UCRL-JC-129738
PREPRINT

# Environmental Compliance Modeling at Lawrence Livermore National Laboratory

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This paper was prepared for submittal to the
***Conference on Stormwater and Related Modeling: Management and Impacts***
***Toronto, Ontario, Canada***
***February 15-20, 1998***

**February 1998**

Lawrence Livermore National Laboratory

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Abstract

This paper presents a post-rehabilitation monitoring and modeling study of the sanitary sewer system at Lawrence Livermore National Laboratory (LLNL). The study evaluated effectiveness of sewer system rehabilitation efforts and defined benchmarks for environmental success. A PCSWMM model for the sanitary sewer system was developed and applied to demonstrate the success of a $5 million rehabilitation effort. It determined that rainfall-dependent inflow and infiltration (RDI&I) had been reduced by 88%, and that system upgrades adequately manage predicted peak flows. An ongoing modeling and analysis program currently assists management in evaluating the system's needs for continuing maintenance and further upgrades. This paper also summarizes a 1989 study that evaluated data collected from December 1, 1988, to January 6, 1989, to determine the adequacy of the LLNL sewer system to accommodate present and future peak flows, and the Sanitary Sewer Rehabilitation (SSR) project, which took place from 1991 through 1995.

## 1. The LLNL Sanitary Sewer System

The sanitary sewer system at LLNL collects and transports wastewater from LLNL and Sandia National Laboratory (SNL) to the City of Livermore's collection system. Wastewater flow from LLNL and SNL is transported through a system of gravity sewers to the northwest corner of the LLNL site. From that point, wastewater enters the City of Livermore's collection system and is ultimately treated in the Livermore Water Reclamation Plant (LWRP).

The LLNL sanitary sewer system collects and transports all building wastewater except known hazardous wastewater, which is disposed of separately. Scheduled and unscheduled releases from some buildings enter the sewer system on a regular basis. The largest releases come from cooling-tower blowdown; this occurs once or twice a week and can contribute up to 300 gallons per minute (gpm), or 1,136 liters per minute (lpm).

The system (Figure 1), which was installed in stages beginning in 1942, comprises mainlines, building laterals, manholes, waste retention facilities, a monitoring station, and a diversion facility. Overall, it contains 56,000 linear feet (17,000 meters) of pipelines (primarily vitrified clay) that range from 4 to 15 inches (10 to 38 cm) in diameter. Building laterals (the pipe from the main line to the first building cleanout) are either 4 or 6 inches (10 or 15 cm) in diameter. The system includes 36,220 linear feet (11,000 meters) of building laterals, 271 manholes, and about 500 cleanouts (primarily in the building laterals).

## 2. Earlier Studies and Rehabilitation Efforts

### 2.1 1989 Study of the Sanitary Sewer System

The LLNL sanitary sewer system was studied in 1988–1989 ($CH_2$MHill, 1990a, b and c) to determine its adequacy to accommodate present and future peak flows. It was then estimated that by the year 2008, the worker population at LLNL would increase to 12,400 and the building area would expand to 6.9 million $ft^2$ (641,000 $m^{2)}$. A 5-week flow monitoring program was implemented at that time to determine existing and projected flow characteristics.

- The LLNL site was divided into 8 drainage basins using 10 flow monitors. Flow components, including RDI&I, were quantified at each location. From the data obtained during this study, design flows, including peak sanitary base flow and an RDI&I allowance, were estimated. The estimates were derived by computer simulation analyses for then-current (1988–1989), 5-year (1993), and 20- year (2008) scenarios.
- Average base flow at each monitoring site was determined by analyzing monitored flow data from dry days. RDI&I for a given rainy day was calculated by subtracting the average base flow from the total flow.
- As part of the 1989 study, $CH_2$MHill contract personnel evaluated historical data and estimated that 4.3% of the rain that falling on the LLNL/SNL sites entered the sanitary sewer system. Researchers asserted that this figure was equivalent to a peak rainfall-induced flow rate of 4.38 million gallons per day (mgd), or 16.6 million liters per day (mld), compared to a peak base-flow rate that they estimated at 0.76 mgd (2.88 mld).
- The $CH_2$MHill study further suggested that:

  1. The sanitary system was sufficient to transport peak dry-weather flow, but;

  2. The system had insufficient capacity to transport projected flows under peak wet-weather conditions (given the amount of RDI&I that would enter the sewer system during a 10- year storm event). This lack of capacity was true for site development conditions existing in 1988–1989, and would be exacerbated by any future growth.

  3. A 55 to 70% reduction in RDI&I could be achieved by improving the sewer system.
- $CH_2$MHill researchers also identified and evaluated various system improvements to correct these deficiencies.

### 2.2 Sanitary Sewer Rehabilitation Project

During the SSR project (Vellinger et al., 1995), LLNL personnel pursued the solutions recommended by $CH_2$MHill by assessing the condition of the system and proposing specific design solutions for rehabilitating the piping. The intended result was compliance with the Porter-Cologne California Water Quality Control Act of 1969 (Porter-Cologne) prohibiting the discharge to the environment of any waste that may potentially adversely impact the quality of waters in California (e.g., no exfiltration is allowed). Federal and local compliance requirements were also considered. The SSR project, therefore, targeted areas with the highest exfiltration potential.

Investigators first performed a detailed analysis of the data generated from smoke tests, flow monitoring, and dye tests; from observations obtained via closed-circuit television; and from water-balance investigations. Sewer pipelines were repaired on a priority-ranked schedule. Direct outdoor connections to the sanitary sewer were identified and re-routed. More than 130 point repairs were completed, 24,000 linear feet (7,300 meters) of sewer mains and laterals were inversion-lined, 42 lateral lines were replaced, and 150 cleanouts and 10 new manholes were installed for site-wide access. Only one building (B251) still employed a direct rain water inflow to the sanitary sewer (from three roof drains). After extensive review and safety consideration the SSR team agreed to allow this connection.

The project's "zero exfiltration" goal was not fully achieved for two reasons:

- $8.5 million cut in SSR funding; and
- zero exfiltration is simply not practical for most existing sanitary sewer system rehabilitations.

