CALIBRATION AND VERIFICATION OF STORMWATER MODELS

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ABSTRACT: Although calibration and verification have long been recognized as important considerations in the use and application of stormwater models, the process can be quite complex and daunting, involving a seemingly infinite number of combinations and permutations of parameters that directly affect the behavior of a model. The process is further complicated as attempts are made to evaluate the effectiveness of altering certain parameters. For example, if a runoff curve number is increased such that a simulated peak stage more closely matches a recorded high water mark, does that necessarily imply that the model is predicting reality more closely? The purpose of this paper is to examine the sensitivity of several modeling parameters during the calibration process and how changes to those parameters affect stage hydrographs. The Integral Square Error (ISE) is used as a tool to evaluate the effectiveness of model alterations on the entire stage hydrograph rather than simply evaluating peak stages or peak flow rates. An urbanized watershed (the Central Drainage Ditch Basin) located within the corporate limits of the City of Tallahassee is used as an example. An approximate 10-year 24-hour storm that occurred in March 1994 is used for calibration purposes. Three other significant storms that also occurred in 1994 are used for verification purposes.

INTRODUCTION

The calibration of a stormwater model typically involves comparing simulated stages, flows and/or volumes of water with observed data for a recent and significant storm event. Depending on the outcome of the comparison, certain model parameters are adjusted such that the predicted values more closely match historical values until a "best fit" is achieved. The process is typically iterative and each subsequent adjustment depends on the outcome of the previous iteration. It can be a tedious, complicated and often frustrating ordeal especially when the historical data are inadequate or of poor quality. The process is further complicated as the modeling professional attempts to interpret and evaluate the effectiveness of each combination and permutation.

Once a final set of modeling parameters has been selected (i.e., the model has been calibrated), it is important to examine the ability of the model to predict other storm events with the same degree of reliability as the calibrated model. This is accomplished by simulating two or three other historical storm events that occurred in the same watershed prior to significant alterations in the basin such as major land use changes or drainage improvements. Preferably, the verification storms should vary from the calibration storm in both magnitude and duration, but they should be long enough and large enough to impact all points in the watershed under consideration. For example, if it has been determined that the travel time in a particular watershed from the most extreme point to its outlet is 3 hours, then perhaps storms of 3-hour duration or greater should be considered for verification purposes. Engineering judgement must be used in the selection of calibration and verification storms.

Stormwater models, especially hydrodynamic models, are quite complex and typically involve thousands of individual data elements and hundreds of judgement calls by the modeling professional. Each of these can and often are questioned. Whether justified or not, if the model has not been demonstrated to predict actual occurrences with some degree of reliability, it will be subject to criticism and difficult to defend. Therefore, the importance of calibration and subsequent verification cannot be overstated.

Successful calibration requires two key elements. First, an accurate and reliable historical record of both rainfall and stream data (stage and/or flow data) for the study area must be available. Continuous recorders are preferred although high water marks will suffice if other records are unavailable. Second, accurate input data for the model including land use and drainage infrastructure consistent with the time period to be used for calibration and verification purposes must be compiled. It makes little sense to use a storm event that occurred in 1960 for calibration of a model prepared based on conditions in 2001.

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The purpose of this paper is to examine the sensitivity of several modeling parameters during the calibration process and how changes to those parameters affect stage hydrographs. These parameters include the peak rate factor (K'), runoff curve numbers, and roughness coefficients along the channels. The Integral Square Error (ISE) is used as a tool to evaluate the effectiveness of model alterations on the entire stage hydrograph rather than simply evaluating peak stages or peak flow rates. A 6-square mile urbanized watershed (the Central Drainage Ditch Basin) located within the corporate limits of the City of Tallahassee is used as an example. Five continuous rain gage recorders (5-minute intervals) are located within this basin as well as four continuous stream gage recorders (also 5-minute intervals).

HISTORICAL DATA

The City of Tallahassee, Leon County and the Northwest Florida Water Management District entered into a tri-party agreement several years ago to jointly fund and monitor a comprehensive network of rain and stream gages. This monitoring program is intended to collect dry weather and storm event discharge data at major outfall locations in Leon County and the City of Tallahassee and is in partial fulfillment of requirements set forth by the USEPA National Pollutant Discharge Elimination System (NPDES). Continuous records of precipitation and stages are maintained to aid in estimating flows, volumes and annual pollutant loads. The data is also used to update hydrologic and flooding conditions as growth and development occurs. The tri-party agreement encompasses a total of 31 recording stations including 16 stream gages that monitor stage and velocity, 3 stream gages that monitor only stage, and 12 rainfall gages. The FY2000 cost for maintaining these 31 stations was $71,286 and equally shared by the City of Tallahassee and Leon County. An equivalent amount of "inkind" services was provided by the Northwest Florida Water Management District.

In addition to the stations described above, the City of Tallahassee maintains 6 other rain gages and 4-10 stream gages. The actual number of stream gages at any point in time varies. They are installed and/or removed depending on specific capital improvement projects. Approximately 25% of a technician's time is dedicated to maintaining these stations.

Figure 1 depicts the comprehensive network of rain and stream gages for the City of Tallahassee and Leon County along with Thiessen polygons for each of the rain gages. The Central Drainage Ditch (CDD) Basin is also shown in Figure 1 and, as indicated, it is situated in the southeastern portion of the City.

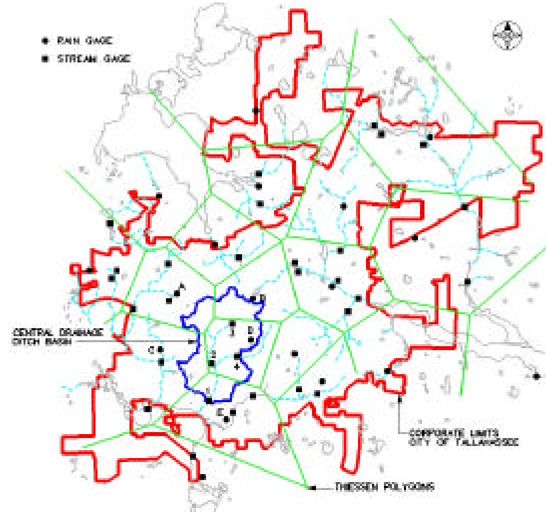


Figure 1. Rain and Stream Gage Network for the City of Tallahassee and Leon County.

An unusually wet year occurred in Tallahassee in 1994 with 3 major storm events, each dropping over 6 inches of rain in 24- to 48-hour periods. Additionally, a fourth storm of lesser magnitude (3 inches in 5 hours) occurred that year. Precipitation totals for these 4 storms at each of the 5 gages affecting the CDD Basin are provided in Table 1. The March 1 storm, largest of the four, was approximately a 10-year event and evenly distributed over the study area. Therefore, it was selected as the calibration storm and the others were used for verification purposes.

	Mar. 1 9:00 am	May 15 11:45 am	Aug. 14 8:00 am	Oct. 1 6:00 pm
Duration	24 hrs	5 hrs	48 hrs	30 hrs
Gage				
A	7.67"	4.04"	6.64"	5.89"
B	7.23"	3.30"	7.27"	6.95"
C	7.85"	4.70"	7.57"	7.32"
D	7.29"	2.63"	5.79"	6.04"
E	7.62"	0.82"	6.30"	5.88"
Area Weighted Totals	7.48"	3.07"	6.53"	6.36"

Table 1. Summary of Precipitation Totals for 1994 Storm Events.

Rainfall data, at 5-minute intervals, was provided by the City of Tallahassee Stormwater Division for the

4 storms and 5 rain gages listed in Table 1. As shown in Figure 2, individual Thiessen polygons for each rain gage cover only a portion of the CDD Basin. In an effort to more accurately account for the non-uniform distribution of rainfall in the study area, rainfall data for individual sub-basins was assigned based on the Thiessen polygon encompassing the sub-basin. Table 2 provides a breakdown of the total area served by each of the 5 rain gages in or near the CDD Basin.

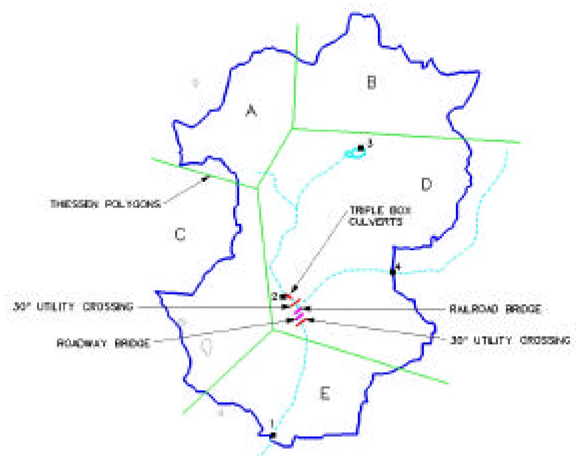


Figure 1. Thiessen Polygons and Stream Gage Locations in the CDD Basin.

Rain Gage	Area (ac)	% of Total
A	573.3	15.34
B	590.4	15.80
C	650.6	17.42
D	1,416.8	37.93
E	504.7	13.51
Totals	3,735.8	100.00

Table 2. Areas Served by Rain Gages.

