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Final
South Broadway Detention Basin Analysis Phase Report

SUBMITTED TO:

Public Works Department
City of Albuquerque
Albuquerque, New Mexico

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C.O.A. PROJECT NO. 4165.90

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FINAL
SOUTH BROADWAY DETENTION BASIN
ANALYSIS PHASE REPORT

Prepared For:
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EXECUTIVE SUMMARY

The City of Albuquerque awarded Resource Technology, Inc. (RTI) a contract to design the north detention pond recommended in the South Broadway Sector Drainage Management Plan (Sept. 1990). At the City's direction, prior to beginning design, RTI investigated three options not included in the Drainage Management Plan.

Briefly stated, the three cases investigated are:

Case I: Divert stormwater flow in Broadway Blvd. from Santa Fe Ave. and north to the detention pond. Route outflow from the pond to the existing Bell/Commercial pump station. Divert flow from east of Broadway Blvd. between Santa Fe Ave and Bell Ave. to the pump station for discharge to the Rio Grande near Bridge St.

Case II: Divert all flow from north of Santa Fe Ave. and also from east of Broadway between Santa Fe Ave. and approximately Lewis Ave. to the pond. Route outflow from the pond to the pump station.

Case III: Divert all flow from north of Santa Fe Ave. and also from east of Broadway between Santa Fe Ave. and approximately Lewis Ave. to the pond. Route outflow from the pond to an existing storm drain in William St. south of Bridge Blvd., instead of pumping to the Rio Grande.

The results of RTI's investigation may be summarized as follows:

Case II is the best of the alternatives studied.

The required ponding volumes for Cases I and II can be accommodated within the lots selected for possible acquisition by the City. The pond for Case III is shallower and needs more surface area; it requires vacating part of the Commercial Ave. right-of-way and relocating utilities.

Compared to Case I, Case II reduces the required capacity of the Bell/Commercial pump station by approximately 53 percent (from 222 to 104 cfs) at an estimated increase in cost of $101,400. The required minimum depth of pond for Case II is greater than for Case I.

Compared to Case II, Case III reduces the required pump capacity only 18 percent (from 104 to 88 cfs) at an increase of approximately $98,200. All 88 cfs of Case III capacity is required for inflow from the local basin.

Final design should allow for uncertainties in the input data and in the computer model.

INTRODUCTION

In September, 1990, Bohannan-Huston Inc. (BHI) submitted the South Broadway Sector Drainage Management Plan Final Developed Conditions Report to the City of Albuquerque, New Mexico. This study, which was conceptual in its level of detail, used the computer model EXTRAN (part of SWMM, EPA's Storm Water Management Model) to route 100-year, 6-hour, developed-condition hydrology through storm sewers, detention ponds, and open channels in the South Broadway area. The report recommended numerous options grouped into several projects for staged construction.

The report's major recommendations include the following projects:

* A north detention pond near Santa Fe Avenue and Commercial Avenue.
* A 72-inch storm sewer in Bell Avenue from Broadway Boulevard to the existing Bell/Commercial pump station.
* Renovation of the Bell/Commercial pump station.
* A 54-inch discharge line from the pump station to the Rio Grande at Bridge Street.
* Diverting Broadway Blvd. flow south of Stadium Blvd. down Trumbull Avenue into one of two existing storm sewers in William Street.
* A south detention pond near Broadway Blvd. and Woodward Street.

Apparently there are environmental problems with the potential sites for the south pond and subsequent to the Final Developed Conditions Report of September, 1990, BHI dropped this project from their recommendations. No report amendment was prepared; however, BHI's most recent computer model (filename TRY.IT), submitted to the City in January, 1991, does not include the south pond.

The City of Albuquerque awarded Resource Technology, Inc. (RTI) the contract to design the north detention pond and directed RTI to further investigate three additional routing cases, which are described below, prior to beginning the pond design. This analysis is based on the hydrology provided in the South Broadway Sector Drainage Management Plan. The purpose of the analysis is to define the requirements for designing the pond and to help refine the requirements for renovating the Bell/Commercial pump station. This report summarizes the investigation.

The area of particular concern lies within the northern portion of the South Broadway area, generally between Stadium Blvd. and Hazeldine Ave., as shown in Figure 1. The level of detail required is only that necessary to allow the City to choose which drainage area and outflow option will be used in the final design of the north detention pond. The cost comparisons presented in this report show only initial cost differences among the three options investigated. They do not reflect any changes related to the pump station renovation or its discharge line, although these costs could be substantial and the City should consider them in deciding among the alternatives.

All three cases assume that required improvements, including any renovations to the pump station and its discharge line, are in place and working. Figure 1 is a schematic showing the locations of pipes and junctions; note that not all pipes and junctions are used in all cases. Figure 2 shows the drainage sub-basins in the north part of the South Broadway Sector. Figure 3A shows the preliminary pond layouts, Figure 4 shows the relevant differences in piping in more detail.

OPTIONS INVESTIGATED

The following three cases were investigated during this study:

Case I Santa Fe -> Pond -> Pump Station

The pond receives all flow from the Broadway system north of and including Santa Fe Ave. and from local Sub-basin SJ-1, which lies north of the pond. Flows from east of Broadway Blvd. approximately between Lewis Ave. and Santa Fe Ave. (Sub-basin SJH106 and offsite Sub-basin APAA1) are taken to the pump station by constructing a new 72-inch line down Bell from Broadway. Run-off from local Sub-basin SJ2 also enters the pump station directly. The 36-inch outlet line from the pond and the 72-inch line in Bell have free discharge into the pump station. The depth of the detention pond would be 12 feet (Figure 3A).

Case II Lewis -> Pond -> Pump Station

Additional flows from east of Broadway Blvd. and north of Lewis Avenue (Sub-basin SJH106 and offsite Sub-basin APAA1) are diverted to the pond. The existing 72-inch line in Broadway Blvd. between Garfield Avenue and Santa Fe Avenue will be reconstructed to flow north and then at the broadway Blvd./Santa Fe Ave. intersection the flow is diverted down Santa Fe Avenue to the north detention pond. To avoid "momentum overload" where flow from the north meets flow from the south, a special junction in Santa Fe Avenue may be required. A new storm sewer in Bell Avenue will not be required. As in Case I, local Sub-basin SJ2 drains directly to the pump station and there is free outfall at the pump station for the pipe from the north detention pond. The depth of the detention pond (Figure 3A) remains 12 feet.

Case III Lewis -> Pond -> Exist Storm Drain

This is similar to Case II, except that the outlet pipe from the detention pond joins an existing 36-inch storm sewer in William Street at Trumbull Avenue, which eventually discharges into the San Jose Drain instead of flowing to the pump station. In this case the pump station handles run-off only from local Sub-basin SJ2 and/or any runoff from the railroad yard that does not reach the pond. A larger pond area (including some road right of way, Figure 3A) with a shallower (8.7 ft.) depth is required to provide a positive gradient to the existing storm drain in William Street.

BASIN MODELING

The South Broadway Sector Drainage Management Plan identified a probable site for the pond adjacent to Commercial Avenue as shown in Figure 4. The irregularly shaped, approximately 3.5-acre site consists of Tracts 55A, 55B, 59, 61, and 64 of MRGCD Map 40 and Tracts A and B of the Guelfi Brothers Land Division. There are two houses, some storage buildings, and one or two good trees but the site is largely vacant. Both the inlet and outlet pipes will be located along the south side of the pond. Design side slopes are 3H:1V. Based on discussion with the City, RTI allowed a 14-foot strip on the south side of the site for access-landscaping-fencing and an 8-foot landscaping-fencing stip on all other sides.

The present investigation assumes fully-developed sub-basin conditions with proposed improvements in place and, in fact, the portion of the sub-basin near and upstream of the pond is almost fully developed at present. Hydrology is derived from the 100-year, 6-hour storm as provided by BHI in the South Broadway Sector DMP, with minor extensions of the hydrographs to allow longer durations, as discussed below.

Although the present study focuses on the portion of the basin north of Stadium Blvd., the entire South Broadway Sector basin is modeled so that the downstream effects of the various options can also be checked. In particular, Case III routes more water to the southern part of the basin than do Cases I and II or any of the options described in the DMP. In line with BHI's final computer model data file submittal to the City ("TRY.IT", January 1991), the present investigation assumes no south detention pond.

In addition to the DMP, BHI supplied the City with IBM-PC compatible diskettes containing data files and the program files needed to run EXTRAN (Version 4.03). However, all of the DMP computer runs including the initial TRY.IT were run on Version 4.04 of EXTRAN. Almost all the input data for the present study came from either the DMP or the diskettes. RTI developed the remainder of the needed data.

RTI conducted field studies which verified that:

- The current (dry season) depth to water table is at least 17 feet.
- The fence-line elevation in the railroad yard west of the site is approximately 4949.6 at the low point where offsite flow enters the site.
- The elevation just east of the fence is approximately 4949.1.

RTI used the data and line, node and manhole designations and labeling developed by BHI except where changes were necessary as follows:

- Reversing slope and flow in Broadway from Garfield Ave. to Santa Fe Ave. (Cases II and III, junctions L14-61 to K14-861, pipes 10C1, 10B2, 10B1).
- Moving the inflow point for Sub-basins SJH 106 and APAA1 to Junction L14-61 (Cases II-III, L14-61 is the new upstream end of pipe 10C1).
- Pond inlet pipe(s): pipes 10A1 & 10D1, junction L14-861D (pipe 10D comes from junction K14-861D, one end matches pond invert; pipe 10D slope & inverts set to provide adequate cover at Cromwell Avenue).
- Pond outlet pipe inverts and lengths (Pipes 404 + 101 become just 101 & total length increases from 2000' to 2200'.)
- Pond area-depth-volume based on 3H:1V slopes instead of vertical walls.
- Printout time steps - to 180 seconds (3 minutes) from 100 & 200 seconds.
- Extending the time of simulation - from 2 hours to 4 hours
- Extending the input hydrographs from 2 hours to 6 hours - only the

recession of the hydrograph was linearly extended, the main portion including the peak flow was not changed.

It was necessary to extend the time of simulation because maximum water surface and maximum outflow from the pond occur at or near 2 hours. In order to do this, it was necessary to extend the input hydrographs in EXTRAN, which are entered manually by the "K3" lines at the end of the input data. Seven-hour hydrographs from hydrologic analyses using HYMO were available for many, but not all, of the subbasins. Almost all of these hydrographs went to zero shortly before or shortly after 6 hours; a few reached zero even earlier. RTI assumed a linear decline from the hydrograph value at 2 hours (as input to EXTRAN) to zero at 6 hours for all sub-basins. This tends to overestimate the inflow rate late in the simulation but the effect is negligible. In practice, then, the computer runs use the first 4 hours of the estimated 6-hour hydrographs.

EXTRAN can model surcharging, backwater effects, and interconnections between pipes, as well as accounting for the water that is stored in pipes, channels, and manholes as they fill and empty. These are capabilities that are needed for studies like this one but that very few models have.

To handle these capabilities EXTRAN employs a variety of sophisticated mathematical techniques and switches amongst them as conditions within a simulation warrant. Many of these techniques are iterative, i.e., "trial and error" or "guess and check". EXTRAN does not always converge to an exact solution when it changes from one technique to another. In particular, the beginning of surcharge, when a pipe changes from flowing not quite full to flowing full and under pressure, may result in numbers that "bounce" around the true value. These instabilities are short-lived in most cases.