## 3. Post-Rehabilitation Study

### 3.1 Goals

The primary goal of the post-rehabilitation study was to evaluate the effectiveness of the SSR project. Study aspects included:

- Duplicating the monitoring conducted in the earlier study by $CH_2$MHill (to the greatest extent possible, given changes in the sanitary sewer system).
- Quantifying the efficacy of the rehabilitation effort by

  -- measuring reductions in RDI&I since measurements by $CH_2$MHill in 1988–1989; and

  -- evaluating the ability of the present system to handle maximum predicted capacity.
- Determining whether peak flows will be less than 1,170 gpm (4,430 lpm). This was important because the City of Livermore has reserved capacity to treat a maximum flow rate of 1,170 gpm from LLNL/SNL.
- Evaluating compliance with the Porter-Cologne Act, specifically regarding the zero exfiltration goal.

## 3.2 Flow Data Analysis

The daily peak flow rate during the monitored period (January 13 through February 15, 1995) was 860 gpm (3,260 lpm). The average daily peak flow during the same period was 593 gpm (2,240 lpm). Flow typically peaks at 600 gpm (2,270 lpm); daily low flows drop to 30 gpm (110 lpm).

Raw data from ISCO flow monitoring devices were converted from MS-DOS files to MS-Excel format for analysis and plotting. Flow data plots for each location, over the entire monitoring period, were closely examined to identify anomalies. Anomalies were compared with information from field logbooks (e.g., when clogged flow meter tubes were observed and cleared) to resolve apparent discrepancies.

Using monitoring program data, investigators quantified the flow components (sanitary flow and RDI&I) for each location. Design flows were then calculated for existing and future (year 2008) conditions, and these were used to assess the adequacy of the collection system under each scenario.

## 3.3 Average Base Flow

### 3.3.1 Choice of Dry-Weather Days

The average base flow (ABF) at each flow monitoring site was developed by analyzing dry-day flow data. Dry days were identified by data from the LLNL meteorological tower, which continuously monitors air temperature, wind speed, and rainfall.

For ABF calculations, a dry day was defined as a midnight-to-midnight period with no rain, preceded by 24 hours of no rain (to minimize the amount of long-term infiltration). The meteorological tower measures rainfall amounts as slight as 0.01 inches (0.0254 cm). For the purposes of this study, such minute rainfall was considered insignificant; therefore, “no rain” was defined as rainfall amounts less than 0.02 inches/hour and less than 0.05 inches/day (less than 0.0508 cm/hour and less than 0.127 cm/day). Selected ABF calculation dry days during the study period were January 18–20, January 29, and February 2–6.

### 3.3.2 Discharges

To more accurately quantify the amount of RDI&I, known large batch discharges (greater than 10,000 gallons [37,850 liters]) were subtracted from the flows. (Such discharges typically result from cooling-tower blowdown, retention tank releases, or treatment facility releases.) Correcting for the reported batch discharges required identifying the affected zones, computing an approximate discharge duration, and converting the reported volume to a discharge rate. Discharge volumes were then subtracted from the basin hydrographs to produce corrected-flow hydrographs. This procedure minimizes variability in the data and allows the accurate detection of a smaller RDI&I. Discharges are summarized in Table 1.

Table 1. Batch releases to the sanitary sewer system.

| Location | Volume (gallons) | Dates | Time | Downstream monitor | Source of batch release |
|---|---|---|---|---|---|
| TFF[a] | 8,450 | 1/9/95 | | 7 | |
| 612 | 5,000 | 1/10/95 | 930 | 1 | Berm water |
| 612 | 1,400 | 1/10/95 | 1330 | 1 | S300 steam cleaning |
| 222 | 4,500 | 1/10/95 | 1330 | 5 | Retention tank |
| 291 | 7,900 | 1/10/95–1/12/95 | | 2 | Cooling-tower blowdown |
| TFF[a] | 21,900 | 1/11/95 | | 7 | |
| 612 | 8,000 | 1/12/95 | 930 | 1 | Retention tank |
| 291 | 12,500 | 1/12/95–1/13/95 | | 2 | Cooling-tower blowdown |
| 222 | 4,000 | 1/13/95 | 1000 | 5 | Retention tank |
| 291 | 100 | 1/13/95–1/17/95 | | 2 | Cooling-tower blowdown |
| 291 | 8,600 | 1/17/95–1/18/95 | | 2 | Cooling-tower blowdown |
| 291 | 14,000 | 1/18/95 | | 2 | Cooling-tower blowdown |
| 153 | 1,850 | 1/18/95 | 830 | 6 | Retention tank |
| 325 | 6,900 | 1/18/95–1/19/95 | | 5 | Cooling-tower blowdown |
| 612 | 43,000 | 1/19/95 | 1340 | 1 | Retention tank |
| 291 | 11,000 | 1/19/95 | | 2 | Cooling-tower blowdown |
| 291 | 34,600 | 1/20/95–1/23/95 | | 2 | Cooling-tower blowdown |
| 612 | 3,500 | 1/24/95 | 1050 | 1 | Retention tank |
| 514 | 990 | 1/24/95 | 1535 | 1 | Retention tank |
| 325 | 4,000 | 1/24/95–1/25/95 | | 5 | Cooling-tower blowdown |
| 514 | 2,000 | 1/25/95 | 1045 | 1 | Retention tank |
| 514 | 2,400 | 1/26/95 | 1045 | 1 | Retention tank |
| 325 | 1,000 | 1/26/95–1/27/95 | | 5 | Cooling-tower blowdown |
| 612 | 5,800 | 1/27/95 | 1315 | 1 | Retention tank |
| 291 | 75,300 | 1/28/95 | 645 | 2 | Cooling-tower blowdown |
| 325 | 5,000 | 1/28/95–1/30/95 | | 5 | Cooling-tower blowdown |
| 151 | 2,490 | 1/31/95 | 1440 | 4 | Retention tank |
| 325 | 2,000 | 1/31/95 | | 5 | Cooling-tower blowdown |
| 325 | 61,000 | 2/1/95 | | 5 | Cooling-tower blowdown |
| TFF[a] | 28,980 | 2/2/95 | | 7 | |
| 325 | 2,000 | 2/2/95 | | 5 | Cooling-tower blowdown |
| TFF[a] | 30,620 | 2/3/95 | | 7 | |
| 612 | 4,000 | 2/3/95 | 1100 | 1 | Retention tank |
| TFF[a] | 22,800 | 2/6/95 | | 7 | |
| TFF[a] | 26,080 | 2/7/95 | | 7 | |
| 514 | 2,270 | 2/7/95 | 1100 | 1 | Retention tank |
| 325 | 2,000 | 2/7/95–2/8/95 | | 5 | Cooling-tower blowdown |
| TFF[a] | 31,670 | 2/9/95 | | 7 | |
| 291 | 3,000 | 2/9/95–2/10/95 | | 2 | Cooling-tower blowdown |
| 325 | 19,000 | 2/9/95–2/10/95 | | 5 | Cooling-tower blowdown |
| TFF[a] | 14,210 | 2/10/95 | | 7 | |

| | | | | | |
|---|---|---|---|---|---|
| 612 | 2,166 | 2/10/95 | 1300 | 1 | Retention tank |
| 291 | 2,400 | 2/10/95–2/13/95 | 0800 2/10 –0800 2/13 | 2 | Cooling-tower blowdown |

