Charles C. S. Song Company, Inc.

Report 95–03

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Optimal Design of the Carriage Lane Outfall Pipe Diameter

Prepared By:

Jianming He, Ph.D.
Charles C. S. Song, Ph.D., P.E.

Charles C. S. Song Company, Inc.
7200 Galpin Lake Road
Excelsior, MN 55331

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REMARKS

Prepared for

HNTB Corporation
Two Renaissance Square
40 North Central Ave., Suite 1100
Phoenix, Arizona 85004

June 1995

HNTB ARCHITECTS ENGINEERS PLANNERS

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July 27, 1995

Mr. Dick Perreault, P.E.
Flood Control District of Maricopa County
2801 West Durango Street
Phoenix, Arizona 85009

RE: Outer Loop Highway
Loop 101/US 60 Traffic Interchange
Baseline Road/Carriage Lane Outfall Project

Subject: Restudy and Downsize the Carriage Lane
Basin Outfall Pipe

Dear Mr. Perreault:

Attached please find a copy of the Dr. Charles Song's report entitled "Optimal Design of The Carriage Lane Outfall Pipe Diameter" and associated HNTB's memorandums for your reference. It was decided, in the meeting held at ADOT today, that the Outfall Pipe will be downsized from the 108-inch pipe to a 84-inch pipe with the same invert elevations.

We'll send you a copy of today's meeting minutes when it is ready. If you have any questions or need more information, please feel free to contact me at 528-4391.

Sincerely,

HNTB Corporation

Bailang Sun

Bailang (Gary) Sun, P.E.
Senior Drainage Engineer

BGS:bms/ FCD01.LTR

cc: John Friel, HNTB

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June 28, 1995

Mr. Stephen Martin, P.E.
Project Manager
Valley Transportation Group
Arizona Department of Transportation
205 South 17th Avenue
Phoenix, Arizona 85007

RE: Contract No. 86-08
Loop 101/US 60 Traffic Interchange

Subject: Carriage Lane Outfall Analysis

Dear Mr. Martin:

HNTB has completed its review of the Charles C.S. Song Company analysis of the optimal design for the Carriage Lane Outfall Pipe Diameter. This analysis, based upon the pumped and continuous flows provided by HNTB and ADOT has resulted in the determination that a 7 foot diameter (84 inch) pipe will accommodate design surges, which is caused by both backflow and pipe pressurization.

The attached three memorandums (dated May 29, 1995, May 31, 1995, June 18, 1995) and Report 95-03 (dated June, 1995) outline the results of our review.

Following is a compilation of this analysis:

1) The minimum pipe size for the system, based upon the pumped flow and 80 cfs continuous allocation, is a 84 inch diameter. This size accommodates pipe flows and surges.

2) The presently designed Carriage Lane Basin Head Structure includes twin 96 inch storage pipes as part of its design. Based upon HDR's proposed 54 inch gravity pipeline for the future Basin "E" outlet in place of the twin 96 inch pipes, we will be requesting confirmation from ADOT/HDR on the replacement of the twin pipes with a single 54 inch pipe. Based upon confirmation of 2), we will proceed with the modification of the Head Structure detail sheet.

3) The deletion of the 50 cfs ADOT continuous allocation (decrease from 80 cfs to 30 cfs) was reviewed by Dr. Song. This decrease in flow did not allow a reduction in pipe diameter from the 84 inch. The pipe surges incurred by a pipe diameter reduction does not justify the reduction. The 84 inch diameter is recommended and will accommodate the 80 cfs continuous allocation.

Partners [illegible]
Associates [illegible]
Offices [illegible]

4) The cost savings of the reduction in pipe diameter from 108 inch to 84 inch would be estimated at $1,500,000, based upon previous unit bid prices.

Based on this review, we are prepared to meet with you and appropriate ADOT representatives to discuss final resolution of this design to move into plan preparation activities.

If you have further questions, please feel free to contact me.

Sincerely,

HOWARD NEEDLES TAMMEN & BERGENDOFF, INC.

John W. Friel, P.E.
Project Manager

cc: Mr. Ray Jordan, ADOT
Dr. C.S. Song, Charles C.S. Song Co., Inc. (w/o Report)
Mr. Gary Sun, HNTB (w/o Report)
File 11057

Attachments: (HNTB Memos dated May 30, 1995 and May 31, 1995)
(Dr. Song Memo dated June 18, 1995)
(HNTB Review dated June 29, 1995)
(Dr. Song Report No. 95-03 dated June 1995)

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ADOT Contract No. 86-08 June 29, 1995

Northwest Outer Loop Highway and US 60
CARRIAGE LANE BASIN OUTFALL PIPE

Summary of Hydraulic Transient Flow Modeling
<u>for the Optimal Design of Pipe Diameter</u>

This summary describes the results of hydraulic transient flow modeling for the optimal design of the Carriage Lane Basin Outfall Pipe diameter studied by the Charles C.S. Song Co. Inc. in June 1995. The purpose of Dr. Song's analysis was to optimize the Outfall Pipe diameter based upon the updated discharge at the Carriage Lane Basin Head Structure. The current design of a 108-inch diameter pipe was based upon a discharge of 230 cfs at the Head Structure. With the updated discharge of 30 or 80 cfs, a smaller pipe diameter will be determined through Dr. Song's analysis. A total of 11 cases were studied and the following is our conclusions and recommendations (Refer to attached Dr. Song's Report).

A. Overflow at the Head Structure occurs for Cases 4, 6, 7, 9 and 11. For a 7' pipe diameter, the amount of overflow into the Basin is about the same (approximately 0.14 acre-feet for 11 minutes) and is independent from the invert elevation (Cases 4 & 6, and Figures 4.5 & 6.5). However, the higher Outfall Pipe invert will lower the absolute maximum water surface elevations (Cases 3 & 5, and Figures 3.4 & 5.4).

B. For a 7' Outfall Pipe, the maximum water elevation will occur near Guadalupe Road at 1192.4 feet, approximately 2.4 higher than the top structure elevation for the dry tunnel condition (Case 3, Figure 3.4). The overflow of 0.14 acre-feet into the Basin will occur at a maximum water elevation of 1189.5 feet, and will last approximately 11 minutes from Minute 25 to Minute 36 with a maximum flow rate of 19.4 cfs (Case 4, Figure 4.5).

C. If the top elevation of 1189.00 feet at the Head Structure can be lowered, the maximum water surface elevations at the Head Structure and near Guadalupe Road will be reduced and the overflow in the Basin will increase.

D. The inflow reduction at the Head Structure from 80 to 30 cfs does somewhat improve the hydraulic transient condition with a 7' Outfall Pipe (Cases 3, 4, 8 & 9). However, very strong surge occurs in the system if the Outfall Pipe diameter is reduced to 6' (Case 10, Figure 10.4).

E. Table 1 shows the absolute maximum water surface elevations in the Price Road Drainage Tunnel system with a 7' Outfall Pipe. In Table 1, the maximum water surface elevations are higher than top elevations at the Head Structure and Guadalupe Road. It should be noted that this maximum water elevation is only occurring for few seconds and the splash out of the drainage structure is just an instant phenomenon. The maximum steady water surface elevations will be used for the Outfall Pipe drainage system design.

F. The recommended Carriage Lane Basin Outfall Pipe diameter is <u>7 feet or 84 inches</u> reinforced concrete pipe. The Head Structure must have a minimum cross-section area of 300 square feet for surge relief reasons.

Table 1
Absolute Maximum Water Surface Elevations
in The Price Road Drainage Tunnel System
With a 7' Carriage Lane Outfall Pipe

Dr. Song's Sta. No.	Structure Description	Invert (ft)	Max. Water Surface Elev. (ft)	Top Elevation (ft)
1	Drop Structure 'A'	1121.00	$1186.27_{(4)}$	1195.00
4	CLBOP Head Structure	1163.70	$1190.24_{(3)}$	1189.00
11	Guadalupe Road	1162.26	$1192.37_{(3)}$	1190.00
29	Baseline Road	1159.37	$1186.94_{(4)}$	1195.27
37	Drop Structure 'B'	1158.00	$1185.21_{(4)}$	1195.55
45	Dropshaft No. 5	1118.00	$1184.33_{(4)}$	1191.50
53	Dropshaft No. 4	1115.37	$1183.03_{(4)}$	1190.80
63	Dropshaft No. 3	1112.07	$1181.32_{(4)}$	1196.35
69	Dropshaft No. 2	1110.09	$1180.30_{(4)}$	1198.90
83	Dropshaft No. 1	1105.47	$1177.44_{(4)}$	1187.75
90	Tunnel Low Point	1103.00	$1176.30_{(4)}$	1182.00

Note: (3) & (4) represent Cases 3 and 4, respectively.

Charles C. S. Song Co. Inc.

Consulting Engineers:
Computational Fluid Dynamics, Modeling Industrial and Environmental Flows
7200 Galpin Lake Road
Excelsior, MN 55331
(612) 627-4599, (612) 474-7984 , Fax (612)- 474-1580

June 18, 1995

Mr. John W. Friel, P.E.
HNTB Cooperation
Two Renaissance Square
40 North Central Ave., Suite 1100
Phoenix, Arizona 85004

RECEIVED
JUN 20 1995
HNTB

Dear Mr. Friel:

Enclosed please find a revised final report for the Carriage Lane Outfall Pipe Analysis project entitled " Optimal Design of the Carriage Lane Outfall Pipe Diameter ".

The original report contained only the cases of 80 cfs continuous allocation at the upstream end. This is because the contract indicated to use 30 cfs or 80 cfs and we believed 80 cfs is the more conservative number to use. At your subsequent request we made additional runs using 30 cfs inflow and included the results in this revised report.

In brief, although reducing inflow from 80 cfs to 30 cfs reduces surge intensity and overflow somewhat, it is not sufficient to reduce the pipe diameter from 7 ft to 6 ft. When a 6 ft diameter pipe is used under the dry condition, two types of surges, one caused by the back flow from the main tunnel and the other caused by pressurization within the 6 ft pipe will occur. This situation should be avoided.

Please also regard this as the invoice for the agreed costs of $10,750.00.

Please contact me or Dr. J. He if have any questions.

Sincerely yours,

Charles C.S. Song

Charles C.S. Song, President
Charles C.S. Song Co. Inc.

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JWF	X		X
BGS	X		X
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DATE: 6-26-95

HNTB HOWARD NEEDLES TAMMEN & BERGENDOFF
ARCHITECTS ENGINEERS PLANNERS

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May 31, 1995

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11057			
DJL	✓		
JWF	✓		
FILE	✓		

Mr. Stephen Martin
Project Manager
Arizona Department of Transportation
205 South 17th Avenue
Phoenix, Arizona 85007

RE: Contract No. 86-08
Northwest Outerloop Highway and US 60

Subject: Carriage Lane Outfall Analysis

Dear Mr. Martin:

HNTB has completed its analysis of the Carriage Lane Outfall System, based upon the HDR memorandum of April 21,1995 and direction provided by ADOT. The attached two memorandums (dated May 29, 1995 and May 30, 1995) outline the results of our review.

Following is a compilation of this analysis:

1) The minimum pipe size for the system, based upon the 100 cfs pumped flow, is a 60 inch diameter. These results have been forwarded to Dr. Song for use in his review of system characteristics.

2) The presently designed Carriage Lane Basin Head Structure includes twin 96 inch storage pipes as part of its design. Based upon HDR's proposed 54 inch gravity pipeline for the future Basin "E" outlet in place of the twin 96 inch pipes, we are requesting confirmation from ADOT/HDR on the replacement of the twin pipes with a single 54 inch pipe. Based upon confirmation of 2), we will proceed with the modification of the Head Structure detail sheet.

An early resolution of the remaining system issues will dictate our plan completion date for the Carriage Lane/Baseline Road 90% submittal. The results from Dr. Song's analysis will be available in mid-June 1995, and will be furnished as they are provided to us.

Partners [illegible]
Associates [illegible]
Offices [illegible]

If you have further questions, please feel free to contact me.

Sincerely,

HOWARD NEEDLES TAMMEN & BERGENDOFF, INC.

John W. Friel, P.E.
Project Manager

cc: Mr. Ray Jordan, ADOT
Dr. C.S. Song, Charles C.S. Song Co., Inc.
Mr. Gary Sun, HNTB
File 11057

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TABLE 1
MINMUM CARRIAGE LANE BASIN OUTFALL PIPE SIZE
WITH 100 cfs PUMPED FLOW DURING OFF-PEAK CONDITIONS

LOCATION	EG LINE (ft)	Q (cfs)	A (ft^2)	WP (ft)	V (fps)	H_v (ft)	L (ft)	H_f (ft)	k	H_{vv} (ft)
SALT RIVER at 10-year water level	1172.120	100								
OPEN CHANNEL with B=10' & z=1	1172.121	100	389.530	53.440	0.2567	0.0010				
40' TRANSITION from 2-13'X14' RCBC to Open Channel	1172.121	100					40		0.21	0.00003
1,460' of 2-13'X14' RCBC TUNNEL RISER	1172.123	100	364.000	108.000	0.2747	0.0012	1460	0.0017		
56.3' TRANSITION from 18' ID Tunnel to 2-13'X14' RCBC	1172.123	100					56.3		0.20	0.0002
15,409' of 18' ID TUNNEL	1172.147	100	254.469	56.549	0.3930	0.0024	15409	0.0244		
13' of 84" Steel Pipe	1172.252	100	38.485	21.991	2.5984	0.1048	13	0.0023	1.00	0.1024
12' TRANSITION from 11'X10.17' RCBC to 84" Steel Pipe	1172.278	100					12		0.28	0.0259
50.5' of 11'X10.17 RCBC CHAMBER - Drop Structure 'B'	1172.279	100	111.870	42.340	0.8939	0.0124	50.5	0.0008		
33' DROP in 10' ID SHAFT - Drop Structure 'B'	1172.293	100	78.540	31.416	1.2732	0.0252	33	0.0012	0.50	0.0126
58' of 8'X10.125' RCBC	1172.307	100	81.000	36.250	1.2346	0.0237	58	0.0023	0.50	0.0118
10' TRANSITION from 108" RCP to 8'X10.125' RCBC	1172.310	100					10		0.20	0.0029
140' of EXISTING 108" RCP (Using H-W Equation)	1172.319	100	63.617	28.274	1.5720	0.0384	140	0.0092		
TRANSITION from 108" RCP to 60" RCP	1172.392	100							0.20	0.0729
10,545' of NEW 60" RCP (Using H-W Equation)	1184.495	100	19.635	15.708	5.0930	0.4028	10545	12.1032		

Notes:
- EG LINE - Energy Line Elevation
- Q - Discharge
- A - Section Area
- WP - Wetted Perimeter
- V - Velocity
- H_v - Velocity Head
- L - Length
- H_f - Friction Losses
- k - Coefficient for Misc. Losses
- H_{vv} - Exit, Entrance, Expansion, Contraction or Bend Losses

HNTB

Phoenix Office

Interoffice Correspondence

To: John Friel

Date: May 30, 1995

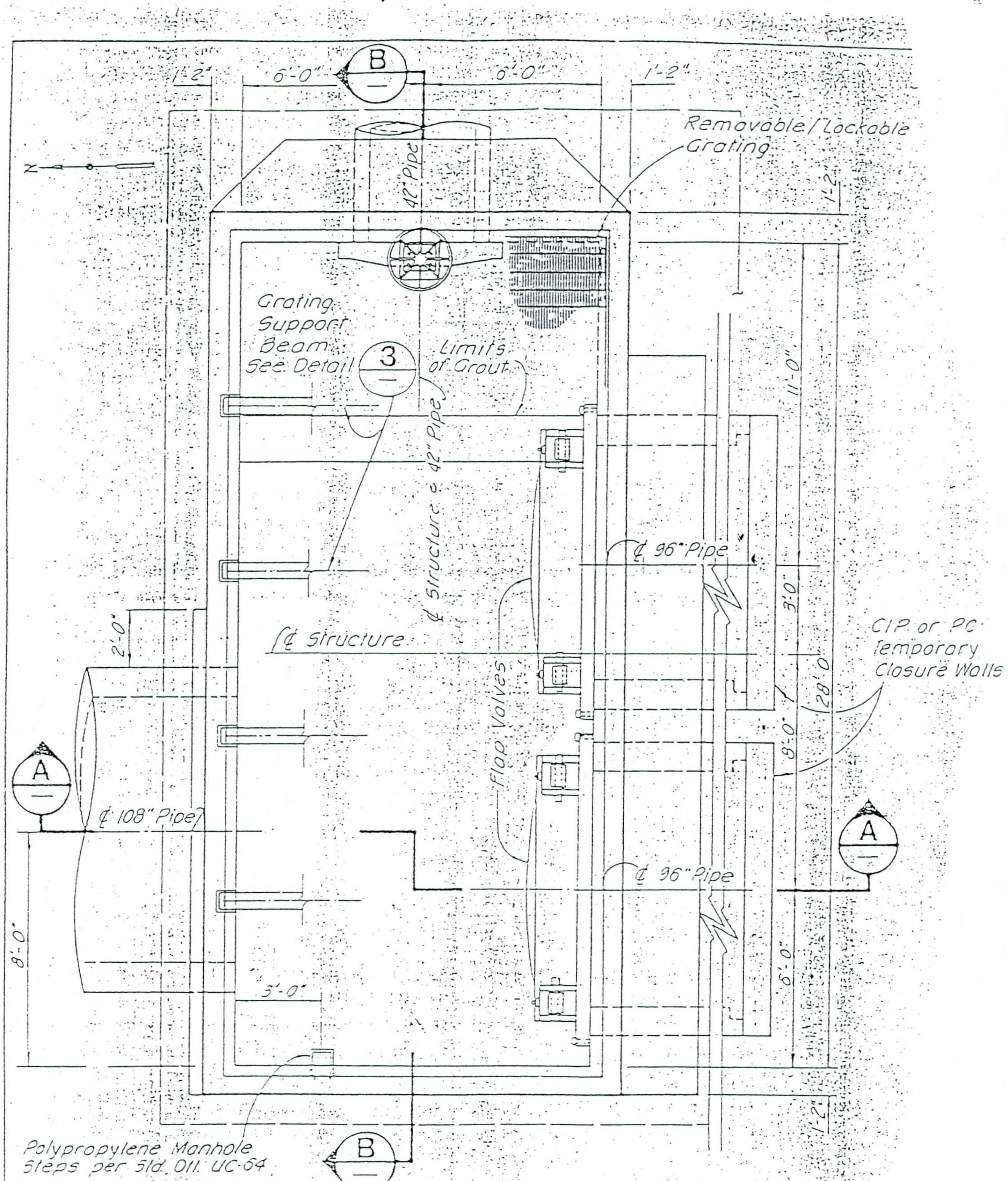
From: Gary Sun GS

Subject: Carriage Lane Basin Outfall Pipe Head
Structure Design Modifications
Loop 101/US 60 Traffic Interchange Phase II Project
HNTB Project No. 11057-DS-027-004

I have reviewed Dr. Jerry Zovne of HDR memorandum to Steve Martin of ADOT dated April 21, 1995 and our current Head Structure design at the Carriage Lane Basin (See Attached Details). Based on the latest design concept, the future Basin 'E' outlet to the Carriage Lane Basin Head Structure is a 54-inch gravity pipeline in lieu of two 96-inch pipes shown on the attached details. These two 96-inch storage pipes would not be required since the Basin 'E' will be convert to a retention basin to provide additional runoff storage. This basin will use a 54-inch pipe for stormwater disposal to the Carriage Lane Basin Outfall Pipe during the tunnel off-peaks. Therefore, I'm requesting a confirmation from ADOT or HDR for replacing two 96-inch pipes with a 54-inch pipe at the Head Structure.

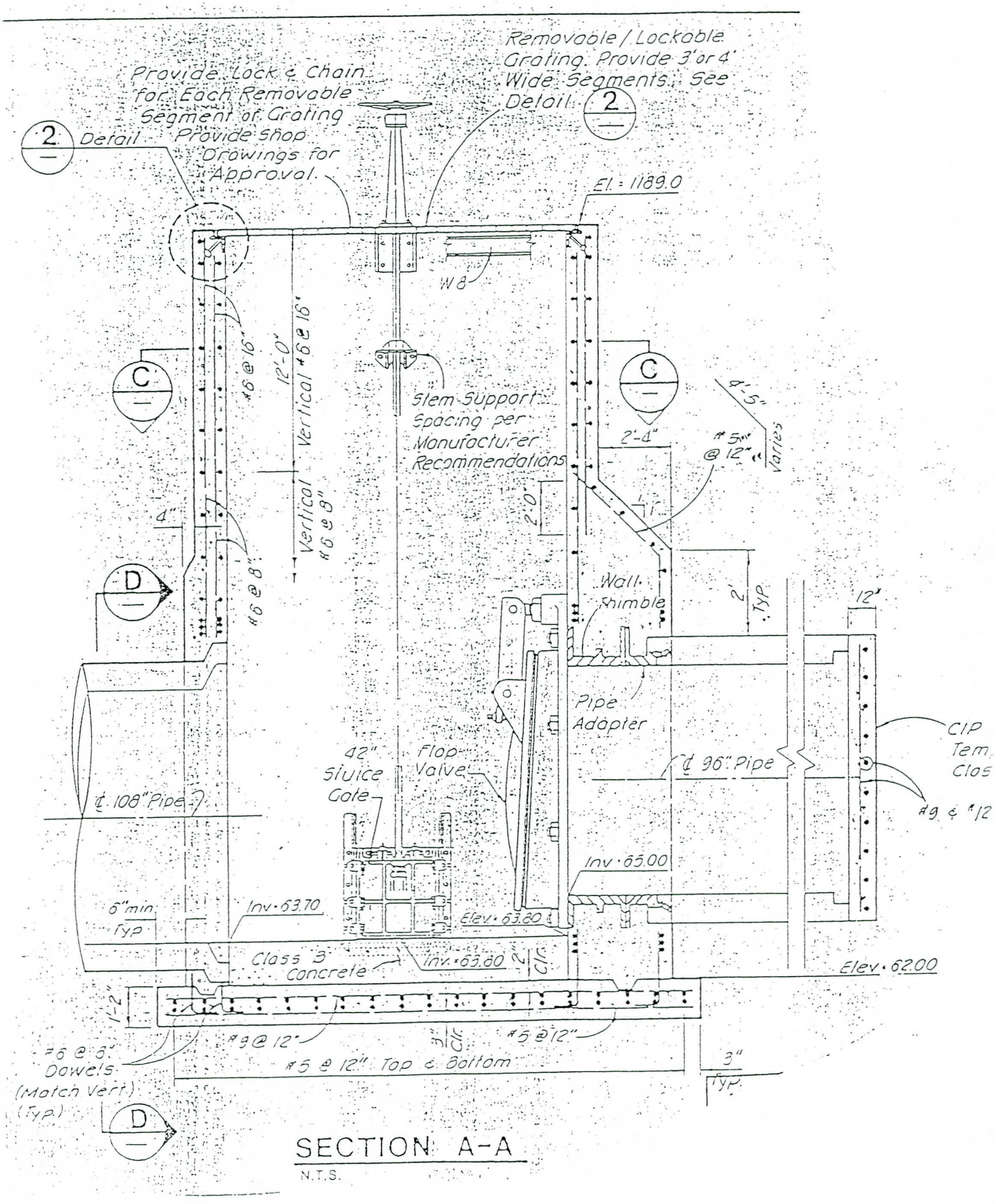
I need this confirmation as soon as possible so that the Head Structure detail sheet can be modified for the upcoming submittal. It should be noted that these two 96-inch pipes will be replaced with a smaller pipe whether or not we downsize the Outfall Pipe. Should you have any questions, please let me know.

BGS / CARR02.COR



PLAN VIEW
HEAD STRUCTURE

N.T.S.



SECTION A-A

N.T.S.

Optimal Design of the Carriage Lane Outfall Pipe Diameter

1. Objective

The primary purpose of this study is to optimize the diameter of the Carriage Lane Outfall Pipe based on the updated discharge using the dynamic transient computer model. The presently–designed pipe diameter (108") is based on the discharge of 230 cfs. The updated discharge is only 30 or 80 cfs. Apparently, the presently–designed pipe diameter is too large for the updated discharge. Therefore, re–evaluation of the pipe diameter is necessary based on the updated information. The basic requirement for the optimal diameter is that the hydraulic performance of the Pipe with a smaller diameter for the 30 or 80 cfs discharge should be equivalent to that of the original design for the 230 cfs discharge.

2. Study Process

This study was started on May 19, 1995 when HNTB Cooperation officially notified Charles C. S. Song Company, Inc. (as a sub–contractor) to proceed with the study based on the accelerated schedule. According to the schedule, on June 5, 1995, the contractor faxed the preliminary results and conclusions to HNTB, and recommended to HNTB that the pipe diameter should not be smaller than 7 ft. This is the final report of the study.

3. Modeling Configuration

The proposed study was based on an assumption that the current design conditions of the tunnel system are the same as the previous modeling configuration except the flow rate and the pipe diameter. However, it was found after the work had been started that the current design conditions are somewhat different from the conditions used in our previous hydraulic transient study. Therefore, this modeling work is based on the updated design parameters, shown as follows:

Main tunnel slope:	0.1097%
Outfall pipe slope:	0.05315%
The maximum inflow rate:	2170 cfs
Tunnel low point elevation:	1103.00 ft (at Station 90)
Tunnel riser high point:	1156.96 ft
Head structure invert:	1163.70 ft (at Station 4)
Head structure weir level:	1189.00 ft
Head structure area:	300 ft^2
Head structure weir length:	20 ft

10-year River flood level: 1172.12 ft

The same station number is used as before.

4. Optimization Method

By changing the pipe diameter, a number of simulation runs were conducted to check its hydraulic performance using the following condition:

CONDITION A: 100-year tunnel flow hydrographs with no flow in Salt River, with outfall at Price Road ('dry' tunnel condition).

After the optimal diameter has been obtained based on the condition described as above, the following condition is used to check the hydraulic performance with the selected pipe diameter.

CONDITION B: 100-year tunnel inflow hydrographs with 10-year Salt River flood level at 1,172.12 feet at Price Road outfall ('wet' tunnel condition).

If the hydraulic performance under Condition B becomes worse, the optimal procedure described before is changed to with Condition B. Then check the results under Condition A. The final optimal pipe diameter should satisfy both Condition A and Condition B.

5. Modeling Results:

Based on the above optimization method, a number of computer simulation runs were conducted. The following 11 typical cases are selected and discussed here to show the hydraulic transient performance of the tunnel system.

Case 1:
pipe diameter = 9 ft
initially 'dry' tunnel
head structure invert at 1163.70 ft
Inflow at the head structure = 80 cfs

Case 2:
pipe diameter = 9 ft
tunnel initially filled at 1172.12 ft ('wet' tunnel)
head structure invert at 1163.70 ft
Inflow at the head structure = 80 cfs

Case 3:
pipe diameter = 7 ft
initially 'dry' tunnel
head structure invert at 1163.70 ft
Inflow at the head structure = 80 cfs

Case 4:
pipe diameter = 7 ft
tunnel initially filled at 1172.12 ft ('wet' tunnel)
head structure invert at 1163.70 ft

Inflow at the head structure = 80 cfs

Case 5: pipe diameter = 7 ft
initially 'dry' tunnel
head structure invert at 1166.70 ft (the pipe elevated 3 ft)
Inflow at the head structure = 80 cfs

Case 6: pipe diameter = 7 ft
tunnel initially filled at 1172.12 ft ('wet' tunnel)
head structure invert at 1166.70 ft (the pipe elevated 3 ft)
Inflow at the head structure = 80 cfs

Case 7: pipe diameter = 6 ft
tunnel initially filled at 1172.12 ft ('wet' tunnel)
head structure invert at 1163.70 ft
Inflow at the head structure = 80 cfs

Case 8: pipe diameter = 7 ft
initially 'dry' tunnel
head structure invert at 1163.70 ft
Inflow at the head structure = 30 cfs

Case 9: pipe diameter = 7 ft
tunnel initially filled at 1172.12 ft ('wet' tunnel)
head structure invert at 1163.70 ft
Inflow at the head structure = 30 cfs

Case 10: pipe diameter = 6 ft
initially 'dry' tunnel
head structure invert at 1163.70 ft
Inflow at the head structure = 30 cfs

Case 11: pipe diameter = 6 ft
tunnel initially filled at 1172.12 ft ('wet' tunnel)
head structure invert at 1163.70 ft
Inflow at the head structure = 30 cfs

All the above cases were run with a continuous allocation (80 cfs in Case 1 to Case 7, and 30 cfs in Case 8 to Case 11) at the head structure and 100-year storm hydrographs at the other dropshafts. The current pipe diameter (9 ft) is tested in Case 1 and Case 2. A recommended pipe diameter (7 ft) is studied in Cases 3 to 6, and Cases 8 & 9. Case 7, Case 10, and Case 11 show the results with a smaller diameter (6 ft). A schematic of the model system configuration is shown in Fig. 1.

Case 1:

Figs 1.1 and 1.2 shows time histories of the water surface elevation at 8 stations (Stations 1, 4, 11, 37, 40, 53, 69, and 90) for Case 1. A strong backward surge can be identified at Station 4 (the head structure) and Station

11 at time = 90 minutes. The peak water level at the head structure is very close to the top elevation of the head structure at 1189.00 ft.

Fig. 1.3 displays five instantaneous grade line along the tunnel system from the head structure to the tunnel low point. The dashed lines represent the location of the dropshafts and their ground elevation. As the figures indicate, the pipe is fully pressurized when the backward sure reaches the head structure. However, after time=170 minutes, the pipe starts to be partially open channel as the inflow to the tunnel system is decreased. The flow in the pipe becomes fully open channel flow at time=245 minutes.

The maximum water surface elevation at each station during the entire flood period is shown in Fig. 1.4. The maximum water surface elevation is much below the top elevation at the dropshafts.

The flow balance is shown in Fig. 1.5. The total inflow includes all the hydrographs into the tunnel system. The outflow to the Salt River starts at t=72.5 minutes. There is no overflow at the head structure in this case.

Case 2:

Figs. 2.1 to 2.5 show the similar figures for Case 2. Since the tunnel system is initially filled at the 10-year flood level (1172.12 ft) of the Salt River, the entire tunnel is almost full except a few stations close to the head structure prior to the 100-year storm. Under this condition, the hydraulic transient performance at the upstream region is greatly improved, as shown in Fig. 2.1, and the maximum water surface elevation is much lower than that in the above case, as indicated in Fig. 2.4.

As shown in the above results, it is possible to reduce the diameter and still give satisfactory hydraulic performance. A number of diameters have been tested. It was found that 7 ft diameter is the minimum size. The following four cases are based on the 7 ft diameter.

Case 3:

The results in Case 3 are shown in Figs. 3.1 to 3.5. This case has the same modeling condition as Case 1 except for the diameter reduction from 9 ft to 7 ft. The results show that the maximum water surface elevation in the pipe becomes higher due to the diameter reduction. At the head structure, the maximum water elevation appears to slightly exceed the top elevation of the structure. But it lasts for less than one minute because no overflow at the structure is found in Fig. 3.5. Note that the overflow data are recorded every one minute.

Case 4:

Figs. 4.1 to 4.5 show the results in Case 4. This case has the same modeling condition Case 2 except for the diameter reduction from 9 ft to 7 ft. Compared with Case 2, the maximum water elevation becomes higher, and very small amount (less than 20 cfs) overflow occurs at the head structure, as indicated in Fig. 4.5.

It is shown from the above two cases that 7 feet could be the minimum diameter for satisfying both 'dry' and 'wet' tunnel conditions. During the process of the diameter determination, HNTB also asked the sub–contractor to evaluate the rise of the pipe invert since a higher pipe invert may be more economical. The following two cases are based on the 7 ft diameter pipe, but the entire pipe invert is moved 3 ft up while the top elevation of the head structure was kept the same.

Case 5:

This is the 'dry' tunnel case with 7 ft diameter pipe and the invert elevated 3 ft. The results are shown in Figs. 5.1 to 5.5. Compared with Case 3, the hydraulic transient condition is greatly improved due to the increased invert elevation. The maximum water surface elevation at the head structure is lower than the top elevation of the structure.

Case 6:

This is the 'wet' tunnel condition corresponding to the above case. As indicated in Figs. 6.1 to 6.5, the results are almost the same as those in Case 4 since the pressurized flow is not affected by the invert elevation.

Case 7:

This case is to demonstrate what will happen if the pipe diameter is less than 7 ft. This example shows the 'wet' tunnel condition with 6 ft diameter pipe. The results are displayed in Figs. 7.1 to 7.5. The overflow at the head structure lasts about two hours and the maximum overflow is up to more than 100 cfs.