In addition to the 5 rain gages, 4 stream gages designated 1, 2, 3 and 4 are located in the CDD Basin (refer to Figure 2). Three of these are located along the CDD and the fourth is on a tributary to the CDD. Gages 1, 2 and 3 are located at the lower end, near the midpoint and at the upper end of the CDD, respectively. These 3 gages were used for calibration purposes while gage 4 was used as a boundary condition for the model. Flow records were obtained from the City of Tallahassee Stormwater Division for gage 4 and specified as inflow hydrographs for the stormwater model.

STORMWATER MODEL

A comprehensive hydrologic and hydraulic computer model was prepared for the CDD Basin to simulate the rainfall-runoff process. Specifically, the Interconnected Channel and Pond Routing Model (ICPR v2.20) was used for all stormwater modeling purposes. This model has been accepted by the Federal Emergency Management Agency (FEMA) for use on flood plain investigations associated with flood insurance applications and it is widely used throughout Florida and the United States. ICPR is distributed by Streamline Technologies, Inc. Information can be found at [www.streamnologies.com.](http://www.streamnologies.com)

The Soil Conservation Service (SCS) unit hydrograph method was used for all sub-basins in the CDD system. This method requires drainage areas, SCS curve numbers, and times of concentration for each sub-basin. Additionally, estimates of directly connected impervious areas (DCIA's) were included in the hydrologic analysis. A total of 80 individual sub-basins were delineated from 1"=200' scale aerial topographic maps (2-foot contour intervals) provided by the City of Tallahassee. These boundaries were refined based on field inspections, a literature review and discussions with city staff. Existing land use was determined from aerial photographs taken within a few years of the calibration and verification storms. Soils information was obtained from the Leon County Soil Survey prepared by the USDA Soil Conservation Service (now the Natural Resource Conservation Service).

Table 3 includes each of the land use types encountered in the study area along with their assumed percentage of directly connected impervious areas (DCIA) and the runoff curve number for the remaining non-DCIA based on normal antecedent moisture conditions. Since one of the calibration parameters is the runoff curve number for non-dcia's, adjustments to curve numbers based on antecedent moisture conditions are provided in Table 4.

The CDD system was discretized into 93 nodes and 147 links. Nodes are discrete locations within the watershed used to define inflow points, boundary conditions, storage areas, changes in channel slope or geometry, or any other points of interest. Runoff hydrographs are loaded at individual nodes in the system and ICPR computes water surface elevations at each node in the model. Links are used to connect nodes together and include pipes, channels, weirs, drop structures, bridges, dam breaches, and rating curves. ICPR calculates flows for each link based on water surface elevations at its connecting nodes. Hydraulic data requirements for the ICPR model include two general types: (1) node data (e.g., location, pond storage or channel overbank storage, initial stages, boundary conditions, etc.); and, (2) link data (e.g., pipe geometry, channel cross-section, weir invert information, etc.).

Two types of boundary conditions are used for the CDD model. The first is a stage-time relationship at the downstream-most node in the model located below stream gage #1. Stage-time relationships for each of the 4 historical storms were provided by the City of Tallahassee Stormwater Division. The second boundary condition is a time-discharge relationship located at the upper end of the eastern tributary to the CDD near stream gage #2. Discharge hydrographs based on historical gaging data at this location for the 4 storms were also provided by the City of Tallahassee Stormwater Division.

Extensive field surveys of the CDD were obtained in the summer and fall of 1997. These included detailed structure geometry for 9 bridges and associated roadway profiles, and cross section data for approximately 17 channel locations. The top-of-bank, toe of slope, channel flow line, water surface elevation, and any other pertinent topographic breaks were collected for each cross section. Construction level surveys were also performed for approximately 3,700 feet of the CDD.

Land Use Number	**Land Use Description**	**% IMP**	**% DCIA**	**Curve Number for Non-DCIA**[1]			
				A	**B**	**C**	**D**
110	SF Res. (1/2-1 ac)	25	15	44.9	64.7	76.4	81.8
120	SF Res. (1/4 ac)	38	22	47.9	66.6	77.6	82.7
130	SF Res. (<=1/6 ac)	60	38	54.3	70.6	80.2	84.7
133	Multi-Family	71	57	47.3	66.2	77.4	82.5
140	Comm. (<30% open)	85	68	49.0	67.3	78.1	83.1
147	Comm. (>30% open)	70	57	47.3	66.2	77.4	82.5
150	Industrial	70	57	47.3	66.2	77.4	82.5
170	Institutional	85	68	49.0	67.3	78.1	83.1
182	Golf Course	25	25	53.2	69.9	79.8	84.3
183	Race Tracks	0	0	49.0	69.0	79.0	84.0
185	Parks and Zoos	0	0	49.0	69.0	79.0	84.0
186	Recreational	0	0	49.0	69.0	79.0	84.0
190	Open Land	0	0	39.6	59.4	70.4	76.3
211	Improved Pasture	0	0	49.0	69.0	79.0	84.0
320	Rangeland (shrub)	0	0	35.0	56.0	70.0	77.0
400	Wooded	0	0	27.9	50.0	63.8	70.4
500	Ponds w/ Berms	95	0	98.0	98.0	98.0	98.0
620	Wetland Forest	95	0	98.0	98.0	98.0	98.0
640	Wetland Marsh	95	0	98.0	98.0	98.0	98.0
812	Railroads	63	50	37.5	55.9	68.3	74.3
814	Roads w/ C&G	100	0	96.4	96.4	96.4	96.4
816	Canals	60	60	47.5	66.6	77.6	82.7
830	Utilities	0	0	49.0	69.0	79.0	84.0

[1] CN's based on AMC II

Table 3. DCIA and SCS Curve Numbers for Non-DCIA by Land Use

CALIBRATION PARAMETERS

As previously stated, three primary parameters were varied during the calibration process of the CDD model: (1) the peak rate factor (K'); (2) curve numbers; and, (3) channel roughness characteristics. The peak rate factor is used in conjunction with the unit hydrograph method to alter the shape and timing of the discharge hydrograph for individual drainage sub-basins. As the peak rate factor increases, the rising and falling legs of the runoff hydrograph become steeper and the peak rate of runoff increases, in effect, reducing storage in the sub-basin. As the peak rate factor decreases, peak flow rates decrease and the volume of runoff is attenuated or pushed farther out in time, thus increasing storage in the sub-basin. Adjusting the peak rate factor alters the overall shape of the discharge hydrograph and when combined with other sub-basin hydrographs, can affect timing and peak flow rates in drainage conveyance systems such as the CDD. Peak rate factors of 256, 323 and 484 were evaluated.

ICPR assumes an initial abstraction of 0.1" over directly connected impervious areas (DCIA's) and then 100% of the rainfall (beyond 0.1") falling on the DCIA appears as runoff. Consequently, use of DCIA's in the CDD model accounts for runoff almost immediately after rainfall commences. This technique is appropriate for highly urbanized areas similar to the CDD Basin.

CN for Normal Conditions (AMC II)	CN for Dry Conditions (AMC I)	CN for Wet Conditions (AMC III)
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

Table 4. Curve Number Adjustments Based on Antecedent Moisture Conditions.

Sub-basin runoff curve numbers in ICPR represent all areas that are not DCIA. Curve numbers can vary from 0 to 100, with a curve number of 0 producing no runoff and a curve number of 100 producing 100% runoff. However, the relationship between curve number and runoff is non-linear and depends on the amount of rainfall among other factors. The curve numbers for non-DCIA's in the CDD Basin were varied during the calibration process discussed in this paper. Three sets of curve numbers were evaluated: (1) CN's based on AMC II (normal conditions); (2) CN's based on AMC I (dry conditions); and, (3) CN's based on the average of normal and dry conditions AMC (I+II)/2.

The third and final parameter adjusted for calibration purposes was Manning's n along the channel system of the CDD. Adjustments to n-values were based on desired increases and reductions in the friction slope along the channel. Since the friction slope is a function of the square of Manning's n, the following equation was used to adjust n-values . Table 5 provides the ranges of n-values used during the calibration process.

$$n' = [\, f \times (n)^2 \,]^{1/2} \qquad (1)$$

where,

n' is the adjusted n-value,
n is the unadjusted n-value, and
f is the adjustment factor (e.g., 1.25 increases the friction slope by 25%).

f=0.25	f=0.50	f=0.75	f=1.00	f=1.25
0.01500	0.02121	0.02598	0.03000	0.03354
0.01750	0.02475	0.03031	0.03500	0.03913
0.02000	0.02828	0.03464	0.04000	0.04472
0.02250	0.03182	0.03897	0.04500	0.05031
0.02500	0.03536	0.04330	0.05000	0.05590
0.02750	0.03889	0.04763	0.05500	0.06149
0.03000	0.04243	0.05196	0.06000	0.06708
0.03250	0.4596	0.05629	0.06500	0.07267
0.03500	0.04950	0.06062	0.07000	0.07826

Table 5. Manning's n as a Function of Friction Slope Adjustment Factor (f).

INTEGRAL SQUARE ERROR

The integral square error, ISE, (Marsalek, et al, 1975) is a useful tool to compare and evaluate various simulations conducted as part of the calibration / verification process. It is a statistical measure that describes the agreement between the time distribution of the observed and computed values of a variable such as flood depth. The ISE is determined from the following equation:

$$ISE = \frac{\left[\sum_{i=1}^{N} (O_i - C_i)^2 \right]^{1/2}}{\sum_{i=1}^{N} O_i} \times 100 \qquad (2)$$

where,

O_i is the observed or recorded depth,
C_i is the computed depth, and
N is the number of observations.

