, while preventing flow into the reservoir at high water level. In general, this can be accomplished by means of bifurcating the inlet/outlet pipe at the valve vault, with an altitude valve (or butterfly valve) on the inlet side, and a check valve on the outlet side that will allow flow out of the reservoir.

The upper northwest corner of Zone 2 is served by a hydropneumatic pump station HPS 2.2 that was constructed when this northern area was isolated from the rest of the Zone 2 system. Recent pipeline improvements to the south have connected this northern area to the rest of Zone 2. The location of HPS 2.2 in the northern part of Zone 2 helps to distribute pumping capacity throughout the zone in the future. In the future, HPS 2.2 could be converted to a booster pump station and the existing pumps replaced with pumps designed to operate within the Zone 2 hydraulic gradeline set by the Zone 2 reservoirs.

In the future, the only hydropneumatic zones will be in Zone 3. Zone 3 North is a very small zone served by an existing hydropneumatic pump station HPS 3.1. HPS 3.1 has adequate capacity to meet buildout needs for this subzone.

Zone 3 Central is currently served by an existing hydropneumatic pump station HPS 3.2. In the future, it is anticipated that there will be gravity reservoir storage for this part of Zone 3. At that time, the pump station can be converted to a booster station that would pump into the zone to re-fill the storage. Until gravity reservoir storage is in place, the hydropneumatic station will be needed.

As part of the Vineyards development in the south part of the City, pumping facilities are being designed and constructed for Zone 3 South. The Zone 3 pump station, HPS 3.3, will be a hydropneumatic station serving the Zone 3 portion of the Vineyards, which is Zone 3 South. It will be located near future Reservoir 2.3.

## Pipelines

The water system maps (plan-size) show the location and diameter of existing and future waterlines. The recommended distribution system shown on the water system maps provides hydraulic capacity for maximum day plus fire flows and peak hour flows.

Most of the future waterlines are located in future alignments. Only the major 12-inch and larger waterlines to serve currently undeveloped areas are shown on the maps. In addition, there will be smaller diameter in-tract improvements to provide service to individual lots.

There are some future waterlines that will be in existing alignments parallel to existing pipelines. These parallel pipelines in existing alignments are needed to provide adequate hydraulic capacity for future demands, in order to move water across the system from the water supply sources and reservoirs. Depending on the condition of the existing pipes, another option would be to replace the existing pipe with a larger pipe rather than installing a parallel pipe.

In general, the City's existing pipelines are adequately sized to meet buildout demands, in conjunction with installation of future pipelines as new areas are served and demand increase. There is one high ground elevation area in Zone 1 served by a pressure reducing station (PRV) from Zone 2 to improve the pressures in this area, which is already developed. The City indicates that pressures in this area under existing conditions are within industry standards with the PRV in operation. However, the model analysis indicates this high elevation area may experience lower pressures in the future under buildout peak hour demands, as discussed below.

The high ground elevation area in Zone 1 is located at the northwest corner of the intersection of Balfour Road and Fairview Avenue adjacent to Zone 2 near PS 2.1. This area is served by an existing pressure regulating station (PRV) between Zone 2 and Zone 1, shown on Figure 5. The existing PRV located near PS 2.1 provides higher pressure water from Zone 2 to this part of Zone 1 in order to maintain adequate pressures. The PRV opens when pressures in that part of Zone 1 fall to 42 psi and closes when pressures rise to 48 psi.