Case 8:

This case has the similar conditions as Case 3 but the inflow at the head structure is reduced from 80 cfs to 30 cfs. The results are shown in Figs. 8.1 to 8.5. It appears that the hydraulic transient condition in this case is improved with the smaller inflow, compared with Case 3.

Case 9:

Similarly, the difference between this case and Case 4 is the deduction of the inflow from 80 cfs to 30 cfs. The results are shown in Figs. 9.1 to 9.5. Due to the smaller inflow, the small amount of overflow from head structure in Case 4 becomes even smaller in this case, as indicated in Fig. 9.5.

The results in the above cases show that the inflow deduction does improve the transient flow condition in the pipe. However, the maximum water surface elevation at the head structure in the both cases is very close to the top elevation of the head structure even though no overflow is found there.

Case 10:

This case is similar to Case 8, but the pipe diameter is reduced to 6 ft. As shown in Fig. 10.1, there are two surge peaks in the pipe. The later one is the same as that in Case 6 due to the backward surge from the main tunnel. The early one is generated in the pipe because the smaller diameter lacks the conveyance to carry the inflow from the two dropshafts (at Stations 11 and 29) to downstream tunnel. This strong surge also causes much higher maximum water surface elevation, as shown in Fig. 10.4. Due to its short period, Fig. 10.5 does not catch the overflow from the head structure in the one minute interval records.

Case 11:

This is the 'wet' tunnel condition corresponding to the above case. The results are shown in Figs. 11.1 to 11.5. Apparently, the 'wet' tunnel condition solves all the surge problems. Compared with Case 7, the overflow from the head structure due to the initial stagnant column still exists, but the overflow due to the pipe conveyance disappears because the 50 cfs deduction in this case is larger than the overflow in Case 7.

6. Conclusions

Based on the above modeling results, the following conclusions are suggested.

(1) The minimum diameter for Carriage Lane Outfall Pipe is 7 feet, which can satisfy both dry river and 10–year flood conditions.

(2) The rise of the pipe invert is favorable to the hydraulic transient performance of the pipe. The results with a rise of 3 feet show the hydraulic surge behavior in the 'dry' river case is greatly improved.

(3) All the modeling results are based on the head structure area of 300 ft^2. A smaller area may lead a stronger surge in the 'dry' river condition. The minimum cross–section area of 300 ft^2 at the head structure as previously determined is recommended.

(4) The inflow deduction at the head structure from 80 cfs to 30 cfs does somewhat improve the hydraulic transient condition in the pipe with 7 ft diameter. But if the pipe diameter is reduced to 6 ft, very strong surge could be generated in the pipe even with 30 cfs inflow. In fact, when the diameter is less than 7 ft, the transient flow features are mostly affected by the large amount of inflow from the two dropshafts (Station 11 and Station 29) due to the mild slop of the pipe.

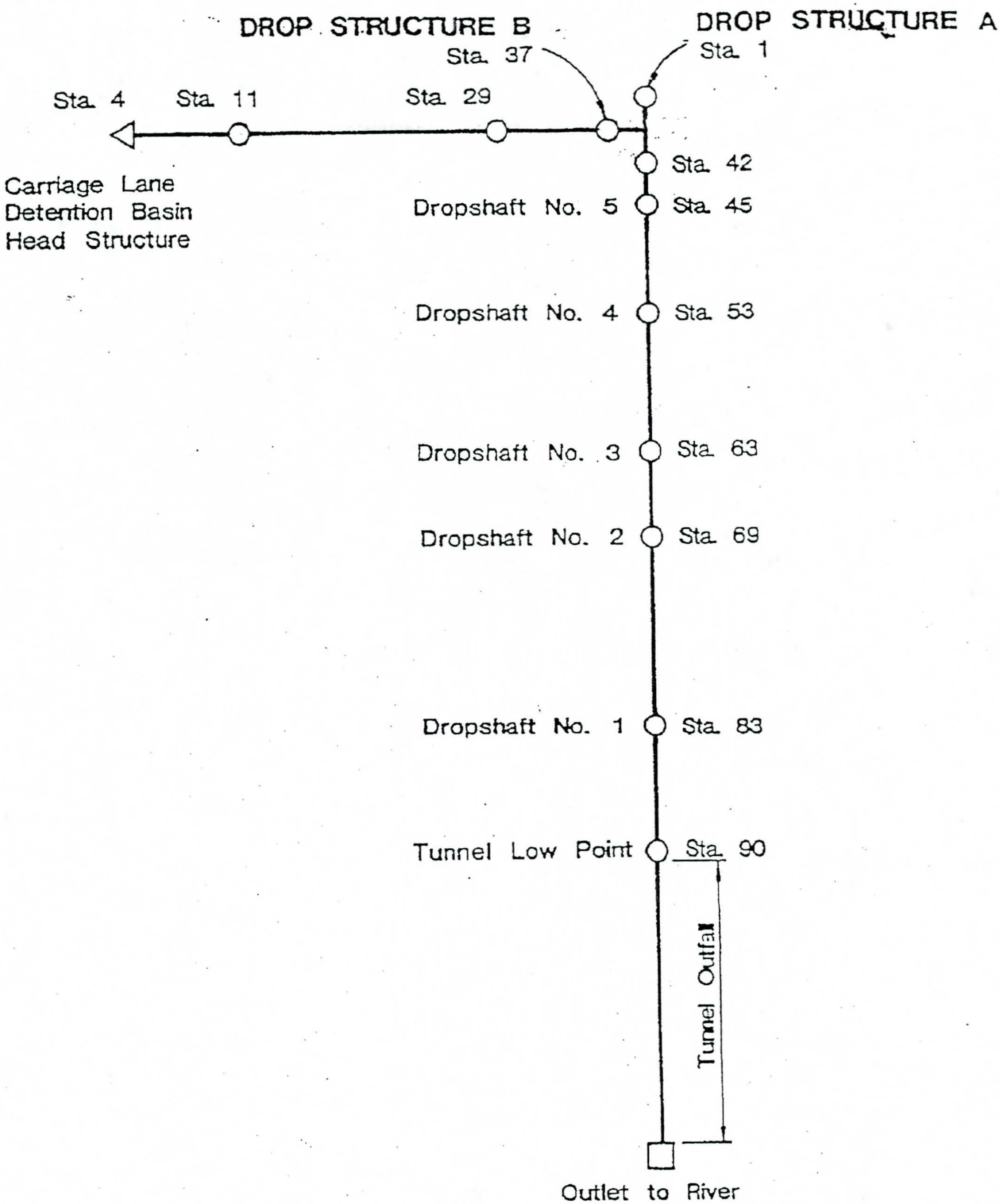


Fig. 1 Tunnel System's Configuration

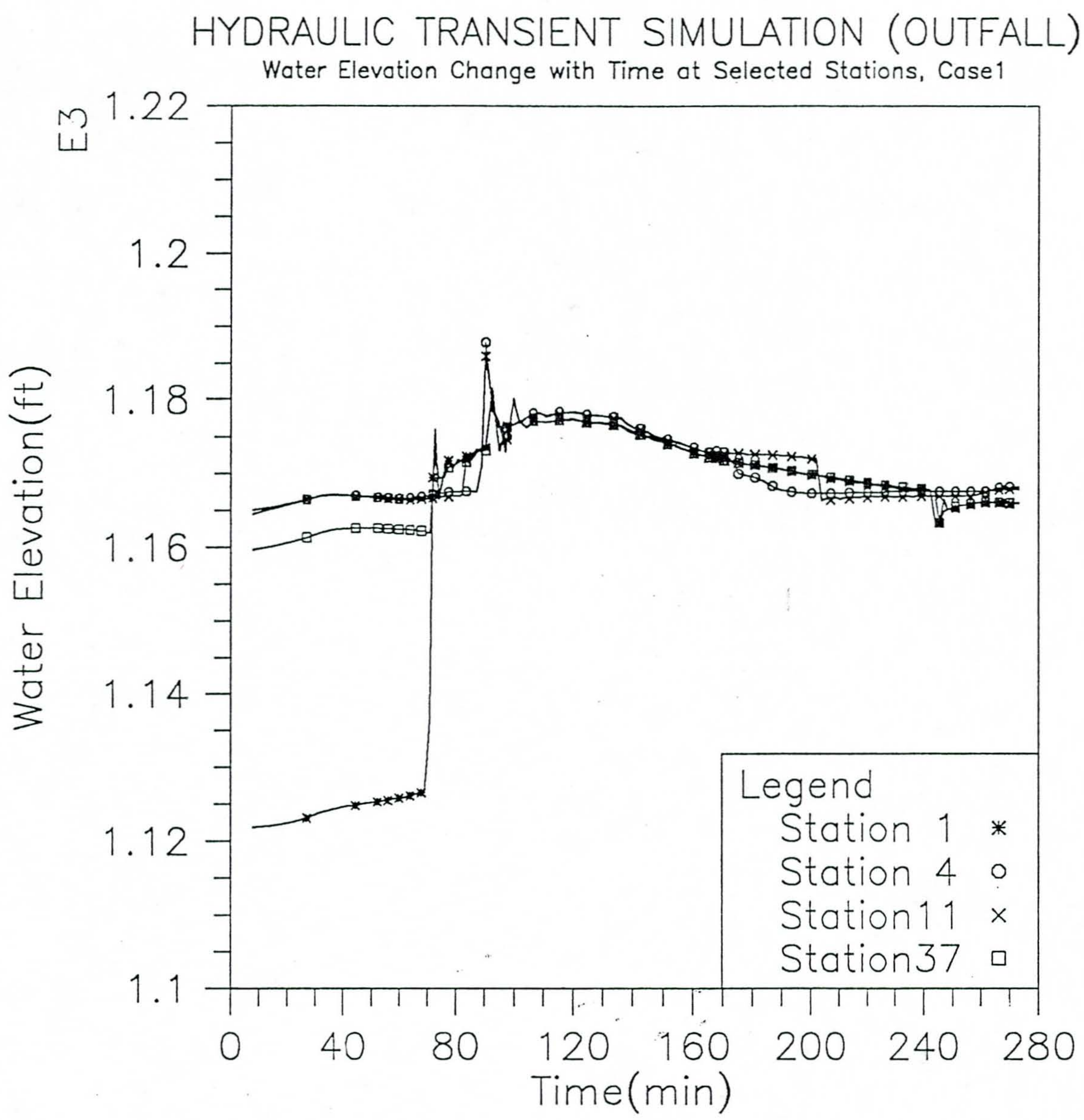


Fig. 1.1 Time variation of water surface elevations at four upstream locations, modeling case: 9 ft diameter and 'dry' tunnel (Case 1).

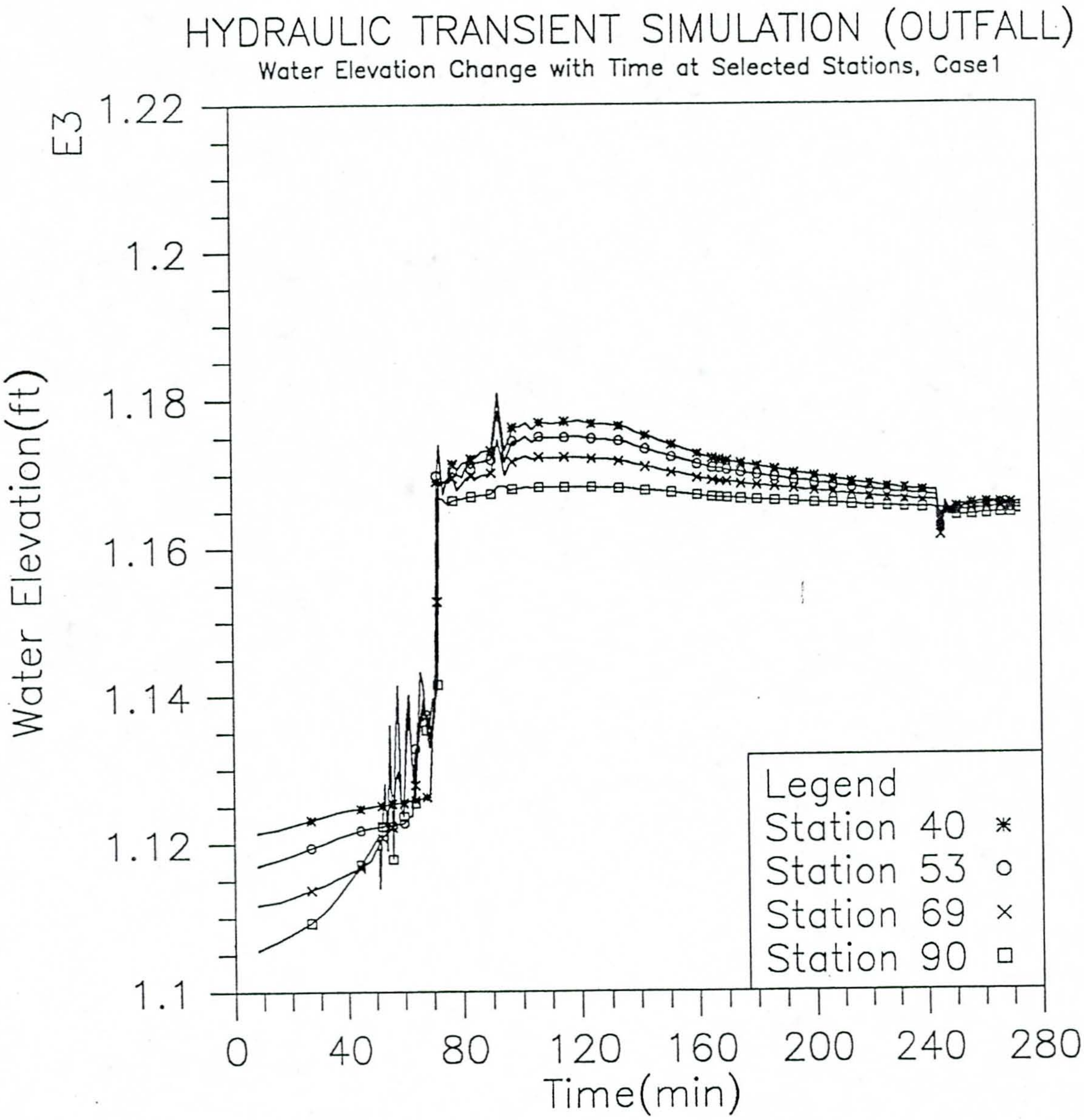


Fig. 1.2 Time variation of water surface elevations at four downstream locations, modeling case: 9 ft diameter and 'dry' tunnel (Case 1).

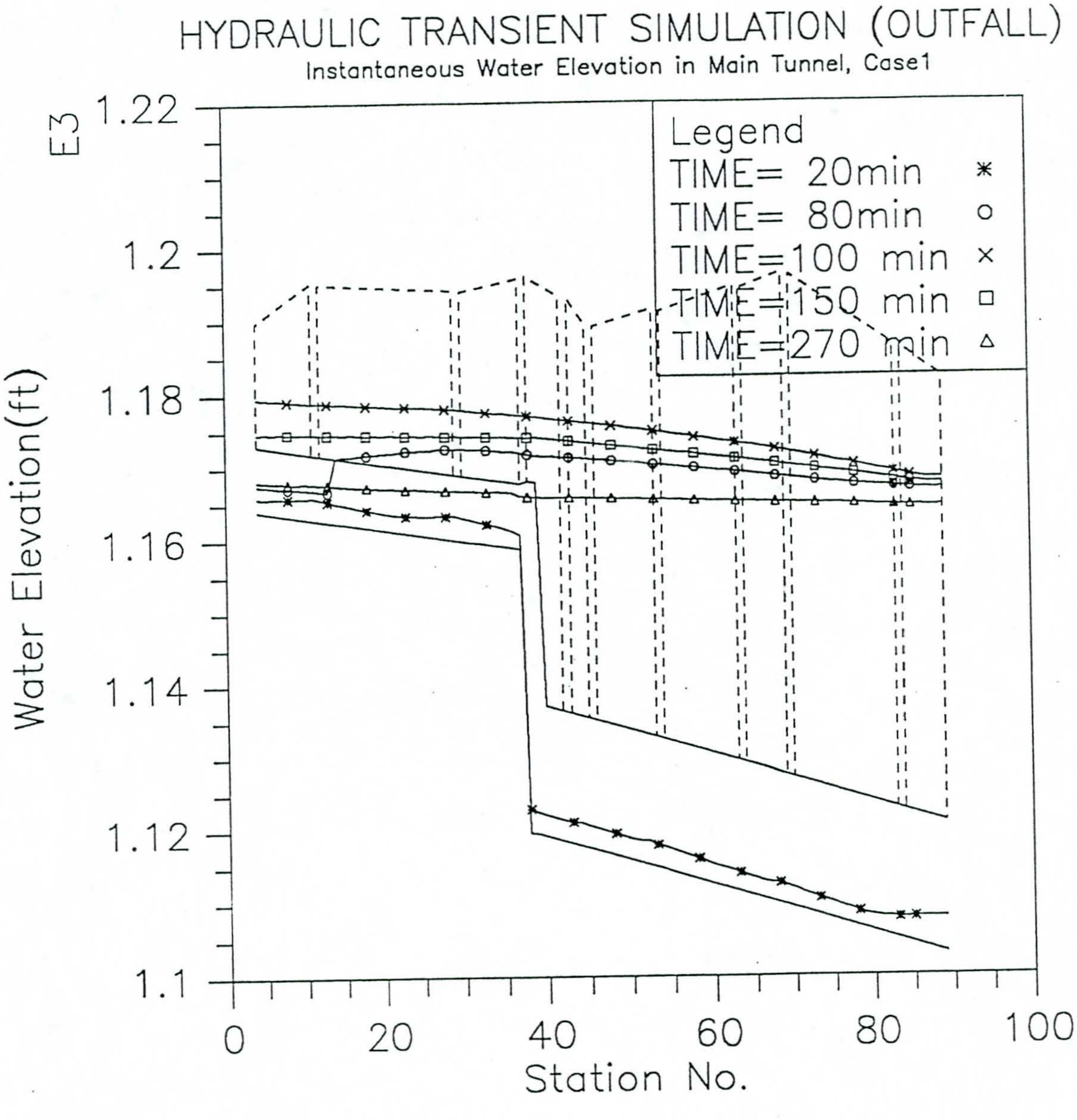


Fig. 1.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 9 ft diameter and 'dry' tunnel (Case 1).

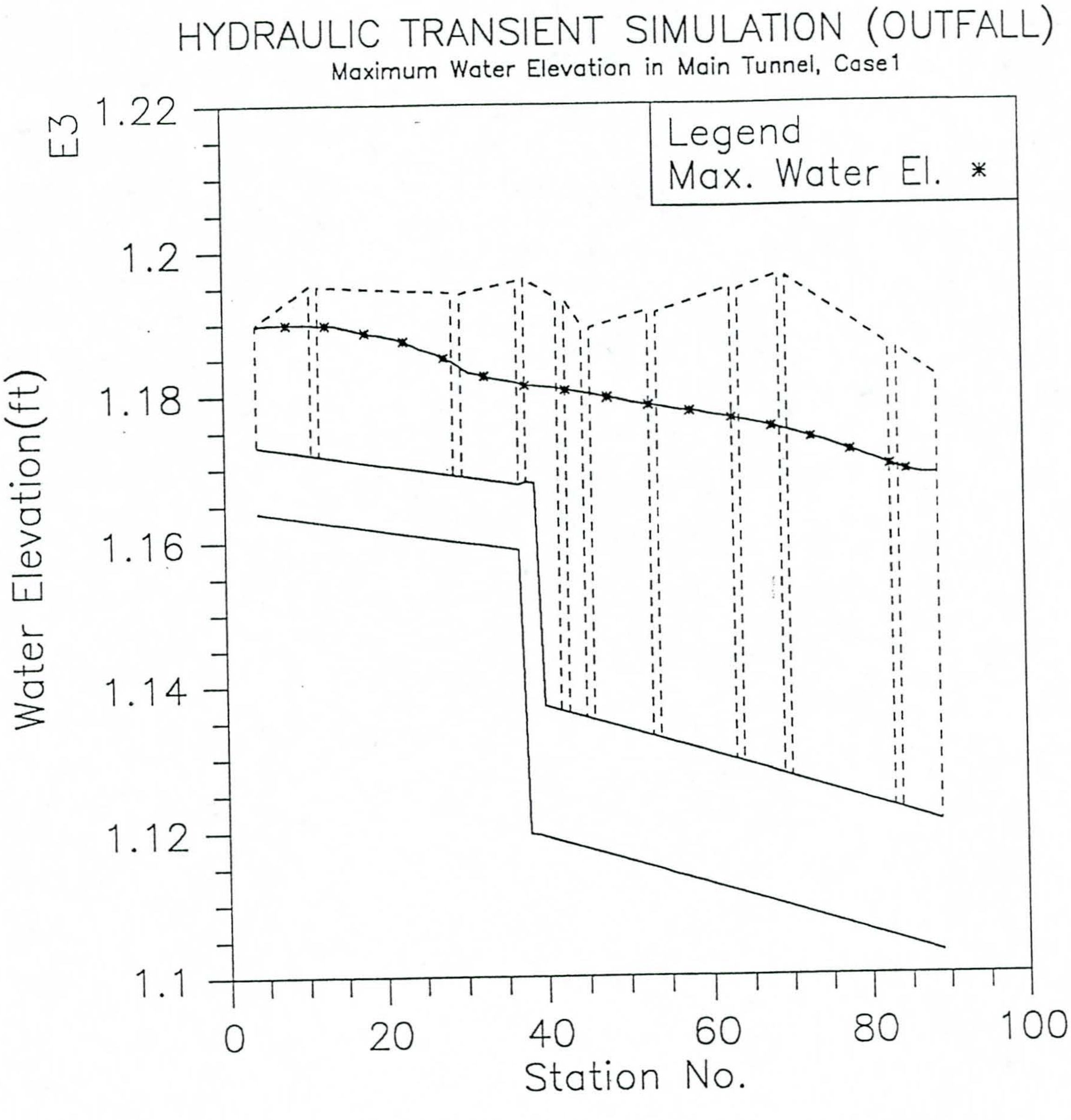


Fig. 1.4 The maximum water surface elevations along the tunnel system, modeling case: 9 ft diameter and 'dry' tunnel (Case 1).

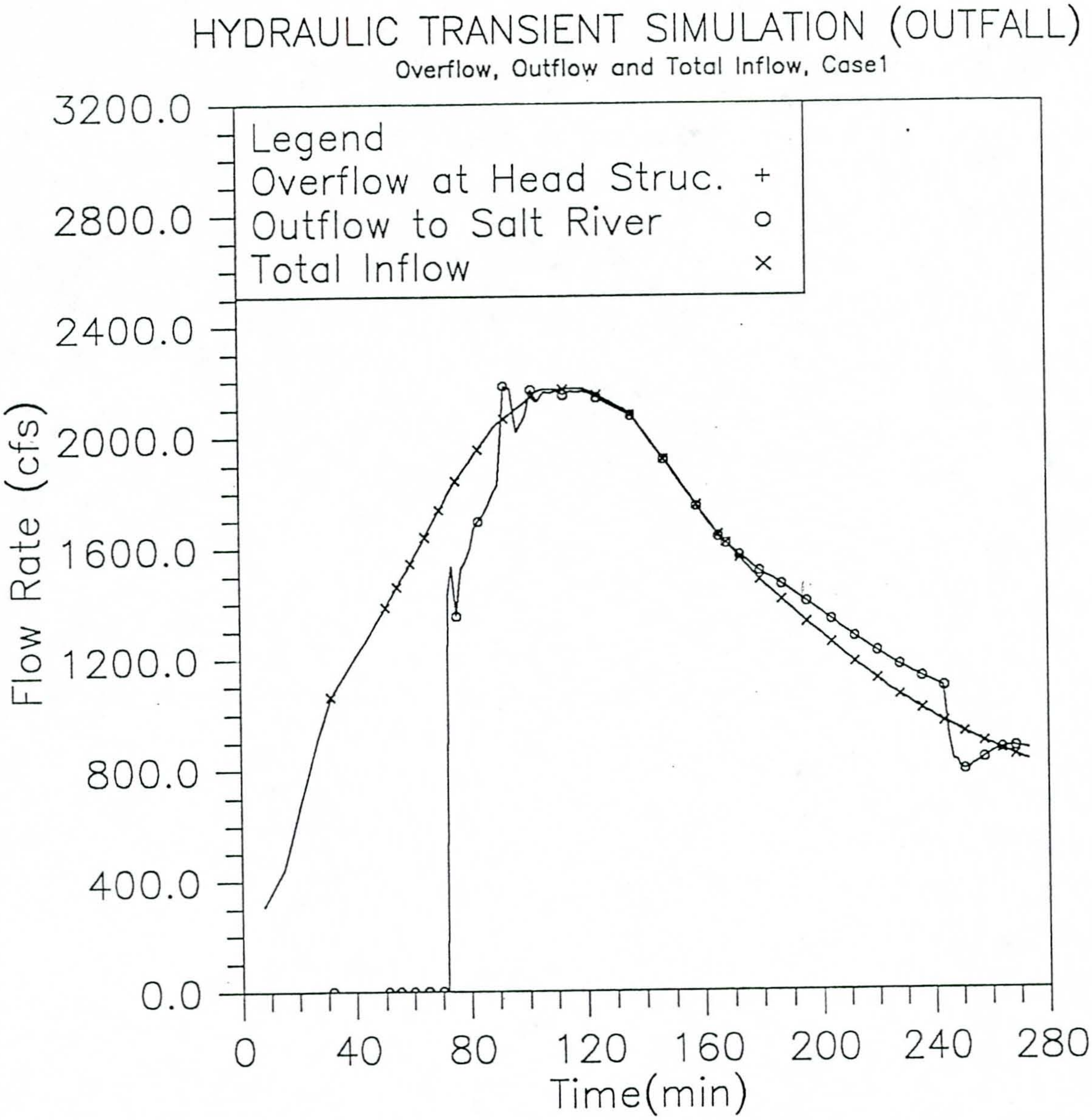


Fig. 1.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 9 ft diameter and 'dry' tunnel (Case 1).

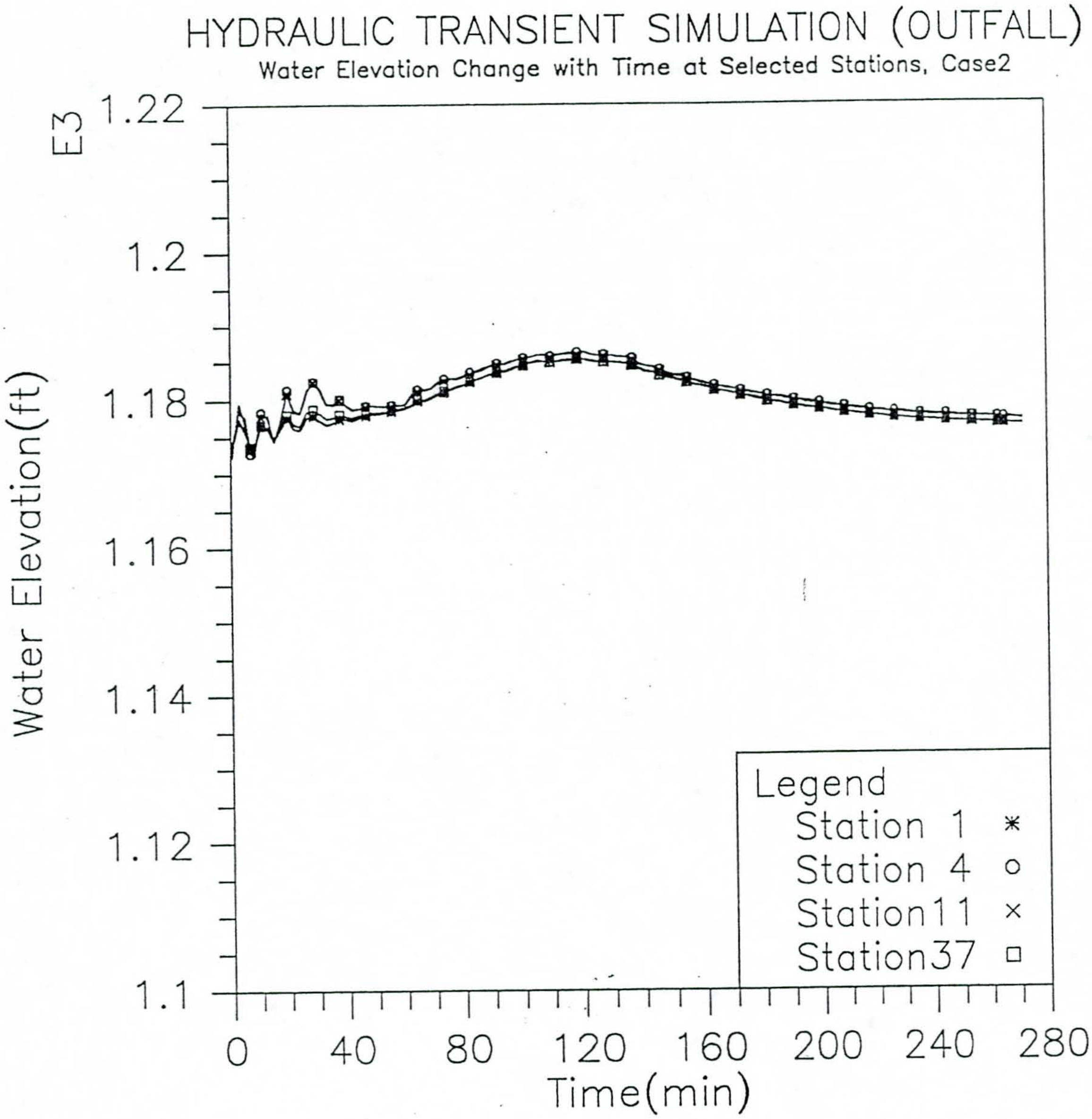


Fig. 2.1 Time variation of water surface elevations at four upstream locations, modeling case: 9 ft diameter and 'wet' tunnel (Case 2).

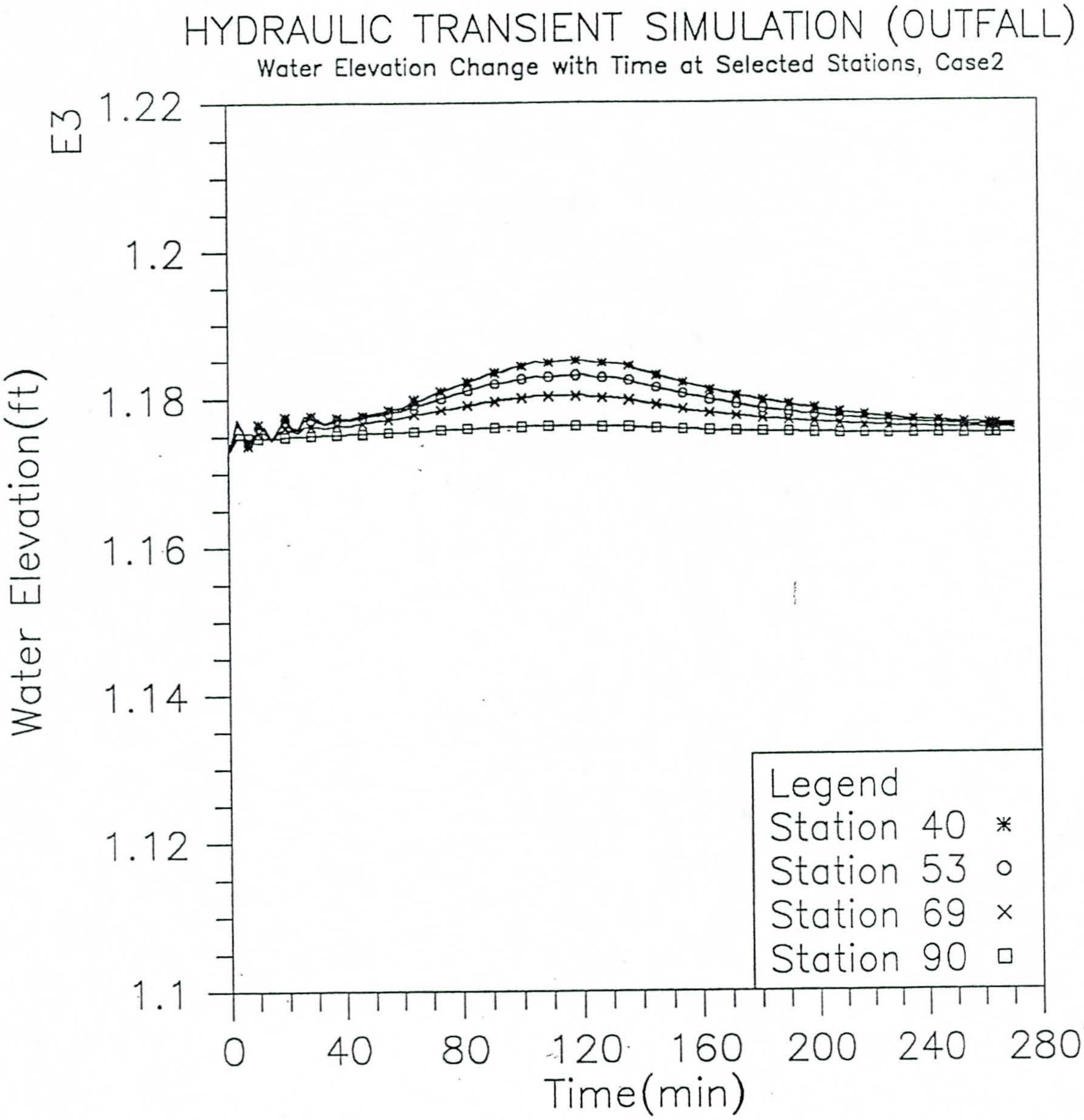


Fig. 2.2 Time variation of water surface elevations at four downstream locations, modeling case: 9 ft diameter and 'wet' tunnel (Case 2).

