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

Project Report No. 445

Mainstream and Des Plains TARP Tunnel System
A Hydraulic Model Study

by

Jianming He and Charles C. S. Song

Prepared for

METROPOLITAN WATER RECLAMATION DISTRICT OF GREATER CHICAGO
111 East Erie Street
Chicago, IL 60611

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I. INTRODUCTION

1.1 Study Objective

Numerical studies of hydraulic transients for the Phase I Mainstream system of the Tunnel and Reservoir Plan (TARP) were conducted in 1988 [1] and 1992 [2]. These studies revealed that due to storage and/or conveyance limitation of the TARP Phase I Mainstream system, flow must be substantially reduced to avoid geysering problems induced by hydraulic transients. Later in 1994 [3], hydraulic transient studies of the preliminary design of Mainstream & Des Plaines TARP Phase II systems were also conducted under various tunnel operation conditions. It was found that a reservoir at the downstream end does little help to reduce the transient problem in the Mainstream system due to the conveyance limitation during the simulated storm event.

The main objective of this study is to investigate the hydraulic event which occurred in the TRAP system on June 1, 1999, resulting in flooding of the Dewatering Valve Chambers at the Mainstream Pumping Station, and damage to the mechanical and electrical equipment therein. The study uses District operation data, reports and findings, photographs of the damage, rain data, as-built facility plans, and all other available, applicable information to characterize the event. In addition to identifying the underlying causes of the event, the study is also to investigate the methods of operation of the Mainstream and Des Plaines TARP tunnel systems, separately and in combination, to optimize CSO pollution capture while avoiding adverse hydraulic transient phenomena such as tunnel geysering, severe pressure surges, etc. in TARP facilities. The study is also to identify possible additional and/or revised control features of the TARP system necessary to achieve the aforementioned optimum operation.

1.2 Work Scope

For the purpose described above, the major tasks of this study are as follows:

A. Modify the existing Mainstream TARP mathematical model to add the North Branch of the Chicago River Tunnel, which was completed in 1998.

B. Using the larger of the two storm events modeled in the mid-1980s as the inflow to the Mainstream Tunnel System, determine whether there will be any surges or operational problems if the tunnels are run interconnected. Assume initially that the Des Plaines Tunnel System takes flow only from the Mainstream Tunnel at its point of connection. Determine the inflow control strategy that must be employed in such a case (i.e., when gates must be closed, which specific gates are to be closed and in what order, etc...).

C. Using the storm event of Task B, model the condition of the Des Plaines Tunnel System initially empty and the tunnel systems interconnected so that flow from the Mainstream System is allowed to flow into the Des Plaines tunnel. On successive runs, model the effects of closing the valves to the Des Plaines tunnel (the two 60-inch valves in the dewatering valve chamber) at various points in time and at different rates of closure. (Interlocks require that the valves close sequentially, rather than simultaneously). Perform all of the Task C runs under two sets of initial conditions: first, assuming the Mainstream System is partially full (at a volume before which flow to the interconnected Des Plaines System would begin) at the onset of the storm event.

D. Compute the impact forces that would be applied to the closing valves and tunnel in the various Task C scenarios. Since the impact force data is prerequisite information for other independent studies desired by the District, this portion of the work should be given priority and completed within 60 days of the Notice to Proceed date of this Contact. At a point not later than 30 days from the Notice to Proceed, an initial estimate of the potential range of the magnitude of these forces shall be provided.

E. Task B, C, and D initially assume that Des Plaines tunnel takes no flow other than that from the Mainstream tunnel. Repeat the above tasks by modifying the scenarios such that the Des Plaines tunnel is also receiving wet weather flow from the upper level sewers (normal tunnel inflow through the Des Plaines tunnel dropshafts) as well as taking flow from the Mainstream tunnel due to the interconnection.

F. Evaluate the algorithm currently used to calculate tunnel fill volume to determine whether or not it is adequate for operation of the two tunnel system in the "interconnected" mode. If inadequate, investigate possible alternatives for improving. For the existing, or any newly proposed, algorithm determine whether additional hydraulic grade line (HGL) measurement stations will be required. Including also any information on specific types of devices/equipment that may be better suited than the existing devices for the kinds of operating conditions that have been or can potentially be experienced in the tunnels and shafts.

G. Model the case of the tunnel system operated separately (i.e., the valves in the dewatering valve chambers closed for both tunnels during storm inflow). For this mode of operation, again evaluate the current tunnel volume algorithm, evaluate its effectiveness, suggest any necessary improvements, and determine the numbers and locations of HGL measurement stations required for each tunnel system.

H. All findings and results to be reported and discussed with the District as they are obtained in order to determine possible additional analyses or revised conditions.

I. Any additional analyses as determined necessary by the district, arising at any point during the study, related to optimizing operation of the tunnel systems.

Such additional studies shall be as directed by the District in writing. Changes to the project's cost and/or duration necessitated by such additional work shall be negotiated between the District and the University.

J. Preparation of final report with findings, conclusions, and recommendations.

II. THE MIXED TRANSIENT FLOW MODEL

2.1 Modeling Equations

The flow to be simulated is very unsteady and features highly dynamic phenomena such as pressurization surge and gysering. The transient flow model used, then, must be able to simultaneously calculate unsteady open channel flows and unsteady pressurized flows, including the abrupt change that occurs at the shock or the surge front.

The well-known St. Venant equations:

$$\frac{\partial y}{\partial t}+v\frac{\partial y}{\partial x}+\frac{c^2}{g}\frac{\partial v}{\partial x}=0 \quad (1)$$

$$g\frac{\partial y}{\partial x}+\frac{\partial y}{\partial t}+v\frac{\partial v}{\partial x}+g(S_f-S_o)=0 \quad (2)$$

are used to represent the unsteady open channel flow. In the above equations, y is the flow depth, v is the flow velocity, c is the gravity wave speed, S_o is the channel slope, S_f is the energy slope, and g is the acceleration due to gravity, x is the distance along a tunnel, and t is time.

The corresponding equations for unsteady pressurized flow are:

$$\frac{\partial y}{\partial t}+v\frac{\partial y}{\partial x}+\frac{a^2}{g}\frac{\partial v}{\partial x}=0 \quad (3)$$

$$g\frac{\partial y}{\partial x}+\frac{\partial y}{\partial t}+v\frac{\partial v}{\partial x}+g(S_f-S_o)=0 \quad (4)$$

in which a is the pressure wave speed, while y takes the meaning of piezometric head measured from the tunnel invert. The systems of the equations (1) - (4) are solved by the method of Characteristics [4].

Because the transition from the open channel flow condition to pressurized flow condition must be abrupt, as in the case of a hydraulic jump, the special shock boundary conditions must be applied. It was shown by Cardle and Song [4], for a pressurization surge or a positive surge, that three characteristic equations plus two shock boundaries can be used to calculate five unknowns at the interface. These five unknowns are v and y on both sides of the interface and the speed of the interface movement. The model can also simulate the negative surge, which occurs during the depressurization process. The detailed physical nature of the process has been discussed by Guo and Song [5].

A number of other boundary conditions representing junctions, dropshafts, upstream ends, downstream end, reservoirs, pump stations, and other accessories are also provided in the model. Inflow hydrographs, outflow conditions, and other active or passive control methods can be also included in the input data file. Flow velocity, depth, discharge, and other variables at any locations and any time may be specified as outputs.

For more than twenty years, the above dynamic transient mixed flow mathematical model has been applied to a number of large sewer tunnel system as a problem solver or design tool. These tunnel systems include Rochester City Sewer System, NY [6], Chicago TARP Phase I [1, 2, 7] and Phase II [3], Phoenix I-10 Tunnel System, AZ [8], Milwaukee Inline Storage System, WI [9, 10], New York Passaic River Flood Protection Tunnel, NY [11], Fall River Tunnel System, MA [12] and Narragansett Bay Commission Tunnel System, RI [13].

2.2 Modeling Configuration

For the mathematical modeling purpose, the simplified Mainstream & Des Plaines TRAP Tunnel system, as shown in Fig. 1, is used. Numbers shown in Fig. 1 are the station numbers used in the model for the purpose of defining different segments of the system. Each junction is represented by three stations for identification of three connecting segments. The entire system is divided into 844 segments of 500 feet each.

2.3 Inflow Hydrographs

The peak inflow rate at each dropshaft and junction is given using the design data for the tunnel systems, which were provided by The Metropolitan Water Reclamation District of Greater Chicago. Their distribution pattern is chosen based on the storm event of Oct. 18, 1985, in the same area. The Keifer's inflow hydrograph for all the dropshafts have been stored. If the design peak inflow at a dropshaft happens to coincide with the peak value of the Keifer's hydrograph, then Keifer's hydrograph is used. If the two peak values don't agree, then the Keifer's hydrograph is considered as a unit hydrograph, and it is scaled up and down according to the ratio of the two peak values.

Using the above method, the hydrograph at each dropshaft and junction can be obtained. Fig. 2 shows the total inflow rate of the generated hydrographs at the Mainstream system, the Des Plaines system, and the combined system.

III. GENERAL HYDRAULIC TRANSIENT CHARACTERISTICS

3.1 Initial Estimate of the Impact Force

To identify the underlying causes of the event occurred in the TRAP system on June 1, 1999, it is important to know the impact force generated by the valve closure in the Des Plaines Valve Chamber. Since the water elevations at both Mainstream and Des Plaines valve chambers were recorded before the valve closure, the peak impact force (F_p) can be estimated based on the fact that the first valve was slammed shut.

$$F_p = \frac{\Delta P}{\gamma} = \frac{aV_2}{g} \tag{5}$$

where a is the wave speed which can be determined based on the air content in the water, V_2 is the flow velocity passing the valve section in Des Plaines valve chamber, and g is the acceleration due to gravity .

The Bernoulli equation between the Mainstream Chamber and the Des Plaines Chamber can be used,

$$y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + h_{loss} \tag{6}$$

where h_{loss} is the total hydraulic loss (all the local and friction losses) from the Mainstream Chamber to the Des Plaines Chamber, which can be expressed as,

$$h_{loss} = h_{valve1} + h_{valve2} + h_{local1} + h_{local2} + h_{1-3} + h_{3-2} + h_3 \tag{7}$$

$$h_{valve1} = k_{valve}\frac{(Q/3)^2}{2g(\pi 2.5^2)^2}$$

$$h_{valve1} = k_{valve}\frac{(Q/2)^2}{2g(\pi 2.5^2)^2}$$

$$h_{1-3} = f\frac{L_{1-3}}{16}\frac{Q^2}{2g(\pi 8^2)^2}$$

$$h_{3-2} = f\frac{L_{3-2}}{13}\frac{Q^2}{2g(\pi 6.5^2)^2}$$

$$h_3 = k_3\frac{Q^2}{2g(\pi 8^2)^2}$$

$$h_{local1} = f\frac{L_{11}}{5}\frac{(Q/3)^2}{2g(\pi 2.5^2)^2} + k_{12}\frac{(Q/3)^2}{2g(\pi 2.5^2)^2} + f\frac{L_{13}}{9}\frac{(Q/3)^2}{2g(\pi 4.5^2)^2} + k_{14}\frac{(Q/3)^2}{2g(\pi 4.5^2)^2}$$

$$h_{local2} = f\frac{L_{21}}{5}\frac{(Q/2)^2}{2g(\pi 2.5^2)^2} + k_{22}\frac{(Q/2)^2}{2g(\pi 2.5^2)^2} + k_{23}\frac{(Q/2)^2}{2g(\pi 4.5^2)^2}$$

By combining all the local and friction losses, the total hydraulic loss from the Mainstream Chamber to the Des Plaines Chamber can be expressed as,

$$h_{loss} = CQ^2 \qquad (8)$$

where C is combined coefficient related to the tunnel geometry and loss coefficients from the Mainstream Chamber and the Des Plaines Chamber. The hydraulic loss coefficients (k) can be found in the hydraulic loss coefficient manual [14].

Based on the recorded water elevation at 22:24, 6/1/99, the peak water surface elevation difference between the Mainstream Chamber and the Des Plaines Chamber just before the valve closure is,

$$y_1 - y_2 = 278.55 - 180.88 = 97.67 \quad \text{ft} \qquad (9)$$

Now Eq. (2) becomes

$$y_1 - y_2 = \frac{(Q/2)^2}{2g(\pi 2.5^2)^2} - \frac{(Q/3)^2}{2g(\pi 2.5^2)^2} + CQ^2 = 97.67$$

Q=2343 cfs

$$V_2 = \frac{Q/2}{\pi 2.5^2} = 59.66 \quad \text{ft/s}$$

The peak impact force can be found,

$$F_p = \frac{\Delta P}{\gamma} = \frac{aV_2}{g} = \frac{59.66}{32.2}a = 1.85a$$

Based on different wave speed, the peak impact force can be calculated.

Wave speed (ft/s)	400	1000	2000	3000	4000
Impact force (ft)	740	1850	3700	5550	7400

3.2 Simulation of June 1, 1999 Event

In order to verify the above estimate of the hydraulic transient condition during the event occurred on June 1, 1999, the first run is to simulate the case at which the Mainstream and Des Plaines tunnel systems are inter-connected, using the hydraulic transient model described above.

Fig. 3 shows the water depth changes at the two valve chambers and their water level difference during the simulated storm event. As shown in the figure, the peak water level difference between the two valve chambers is about 105 ft. This value is close to the peak value of 98.72 ft recorded at 10:22 p.m.. According to the record, the accident occured at 10:24 p.m.. The flow rate from the Mainstream valve chamber to the Des Plaines valve chamber is shown in Fig. 4. During the initial storm stage, the discharge is negative, and the water flows from Des Plaines tunnel to Mainstream tunnel since the invert elevation of the Des Plaines tunnel is higher than that of the Mainstream tunnel. As the storm inflow to the tunnel systems increases, water level at the Mainstream tunnel rises more rapidly than that in Des Plaines tunnel. As a results, the flow reverses its direction, and the flow rate from the Mainstream tunnel to the Des Plaines tunnel rapidly increases to its peak (2100 cfs), which is also close to the value (2343) estimated under Section 3.1.

Based on the simulation results, it is believed that the valve closing buttons were pressed at the peak flow rate. In fact, the water level difference decreases also rapidly from its peak as the water level in Des Plaines tunnel rises. On the other hand, due to the tunnel and valve size limitation, the flow rate between the two tunnel system is relatively small compared with their huge inflow to the tunnel, and the peak period only lasts for about 45 minutes. It is also found that this small amount and short period flow has little effect on the transient surge behavior in both tunnel system. Therefore, it is suggested that the valves not be operated during a big storm event. The controllable flow by operating the valve is relatively small.

3.3 Effect of Valve Closing Time on Impact Force

As stated before, the impact force estimate Eq. (9) is based on the assumption that the valve is slammed shut, which is the worst scenario. To evaluate the effect of valve close time on the impact force, the hydraulic transient model as described before is needed. However, since the entire tunnel system model is too large, and the hydraulic surge conditions in the tunnels have little effect on the impact force generated by the valve closing during the short valve closing period, the impact force model excludes the two main tunnel systems. For the small system, the segment length between two model stations can be reduced to 50 ft from 500 ft for better accuracy. The small system can was used to simulate the valve closure event using the peak water level difference between the two the valve chamber from the entire system model as the initial conditions.

As mentioned before, the impact force is also affected by the wave speed, which is related to the air content in the water. Based different valve closing time and wave speeds, the peak impact force is shown in Fig. 5. The results show that the impact force is significantly affected by both valve closing time and wave speed. Normally, the air content in a sewer tunnel system during a large storm event is normally assumed to be 0.5 to 2%, i.e., wave speed of 1500 to 400 ft/s is used in a hydraulic transient simulation.

3.4 Interconnected Tunnels (Case 1)

When the two tunnel systems are interconnected (i.e., all the valves in the two valve chambers are open), the flow direction at the connection depends on the water level between the two valve chamber. Part of the simulation results have been shown in Section 3.2. Fig. 6 shows the time variations of the water surface elevation at four upper level locations (the upstream end, Howard CS, CS-2, and DS-91) of the Mainstream tunnel system. There is a very strong surge at CS-2. Fig. 7 shows the water level changes at five downstream locations (DS-75, DS-50, DS-17, DS-8, and MS valve cahmber0 of the Mainstream tunnel system. The surge peak at MS valve chamber is due to the flow direction change from MS-to-DSP to DSP-to-MS as shown in Figs 3 and 4. The hydraulic grade lines along the main Mainstream tunnel (excluding the branches) at different time are shown in Fig. 8.

From theses figures, it is noticed that the Mainstream tunnel is first pressurized from the downstream end at t=85 minutes. As the tunnel continues to be filled and the pressurized portion expands, surge develops the interface between the pressurized zone and the free surface zone. The magnitude of the surge increases as the inflow rate increases. When the surge is close to the upstream end at t=120 mins, the pressure difference between the surge front and the downstream area rises to 215 ft, as shown in Fig. 8. The big water level difference pushes a large amount of flow from downstream area to upstream area. When the surge front arrives at the upstream end, the large amount of backflow causes water to be squeezed into the dropshaft so that the water level rises rapidly inside the dropshaft.

Fig. 9 shows the water level changes with time at five locations (DS-1, CS-2, DS20I, DS-35 and DSP valve chamber) of the Des Plaines tunnel system. The hydraulic grade lines along the main Des Plaines tunnel (excluding the branches) at different time are shown in Fig. 10. The main hydraulic transient characteristics in the Mainstream tunnel can be also found in the Des Plaines tunnel system. However, since the inflow to the Des Plaines tunnel system is smaller related to its tunnel capacity, compared with the Mainstream system, the surge condition in the Des Plaines tunnel is weaker. When the Mainstream tunnel is fully pressurized at t=130 mins, the Des Plaines tunnel is still partially full, and the pressure difference between surge front and the downstream area is much smaller.

As shown in Section 3.2, due to the interconnection between the two tunnel systems, the flow direction between the two tunnel changes twice. When the water

IV. CONCLUSIONS

The main hydraulic transient characteristics based on the study are summarized as follows.

1. When the Mainstream tunnel and the Des Plaines tunnel are interconnected, the flow direction between the two tunnel changes twice. Water first flows from the Des Plaines tunnel to the Mainstream tunnel since the Des Plaines tunnel has a higher invert elevation than the Mainstream tunnel. As the water level in the Mainstream tunnel rise faster than that in the Des Plaines tunnel, the water in the Mainstream tunnel flows back to the Des Plaines tunnel, and quickly reaches its peak of 2100 cfs. After the peak, the flow rate decreases rapidly, and then flow is from the Des Plaines tunnel to Mainstream tunnel again.

2. The valve closure on June 1, 1999 happened to occur at the peak flow of 2100 cfs from the Mainstream tunnel to the Des Plaines tunnel. Since the flow from the Mainstream tunnel to the Des Plaines tunnel only lasts about 45 minutes without any valve operation, and the flow rate has no effect on the hydraulic transient surge in both tunnels since it is relatively small compared with the inflow to the tunnels, it is highly recommended that the valves in both valve chambers not be operated during the peak flow period (less than 45 minutes).

3. The impact force generated by the valve closure in the Des Plaines valve chamber is dependent on the valve closure time and the wave speed which is related to the air content in the storm sewer water. For a normal condition, it is assumed that the air content in water is about 0.5% to 2% (wave speed: a= 400 to 1000 ft/s) for a large storm event. If the first valve was slammed shut, the potential impact force,

$$F=aV/g=1.66a$$

Wave speed (ft/s)	400	1000	2000	3000	4000
Impact Force (ft)	664	1660	3320	4980	6640

If it is not slammed shut, the effect of the valve closure time is as follows:

Impact force (ft) Change with Wave speed and Valve Closure Time (T)

Air content (%)	Wave speed(ft/s)	T=0.1m	T=0.25	T=0.5	T=1.0	T=2.0
2	400	689	527	452	405	373
1	600	1030	805	603	502	456
0.5	1000	1607	1209	906	753	652
0.2	1500	2520	1810	1306	1103	953

4. During the tested hydrogrpah storm event, both the Mainstream tunnel and the Des Plaines tunnel may experience strong hydraulic transient problems no mater whether they are interconnected or independent.

5. The effect of the interconnection between the Mainstream tunnel and the Des Plaines tunnel on the hydraulic transient surge is very small since the flow between the two tunnel is not significant compared with the inflow rate to the tunnels. If there is no wet weather inflow to the Des Plaines tunnel (i.e., the Des Plaines tunnel only takes flow from the Mainstream tunnel), the hydraulic transient surge in the Mainstream tunnel is not improved due to the geometrical limitation of the connection and valve size.

6. Inflow control is an effective way to avoid the surge. For the tested design hydrograph, the suggested inflow control procedure for the Mainstream tunnel system is that the gates at DS-27, DS-28, and DS-29 starts to close when the tunnel system is 20% full if the Mainstream tunnel is independently operated, and 35% full if the two tunnels are interconnected. All other control gate should start to be closed at 50% full for both independent and interconnected tunnels. For the Des Plaines tunnel, all the gates should start to close at 70% full of the Des Plaines tunnel capacity.

7. The valve closure procedure in the Des Plaines valve chamber has no significant effect on the hydraulic transient characteristics in both tunnels since the valve closure is too short and too far from the upstream area where the surge problem occurs.