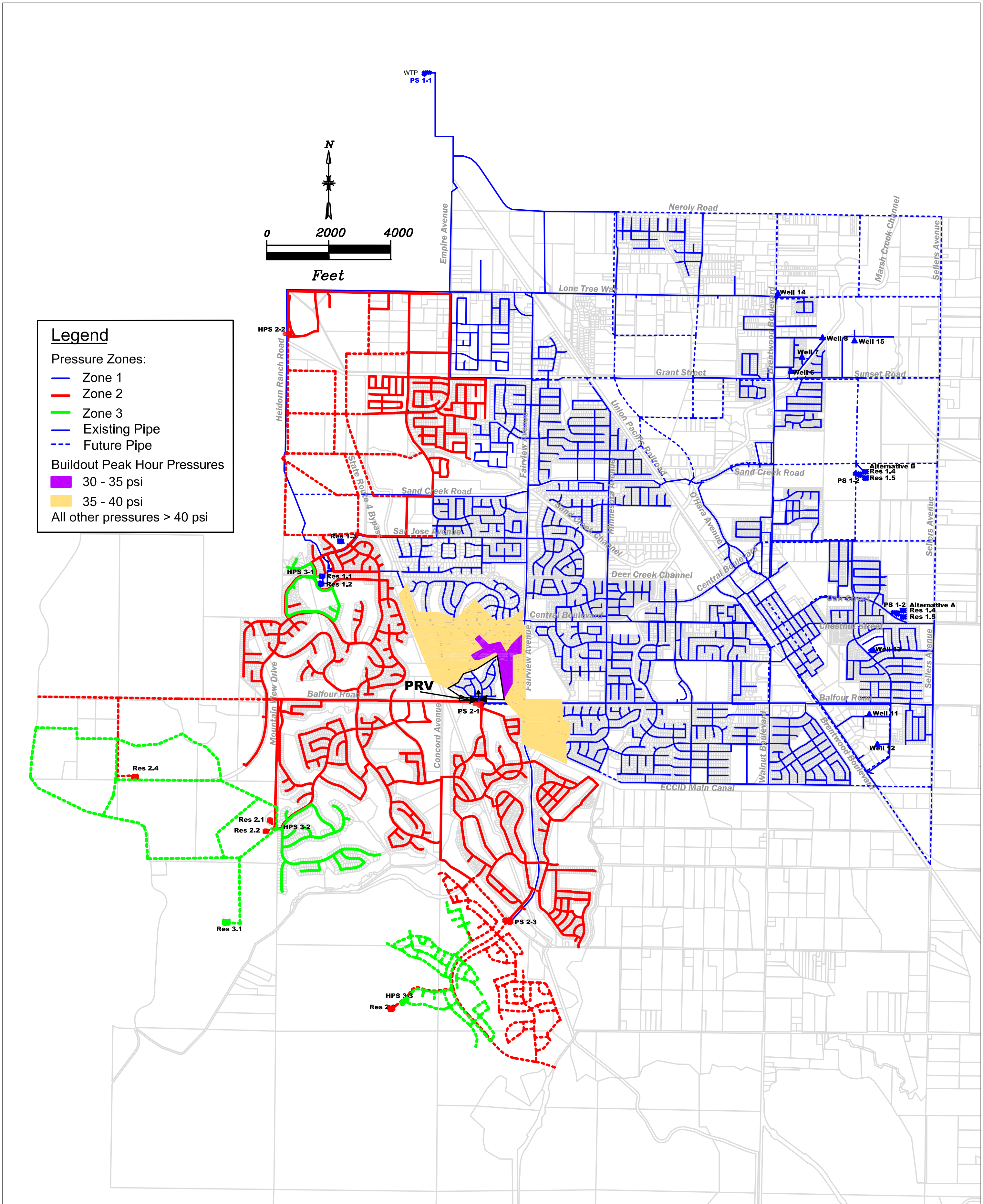
Under buildout peak hour demand conditions, the model analysis indicates that parts of this high elevation area may experience peak hour pressures between 30 to 40 psi, as shown on Figure 5. The PRV increased pressures to above 40 psi in the immediate vicinity of the PRV, and improved the pressures in high elevation area so that all buildout peak hour pressures were greater than 30 psi.