Table 1. Batch releases to the sanitary sewer system (continued).

| Location | Volume (gallons) | Dates | Time | Downstream monitor | Source of batch release |
|---|---|---|---|---|---|
| 325 | 113,000 | 2/10/95–2/13/95 | | 5 | Cooling-tower blowdown |
| TFF[a] | 22,320 | 2/13/95 | | 7 | |
| 514 | 1,200 | 2/13/95 | 1030 | 1 | Retention tank |
| 291 | 7,000 | 2/13/95–2/14/95 | 0800 2/13 –0800 2/14 | 2 | Cooling-tower blowdown |
| TFF[a] | 21,790 | 2/14/95 | | 7 | |
| 291 | 5,700 | 2/14/95–2/15/95 | 0800 2/14 –0800 2/15 | 2 | Cooling-tower blowdown |
| TFF[a] | 29,750 | 2/15/95 | | 7 | |
| 514 | 1,381 | 2/16/95 | 1410 | 1 | Retention tank |
| 325 | 14,000 | 2/16/95–2/17/95 | | 5 | Cooling-tower blowdown |
| 325 | 80,000 | 2/17/95–2/21/95 | | 5 | Cooling-tower blowdown |
| 325 | 30,000 | 2/21/95–2/22/95 | | 5 | Cooling-tower blowdown |
| 514 | 1,030 | 2/22/95 | 1355 | 1 | Retention tank |
| 325 | 12,000 | 2/22/95–2/23/95 | | 5 | Cooling-tower blowdown |
| 612 | 5,000 | 2/23/95 | 900 | 1 | Retention tank |

[a] TFF = Treatment Facility F.

### 3.3.3 Other Considerations

Additionally, investigators discounted days on which a known problem occurred with the monitor (such as a clogged tube) and days with unexplained flow irregularities. For example, data from Monitor 6 were highly irregular, perhaps relating to the application of Manning's equation to the data. LLNL monitors measure depth, not flow; Manning's equation is then used to convert depth to flow. Manning's equation, however, requires certain conditions for accurate application, including a length of uninterrupted upgradient pipe of constant slope, and no change in slope downgradient or upgradient of the monitored location. It may be that these conditions were not met at Monitor 6. Regardless of the cause, researchers decided that the data were too erratic to be trusted. Therefore, data from a permanent flow monitoring station in Building 196 were used in place of Monitor 6 data. Building 196 was considered an equivalent location because both Monitor 6 and the monitor in Building 196 measure the combined sanitary sewer flow just before that flow leaves the LLNL site.

### 3.3.4 Calculation

After the above screenings and adjustments, data for the remaining days were averaged to create an ABF for each basin and for the LLNL/SNL site as a whole. Weekdays and weekends were treated separately to produce discrete hydrographs for weekday ABF and weekend ABF. Because of differences between the data collection schemes at the Building 196 flow monitoring station and at the basin monitors, data from the basin monitors were stored and averaged on a 15-minute basis, and site-wide data were stored and averaged on a 5-minute basis.

#### 3.3.5 Scaling

Basins 1–5, 7, 8, and 10 all flow into Basin 6. Basin 10 is SNL. Basin 9 is no longer used, because the corresponding sewer line in the 1989 study has since been abandoned. Basin 6 flow can thus be obtained by summing the flows in the upgradient basins and subtracting this total from the Building 196 flow. Originally, however, when the ABF calculation was complete, the upgradient flow sum was greater than the Building 196 flow.

The investigating team believes that some of this discrepancy can be attributed to inaccuracy because the basin monitors measured only depth. A velocity factor was obtained by placing dye in the manholes upstream of the monitoring locations and measuring its travel time to the next downstream manhole. Based in part on the measured velocity, scale factors were applied to the flow results in order to achieve a mass balance. The scaled data were used to recalculate the ABF and in all subsequent plots and calculations.

### 3.4 Rainfall Dependent Infiltration and Inflow (RDI&I)

RDI&I is the volume of storm-induced ground water and storm water runoff that infiltrates the sanitary sewer collection system during and following rain events. It is determined by subtracting the ABF from the wet-weather hydrograph figure for a specific day.

#### 3.4.1 Choice of Wet-Weather Days

As with the ABF calculation, researchers discounted large batch discharges and anomalies in this wet-weather flow data. Initially, the team used data from a storm on January 23–24. This was one of the largest (0.53 inches [1.35 cm] of rain) and most intense weekday storms during the monitoring period. For some basins, data problems on these dates could not be resolved; for those affected basins data from January 27 were substituted. On this date, rainfall was 0.55 inches (1.4 cm), but it fell over a longer time period.

Because a larger (0.8 inch [2.03 cm]) storm occurred on the weekend of January 14–15, RDI&I analysis was also conducted for the weekend flows. Finally, an even larger storm (1.7 inch [4.31 cm]) occurred January 9–10. Although flow data by basin were not collected at that time, investigators wished to examine the effects of larger storms on RDI&I, and therefore used data from the Building 196 monitor to calculate site-wide RDI&I.

For each date, a time period from the first measurable (0.01 inch [0.025 cm]) rainfall until 12 hours after the last measurable rainfall was used in the RDI&I calculation. The 12-hour period (consistent with the 1989 study) ensures that all long-term RDI&I was included.

Figure 2 presents the RDI&I data for January 23, 1995, for the entire LLNL/SNL site. This plot represents the period from 6:00 p.m. January 23 through 6:00 p.m. January 24. To properly couple the wet day with the ABF, the portion of the storm that occurred on January 23 was plotted to the right of the portion that occurred on January 24. Any wet-weather flow greater than the ABF was assumed to be RDI&I. The area under the RDI&I curve, then, gives the total volume of RDI&I for the storm event.