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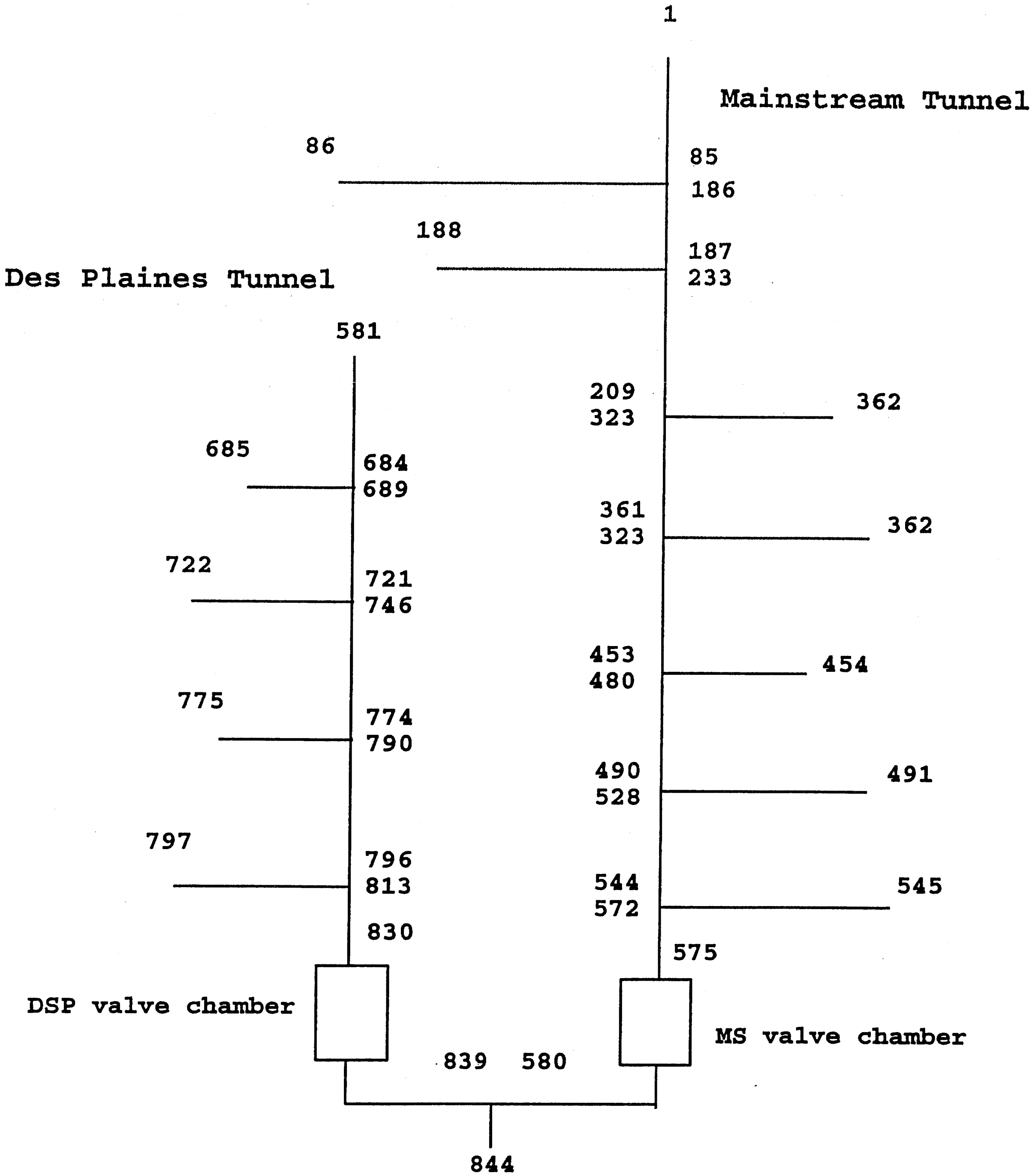

Fig. 1 Schematic of the Mainstream tunnel and Des Plaines tunnel systems for modeling purpose

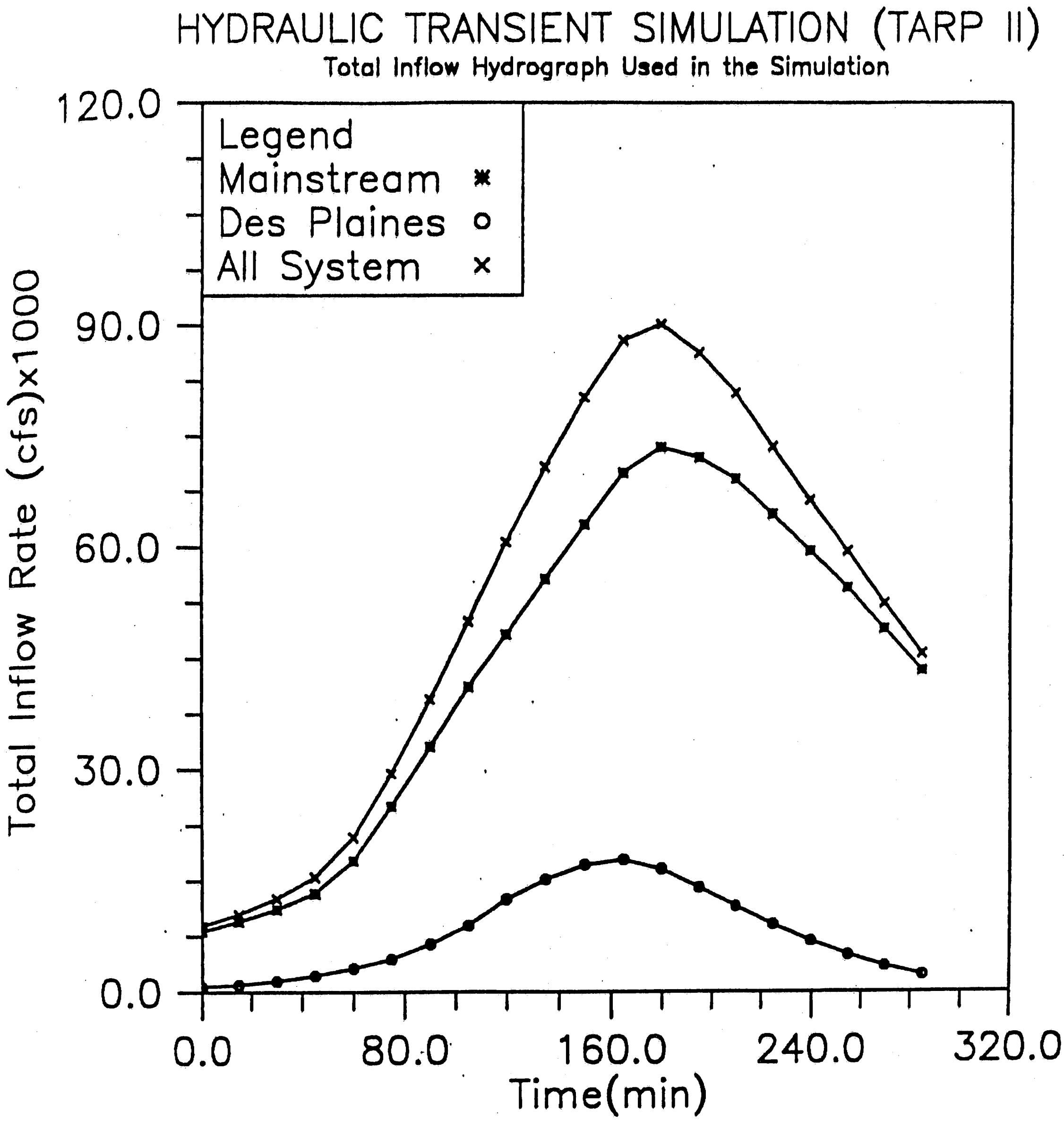

Fig. 2 Total inflow hydrodraphs to the tunnel systems based on the design values and Oct. 18, 1985 storm.

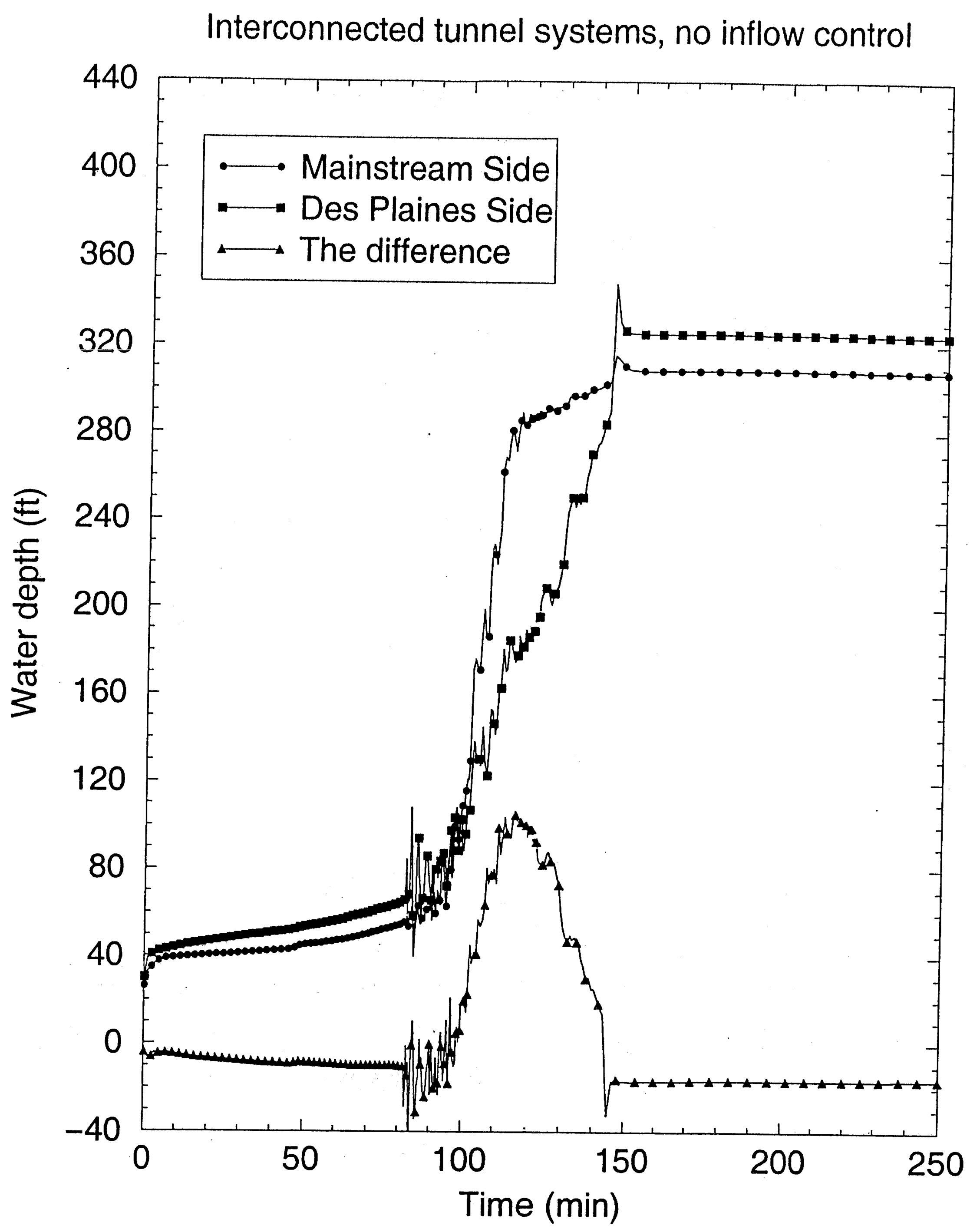

Fig. 3 Water depth changes with time at the two valve chambers and their water level difference, interconnected tunnels.

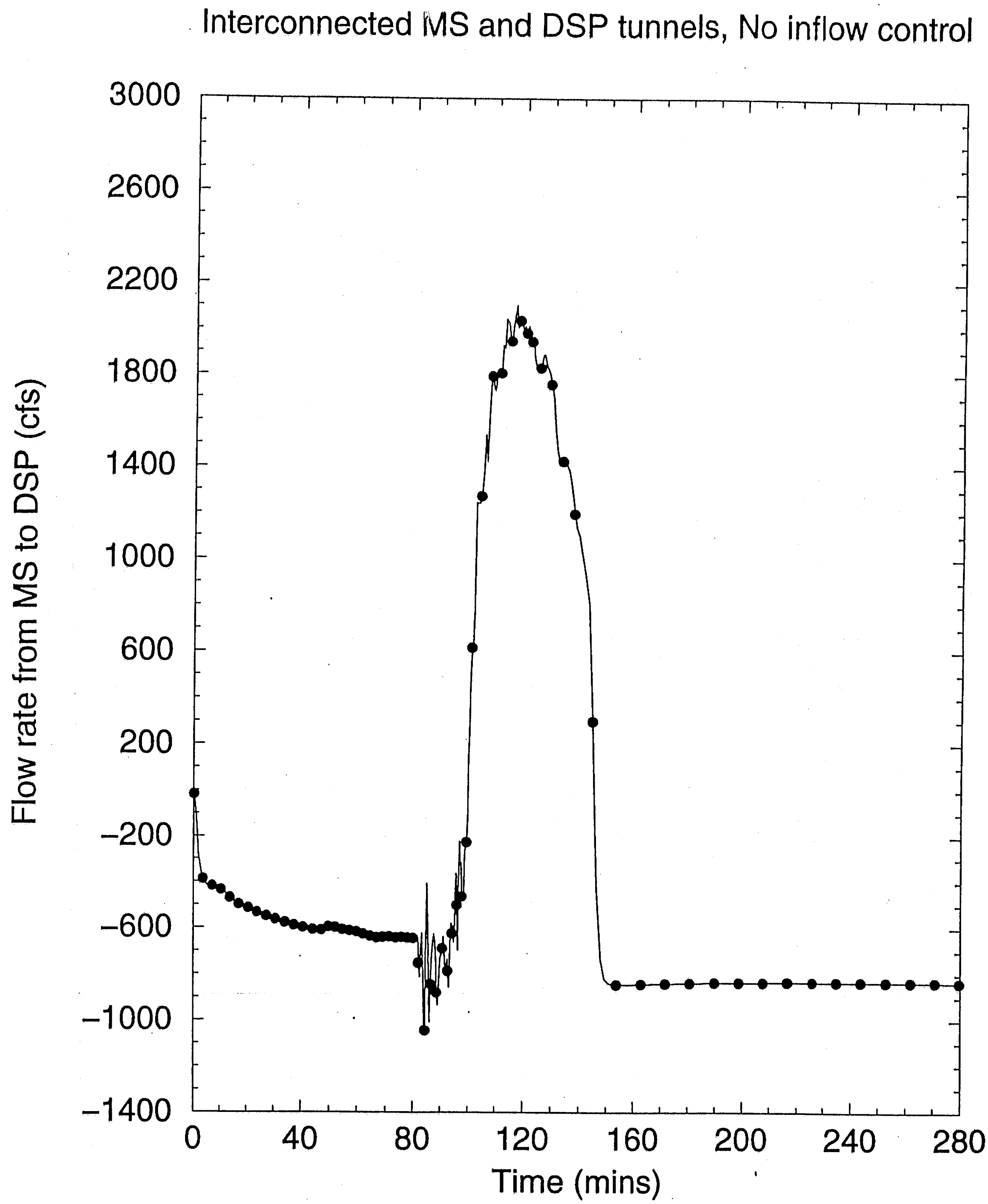

Fig. 4 Flow rate changes between the interconnected tunnels.

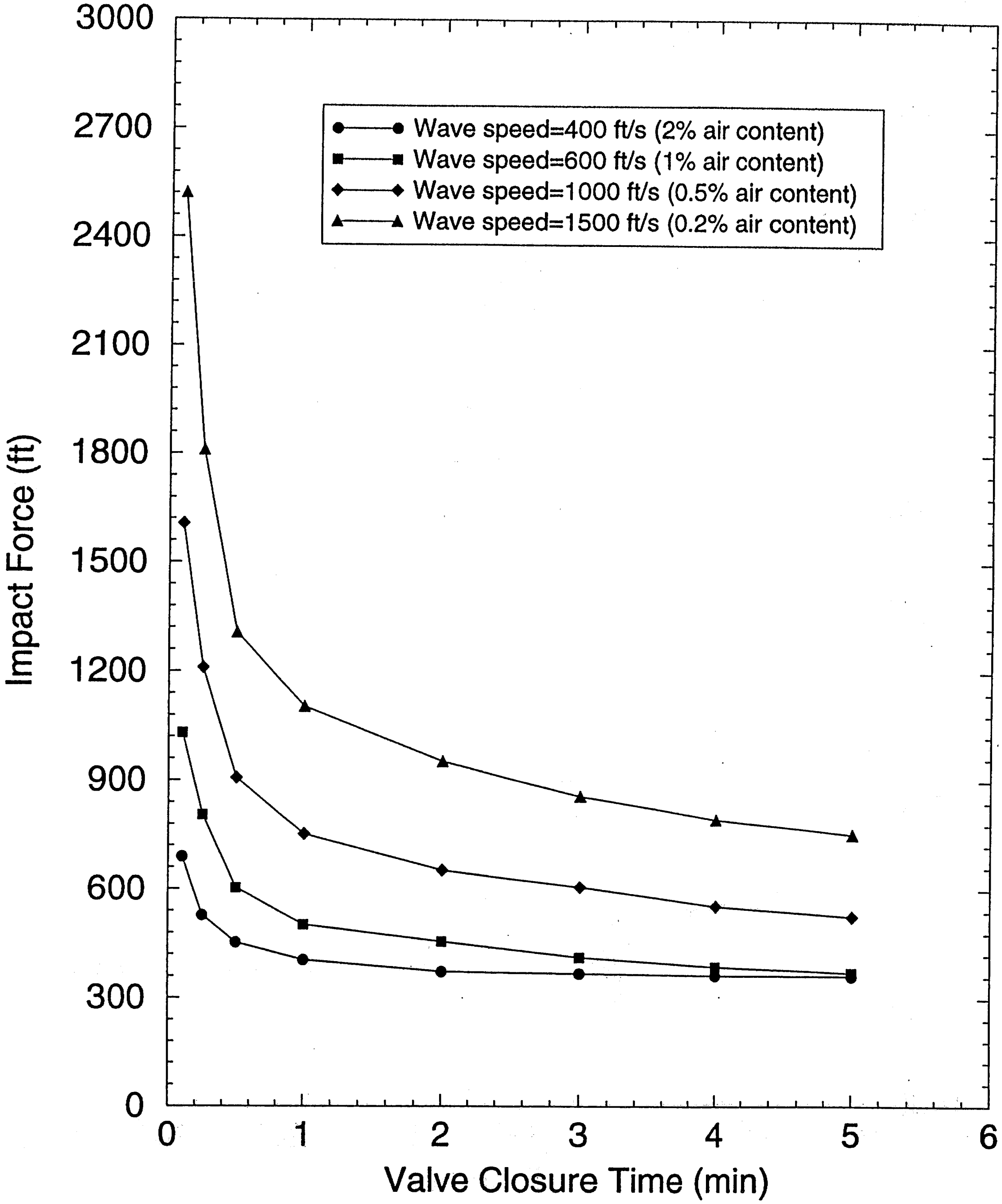

Fig. 5 Peak impact force generated by the valve closure in the Des Plaines valve chamber.

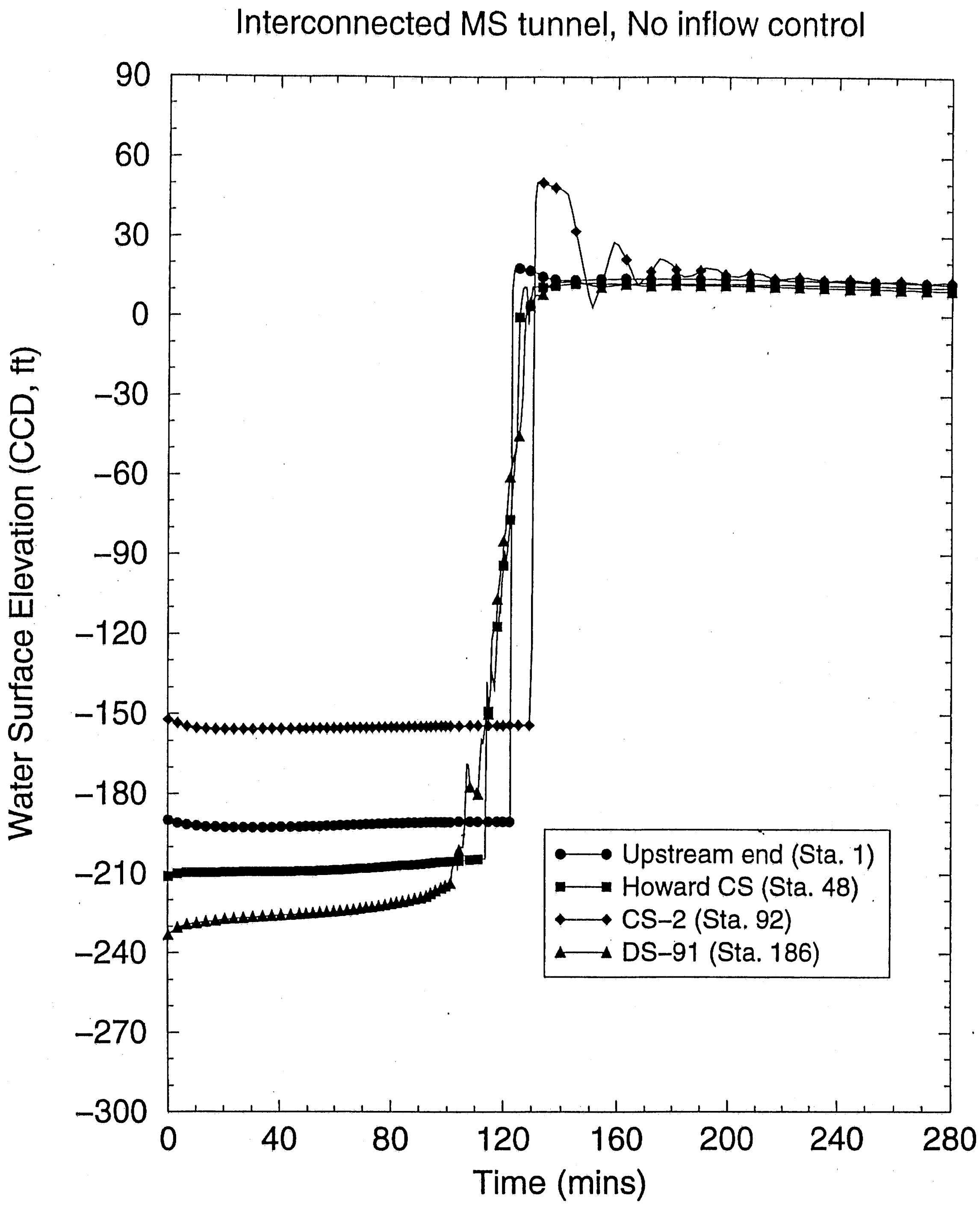

Fig. 6 Water elevation variations with time at 4 locations (the upstream end, Howard CS, CS-2, and DS-91) of the Mainstream tunnel, Case 1: interconnected tunnels, no inflow control.