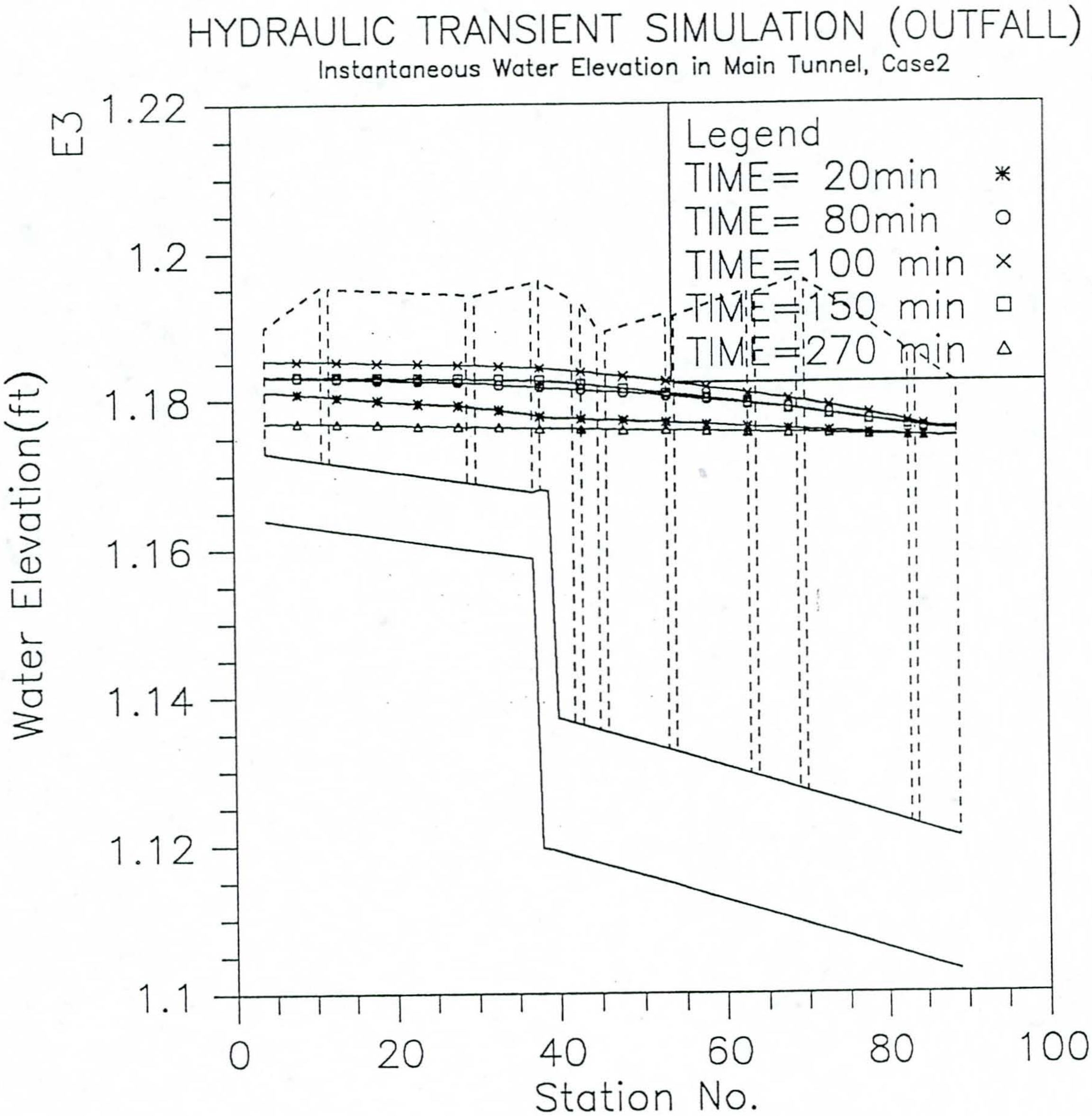


Fig. 2.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 9 ft diameter and 'wet' tunnel (Case 2).

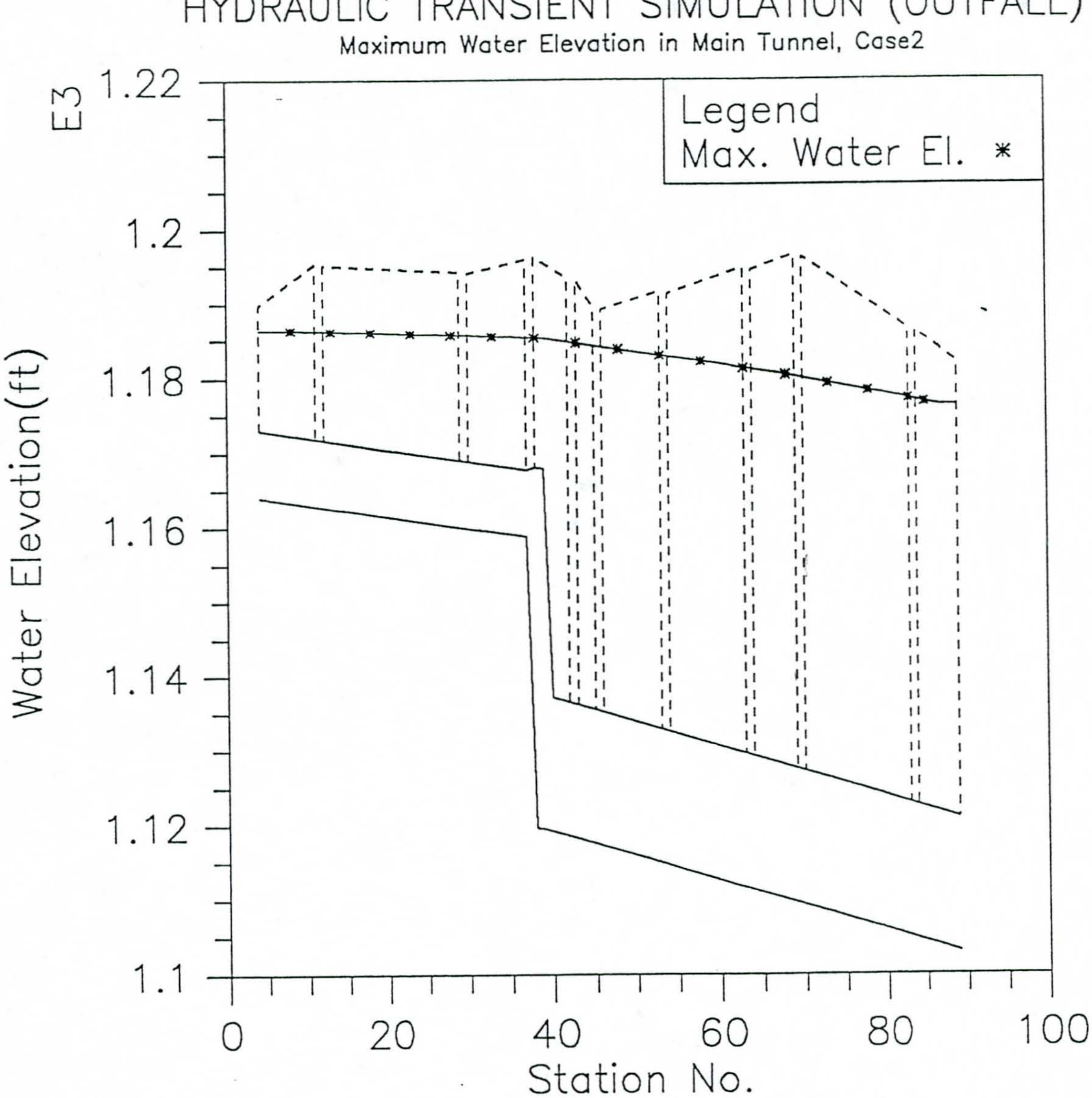


Fig. 2.4 The maximum water surface elevations along the tunnel system, modeling case: 9 ft diameter and 'wet' tunnel (Case 2).

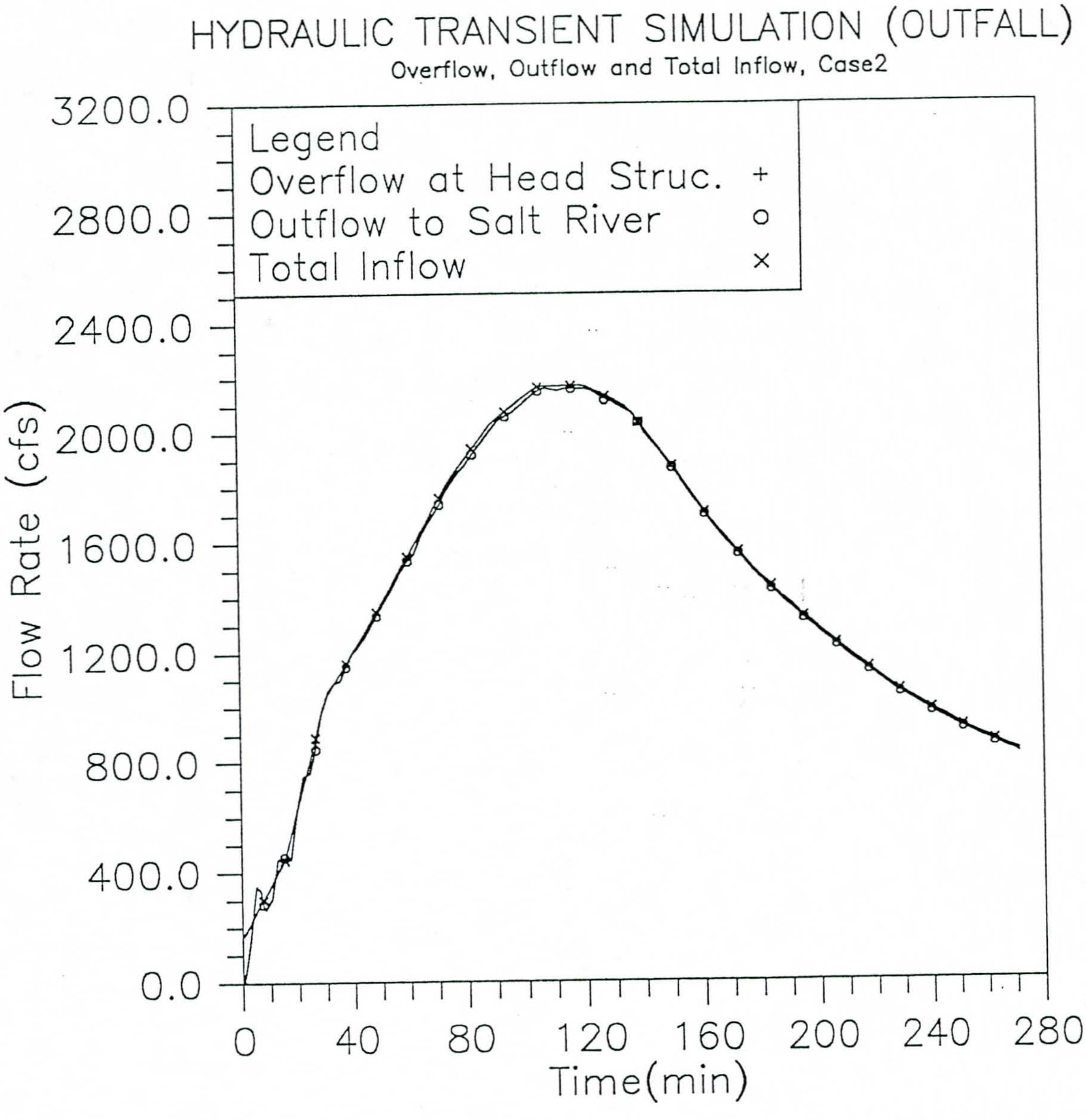


Fig. 2.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 9 ft diameter and 'wet' tunnel (Case 2).

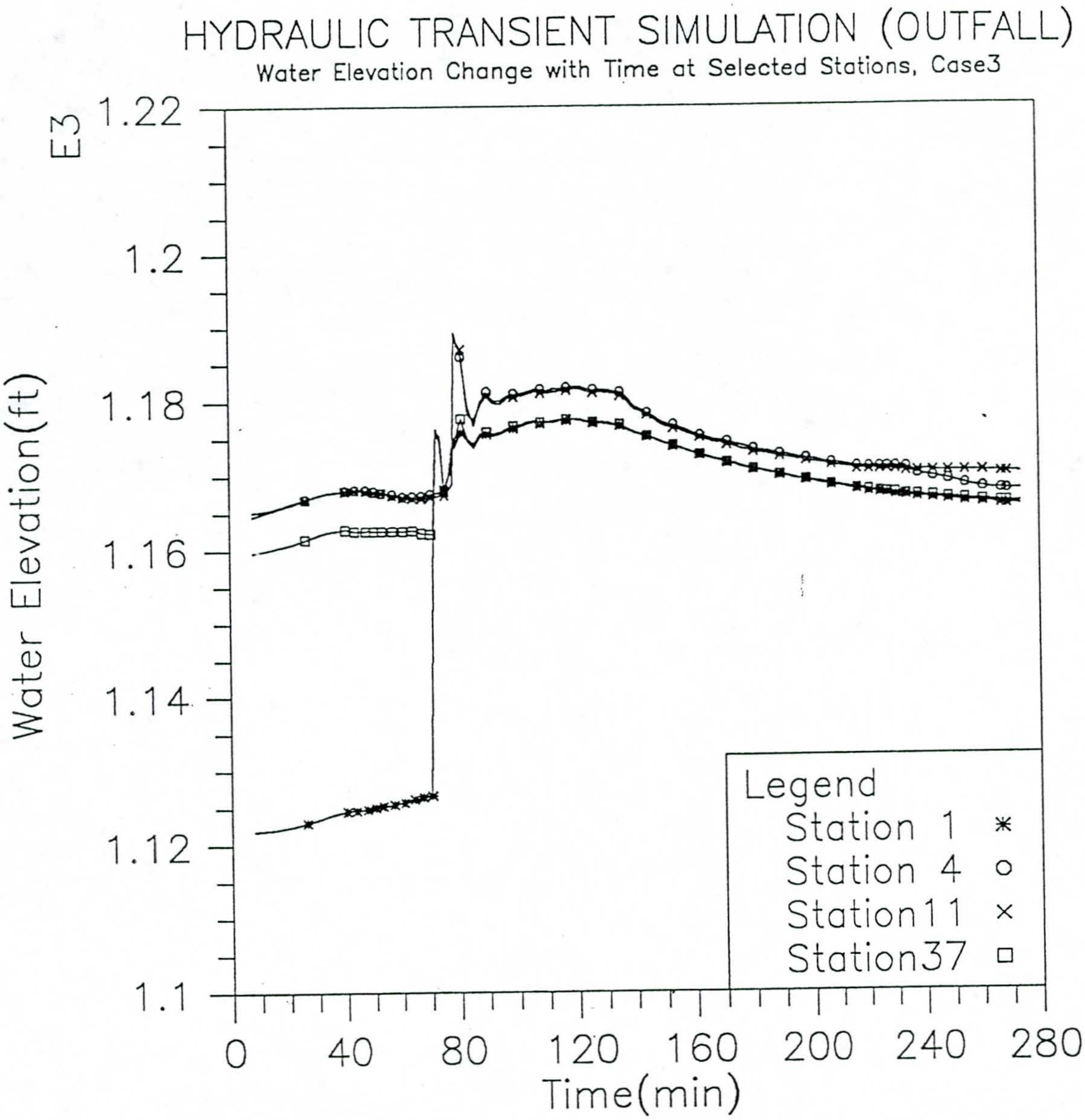


Fig. 3.1 Time variation of water surface elevations at four upstream locations, modeling case: 7 ft diameter and 'dry' tunnel (Case 3).

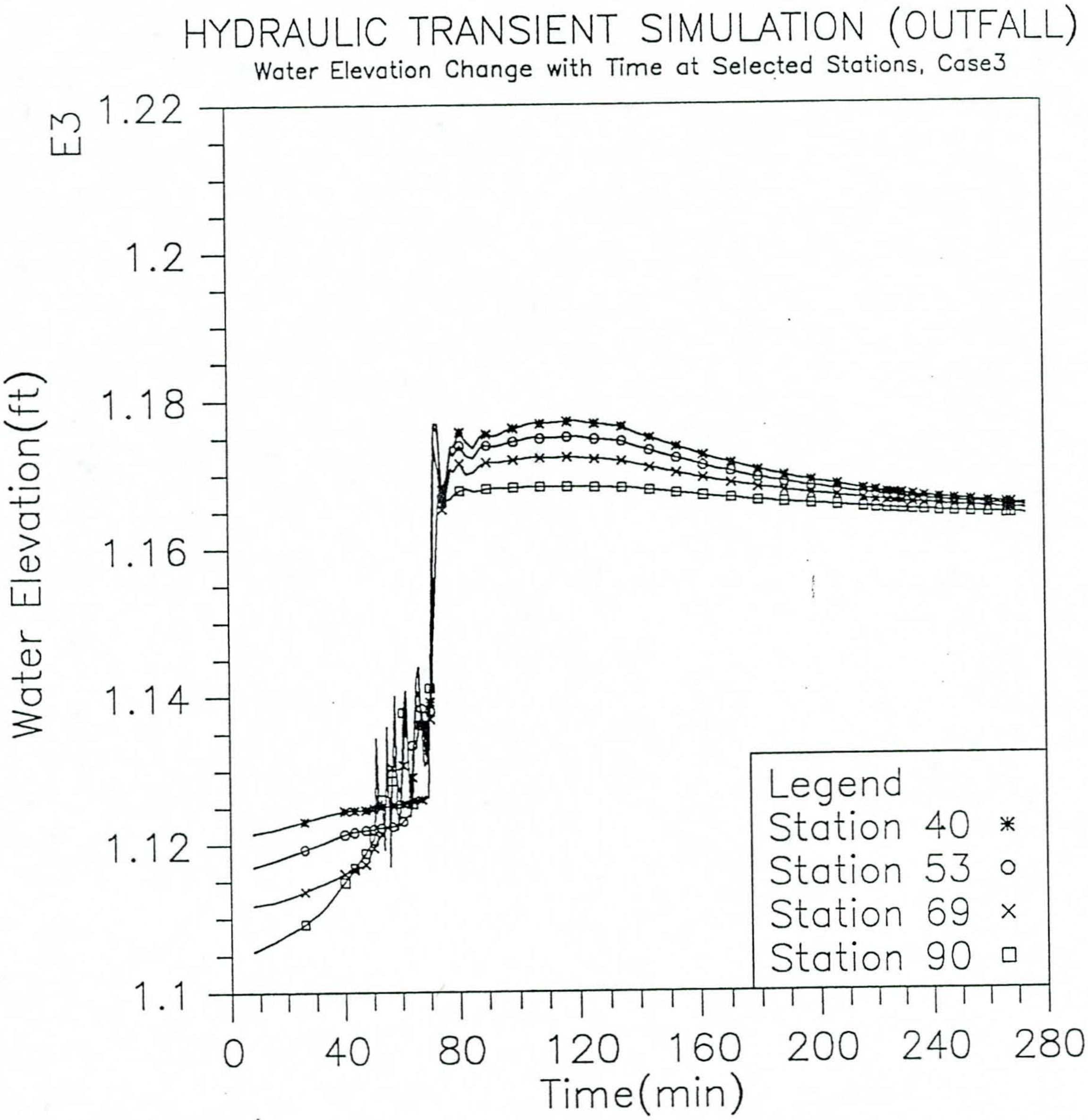


Fig. 3.2 Time variation of water surface elevations at four downstream locations, modeling case: 7 ft diameter and 'dry' tunnel (Case 3).

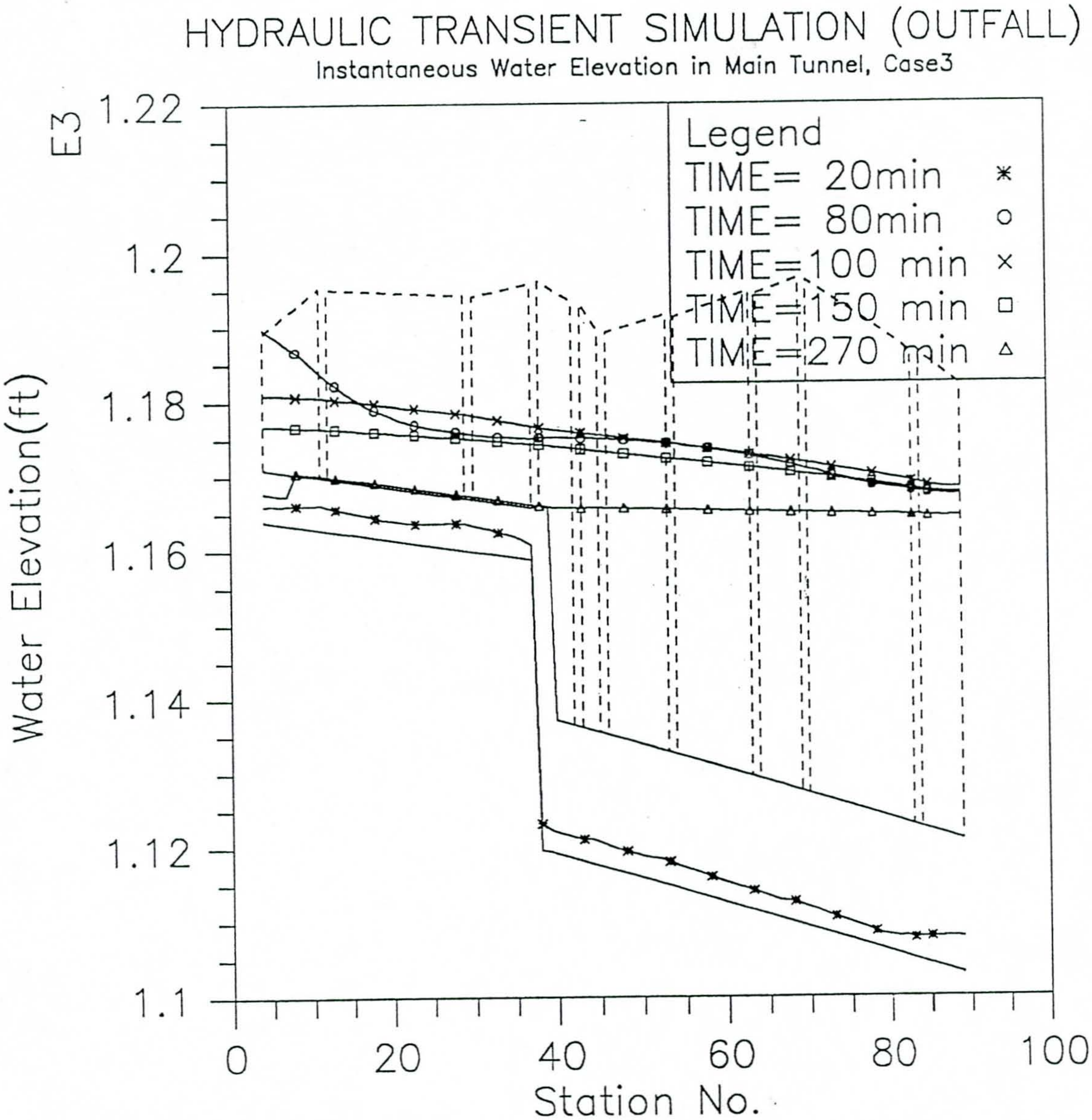


Fig. 3.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 7 ft diameter and 'dry' tunnel (Case 3).

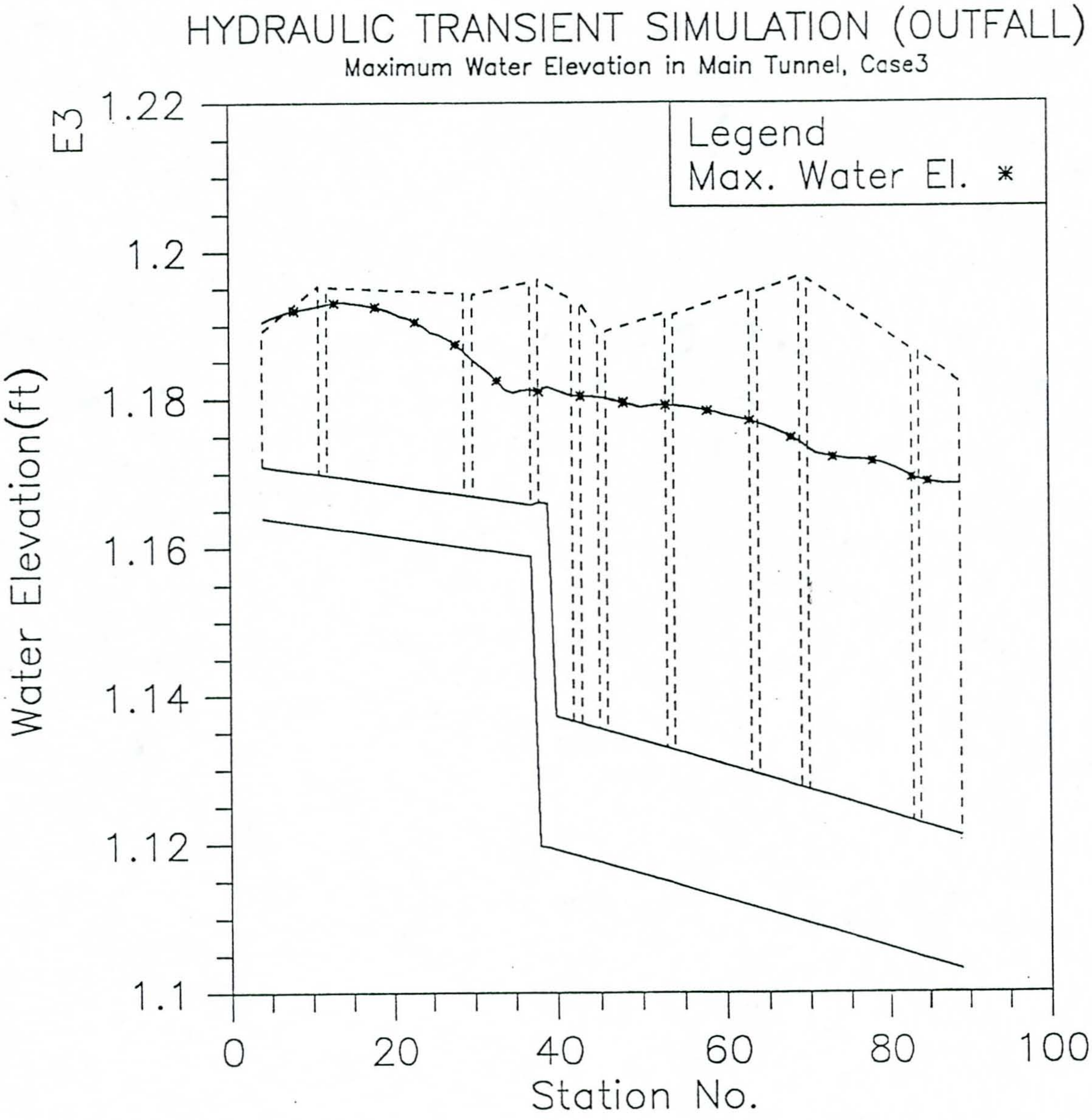


Fig. 3.4 The maximum water surface elevations along the tunnel system, modeling case: 7 ft diameter and 'dry' tunnel (Case 3).

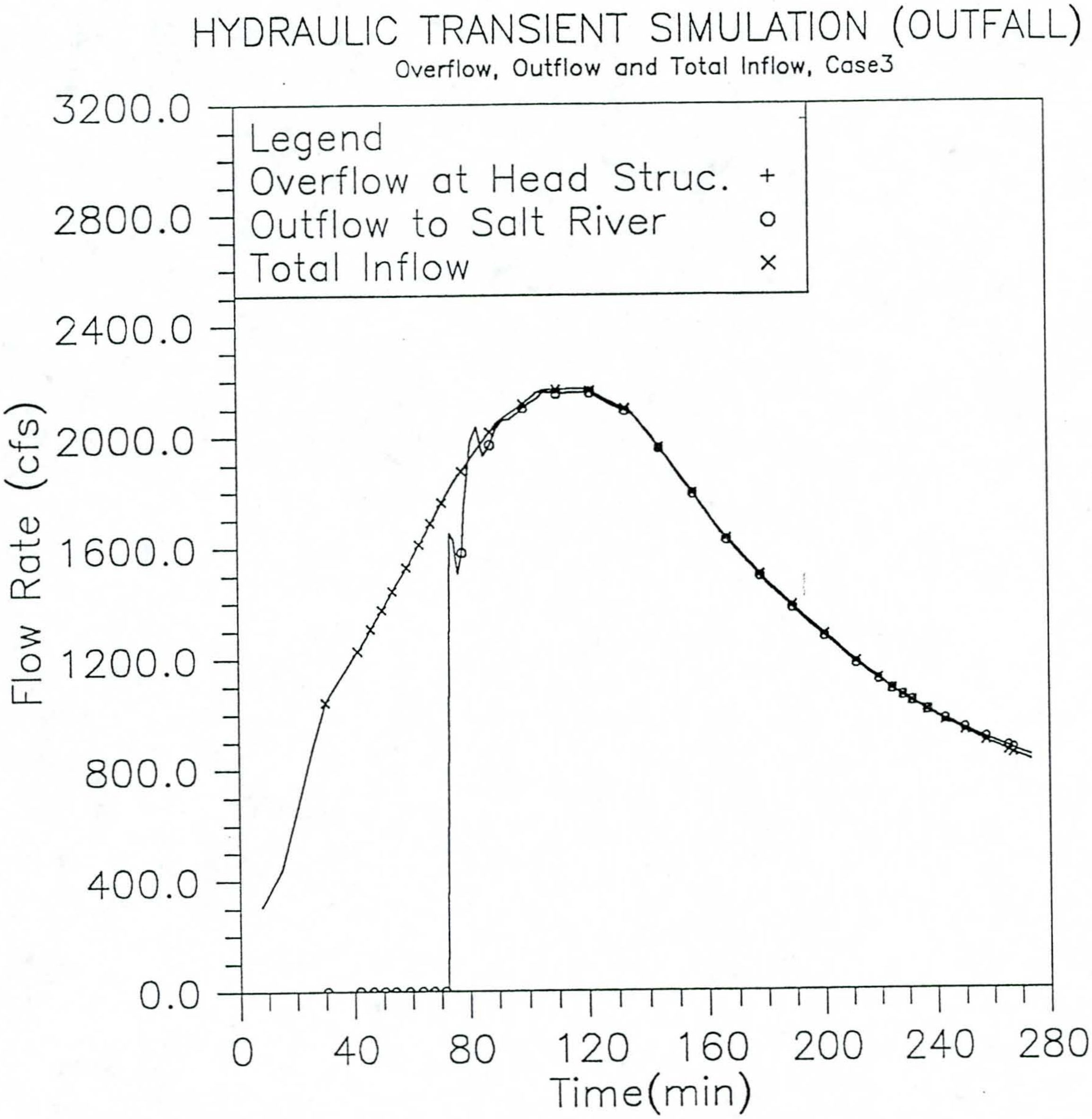


Fig. 3.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 7 ft diameter and 'dry' tunnel (Case 3).