3.4.2 R-value

The absolute volume of RDI&I will, of course, also depend on the size of the storm. All else being equal, it is generally assumed, for example, that a doubling in the rainfall volume will result in a doubling of the RDI&I volume. RDI&I volumes, therefore, are often scaled by dividing by the rainfall volume:

$$R = \frac{\text{RDI\&I volume}}{\text{Rainfall volume}} \quad (1)$$

This unitless R-value then provides a measure of RDI&I that can be used to compare different storm events and different basins.

R-values were developed for each of the eight basins and for the LLNL/SNL sites as a whole. R-values are presented in Table 2. For each storm event, the amount of rainfall is given, followed by the LLNL/SNL site-wide R-value, then the basin R-values. The R-value for LLNL/SNL ranged from 0.22 to 0.91, with the average being about 0.5.

**Table 2. Calculated R-values[a] for selected storm events.**

| **Date:** | **1/9/95–1/10/95** | **1/14/95–1/15/95 (weekend)** | **1/23/95–-1/24/95 or 1/27/95** |
|---|---|---|---|
| **Rainfall:** | **1.7 in.** | **0.8 in.** | **0.5 in.** |
| **R-values:** | | | |
| **Site-wide:** | 0.22[b] | 0.91[b] | 0.36[b] |
| **By basin:** | | | |
| **L1** | na[c] | 0.57 | 0.84 |
| **L2** | na | 1.43 | 0.14 |
| **L3** | na | 0.43 | 1.00 |
| **L4** | na | 0.52 | 0.89 |
| **L5** | na | 1.57 | 0.12 |
| **L6** | na | 1.56 | 0.34 |
| **L7** | na | 0.93 | 0.04 |
| **L8** | na | 0.10 | 0.79 |
| **SNL10** | na | 0.85 | 0.02 |

[a] Where R = a unitless value that provides a measure of rainfall-dependent infiltration and inflow (RDI&I).

[b] Value based on Building 196 data.
[c] na = data not available.

### 3.4.3 Sources of Uncertainty

RDI&I volume depends on the physical condition of the sanitary sewer system and on the number and type of storm water sources. RDI&I volume entering a system is also highly dependent on the soil conditions antecedent to a storm event. If the soil is already saturated, RDI&I volume is maximal. Conversely, if the soil is dry, much of the rainfall is absorbed by the soil, and RDI&I is minimal. During the 1989 study, soil conditions were relatively dry. Therefore, $CH_2$MHill used historical data to scale the results obtained during that study to comparable values, given saturated soils. During the current study, soil conditions were relatively saturated; it can, therefore, be assumed that the RDI&I value is at or near maximum.

A number of potential sources of uncertainty could affect both the calculation of the base flow and RDI&I.

- Undocumented batch discharges on a dry day would increase the calculated base flow, thereby decreasing the calculated RDI&I. Conversely, undocumented batch discharges during a rain event would increase the wet-weather flow, potentially be counted as part of the RDI&I, and could result in an over-estimation of the RDI&I.
- Clogged tubes or other operational problems with the flow monitors, shifts in recorded flows due to the "drift" in the bubble rate or to recalibration, and, in general, the accuracy of the ISCO flow monitors all contribute to the uncertainty. Similarly, Building 196 data are accurate to about ±30 gpm (110 lpm).
- Variability in daily base flow (due to day of week, number of people on site, different operations, etc.) and soil conditions (as mentioned above) add to the uncertainty as well. For example, because the soil was saturated even on dry days, base-flow data may have included long-term infiltration, resulting in an over-estimation of base flow and a corresponding under-estimation of RDI&I.
- Finally, the R-value calculation assumes that the "non-rain" flow (i.e., the sanitary sewer flow resulting from LLNL/SNL operations) during the storm event is equal to the ABF calculation based on the dry days. Thus, variation in non-rain flow contributes to uncertainty in the calculated RDI&I.

## 3.5 The Simulation Model

### 3.5.1 Comparison to 1989 Study

The simulation model used in the 1989 study is proprietary to $CH_2$MHill. Because this contractor is not involved in the current study, the same model could not be used. Instead, LLNL researchers used the Personal Computer Storm Water Management Model (PCSWMM) to develop a dynamic, hydraulic-routing model of the site's sanitary sewer system.

To ensure that changes in results between the two studies could not be attributed to differences in the models, the PCSWMM model was first run using 1988–1989 conditions. To model the LLNL system, the team keyed approximately 450 manholes and pipe segments, along with their relative positions, into PCSWMM data files. For each pipe segment, investigators entered length, radius, and slope. These data were based primarily on printouts from $CH_2$MHill model runs and, in part, on maps from the time. For the 1988–1989 condition, $CH_2$MHill assumed that the Manning's roughness coefficient for all pipes was 0.013. LLNL researchers used the same value in this comparison of the two models.

The 1988–1989 peak-flow condition was simulated using the new model; results were virtually identical to $CH_2$MHill results. Thus, investigators confirmed that the two models produce comparable results.

#### 3.5.2 Hydraulic Model Update

Next, it was necessary to update the model to 1995 conditions. Most changes resulted from lining the pipes, i.e., the pipes now had reduced inside radii and different Manning's roughness coefficients. Two types of lining had been used, and each had a different thickness. To update the PCSWMM files, a current site map showing locations of the two types of lining was used.

In addition, the PCSWMM files were changed to reflect the numerous places where pipes had been eliminated or added or manholes had been added.

#### 3.5.3 Design Flow Determinations

The same method was used to quantify design wastewater flows at LLNL and SNL in both studies. That is, design flows included peak sanitary base flow and an RDI&I allowance, but did not allow for batch discharges. Wastewater flow was determined for both existing conditions and future conditions (year 2008). Future development information was obtained from the *Lawrence Livermore National Laboratory Site Development Plan (Calendar Year 1995)* (Lawrence Livermore National Laboratory, 1995b), and from the *Sandia National Laboratories' 1994 Site Development Plan* (Sandia National Laboratory, 1994).

Refer to Tables 3 and 4 for the design flows derived as explained in the following paragraphs.

#### 3.5.4 Design Base Flow

Although growth was predicted at the time the model was run, this growth was based on unfunded projects. Growth may in fact be relatively flat. Current data were used to update the wastewater flow for each building in the 1989 study so as to reflect existing conditions. The

*Associate Director Responsibility Report First Quarter Fiscal Year (FY) 1995* and *Retired Buildings/Trailers* (Lawrence Livermore National Laboratory, 1995) provided update information for LLNL. Existing developed condition information for SNL was obtained from the *Sandia National Laboratories' 1994 Site Development Plan* (Sandia National Laboratory, 1994).

