UNIVERSITY OF MINNESOTA
ST. ANTHONY FALLS LABORATORY
Engineering, Environmental and Geophysical Fluid Dynamics

Project Report No. 431

Hydraulic Model Study of the Market Avenue Retention Basin (MARB)

by

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Prepared for

DELL ENGINEERING, INC.
Holland, Michigan

and

CITY OF GRAND RAPIDS
Michigan

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ACKNOWLEDGEMENTS

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INTRODUCTION

This report presents the results of a physical model study of the Market Avenue Retention Basin (MARB) located in Grand Rapids, Michigan. The model study was conducted for Dell Engineering, Inc. and the City of Grand Rapids, MI.

The MARB described herein is a combined sewer overflow retention basin located in Grand Rapids, MI. The facility consists of a pumping station with ten pumps and three retention basins. The sump is designed to function through a ten foot operating range, with an additional pump coming on-line with each one foot rise in the sump level. The water enters the first basin through any of the ten pumps and discharges into Basin Two through an outfall channel when the elevation reaches 622 ft. Basin Two is a large triangular shaped area with eight elevated channels that water is allowed to overtop and enter Basin Three. Because this is a combined sewer plant the number of pumps in operation, and hence the pumping rate, varies with the magnitude of the storm event. Each pump is rated at 105 million gallons per day (MGD), giving the maximum basin capacity of 1050 MGD when all ten pumps are in operation.

The initial scope of this study focused on the hydraulic efficiency of Basin Two as a secondary sedimentation basin. The objective was to determine if short-circuiting occurred, and if so, to test various baffle arrangements to mitigate the short-circuiting. During the assessment process, it became clear that in addition to its capacity as a settling compartment, Basin Two was also being used as a disinfection chamber. With this in mind, and since it is important for disinfection to occur properly, sufficient chlorine contact time with all effluent is necessary. Since the model study of the existing facility confirmed that the necessary contact time was not being achieved, alternative baffle designs were developed to increase the residence time to that needed for satisfactory disinfection. The results of this model study illustrate the efficiency of Basin Two as a sedimentation basin as well as a retention basin for chlorine disinfection. The information obtained from the experiments is given in the results section of the report and except where noted, all data has been scaled to existing facility values.

A forty-minute video presentation supplements this report. The video shows the experimental runs for the three most favorable baffle designs at both 30% and 100% of the design flow.

Model Design and Calibration

Model Design

A physical model including the outlet channel of Basin One, all of Basin Two, and enough of Basin Three to intercept all Basin Two discharge channels was constructed at a 1-to-15 geometrical scale. Figure 1 shows a plan view of the facility highlighting the area that was modeled in the laboratory. The length ratio was selected such that the water depths and velocities in the model provide a typical Reynolds number of 7×10^4 at 100% of the design flow. At this level of Reynolds number, the model flows are sufficiently in the turbulent flow regime to adequately reflect the hydraulics of the existing facility basin. The model was constructed of plywood with no internal framing that would materially affect the model performance. The sharp-crested weirs on the discharge channels in Basin Two were constructed of sheet metal. A plan view of the model is shown in Figure 2.

In the existing facility, Basin Two has multiple slopes leading from an elevation of 601 feet down to a trough with an average side elevation of about 600 feet. The effect of these slopes on the hydraulic capacity of the system is insignificant when the water surface level is approximately at 621 feet. Thus, the basin floor of the model was constructed flat at an elevation of 601 feet. Using the upper elevation of 601 feet for the floor was slightly conservative in its effect on the overall hydraulic capacity.

Gravitational and inertial forces predominantly affect flow phenomena involving free surface flow such as in the retention basin. Therefore, the model was designed and built using Froude similarity, which effectively balances gravitational and inertial forces. The similarity allows the scale ratios of several flow parameters to be determined, such as:

Froude Number: $$F_r = \frac{F_{MODEL}}{F_{EXISTING}} = 1$$

Length: $$l_r = \frac{l_{MODEL}}{l_{EXISTING}} = \frac{1}{15} = 0.067$$

Flow: $$Q_r = \frac{Q_{MODEL}}{Q_{EXISTING}} = (l_r^{5/2}) = 1.15\text{x}10^{-3}$$

Velocity: $$V_r = \frac{V_{MODEL}}{V_{EXISTING}} = \sqrt{l_r} = \sqrt{.067} = 0.258$$

Time $$t_r = \frac{t_{MODEL}}{t_{EXISTING}} = \sqrt{l_r} = \sqrt{.067} = 0.258$$

Typical model Reynolds number ranged from 5×10^4 to 1×10^5 in the outfall of Basin One, confirming fully turbulent flow.

Grain size analysis was performed on the samples attained from the Basin Two floor of the facility in Grand Rapids. From that, the settling velocity of the mean diameter particles was considered in order to replicate as close as possible the particulate matter in the sewer inflow. Of the materials available, unexpanded polystyrene beads were selected to best model the influent particulate matter. The beads have a specific gravity of 1.02 and a mean particle diameter of 0.85 mm. Calculations for the determination of the model sediment are shown in Appendix B.

Model Calibration

As stated earlier, the full length of Basin Three of the retention facility was not modeled. A sharp-crested weir was added in the model to the downstream end of Basin Three to provide tailwater control. The location of the weir can be seen in the plan view of the model in Figure 2. To determine the correct height of the weir, an estimated height was calculated using the equation below taken from *Open Channel Flow* (Chaudry 1996).

$$q = \frac{2}{3} C_d \sqrt{2g} H_o^{\frac{3}{2}} \quad \text{in which} \quad C_d = 0.611 + 0.08 \frac{H_o}{P}$$

where q is the specific discharge in cfs per foot width,
Cd is the coefficient of discharge,
g is the acceleration due to gravity in ft/s^2,
Ho is depth of water over the weir in ft, and
P is the height of the weir in ft.

Once the weir was constructed, the water surface elevation was obtained in Basin Three at 100% of the design flow and compared to the existing facility elevations given in the design drawings. Final adjustments were made to the height of the weir until the water surface elevations in the model matched those on the design drawings.

A broad-crested weir was constructed between the headbox and the modeled portion of the Basin One outfall channel to obtain uniform velocities entering Basin Two. Velocities were measured using a pygmy meter at six locations in the outfall cross section. From these measurements, the mean velocity at the outfall at 100% of design flow was calculated to be 4.7 feet per second (fps). This shows reasonable agreement with the theoretical mean velocity of 5.0 fps. A list of the recorded velocities is shown below in Table 1.

Table 1 - Basin One Outfall Velocities.

CROSS SECTION OF BASIN ONE OUTFALL CHANNEL LOOKING INTO BASIN TWO		
	1/3 width from left wall	1/3 width from right wall
20% of depth	A	B
50% of depth	C	D
80% of depth	E	F

BASIN ONE OUTFALL VELOCITIES		
Location	Velocity	Difference from mean
	(fps)	(%)
A	5.1	10.5
A	4.7	1.3
B	4.4	-4.5
B	4.6	-0.7
C	4.9	6.3
C	4.9	6.3
D	4.1	-12.0
D	4.3	-7.8
E	5.0	8.0
E	4.8	3.0
F	4.4	-6.1
F	4.4	-4.5

In addition to measuring the residence times for each run, the water surface elevations were recorded using point gauges measuring to 0.001 of a foot. The locations in the model where the elevation readings were taken are shown in Figure 2.

Second Test Series

The second series of tests addresses the issues of proper disinfection unveiled in the modified scope of work. After reviewing the series one model runs, it was evident that the residence time in Basin Two was insufficient for adequate disinfection. Hence, the objective for the second tests was to develop a configuration providing the longest residence time. An initial design involving three baffle walls was provided by the City of Grand Rapids and tested in the model. Ten additional runs were performed in which the baffle walls were slightly modified in length and/or direction based on the experimental conclusions of the previous designs. To provide results for a typical storm event, as well as the maximum allowable discharge, each of the configurations was tested at 30% and 100% of the flow design. One of the more favorable designs, Design 3J, was then further tested in the model. Submerged drift bodies were placed in Basin Two and timed over a given length of travel in order to determine the point velocities at several locations throughout the basin. Flow patterns were visually determined using both drift bodies and dye. To determine the true effectiveness of each of the various baffle configurations, flow patterns and point velocities were also determined for the base condition, Design MT. Figures 3 through 6 show the flow patterns and the velocities for both Design MT and Design 3J at approximately 2.5 feet beneath the water surface. In addition to the water surface elevations taken at the four locations previously described, elevations were recorded at an additional site, #2 in Basin Two. Additional point gauge readings were taken since the velocity at this location was negligible for all of the baffle configurations and, therefore, calculating the head loss across Basin Two was simplified. Figures 7 and 8 show the water surface elevation comparisons for Design MT and Design 3J, respectively.

Optimization Test Series

After reviewing several of the designs, Dell Engineering and the City of Grand Rapids requested that additional baffle configurations be tested in the model. The criteria for the new designs called for the total wall length to be less than one half of the previous configurations and the residence time to be greater than fifteen minutes at 30% flow. With that, the next experimental run, Design #4, consisted only of three turning vanes at the Basin One outfall into Basin Two. Figures 9 and 10 show the vane layout, residence times for both 30% and 100% of the design flow, and the water surface elevation comparisons for Design #4. Since the residence time for the 30% run in Design #4 was longer than that required, an additional configuration, Design #5, was tested to determine the shortest permissible wall length. The altogether wall length had been reduced by over 80% while still maintaining a sufficient residence time for disinfection. A plan view of Design #5 showing the vane layout and the residence times for both the 30% and 100% flow conditions is shown in Figure 11. The water surface elevation comparisons are shown in Figure 12. Table 3 summarizes the baffle wall length and residence times for each of the final three designs. It should be noted that all residence times reported are the average of at least five experimental runs for that condition.

Table 3 – Summary Information for Final Experimental Runs

	Total Baffle Wall Length (ft)	Residence Time (min)	Differences in Water Surface Elevations from #2 Location		
			East Location (ft)	West Location (ft)	#1 Location (ft)
Design MT at 30% Flow	---	9.0	0.27	0.27	0.12
Design MT at 100% Flow	---	3.2	0.30	0.22	0.12
Design 3J at 30% Flow	650	24.8	0.24	0.15	0.13
Design 3J at 100% Flow	650	8.3	0.27	0.21	0.24
Design 4 at 30% Flow	225	17.0	0.18	0.12	0.03
Design 4 at 100% Flow	225	5.6	0.25	0.15	0.08
Design 5 at 30% Flow	114	15.1	0.21	0.10	0.07
Design 5 at 100% Flow	114	4.7	0.22	0.16	0.04

Momentum Force Test Series

Lastly, experimental tests were done in the model to assist in the determination of the impact force on the turning vanes at the outfall of Basin One. Water depth in the Basin One outfall channel was measured for three conditions; (1) Basin Two completely empty, (2) Basin Two approximately half-filled, and (3) Basin Two completely filled. The force was then calculated using the equation below taken from *Open Channel Flow* (Chaudry 1996).

$$F = \rho Q V_x$$

where F is the impact force due to momentum flux in lb force,
ρ is the density of water in slugs/ft^3,
Q is the discharge in cfs, and
V_x is the mean velocity in the direction of the flow in ft/s.