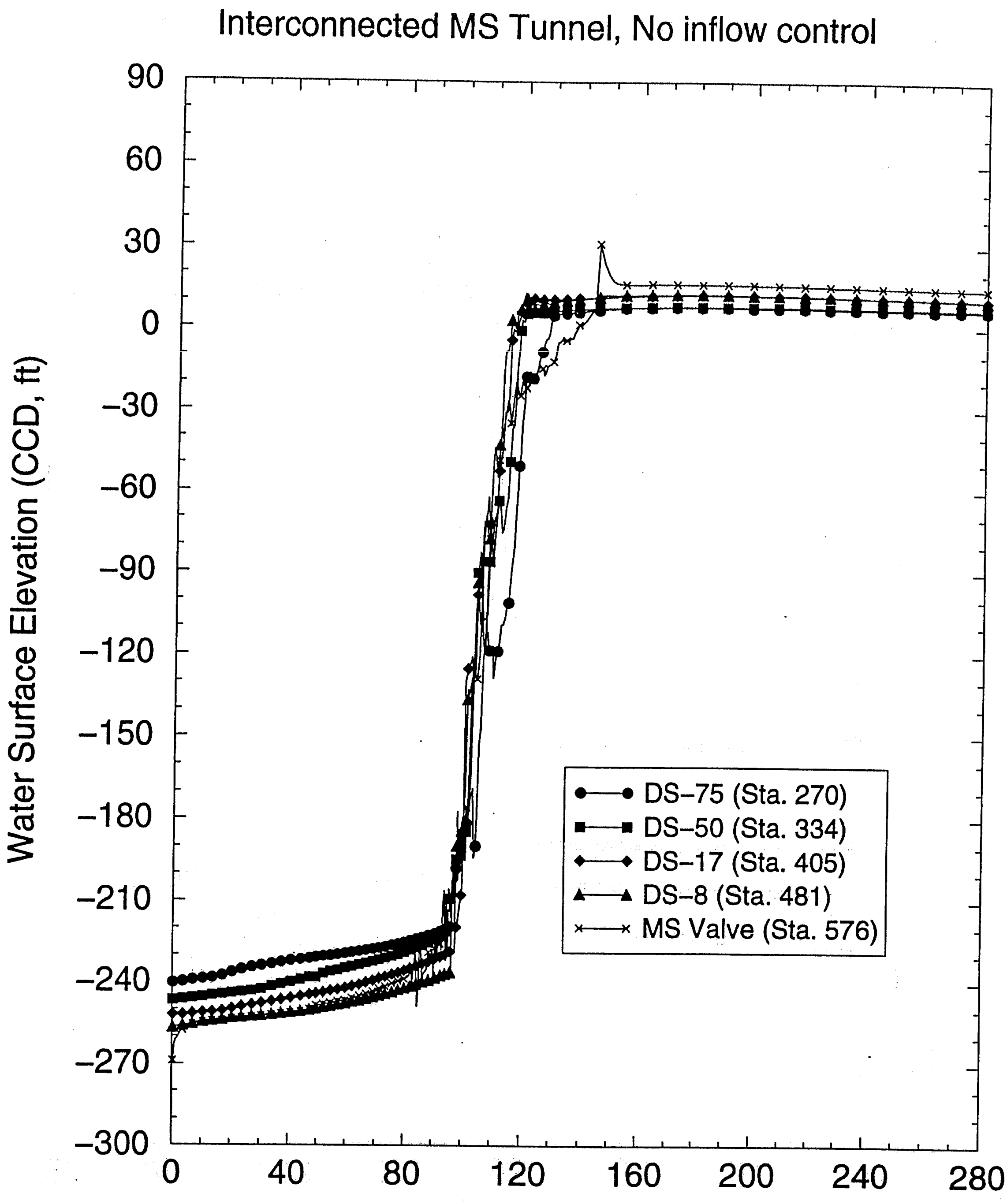

Fig. 7 Water elevation variations with time at 5 locations (DS-75, DS-50, DS-17, DS-8, and the MS valve chamber) of the Mainstream tunnel, Case 1: interconnected tunnels, no inflow control.

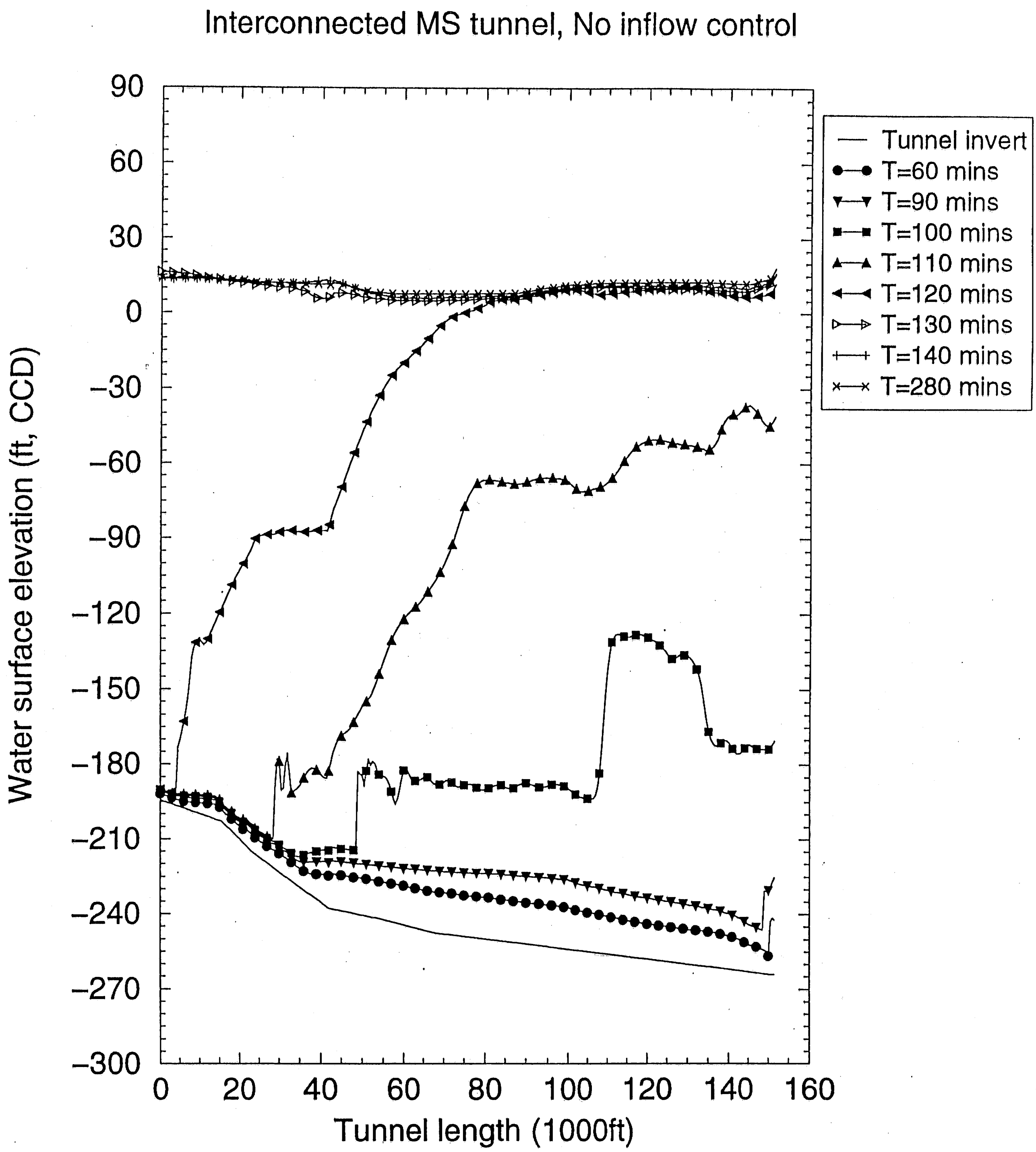

Fig. 8 Instantaneous hydraulic grade lines along the Mainstream tunnel, Case 1: interconnected tunnels, no inflow control.

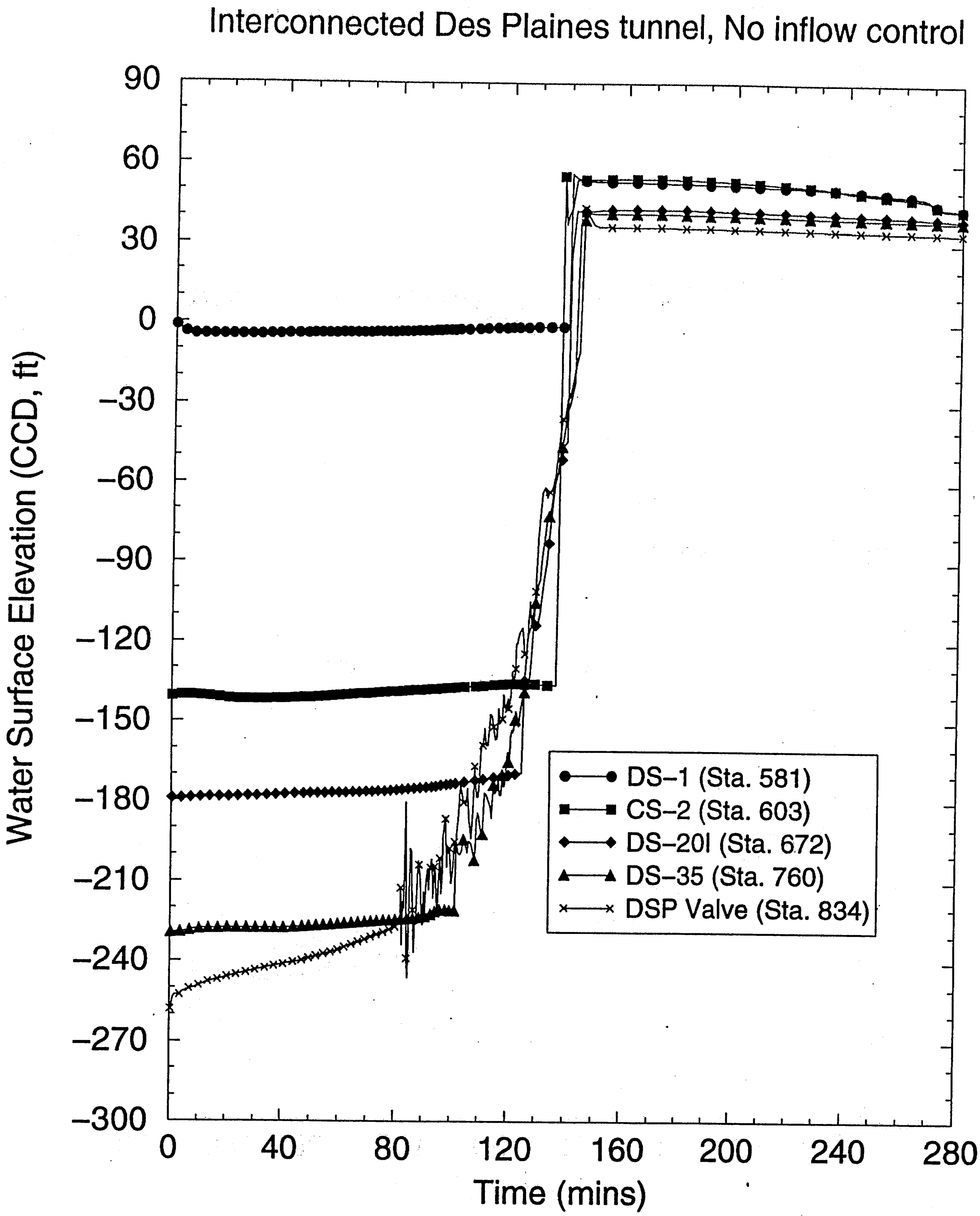

Fig. 9 Water elevation variations with time at 5 locations (DS-1, CS-2, DS20I, DS-35, and the DSP valve chamber) of the Des Plaines tunnel, Case 1: interconnected tunnels, no inflow control.

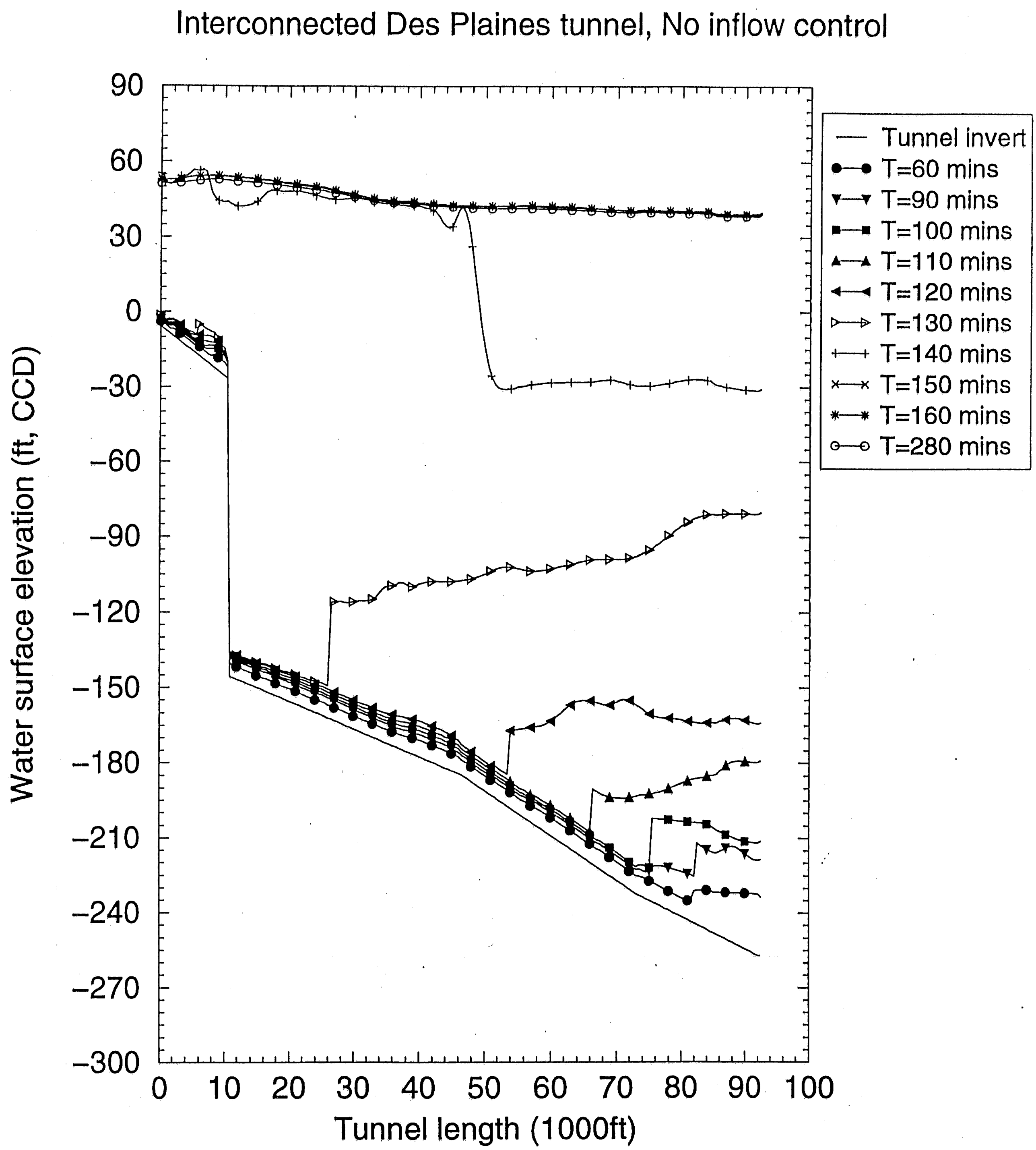

Fig. 10 Instantaneous hydraulic grade lines along the Des Plaines tunnel, Case 1: interconnected tunnels, no inflow control.

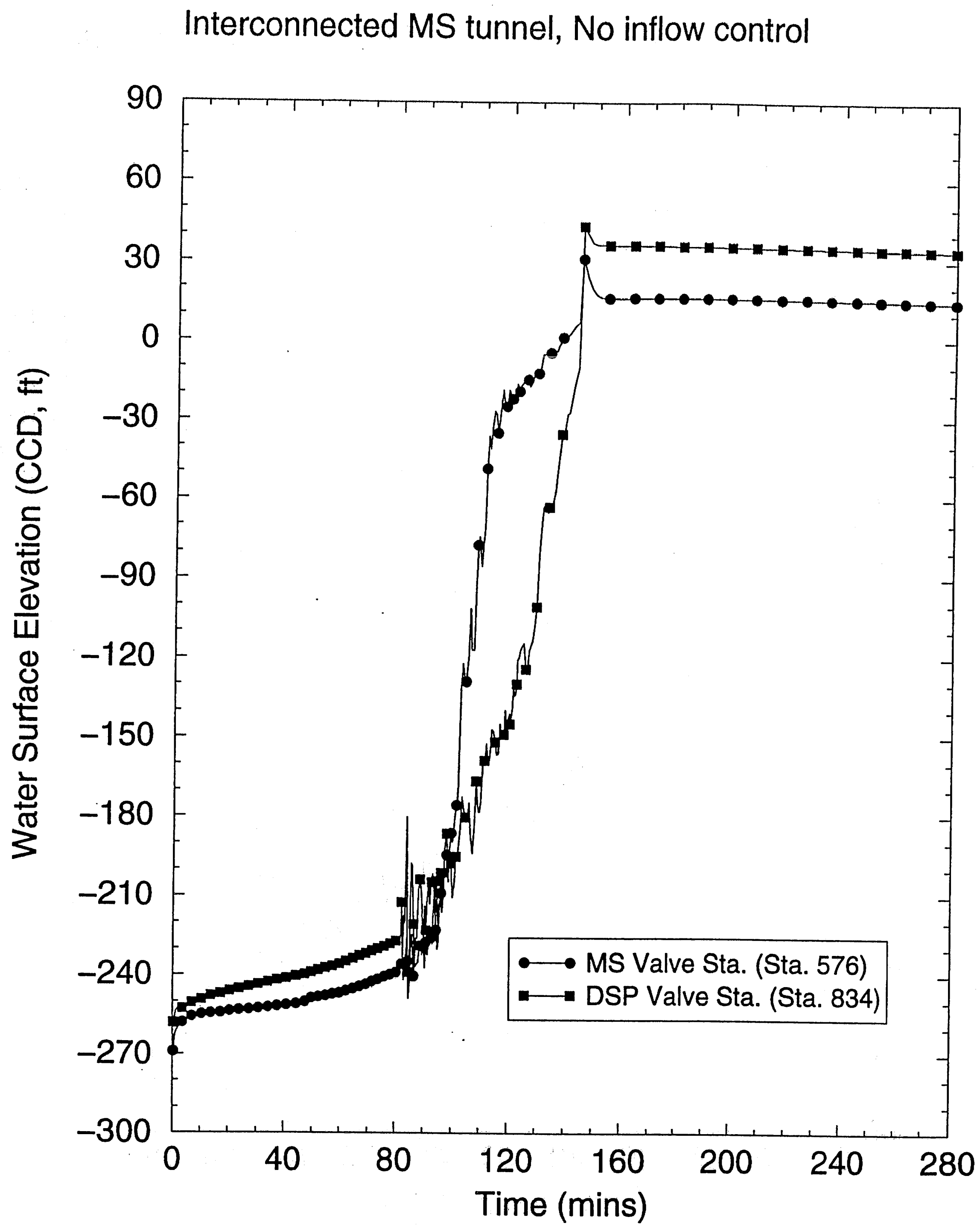

Fig. 11 Water elevation variations with time at the two valve chambers, Case 1: interconnected tunnels, no inflow control.