Smaller ISE's indicate better agreement between observed and computed values. The following ratings have been recommended by Sarma, Delleur and Rao (1969):

0.0% ≤ ISE ≤ 3.0%		excellent
3.0% ≤ ISE ≤ 6.0%		very good
6.0% ≤ ISE ≤ 10.0%		good
10.0% ≤ ISE ≤ 25.0%		fair
25.0% ≤ ISE		poor

The ISE was computed for each of the 3 gages used for calibration purposes and for each set of parameters that were evaluated.

SENSITIVITY TO PEAK RATE FACTOR, K'

As previously stated, sensitivity of the CDD to peak rate factor, K', was evaluated by simulating the March 1, 1994 storm for various peak factors including K'=256, K'=323 and K'=484. Table 6 provides a summary of observed maximum stages, computed maximum stages and the corresponding ISE values for the peak rate factors investigated. These were based on curve numbers for the AMC II condition (normal antecedent moisture condition).

	Gage #1	Gage #2	Gage #3
Max. Stage			
Observed	40.25	58.05	84.08
K'=256	40.68	59.14	83.74
K'=323	40.77	59.22	83.83
K'=484	40.94	59.29	84.09
ISE			
K'=256	1.1620	6.3291	2.1103
K'=323	1.1293	6.0599	2.8958
K'=484	1.1216	5.6943	4.1414

Table 6. Summary of Maximum Stages and ISE Values for Various Peak Rate Factors Under AMC II Conditions.

Maximum stages do not appear to be sensitive to peak rate factor. The differences in maximum stages between K'=484 and K'=256 are only 0.26', 0.15' and 0.35' at gages 1, 2 and 3, respectively. It is notable that as K' is increased from 256 to 484, maximum stages at gages 1 and 2 increase farther above observed stages, yet the ISE values decrease suggesting a better overall fit to historical data. However, according to criteria for ISE values, the gage 1 simulated stages for all 3 K' values are considered "excellent". Simulation results for gage 2 are considered "good" for peak rate factors 256 and 323 and "very good" for K'=484. Figures 3 and 4 depict graphical comparisons between observed and simulated stage hydrographs at gages 1 and 2. In general, it appears that predicted stages for this set of simulations are higher than observed for gages 1 and 2, possibly caused by higher runoff rates and/or higher roughness in the channel. Lowering curve numbers should cause an overall decrease in stages while lowering roughness coefficients along the channel would likely reduce flood levels in the upper reaches and increase levels in the lower reaches.

Figure 3. Sensitivity to K' for AMC II
Stream Gage #1, Orange Avenue (March 1, 1994 Storm)

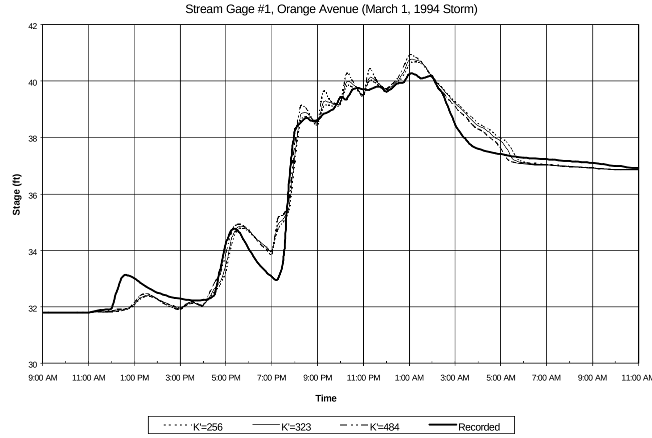
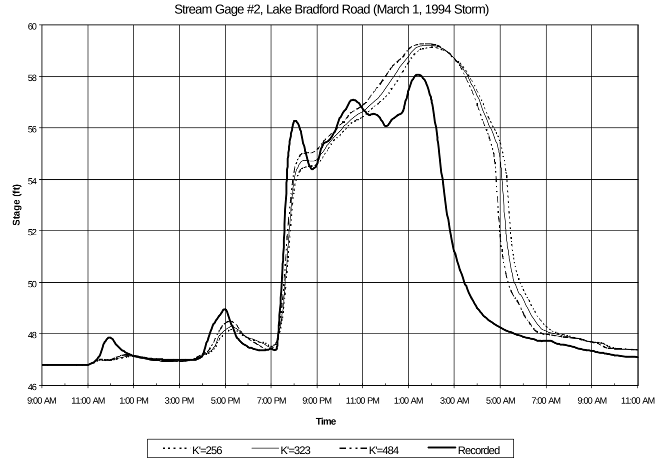
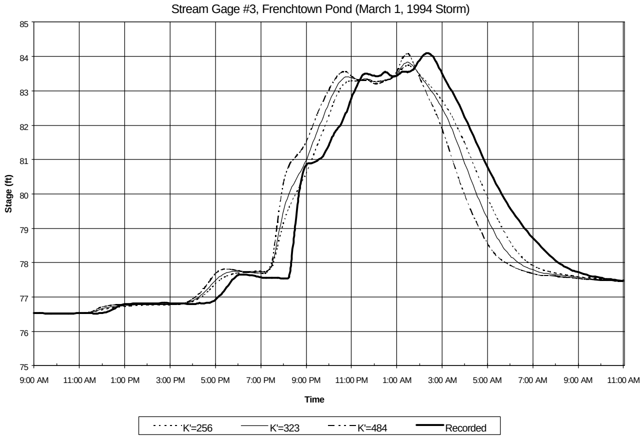


Figure 4. Sensitivity to K' for AMC II
Stream Gage #2, Lake Bradford Road (March 1, 1994 Storm)



The effect of peak rate factor on the ISE value for gage 3 is the opposite of that observed for gages 1 and 2. ISE values increase with K' meaning that a worse fit to the observed data occurs as K' is increased although all 3 ISE values are considered either "excellent" or "very good". Examination of the stage hydrographs for gage 3 (see Figure 5) indicate a phase shift of approximately 1 hour between the simulated and observed hydrographs. Since gage 3 is located in a large detention pond at the headwaters of the CDD and recieves runoff from a single drainage sub-basin, it is possible in ICPR to "shift" the inflow hydrograph by an hour. Doing so for K'=484 reduces the ISE from 4.1414 to 1.6338 significantly improving the fit. However, the possibility of data error must be considered. It is conceivable that the reported rainfall data for this particular basin was shifted by an hour possibly due to a power outage. Rather than introduce a time shift at this point in the calibration process, the writer believes it would be more appropriate to examine this area again after the validation simulations are completed.

Figure 5. Sensitivity to K' for AMC II
Stream Gage #3, Frenchtown Pond (March 1, 1994 Storm)



SENSITIVITY TO RUNOFF CURVE NUMBERS

As indicated in the previous section, predicted stages along the CDD generally appear higher than observed levels for the 3 peak rate factors examined. K'=484 seems to produce the best overall fit along the CDD according to ISE values although peak stages are more than 1 foot higher than observed maximums at gage 2. Therefore, it seems appropriate to reduce the runoff volume (i.e., reduce runoff curve numbers) and hold the peak rate factor to K'=484. In an effort to methodically reduce the runoff curve numbers, Table 4 was used as a guide. In addition to the AMC II conditions evaluated in the previous section, AMC I and AMC (I+II)/2 were added to the calibration set.

Table 7 provides a summary of the results of this analysis. Maximum stages at each of the 3 gages have now been "bracketed" meaning there are sets of calibration parameters that predict maximum stages above and below the observed peak flood levels. This is important because it narrows the search for an acceptable set of modeling parameters. Likewise, the ISE values are considered "very good" to "excellent" in all cases. Based on the 3 scenarios evaluated, the AMC (I+II)/2 with a K'=484 appears to be the "best fit" overall because in addition to the more than acceptable ISE values, simulated peak stages for the 3 gages are within 6 inches of observed levels. This places the computed maximum depths at all 3 gages less than 4% different from observed maximum depths.

	Gage #1	Gage #2	Gage #3
Max. Stage			
Observed	40.25	58.05	84.08
AMC I	40.13	57.25	83.54
AMC (I+II)/2	40.51	58.49	83.81
AMC II	40.94	59.29	84.09
ISE			
AMC I	1.1179	4.6296	4.5108
AMC (I+II)/2	1.0240	5.3221	4.0808
AMC II	1.1216	5.6943	4.1414

Table 7. Summary of Maximum Stages and ISE Values for Various AMC's with K'=484.

The model appears to be more sensitive to curve numbers than peak rate factor, especially at gage 2. Gage 2 is located approximately midway along the CDD and upstream of a major highway crossing with large box culverts. There are numerous other impediments to flow immediately downstream of the gage including two 30-inch aerial sewer crossings, a railroad crossing, a roadway bridge crossing and sandwiched between all of these is a confluence with a major tributary system to the east. All of these combine to create complex hydraulic and tailwater influences on gage 2.

Figures 6 and 7 depict observed and simulated stage hydrographs at gages 1 and 2. Visually, AMC (I+II)/2 appears to fit better at gage 1 and this is supported by the lower ISE value. Although AMC (I+II)/2 fits better for gage 2 on the rising leg and along the peaks, it tends to lag the observed hydrograph on the recession limb. Figure 8 shows the observed and simulated stage hydrographs for gage 3.