The results of the calculations are shown in Table 4.

Table 4 – Impact Force at Basin One Outfall

	No Tail Water (Basin Two Empty)	Some Tail Water (Basin Two half-filled)	Full Tail Water (Basin Two filled)
Water Depth in the Outfall Channel, d	4.4 ft	7.5 ft	17.8 ft
Mean Velocity in the Outfall Channel, V_x	20.4 fps	11.9 fps	5.0 fps
Force due to Momentum, F	64400 lb	37600 lb	15800 lb
Elevation of the Centroid of Force	606 ft	608ft	613 ft

TEST RESULTS AND CONCLUSIONS

The value of physical modeling of systems with three-dimensional flow conditions is demonstrated in this study. Flow visualization and the ease of modifying the model offered the ability to explore many possible design alternatives in order to obtain a good solution to the problems in the basin. The first segment of the model study focused on the hydraulic efficiency of Basin Two as a sedimentation compartment. The results of the experimental runs in the model are summarized in Table 2.

The model tests of Design MT, or the existing design, offered insight as to the development of the baffle design alternatives. The objective of the second series of tests was to develop various baffle configurations to minimize the short-circuiting, and thus, increase the residence time in the basin. After testing several baffle configurations in the model, the three baffle configurations that were selected for consideration were Design 3J, Design 4, and Design 5. A summary of the experimental results for the three designs is shown in Table 3.

Care should be taken when comparing water surface elevations between locations in the model experiments. That is, the ability to accurately measure elevations in the model to 0.001 ft yields a 0.015 ft discrepancy when scaled to existing facility values

The impact force on the turning vanes due to momentum in the Basin One outfall channel was calculated for three conditions. The results of the computations are shown in Table 4.