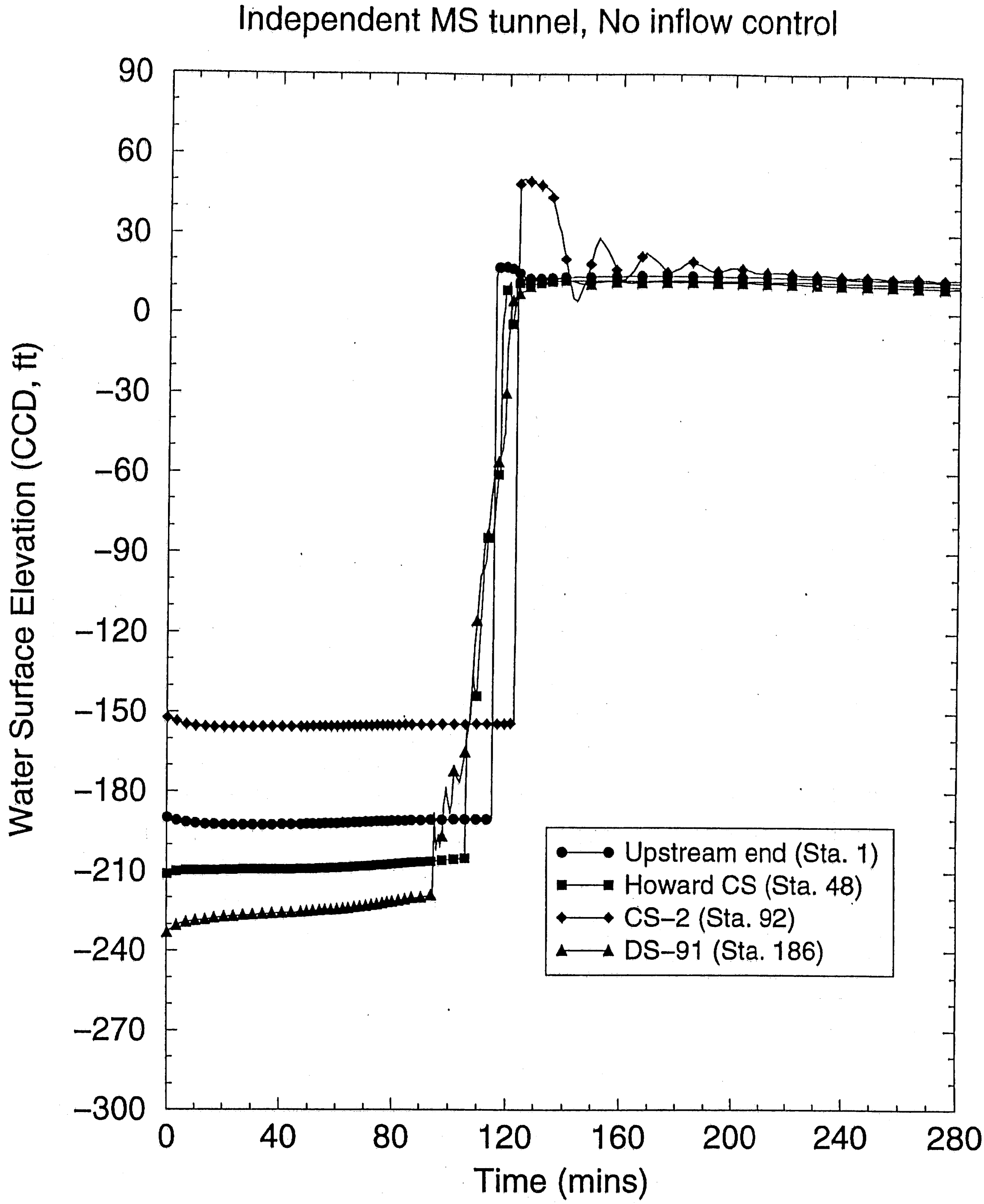

Fig. 12 Water elevation variations with time at 4 locations (the upstream end, Howard CS, CS-2, and DS-91) of the Mainstream tunnel, Case 2: independent tunnels, no inflow control.

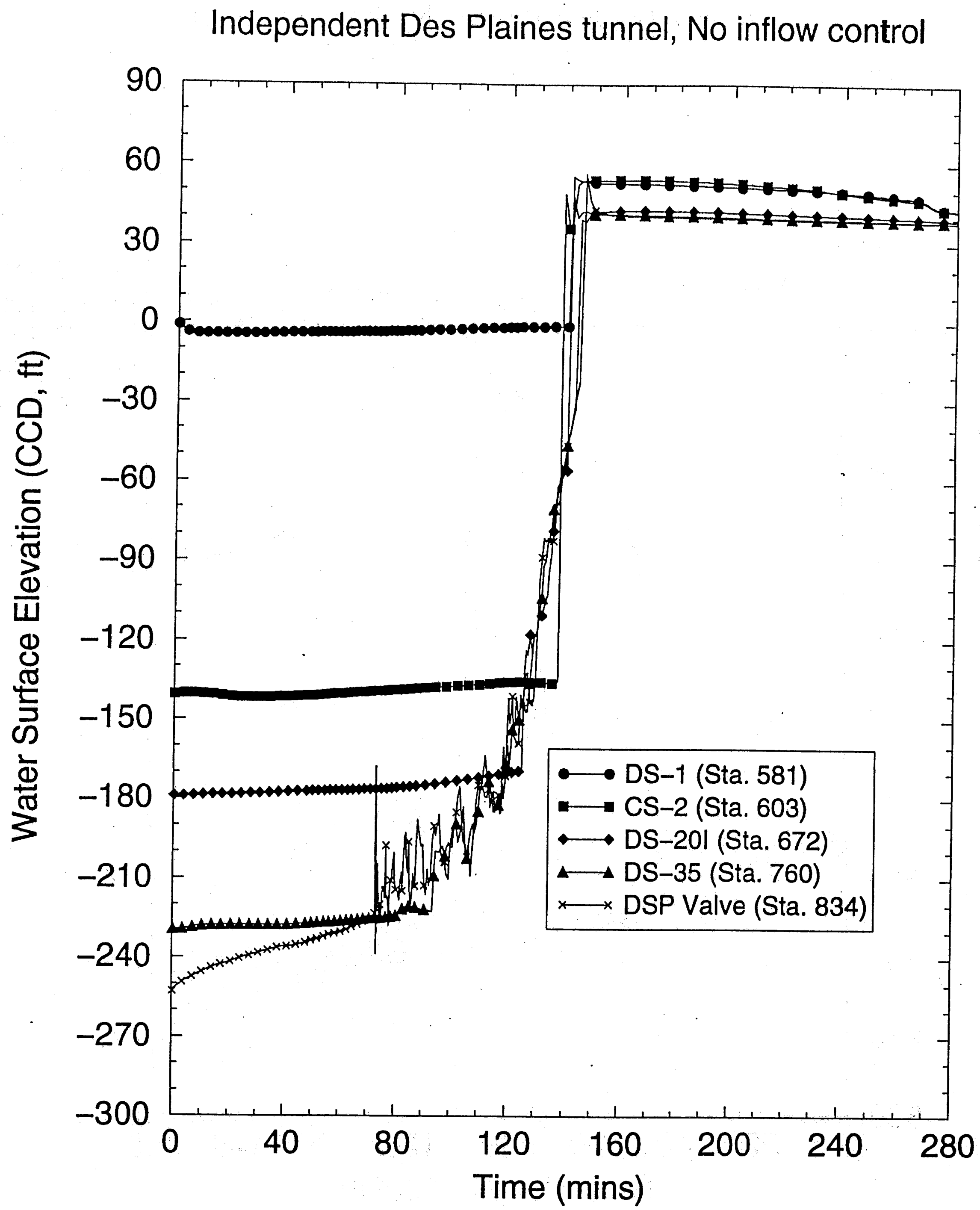

Fig. 15 Water elevation variations with time at 5 locations (DS-1, CS-2, DS20I, DS-35, and the DSP valve chamber) of the Des Plaines tunnel, Case 2: independent tunnels, no inflow control.

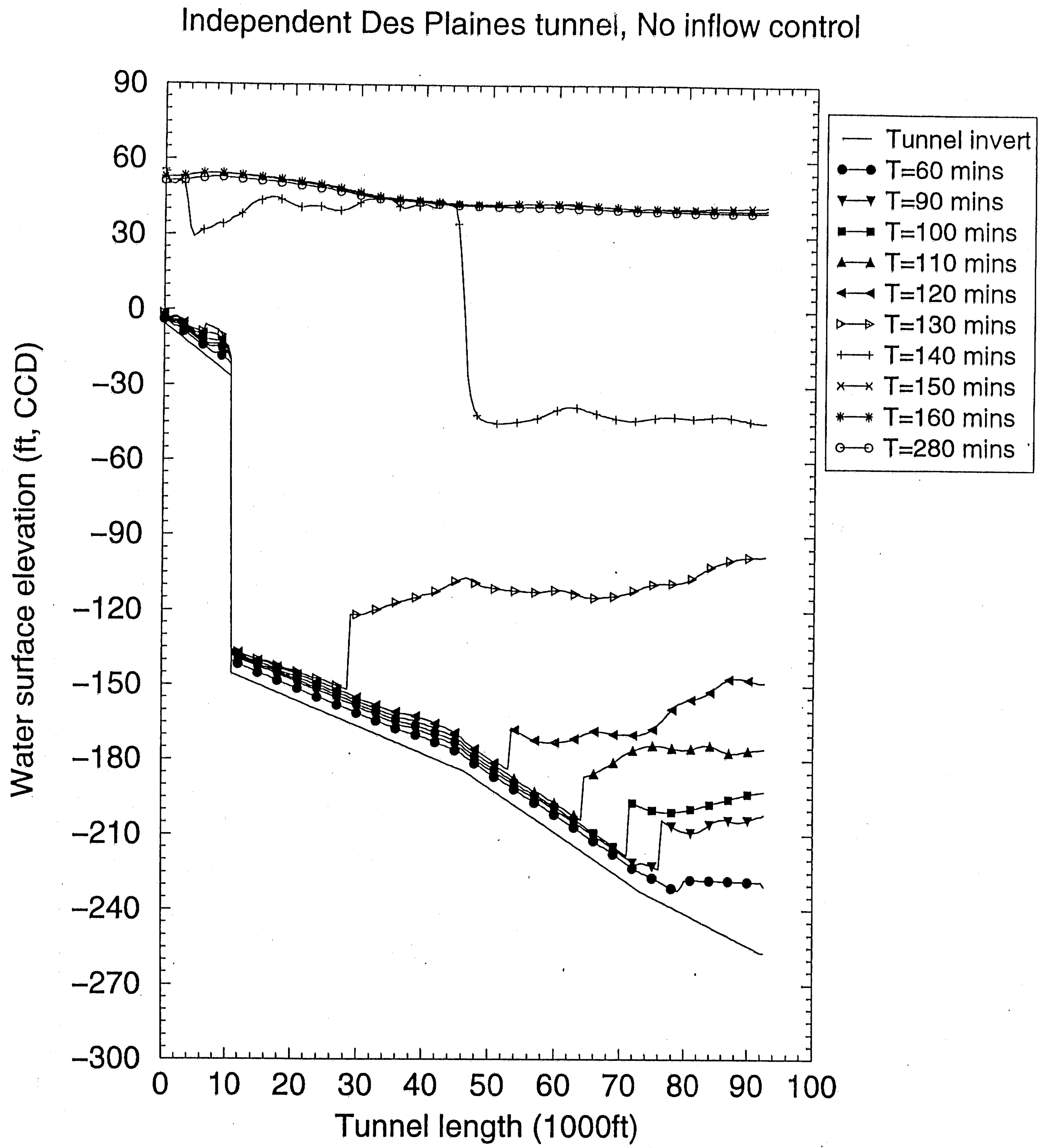

Fig. 16 Instantaneous hydraulic grade lines along the Des Plaines tunnel, Case 2: independent tunnels, no inflow control.

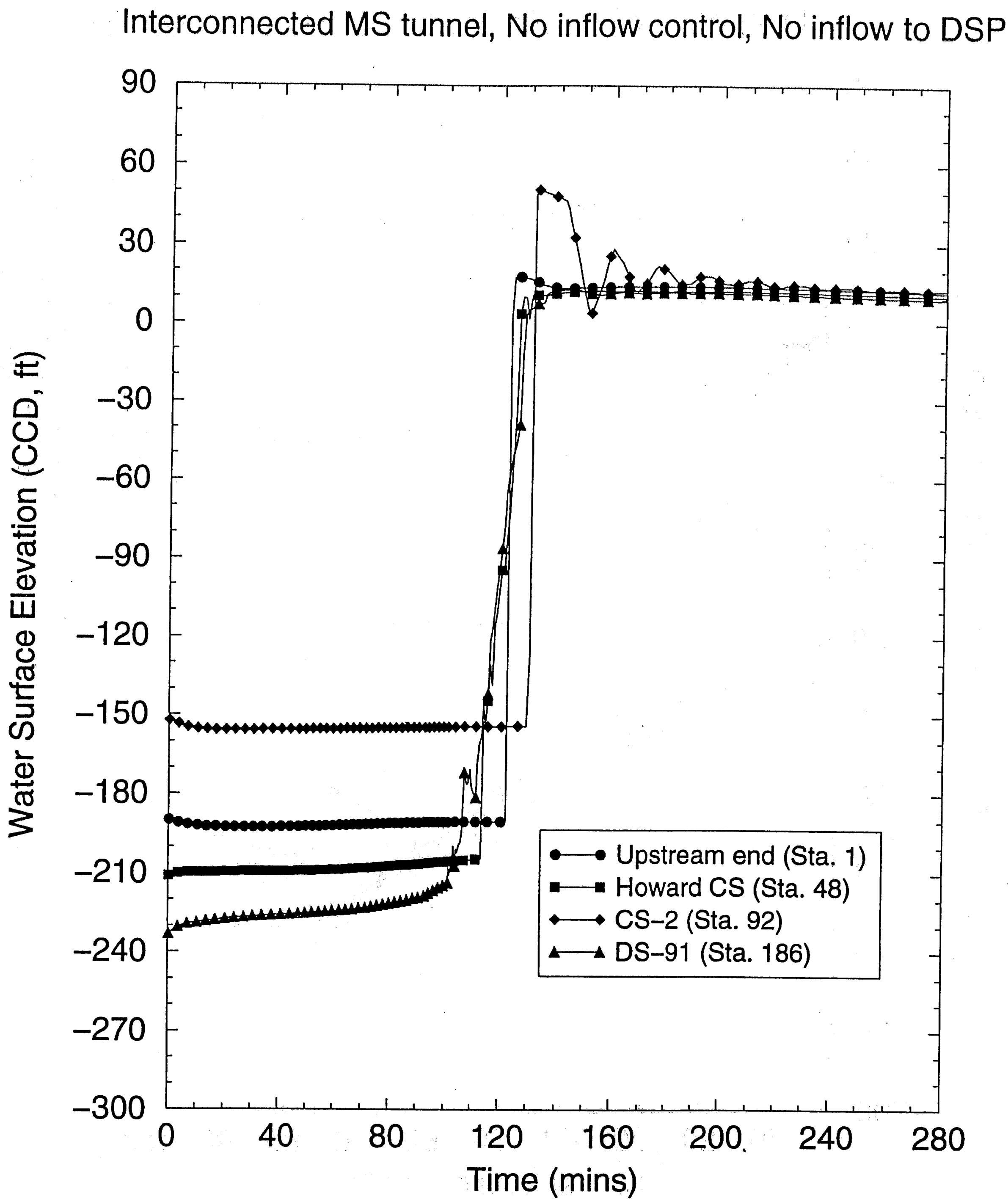

Fig. 17 Water elevation variations with time at 4 locations (the upstream end, Howard CS, CS-2, and DS-91) of the Mainstream tunnel, Case 3: interconnected tunnels, no inflow control, no wet weather inflow to the DSP tunnel.

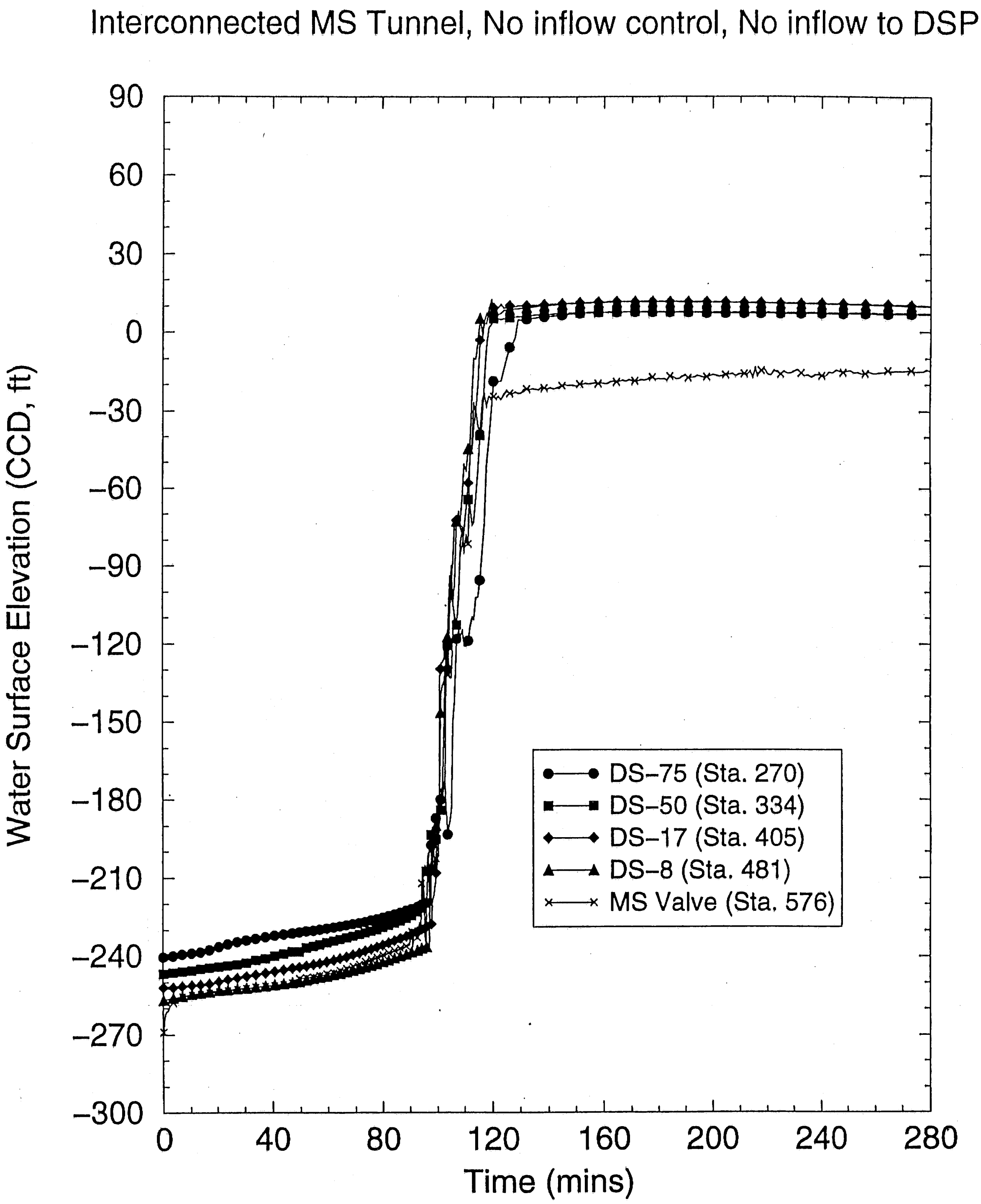

Fig. 18 Water elevation variations with time at 5 locations (DS-75, DS-50, DS-17, DS-8, and the MS valve chamber) of the Mainstream tunnel, Case 3: interconnected tunnels, no inflow control, no wet weather inflow to the DSP tunnel.