SENSITIVITY TO FRICTION SLOPE

As indicated in Table 8, altering the friction slope, Sf, has no impact on gage 3 because it is located upstream in a detention beyond the influence of these changes. Maximum flood levels at gage 1 vary only slightly, increasing as the friction slope is decreased and decreasing as the friction slope is increased. This seems to be opposite of what one might expect, but it must be remembered that this gage is located at the downstream end of the CDD. By decreasing the friction slope (i.e., reducing Manning's n) water is moved more efficiently from the upper portion of the basin to the lower portion, thereby increasing flow stages downstream.

The impact at gage 2 is a little more significant than gage 1, but relatively minor overall. Increasing the friction by 25% above the base values causes only a 0.29' difference in maximum stages or about a 2.6% difference in flood depths. Reducing the friction slope by 25% lowers the maximum water level by 0.33' or 2.9%.

Examination of the ISE values indicate an "excellent" fit for gage 1 and "very good" fits for gages 2 and 3 in all cases. Stage hydrographs for gages 1 and 2 are depicted in Figures 9 and 10, respectively. Figure 10 includes a scenario for reduction of the friction slope by 75%. Although this is a little drastic, it indicates that maximum stages are pulled down too far at gage 2 and that stormwater is probably moved too efficiently downstream.

	Gage #1	Gage #2	Gage #3
Max. Stage			
Observed	40.25	58.05	84.08
Sf -25%	40.71	58.16	83.81
Sf Base	40.51	58.49	83.81
Sf +25%	40.43	58.78	83.81
ISE			
Sf -25%	1.0180	4.9041	4.0808
Sf Base	1.0240	5.3221	4.0808
Sf +25%	1.0481	5.5868	4.0808

Table 8. Summary of Maximum Stages and ISE Values for Various Adjustments to the Friction Slope, Sf, with K'=484 and AMC (I+II)/2.

Figure 6. Sensitivity to AMC for K'=484
Stream Gage #1, Orange Avenue (March 1, 1994 Storm)

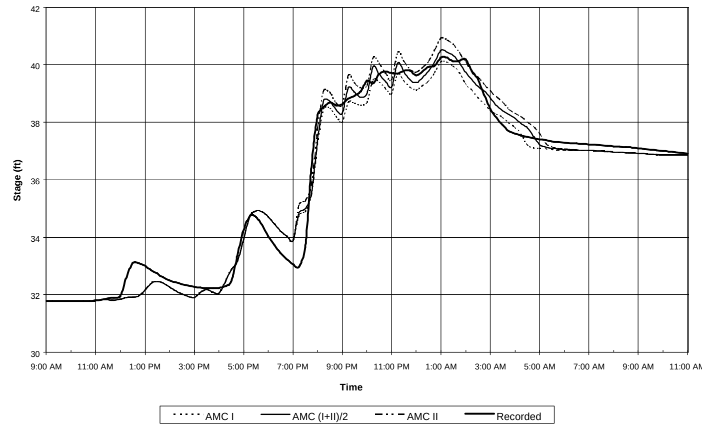


Figure 7. Sensitivity to AMC for K'=484
Stream Gage #2, Lake Bradford Road (March 1, 1994 Storm)

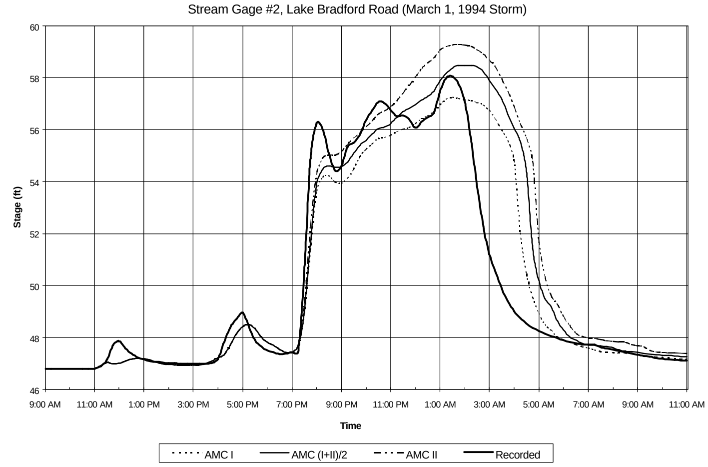
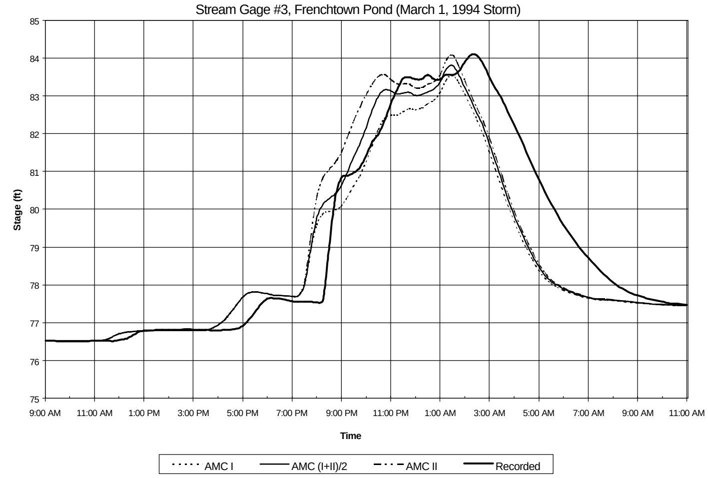


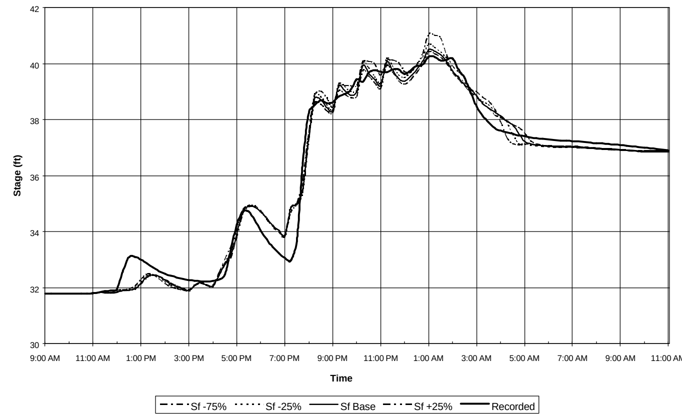
Figure 8. Sensitivity to AMC for K'=484
Stream Gage #3, Frenchtown Pond (March 1, 1994 Storm)



SUMMARY OF CALIBRATION

Of the three calibration parameters evaluated, the CDD Basin is most sensitive to runoff curve numbers and least sensitive to peak rate factor. The best overall fit in terms of ISE values and the ability to predict observed maximum flood levels occurs when K'=484 and for runoff curve numbers corresponding to AMC (I+II)/2. Although reducing the friction slope, Sf, by 25% improved the simulations slightly, it is the writer's belief that the higher n-values (Sf Base) should be used for subsequent verification purposes because the March 1, 1994 calibration occurred at the end of Winter and prior to the growing season. Higher n-values probably existed in May, August and October 1994.

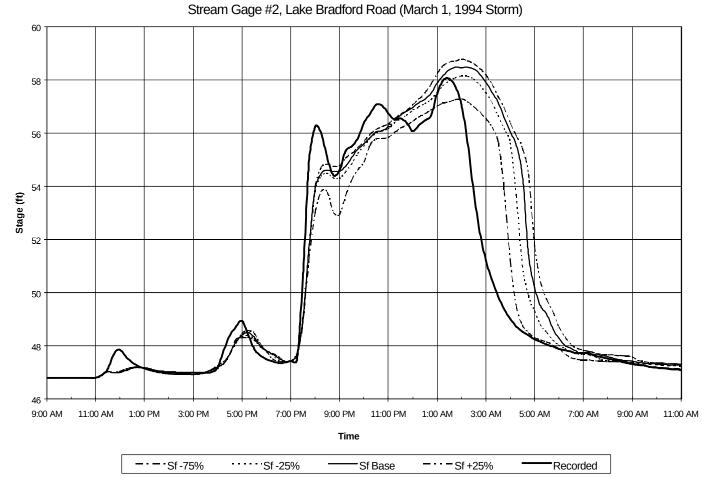
Figure 9. Sensitivity to Friction Slope for AMC (I+II)/2 and K'=484
Stream Gage #1, Orange Avenue (March 1, 1994 Storm)



MODEL VERIFICATION

A summary of maximum stages and associated ISE values for each of the three verification storms is provided in Table 9. The ISE values indicate "excellent" and "very good" fits to the observed data in all cases.