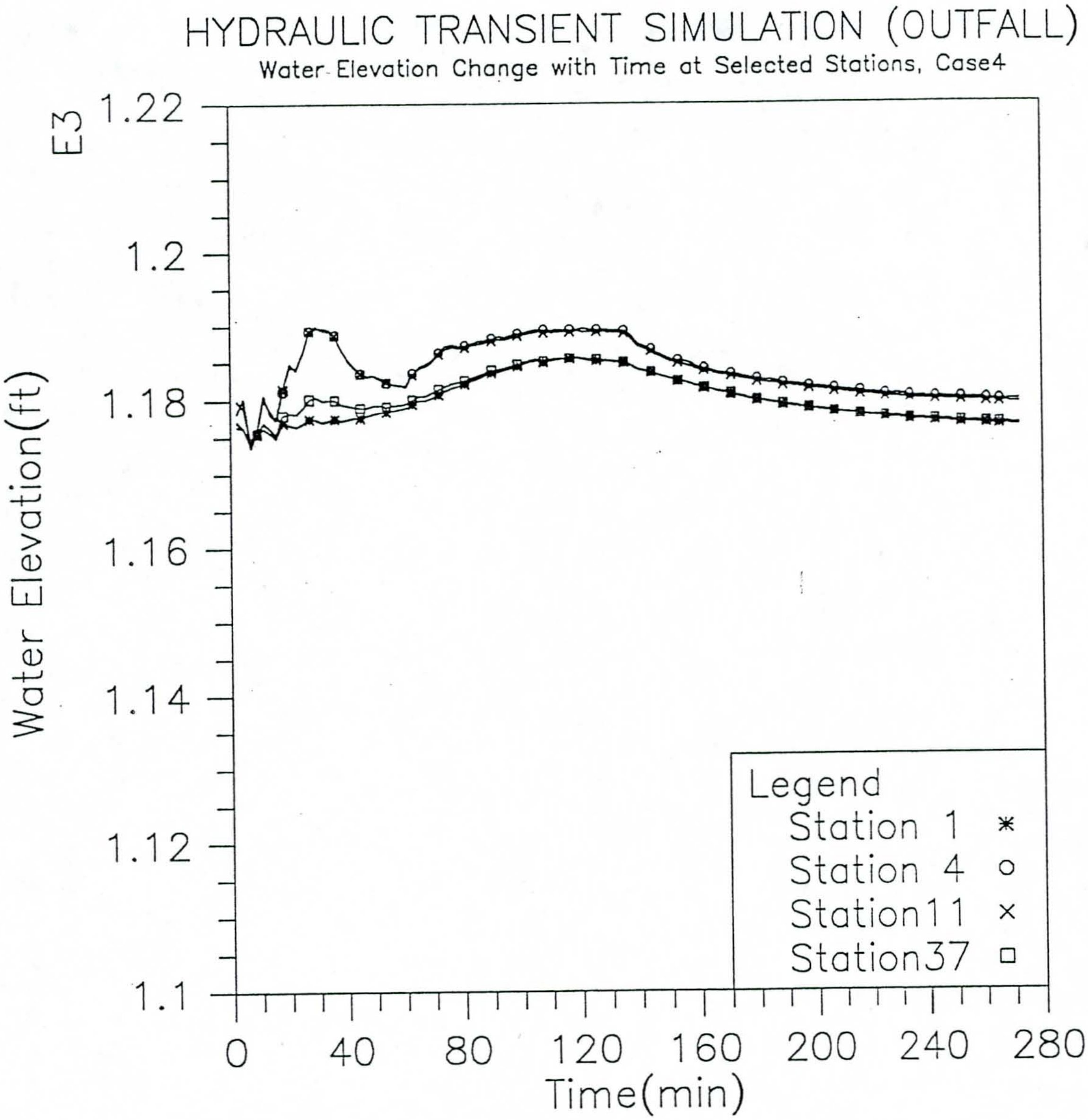


Fig. 4.1 Time variation of water surface elevations at four upstream locations, modeling case: 7 ft diameter and 'wet' tunnel (Case 4).

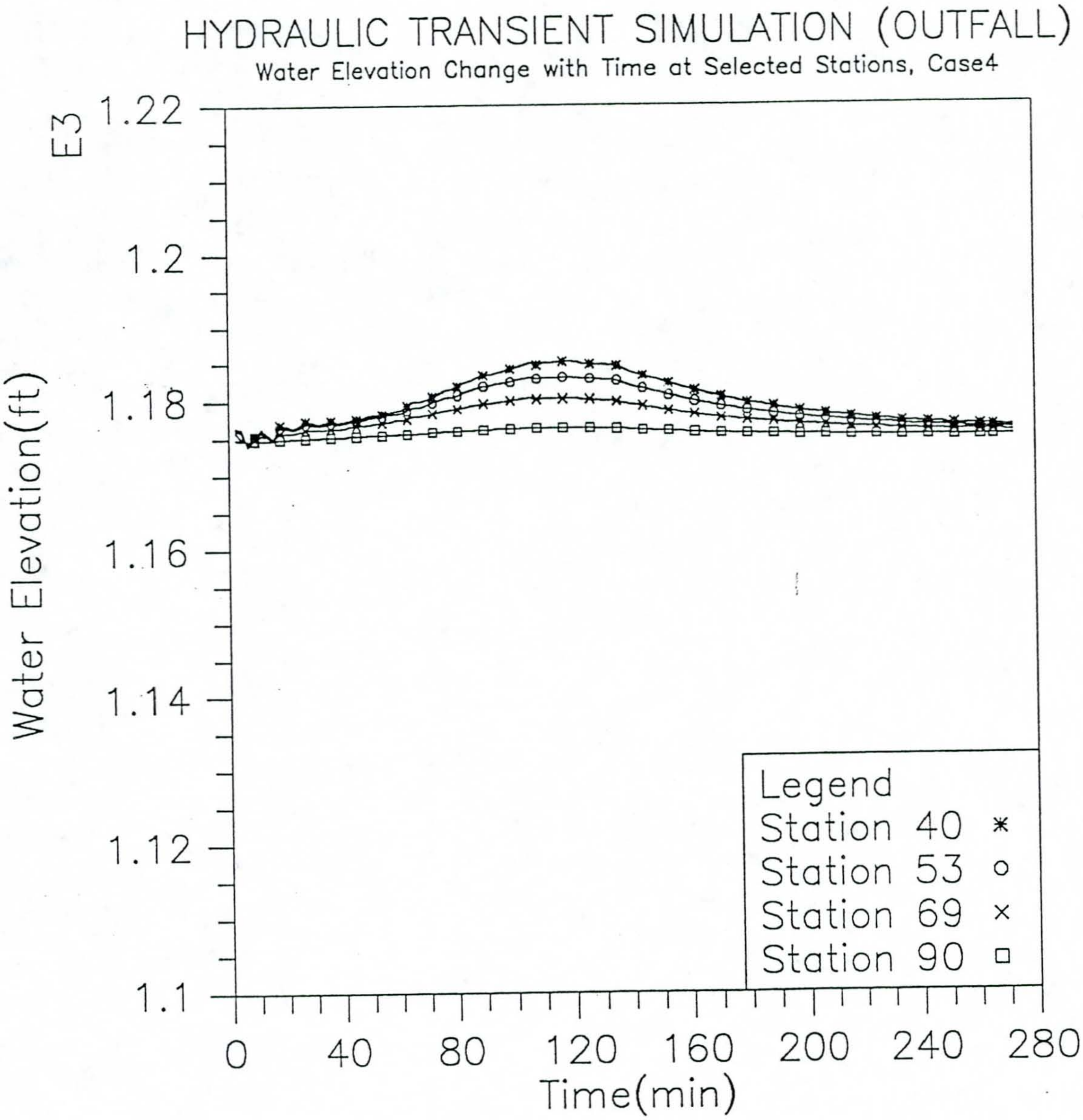


Fig. 4.2 Time variation of water surface elevations at four downstream locations, modeling case: 7 ft diameter and 'wet' tunnel (Case 4).

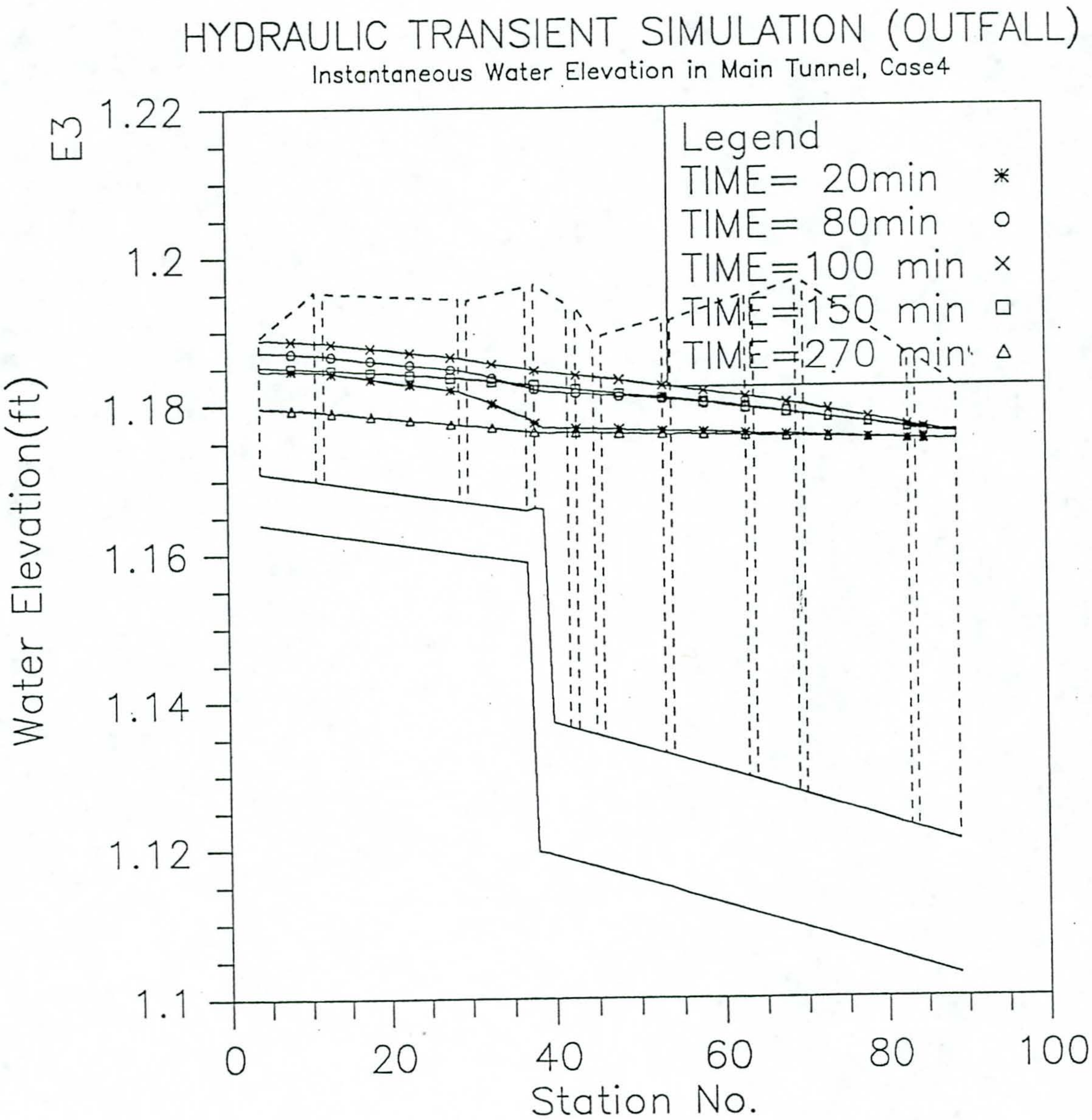


Fig. 4.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 7 ft diameter and 'wet' tunnel (Case 4).

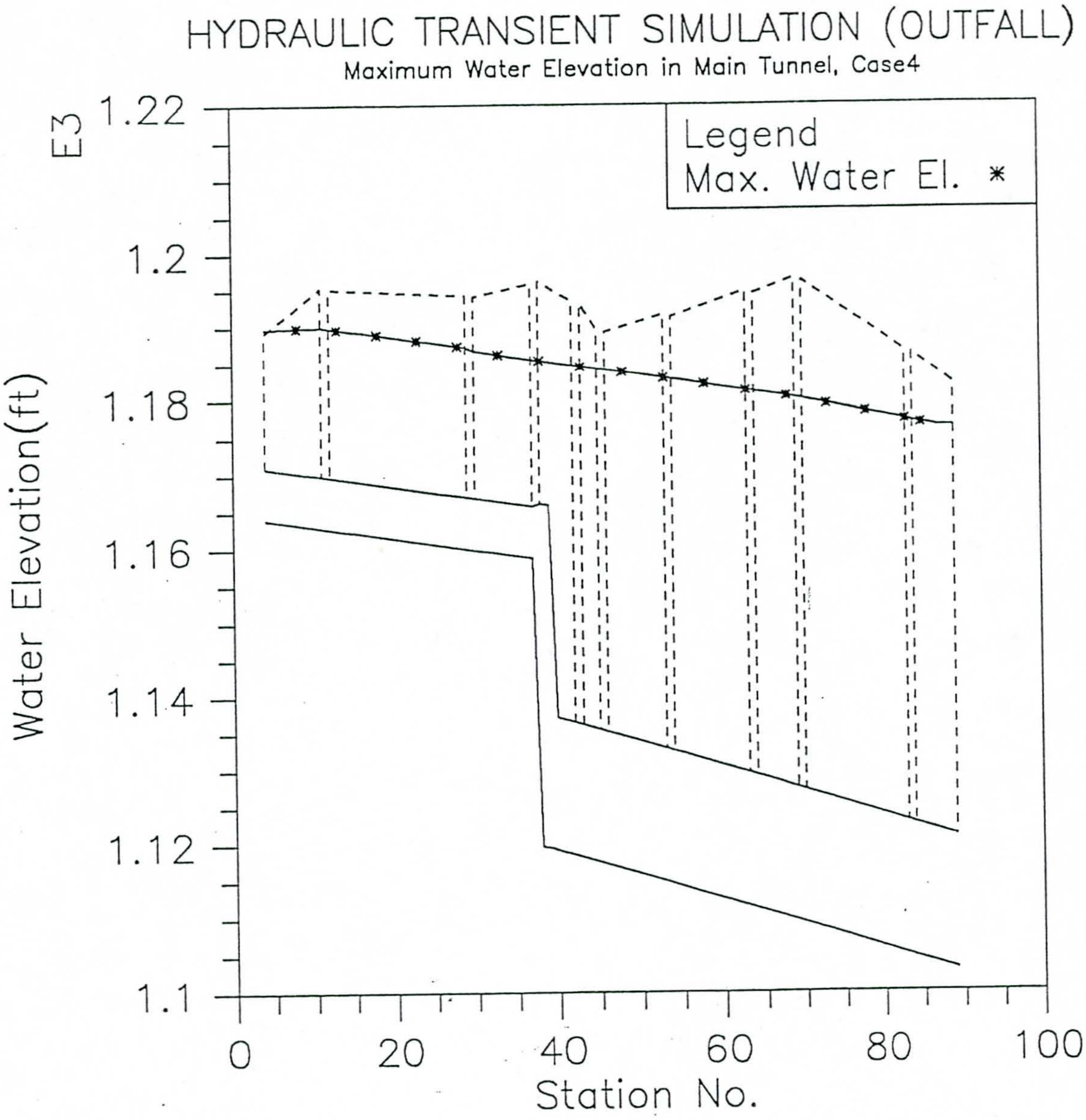


Fig. 4.4 The maximum water surface elevations along the tunnel system, modeling case: 7 ft diameter and 'wet' tunnel (Case 4).

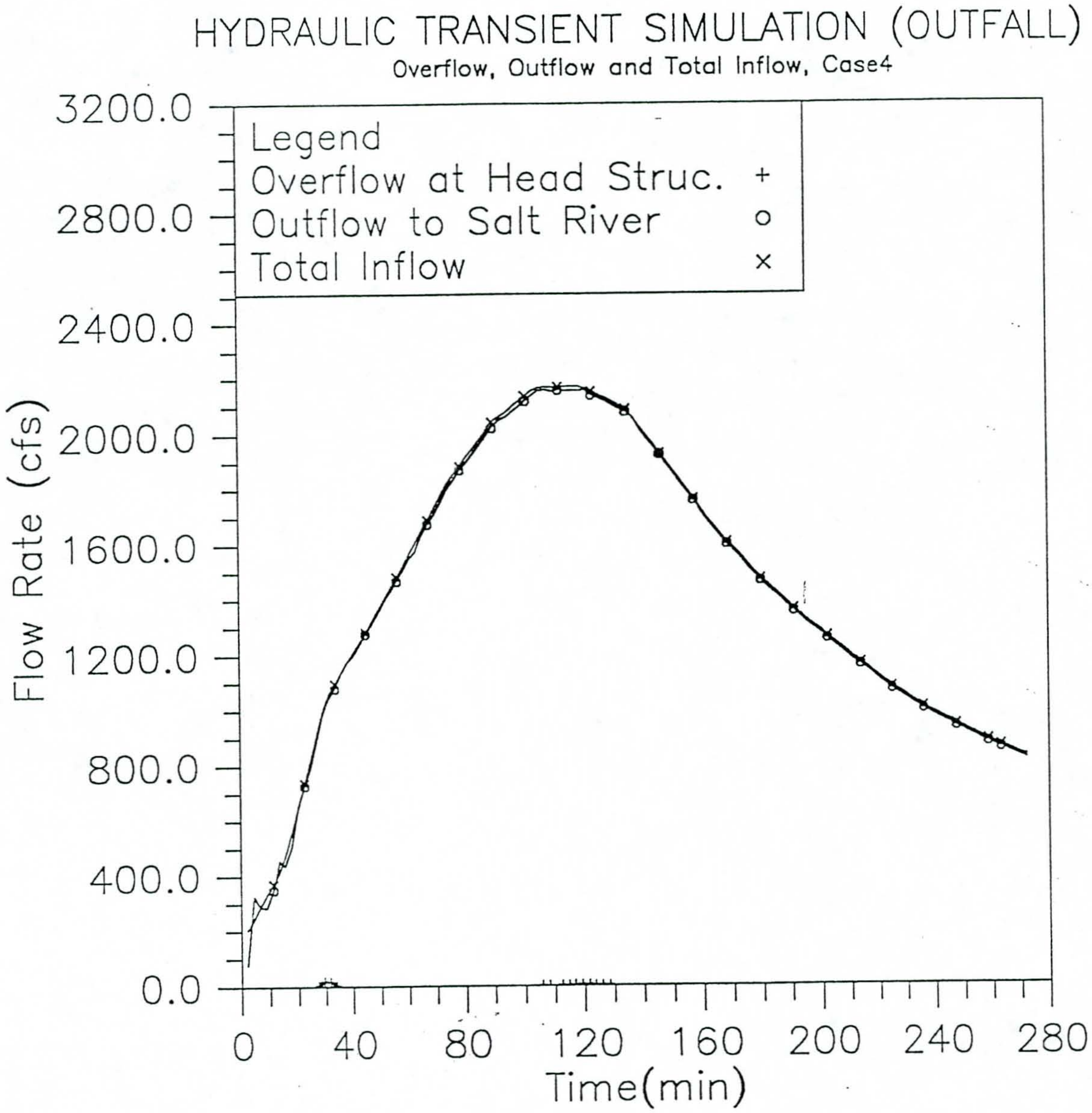


Fig. 4.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 7 ft diameter and 'wet' tunnel (Case 4).

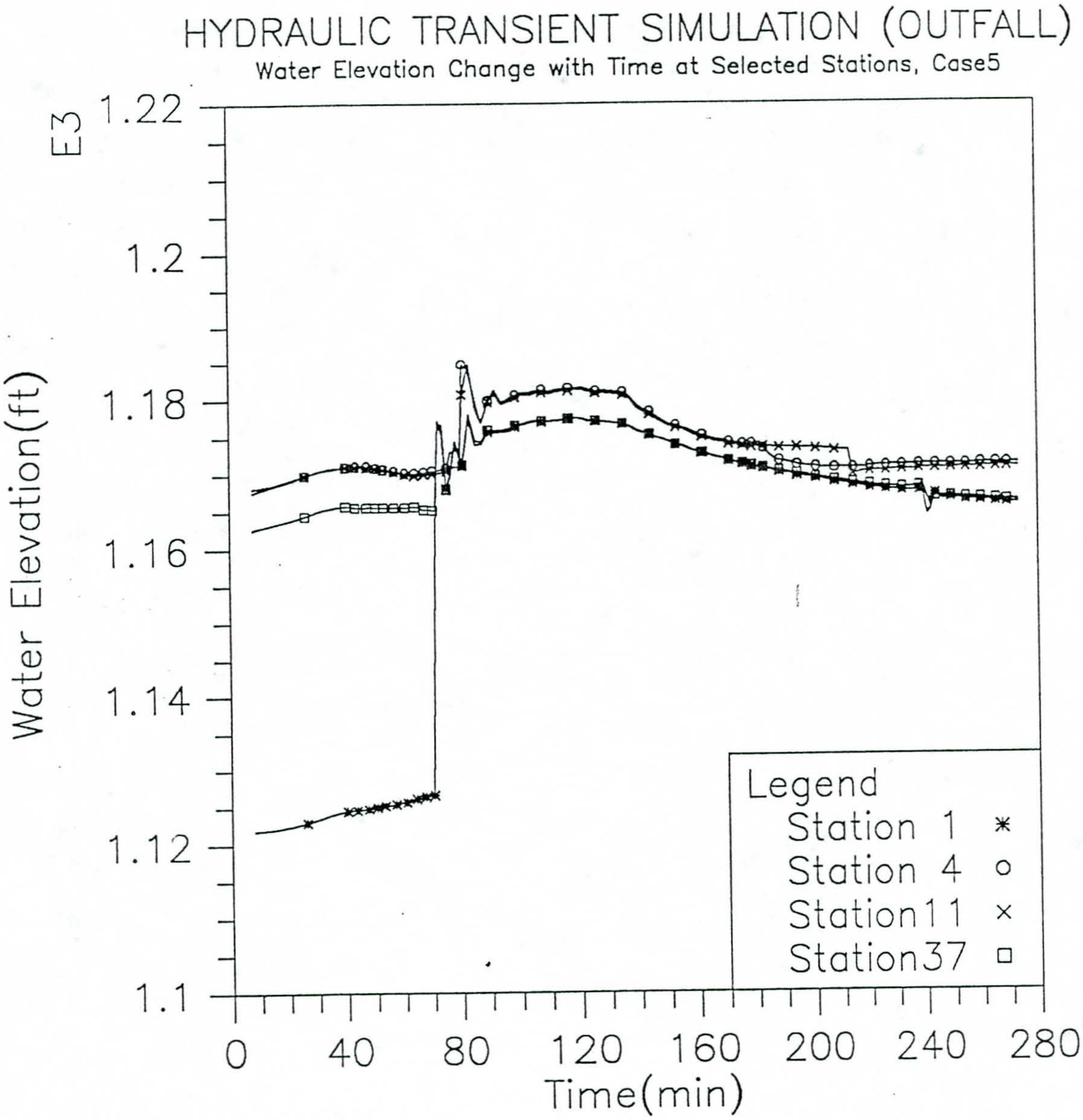


Fig. 5.1 Time variation of water surface elevations at four upstream locations, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 5).

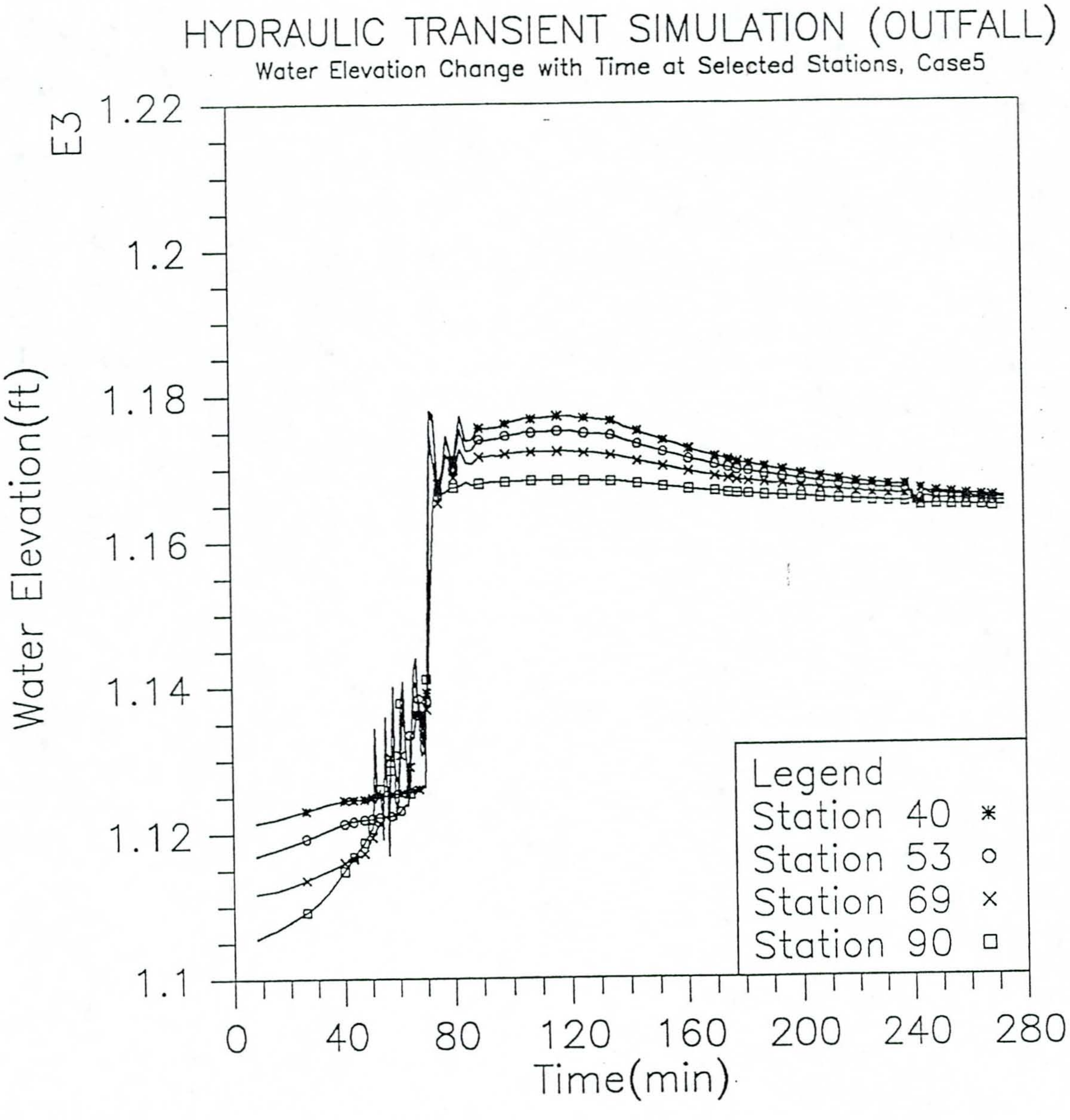


Fig. 5.2 Time variation of water surface elevations at four downstream locations, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 5).

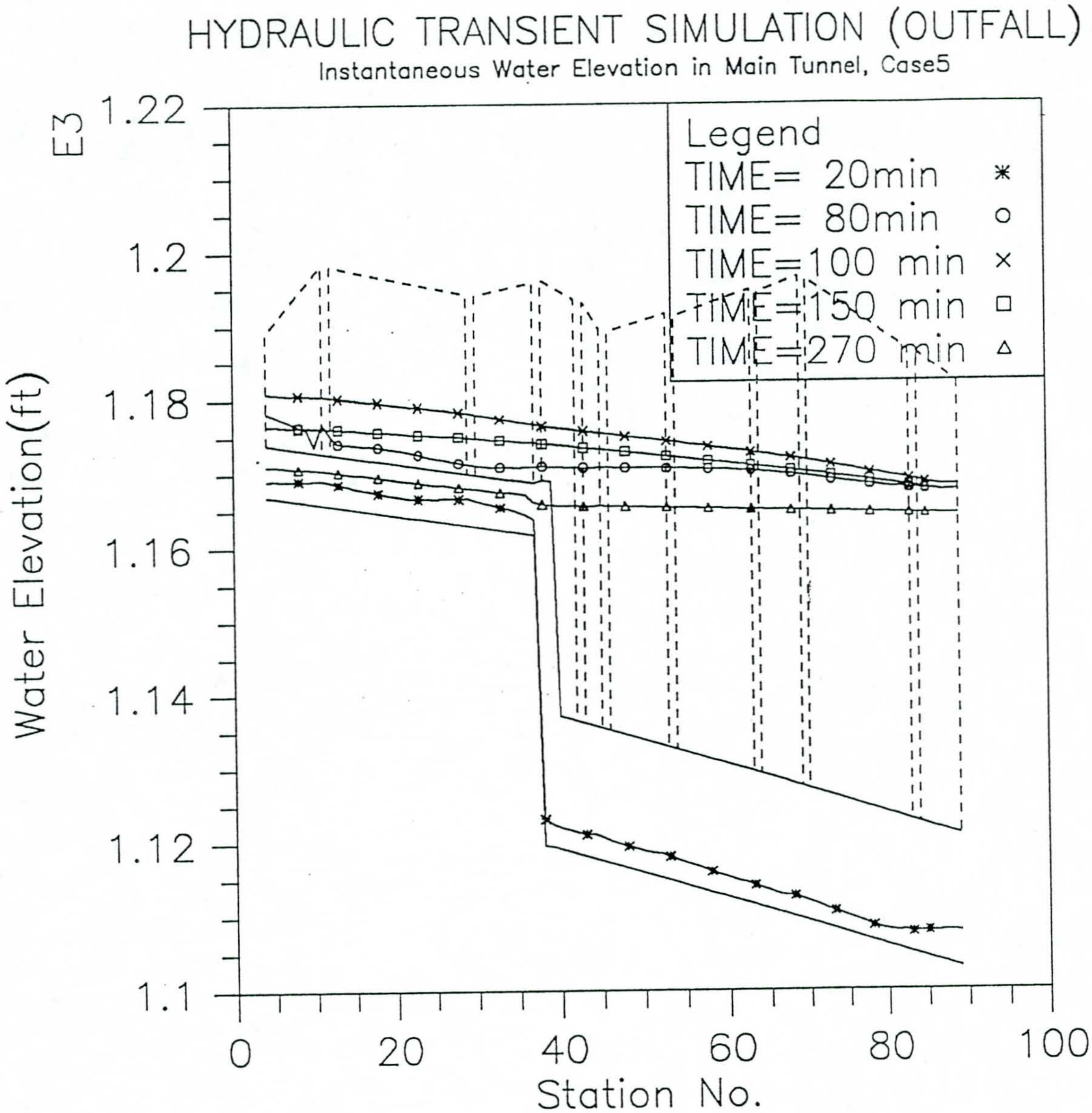


Fig. 5.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 5).