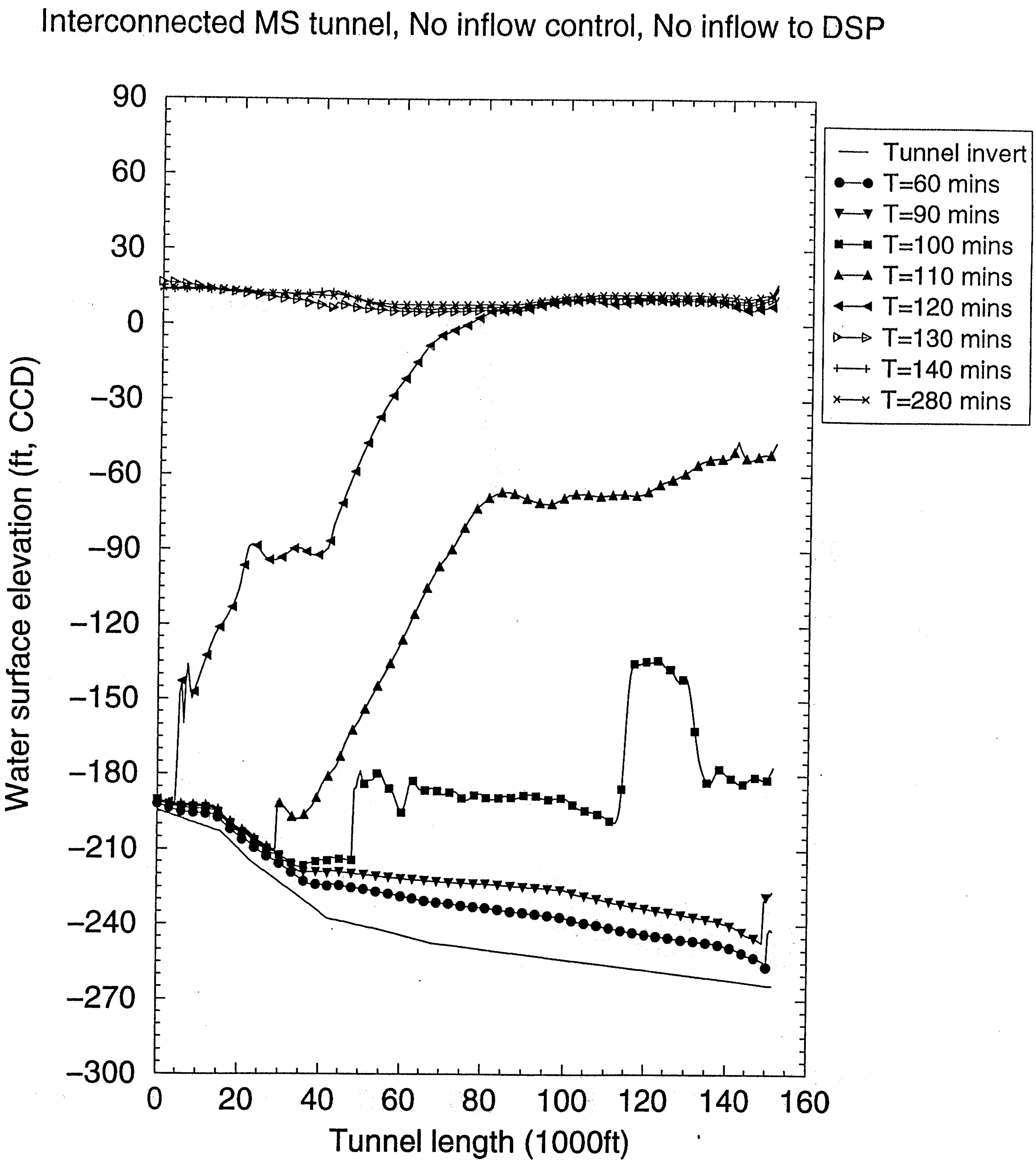

Fig. 19 Instantaneous hydraulic grade lines along the Mainstream tunnel, Case 3: interconnected tunnels, no inflow control, no wet weather inflow to the DSP tunnel.

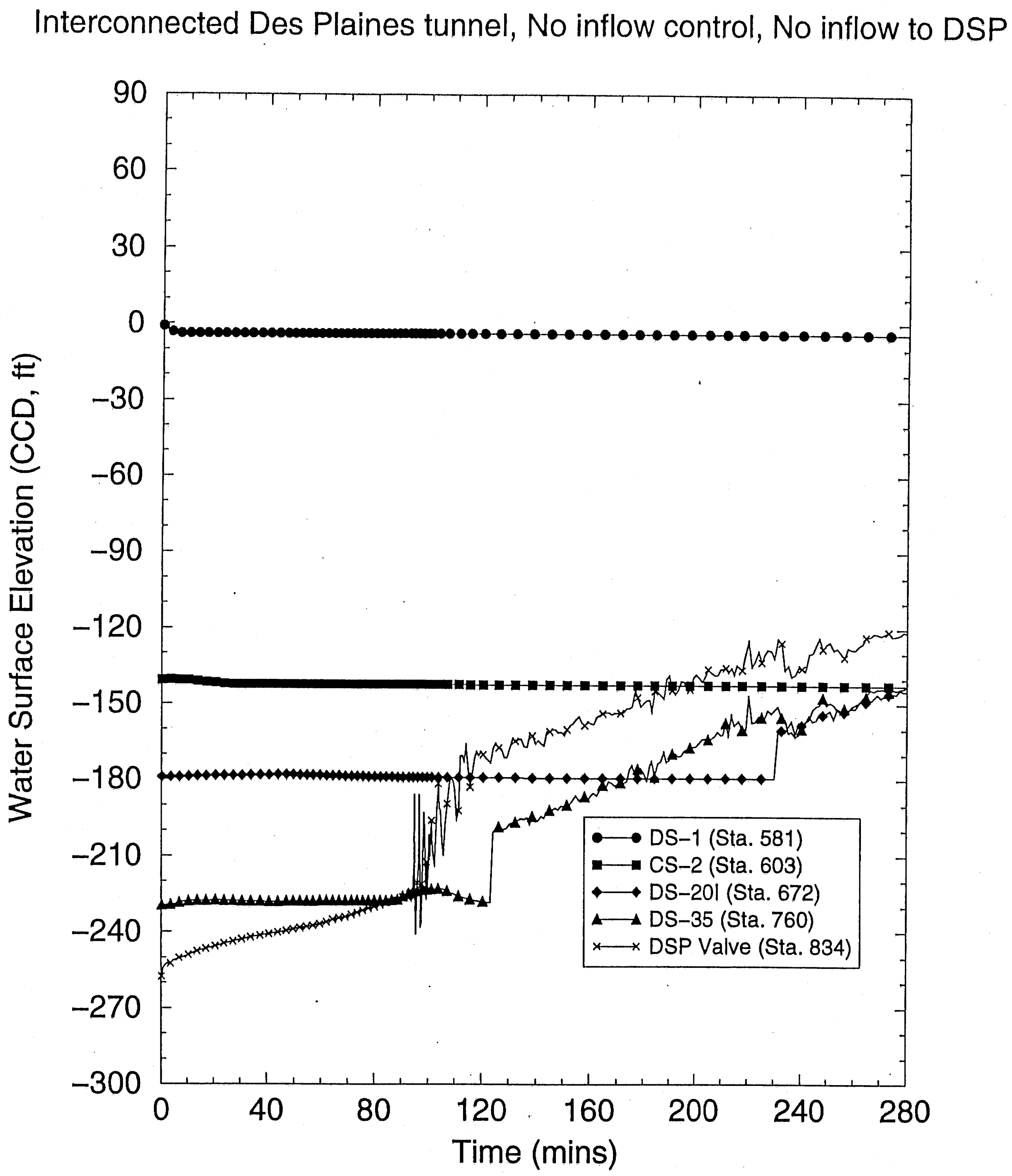

Fig. 20 Water elevation variations with time at 5 locations (DS-1, CS-2, DS20I, DS-35, and the DSP valve chamber) of the Des Plaines tunnel, Case 3: interconnected tunnels, no inflow control, no wet weather inflow to the DSP tunnel.

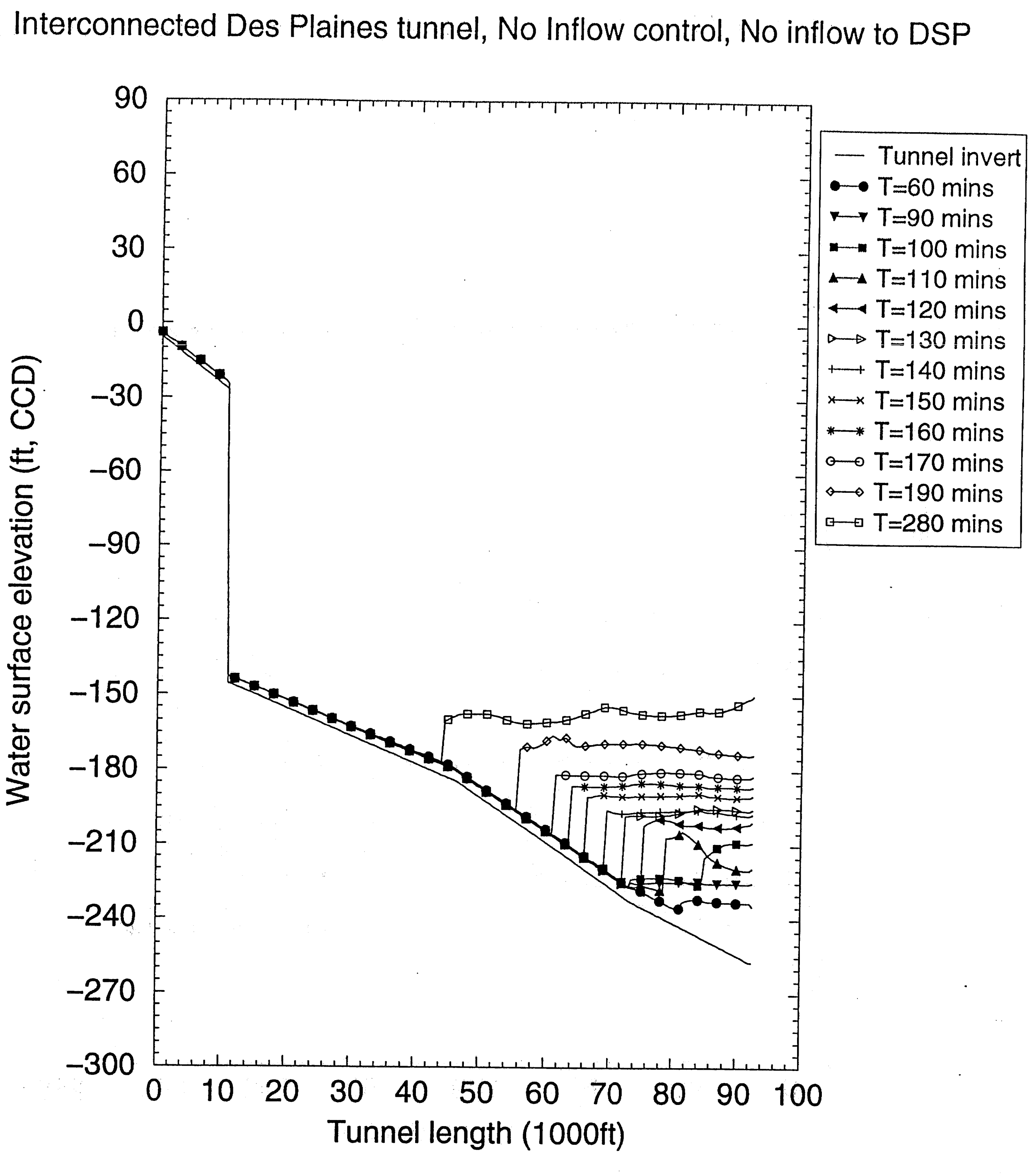

Fig. 21 Instantaneous hydraulic grade lines along the Des Plaines tunnel, Case 3: interconnected tunnels, no inflow control, no wet weather inflow to the DSP tunnel.

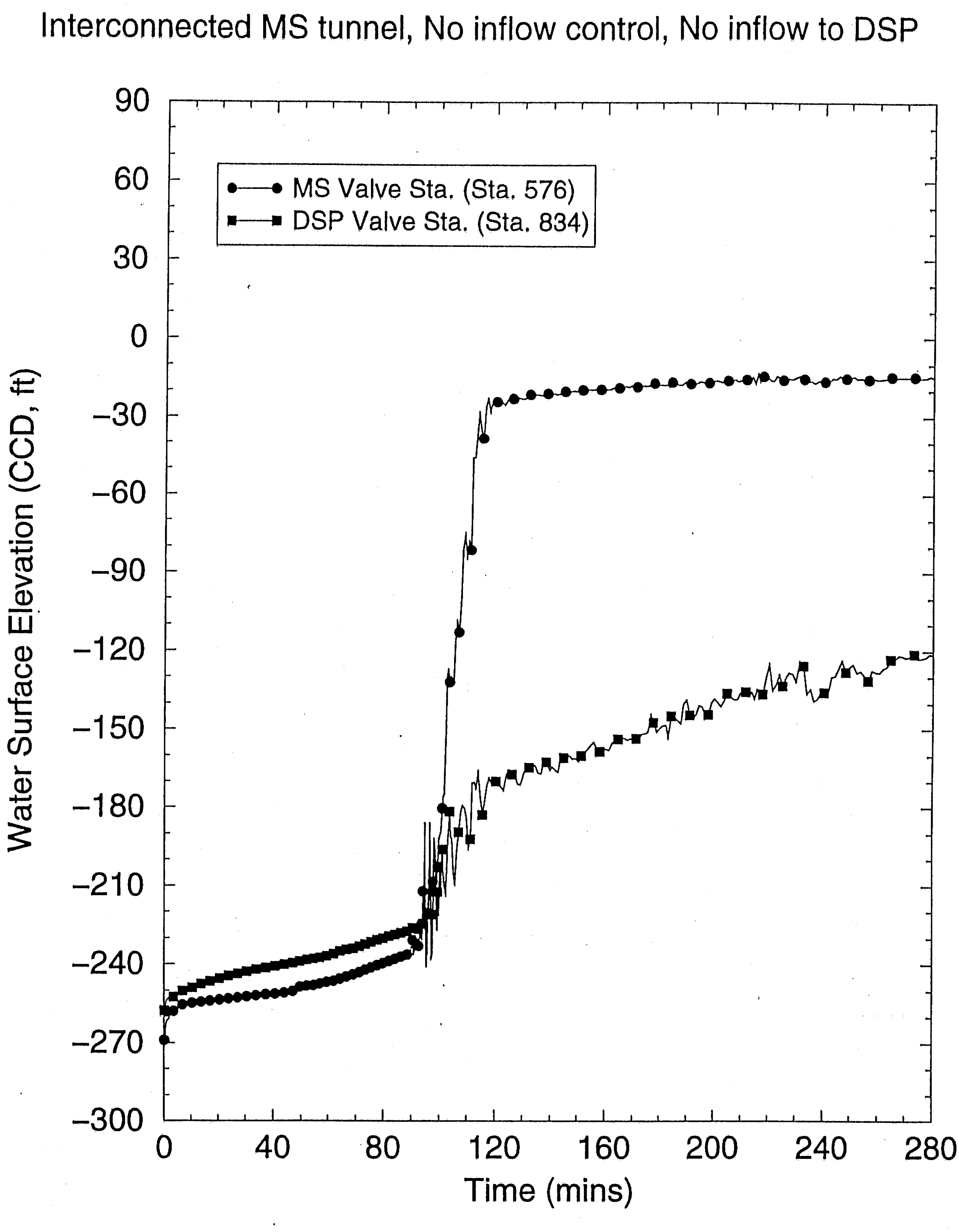

Fig. 22 Water elevation variations with time at the two valve chambers, Case 3: interconnected tunnels, no inflow control, no wet weather inflow to the DSP tunnel.

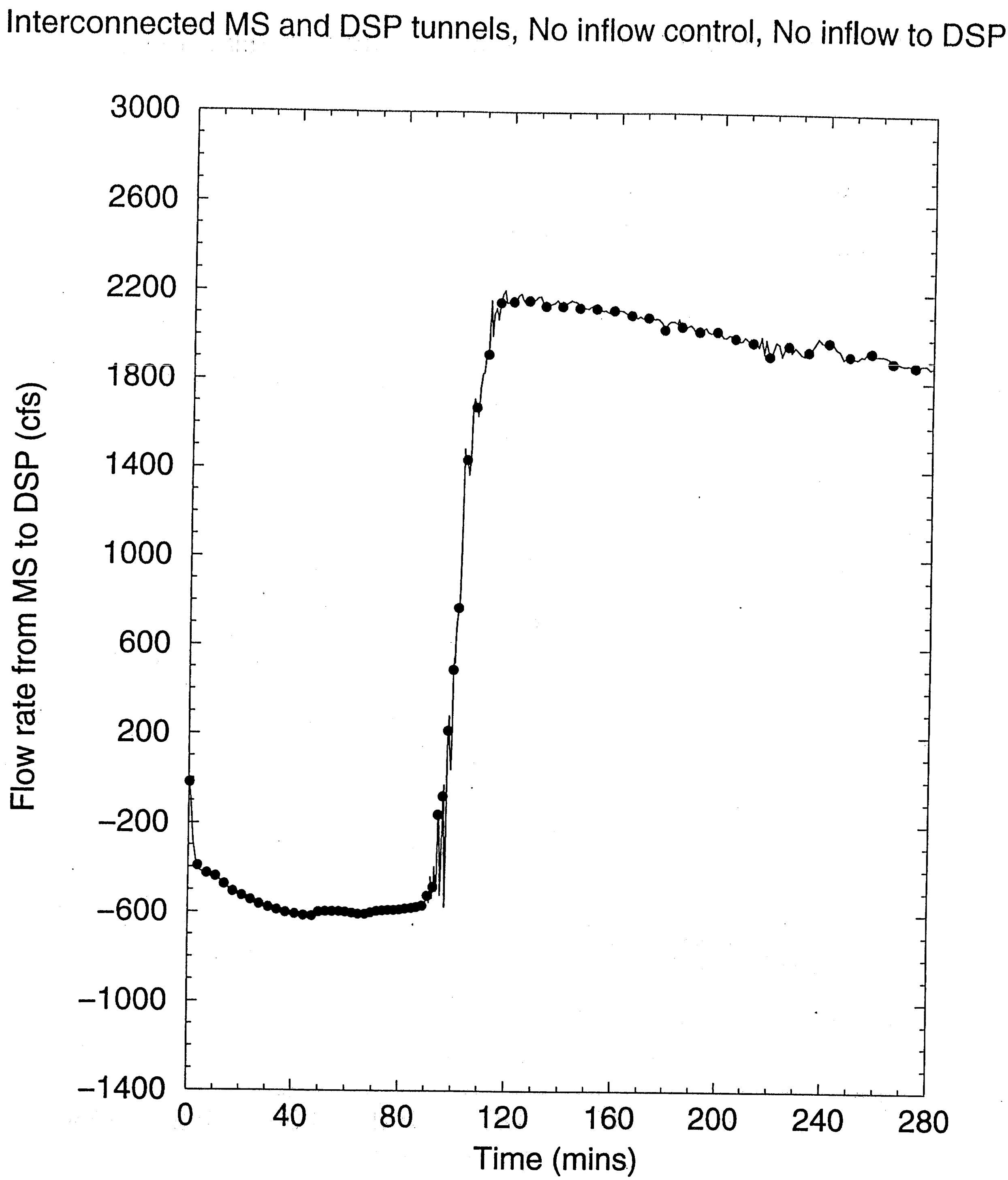

Fig. 23 Flow rate changes between the interconnected tunnels, Case 3: interconnected tunnels, no inflow control, no wet weather inflow to the DSP tunnel.

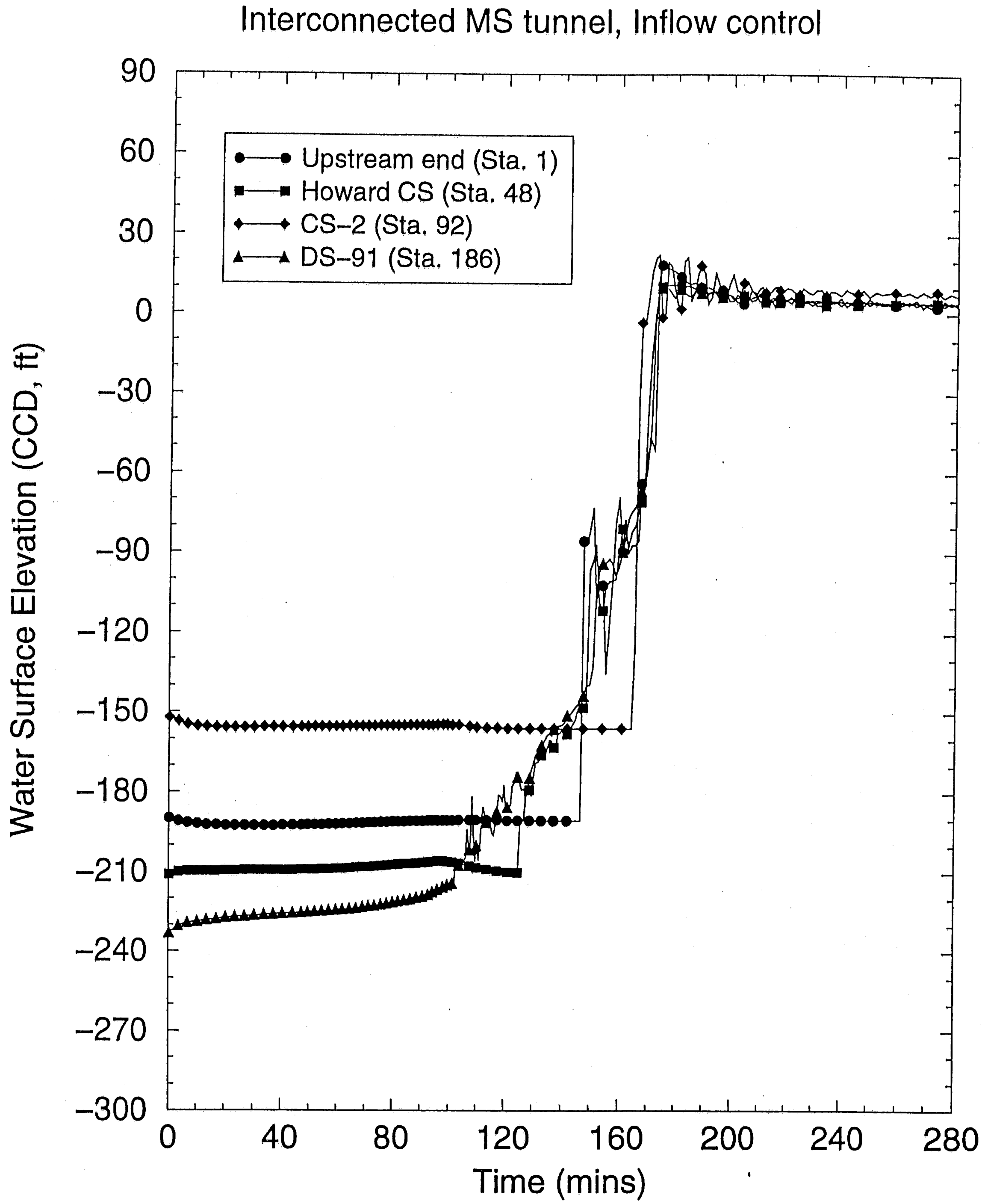

Fig. 24 Water elevation variations with time at 4 locations (the upstream end, Howard CS, CS-2, and DS-91) of the Mainstream tunnel, Case 4: interconnected tunnels, inflow control.