FIGURES

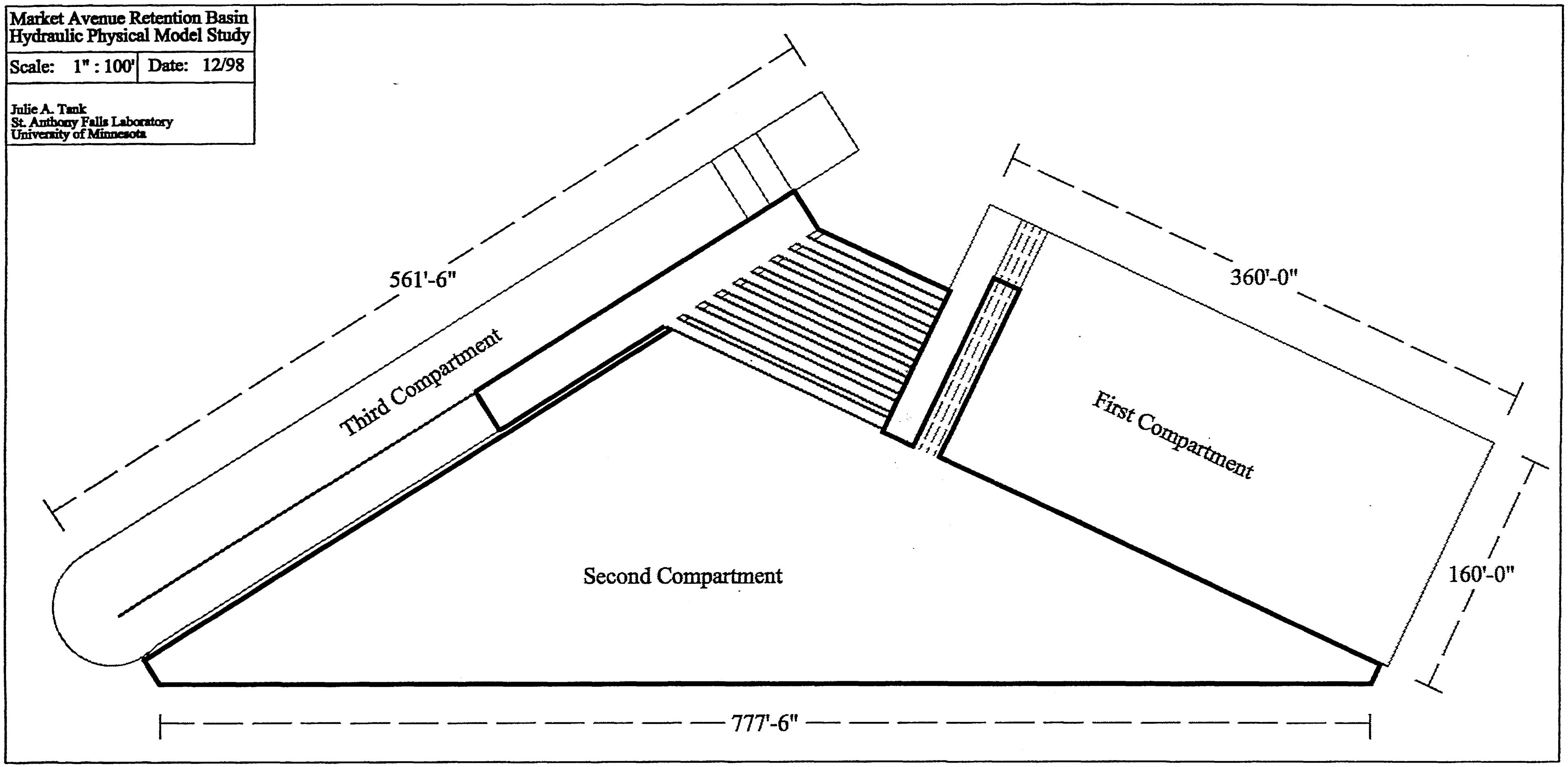

Figure 1 - Plan view of MARB Facility Highlighting the Model Study Area

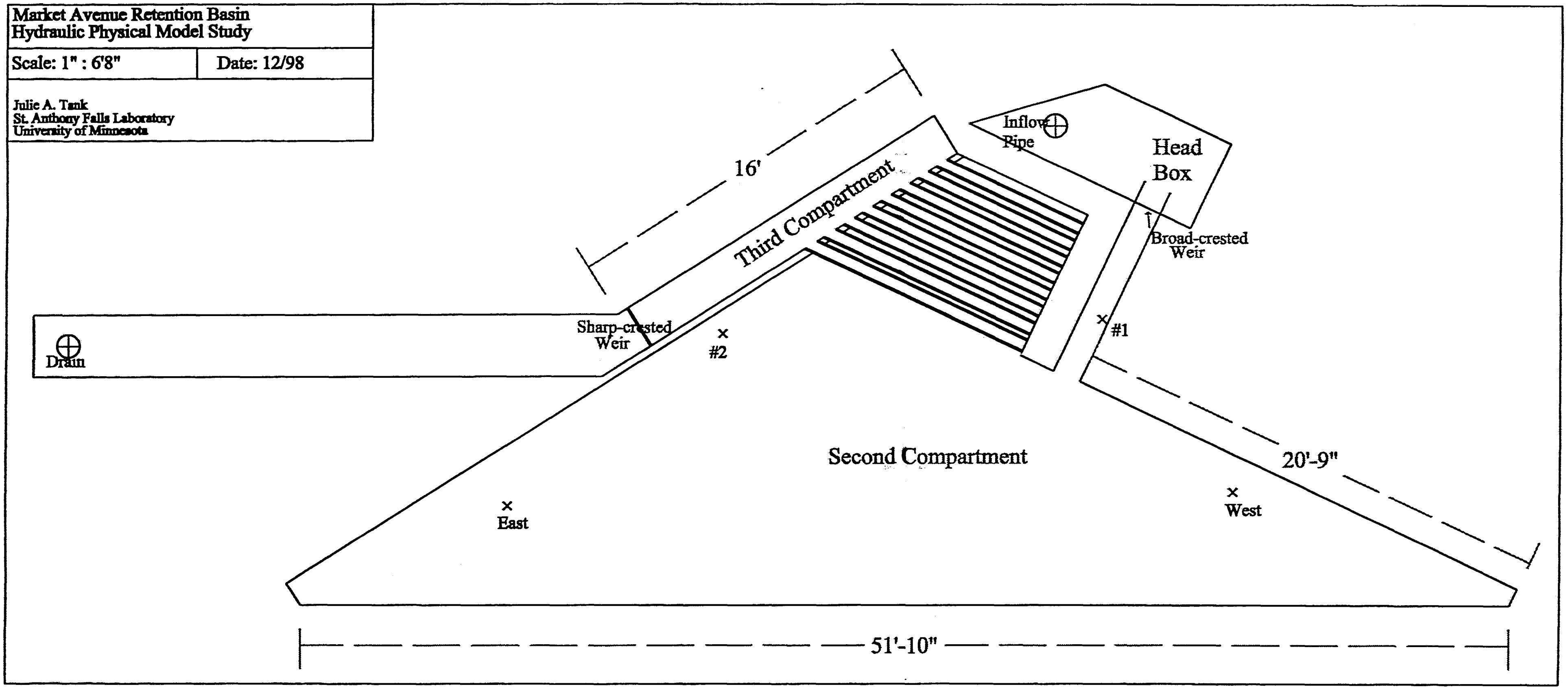

Figure 2 - Plan view of MARB Model

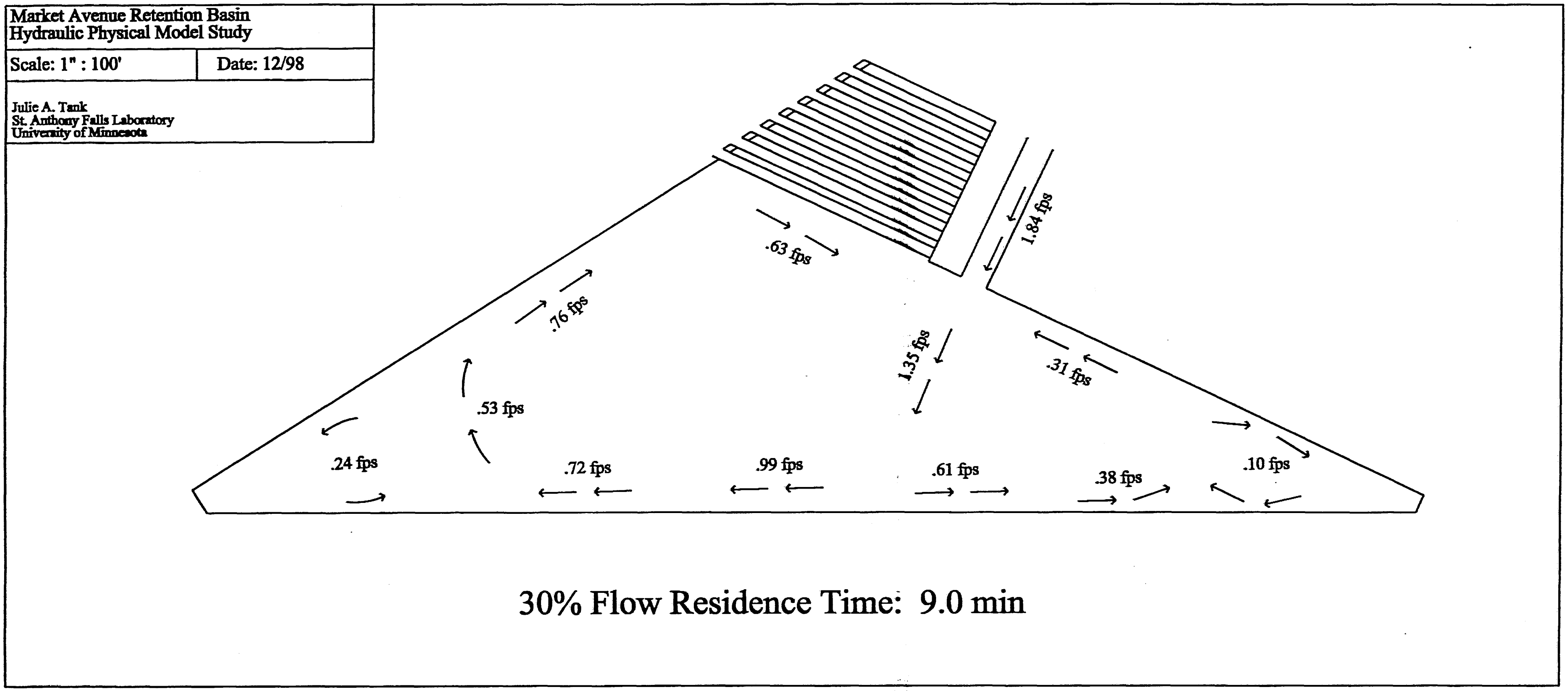

Figure 3 - Design MT, Flow Patterns and Point Velocities at 30% Flow

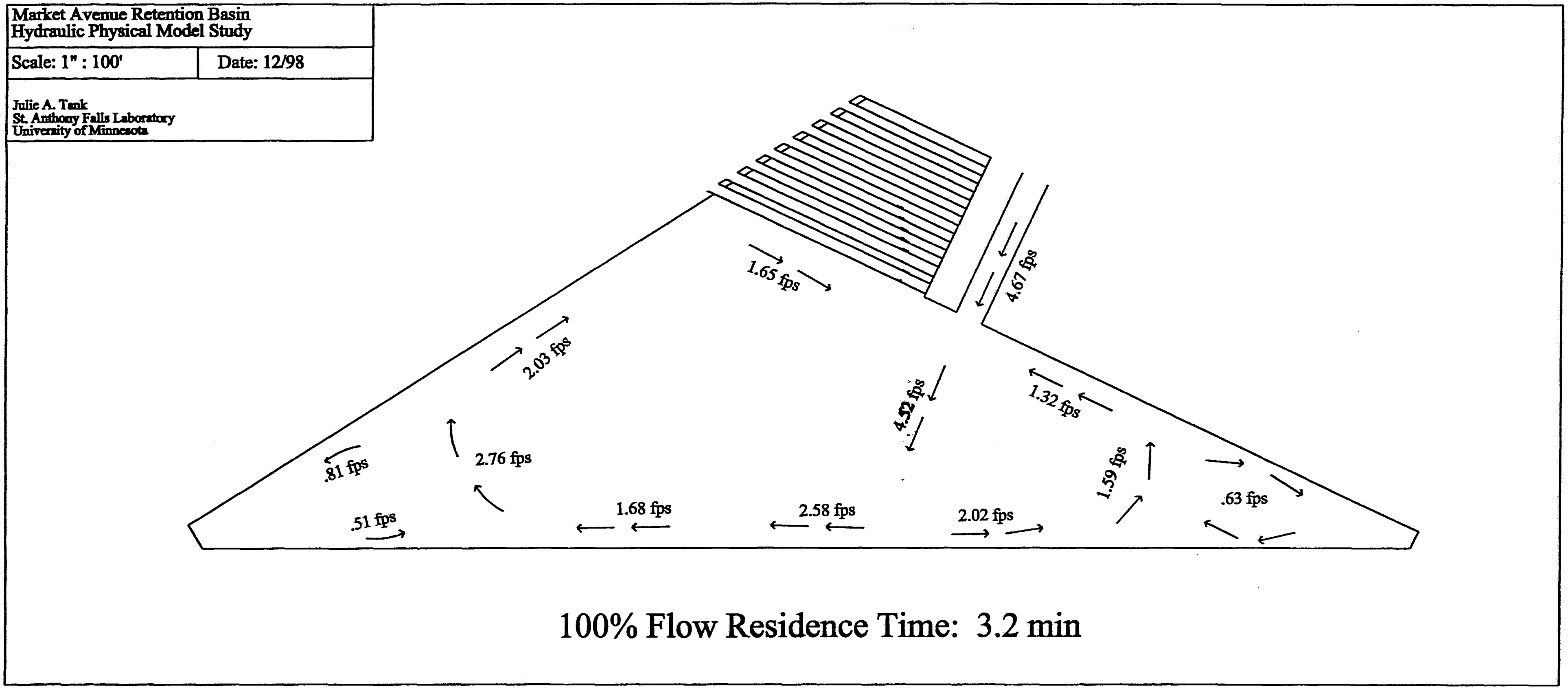

Figure 4 - Design MT, Flow Patterns and Point Velocities for 100% Flow

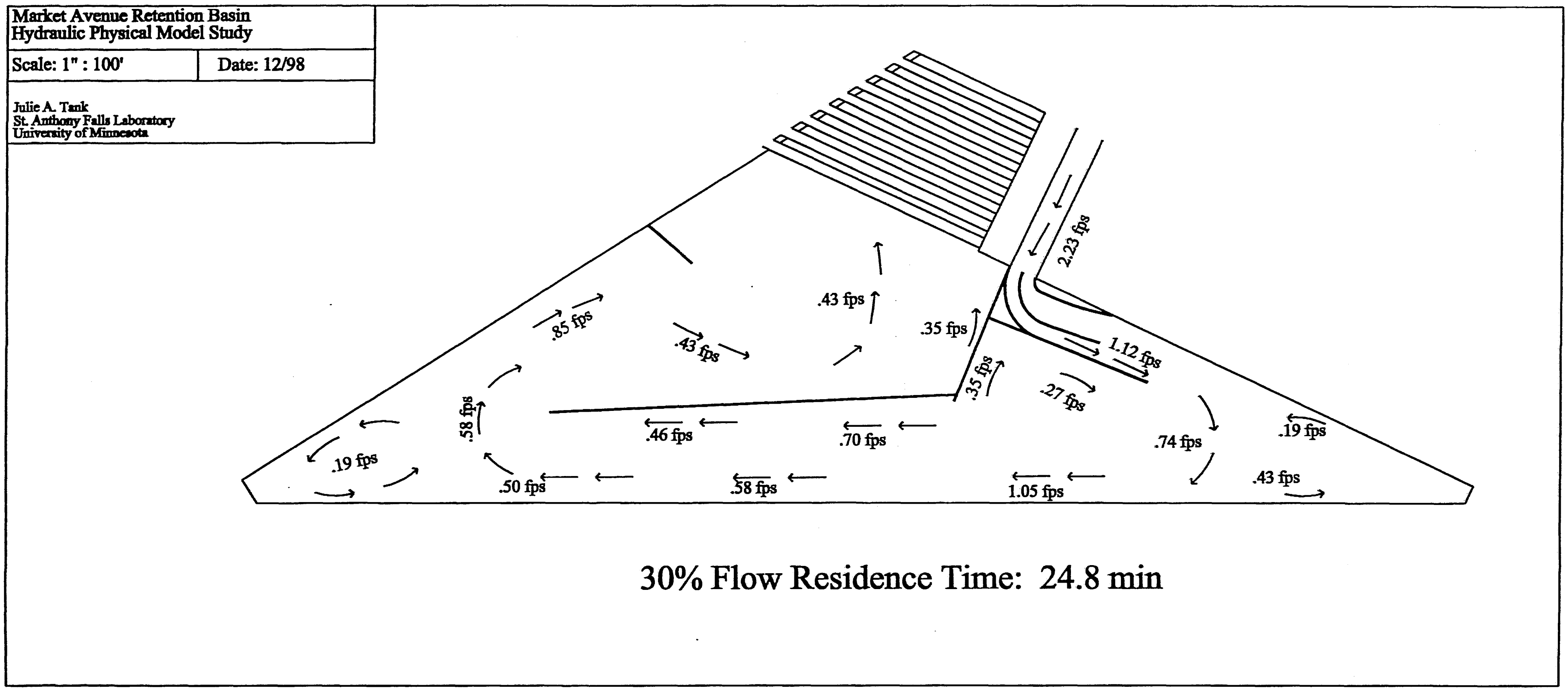

Figure 5 - Baffle Design 3J, Flow Patterns and Point Velocities for 30% Flow