**Table 3. Design flows for current (1995) scenario.**

| **Basin** | **ABF[a] (mgd)** | **PDWF[b] (mgd)** | **Peak RDI&I[c] (mgd)** | **PWWF[d] (mgd)** |
|---|---|---|---|---|
| L1 | 0.03071 | 0.07328 | 0.14929 | 0.22257 |
| L2 | 0.03114 | 0.07415 | 0.00776 | 0.08191 |
| L3 | 0.01248 | 0.03399 | 0.05623 | 0.09022 |
| L4 | 0.02167 | 0.05443 | 0.06850 | 0.12293 |
| L5 | 0.07475 | 0.15650 | 0.01680 | 0.17330 |
| L6 | 0.01915 | 0.04898 | 0.05997 | 0.10895 |
| L7 | 0.03324 | 0.07840 | 0.00394 | 0.08234 |
| L8 | 0.03033 | 0.07250 | 0.09500 | 0.16750 |
| SNL | 0.05113 | 0.11319 | 0.00084 | 0.11403 |
| **Total:** | 0.30459 | 0.70542 | 0.45833 | 1.16375 |

[a] ABF = average base flow.
[b] PDWF = peak dry-weather flow.
[c] RDI&I = rainfall-dependent infiltration and inflow.
[d] PWWF = peak wet-weather flow.

**Table 4. Design flows for future (year 2008) scenario.**

| **Basin** | **ABF[a] (mgd)** | **PDWF[b] (mgd)** | **Peak RDI&I[c] (mgd)** | **PWWF[d] (mgd)** |
|---|---|---|---|---|
| L1 | 0.05889 | 0.12769 | 0.14929 | 0.27698 |
| L2 | 0.05479 | 0.12007 | 0.00776 | 0.12783 |
| L3 | 0.01962 | 0.05001 | 0.05623 | 0.10624 |
| L4 | 0.03397 | 0.07987 | 0.06850 | 0.14837 |
| L5 | 0.08636 | 0.17701 | 0.01680 | 0.19381 |
| L6 | 0.10328 | 0.20620 | 0.05997 | 0.26617 |
| L7 | 0.04259 | 0.09686 | 0.00394 | 0.10080 |
| L8 | 0.05455 | 0.11962 | 0.09500 | 0.21462 |
| SNL | 0.05196 | 0.11477 | 0.00084 | 0.11561 |
| **Total:** | 0.50602 | 1.09210 | 0.45833 | 1.55043 |

[a] ABF = average base flow.
[b] PDWF = peak dry-weather flow.
[c] RDI&I = rainfall-dependent infiltration and inflow.
[d] PWWF = peak wet-weather flow.

There are 173 permanent buildings and 331 temporary structures at LLNL, for a total of about 5.7 million $ft^2$ (530,000 $m^2$). Flows were distributed to individual manholes on the basis of the buildings connected to or contributing to the particular manhole, as follows.

- Sanitary flow was estimated for each building at LLNL on the basis of the net area of the building and the number of personnel in it.
- Unit flow rates in gallons per square foot per day (gpsfd) or gallons per capita per day (gpcd) were applied to each building depending on its use.
- Each building was assigned a use code and associated unit flow rate as defined in Table 4.

**Table 4. Building uses and associated flow rates.**

| Building use | Use code | Associated flow rate |
|---|---|---|
| No sanitary flow | N | None |
| Office | O | • 0.0607 gpsfd[a] (2.47 lpsmd[b]) plus 14.9 gpcd[c] (56.4 lpcd[d]) if both population and area are known for a building.<br>• 0.121 gpsfd (4.93 lpsmd) if only area is known. |
| Laboratory | L | 0.161 gpsfd (6.56 lpsmd) |
| Heavy sanitary contributor | H | 0.243 gpsfd (9.90 lpsmd) |

[a] gpsfd = gallons per square foot per day.

[b]lpsfd = gallons per square foot per day.

[c]gpcd = gallons per capita per day.

[d]lpcd = liters per capita per day.

### 3.5.5 Future Sanitary Flows

Data from the most recent *LLNL Site Development Plan* (Lawrence Livermore National Laboratory, 1995b) were used to for project peak dry-weather and wet-weather flows for the year 2008. Each building was assigned a use code and an assumed discharge manhole. The unit flow rates presented in the existing condition were applied to the future buildings to determine the average flow. Future RDI&I for each basin was assumed to remain constant at existing levels. Based on the Site Planning 1995 draft of the Line-Item Construction Plan Project Summary (included in Lawrence Livermore National Laboratory, 1995b), these buildings will total approximately 1.5 million $ft^2$ (139,350 $m^2$).

### 3.5.6 Design Flows for the 10-year Storm Event

Following the example of the 1989 study, I selected the 10-year, 4-hour storm as the design storm event. In 1989, this choice had been based on criteria used by the San Francisco Regional Water Quality Control Board (Regional Board). At that time, the Regional Board had allowed predicted sanitary sewer overflows at intervals ranging from 5 to 20 years. For the East Bay

planning community, the Regional Board had recommended allowing overflows in sensitive areas in 10- to 20-year intervals. Because LLNL is considered a sensitive area, a 10-year design storm was selected. Currently, the Regional Board acknowledges the existence of overflows, but does not make specific allowances.
For the Livermore area, the 10-year storm would have an average intensity of 0.32 inches/hour (0.81 cm/hour) for a 4-hour period. The peak hourly RDI&I responses developed for each basin for the design storm event were based on storm characteristics determined from flow monitoring data for 1995. That is, the same percentage of rainfall was assumed to enter the sanitary sewer system of each basin as was determined for the January 23–24 and January 27 storms. Future RDI&I for each basin was assumed to remain constant at existing levels.

Current and future peak wet-weather flows were then obtained by adding RDI&I to current and peak sanitary flows, respectively.

(2) $PDWF = PSF + GWI,$

(3) $PWWF = PSF + RDI\&I,$

where,

PDWF = peak dry-weather flow (in mgd),
PWWF = peak wet-weather flow ( in mgd),
GWI = ground water infiltration (in mgd), and
PSF = peak sanitary flow (in mgd).

Because the 1989 study had determined that ground water infiltration is not present in the LLNL collection system, it was not considered in 1995 (i.e., PDWF = PSF).