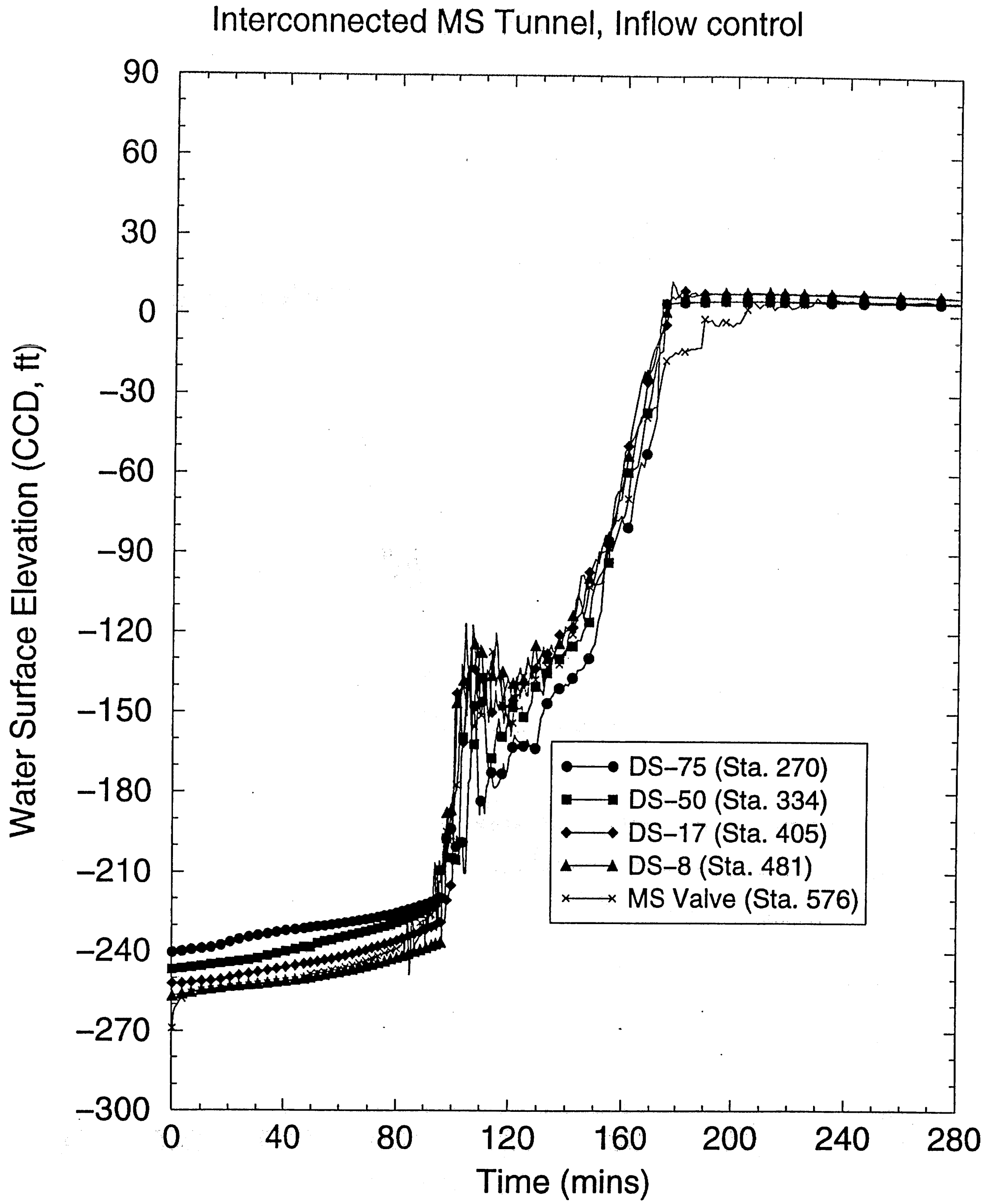

Fig. 25 Water elevation variations with time at 5 locations (DS-75, DS-50, DS-17, DS-8, and the MS valve chamber) of the Mainstream tunnel, Case 4: interconnected tunnels, inflow control.

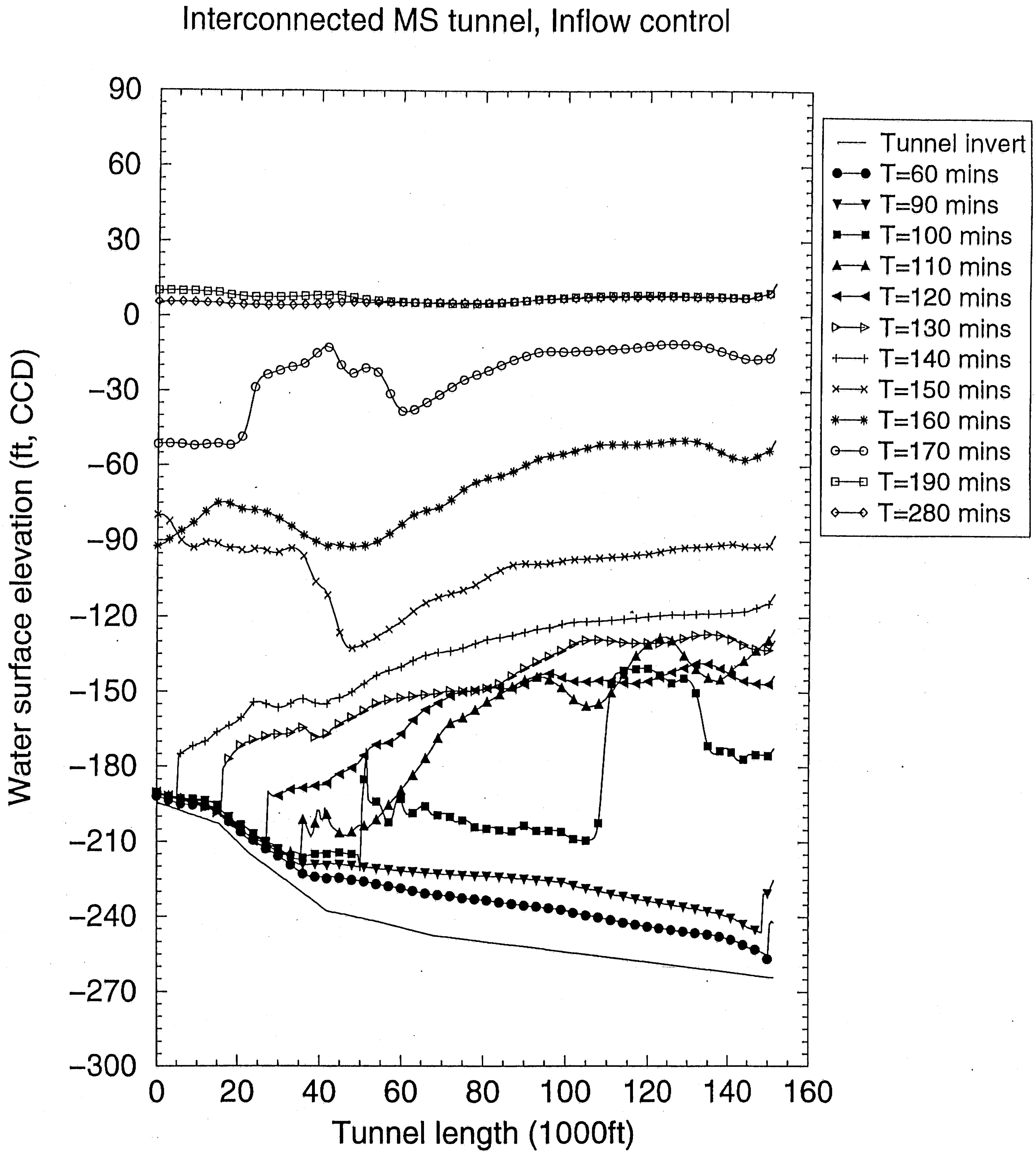

Fig. 26 Instantaneous hydraulic grade lines along the Mainstream tunnel, Case 4: interconnected tunnels, inflow control.

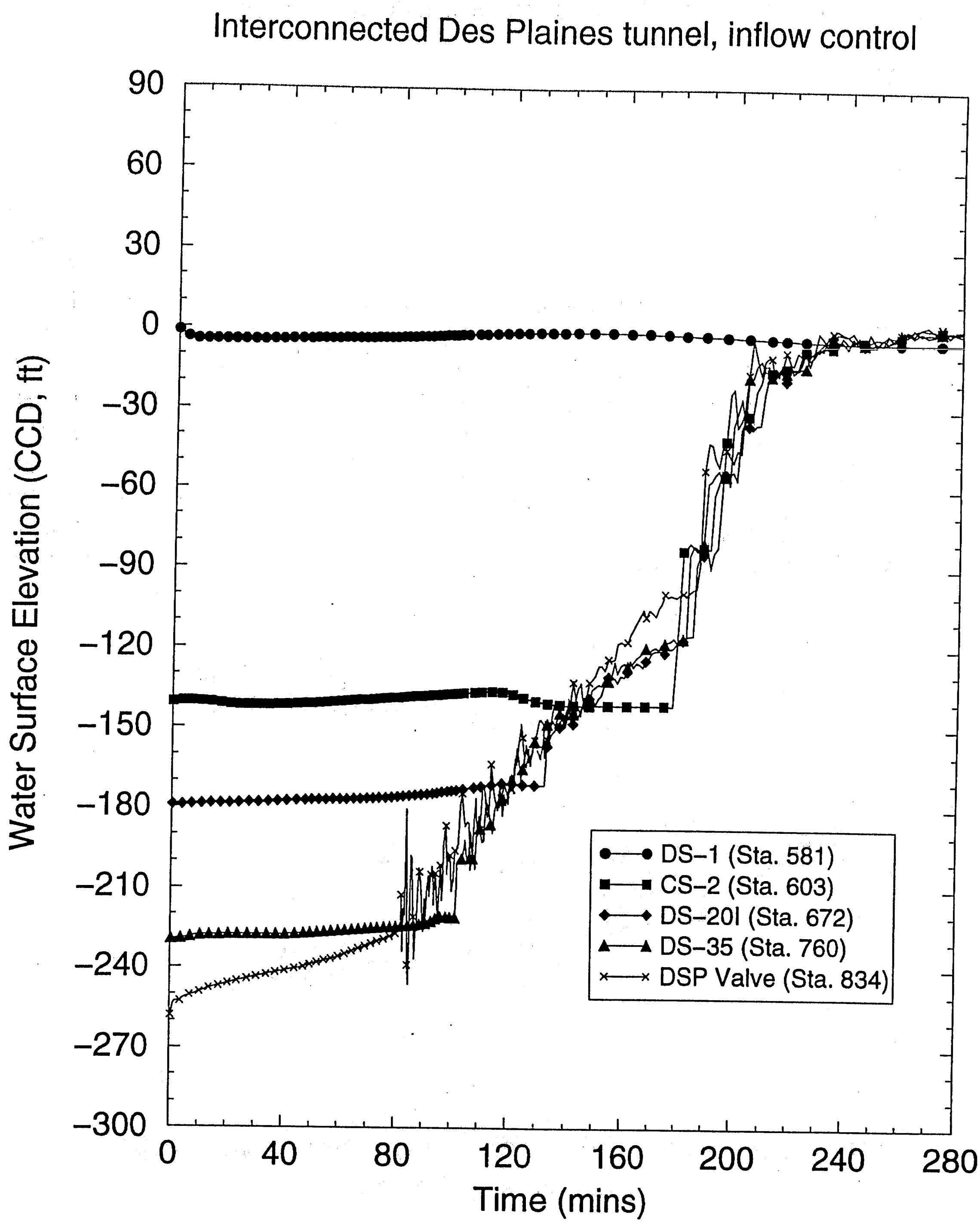

Fig. 27 Water elevation variations with time at 5 locations (DS-1, CS-2, DS20I, DS-35, and the DSP valve chamber) of the Des Plaines tunnel, Case 4: interconnected tunnels, inflow control.

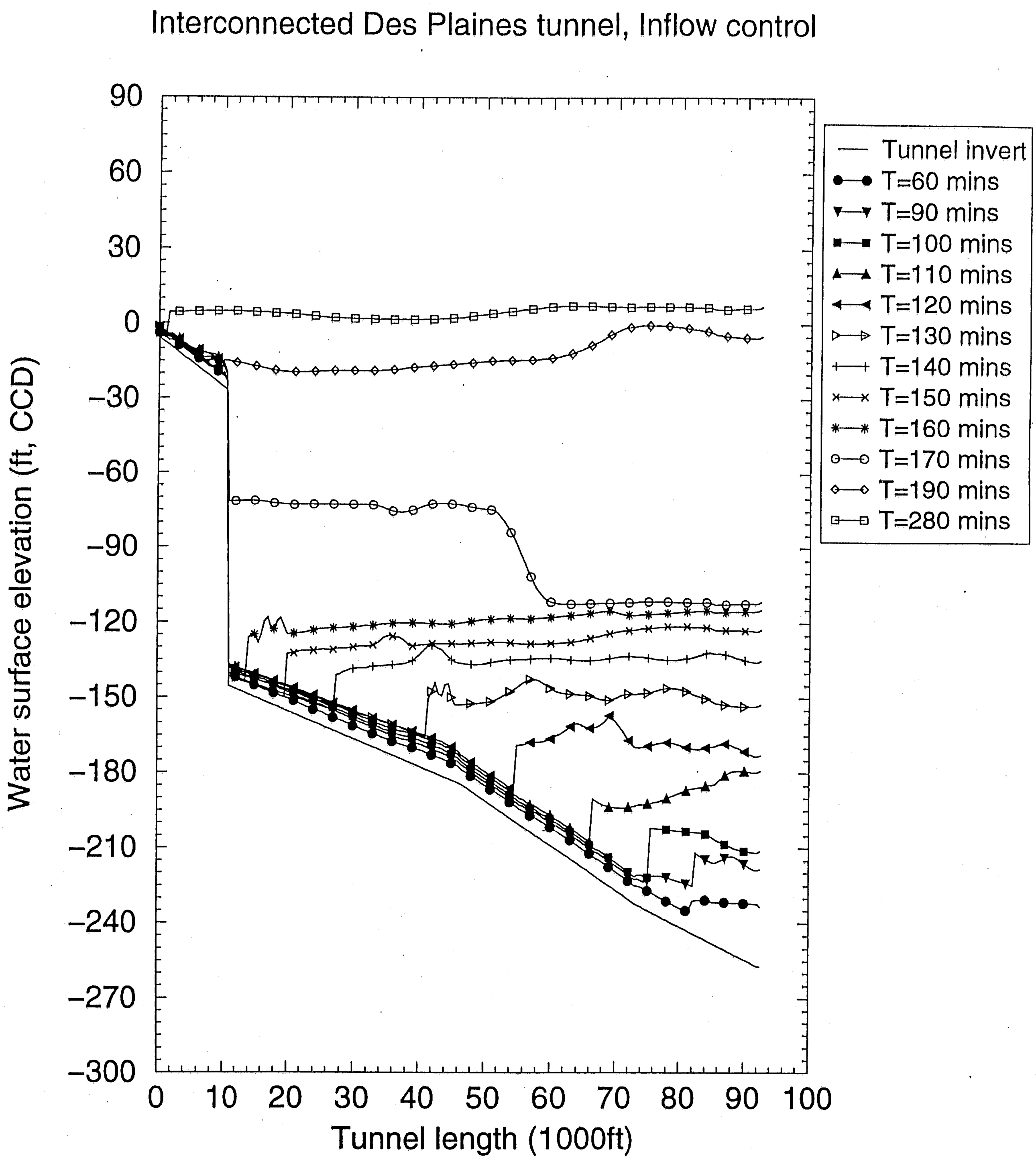

Fig. 28 Instantaneous hydraulic grade lines along the Des Plaines tunnel, Case 4: interconnected tunnels, inflow control.

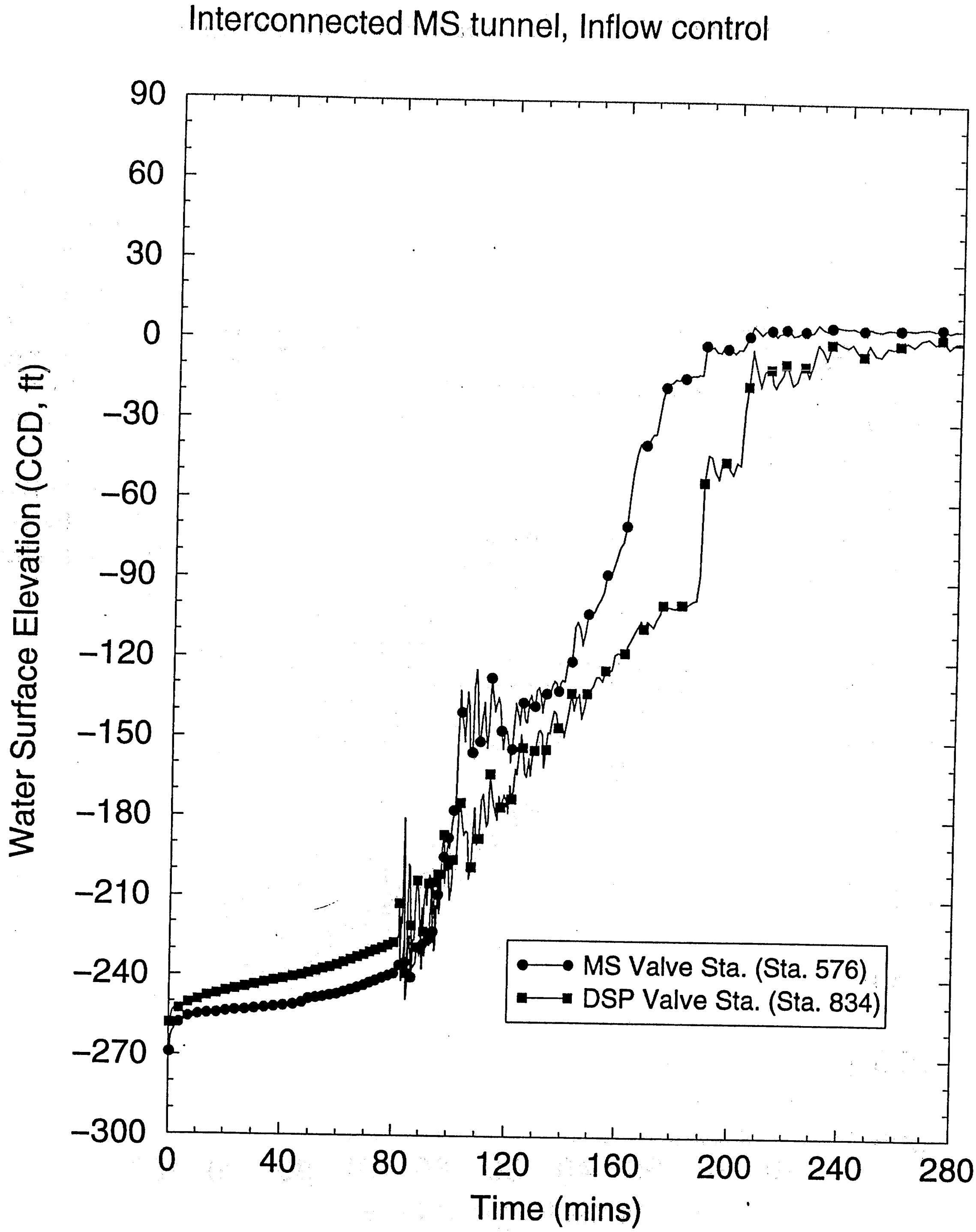

Fig. 29 Water elevation variations with time at the two valve chambers, Case 4: interconnected tunnels, inflow control.