While the buildout peak hour pressures of 30 to 40 psi in the high elevation area do not meet the City's minimum pressure criterion of 40 psi, they are within the typical range used by other water agencies. Typically, pressures above 30 psi during the peak hour of the maximum day are considered acceptable for existing areas. As demands increase in the future, the City should watch pressures in this area to determine if there may be a need for other measures. More detailed investigation would be needed to determine specific improvements, which may potentially include higher pressure settings at the existing PRV, adding additional PRVs from other Zone 2 locations adjacent to the area, pipe improvements to reduce headlosses downstream of the PRV, or rezoning the high elevation area to Zone 2 and isolating it completely from Zone 1 to avoid recirculation of water between the zones at PS 2-1, which may impact Zone 2 pumping and storage capacities.

The model results indicated that there are short segments of existing pipes where headlosses exceed the criterion of 10 ft/1000 ft. However, those pipes do not need to be upsized since pressures in those areas exceed the 40 psi minimum pressure criterion. In addition, upsizing those pipes would not significantly improve hydraulic conditions in those areas.

Figure 6 shows the location of the fire flows applied at representative locations on the existing system in each zone for residential and non-residential uses. All locations could provide the required flow at the required pressure. The overall City transmission system is capable of supplying the required fire flows at adequate pressures. Future pipelines to serve new development or redevelopment areas will be designed to provide adequate fire flows.

CDM's scope and analysis did not include detailed analysis of localized improvements to meet fire flows at all locations in the existing system, e.g., all small diameter dead end pipes that were analyzed in the June 2003 draft plan. CDM did not re-run or check the previous fire flow analyses done for the existing system as described in the June 2003 draft report.