For the 1989 study, $CH_2MHill$ had developed an equation that related peak sanitary flow to average base flow (ABF). This equation is based on a regression between the two variables, using data from the 10 basins. The present study preserved the same formula to estimate peak sanitary flows:

(4) $PSF = 1.43\ (ABF)^{0.853}\ .$

For the design average base flow, the peaks of each contributing flow component were assumed to occur simultaneously. That is, the peak RDI&I resulting from the 10-year storm would occur at the same time as the peak daily sanitary flow. The probability of the two events occurring simultaneously once during a 20-year period was reported in the 1989 study as 8%, corresponding to a recurrence interval of 12 years.

To determine the capacity deficiencies for existing and future sanitary flows, PCSWMM routed design flows through the collection system. As in the 1989 study, the system was divided into eight drainage basins based on the locations of the flow monitors. Design flows were distributed through each basin by assigning appropriate flows to each manhole. Peak base flow was distributed according to the estimated building flows already described. Peak wet-weather flow for each manhole was determined as follows.

- The rainfall rate (inches/hour) was converted to a volume rate (cubic feet per second, or cfs) by multiplying the rainfall rate by the area of the drainage basin (with appropriate unit conversions).
- This volume rate was converted to an RDI&I rate (cfs) per basin, by multiplying it by the R-value for that basin.
- Finally, the RDI&I rate was distributed throughout the basin in proportion to the length of pipe upgradient from a given manhole. That is, if the pipe length upgradient of a particular manhole represented 5% of the pipe length modeled for the basin, then 5% of the RDI&I was assigned to that manhole.

### 3.5.7 Scenarios

Scenarios modeled included 1989 conditions, current (1995) peak dry- and wet-weather flows, and future (year 2008) peak dry- and wet-weather flows. Peak dry- and wet-weather flows were simulated together by running the model at peak dry-weather flow conditions and then superimposing on this the 4-hour, 10-year storm event RDI&I. A 24-hour simulation was used, with peak dry-weather flows as input from 12 a.m. to 10 a.m. and 2 p.m. to 12 a.m., and peak wet-weather flows as input from 10 a.m. to 2 p.m.

## 3.6 Results

### 3.6.1 Capacity

The capacity of each pipe segment was determined by PCSWMM according to Manning’s equation. The maximum capacity of LLNL pipes (flowing full with no overflow) was calculated as 1,215 gpm (4,600 lpm). To identify deficiencies, the routed flow to each segment was compared to the capacity.

Locations near capacity are summarized in Table 5. Locations downgradient from Building 196 (pipes leading to manholes 155 and 156) were at as much as 50 to 60% of their capacity. In the peak-flow condition, some locations upgradient of Building 196 achieved approximately 75% of capacity.

**Table 5. Pipes near capacity in future peak-flow scenario (year 2008).**

| Downgradient manhole[a] | Theoretical maximum capacity (cfs)[b] | Modeled maximum flow (cfs) | Percent of capacity[c] |
|---|---|---|---|
| 191 | 0.86 | 0.646 | 0.75 |
| 192 | 0.85 | 0.629 | 0.74 |
| 193 | 0.9 | 0.627 | 0.70 |
| 88 | 1 | 0.619 | 0.62 |
| 89 | 1 | 0.611 | 0.61 |
| 156 | 2.7 | 1.6 | 0.59 |
| 50 | 3.4 | 1.953 | 0.57 |
| 190 | 1.2 | 0.648 | 0.54 |
| 155 | 3.2 | 1.6 | 0.50 |
| 292 | 3.4 | 1.59 | 0.47 |
| 293 | 3.4 | 1.59 | 0.47 |

[a] Each pipe is identified by the manhole immediately downgradient from it.
[b] Where 1 $ft^3$/s (cfs ) = 448.86 gallons/min (gpm).
[c] Ratio of maximum flow to capacity.

### 3.6.2 Flow

Tables 6 and 7 summarize the RDI&I results and compare them with results from the 1989 study. RDI&I values per drainage basin are based on the results of the January 23–24 and January 27 storms. Peak dry-weather flow was about the same in 1995 as it was in 1989. Peak dry-weather flows are predicted to increase slightly by the year 2008. Most notable is that the 1995 peak RDI&I (for the 10-year storm) is estimated to be 0.46 mgd (320 gpm, or 1,200 lpm), compared to 4.38 mgd (3,000 gpm, or 11,500 lpm) in 1987. This represents an 88% improvement in the amount of rainwater entering the sanitary sewer. Maximum flow leaving the site in the 1995 was estimated at 1.17 mgd (820 gpm, or 3,100 lpm), well below the discharge limit (1,170 gpm, or 4,430 lpm). The estimated peak year 2008 flow (1.55 mgd, 1,076 gpm, or 4073 lpm), however, is only slightly below the discharge limit.

### 3.6.3 Uncertainty

A number of assumptions were made to develop and run the model. The 10-year storm was used as the source of RDI&I water, and it was assumed that peak RDI&I occurs simultaneously with peak base flow (i.e., between 10:00 a.m. and 2:00 p.m.). In addition, the model assumed that no batch discharges to the sanitary sewer system occurred during storm events.

**Table 6. Comparison of 1987 and 1995 flows.**

| Basin | Peak dry-weather flow (mgd) | | Peak RDI&I[a] (mgd) | | Peak wet-weather flow (mgd) | |
|---|---|---|---|---|---|---|
| | 1987 | **1995** | 1987 | **1995** | 1987 | **1995** |
| L1 | 0.0609 | **0.0733** | 0.2995 | **0.1493** | 0.3604 | **0.2226** |
| L2 | 0.0827 | **0.0742** | 0.2294 | **0.0078** | 0.3121 | **0.0819** |
| L3 | 0.0387 | **0.0340** | 0.2672 | **0.0562** | 0.3059 | **0.0902** |
| L4 | 0.0431 | **0.0544** | 0.5537 | **0.0685** | 0.5968 | **0.1229** |
| L5 | 0.1593 | **0.1565** | 0.7426 | **0.0168** | 0.9019 | **0.1733** |
| L6 | 0.1143 | **0.0490** | 1.4972 | **0.0600** | 1.6115 | **0.1090** |
| L7 | 0.0867 | **0.0784** | 0.2459 | **0.0039** | 0.3326 | **0.0823** |
| L8 | 0.0769 | **0.0725** | 0.2637 | **0.0950** | 0.3406 | **0.1675** |
| SNL | 0.1133 | **0.1132** | 0.2765 | **0.0008** | 0.3898 | **0.1140** |
| **Total:** | 0.7759 | **0.7054** | 4.3757 | **0.4583** | 5.1516 | **1.1638** |

[a] RDI&I = rainfall-dependent infiltration and inflow.