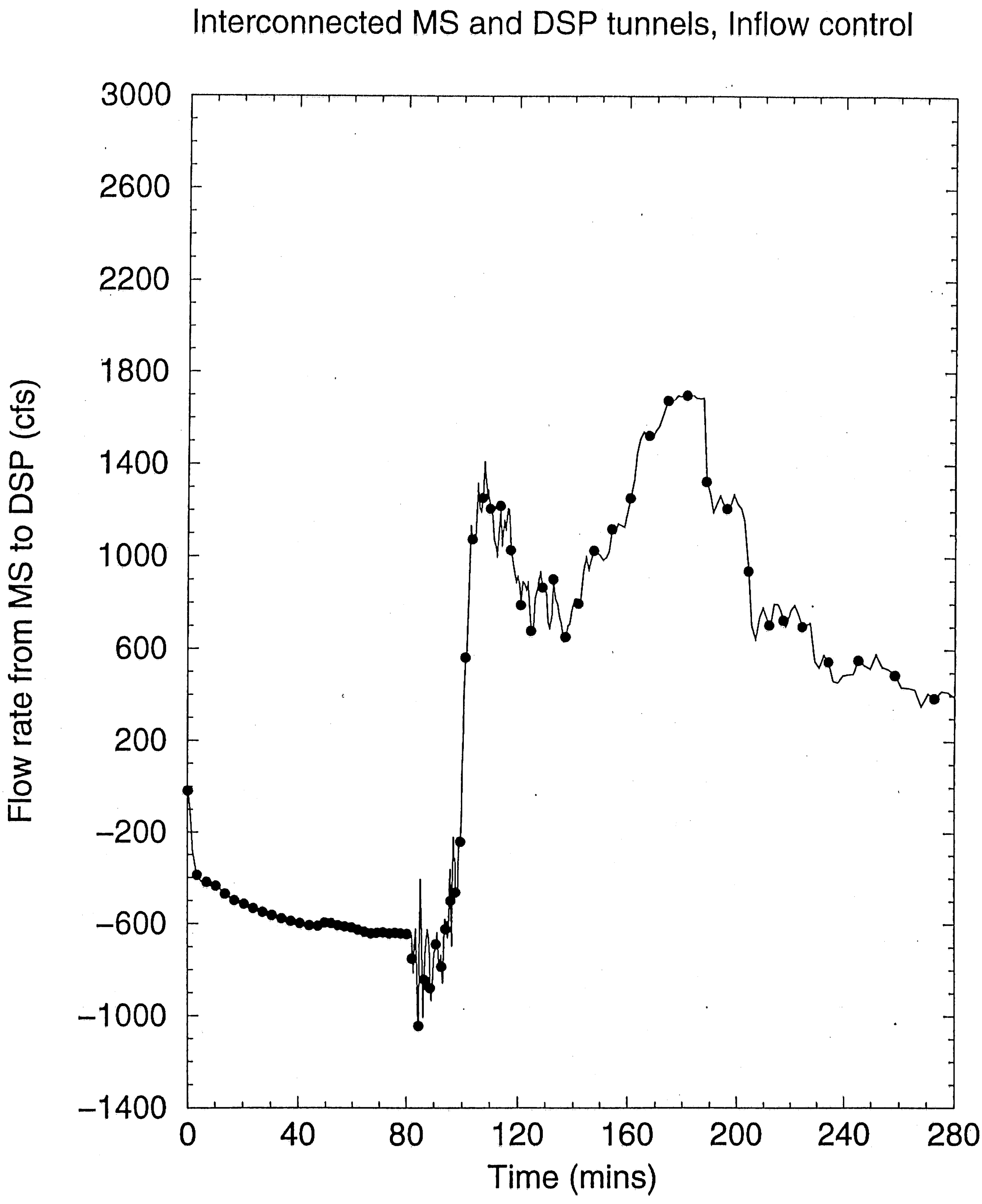

Fig. 30 Flow rate changes between the interconnected tunnels, Case 4: interconnected tunnels, inflow control.

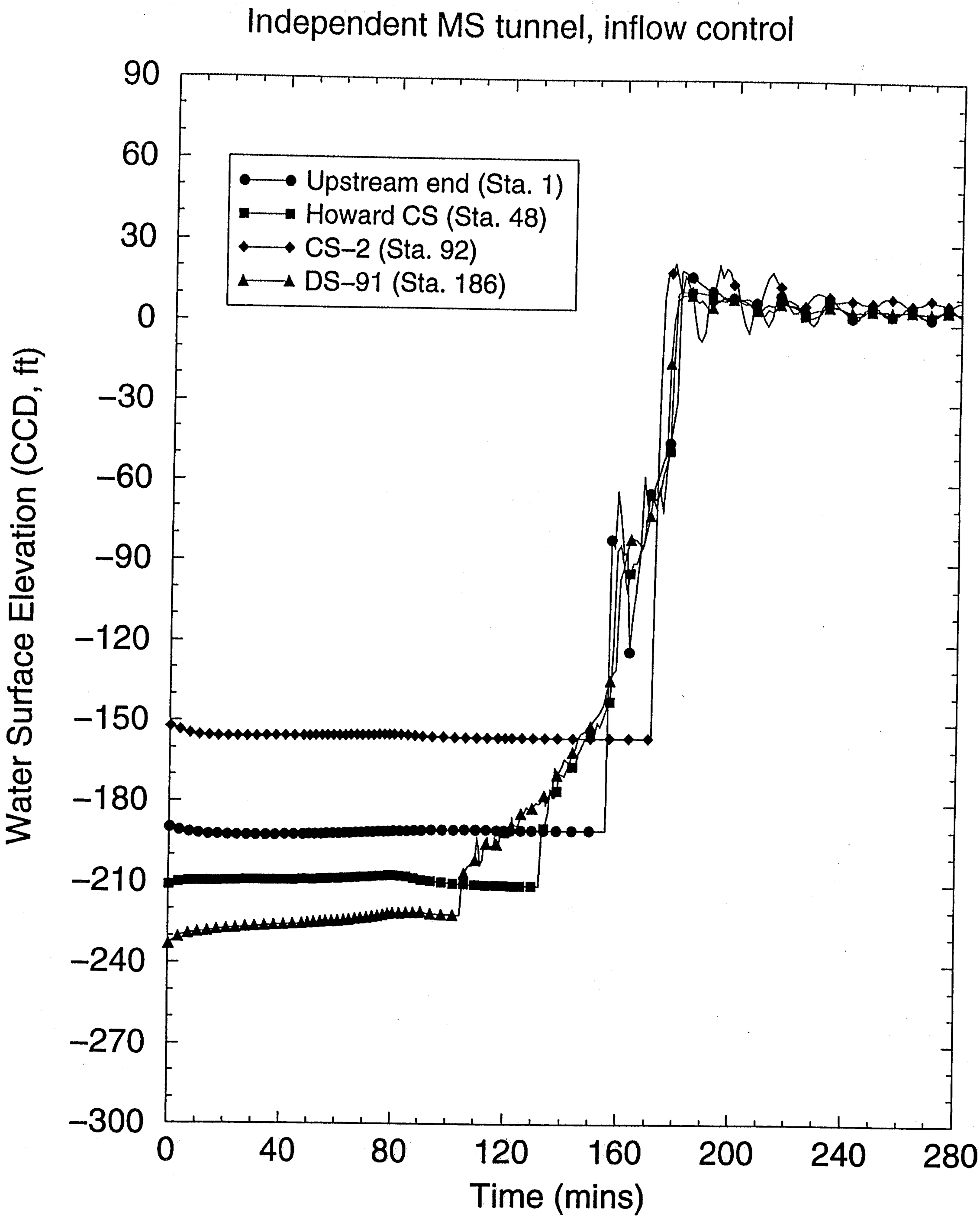

Fig. 31 Water elevation variations with time at 4 locations (the upstream end, Howard CS, CS-2, and DS-91) of the Mainstream tunnel, Case 5: independent tunnels, inflow control.

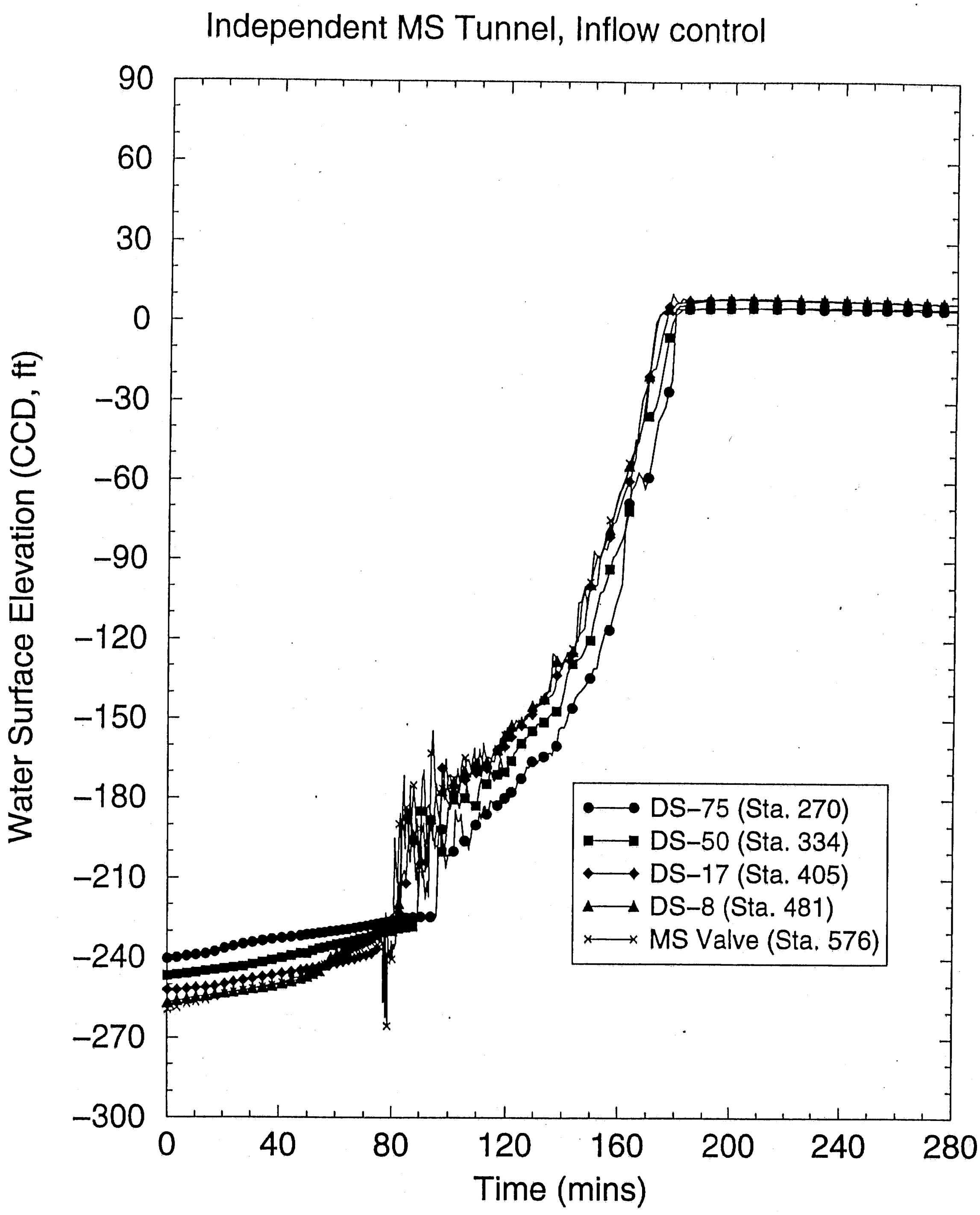

Fig. 32 Water elevation variations with time at 5 locations (DS-75, DS-50, DS-17, DS-8, and the MS valve chamber) of the Mainstream tunnel, Case 5: independent tunnels, inflow control.

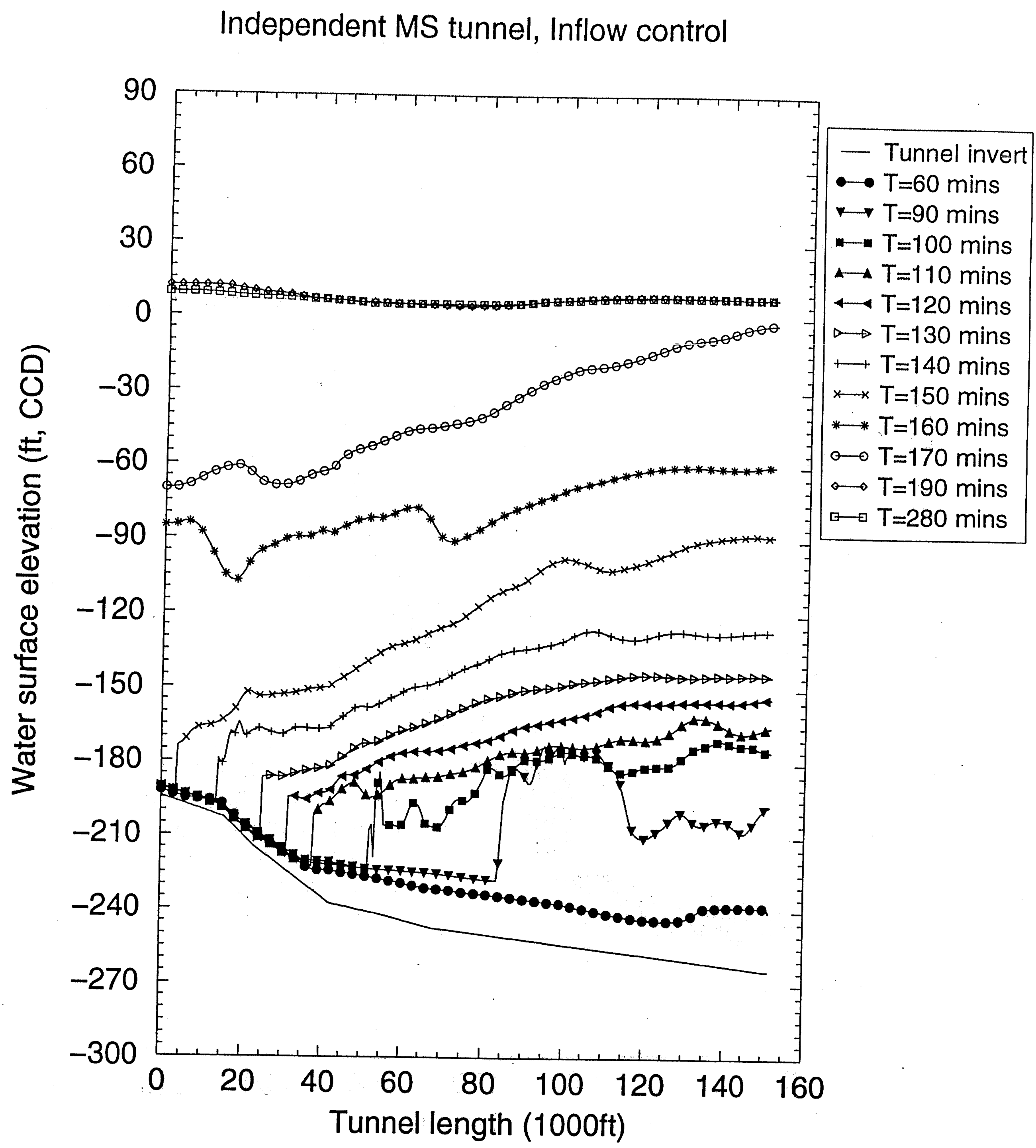

Fig. 33 Instantaneous hydraulic grade lines along the Mainstream tunnel, Case 5: independent tunnels, inflow control.

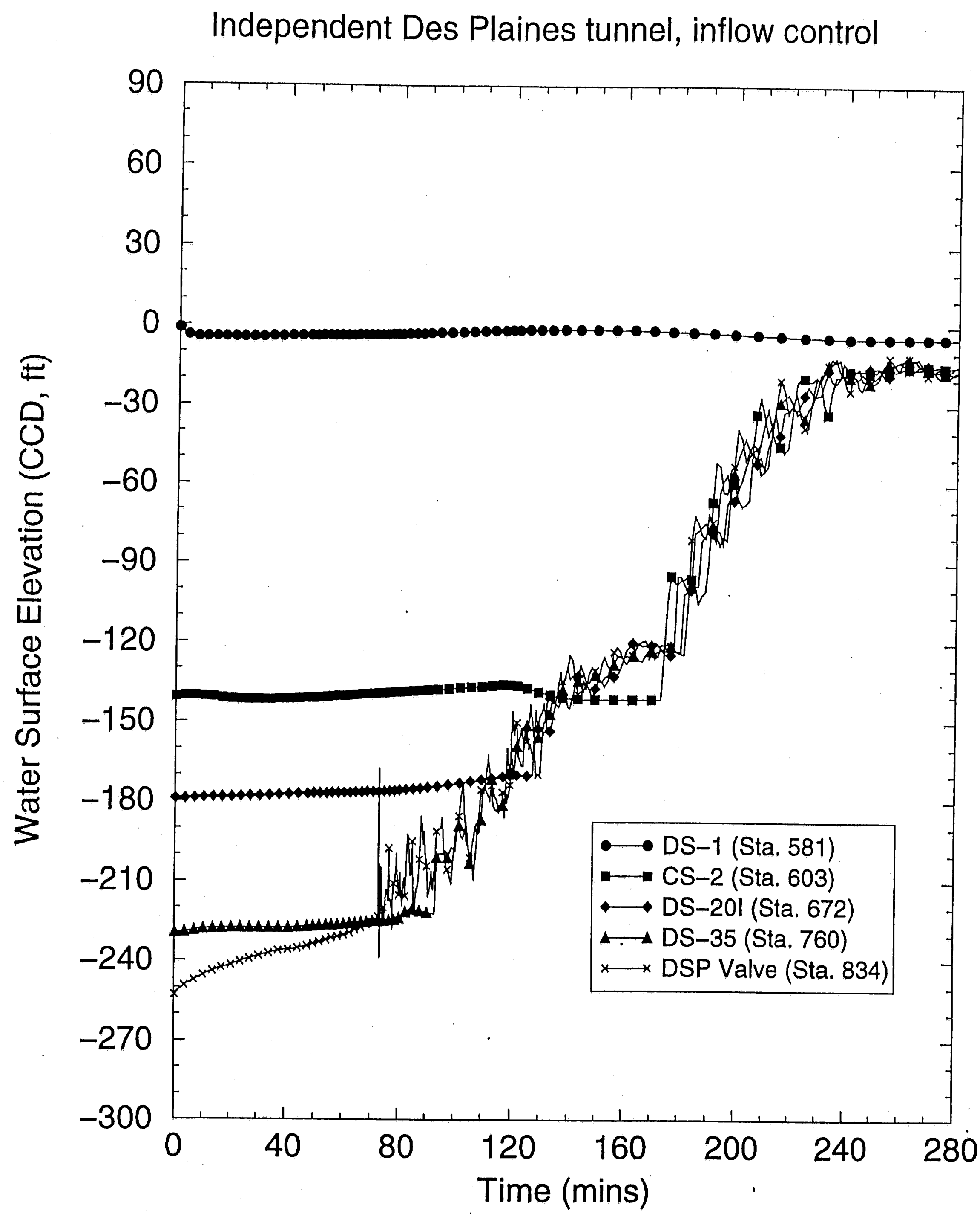

Fig. 34 Water elevation variations with time at 5 locations (DS-1, CS-2, DS20I, DS-35, and the DSP valve chamber) of the Des Plaines tunnel, Case 5: independent tunnels, inflow control.

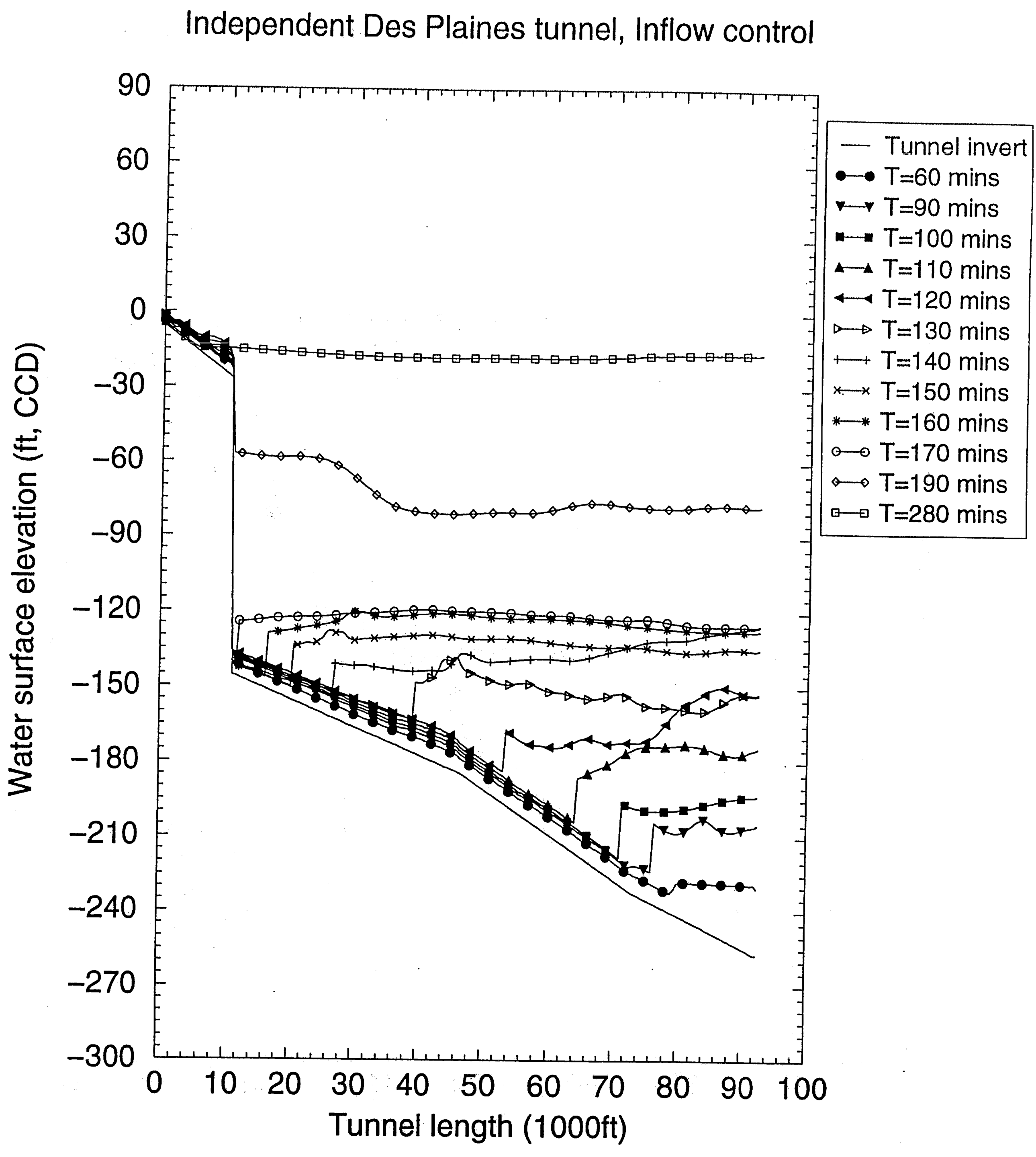

Fig. 35 Instantaneous hydraulic grade lines along the Des Plaines tunnel, Case 5: independent tunnels, inflow control.