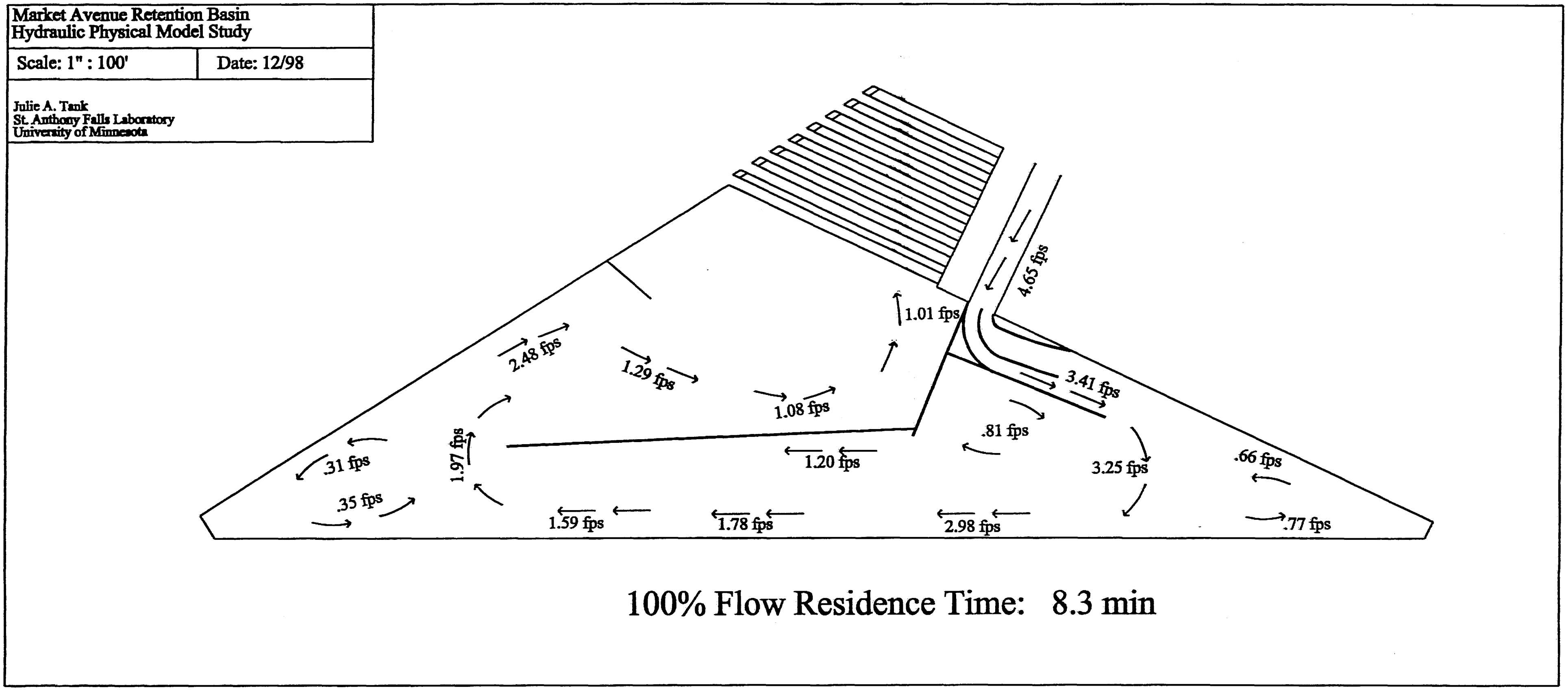

Figure 6 - Baffle Design 3J, Flow Patterns and Point Velocities for 100% Flow

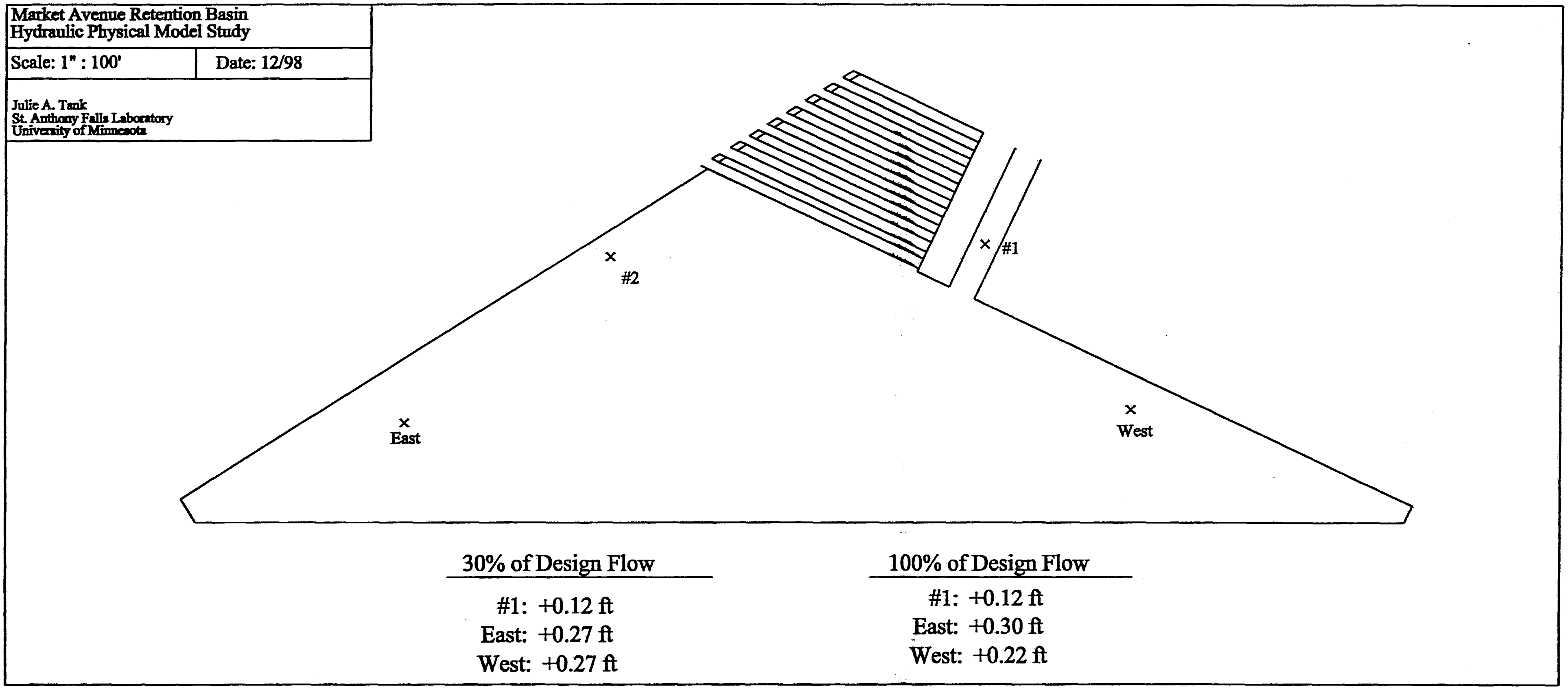

Figure 7 - Design MT, Water Surface Elevation Comparisons

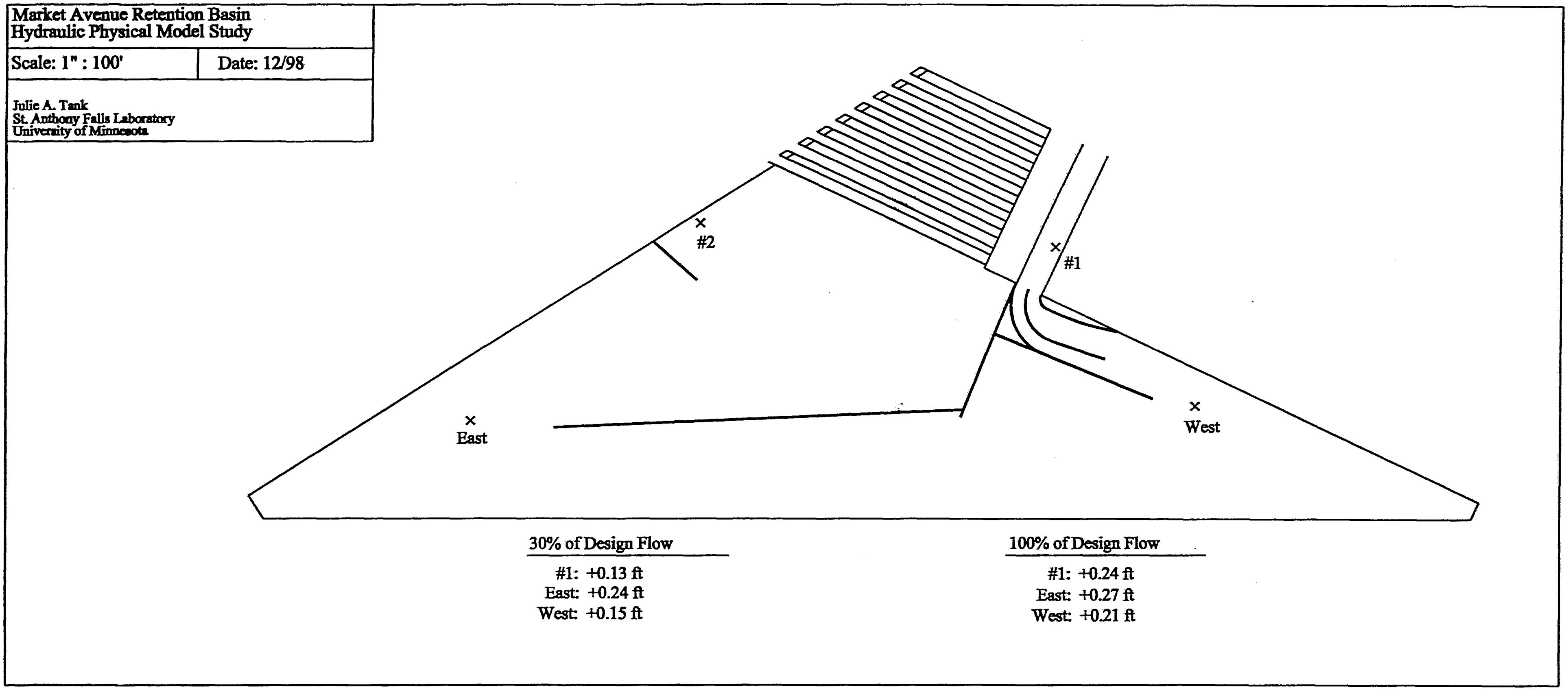

Figure 8 - Baffle Design 3J, Water Surface Elevation Comparisons

Market Avenue Retention Basin
Hydraulic Physical Model Study

Scale: 1" : 100'	Date: 12/98

Julie A. Tank
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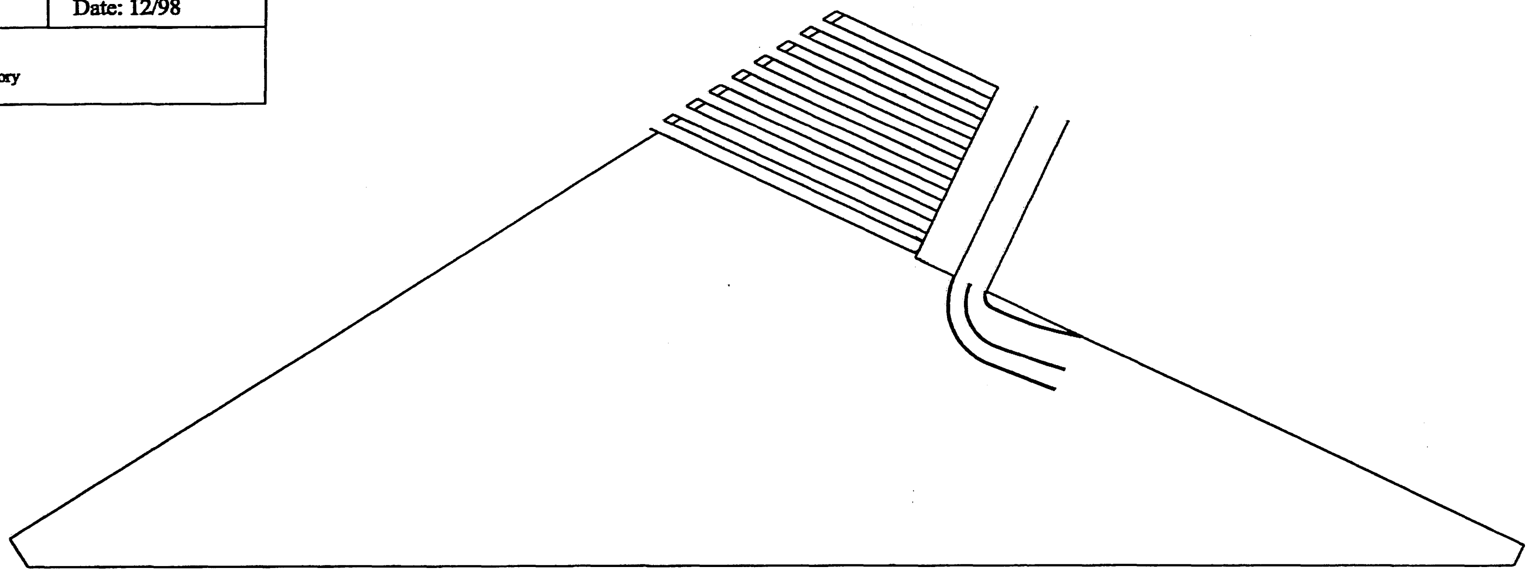