Most of the graphs in this report are based on results printed at 3 minute intervals and may not reflect a "bouncing peak" that occurs between printout intervals. The summary tables do record maximum values; however, these may not reflect the true peaks as the computational convergence may not have occurred.

This study uses the same EXTRAN parameters as did the DMP study except for printout locations, printout intervals, and length of simulation. The important values are:

Integration Cycles	720
Length of Integration Steps	20 sec
Create Equivalent Conduits	1 (yes)
Intermediate Printout Intervals	9 cycles (3 min)
Summary Printout Intervals	9 cycles (3 min)
ITMAX - Maximun Iterations	30 (per node per cycle)
SURTOL - Surcharge Tolerance	0.05 (fraction)
ISOL - Code for Solution Method	0 (0 => Explicit Method)

STUDY PROCEDURES AND CONDITIONS

RTI created conceptual but realistic pond layouts based on recorded plats, topographic maps, field studies, and discussions with the City as described in the previous section. From the conceptual designs RTI developed depth-area-volume relations for input into EXTRAN. The ponds are shown in plan view in Figure 3A while Figure 3B shows the depth-area-volume relations. RTI used the DMP estimate for top-of-pond elevation at 4949.1. The bottom-of-pond elevation is 4937 for Cases I and II.

RTI also conducted field tests which showed that the water table in April 1990 was below elevation 4932, which is five feet below the proposed pond invert elevation.

The actual street right-of-way in Commercial Ave. north of the site is probably 40 feet or more as indicated by the zone atlas and by sketches provided by the City for a waterline installation. In order to provide continued access in the public right-of-way to the existing machine shop on the north side of Commercial St., for Case III (with the pond enlarged into the right-of-way) RTI assumed a 30-foot right-of-way and used a 14-foot offset instead of the 8-foot offset used in Cases I and II. This is roughly equivalent to assuming a 36-foot access lane.

Pond input parameters for this study may be summarized as shown below. Table 1 presents a comparison of input and output values.

	Water Surface Area acres	Design Depth feet	Design Capacity acre-feet
Case I	2.85	12.1	27.09
Case II	2.85	12.1	27.09
Case III	3.43	8.7	25.73

The pond invert for Case III is based on a pipe slope of approximately 1 foot in 2000 feet between the pond and the existing storm drain in William St. at Trumbull Ave. The pond invert for Cases I and II is based on a pipe slope of approximately 1 foot in 2000 feet between the pond and the pump station and on free discharge from the pipe into the pump station. According to the DMP, all pumps will be on, and capable of keeping up with all inflow to the pump station, when the water surface in the sump reaches elevation 4936.0, equivalent to a 14-foot depth. Since any water surface in the sump lower than the elevation of critical depth in the incoming pipe allows for free discharge, and since critical depth in a 36-inch pipe is about 1.5 to 1.75 feet for the range of peak pond outflow in this investigation, the 4936.0 design invert at the sump provides a measure of safety as to the free-discharge assumption.

Most of the storm drain pipes in the area are RCP (reinforced concrete pipe), for which the design Manning's roughness is usually taken as 0.013. The DMP used 0.015, an increase of 15%, to account for manholes and other head loses. This is a reasonable value for present purposes.

Design conditions for this study may be summarized as follows:

100-year 6-hour rainfall (2.4")

Fully developed conditions
Improvements per DMP in place
4-hour simulation
Manning's n = 0.015
Top-of-pond elevation 4949.1
All overflow from junctions leaves the system

For this study and the DMP, the overflows in the upper system (Edith and Roma, Broadway and Marquette) are major. According to the DMP they would leave the drainage area of the South Broadway system as overland flow, so it is appropriate not to consider them further. The other overflows appear to be small enough to neglect. For example, MH L14-334 at William Street and Wheeler Avenue has the next longest overflow time (6 minutes for Case II); this junction has a peak listed overflow rate of approximately 4.5 cfs and a cumulative volume of 1260 cu ft, roughly 0.03 ac-ft. For this study, all water at flooded junctions (i.e. manholes) leaves the system. Perhaps the most reliable ways in the EXTRAN to keep this water in the system would be to increase conduit capacity or to calculate the overflow hydrograph and manually enter it downstream with a suitable lag, but the effort is not justfied for this study.

RESULTS

The 2-inch stack of computer printouts accompanying this report contains a wealth of results. The graphs and tables at the end of this report present most of the important information in a more compact and useful form. Most of these graphs, except Fig. 6, are based on printout at three-minute simulation intervals and may not reflect instantaneous peaks shown in the summary tables. The following discussions are based on these graphs.

The proposed pipe and HGL (Hydraulic Grade Line) profiles for the inflow and outflow conduits for Cases I, II, and III are presented in Figure 5. The approximate ground elevations along the pipes are also shown. In those locations where the HGL is close to the ground surface, bolted manhole covers may be necessary.

Figure 6 is a bar graph showing all junctions (manholes, or MH) calculated to be flooded in Case I, II, or III, listed in a generally downstream direction. "Flooded manhole" is a modeling convention; usually in practice water either would be unable to enter the system or would flow out from an inlet or a manhole cover. In this analysis all overflow at a flooded manhole leaves the system and is not accounted for any further.

Figure 6 shows that routing the pond outlet water to the existing storm drain in William St. under Case III has no significant impact on manhole flooding downstream from the outlet pipe. The computer results must be kept in perspective because Figure 6 suggests that flooding at the downstream manholes may even be reduced in Case III. Furthermore, the differences between Case I and Case II affect only water removed at the Bell/Commercial pump station, upstream of Stadium Blvd; although they should have no effect downstream, the output shows minor differences.

Flooding at the upper end of the system (MH J14-993 and K14-71) is significant (roughly 1.2 ac. ft. total) but physically would leave the watershed as overland flow to the northwest; therefore, the model is a good representation of the actual loss. Improvements to the system to reduce flooding in this area would impact the capacity of the system farther downstream. The rest of the calculated flooding is very minor and in reality would probably re-enter the system after a few minutes.

Figure 7 shows the calculated flow in pipe 17 (Figure 1), the existing 36-inch storm drain in William St. south of Trumbull. (There is also a parallel 72-inch pipe in William south from Trumbull, pipe 15 in Figure 1.) The 10-inch discharge line from the sump at the Bell-Commercial pump station connects to approximately 550 ft of 24-inch line which leads to pipe 17; this lead-in line has no calculated inflow and is not modeled. The inflow to pipe 17 occurs at its south end, MH L14-349. The computer model sees an initially empty pipe, and water input at the south end initially spreads out both north and south to fill the pipe. This is the explanation for the negative flows at the start of simulation. Figure 7 also shows some instability at the start of simulation. Other than that, the figure shows what is expected: no flow once the pipe has filled under Cases I and II, and approximately 28 cfs of pond outflow under Case III.

The next figure, Figure 8, shows the water surface elevations at both ends of that same pipe. It is based on printed output and shows less flooding than does Fig. 6, which comes from the summary output tables. Fig. 8 suggests that the flooding at MH L14-345 results from a computational momentum surge which might disappear with a more detailed set of inputs.

Table 1 provides a concise description of pond characteristics for the several cases, including hydraulic data and cost comparisons. Figure 3A shows the plan views and Figures 1 and 4 show the location of the proposed pond. Figure 9 shows the inflow and outflow hydrographs to the ponds. Peak inflow rates ranging from approximately 400 to 600 cfs for Cases I through III are reduced to peak outflows of about 28 cfs. The total inflow rates shown are based on the three-minute output (not the summary table) and include inflows directly from local Sub-basin SJ1, which adjoins the pond on the north.

Figure 10 shows the water surface elevations in the pond for the three cases. In all cases the pond inverts are set at approximately minimum elevations for their outlet pipe conditions - that is, the ponds are sized for maximum volume. In Case I, without diverting flow from Sub-basins SJH-106 and APAA1 north in Broadway and then to the pond, the maximum water surface elevation is more than 4 feet below the top of pond elevation and the depth could be reduced in final design. Case II, with the extra diversion, shows about 1.8 feet of excess capacity, which seems reasonable at this stage, since some of the excess might disappear in the details of final design and the rest could remain as a safety factor, including freeboard.

Case III has the shallower depth required for gravity flow to the existing storm drain in William Street at Trumbull. It requires more surface area to compensate for the reduced depth. In this analysis the extra area is assumed to come from Commercial St. right-of-way west and north of the main pond site. The approximately 1 foot of excess capacity is acceptable. Figure 10 also shows a line which simply represents the unsuccessful attempt to combine the shallow (8.7') depth of Case III with the small pond footprint of Cases I and II. Analysis in EXTRAN shows that under these conditions the pond would overflow for more than an hour, with peak rate of 93.4 cfs and a cumulative volume of 130,000 cu. ft. (2.98 ac. ft). Given the 8.7-foot pond depth, even using up all of the 8-foot buffer around the pond perimeter would not provide the additional 2.98 ac-ft required. A careful expansion into the Commercial Street right of way might avoid some of the utility line relocation now associated with Case III.

Figure 11 shows the major decrease in required pump station capacity for Case II compared to Case I. In Case I flow from the local sub-basin (SJ2, Qp = 88 cfs) and from Sub-basin SJH-106 east of Broadway both reach the pump station very quickly. The second hump in Case I represents inflow from Sub-basin APAA1, which originates on the east side of Interstate 25 and does not begin contributing flow until 25 minutes into the simulation. The curve for Case III represents only local (Sub-basin SJ2) inflow into the pump station; the pond outflow is conveyed directly to the storm sewer in William Street. For the 100-year 6-hour storm, BHI previously calculated the total runoff for Sub-basin SJ2 as 4.3 acre-feet. The peak pumping capacity of 88 cfs seems very expensive for a sub-basin this small. Given this pumping requirement and the extra size and expense of the shallower pond, Case III is not attractive compared to Case II.

DISCUSSION

Final design for the pond, the pump station, and other improvements identified in the South Broadway Sector DMP should allow for considerable uncertainty in the input data and in EXTRAN. New hydrologic methods which are under consideration by the City may increase design inflows. The significant overflow now calculated for the upper part of the basin (nodes J14993 and K1471 especially) currently leaves the basin as overland flow. Future improvements may keep this water in the basin, putting additional demands on the pipes and the pond. It is not clear how the calculated flow from Sub-basin SJ1 to the north of the pond will actually enter the pond; some flow may bypass the pond via the railroad yard. On the other hand, the railroad yard may contribute flow not currently counted; that is, possibly part of the railroad yard should be included in Sub-basin SJ1. Lack of inlet capacity may be a problem in some places.

EXTRAN, which uses iterative methods to solve for heads and flows throughout the sub-basin, sometimes oscillates back and forth and fails to converge to a true solution. This is most common when junctions (manholes) first start to surcharge or flood. Unfortunately, this happens close to the time of peak for the inflow hydrographs in the South Broadway Sector area and EXTRAN results for peak flow and depth must be considered approximate. Refining the inflow hydrographs further, so that inflows from a sub-basin be broken down into several smaller flows, would probably reduce the calculated peak flows as well as the amount of surcharging and overflow. However, this does not seem justified given the other uncertainties involved.