**Table 7. Comparison of 1987 and 2008 flows.**

| Basin | Peak dry-weather flow (mgd) | | Peak RDI&I[a] (mgd) | | Peak wet-weather flow (mgd) | |
|---|---|---|---|---|---|---|
| | 1987 | **2008** | 1987 | **2008** | 1987 | **2008** |
| L1 | 0.0609 | **0.12769** | 0.2995 | **0.14929** | 0.3604 | **0.27698** |
| L2 | 0.0827 | **0.12007** | 0.2294 | **0.00776** | 0.3121 | **0.12783** |
| L3 | 0.0387 | **0.05001** | 0.2672 | **0.05623** | 0.3059 | **0.10624** |
| L4 | 0.0431 | **0.07987** | 0.5537 | **0.06850** | 0.5968 | **0.14837** |
| L5 | 0.1593 | **0.17701** | 0.7426 | **0.01680** | 0.9019 | **0.19381** |
| L6 | 0.1143 | **0.20620** | 1.4972 | **0.05997** | 1.6115 | **0.26617** |
| L7 | 0.0867 | **0.09686** | 0.2459 | **0.00394** | 0.3326 | **0.10080** |
| L8 | 0.0769 | **0.11962** | 0.2637 | **0.09500** | 0.3406 | **0.21462** |
| SNL | 0.1133 | **0.11477** | 0.2765 | **0.00084** | 0.3898 | **0.11561** |
| **Total:** | 0.7759 | **1.09210** | 4.3757 | **0.45833** | 5.1516 | **1.55043** |

[a] RDI&I = rainfall-dependent infiltration and inflow.

## 3.7 Conclusions and Recommendations

We believe that we successfully duplicated the monitoring and modeling techniques used in the 1989 baseline study. We believe that (with the caveats detailed below) our study accurately represents current conditions at the LLNL/SNL site and permits evaluation of improvements made to the sanitary sewer system as a result of the SSR project.

Our conclusions can be summarized in the following statements.

- The SSR project reduced calculated values of RDI&I by 88% compared to the values derived in the 1989 study.
- The LLNL sanitary sewer system has sufficient capacity to transport present and projected (year 2008) peak dry-weather flows.
- The system also has sufficient capacity (1,215 gpm [4,600 lpm]) to transport projected peak wet-weather flows (i.e., RDI&I from a 10-year storm) both now and after future (year 2008) development; however, some pipes will be at 75% of their theoretical capacity.
- Predicted peak flows (820 gpm now and 1,076 gpm in 2008 [3,100 lpm now and 4,073 lpm 2008]) are less than the maximum reserved for the LLNL/SNL system by the City of Livermore (1,170 gpm, or 4,428 lpm).
- Although the focus of this study has been on infiltration, rather than on exfiltration, it is generally true that reduction in infiltration is accompanied by reduction in exfiltration. This study indicates that the rehabilitation effort has resulted in an appreciable reduction in infiltration. Therefore, although exfiltration has not been eliminated, it has been reduced to levels as low as reasonably attainable. The SSR project has achieved the Building Drain Repair objective of reducing exfiltration of waste water into the surrounding soil.

Some uncertainties, however, lead to the specific recommendations bulleted below.

In this study, LLNL evaluated three storms on a site-wide basis and two storms on a basin-by-basin basis (essentially three and two data points, respectively). Although we succeeded in quantifying the LLNL/SNL site-wide RDI&I in a fairly narrow range (R-values of 0.22 to 0.91), overall variability RDI&I has not been computed. A greater degree of uncertainty is associated with the by-basin R-values. (Basin 6 flow, because calculations for it are based upon the flow at all other locations, is the most difficult to quantify adequately.) There is no guarantee that the maximum RDI&I has been determined.

- We recommend a minimum ongoing program of representative, annual monitoring of wet-season and dry-season flow. This further collection of data would verify conclusions reached in this report, maintain current flow data, permit better quantification of the RDI&I (including establishing its upper bound), identify basins having the greatest problems with capacity or with RDI&I, and determine the probability that the flow could exceed either the capacity of the system or the discharge rate reserved by the City of Livermore.

∑ We concur with the 1989 study's recommendation that significant batch discharges not occur during rain storms.

## 3.8 1996–1997 Continuation Study

To help management evaluate the sewer system's needs for continuing maintenance and further upgrades, RDI&I was analyzed for the 1996–1997 rainy season (Brandstetter, 1998). Results are summarized in Table 7. RDI&I estimated for the 1996–1997 season agree extremely well with the values calculated in the 1995 study. Although no industry standard has been established for RDI&I percentages, these values are well below those typically measured in municipalities and large industrial complexes. The large volume of rain in any given storm event, however, represents substantial flows. The three storm events evaluated in 1996–97 produced RDI&I of 4,000, 15,000 and 118,000 gallons (15,000, 57,000 and 731,000 liters). A 10-year storm event (1.28 inches, or 3.25 cm of rain) with an RDI&I percentage of 0.5% would contribute 107,000 gallons (405,000 liters) of water to the LLNL sanitary sewer system. If the RDI&I on that particular day was at the high end of those measured (that is, a worst case scenario of 0.9%), the volume contributed would exceed 193,000 gallons (731,000 liters), an increase of over 60% in the sewer flow. Thus, the contribution of rain to the sanitary sewer must continue to be considered whenever flow capacity or related issues are considered, even though RDI&I percentages are well below any values considered unacceptable.