30% Flow Residence Time: 17.0 min

100% Flow Residence Time: 5.6 min

Figure 9 - Baffle Desgin #4, Residence Times

Market Avenue Retention Basin
Hydraulic Physical Model Study

Scale: 1" : 100'	Date: 12/98

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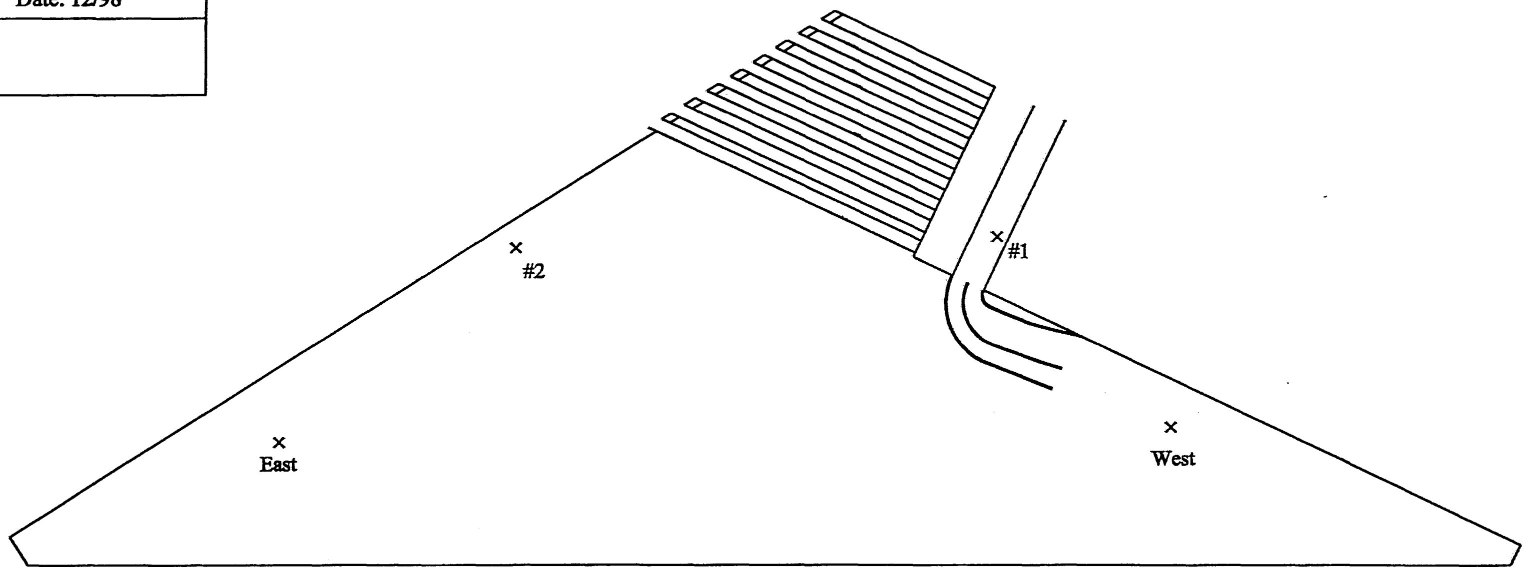

30% of Design Flow	100% of Design Flow
#1: +0.03 ft	#1: +0.08 ft
East: +0.18 ft	East: +0.25 ft
West: +0.12 ft	West: +0.15 ft

Figure 10 - Baffle Design #4, Water Surface Elevation Comparisons

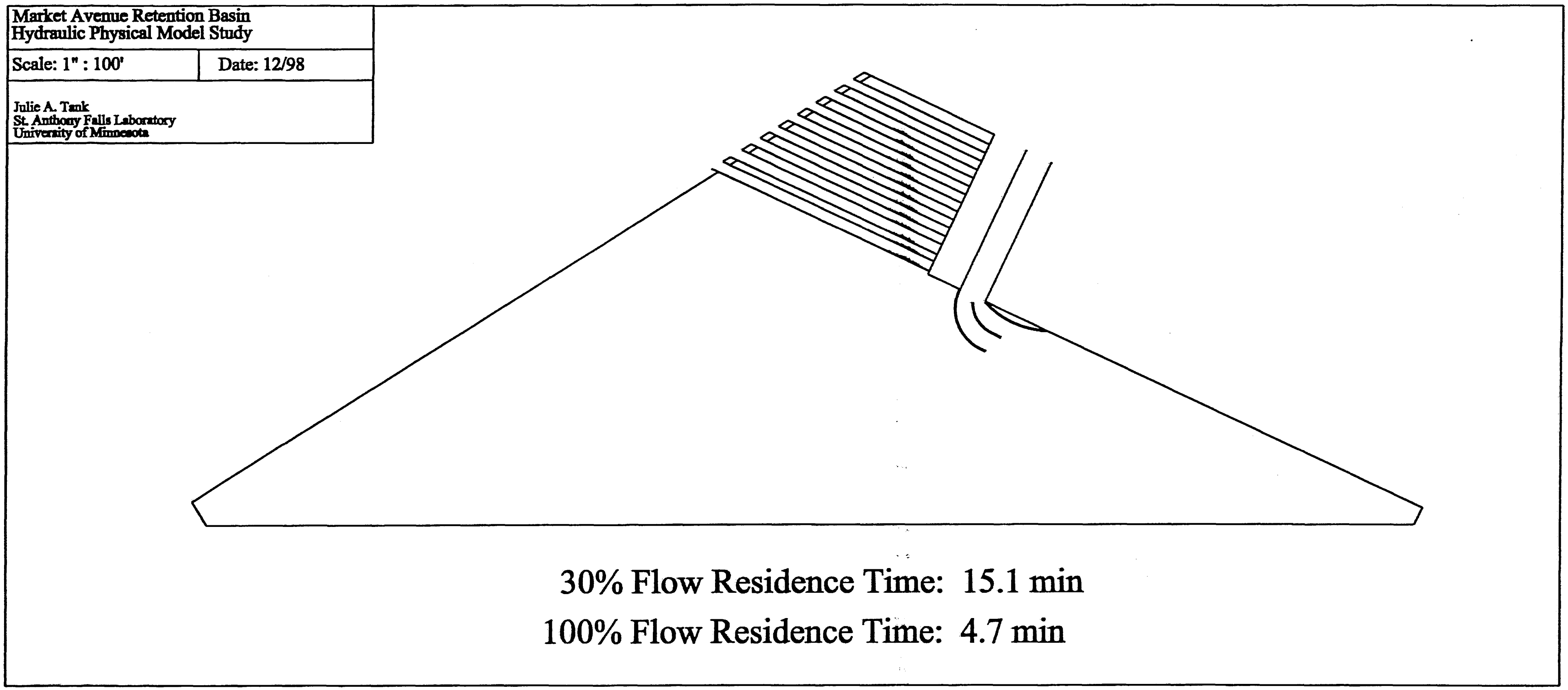

Figure 11 - Baffle Design #5, Residence Times

Market Avenue Retention Basin
Hydraulic Physical Model Study

Scale: 1" : 100' | Date: 12/98

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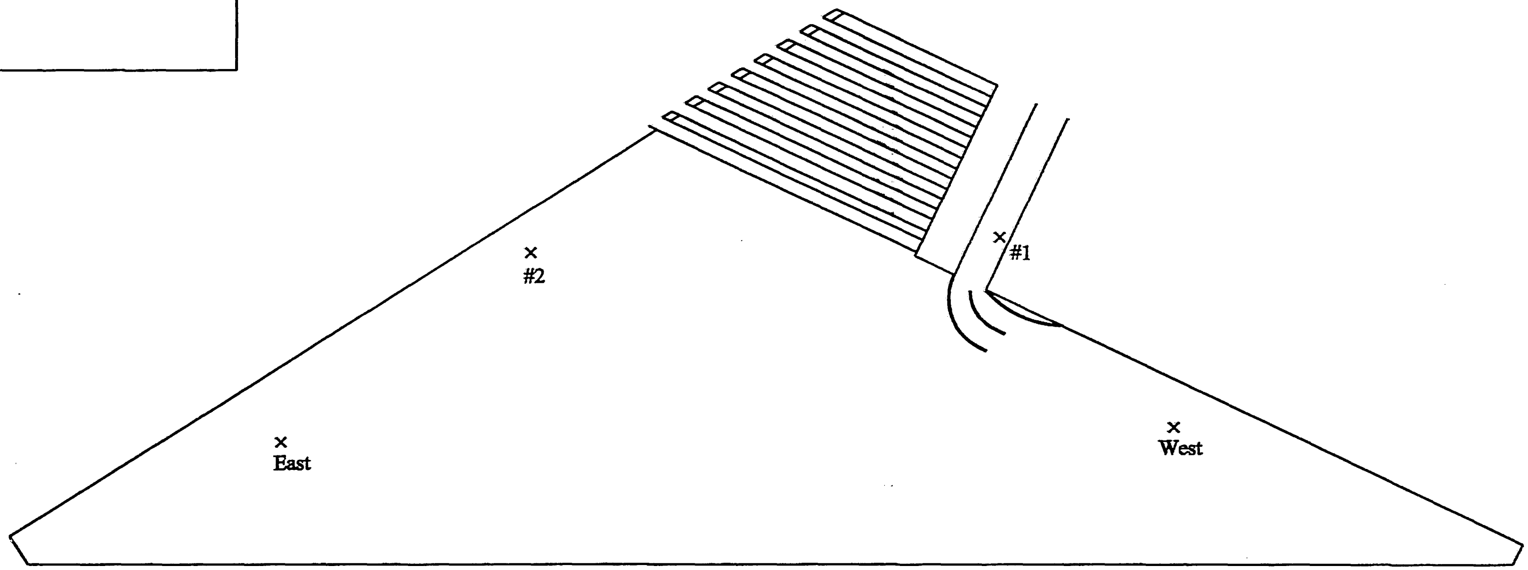
University of Minnesota



30% of Design Flow	100% of Design Flow
#1: +0.07 ft	#1: +0.04 ft
East: +0.21 ft	East: +0.22 ft
West: +0.10 ft	West: +0.16 ft