There is a surprising lack of information available about the existing utilities in Broadway Blvd, Commercial Ave., William St. and nearby. Final design will require field verification for several locations. The most troubling possibility is a conflict between sanitary sewer and storm drain pipe in Broadway. However, the elevation change between Broadway Blvd. at Santa Fe Ave. and the pond invert is over 18 feet and construction should not be seriously affected.

The pond location proposed by the City seems to be the best (if not the only adequate) site available in this area, especially considering the benefits of routing Sub-basins SJH-106 and APAA1 to the pond. However, there may be local opposition, especially from property owners, and it may be desirable to pay particular attention to landscaping. With enough room and enough public interest the site might be developed as a green space. A wetlands approach could be used to improve the quality of stormwater leaving the pond, especially after small storms. This is partly a final design issue but also goes to the question of how much land the City should acquire.

The conceptual inlet to the pond is via a 10-foot strip extending north from Santa Fe Ave. approximately one-half block west of Broadway Blvd. This location is impractical for construction and marginal for maintenance access. The City has expressed interest in acquiring the property between this strip and Broadway Blvd, most of which is approximately 6 feet higher than the maximum pond elevation. Acquiring this parcel would benefit the pond but might expose it more to public view from Broadway Blvd. Consequently, this property is not included as part of the ponding area at this time.

The location and routing of the pond outlet do not matter much for modeling but are of practical concern. The DMP recommended an alignment in Commercial Avenue but the right-of-way west of the pond and south to Pacific Ave. is narrow and ill-defined, and is crowded with existing utilities. An alignment starting in John Street (John-Garfield-William) has the advantage of requiring the shortest easement through private property. However, this alignment is longer and requires more bends (and associated head loss) than the other choices, and would put construction directly in front of the area's largest park. It would also require design care in matching inlet and outlet structures.

The preferred outlet routing is along the William St. alignment to Pacific Avenue, west in Pacific to Commercial Avenue, and south in the west lane of Commercial to an existing manhole at Bell Ave. In both Pacific and Commercial the outlet line would replace the existing storm drain and would accept inflow from the existing catch basins. A major advantage of this routing is that it requires disrupting fewer utilities services, since there are no residences on the west side of Commercial, only the railroad yard.

This alignment does require a fairly long construction-drainage easement across a mostly empty lot that is part of an auto wrecking yard, as well as a short one across a second lot. (Normal maintenance access for the pond would not cross these lots.) The last block of William Street is currently unpaved and could be paved as part of this project, which might be attractive to local residents.

Especially for Cases I and II, the invert of the pond is considerably lower than the invert of the storm drain in Broadway, which has a flatter slope just north of Santa Fe Avenue (0.1% and 0.08%) than it has farther upstream. The diversion to the pond could be arranged to allow an increase in slope and capacity in this line when it is upgraded to 84 inches. Considering the depth available, the benefit could extend fairly far upstream. Also a steeper slope in the north-flowing section of Broadway (all cases except Case I) might allow a smaller pipe size, although this would mean not reusing the existing materials.

Directing the flow to the pond will require major work at the intersection of Broadway Blvd. and Santa Fe Ave., including one or more special structures. Flow will reach this intersection from north, south, and east in Case II or Case III. The design must provide for the forces and heads encountered in turning the flows and must prevent unacceptable interference between the several incoming branches.

The ponds modeled in this study each hold more than 10 acre-feet of water. Case I and Case II are more than 10 feet deep (although Case I could be kept shallower than 10 feet). However, the ponds are basically holes in the ground rather than dams, and may not fall under purview of the State Engineer's dam criteria, provided they empty in a reasonable period of time.

Potentially more troublesome is the local flood zone near the proposed pond. The South Broadway Sector DMP shows a recalculated 100-year flood zone, for existing conditions, that is much larger than is shown on Panel 28 of the 1983 FEMA maps; this is apparently a reflection of improved analysis methods. Table 3 in the DMP indicates no nearby flood areas for improved conditions. It remains to field verify that the proposed top-of-pond elevation will not adversely affect local drainage; some regrading may be necessary to maintain drainage from adjacent properties.

COSTS

Table 1 indicates that Case II costs approximately $101,400 more than Case I and that Case III costs approximately $98,200 more than Case II. These figures are based on the construction items shown in Table 2 using City of Albuquerque City Engineer's Estimated Unit Prices (1990) and engineering judgment.

<u>These cost comparisons refer only to initial cost differences for constructing the pond and the inlet and outlet piping.</u> The cost comparisons do not include items common to all three scenarios and are not estimates of complete construction costs. Nor do they reflect any differences related to the pump station renovation, its discharge line, or energy costs, although these amounts could be substantial and the City should consider them in choosing between scenarios. According to the results of this study, Cases II and III have less than half the pumping requirements of Case I, and thus lower pumping costs.

The information on Table 2 is based on the following assumptions.

1. The cost of removing the existing 72" diameter storm drain in Broadway is offset by being able to reuse most of the pipe, <u>or</u>, if the existing pipe cannot be reused, the new pipe can be smaller and less expensive.

2. An extra manhole, 100 feet of 72" line, and a junction box in Santa Fe Ave. will be used to bring flow into the pond from both north and

south. Final design may be simpler and less expensive.

3. Case II will require raising and rebuilding Commercial Ave. west of the pond. This may not be necessary if the top one foot or so of pond capacity is not needed, or if the street can remain in the freeboard area.

4. Enlarging the pond into the Commercial Avenue right-of-way would require replacing the existing water and sanitary sewer lines.

Table 2 contains no costs for relocating gas lines.

RECOMMENDATIONS

On the basis of the supplemental analyses described in this report, the Case II alternative is the best design for the North Detention Pond.

Case II, which allows for the reversal of the storm drain north along Broadway Blvd. from Lewis Street to Santa Fe Avenue, provides for full use of the proposed pond area and volumes (at a depth of 12 feet) without encroaching onto the right-of-way of Commercial Avenue. The additional depth in the pond will allow a steeper slope for the proposed 84-inch inflow pipe with a resultant improvement in hydraulic performance.

When the storm drains in Broadway Blvd. north of Santa Fe Avenue are upgraded from 72 inches to 84 inches, as recommended in the DMP, the excessively reduced slopes from Santa Fe Avenue to Iron Avenue should be reconstructed at a minimum slope of 0.0020 ft/ft, which would continue the slope of the upstream pipe. The 72-inch storm drain in Broadway Blvd. from Santa Fe Avenue to Lewis Street should be excavated and the pipe reinstalled at a reverse slope so that the pipe will drain to the north and provide adequate ground clearance at Cromwell Avenue. Final design may show that a smaller new pipe would be as effective.

A hydraulically efficient junction box will have to be designed in or near the Broadway - Santa Fe Avenue intersection to allow the southerly and northerly flow in Broadway Blvd. to be diverted west along Santa Fe Avenue with minimal head loss.

The inlet pipe outfall must be designed to prevent build-up of sediment in the pipe or erosion around the end of the pipe. Similarly, the outlet pipe entrance should be protected from erosion and from development of hydraulic transients such as vortices. Also, both inlet and outlet will require safety and trash guards, particularly if the pond area is made available for recreational use, which is also recommended.

The pond effectiveness is optimal in Case II where the inflow hydrograph has the highest peak (greater than 500 cfs) and an outflow rate (28 cfs) which is only slightly greater than the outflow rate for the other cases. Also, the Case II pond makes maximum usage of the available area and volume because the peak ponding elevation with freeboard is approximately the same as the top of the pond.

Another reason for recommending the Case II pond design is the effective use of the Bell/Commercial pump station. The maximum required pumping capacity for local inflow from Area SJ2 alone (Case III) is 88 cfs; this is only slightly less than the 104 cfs required with the addition of pond outflows (Case II). However, the pumping rate in Case II will last significantly longer.

The outlet pipe should extend south in an easement across private property and along William Street to the Bell/Commercial pump station. The William Street alignment is preferable to the John Street alignment because of its reduced number of bends along the minimal pipe slope (approximately 0.0005 ft/ft) provided.

The emergency spillway for the pond should be located along Commercial Avenue so that flows in excess of pond design will continue along Commercial Avenue, which is the historic path for flood flows.

The design of the pond and its appurtenances must be coordinated with the State Engineer Office.

APPENDIX

TABLE 1

COMPARISON OF POND ALTERNATIVES

CASE: *	I	II	III
ONSITE/+ ROW	ON	ON	+ ROW
TOP AREA (ac)	2.85	2.85	3.43
BOTTOM AREA (ac)	1.71	1.71	2.51
CAPACITY AVAIL (ac-ft)	27.09	27.09	25.73
CAPACITY USED (ac-ft)	16.17	22.65	23.23
MAX DEPTH AVAIL. (ft)	12.1	12.1	8.7
TOP ELEV. (msl ft)	4949.10	4949.10	4949.10
BOTTOM ELEV. (msl ft)	4937.00	4937.00	4940.40
MAX WS ELEV. (msl ft)	4944.85	4947.32	4948.32
TIME TO MAX WS (hrs:min)	1:52	1:57	2:15
QMAX IN (cfs) (a)	394	592	521
QMAX OUT (cfs)	29.50	35.00	27.77
QMAX @ PUMP STATION, (cfs)	222	104	88
COST COMPARISON (b)	290,720	392,100	490,300

* CASE I: SANTA FE -> POND -> PUMP STATION
II: LEWIS -> POND -> PUMP STATION
III: LEWIS -> POND -> EXISTING STORM DRAIN

NOTES:

(a) Q MAX IN INCLUDES SUB-BASIN SJ1 HYDROGRAPH

(b) THESE FIGURES REFER ONLY TO INITIAL COST DIFFERENCES FOR CONSTRUCTING THE POND AND THE INLET AND OUTLET PIPING. THE DO NOT REFLECT DIFFERENCES RELATED TO THE PUMP STATION OR ITS DISCHARGE LINE.