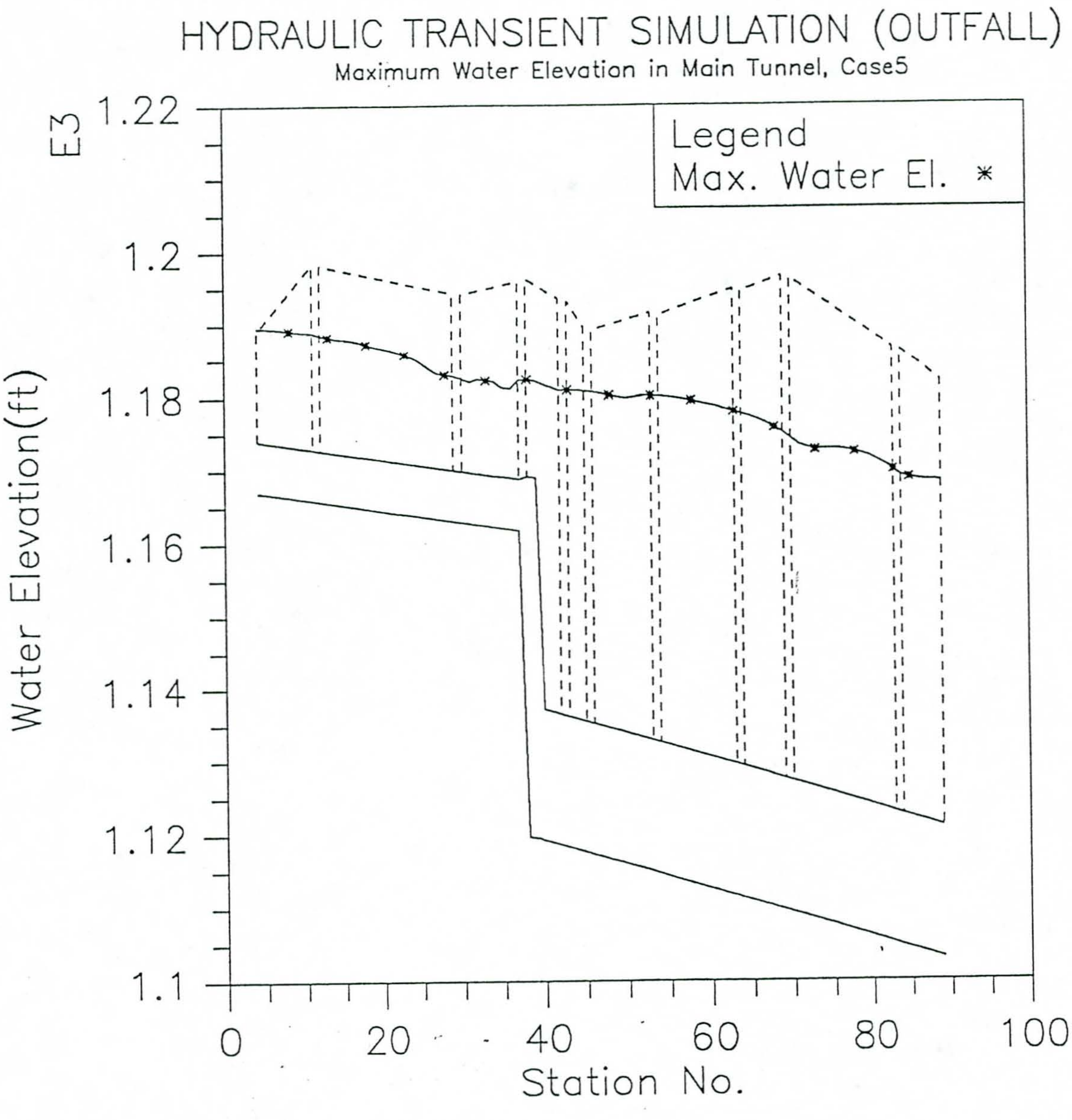


Fig. 5.4 The maximum water surface elevations along the tunnel system, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 5).

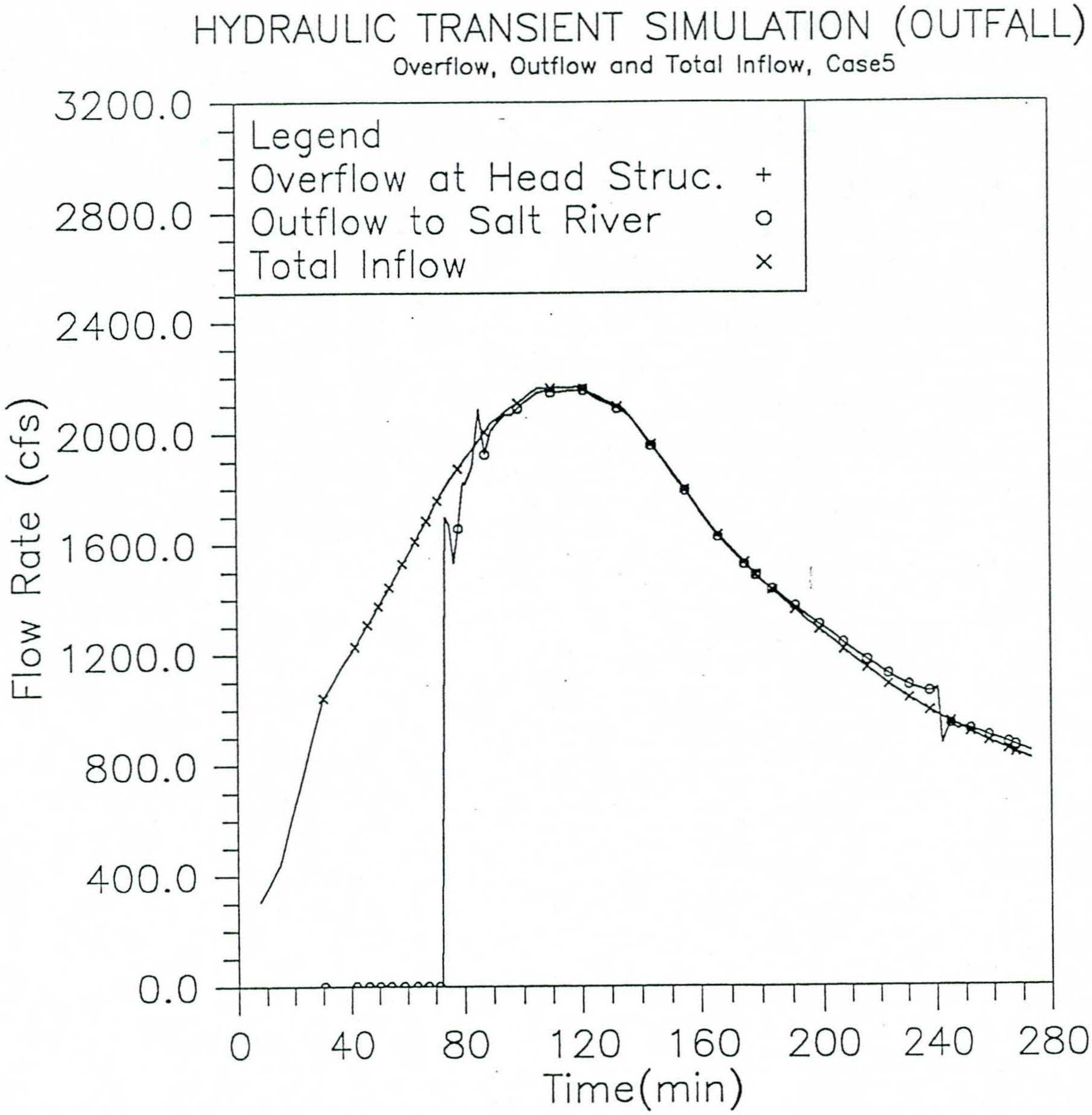


Fig. 5.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 5).

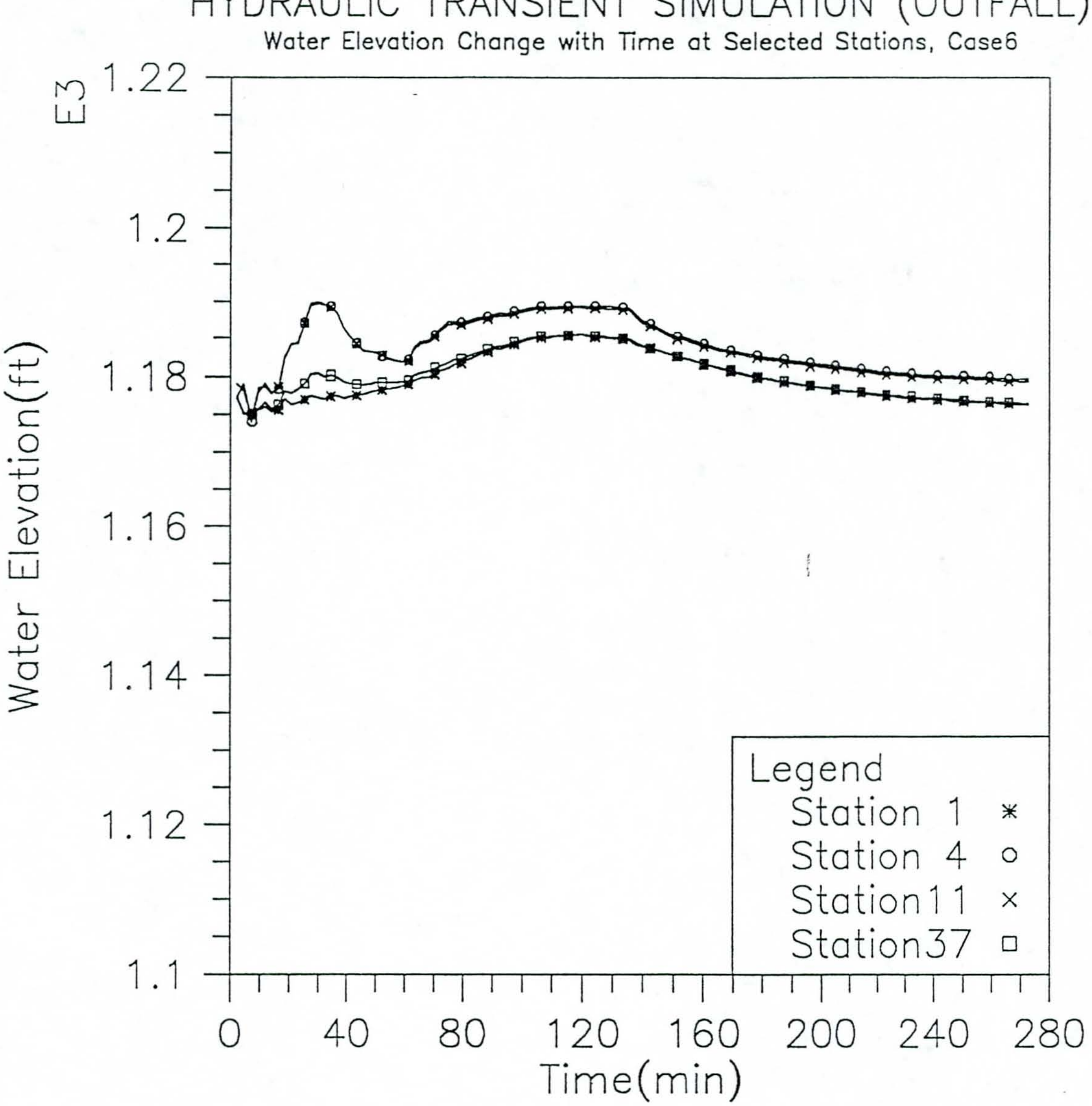


Fig. 6.1 Time variation of water surface elevations at four upstream locations, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 6).

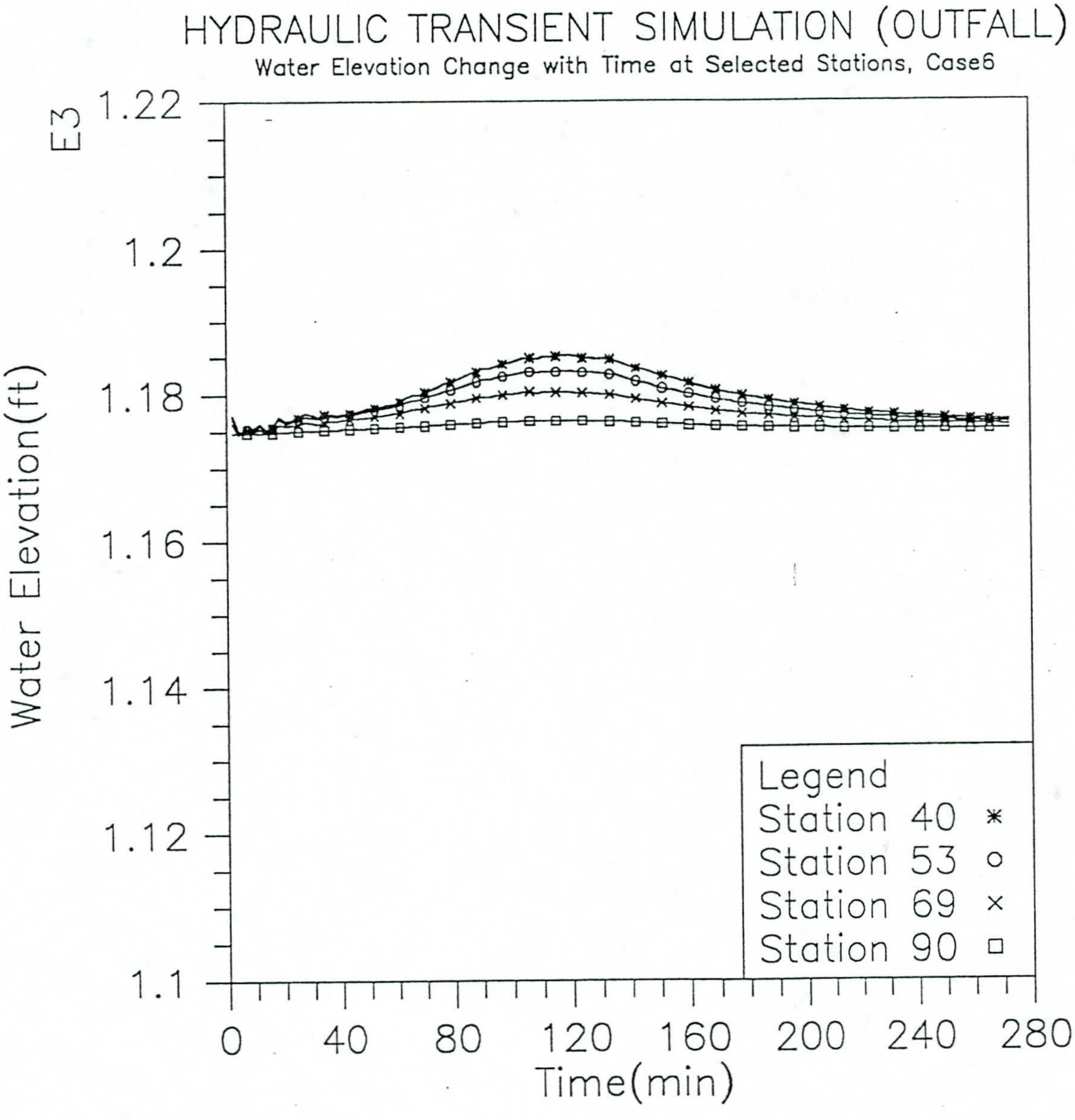


Fig. 6.2 Time variation of water surface elevations at four downstream locations, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 6).

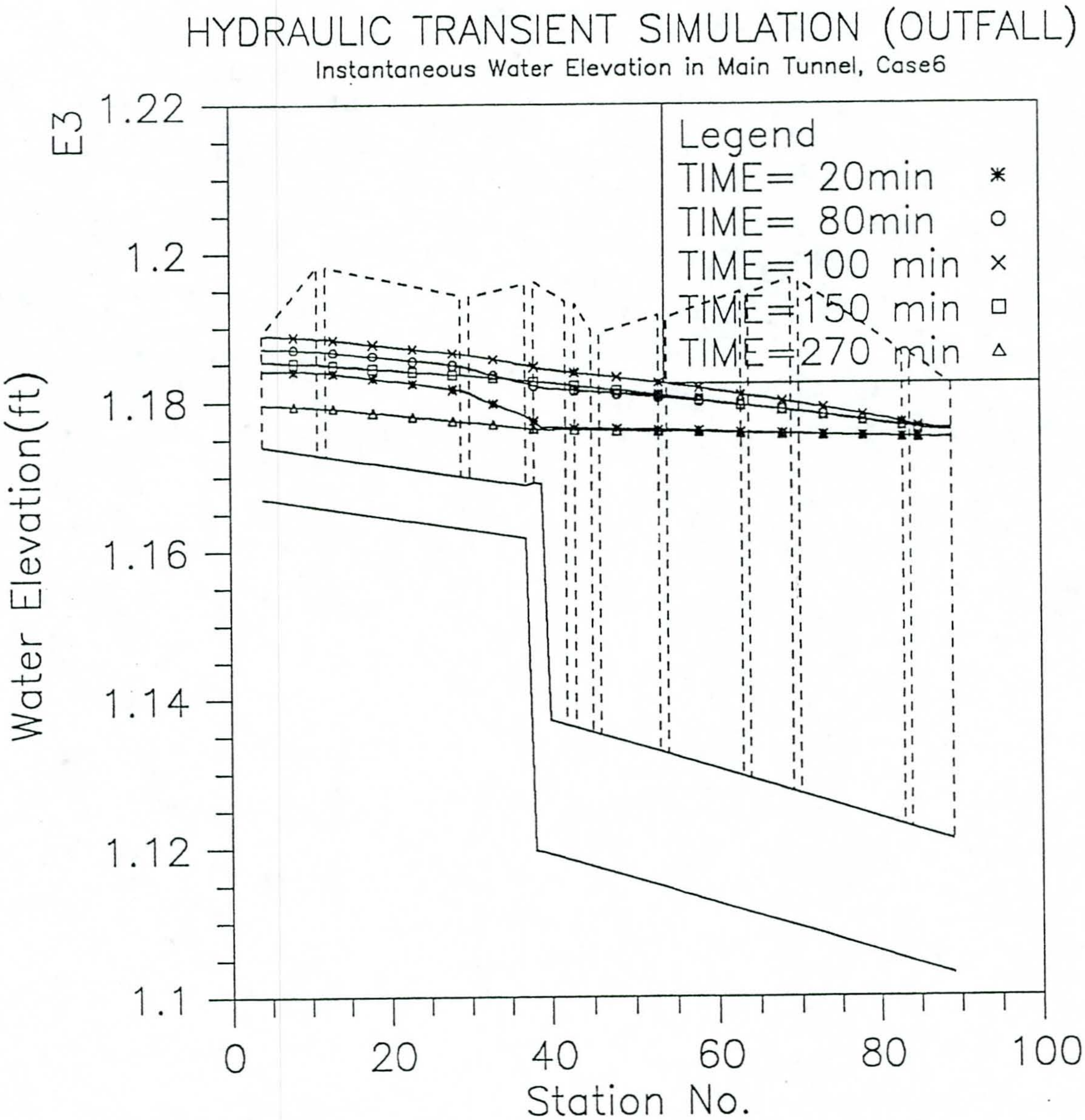


Fig. 6.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 6).

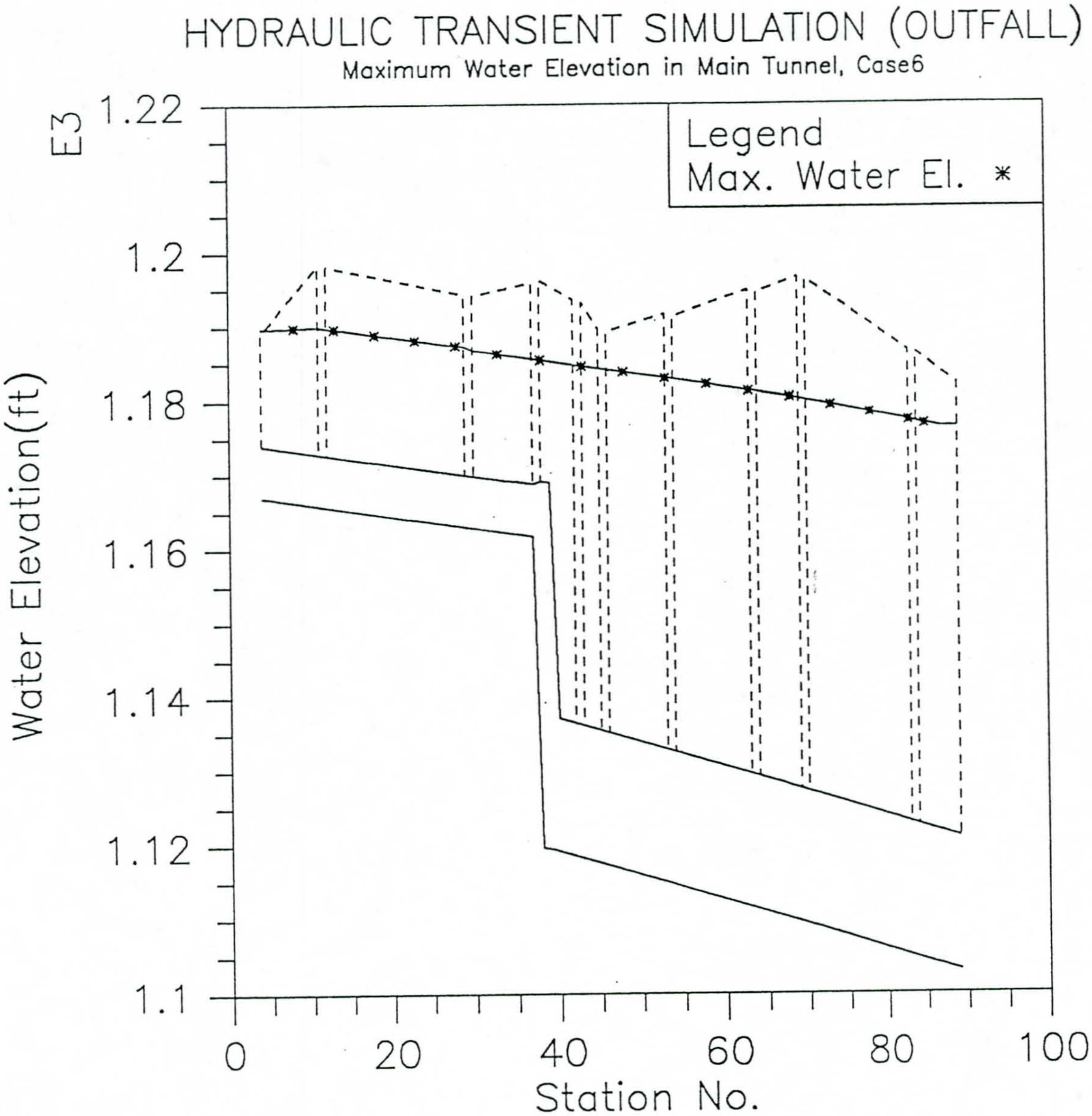


Fig. 6.4 The maximum water surface elevations along the tunnel system, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 6).

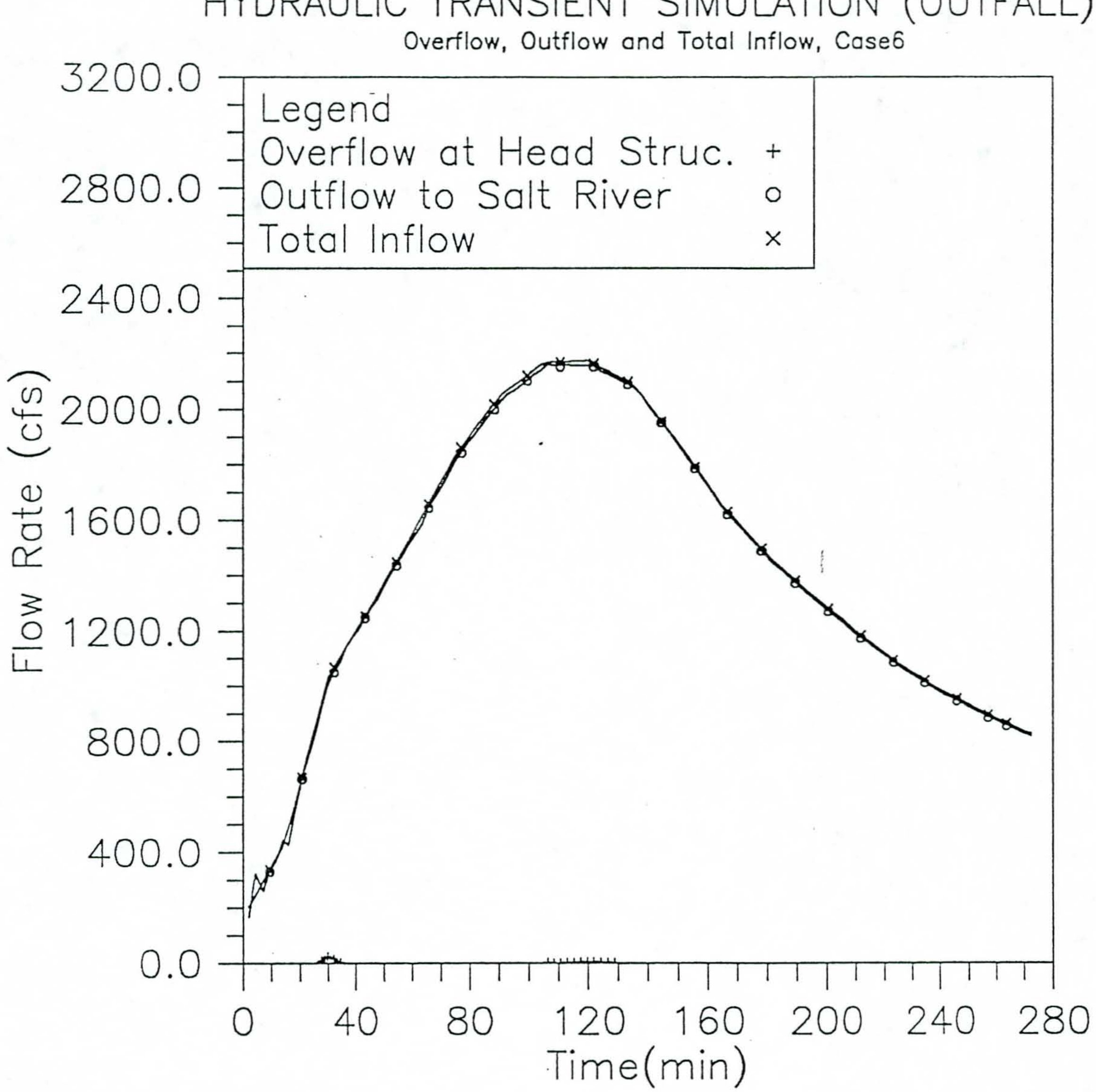


Fig. 6.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 7 ft diameter, 'dry' tunnel, and 3 ft rise of pipe invert (Case 6).

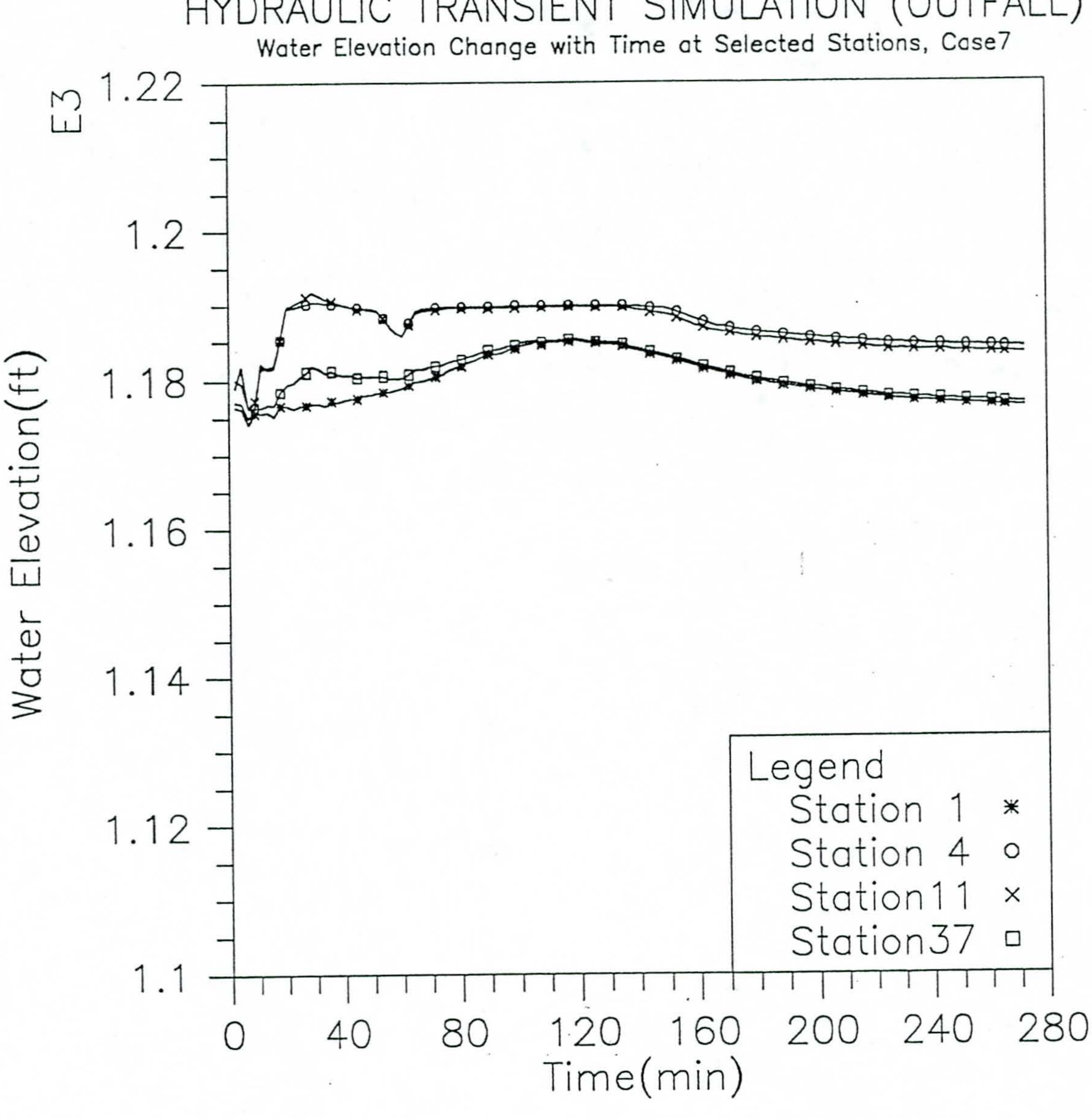


Fig. 7.1 Time variation of water surface elevations at four upstream locations, modeling case: 6 ft diameter and 'wet' tunnel (Case 7).

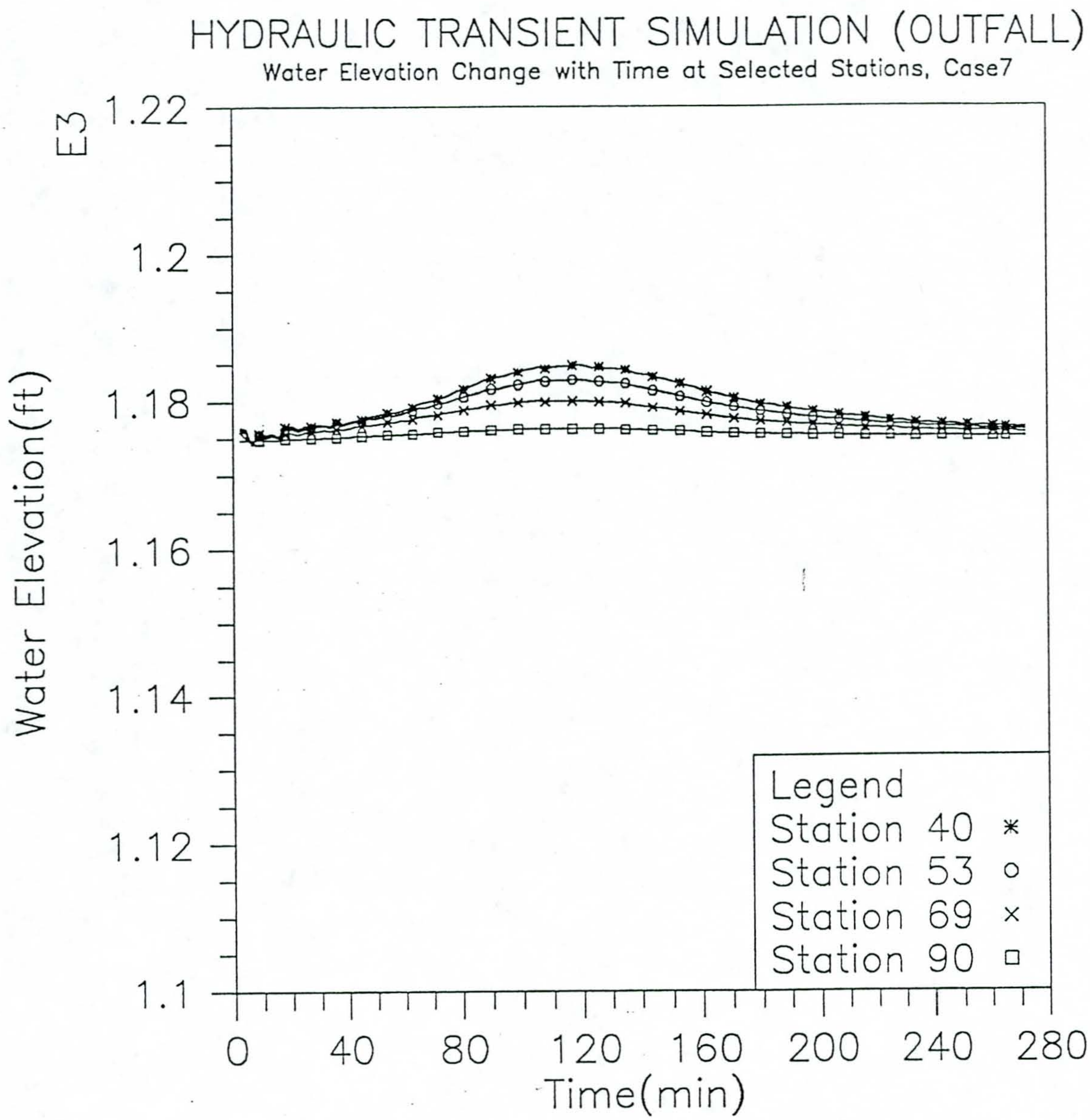


Fig. 7.2 Time variation of water surface elevations at four downstream locations, modeling case: 6 ft diameter and 'wet' tunnel (Case 7).