Figure 12 - Baffle Design #5, Water Surface Elevation Comparisons

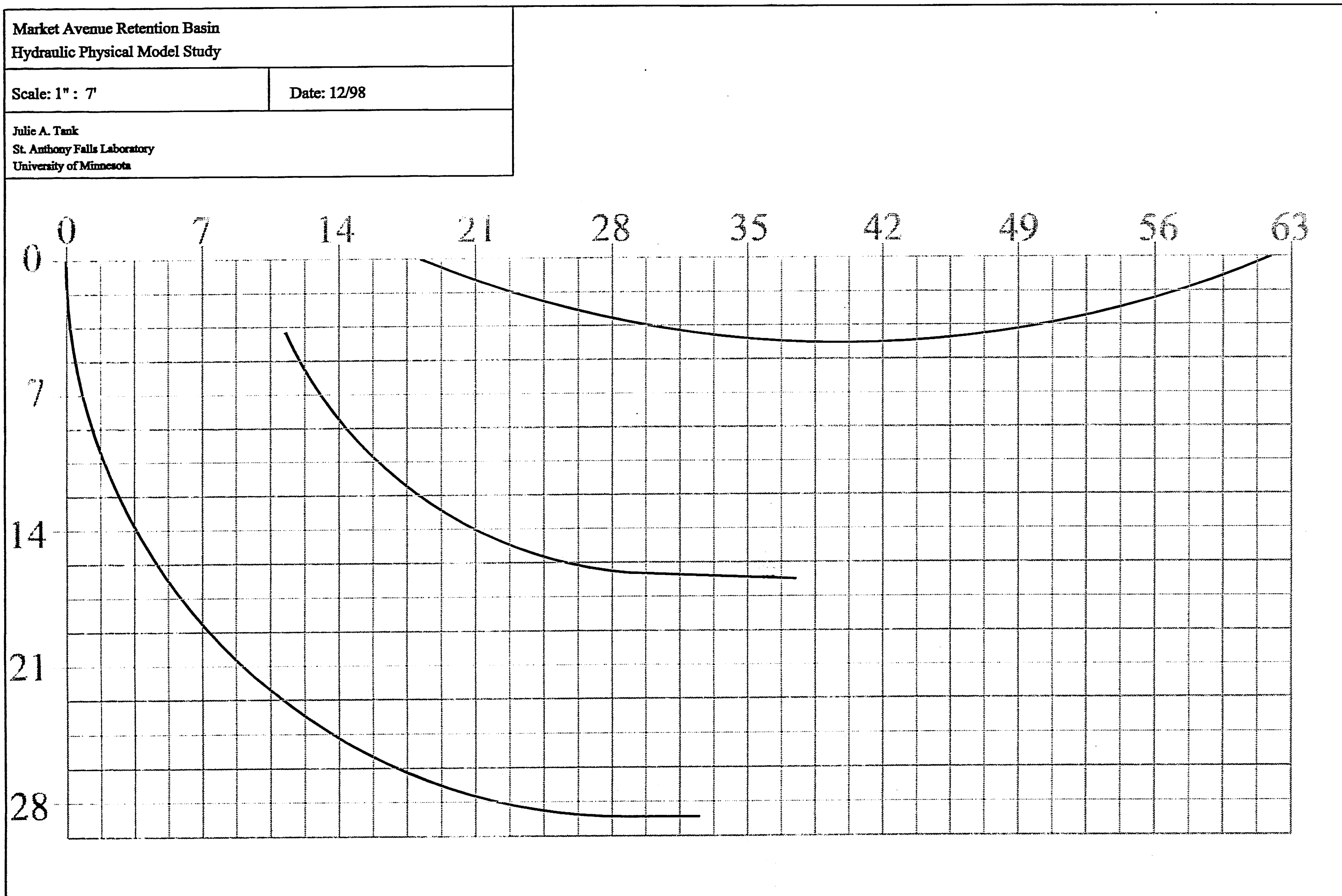


Figure 13 - Baffle Design #5, Turning Vanes Enlarged View

REFERENCES

1. Chaudry, M. Hanif. *Open-Channel Flow.* Prentice Hall, Englewood Cliffs, NJ 07632

2. ASCE Task Committee. *Sedimentation Engineering, ASCE Manual for Engineering Practice No. 54.* New York: American Society of Civil Engineers.

APPENDICES:

Appendix A: Photos

Photo 1. Overview of MARB Model.
Photo 2. Plan View of Design #5 Turning Vanes.
Photo 3. Design #5 Turning Vanes.
Photo 4. Basin Two Elevated Discharge Trough.
Photo 5. Side View of Basin Two Elevated Discharge Troughs.

Appendix B: Particulate Matter Calculations

Appendix C: Effluent Suspended Sediment Concentration Calculations

Appendix A: PHOTOS

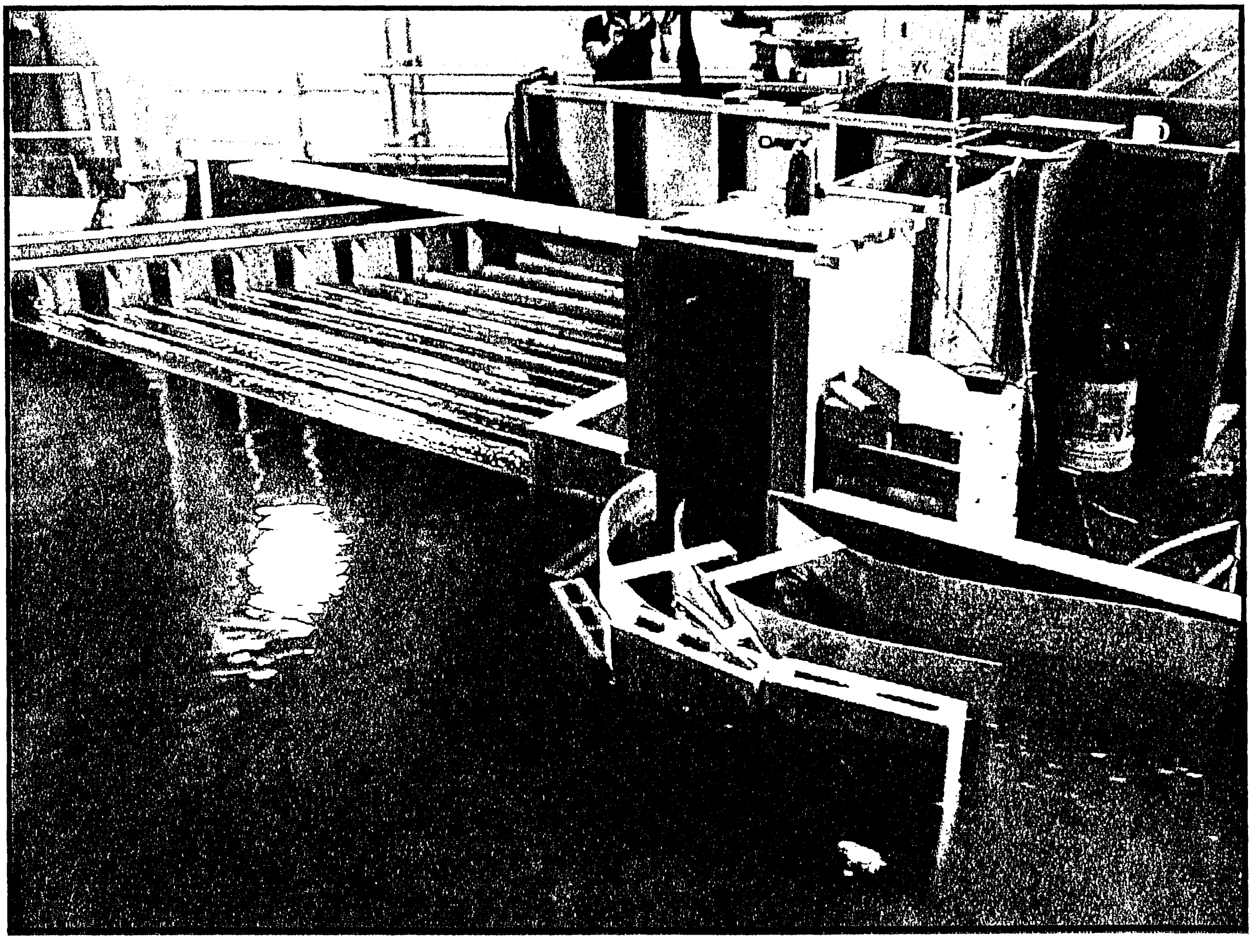


Photo 1 - Overview of MARB Model.

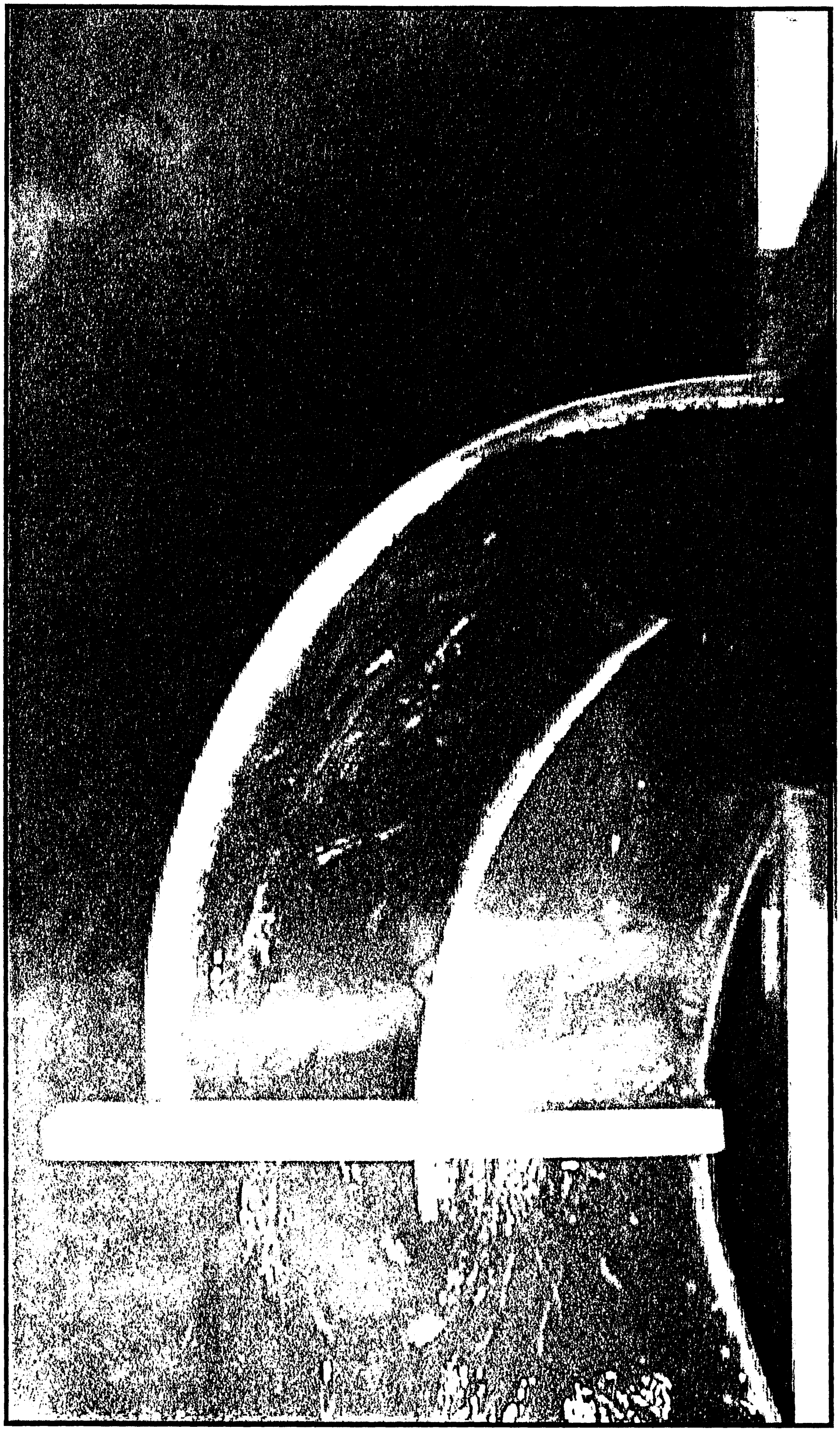


Photo 2 - Plan View of Design #5 Turning Vanes.