TABLE 2 COST DIFFERENCES
(COSTS FOR ITEMS IN COMMON NOT INCLUDED)

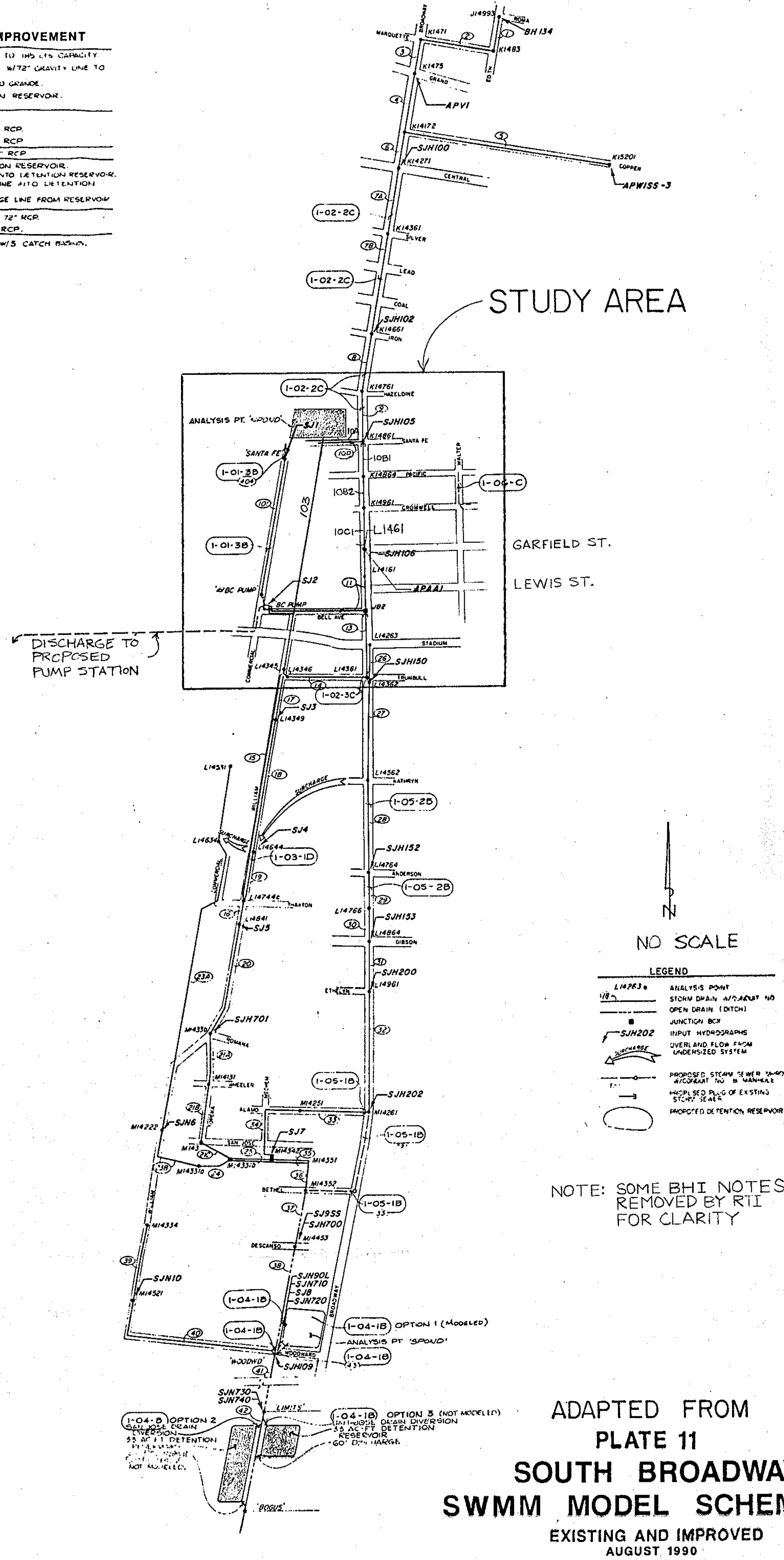
OPTION I	SANTA FE -> POND -> PUMP STATION	UNIT	QUANTITY	PRICE	AMOUNT
ITEM #					
0910.103	36" RCP REMOVE AND DISPOSE (BELL, BWAY TO PUMP STATION)	LF	1005	6.80	6834.00
0910.29	72" RCP (BELL, BWAY TO PUMP STATION)	LF	1005	125.00	125625.00
0701.20	TRENCHING FOR 48" PIPE $0.33 X 48 = 15.84	LF	1005	15.84	15919.20
	REBUILD JUNCTION BOX IN BWAY, INC. REPAVING	LS	1	6000.00	6000.00
0920.15	6' MH (BELL AT JOHN)	EA	1	2500.00	2500.00
0343.11	REMOVE AND REPLACE RESIDENTIAL PAVING, ASSUME 12' WIDTH	SY	1340	18.00	24120.00
	TRAFFIC CONTROL	LS	1	3000.00	3000.00
	EXCAV & DISP EXCESS MTL-REQ'D CAP (FROM 0601.02)	AF	16.17	6600.00	106722.00
TOTAL	**				290270.20

OPTION II	LEWIS -> POND -> PUMP STATION				
ITEM #					
0910.29	REM AND RELAY 72" RCP (BWAY GARFIELD TO SANTA FE)	LS	1035	125.00	129375.00
	(PRICED AS NEW INSTALLATION - REUSE MOST PIPE)				
0701.21	TRENCHING FOR 72" PIPE (BWAY GARFIELD TO SANTA FE)	LF	1035	34.56	35769.60
0920.15	2ND MH BWAY SANTA FE	EA	1	2500.00	2500.00
0910.29	72" RCP ~100 LF (PART WAY DOWN SANTA FE FROM BWAY)	LF	100	125.00	12500.00
0701.21	ADDITIONAL FOR TRENCH 72" (ASSUME 1/2 NORMAL PRICE)	LF	100	17.28	1728.00
	SANTA FE 72' & 84 IN 84 OUT? JUNCTION BOX	LS	1	5000.00	5000.00
0343.13	REMOVE AND REPLACE ARTERIAL PAVING ASSUME 12' WIDTH FOR 72" PIPE	SY	1380	21.00	28980.00
0343.11	ADDITIONAL REMOVE AND REPLACE RESIDENTIAL PAVING 9' WIDTH		100	18.00	1800.00
0343.11	REMOVE AND REPLACE RESIDENTIAL PAVING 20' (COMMERIAL AVE NW OF POND)		1000	18.00	18000.00
	CAP ENDS 72" PIPE	EA	2	500.00	1000.00
	TRAFFIC CONTROL	LS	1	6000.00	6000.00
	EXCAV & DISP EXCESS MTL-REQ'D CAP (FROM 0601.02)	AF	22.65	6600.00	149490.00
TOTAL	**				392142.60

OPTION III	LEWIS -> POND -> EXIST STORM DRAIN				
ITEM #					
	ALL OF OPTION II				392142.60
	PLUS				
0910.17	36 RCP (WILLIAM, BELL TO L14 345)	LF	700	44.00	30800.00
0701.10	TRENCHING < 8' FOR 36" PIPE	LF	700	10.50	7350.00
0343.11	REM AND REP PAVING ASSUME 9' FOR 36" PIPE (WILLIAM, TRUMBULL TO BELL)	SY	700	18.00	12600.00
0901.73	REMOVE AND DISPOSE OF 24" RCP (WILLIAM NEAR TRUMBULL)	LF	700	11.00	7700.00
0920.14	2 MH 6' DIA.	EA	2	2250.00	4500.00
	TRAFFIC CONTROL	LS	1	1000.00	1000.00
	ADDITIONAL ROW/VACATION	LS	1	10000.00	10000.00
	UTILITIES PROTECTION/RELOCATION....................				
0801.54	REMOVE AND RETURN WATER LINE 6"	LF	1095	3.00	3285.00
0801.02	WATERLINE C-900, CLASS 150 6" = TRENCHING <6'	LF	1095	8.40	9198.00
0343.01	REMOVE AND DISPOSE PAVEMENT 20'	SY	2555	1.90	4854.50
0801.117	REMOVE AND RELAY FIRE HYDRANT		2	600.00	1200.00
0901.03	NEW 8" P.V.C. SANITARY SEWER	LF	1095	5.40	5913.00
0901.71	REMOVE AND DISPOSAL OF 8" LINE	LF	1095	5.30	5803.50
0701.01	TRENCHING FOR 8" SAS <8'	LF	1095	7.90	8650.50
0920.07	MANHOLE 4' DIA. (6 TO 10' DEEP)	EA	2	1650.00	3300.00
	ADD. EXCAV & DISP EXCESS MTL-REQ'D CAP (FROM 0601.02) (23.23 AF TOTAL)	AF	.58	6600.00	3828.00
	MINUS				
0910.17	36" RCP (BELL, WILLIAM TO PUMP STATION)	LF	300	-43.00	-12900.00
0701.10	TRENCHING <8' FOR 36" PIPE	LF	300	-10.50	-3150.00
0343.11	REM AND REP PAVING, ASSUME 9' WIDTH (BELL, WILLIAM TO PUMP STATION)	SY	300	-18.00	-5400.00
	TRAFFIC CONTROL (WILLIAM TO PUMP STATION)	LS	1	-400.00	-400.00
TOTAL	**				490275.10

File name: COSTCOMP

PROJECT	PROPOSED IMPROVEMENT
1-01-1B	UPGRADE PUMPING STATION TO 185 cfs CAPACITY.
1-01-2B	REPLACE 36" FORCE MAIN W/72" GRAVITY LINE TO PUMP STATION BUILD FORCE MAIN TO RIO GRANDE.
1-01-3B	BUILD 24 ac-ft DETENTION RESERVOIR. INSTALL 36" RCP.
1-02-1C	INSTALL 84" RCP.
1-02-2C	REPLACE 60" RCP W/ 84" RCP.
1-02-3C	DIVERT 48" RCP TO 72" RCP
1-03-1D	REPLACE 36 RCP w/ 72" RCP
1-04-1B	BUILD 33 ac-ft DETENTION RESERVOIR. DIVERT SAN JOSE DRAIN INTO DETENTION RESERVOIR. DIVERT 36" WOODWARD LINE INTO DETENTION RESERVOIR. INSTALL 60" RCP DISCHARGE LINE FROM RESERVOIR TO SAN JOSE DRAIN
1-05-1B	PLUG 54" RCP & INSTALL 72" RCP.
1-05-2B	REPLACE 60" RCP w/ 72" RCP.
1-06-C	BUILD 24" STORM DRAIN W/3 CATCH BASINS.