**Table 7.** Storm event and RDI&I summary.

| Date | Rainfall (inches) | | RDI&I |
|---|---|---|---|
| | Total | Used for RDI&I | |
| 12/5/96 | 0.29 | 0.17 | 0.14% |
| 12/13/96 | 0.19 | 0.19 | 0.50% |
| 1/22/97 | 0.86 | 0.86 | 0.82% |
| Average | 0.45 | 0.41 | 0.49% |

Annual repetition of this study will continue to provide valuable information. LLNL's RDI&I computation is based on six data points. Repetition for at least another two years is needed before we can be confident that researchers have accurately quantified both the mean and the range of the RDI&I. (It is essential to know not only the mean, but to know what the worst case scenario could be.) In addition, to date, only one storm with rainfall greater than 1 inch (2.54 cm) has been evaluated. Although it is common for LLNL to experience rainfall greater than 2 and sometimes even 3 inches (5 cm, 7.6 cm) in a 24-hour period, thus far large storm events have not met the criteria for the RDI&I calculation. Continuation of this type of study is necessary in order to evaluate the impact of such larger storm events. Furthermore, this type of study represents a proactive approach—the RDI&I studies can provide a warning of possible inflow or infiltration problems—rather than waiting until problems become excessive or obvious. By continuing to document a low RDI&I percentage, LLNL can confirm that administrative and other controls are adequate, and that there is no need for extensive source investigation. Finally, because the data required for this study are collected routinely for other purposes, the only cost for this study is the analysis itself. Therefore continuing RDI&I studies provide real benefits at a relatively low cost.

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## Acronyms and Terminology

| | |
|---|---|
| ABF | average base flow. Estimated flow on a typical day during conditions of no rain and no batch discharges. |
| base flow | Flow during conditions of no rain and no batch discharges |
| batch discharge | A manual discharge to the sanitary sewer system, resulting from cooling-tower blowdown, retention-tank releases, or treatment-facility releases. |
| cfs | cubic feet per second |
| design flow | Estimated flow capacity required in the sanitary sewer system, accounting for base flow and rainfall dependant infiltration and inflow. |
| DOE | United States Department of Energy |
| $ft^2$ | square feet |
| gpcd | gallons per capita per day |
| gpm | gallons per minute |
| gpsfd | gallons per square foot per day |
| GWI | ground water infiltration |
| lpcd | liters per capita per day |
| lpm | liters per minute |
| lpsmd | liters per square foot per day |
| LLNL | Lawrence Livermore National Laboratory |
| LWRP | Livermore Water Reclamation Plant |
| $m^2$ | square meters |
| mgd | million gallons per day |
| mld | million liters per day |
| peak base flow | highest flow during conditions of no rain and no batch discharges |
| PDWF | peak dry weather flow, equivalent to peak base flow |
| PSF | peak sanitary flow |
| PWWF | peak wet weather flow |
| RDI&I | rainfall dependant infiltration and inflow. The amount of rainfall that enters the sanitary sewer. |
| R-value | the ratio of RDI&I volume divided by the total rainfall volume |
| SNL | Sandia National Laboratory |
| SSR | Sanitary Sewer Rehabilitation project. A 5-year project to upgrade and improve the sanitary sewer system at Lawrence Livermore National Laboratory. |
| TFF | Treatment Facility F. A facility that treats ground water to remove contaminants. |

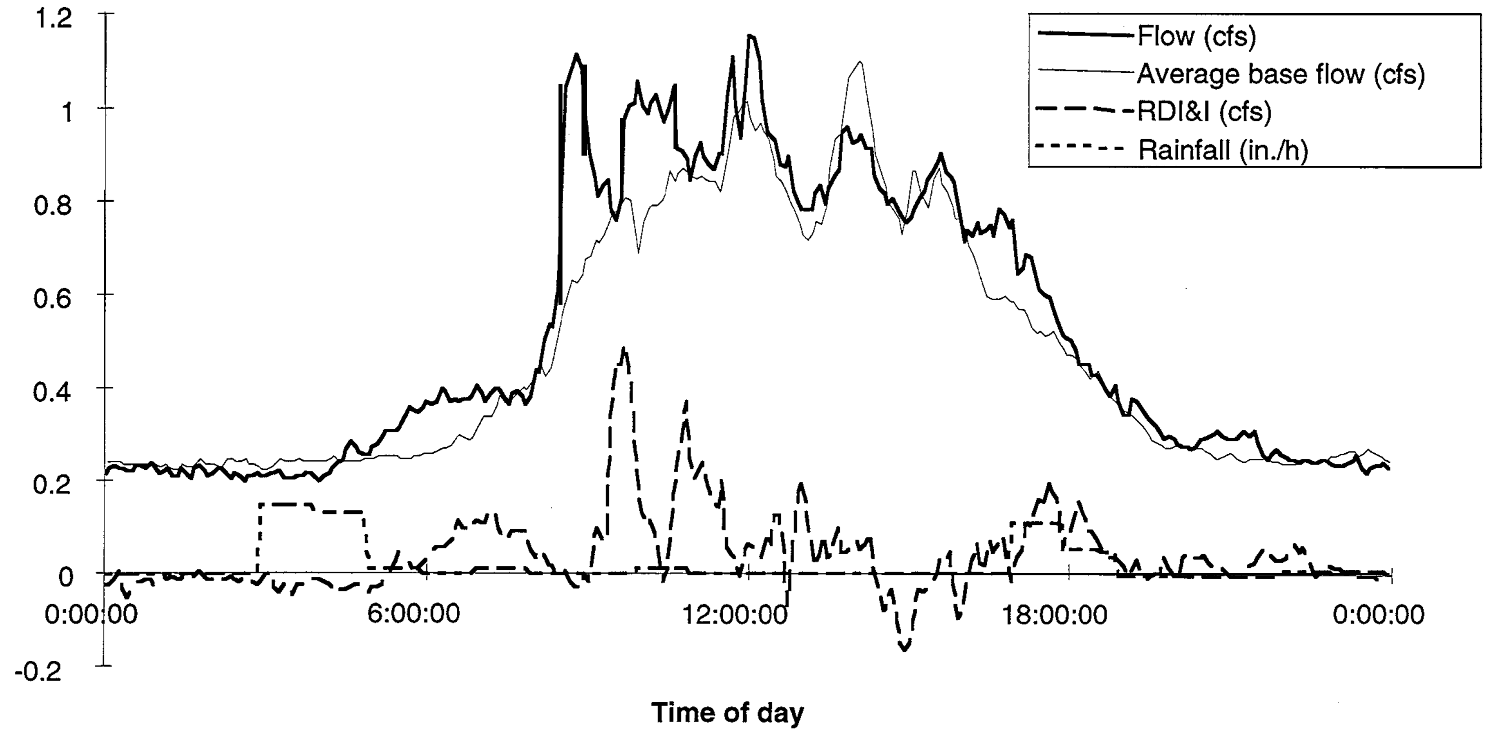


**Figure 2. LLNL/SNL site-wide data for January 23, 1995: rainfall, measured flow, average base flow (ABF), and rainfall-dependent infiltration and inflow (RDI&I).**

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