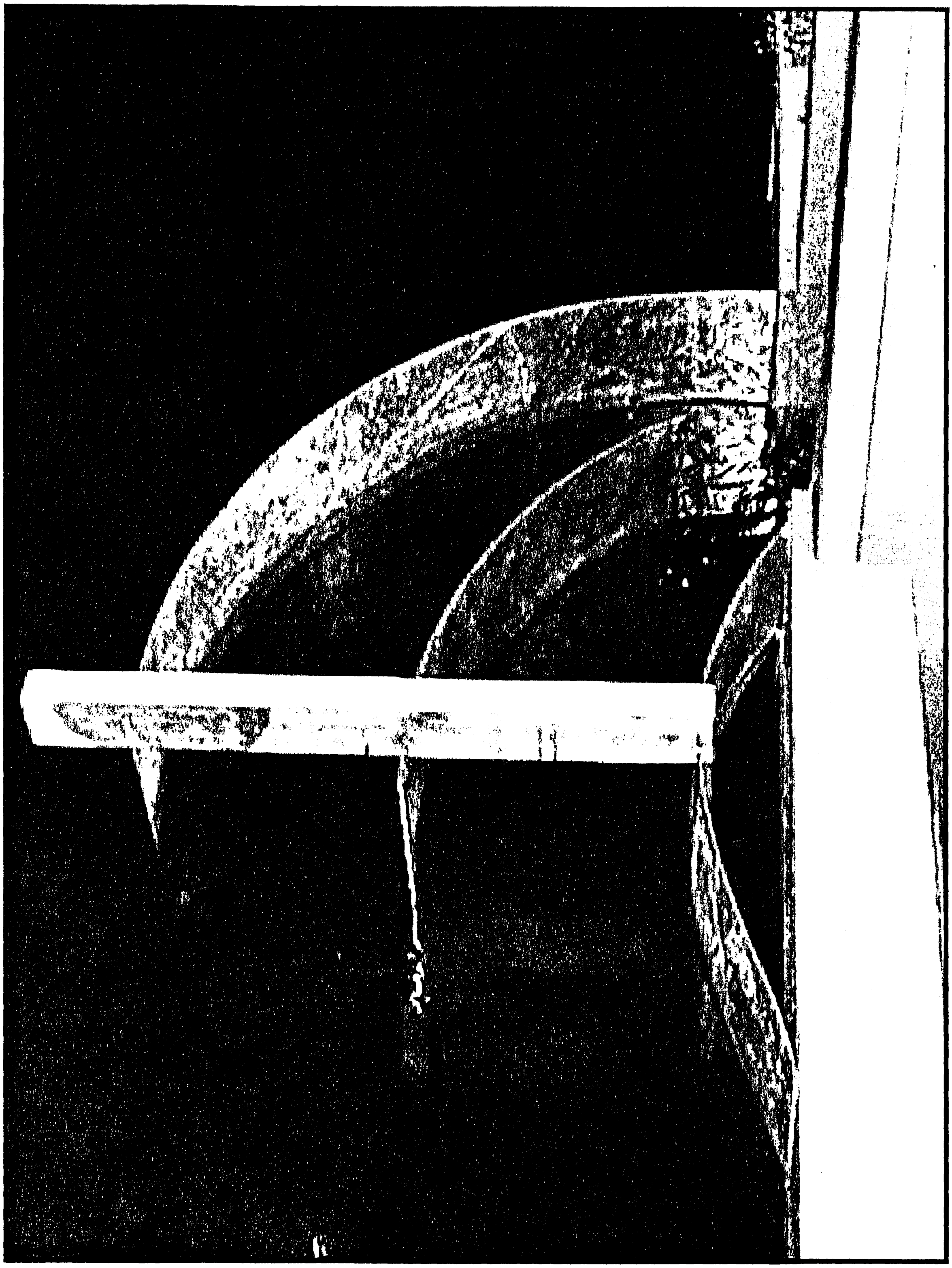


Photo 3 - Design # 5 Turning Vanes.

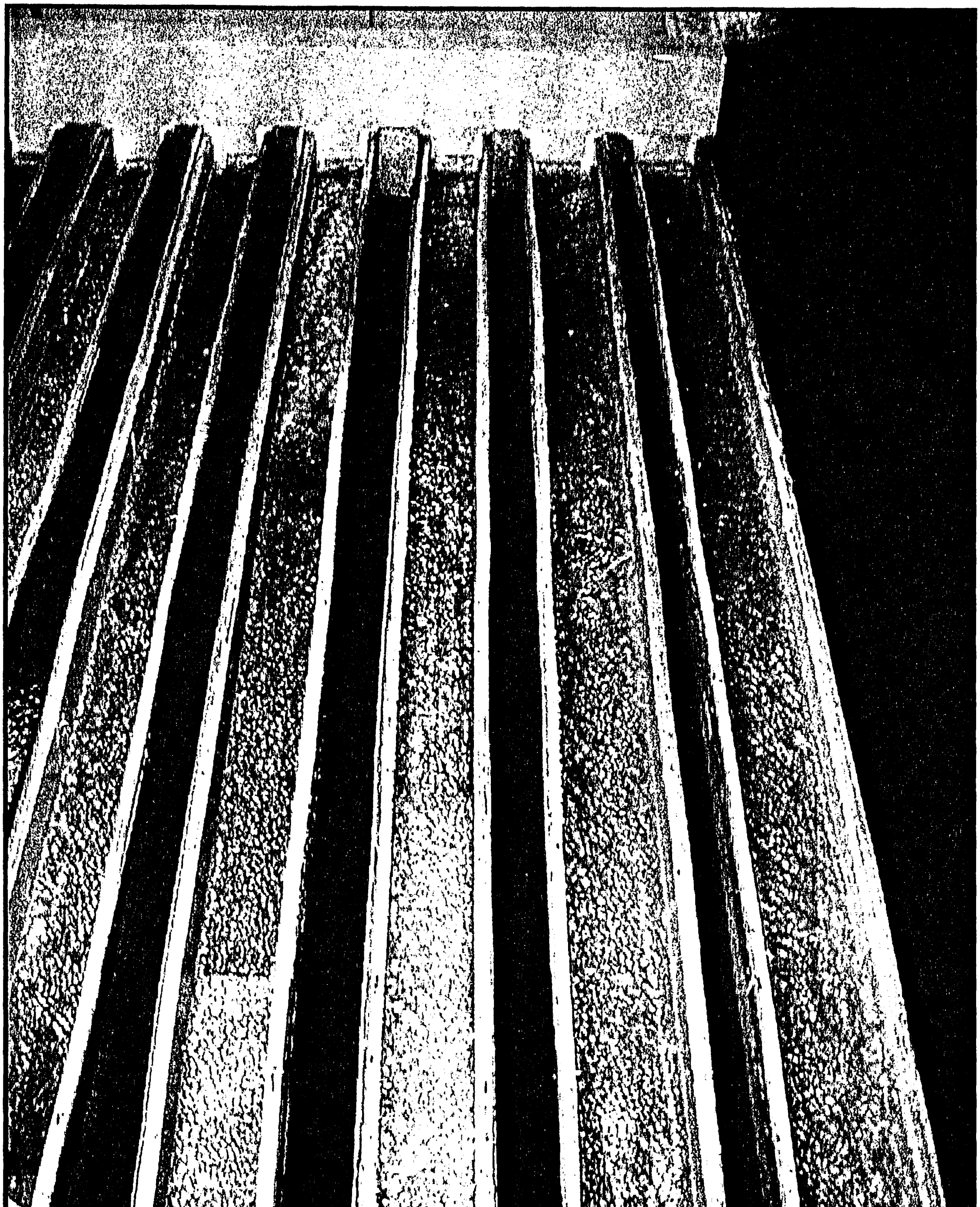


Photo 4 - Basin Two Elevated Discharge Trough.