ADAPTED FROM

PLATE 11

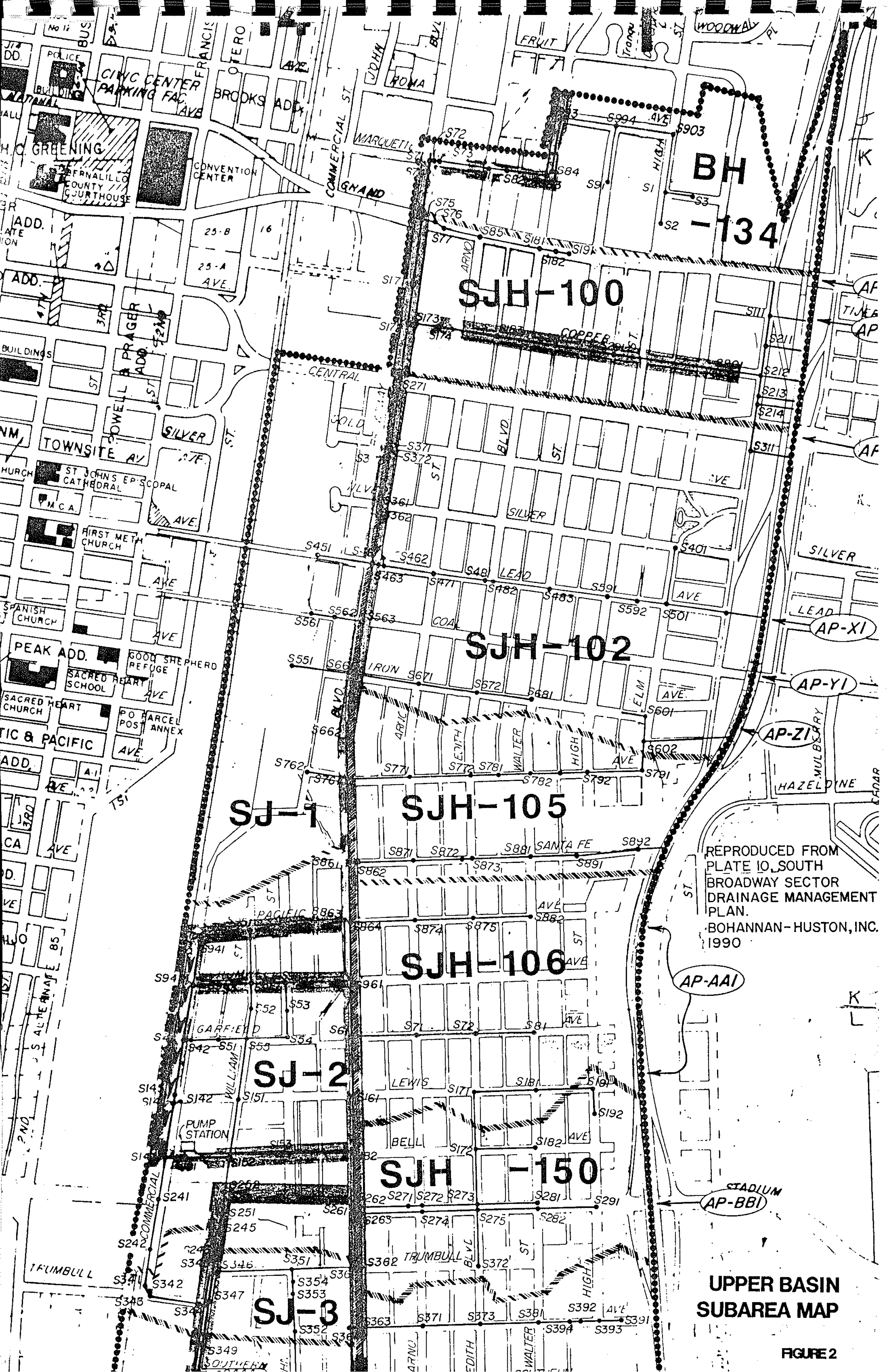
SOUTH BROADWAY SWMM MODEL SCHEMATIC

EXISTING AND IMPROVED

AUGUST 1990

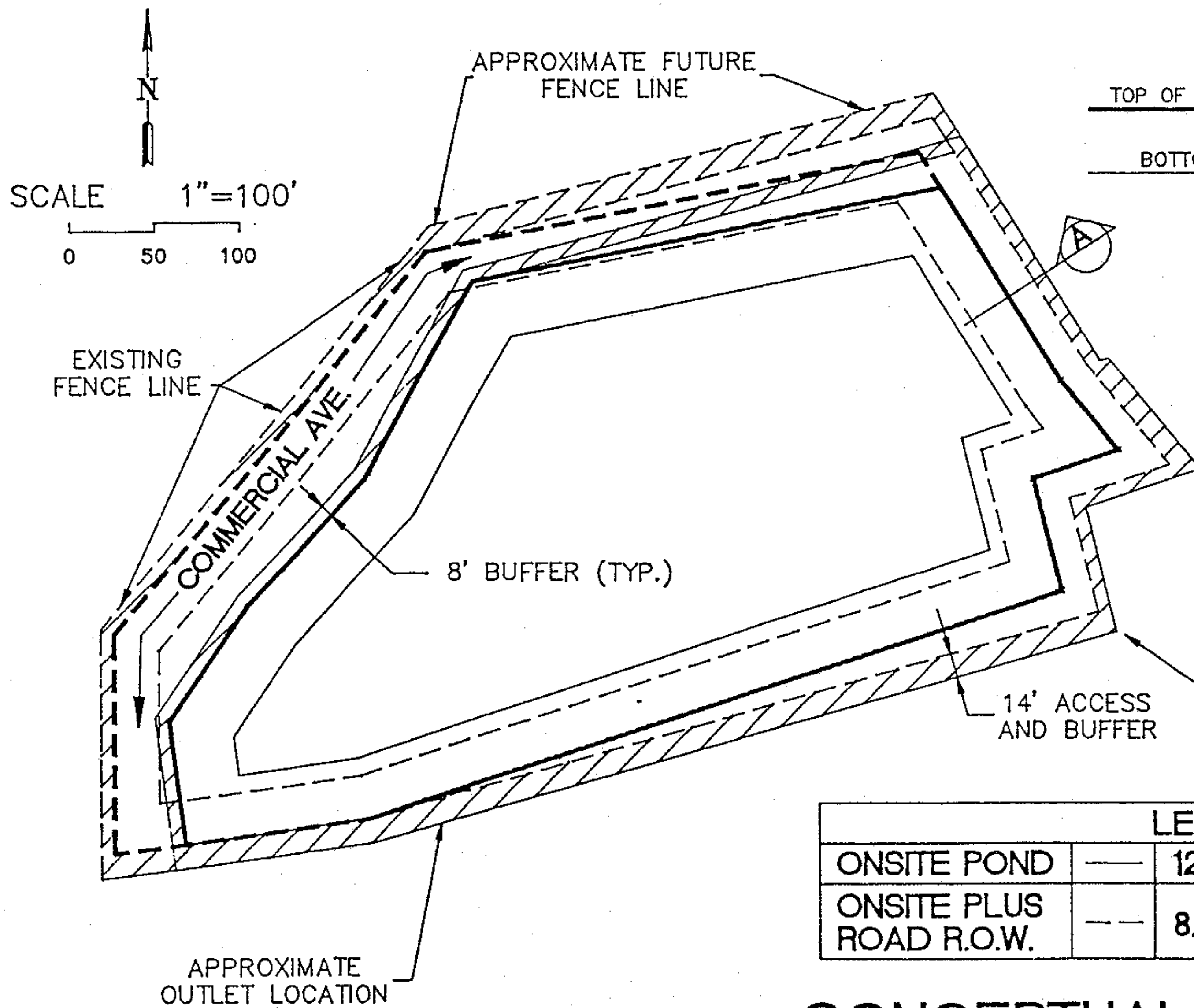
BOHANNAN-HUSTON, INC.

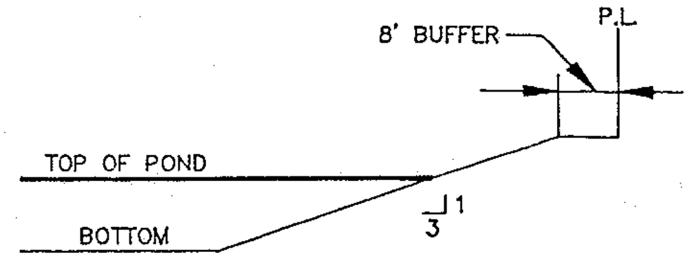
FIGURE 1



UPPER BASIN SUBAREA MAP

FIGURE 2





SECTION A
N.T.S.

LEGEND			
ONSITE POND	——	12' DEPTH	CASE I AND II
ONSITE PLUS ROAD R.O.W.	– –	8.7' DEPTH	CASE III

CONCEPTUAL POND DESIGNS

SCALE: 1" = 100'