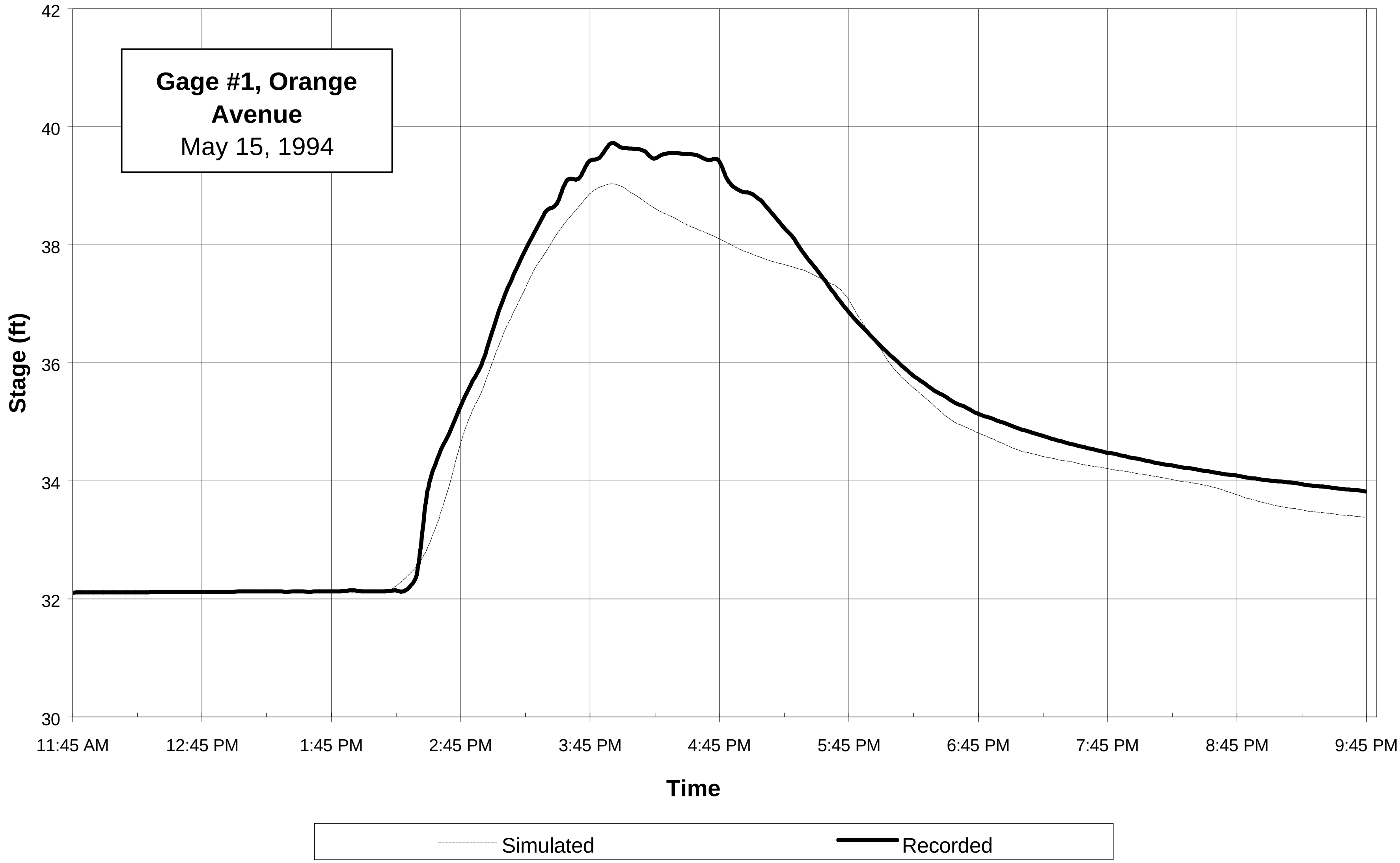
Figure 10. Sensitivity to Friction Slope for AMC (I+II)/2 and K'=484
Stream Gage #2, Lake Bradford Road (March 1, 1994 Storm)



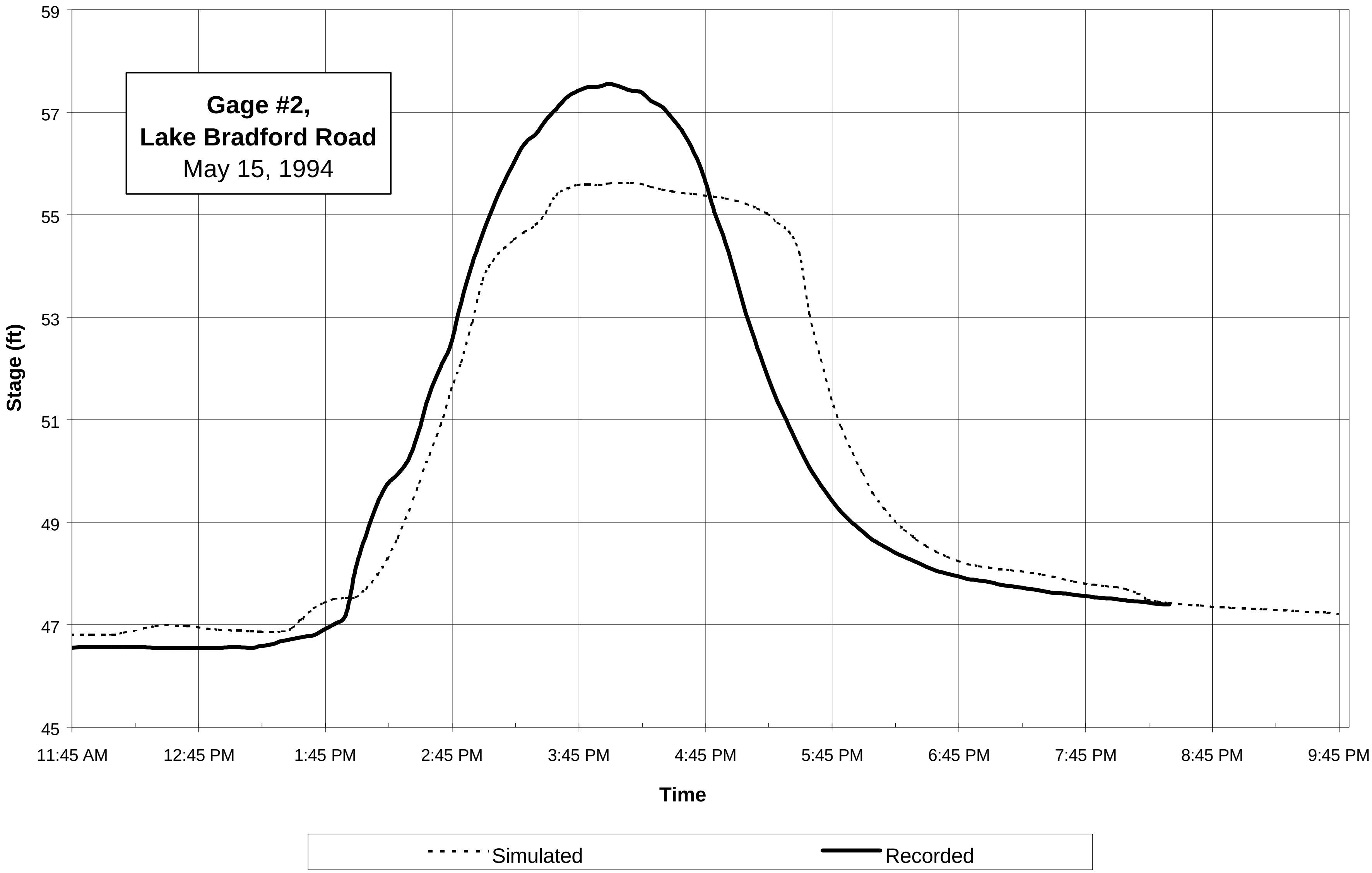
	Gage #1	Gage #2	Gage #3
Max. Stage			
May 15, 1994			
Observed	39.72	57.55	83.49
Simulated	39.03	55.62	82.81
Aug. 14, 1994			
Observed	40.18	56.05	82.26
Simulated	39.27	55.89	82.96
Oct. 1, 1994			
Observed	40.02	56.41	82.76
Simulated	39.07	55.45	83.18
ISE			
May 15, 1994	1.5073	3.4820	3.0968
Aug. 14, 1994	1.2717	3.3501	1.0843
Oct. 1, 1994	0.9897	2.7125	1.7328

Table 9. Summary of Maximum Stages and ISE Values for Verification Storms, (with K'=484, AMC (I+II)/2 and Sf Base).