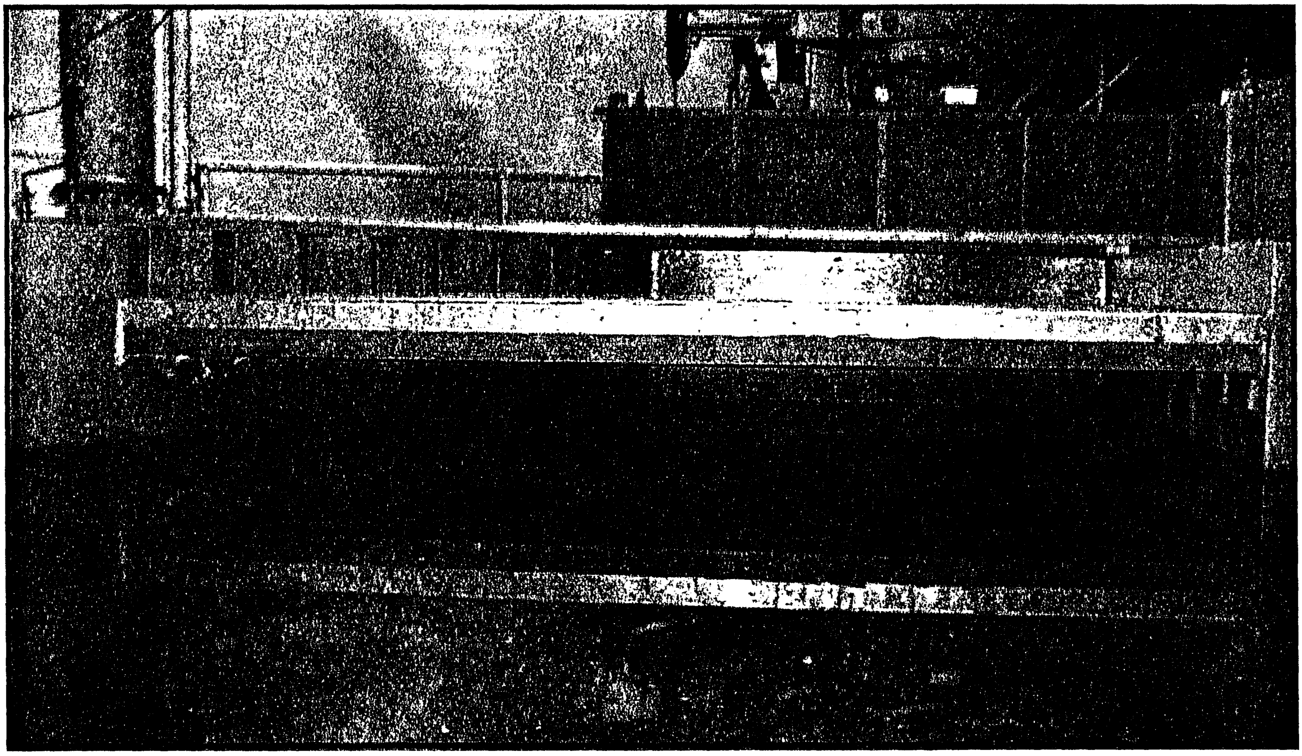
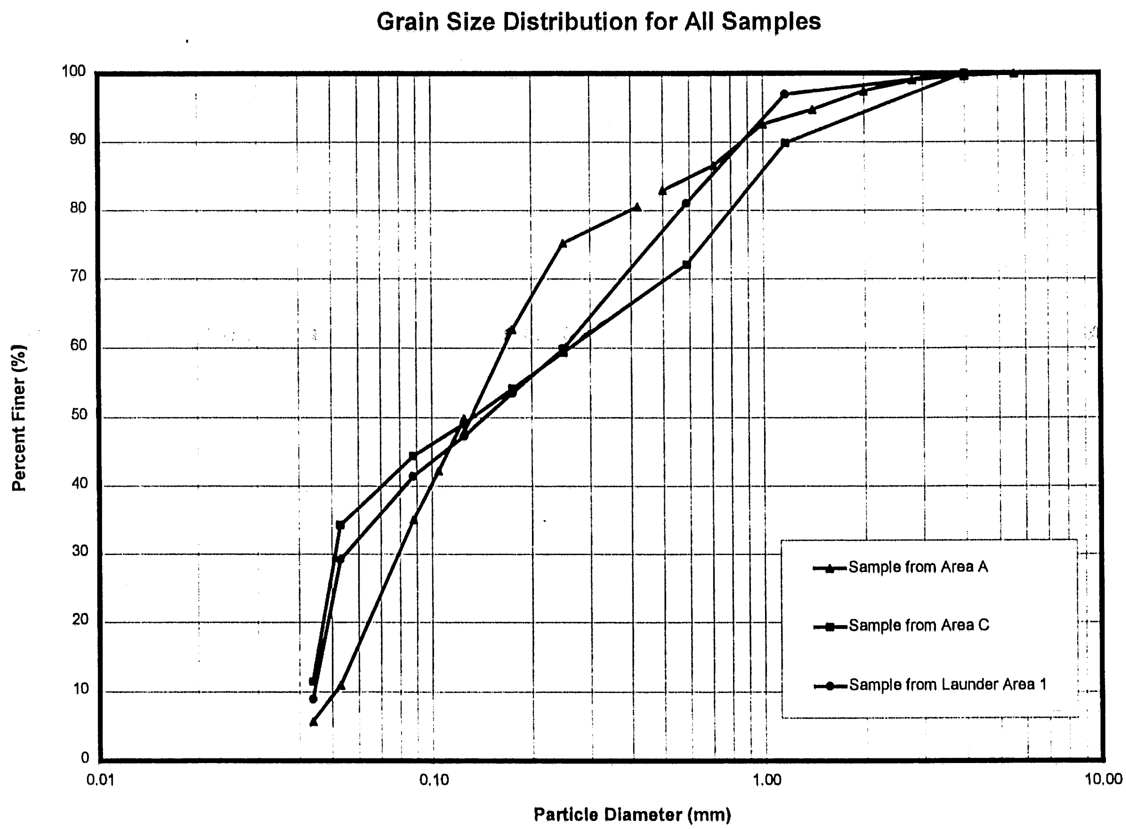


Photo 5 - Side View of Basin Two Elevated Discharge Troughs.

APPENDIX B: PARTICULATE MATTER ANALYSIS

A plot of the existing facility particulate matter particle size distribution is shown below.



The particulate matter taken from the deposits in basin two of the MARB had a specific gravity of 2.07 and a mean diameter of 0.13 mm. The particle size distributions and specific gravity calculations are shown on the above graph. In order to model this particulate matter efficiently, the settling velocity of the material must be scaled according to the scale ratios expressed in the model design section of the report. Calculations for determining the settling velocity and Reynolds number for the existing facility residue are as follows:

1. Estimate a settling velocity using Stokes Law, equation (2.2) in *Sedimentation Engineering(ASCE Manual 54)*:

$$w = \frac{gd^2}{18\upsilon}(\frac{\gamma_s - \gamma}{\gamma}) = 0.0348 fps$$

where g is the acceleration due to gravity in ft/s^2,
d is the mean grain size diameter in ft,
ν is the kinematic viscosity of water in ft^2/s,
γ is the specific weight of the sediment, and
γ is the specific weight of the fluid.

2. Calculate the particle Reynolds number and determine a drag coefficient from Fig. 2.1 in *Sedimentation Engineering(ASCE Manual 54)*:

$$R = \frac{wd}{\upsilon} = 1.484 \qquad C_d = 15$$

where w is the settling velocity of the particle determined in step 1.

3. Use equation (2.3) in *Sedimentation Engineering(ASCE Manual 54)* to calculate a new settling velocity using the drag coefficient determined above:

$$w = \sqrt{\frac{4}{3}\frac{gd}{C_d}(\frac{\gamma_s - \gamma}{\gamma})} = 0.036 fps$$

4. Calculate the new particle Reynolds number and determine the corresponding drag coefficient:

$$R = \frac{wd}{\upsilon} = 1.541 \qquad C_d = 15$$

5. No further iterations are necessary since the drag coefficients resulting from each Reynolds number calculations have converged.

Now to correctly model the sediment, the scale ratio for velocity, as given in the Model Design section of the report, is used. The calculation is as follows:

$$w_{MODEL} = \frac{w_{EXISTING}}{\sqrt{l_r}} = 0.0093 fps$$

ERRATA Sheet for Project Report No. 431

Page 1, para. 3, should read:

The initial scope of this study focused on the hydraulic efficiency of Basin Two as a secondary sedimentation basin. The objective was to determine if short-circuiting occurred, and if so, to test various baffle arrangements to mitigate the short-circuiting. During the assessment process, it became clear that in addition to its capacity as a settling compartment, Basin Two was also being used as a disinfection chamber. Since the model study of the existing facility established that the optimum contact time was not being achieved, alternative baffle designs were developed to increase the residence time. The results of this model study illustrate the efficiency of Basin Two as a sedimentation basin as well as a retention basin for chlorine disinfection. The information obtained from the experiments is given in the results section of the report and except where noted, all data has been scaled to existing facility values.

Page 6, para. 2, sentences 2 and 3 should read:

After reviewing the series one model runs, it was evident that the residence time in Basin Two was insufficient for adequate disinfection without the addition of surplus chlorine. Hence, the objective for the second tests was to develop a configuration providing the longest residence time, i.e. reducing chlorine needs.

The procedure for determining settling velocity shown above was then used to calculate the settling velocities for various materials at hand. The summary table below shows the calculated values.

	Mean Diameter (mm)	**Specific Gravity**	**Settling Velocity (fps)**	**Particle Reynolds Number**
Fine Sand	0.11	2.65	0.04	1.5
Coal Slag	0.50	1.4	0.10	15.9
Polystyrene Beads	0.85	1.02	0.02	4.8

Looking at the available materials in the table above, the polystyrene beads have a settling velocity nearest that of the existing facility particulate matter. Therefore, they were chosen to model the particulate matter in the sewer inflow.

APPENDIX C: EFFLUENT SUSPENDED SEDIMENT CONCENTRATION CALCULATIONS

1. Estimate the volume of Basin One:

$$V = lwd = (370ft)(162ft)(21ft) = 1{,}258{,}740ft^3$$

where l is the length of Basin One in ft,
w is width of Basin One in ft, and
d is the approximate depth of Basin One at 100% of design flow in ft.

2. Calculate the cross sectional velocity:

$$V = \frac{Q}{wd} = \frac{1625cfs}{(162ft)(21ft)} = 0.48fps$$

where Q is the discharge at 100% of the design flow in cfs.

3. Calculate the theoretical residence time for Basin One:

$$\Theta = \frac{l}{V} = \frac{370ft}{0.48fps} = 775s$$

4. Calculate the settling velocity required for a particle to start at the surface as it enters Basin One and reach the bottom upon leaving Basin One:

$$w = \frac{d}{\Theta} = \frac{21ft}{775s} = 0.0271fps$$

5. Calculate the mean grain size diameter that settles at the above rate using the specific gravity previously determined (Appendix B) and Stokes' Law:

$$d = \sqrt{\frac{18\upsilon w}{g\left(\frac{(\gamma_s - \gamma)}{\gamma}\right)}} = 0.0004 ft = 0.115 mm$$

6. Determine the percent of sediment greater than a mean diameter of 0.115mm from the particle size distribution plot in Appendix B. This value, 52%, represents the amount of sediment that will deposit in Basin One under these conditions.

7. Calculate the Basin One theoretical effluent suspended sediment concentration for a Basin One influent concentration of 300ppm:

$$C_1 = 300 ppm - (300 ppm)(52\%) = 144 ppm$$

8. Calculate the settling velocity for a particle to reach the floor of Basin Two based on the residence time measured in the model and scaled to existing facility values:

$$w = \frac{21 ft}{193.6 s} = 0.1084 fps$$

9. Using the procedure outlined in step 5 determine the grain size diameter that would settle at this rate given the specific gravity of the captured sediment is 2.07:

$$d = \sqrt{\frac{18\upsilon w}{g\left(\frac{(\gamma_s - \gamma)}{\gamma}\right)}} = 0.001 ft = 0.312 mm$$

10. Using the same procedure explained in step 6, the percent of sediment coarser than 0.312mm is 31%.

11. Calculate the Basin Two suspended sediment concentration:

$$C_2 = 300 ppm - (300 ppm)(31\%) = 207 ppm$$

12. Notice that the Basin Two effluent suspended sediment concentration is higher than that of Basin One. This shows that the existing design does not provide a residence time long enough to

allow sediment coming from Basin One to settle out before leaving Basin Two. Specifically, without the addition of baffle walls to increase the residence time, the results of the model tests show that Basin Two does not reduce the suspended sediment concentration.