FIGURE 3

S BROADWAY POND

DEPTH / AREA / VOLUME

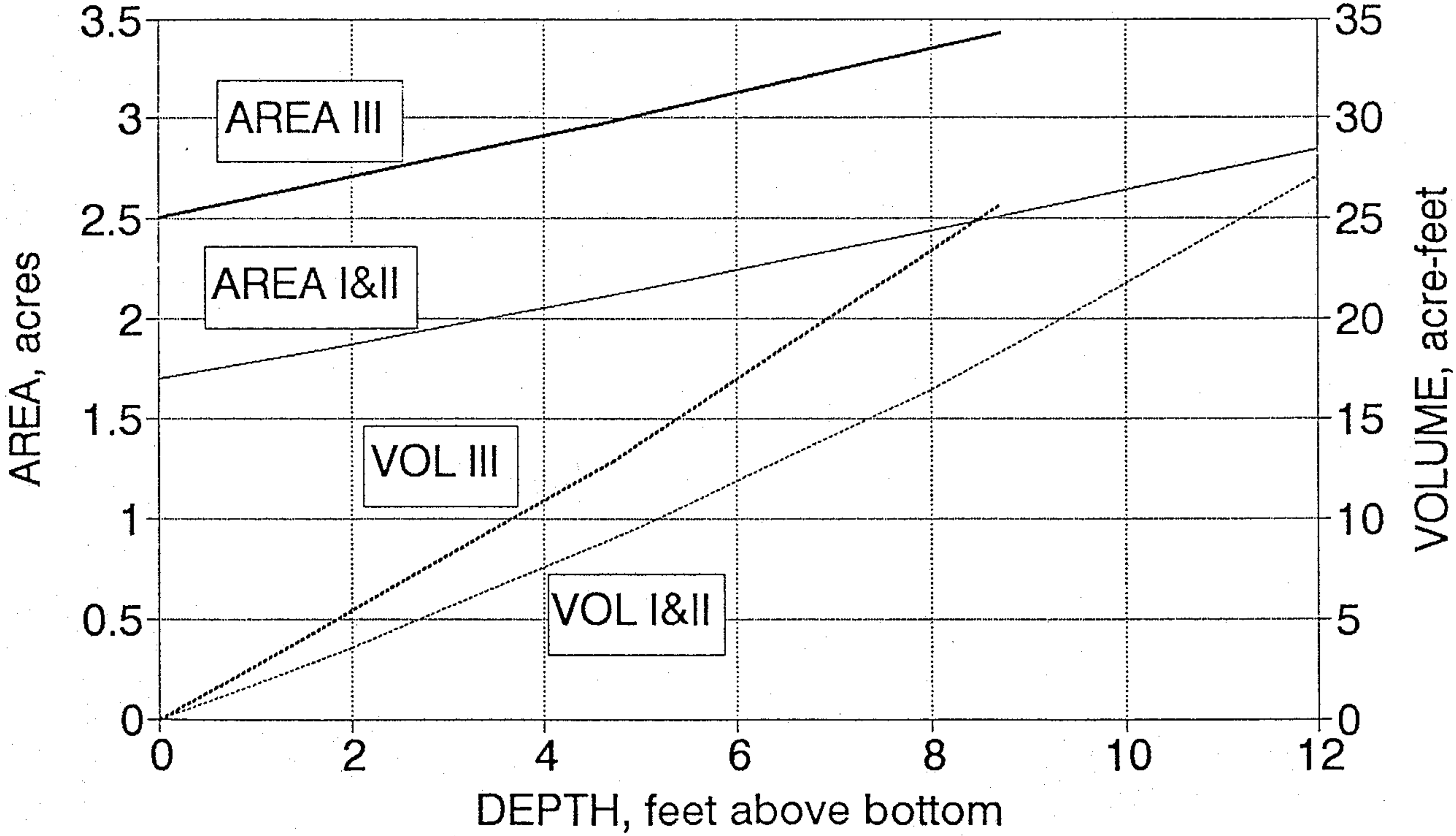
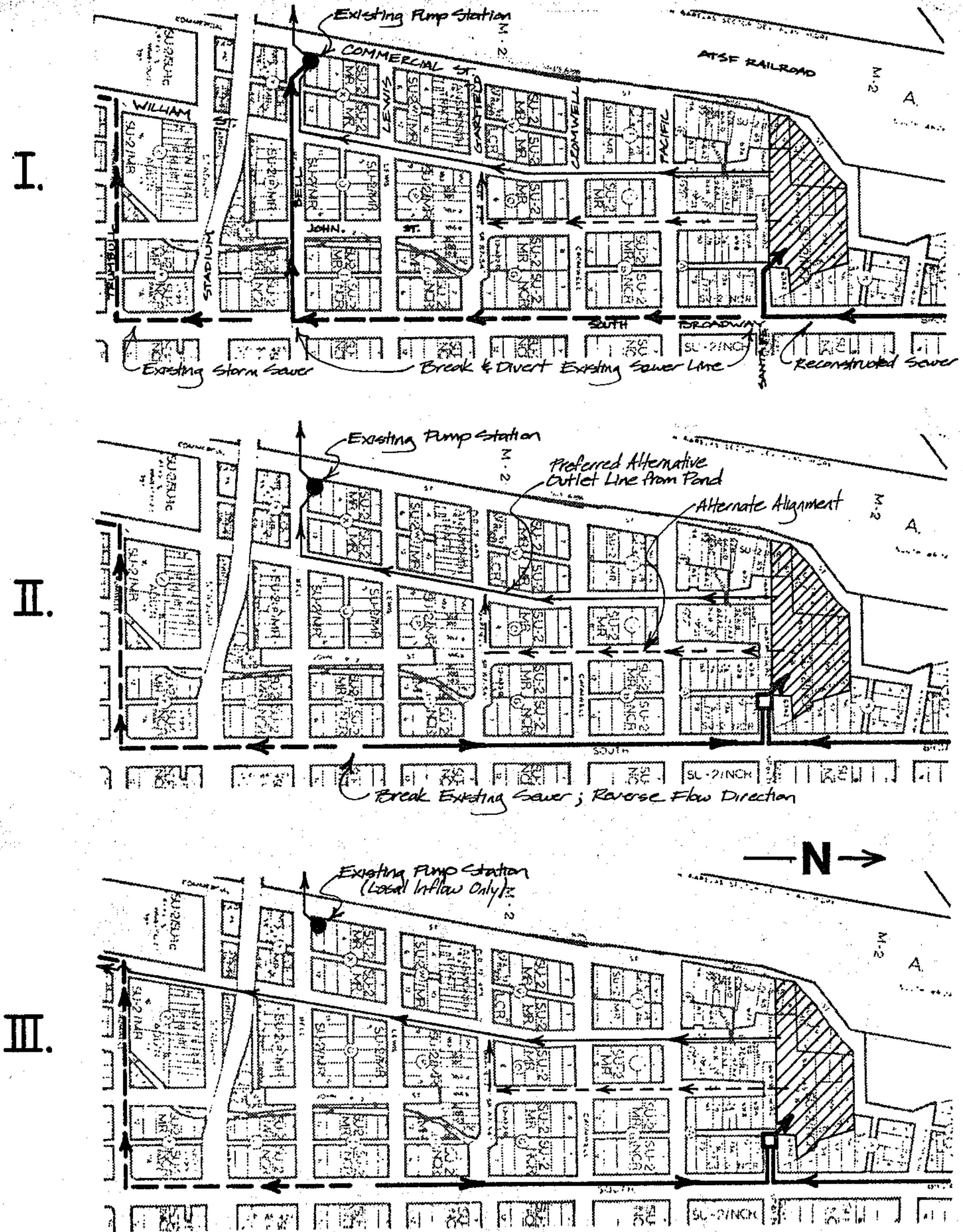
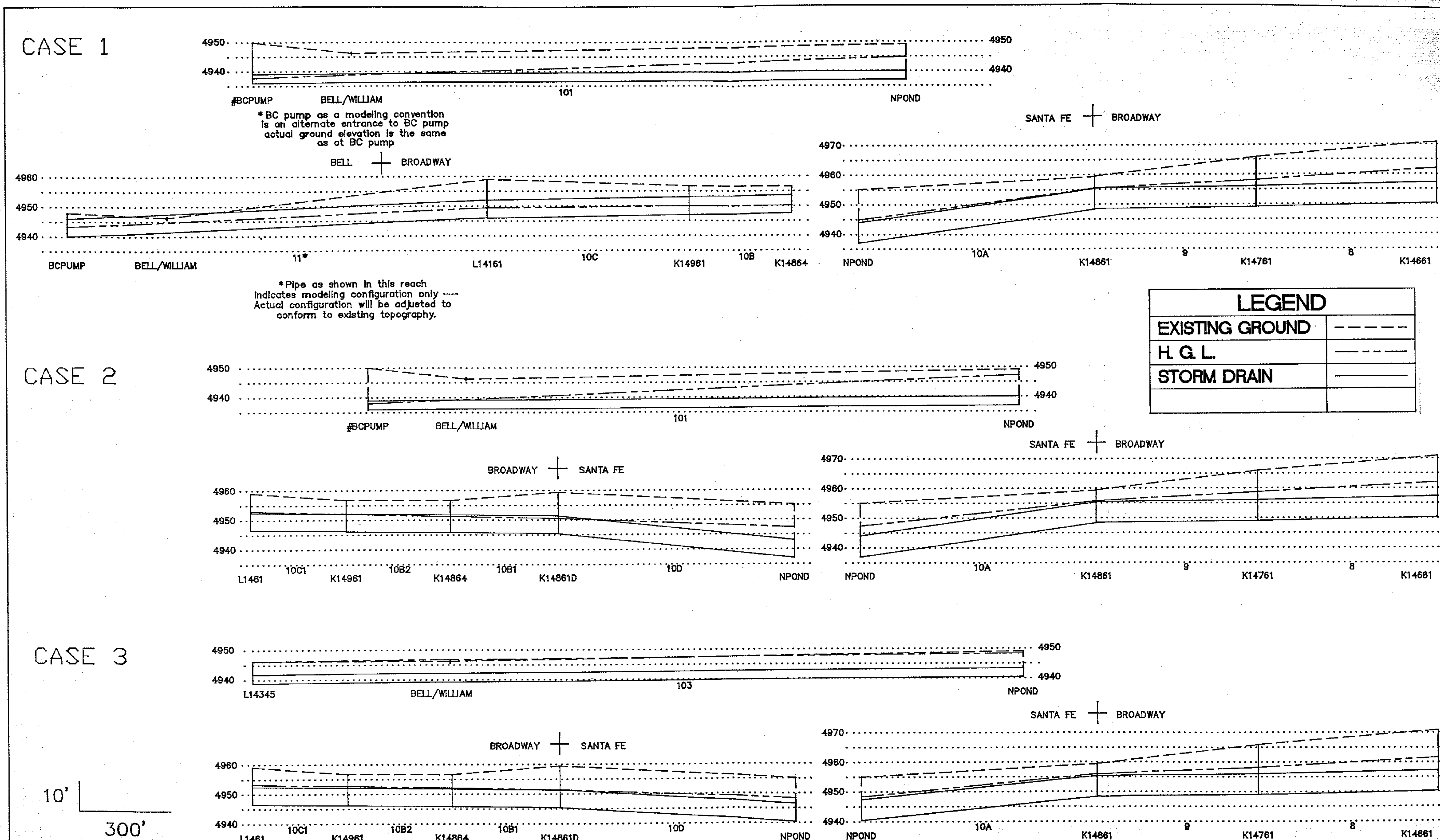