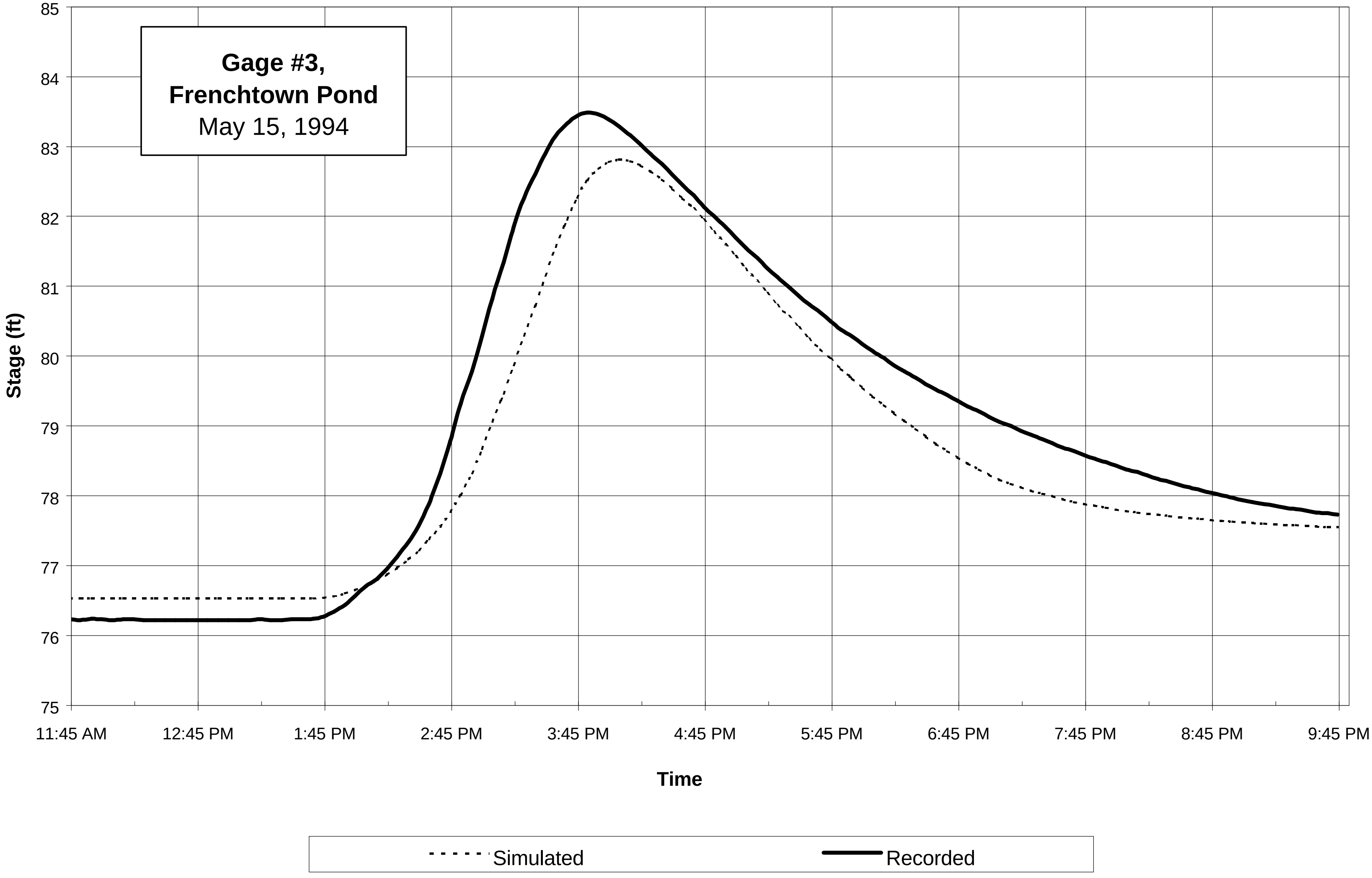
Comparisons of observed and simulated stage hydrographs for each storm can be found on the pages following this paper.



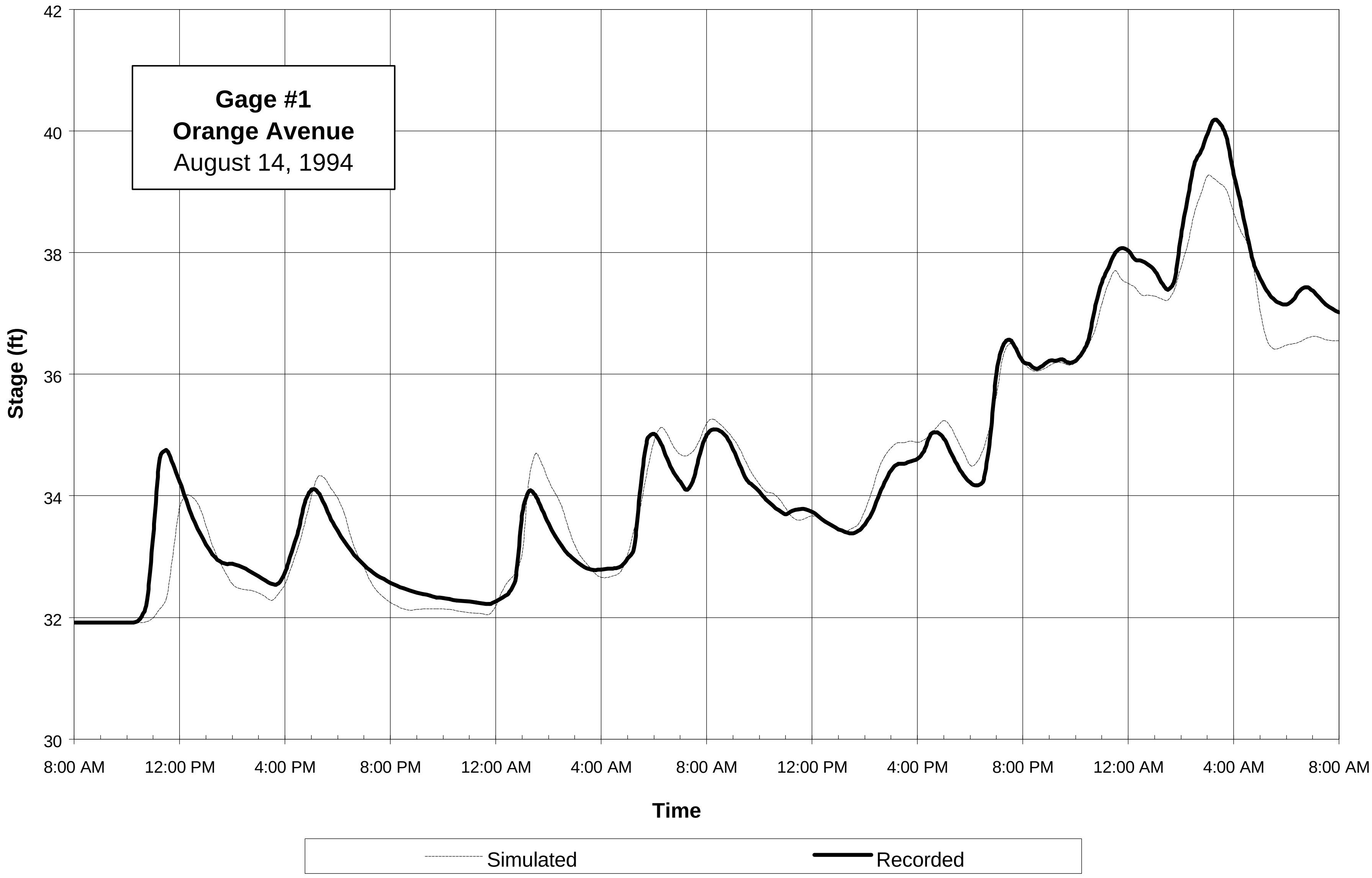
[1] Singhofen & Associates, Inc. 6961 University Blvd., Winter Park, FL 32792 . E-mail: pete@streamnologies.com



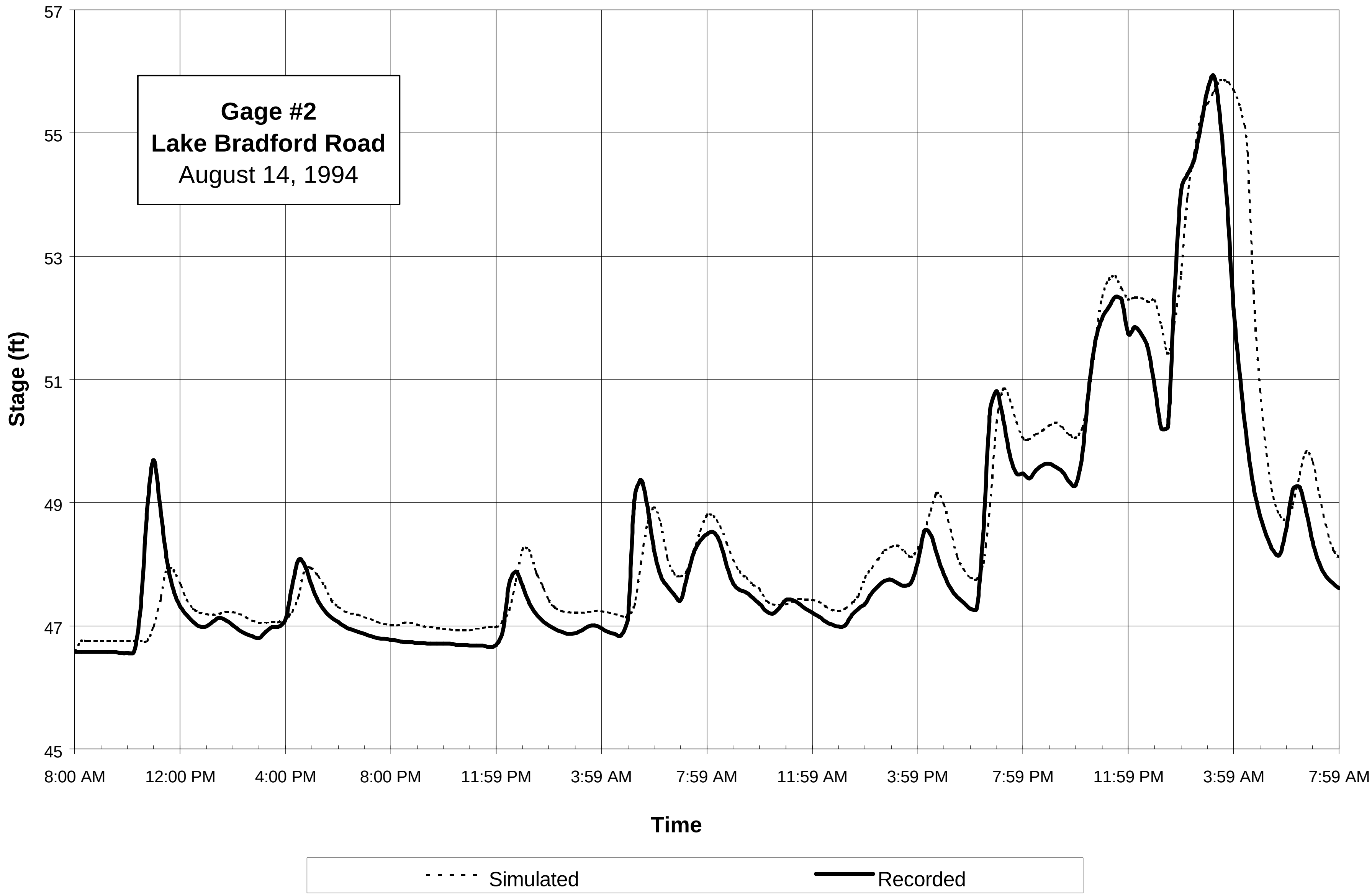
Gage #2,
Lake Bradford Road
May 15, 1994
59
57
55
53
51
49
47
45
Stage (ft)
11:45 AM
12:45 PM
1:45 PM
2:45 PM
3:45 PM
4:45 PM
5:45 PM
6:45 PM
7:45 PM
8:45 PM
9:45 PM
Time
Simulated
Recorded