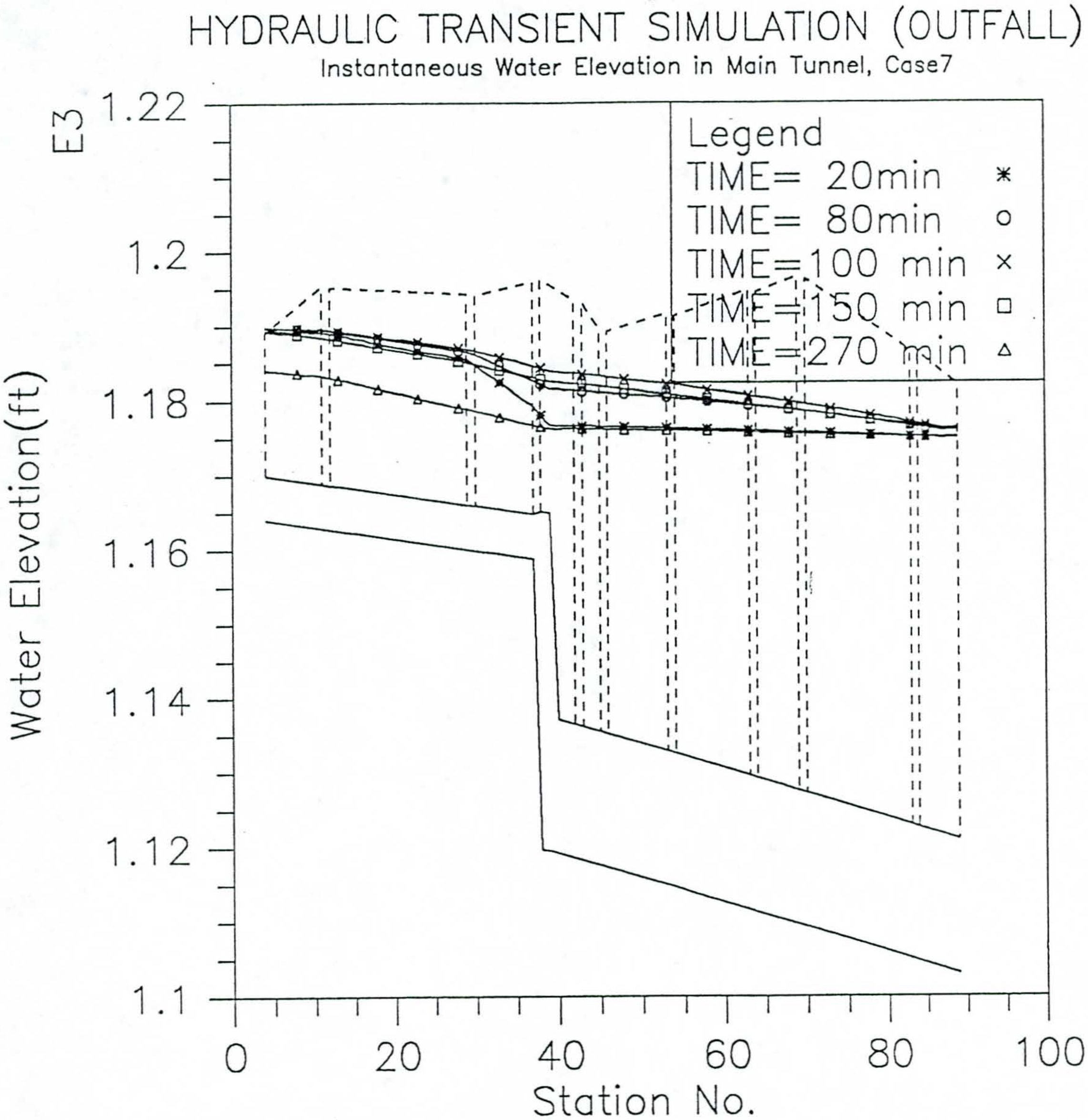


Fig. 7.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 6 ft diameter and 'wet' tunnel (Case 7).

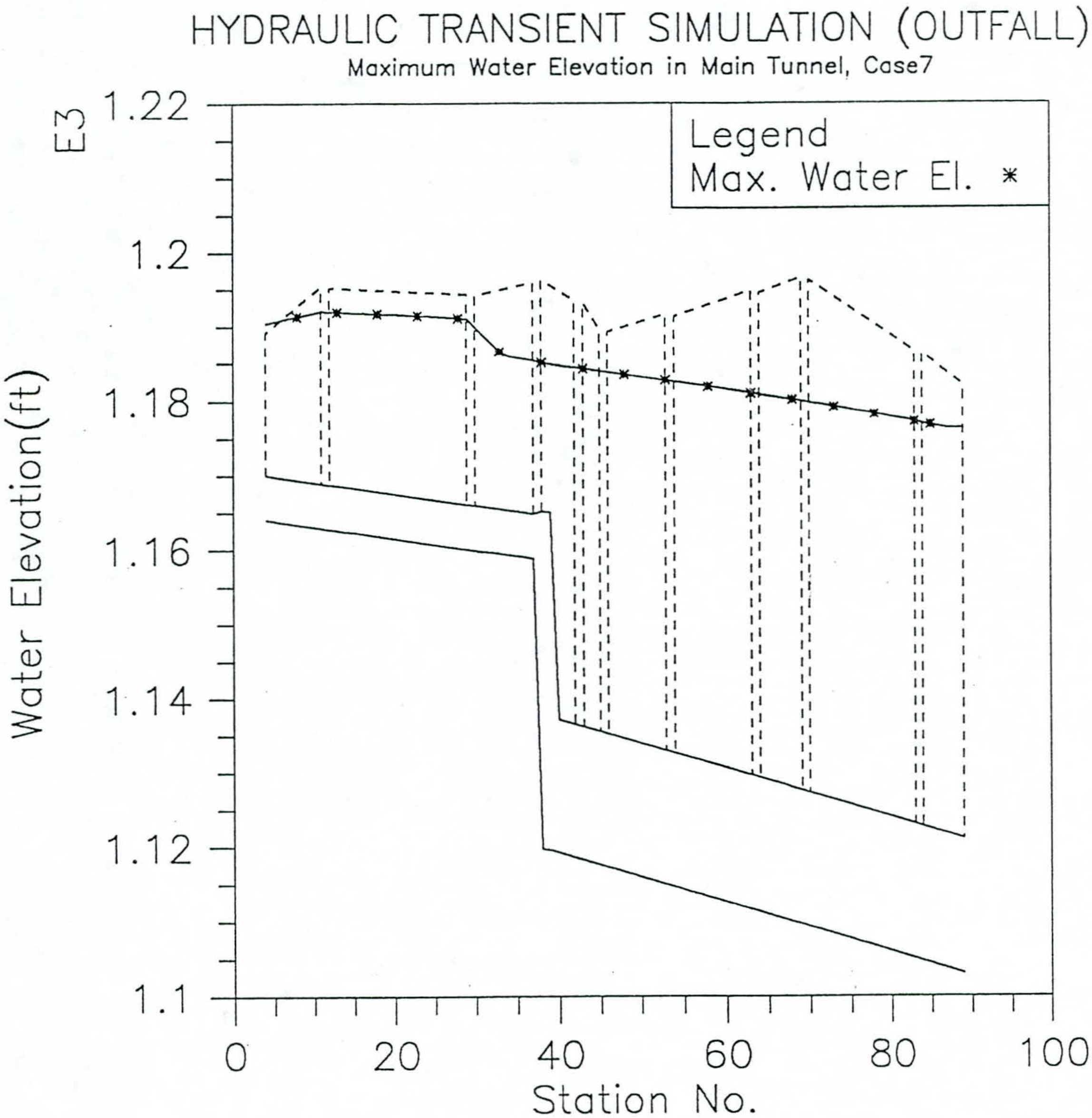


Fig. 7.4 The maximum water surface elevations along the tunnel system, modeling case: 6 ft diameter and 'wet' tunnel (Case 7).

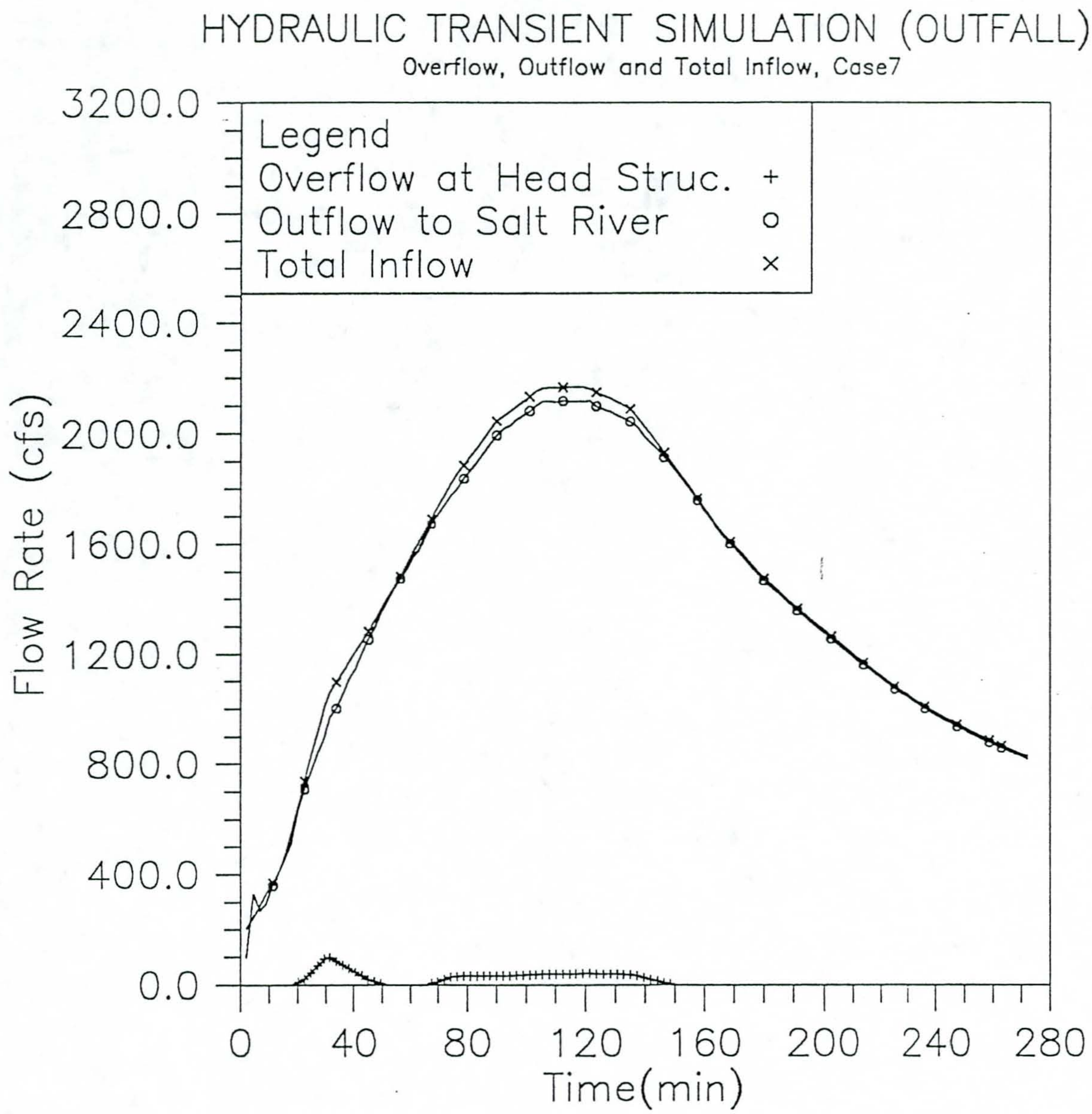


Fig. 7.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 6 ft diameter and 'wet' tunnel (Case 7).

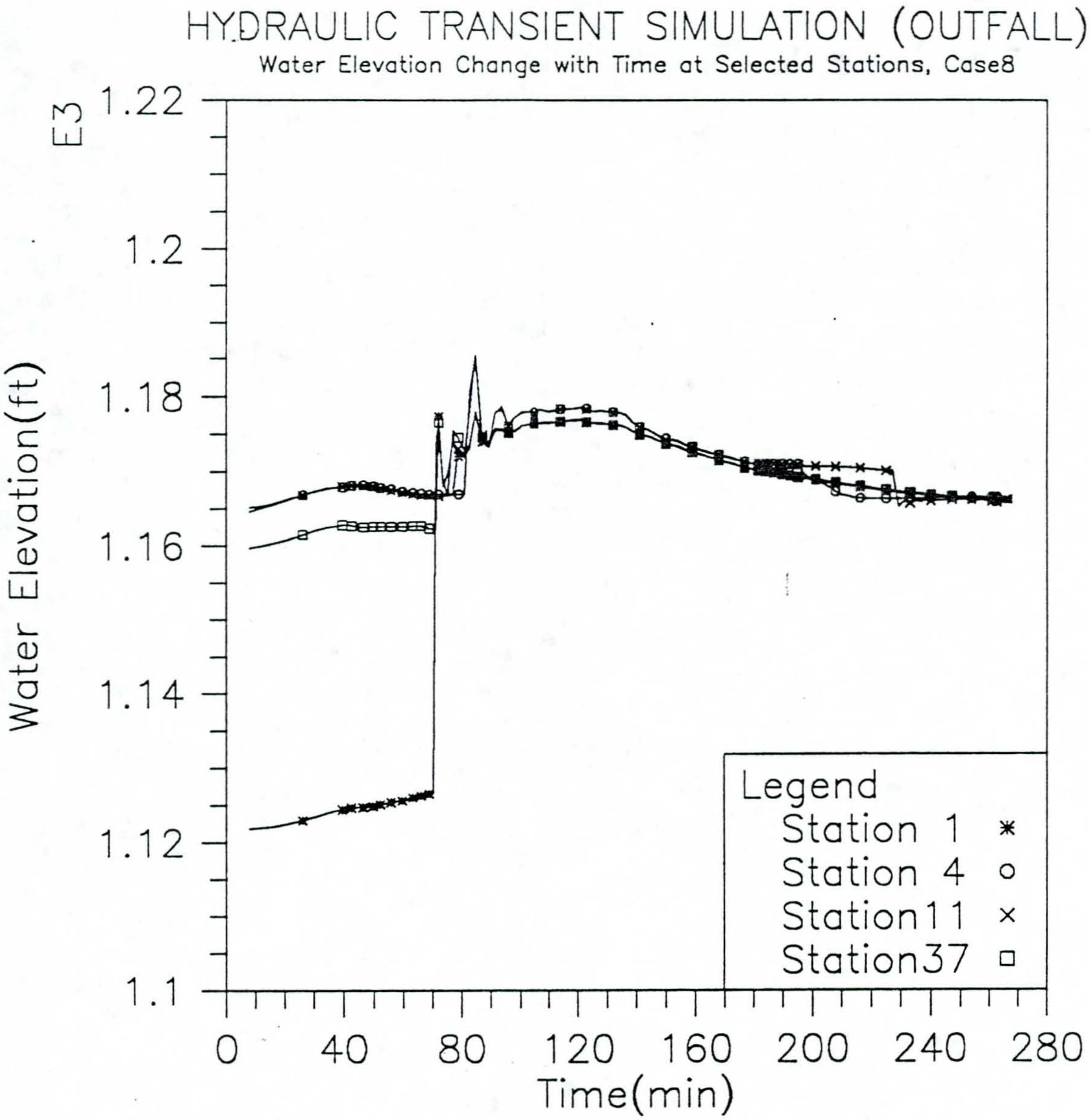


Fig. 8.1 Time variation of water surface elevations at four upstream locations, modeling case: 7 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 8).

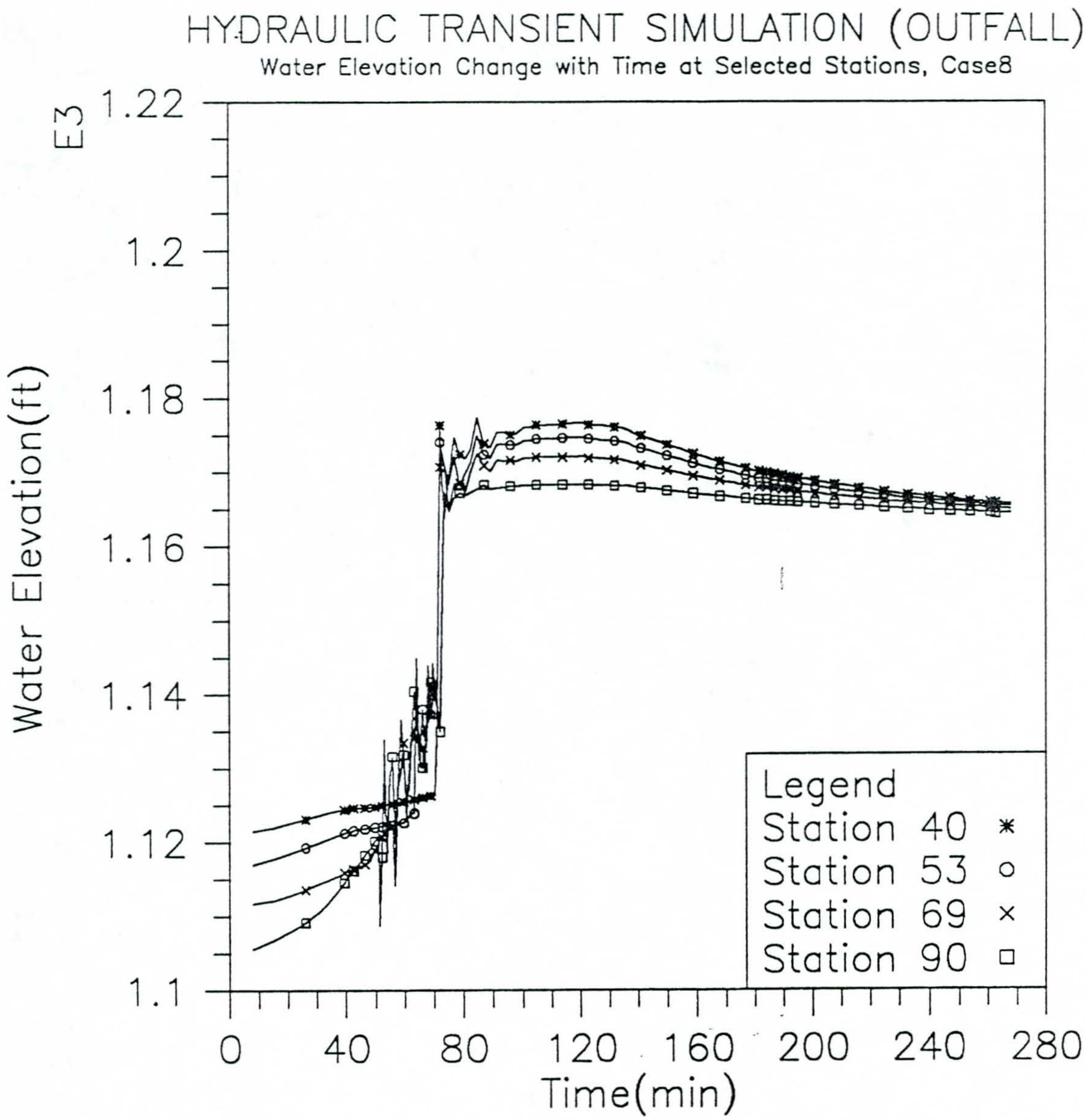


Fig. 8.2 Time variation of water surface elevations at four downstream locations, modeling case: 7 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 8).

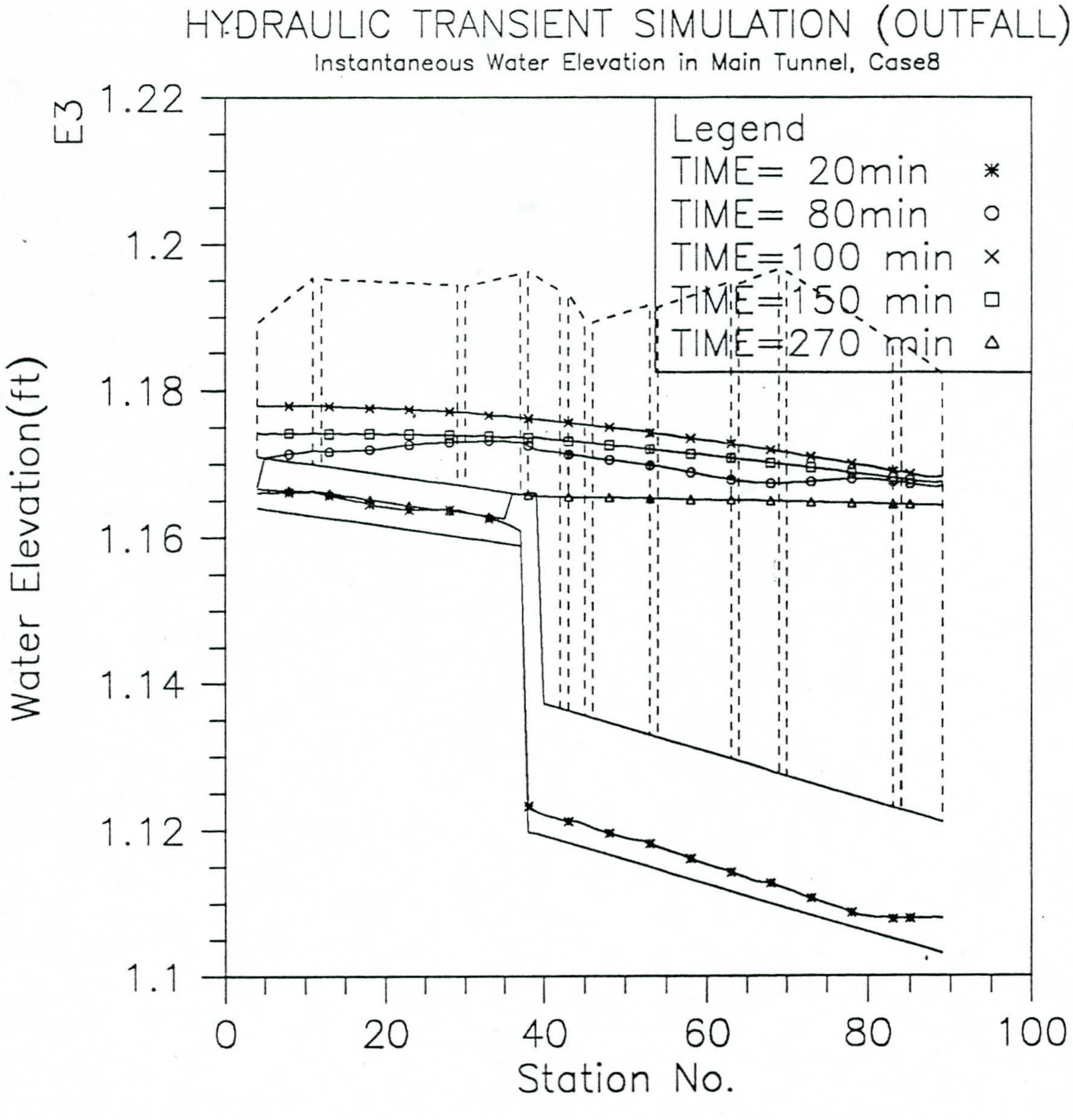


Fig. 8.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 7 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 8).

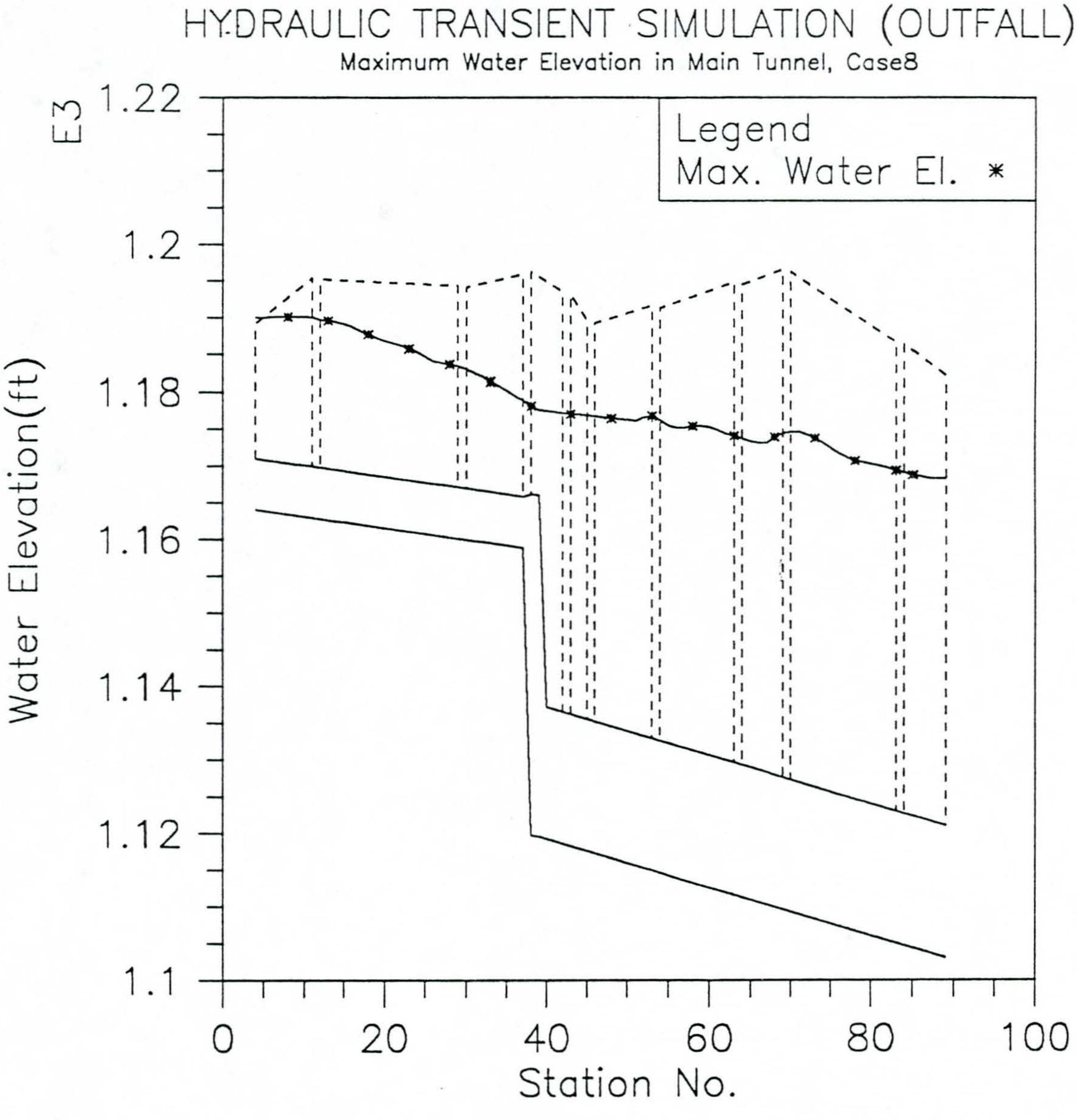


Fig. 8.4 The maximum water surface elevations along the tunnel system, modeling case: 7 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 8).

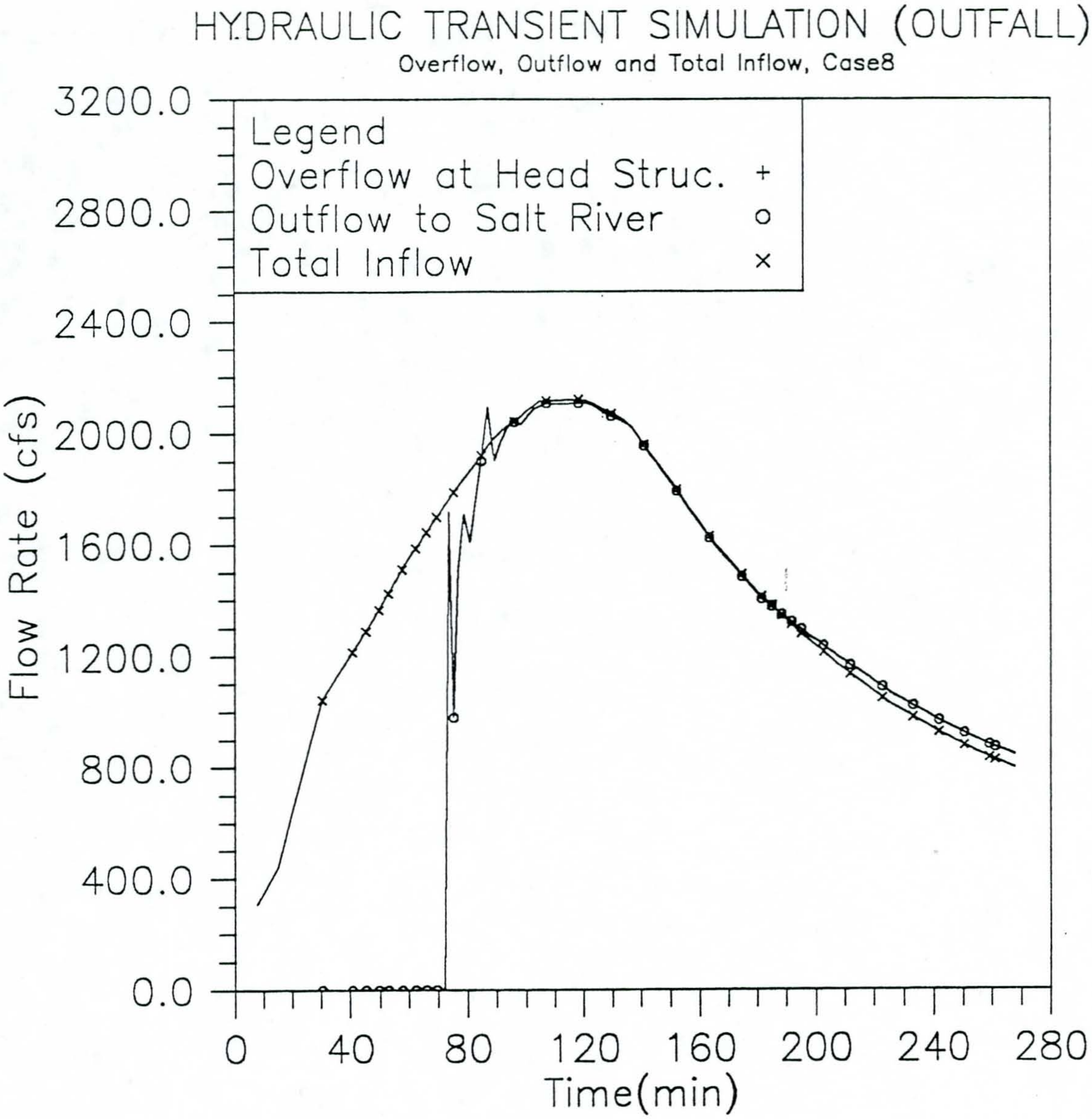


Fig. 8.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 7 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 8).