FIGURE 3A



FLOW SCHEMATIC FOR CASES I,II &III

FIGURE 4



SCALE: 1"=300' H
1"=30'V

STORM DRAIN PROFILES FOR POND INFLOW AND OUTFLOW PIPES

FIGURE 5

MANHOLE FLOODING

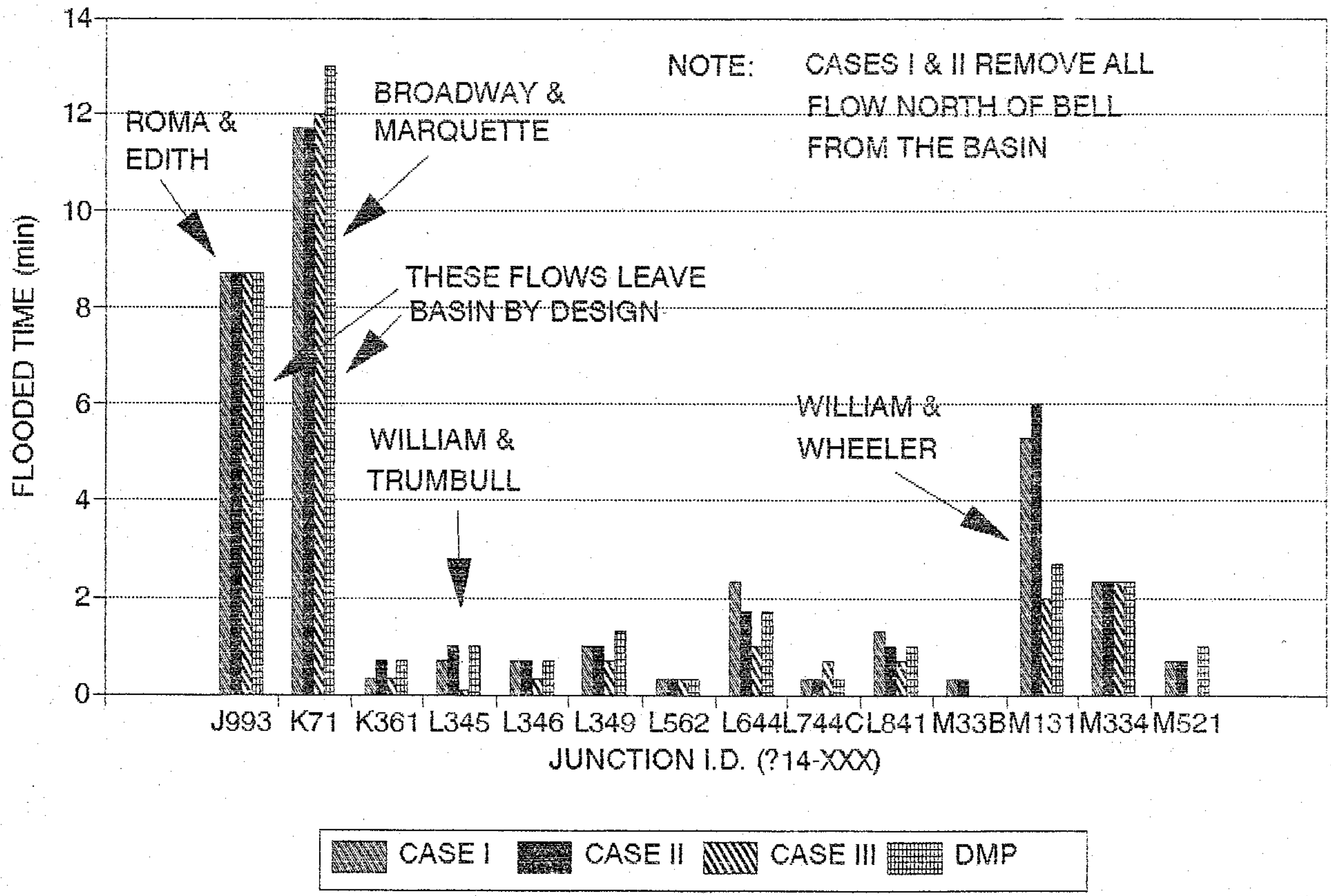


FIGURE 6

IMPACT OF POND OUTFLOW
ON WILLIAM St. STORM DRAIN - FLOW RATES

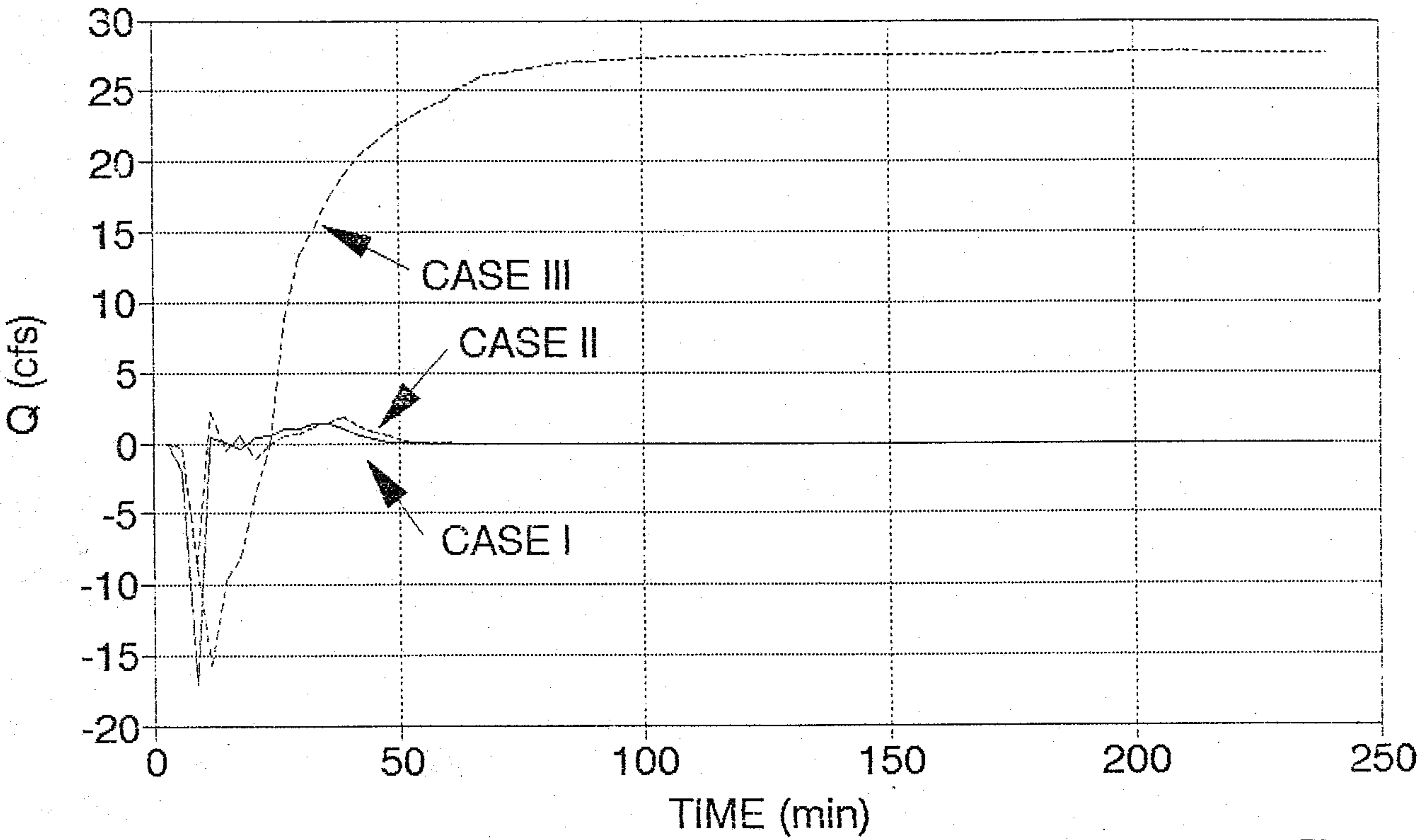


FIGURE 7

IMPACT OF POND OUTFLOW
ON WILLIAM St. STORM DRAIN - FLOW ELEV.