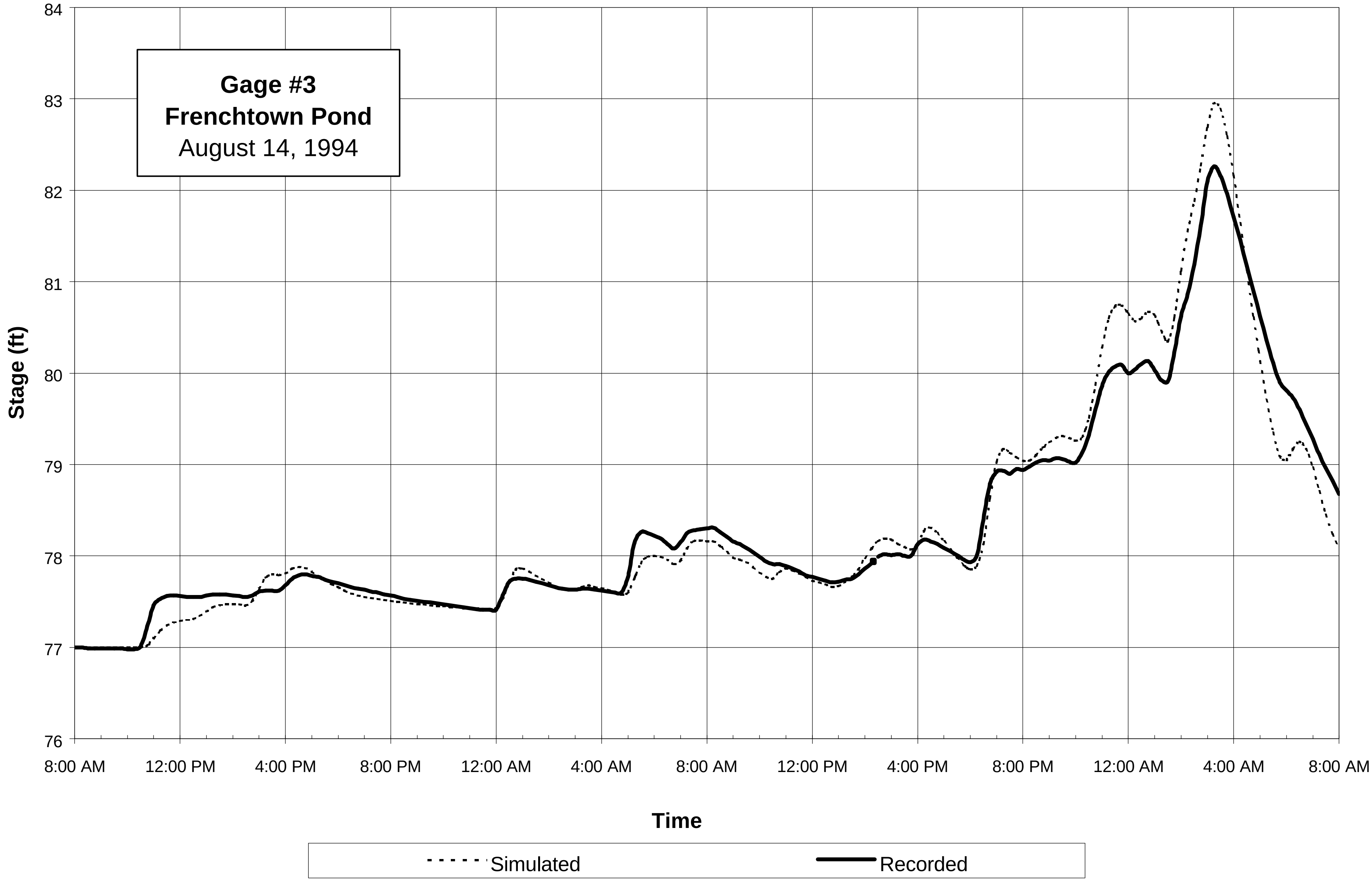
Gage #3,
Frenchtown Pond
May 15, 1994
Stage (ft)
85
84
83
82
81
80
79
78
77
76
75
11:45 AM
12:45 PM
1:45 PM
2:45 PM
3:45 PM
4:45 PM
5:45 PM
6:45 PM
7:45 PM
8:45 PM
9:45 PM
Time
Simulated
Recorded