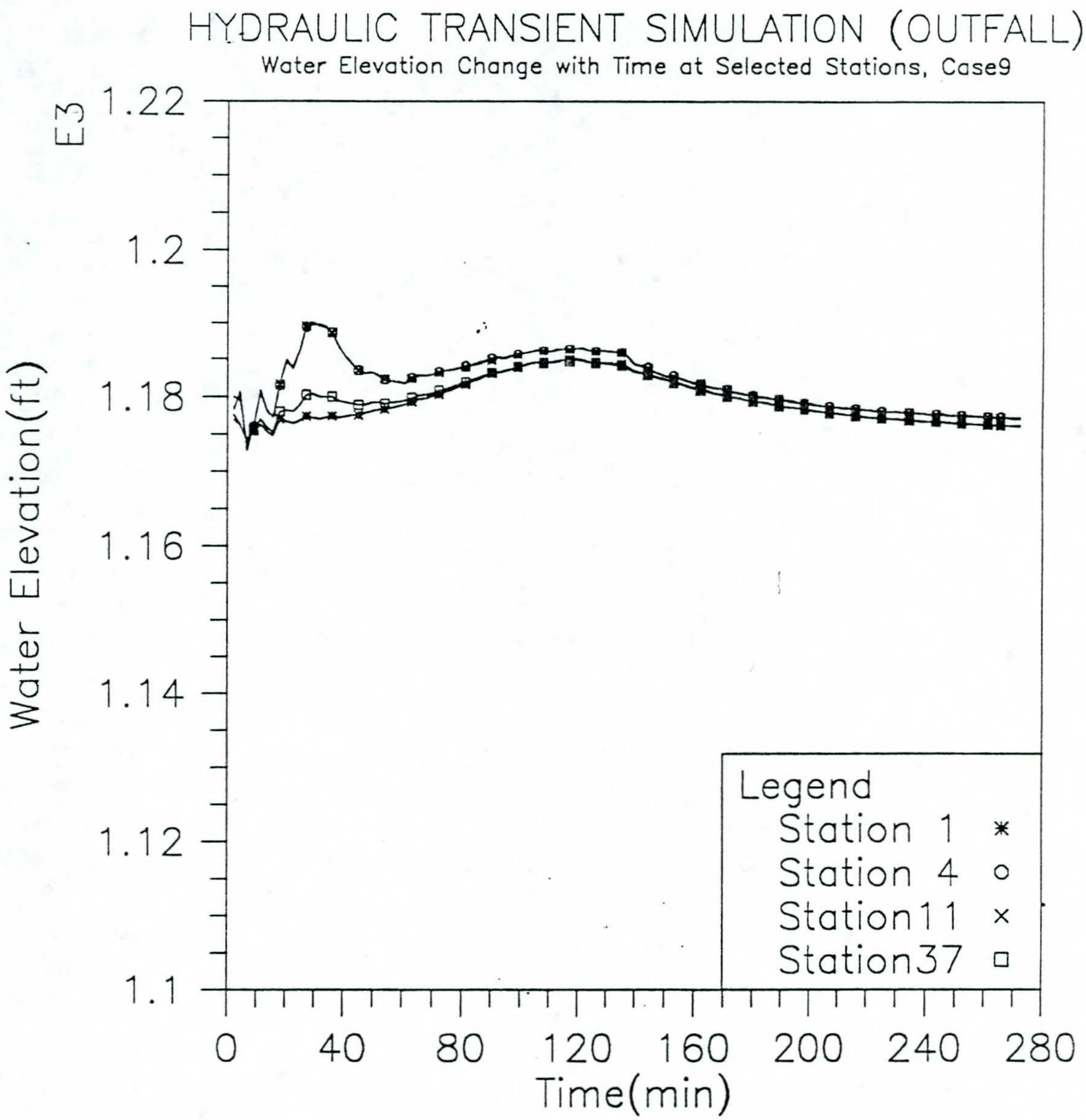


Fig. 9.1 Time variation of water surface elevations at four upstream locations, modeling case: 7 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 9).

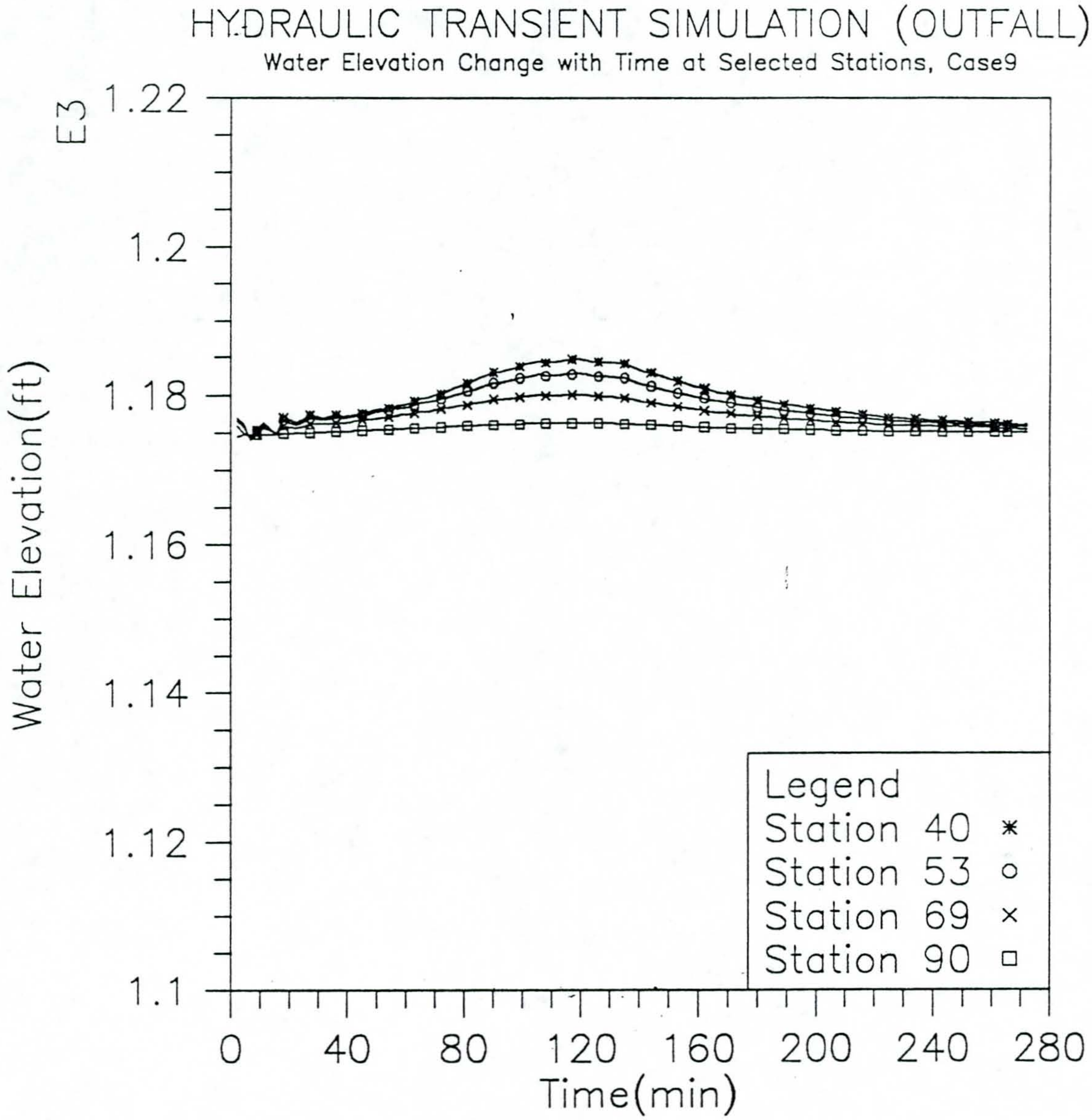


Fig. 9.2 Time variation of water surface elevations at four downstream locations, modeling case: 7 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 9).

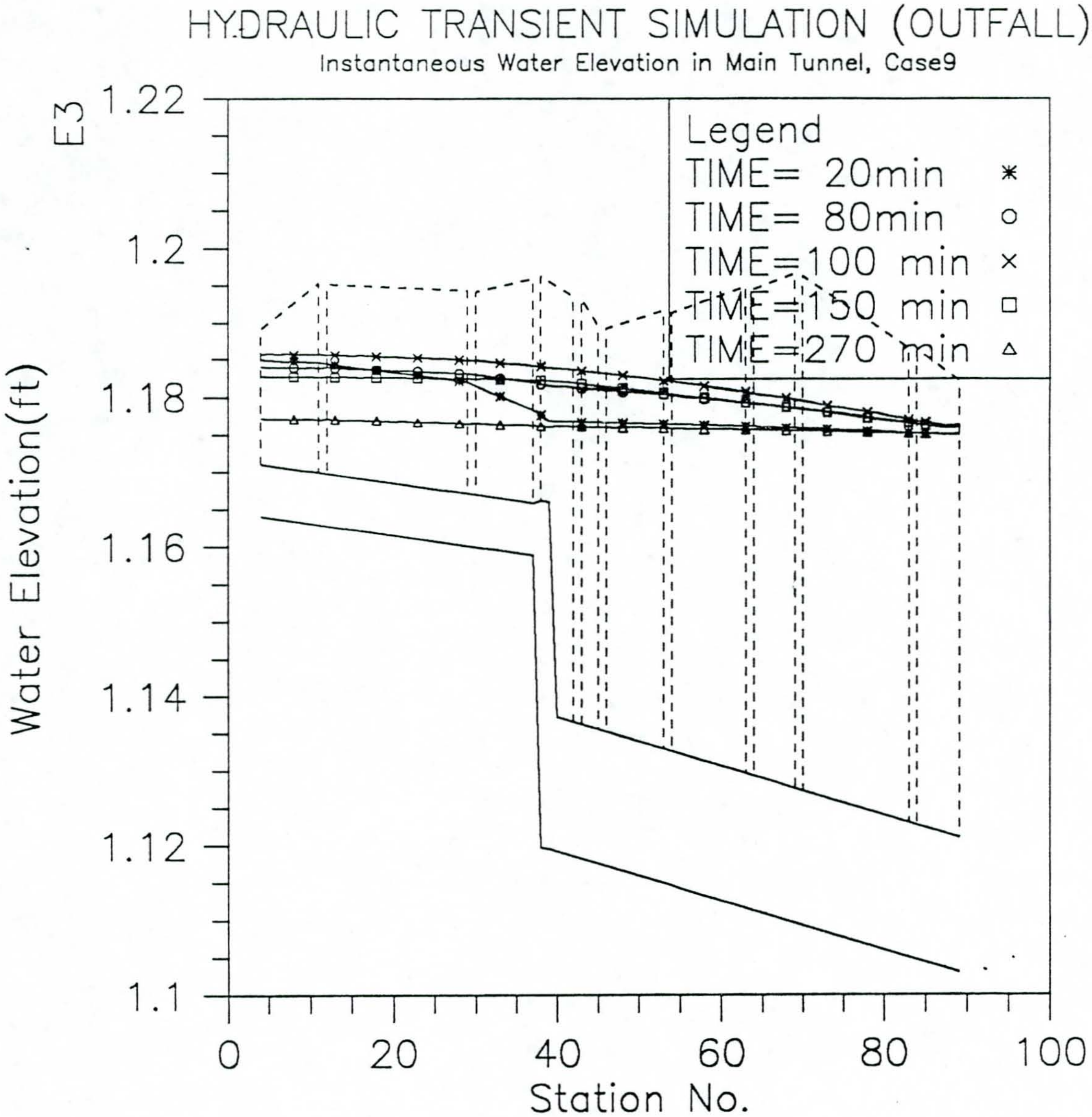


Fig. 9.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 7 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 9).

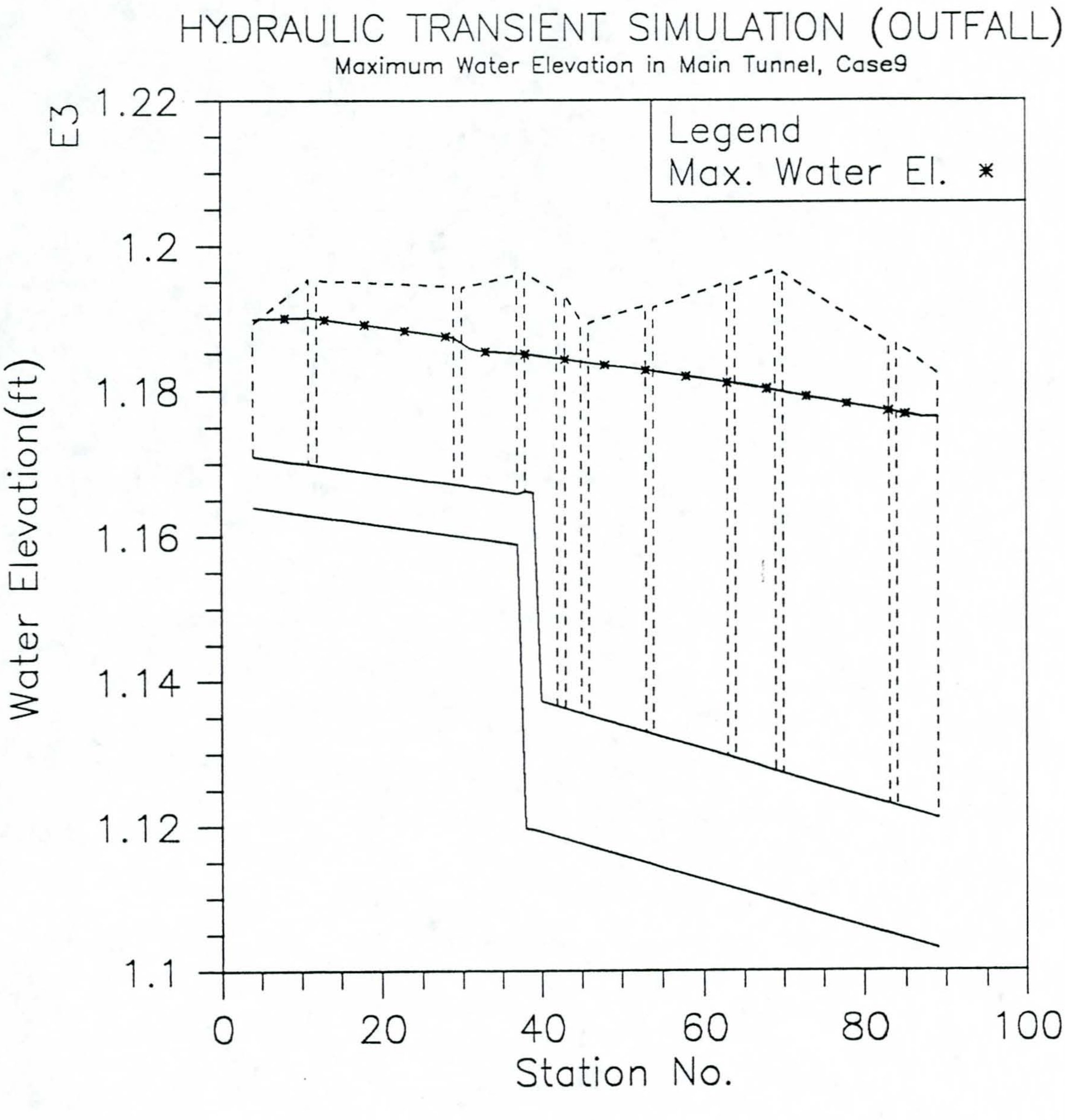


Fig. 9.4 The maximum water surface elevations along the tunnel system, modeling case: 7 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 9).

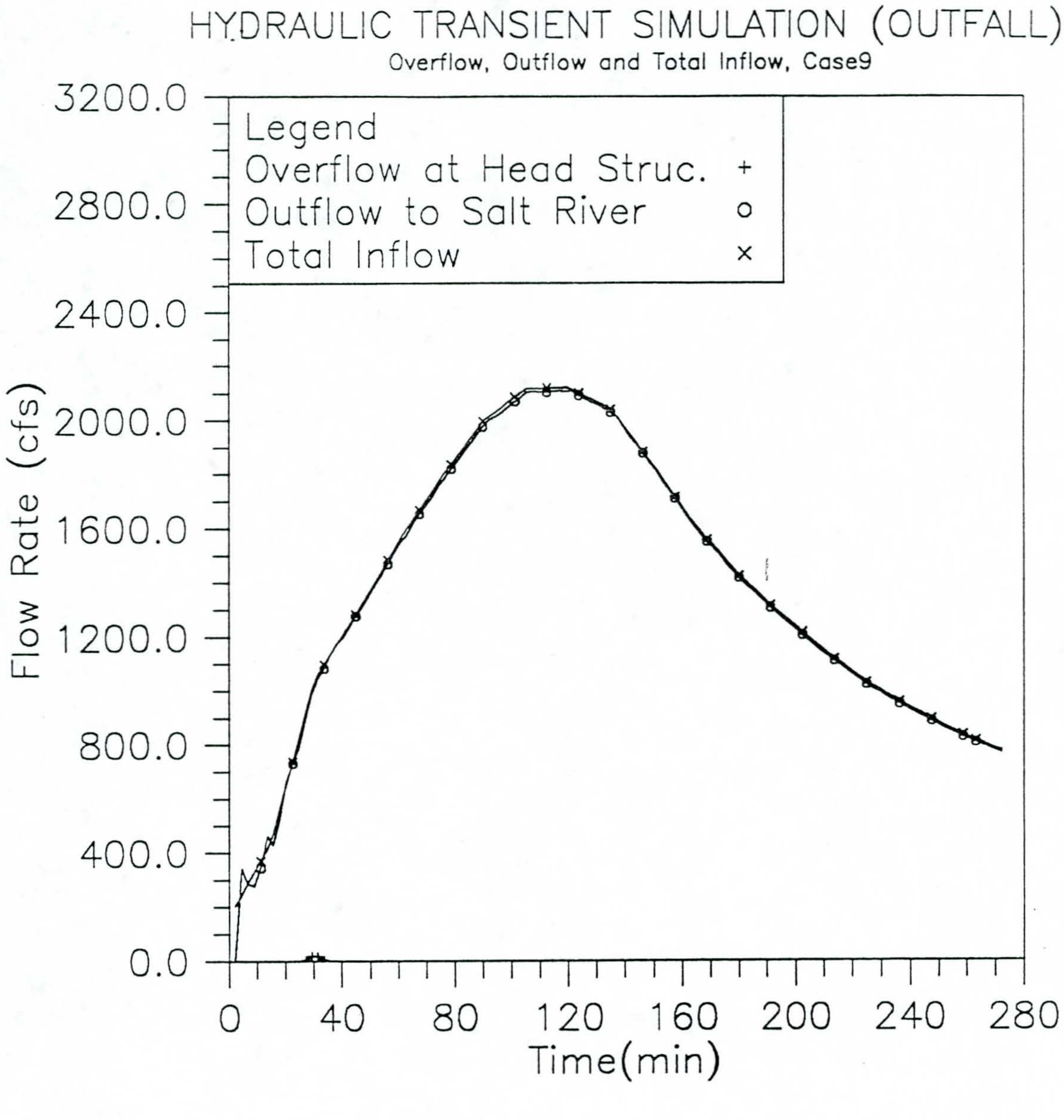


Fig. 9.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 7 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 9).

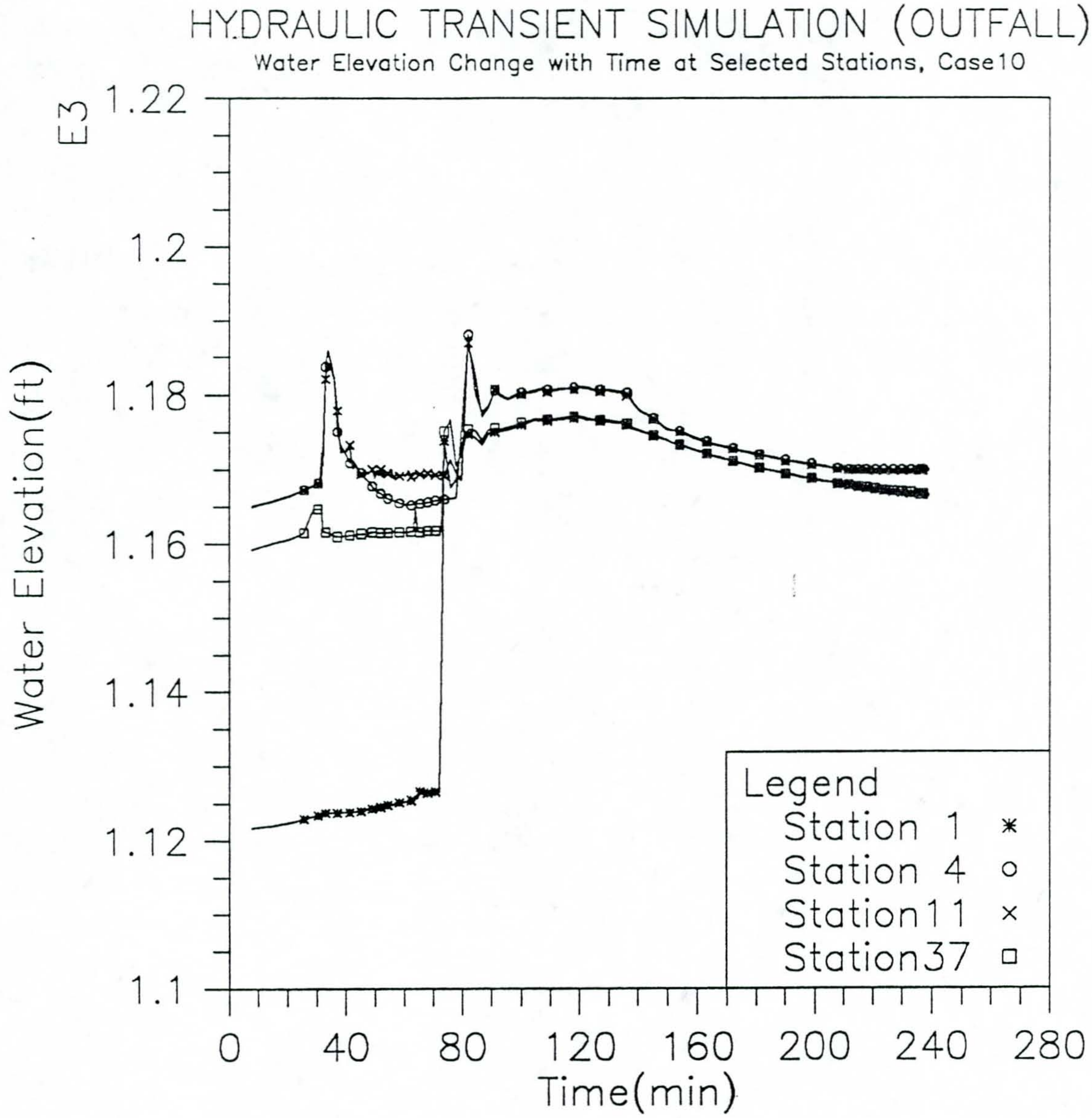


Fig. 10.1 Time variation of water surface elevations at four upstream locations, modeling case: 6 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 10).

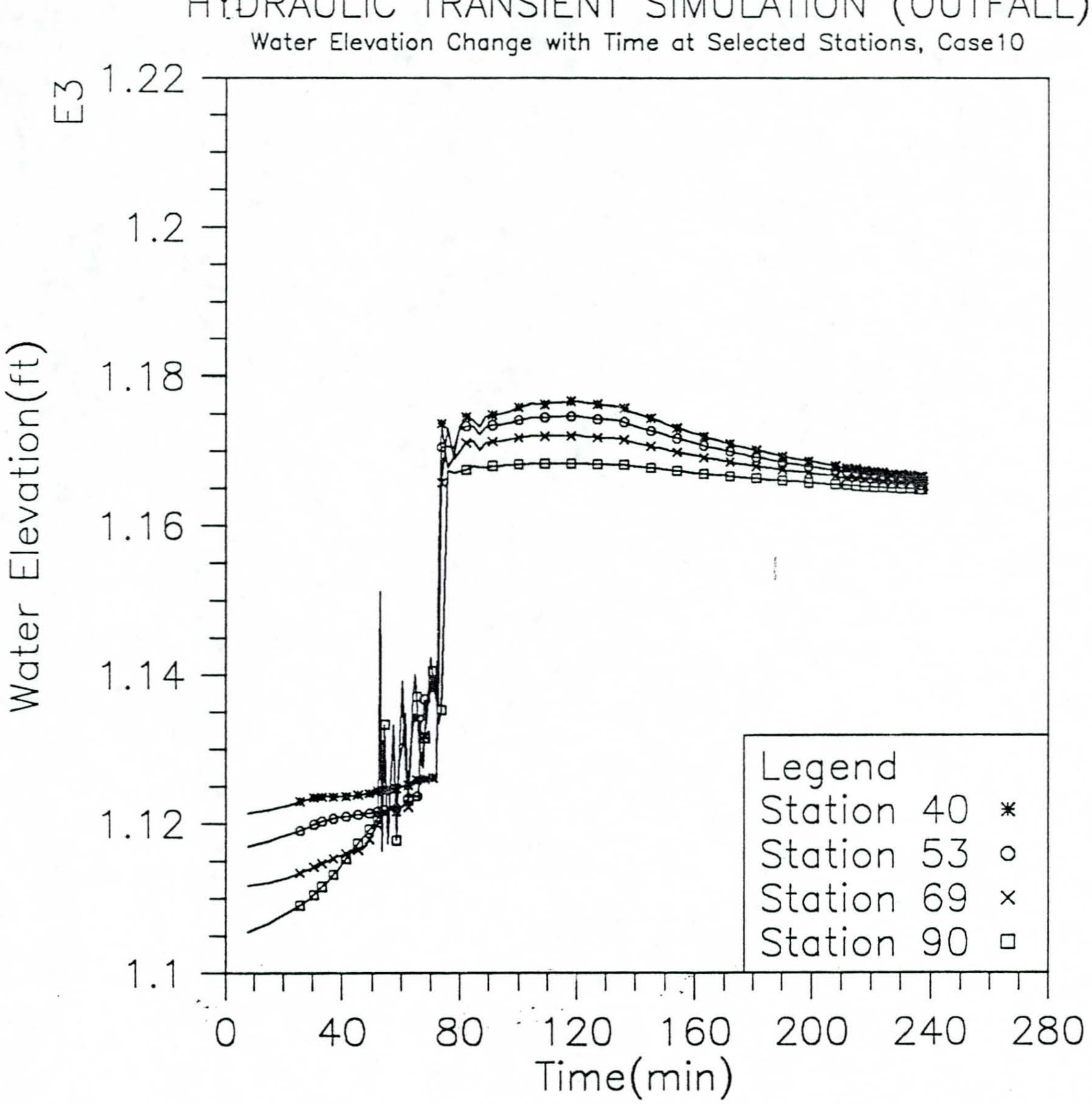


Fig. 10.2 Time variation of water surface elevations at four downstream locations, modeling case: 6 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 10).

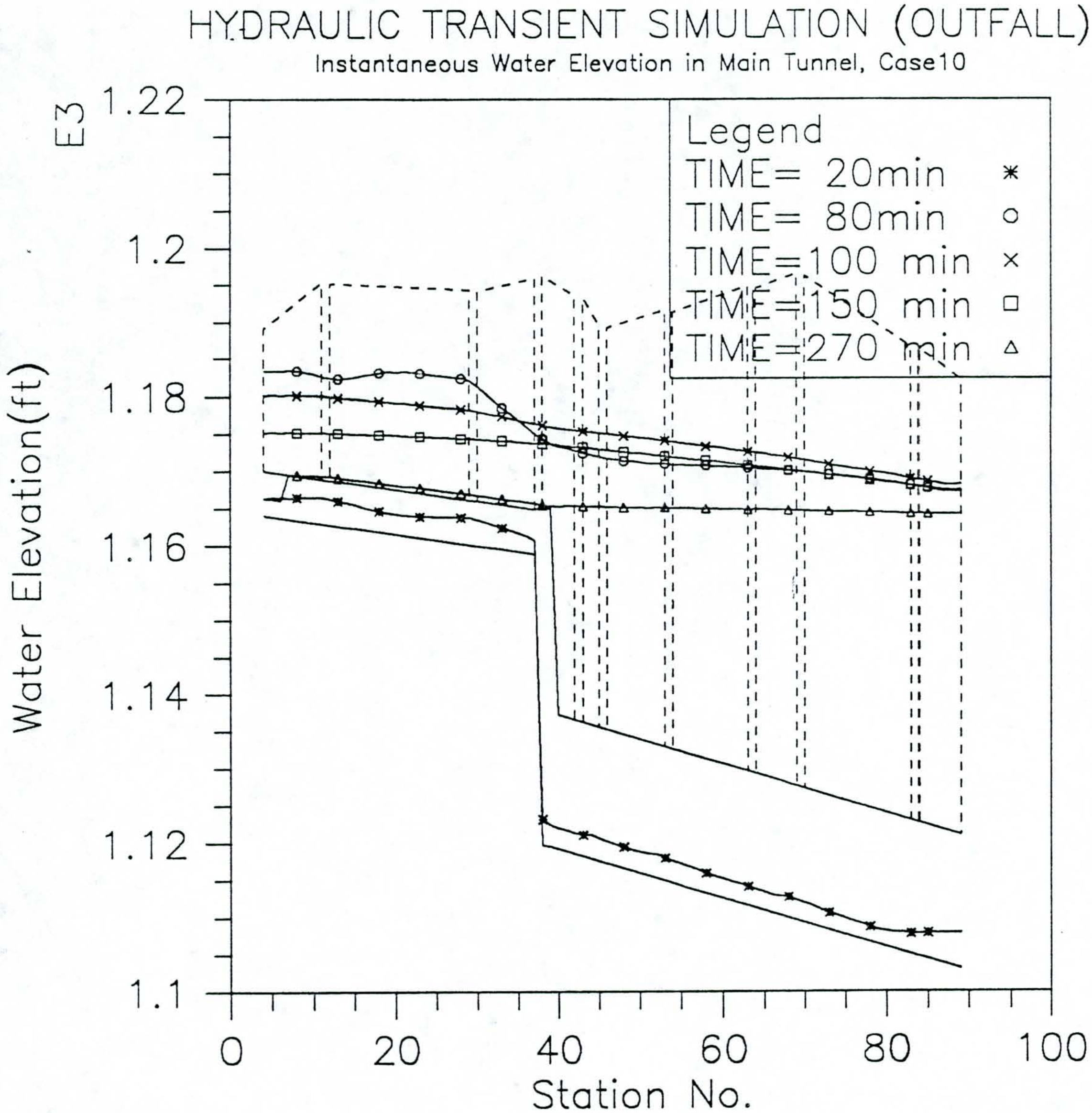


Fig. 10.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 6 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 10).

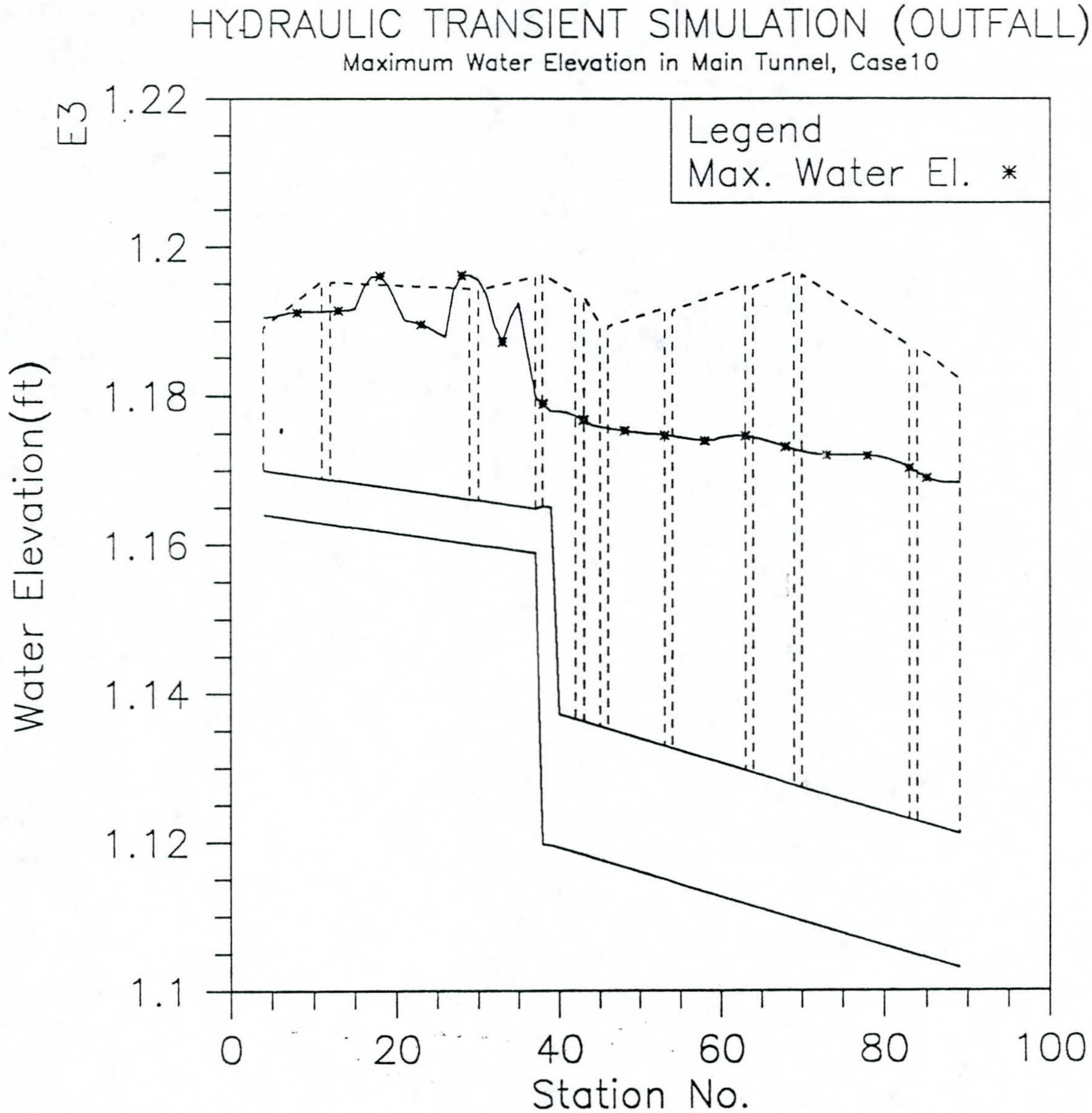


Fig. 10.4 The maximum water surface elevations along the tunnel system, modeling case: 6 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 10).