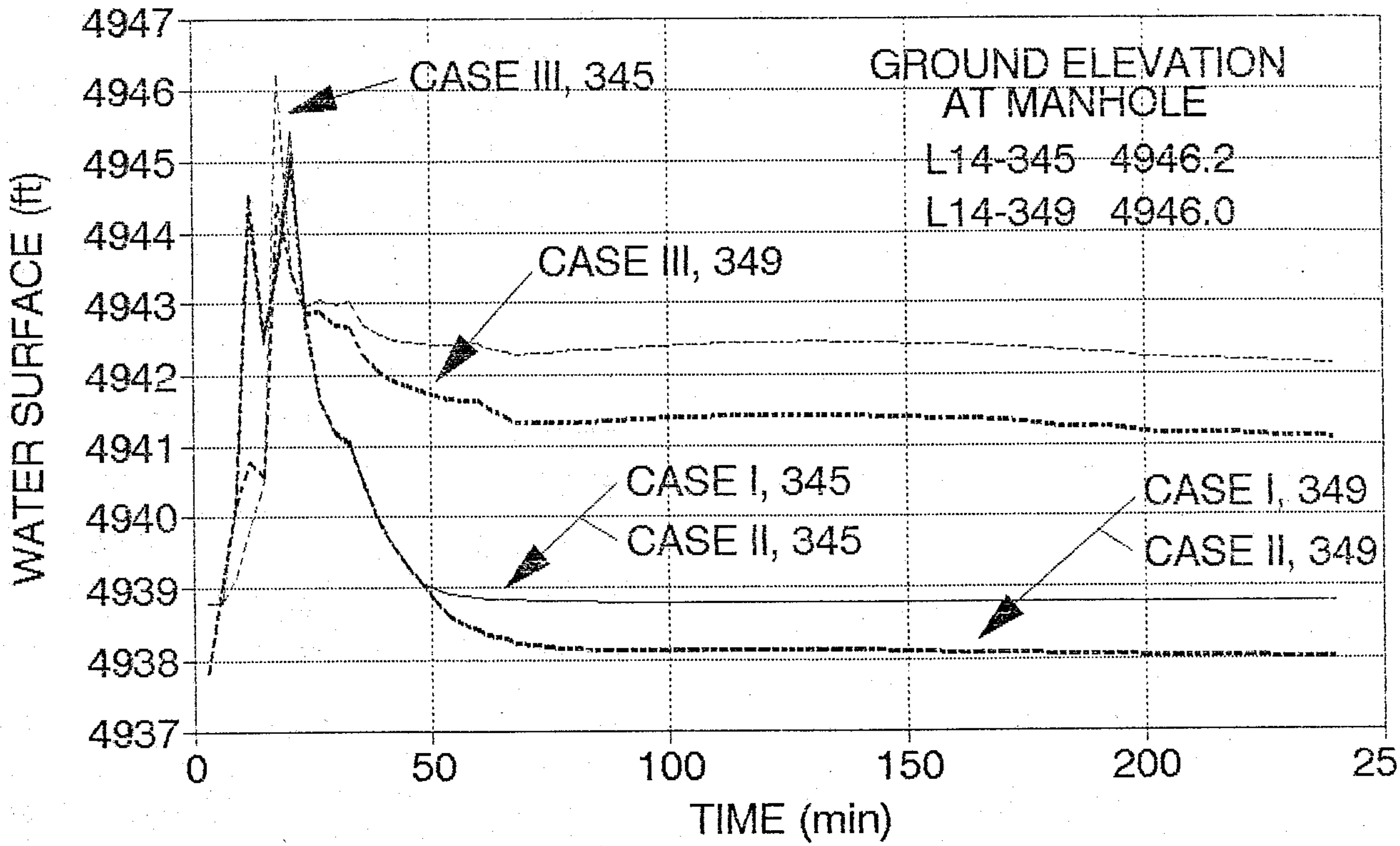


FIGURE 8

NORTH POND

INFLOW / OUTFLOW HYDROGRAPHS

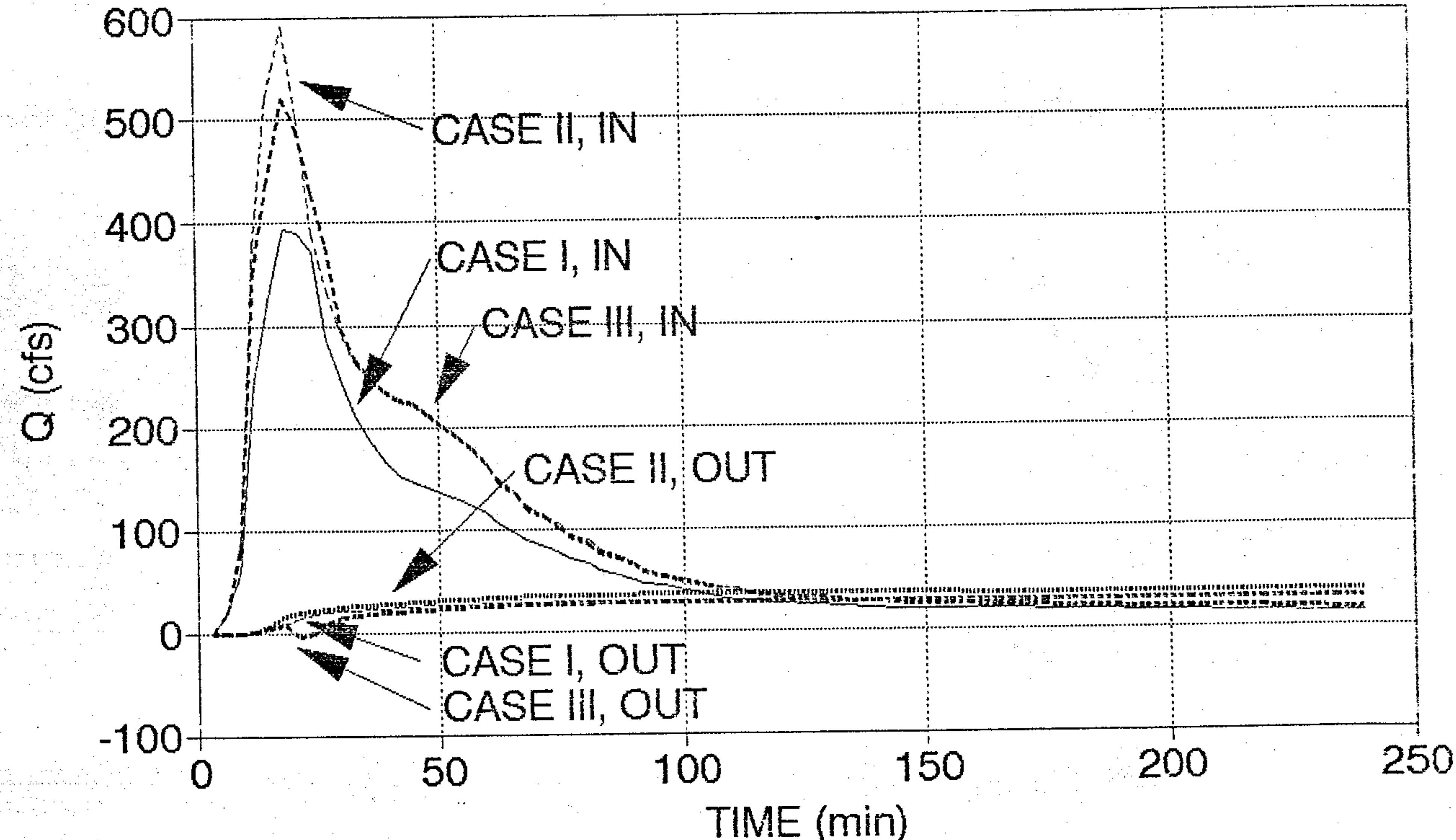


FIGURE 9

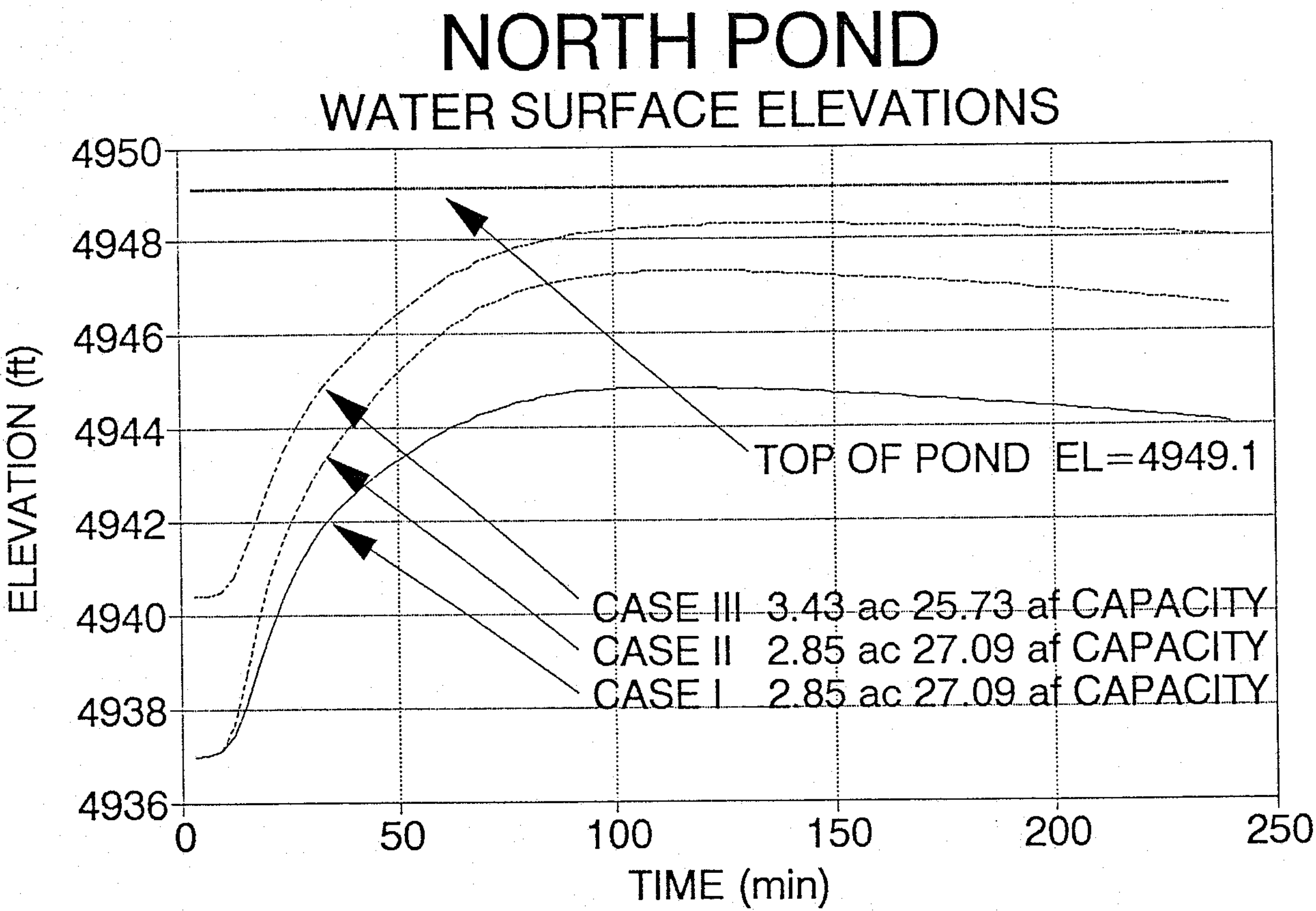

NORTH POND
WATER SURFACE ELEVATIONS
ELEVATION (ft)
4950
4948
4946
4944
4942
4940
4938
4936
0
50
100
150
200
250
TIME (min)
TOP OF POND EL=4949.1
CASE III 3.43 ac 25.73 af CAPACITY
CASE II 2.85 ac 27.09 af CAPACITY
CASE I 2.85 ac 27.09 af CAPACITY

FIGURE 1O

B/C PUMP STATION

TOTAL FLOW RATES

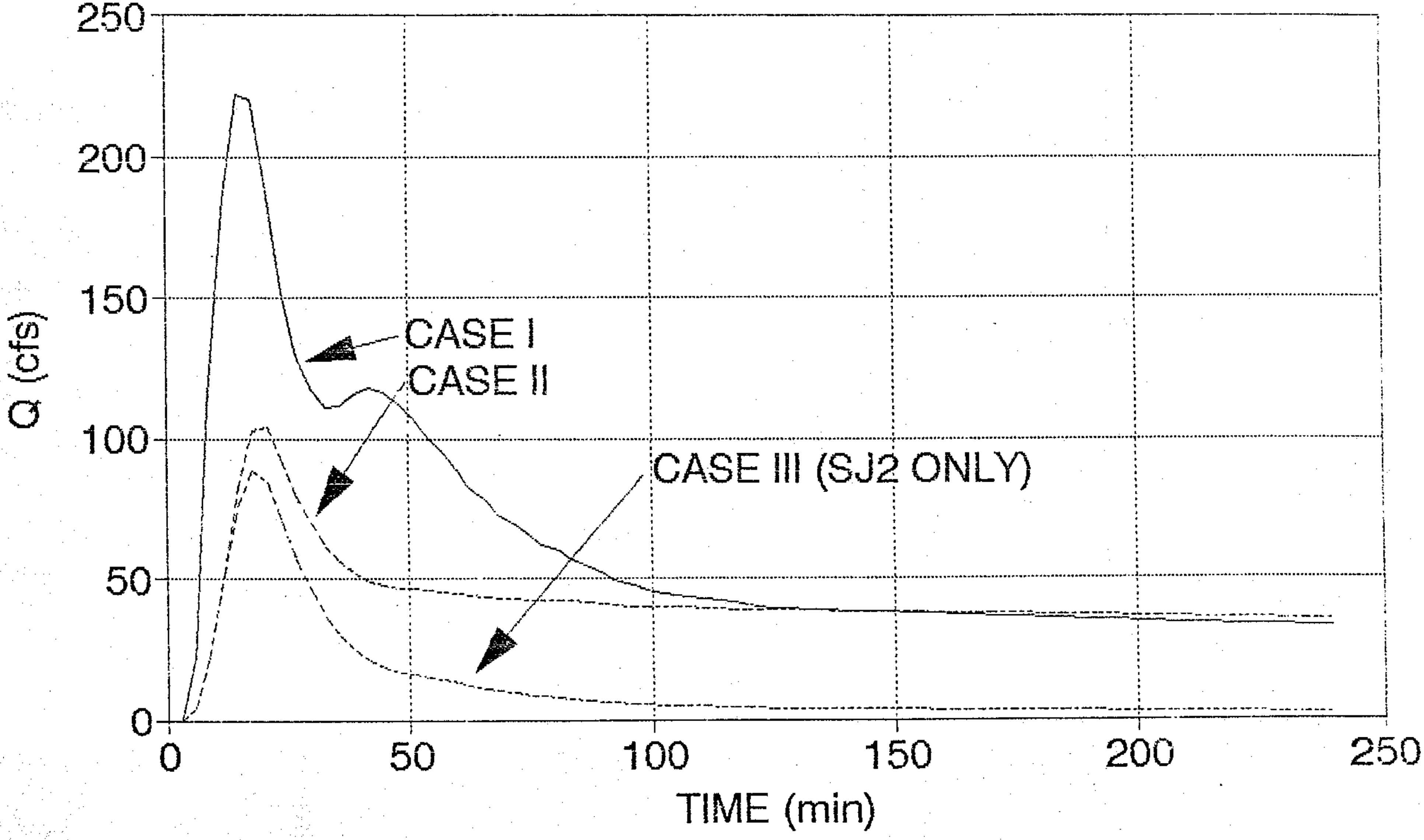


FIGURE 11

ENGINEERS ARCHITECTS PHOTOGRAMMETRISTS

BOHANNAN-HUSTON INC.

COURTYARD I, 7500 JEFFERSON STREET, N.E. ALBUQUERQUE, NM 87109
TEL (505) 823-1000 FAX (505) 821-0892

/

January 24, 1991

1.2 MByte IBM Format

Mr. Loren Meinz
City of Albuquerque
P.O. Box 1293
Albuquerque, NM 87103

Re: South Broadway Drainage Study Computer Files

Dear Loren:

I am enclosing the following computer files relating to the South Broadway Drainage Study:

NSPOND.IN: SWMM input file of improved conditions modeling both a North and South detention reservoir. Fully developed land usage, 100-year, 6-hour storm.

NSPOND.OUT Output file from NSPOND.IN.

BROADWAY.IN SWMM input file of existing drainage system, fully developed land usage, 100-year, 6-hour storm.

BROADTEN.IN SWMM input file of existing drainage system, fully developed land usage, 10-year, 6-hour storm.

TRY.IT SWMM input file of improved conditions modeling a north detention reservoir but no south reservoir. This scenario produces the San Jose Drain design flow. 100-year, 6-hour storm, fully developed land usage.

TRYIT.OUT Output file from TRY.IT

....

100YRHYD.HYM HYMO input file of basin hydrology, existing land usage, 100-yr, 6-hour storm.

100YRHYD.OUT Output file from 100YRHYD.HYM.

10YRHYD.HYM HYMO input file of basin hydrology, existing land usage, 10-yr, 6-hour storm.

100YRHYDIMP.HYM HYMO input file of basin hydrology, fully developed land usage, 100-yr, 6-hour storm.

10YRHYDIMP.HYM HYMO input file of basin hydrology, fully developed land usage, 10-yr, 6-hour storm.

It is my understanding that, with this submittal, all contractual obligations are fulfilled. I've enjoyed working with you on this project and look forward to future collaborations.

Cordially,

Butch Gerbrandt

Butch Gerbrandt, P.E.
Project Manager

cc: Kapil Goyal

BG/al

Job No. 8920803

INFORMATION SOURCES

The first two items listed below were the major sources for this study:

South Broadway Sector Drainage Management Plan - Developed Conditions Report Prepared for the City of Albuquerque by Bohannan-Huston Inc., Sept. 1990.

Computerized input, output, and EXTRAN program files for the South Broadway Sector Drainage Management Plan. Submitted to the City of Albuquerque by Bohannan-Huston, Inc., January 1991.

STORM WATER MANAGEMENT MODEL USER'S MANUAL VERSION 4: EXTRAN ADDENDUM Environmental Protection Agency 1988, No. EPA/600/3-88/001b

Discussions with City of Albuquerque staff

Discussions with other experienced EXTRAN users

City of Albuquerque Zoning Atlas

Recorded plats

City of Albuquerque Phototopographic maps (K-14 & L-14)

City of Albuquerque records of utility projects

Field investigations, including tests for depth to groundwater and mean sealevel elevation for inlet low points (at railroad yard fence)