CDM

**Figure 5**
**Zone 1 High Elevation Area**

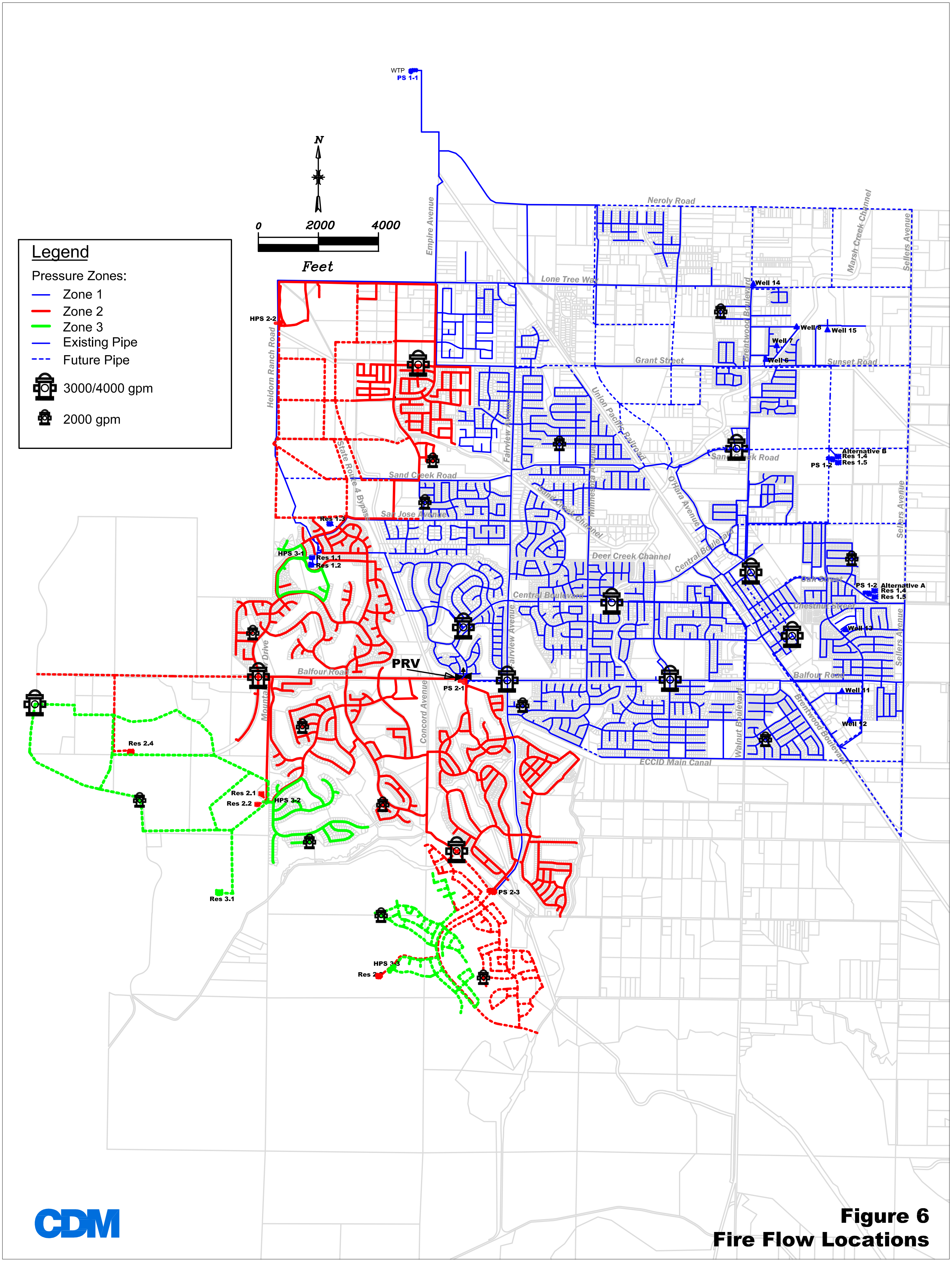



**Figure 6**
**Fire Flow Locations**

The June 2003 draft report stated that there were 69 locations (nodes) on existing pipes where a 2000 gpm fire flow could not be delivered at 20 psi residual pressure, but did not give specific node locations. In general, the report stated that these locations occurred at Zone 1 nodes located above the upper service elevation of 110 feet, and on small diameter (4-inch, 6-inch and 8-inch) dead end pipes. It would not be possible to provide the minimum 2000 gpm fire flow from any pipes in the system with similar conditions. However, this is only an issue if there is a hydrant(s) located on the pipes. If not, the pipes would be acceptable for domestic service only. Detailed information on fire hydrant locations is needed to conduct detailed fire flow analyses for localized areas.

Below are general recommendations based on CDM's analysis of the City's system and our experience with other water systems.

- For residential fire flows of 2000 gpm:
  - Looped pipelines will provide adequate fire flows. It is only smaller diameter dead-end pipes that may be inadequate.
  - In general, 6-inch and smaller pipes should be looped to be provide adequate fire flows. Fire hydrants should not be located on the smaller dead-end lines (except at the fire service itself). As a rule of thumb, a 6-inch dead-end line cannot have a hydrant located further than 175 feet from the start to provide a residual pressure of 20 psi assuming the pressure at the start of the dead-end line is 40 psi. If the pressure at the start of the dead-end is lower than 40 psi, then the allowable distance would be shorter.
  - 8-inch pipes should also be looped unless short enough to limit the total amount of headloss and provide adequate pressures. As a rule of thumb, an 8-inch dead-end line cannot have a hydrant located further than 700 feet from the start to provide a residual pressure of 20 psi assuming the pressure at the start of the dead-end line is 40 psi. If the pressure at the start of the dead-end is lower than 40 psi, then the allowable distance would be shorter.
- For high density residential and non-residential fire flows of 3000 gpm and higher:
  - Looped pipelines are needed provide adequate fire flows. Typically 12-inch or greater pipelines would be required.

Suggestions for resolving localized fire flow deficiencies that may be identified in the future include:

- Replacing or paralleling the small diameter pipes with larger pipes if there are hydrants located on these pipes. If there are no hydrants located on the pipes, then the small diameter pipes would be acceptable for domestic service.

- Looping the small diameter pipes if there are other pipes in the vicinity. This would also improve water quality by eliminating dead ends.
- Connecting high elevation areas to Zone 2 if there are adjacent Zone 2 pipelines by either permanently re-zoning the areas to Zone 2, or installing an emergency pressure reducing station from Zone 2 to be activated only if pressure in the area drops below a certain level under high demand conditions indicating a fire flow situation.