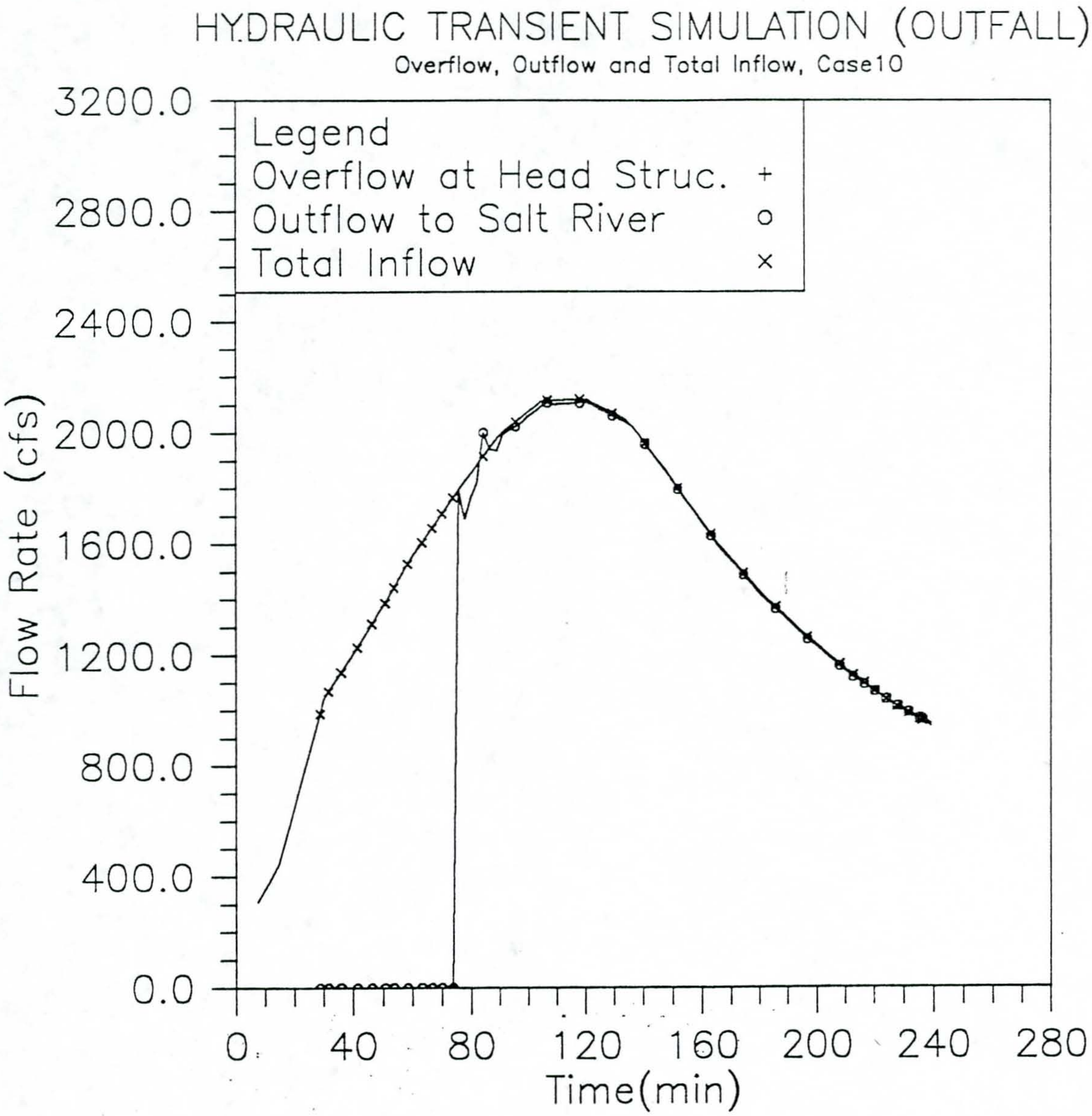


Fig. 10.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 6 ft diameter, 'dry' tunnel, and 30 cfs inflow at the head structure (Case 10).

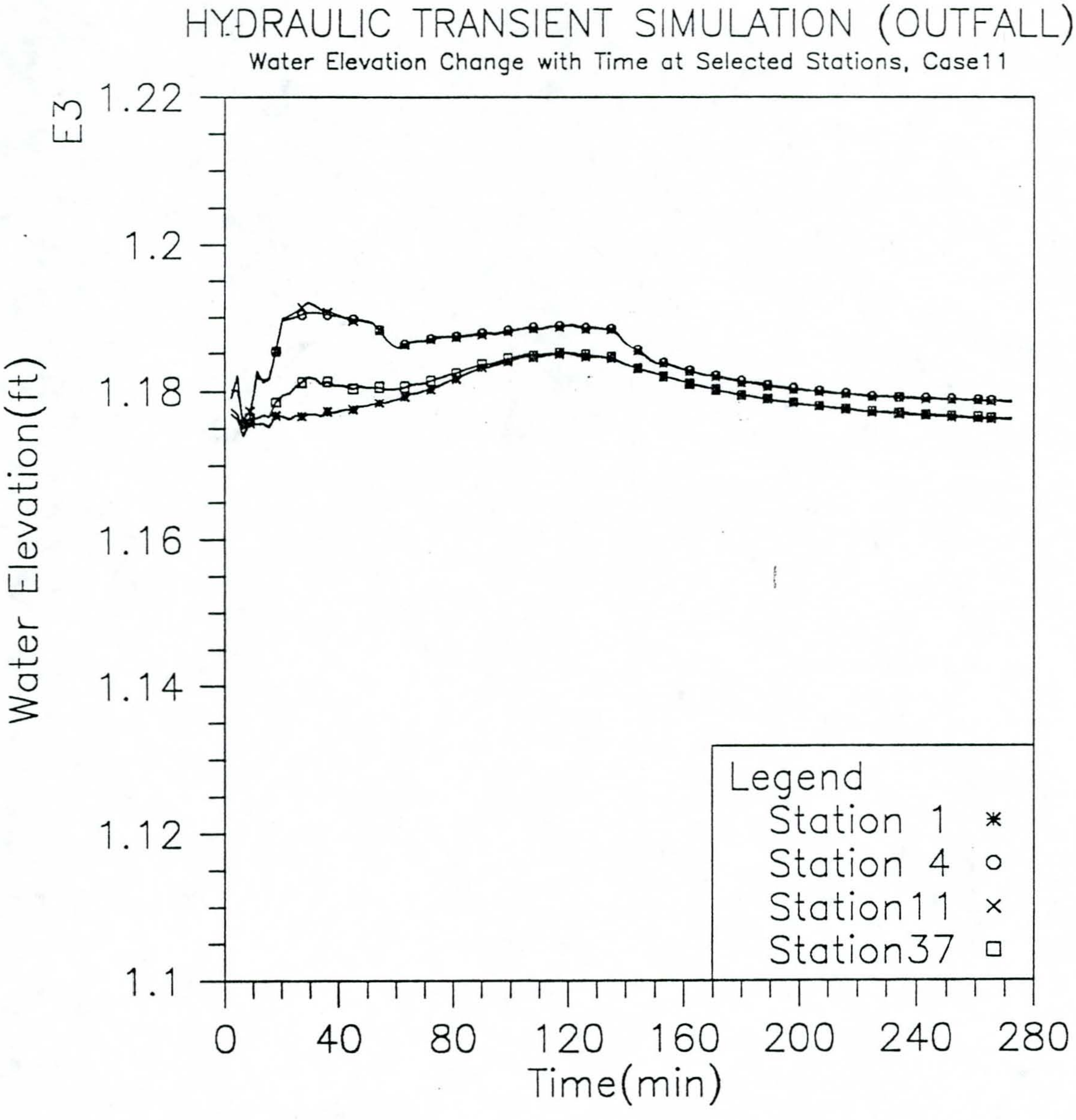


Fig. 11.1 Time variation of water surface elevations at four upstream locations, modeling case: 6 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 11).

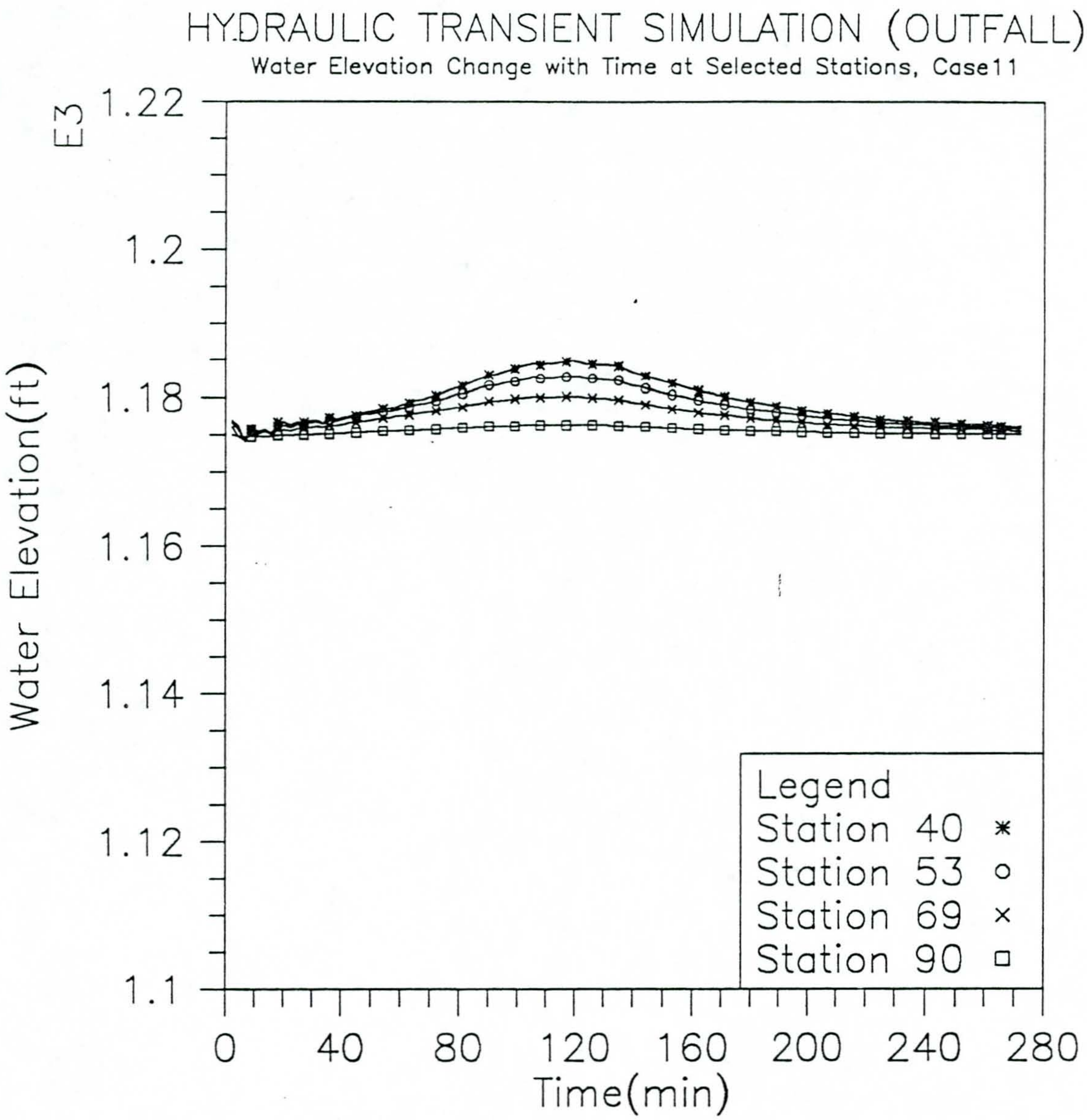


Fig. 11.2 Time variation of water surface elevations at four downstream locations, modeling case: 6 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 11).

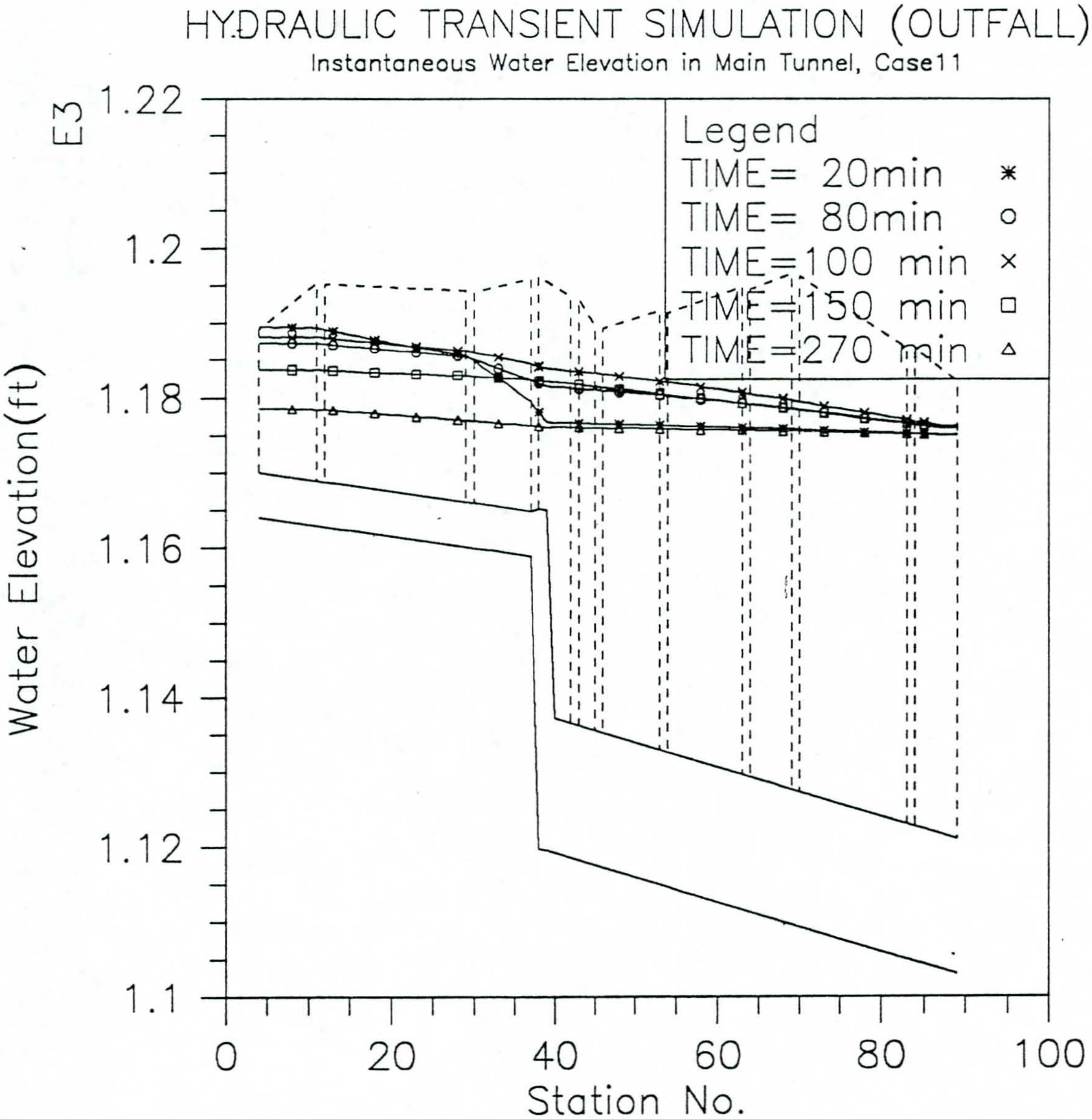


Fig. 11.3 Instantaneous hydraulic gradelines along the tunnel system, modeling case: 6 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 11).

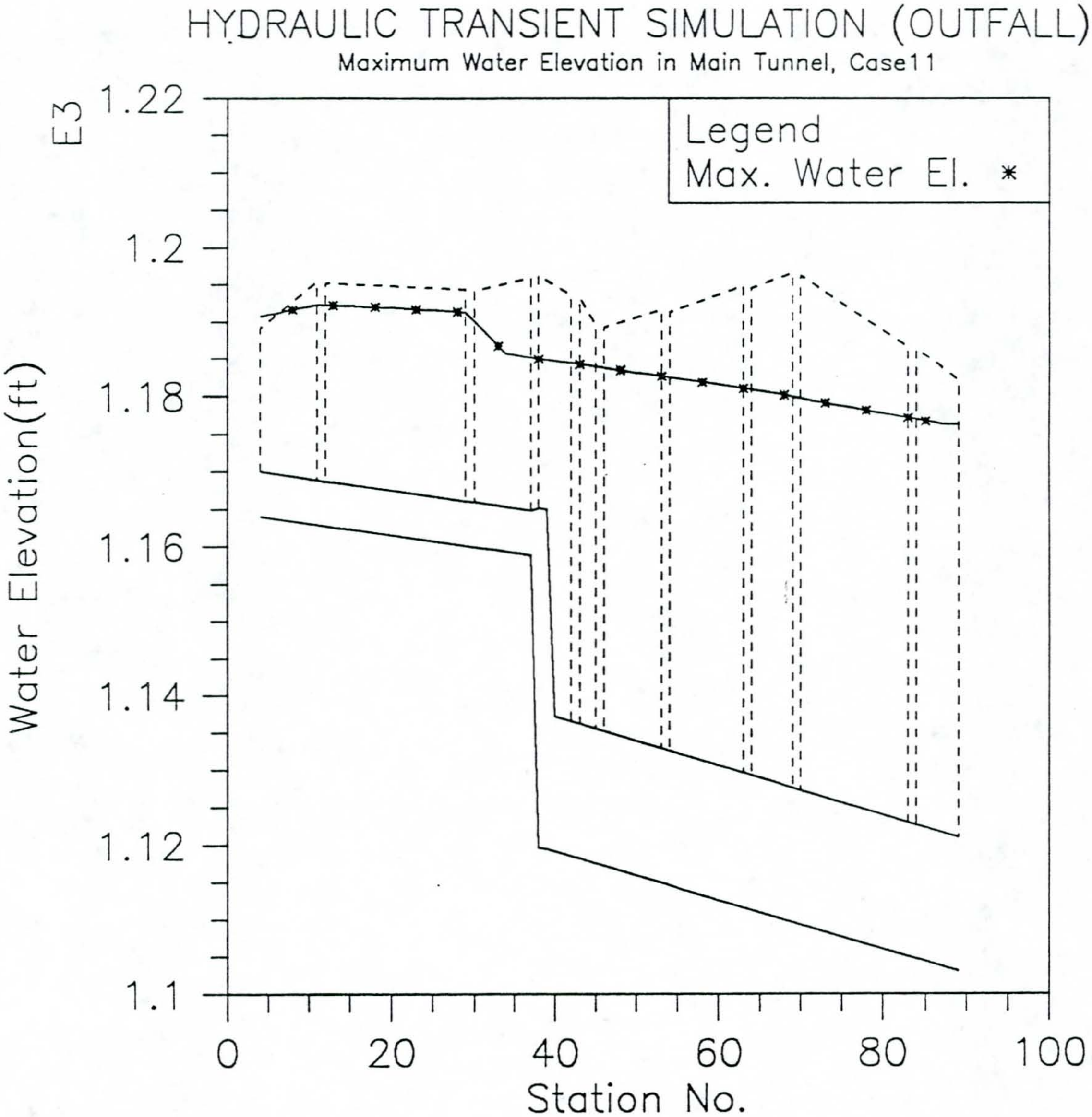


Fig. 11.4 The maximum water surface elevations along the tunnel system, modeling case: 6 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 11).

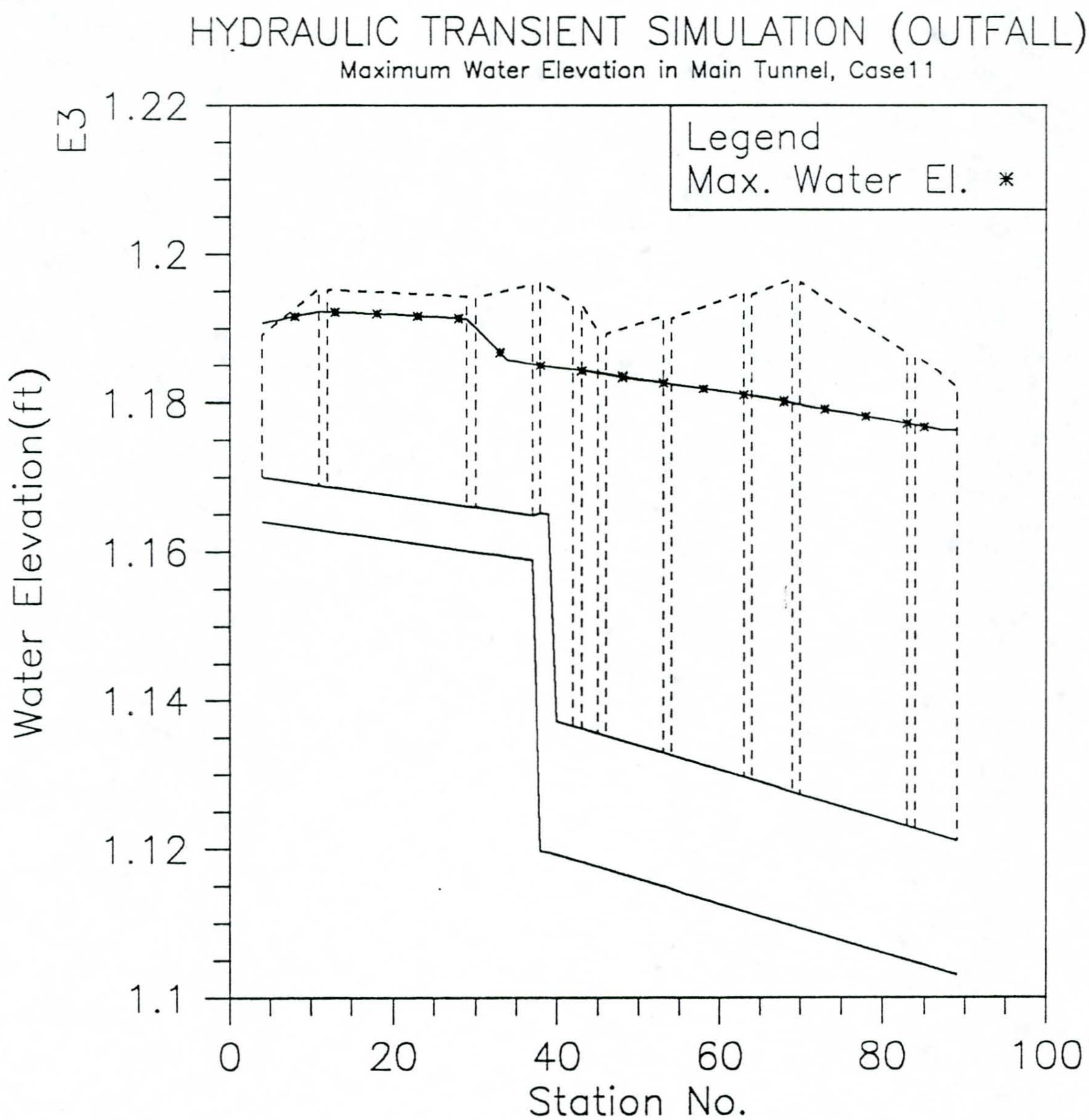


Fig. 11.4 The maximum water surface elevations along the tunnel system, modeling case: 6 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 11).

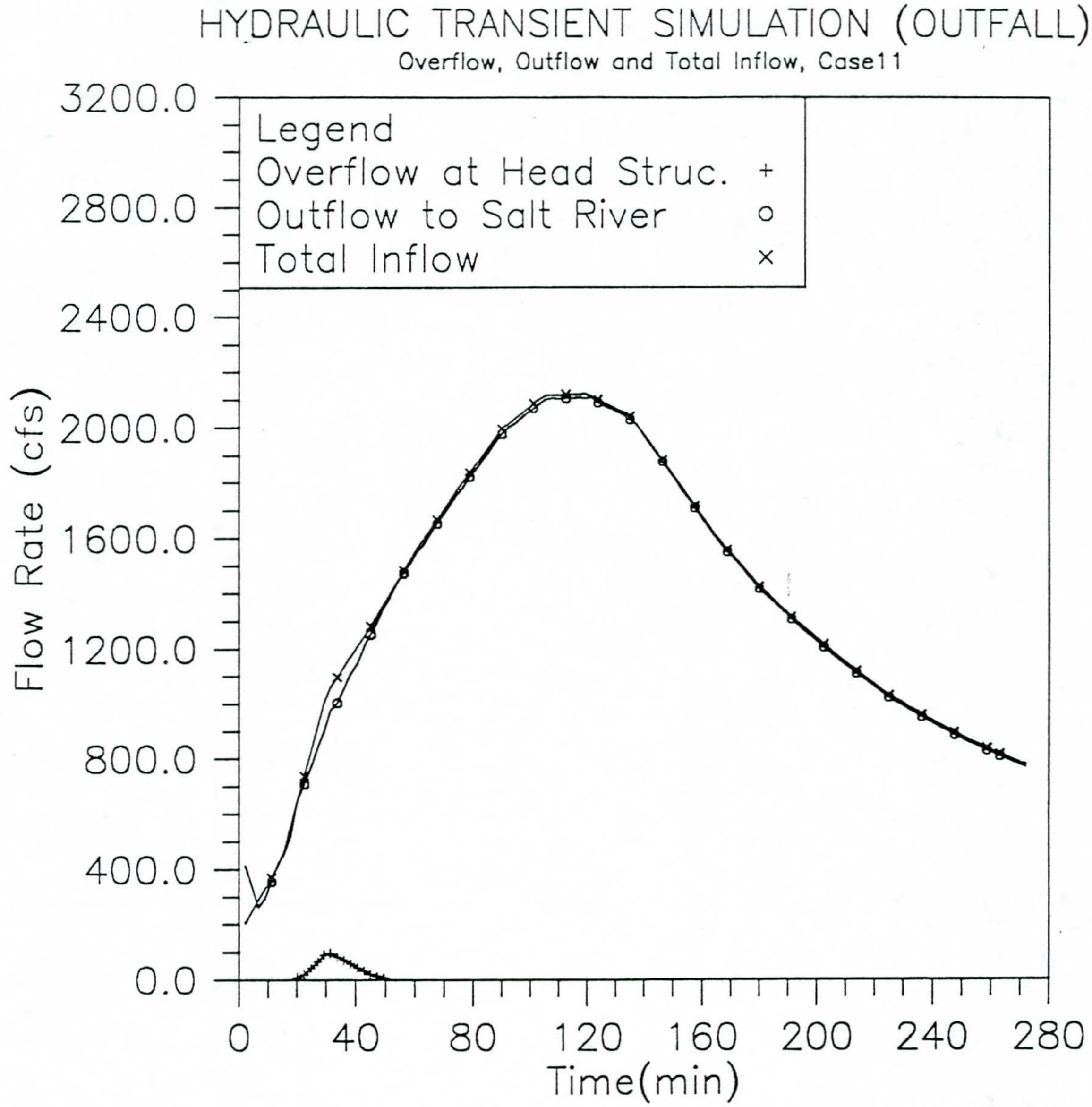


Fig. 11.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 6 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 11).

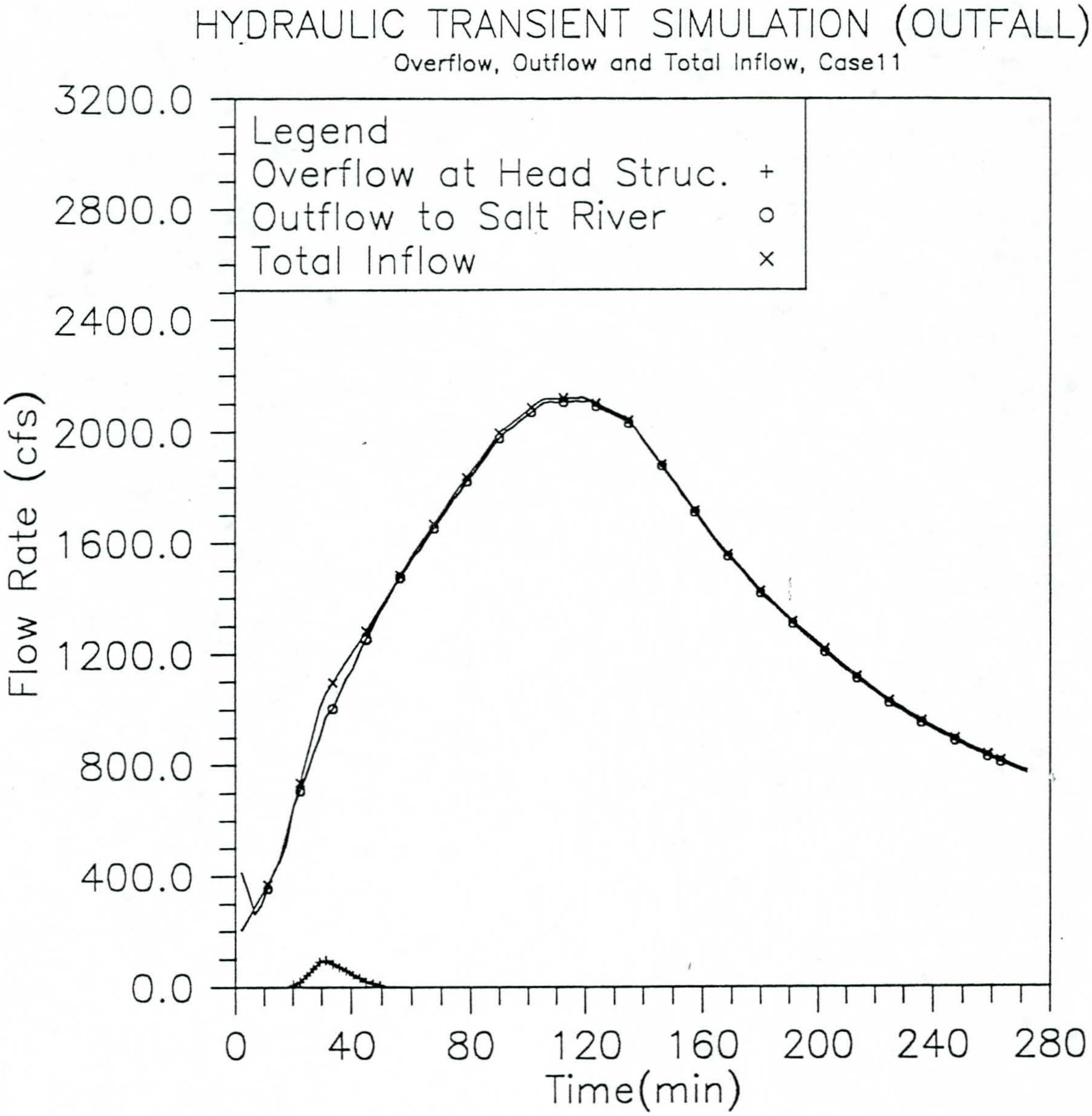


Fig. 11.5 The total flow balance: total inflow to the tunnel system, outflow to the river, and overflow at the head structure, modeling case: 6 ft diameter, 'wet' tunnel, and 30 cfs inflow at the head structure (Case 11).