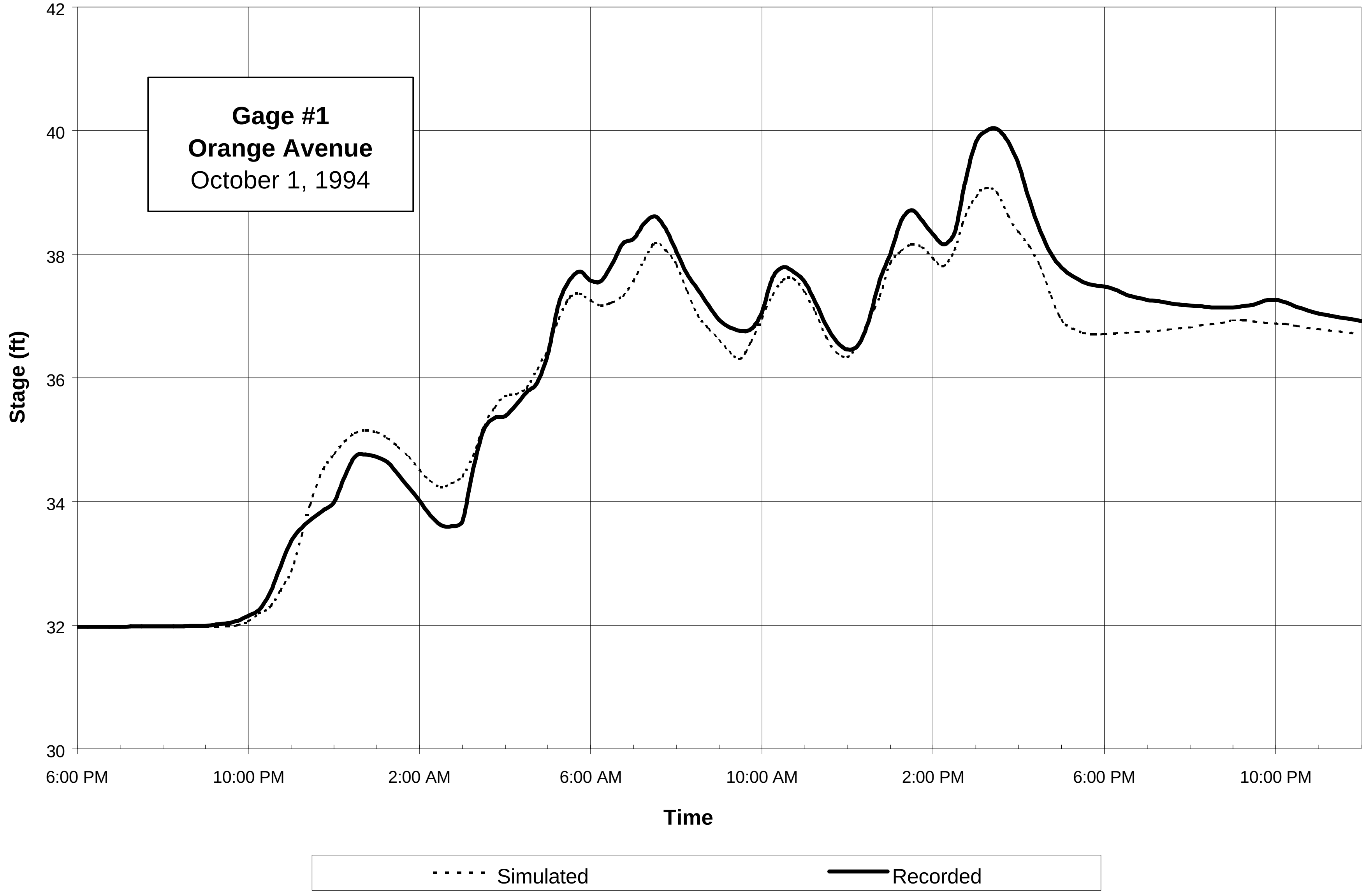


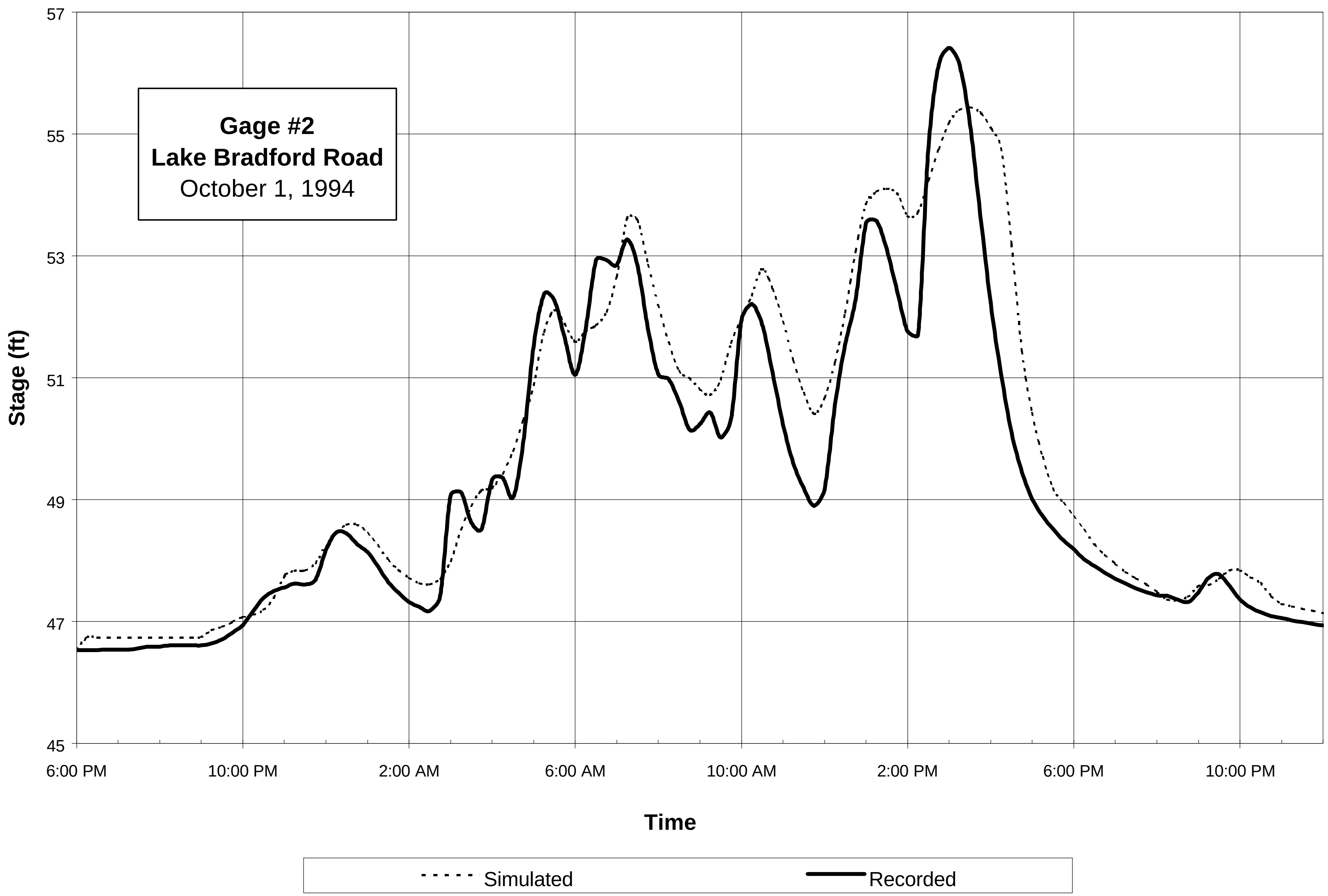
Gage #1
Orange Avenue
August 14, 1994
Stage (ft)
42
40
38
36
34
32
30
8:00 AM
12:00 PM
4:00 PM
8:00 PM
12:00 AM
4:00 AM
8:00 AM
12:00 PM
4:00 PM
8:00 PM
12:00 AM
4:00 AM
8:00 AM
Time
Simulated
Recorded

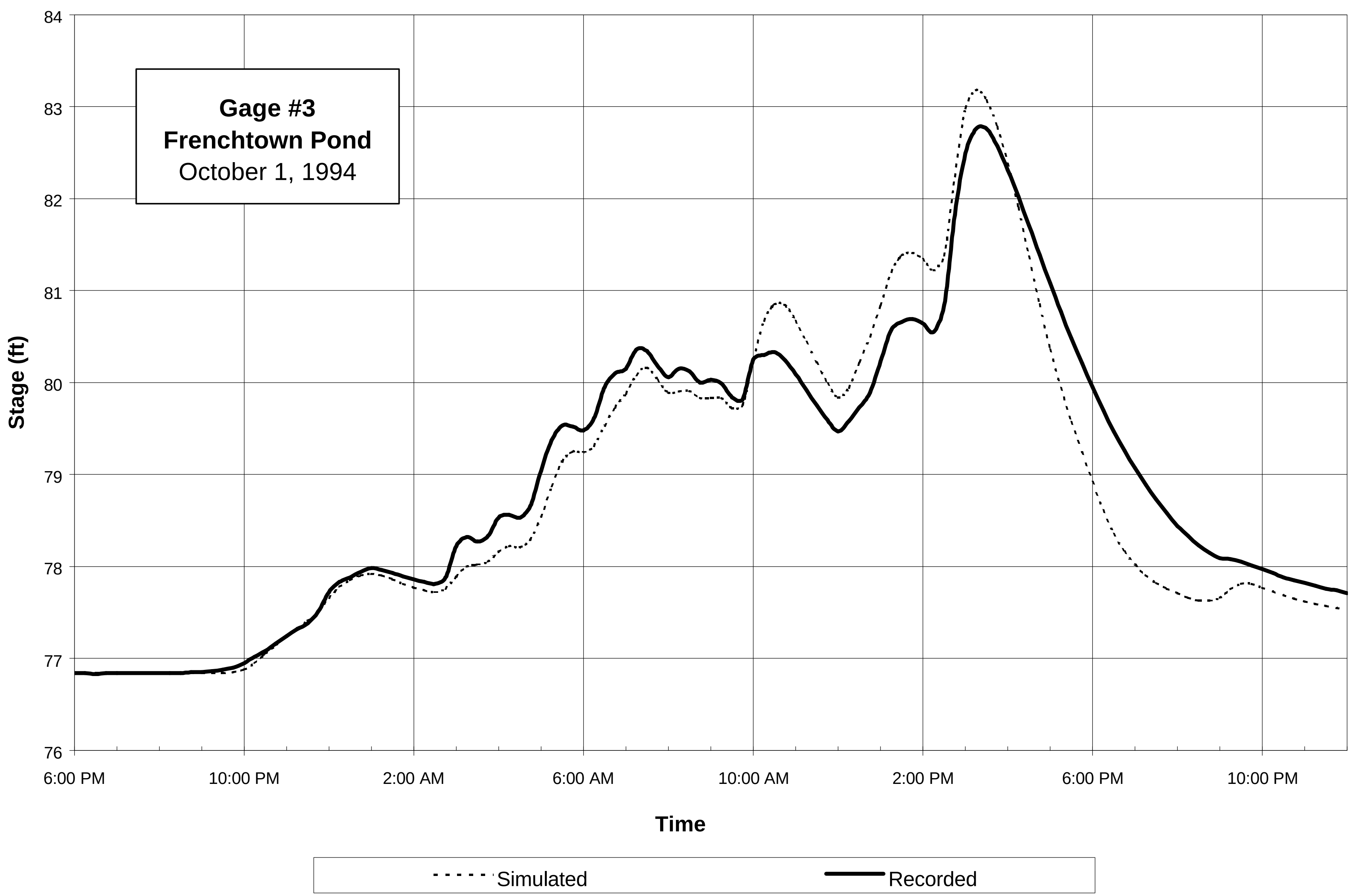


Gage #2
Lake Bradford Road
August 14, 1994
Stage (ft)
57
55
53
51
49
47
45
8:00 AM
12:00 PM
4:00 PM
8:00 PM
11:59 PM
3:59 AM
7:59 AM
11:59 AM
3:59 PM
7:59 PM
11:59 PM
3:59 AM
7:59 AM
Time
Simulated
Recorded


Gage #3
Frenchtown Pond
August 14, 1994
84
83
82
81
80
79
78
77
76
Stage (ft)
8:00 AM
12:00 PM
4:00 PM
8:00 PM
12:00 AM
4:00 AM
8:00 AM
12:00 PM
4:00 PM
8:00 PM
12:00 AM
4:00 AM
8:00 AM
Time
Simulated
Recorded


Gage #1
Orange Avenue
October 1, 1994
Stage (ft)
42
40
38
36
34
32
30
6:00 PM
10:00 PM
2:00 AM
6:00 AM
10:00 AM
2:00 PM
6:00 PM
10:00 PM
Time
Simulated
Recorded


Gage #2
Lake Bradford Road
October 1, 1994
57
55
53
51
49
47
45
Stage (ft)
6:00 PM
10:00 PM
2:00 AM
6:00 AM
10:00 AM
2:00 PM
6:00 PM
10:00 PM
Time
Simulated
Recorded



Gage #3
Frenchtown Pond
October 1, 1994
84
83
82
81
80
79
78
77
76
Stage (ft)
6:00 PM
10:00 PM
2:00 AM
6:00 AM
10:00 AM
2:00 PM
6:00 PM
10:00 PM
Time
Simulated
Recorded