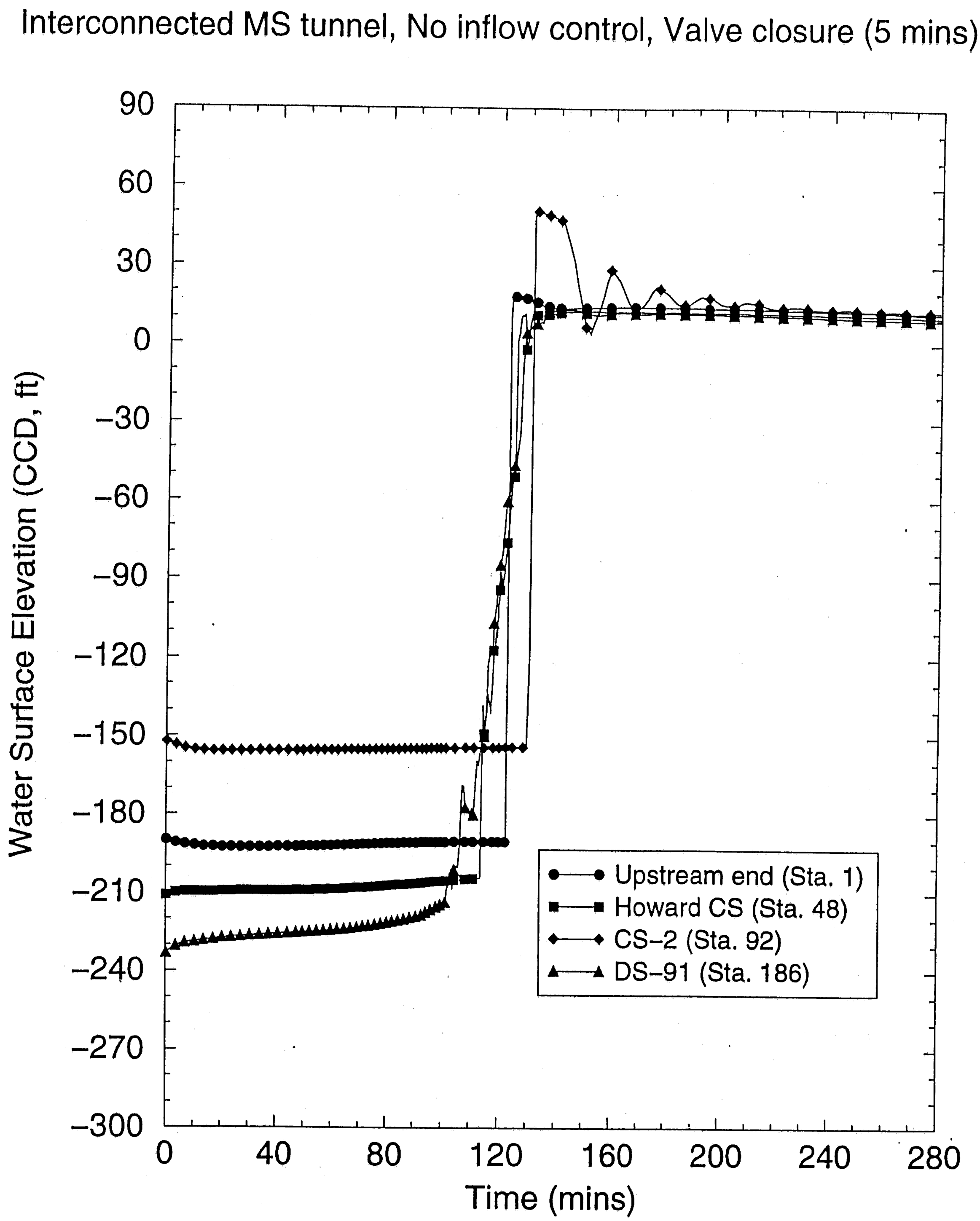

Fig. 36 Water elevation variations with time at 4 locations (the upstream end, Howard CS, CS-2, and DS-91) of the Mainstream tunnel, Case 6: interconnected tunnels, no inflow control, valve closure in 5 mins.

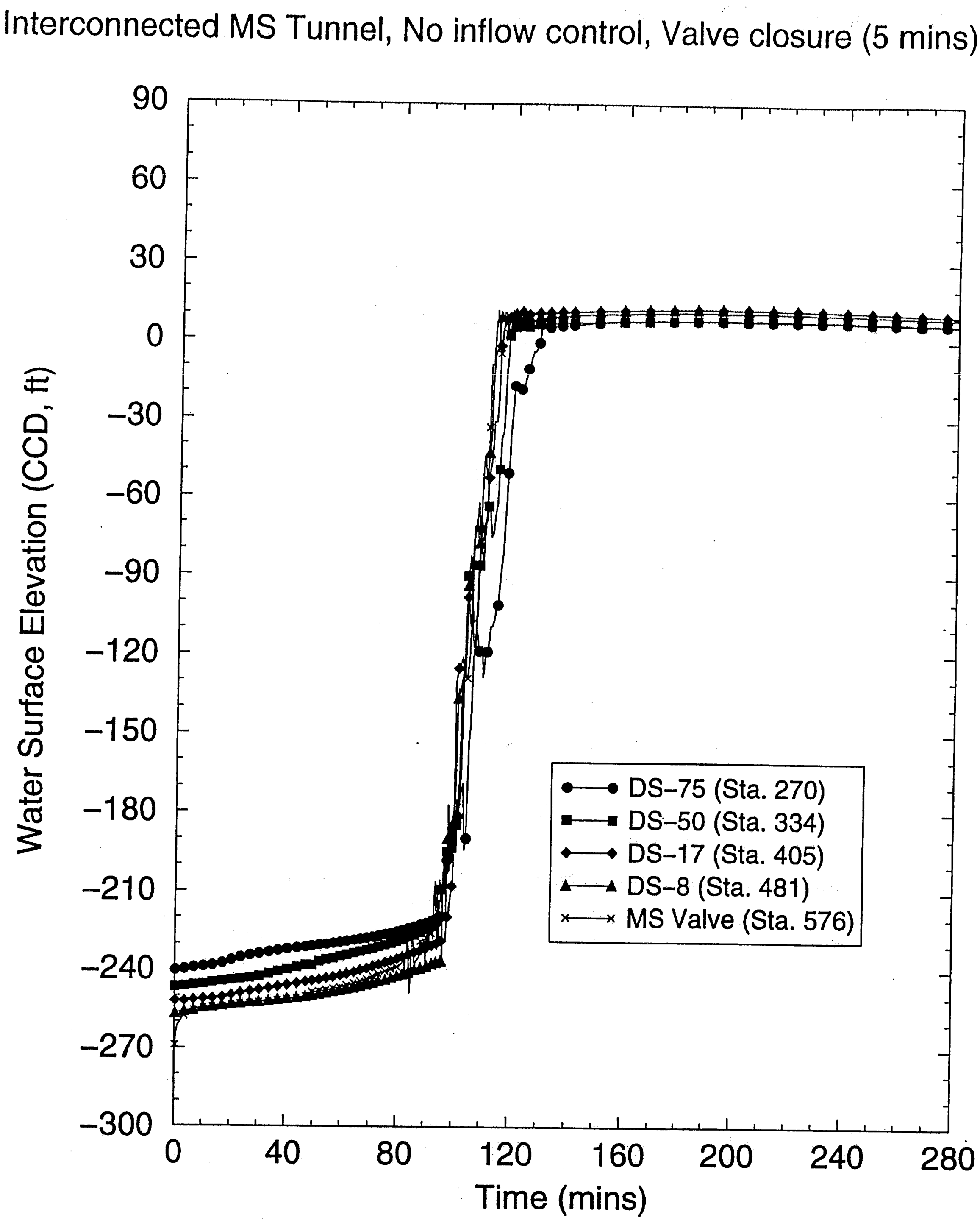

Fig. 37 Water elevation variations with time at 5 locations (DS-75, DS-50, DS-17, DS-8, and the MS valve chamber) of the Mainstream tunnel, Case 6: interconnected tunnels, no inflow control, valve closure in 5 mins.

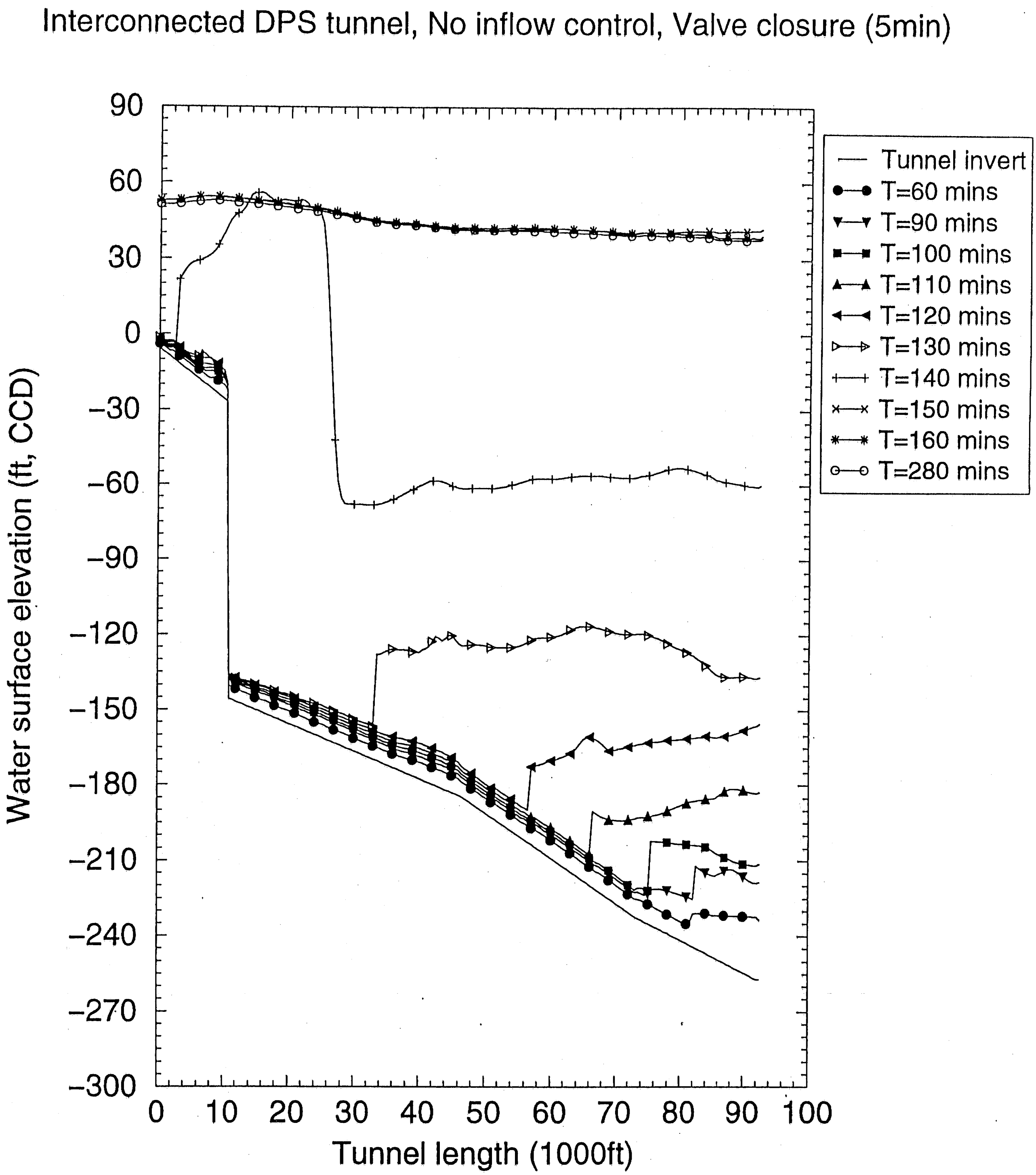

Fig. 38 Instantaneous hydraulic grade lines along the Mainstream tunnel, Case 6: interconnected tunnels, no inflow control, valve closure in 5 mins.

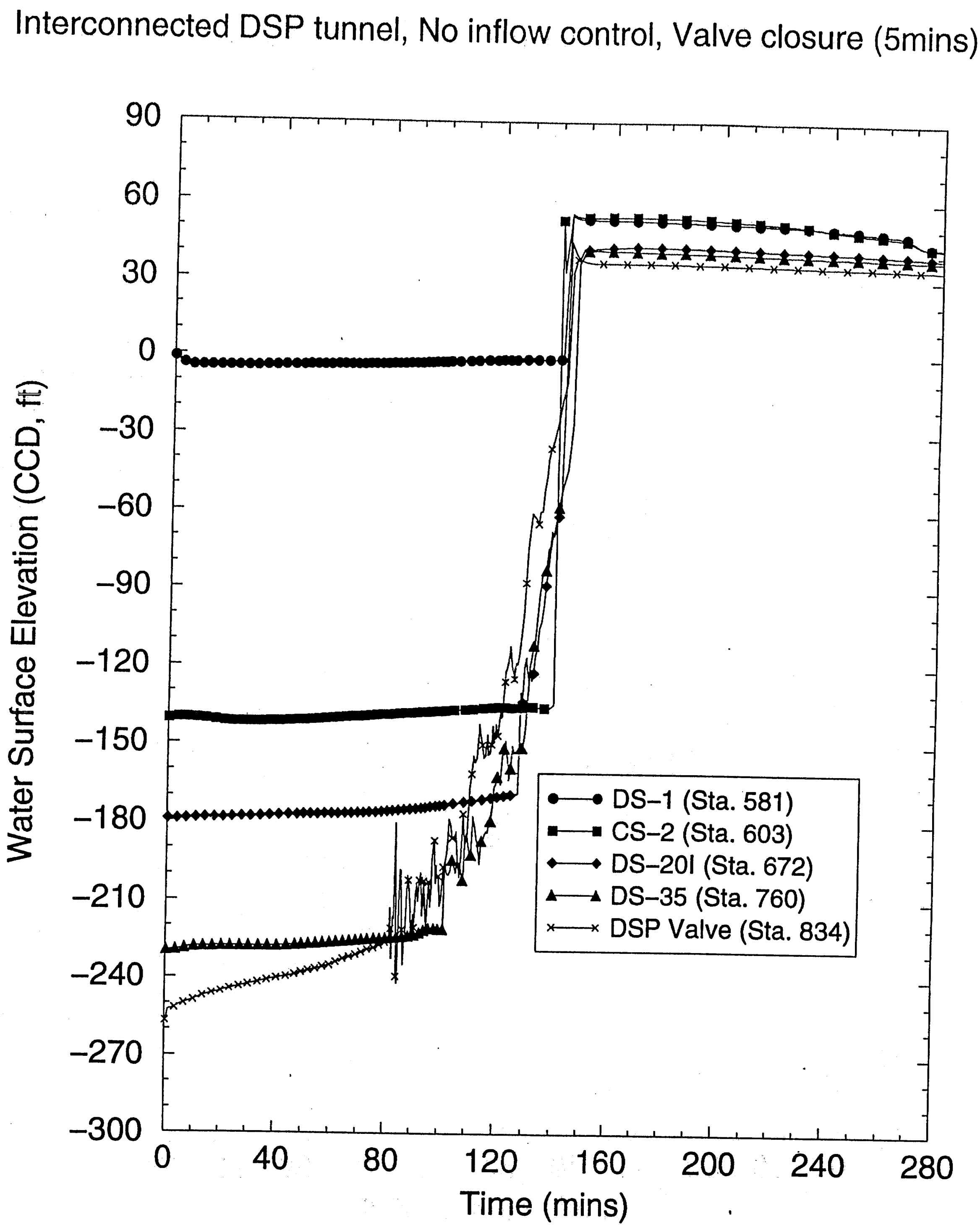

Fig. 39 Water elevation variations with time at 5 locations (DS-1, CS-2, DS20I, DS-35, and the DSP valve chamber) of the Des Plaines tunnel, Case 6: interconnected tunnels, no inflow control, valve closure in 5 mins.

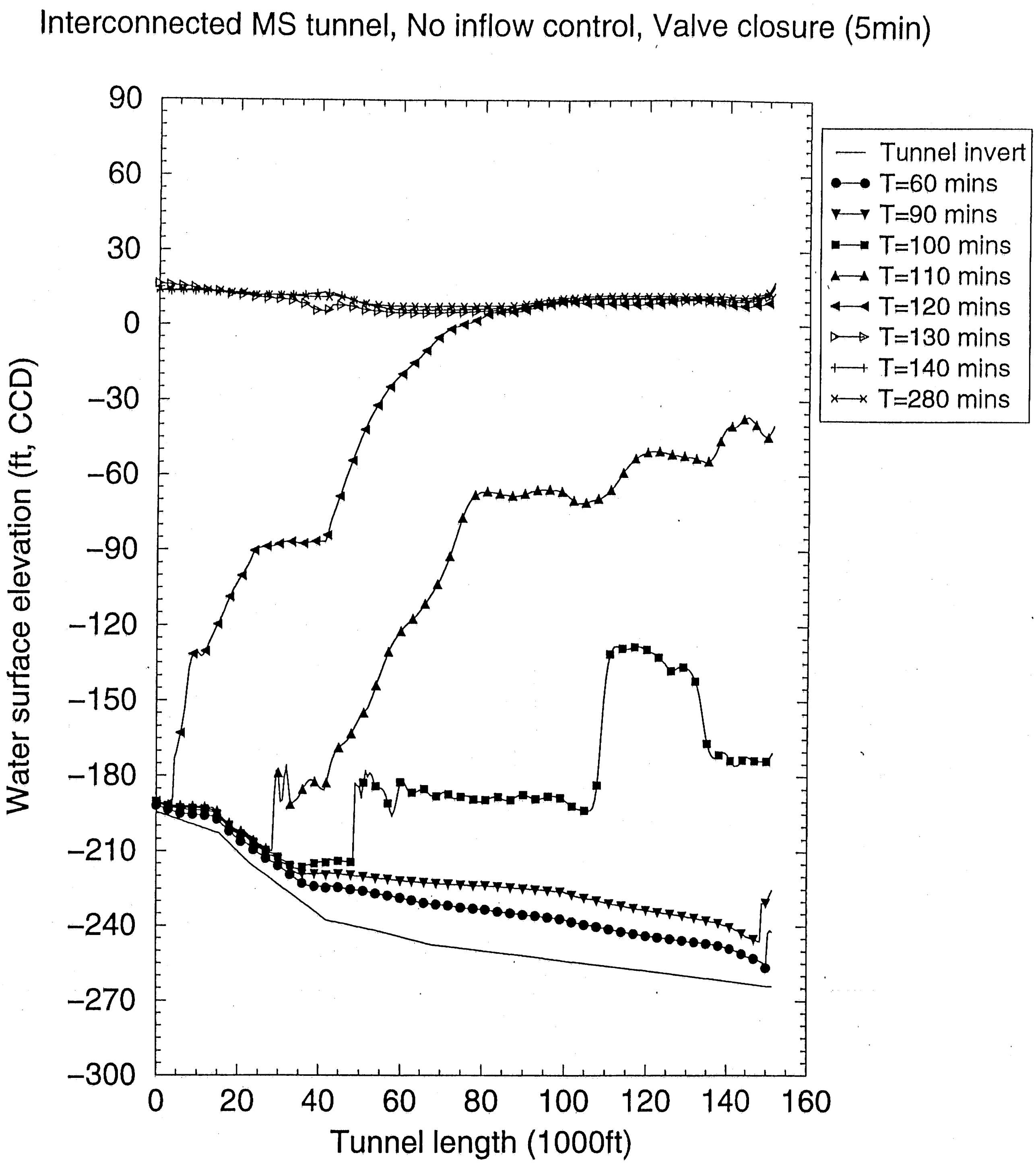

Fig. 40 Instantaneous hydraulic grade lines along the Des Plaines tunnel, Case 6: interconnected tunnels, no inflow control, valve closure in 5 mins.

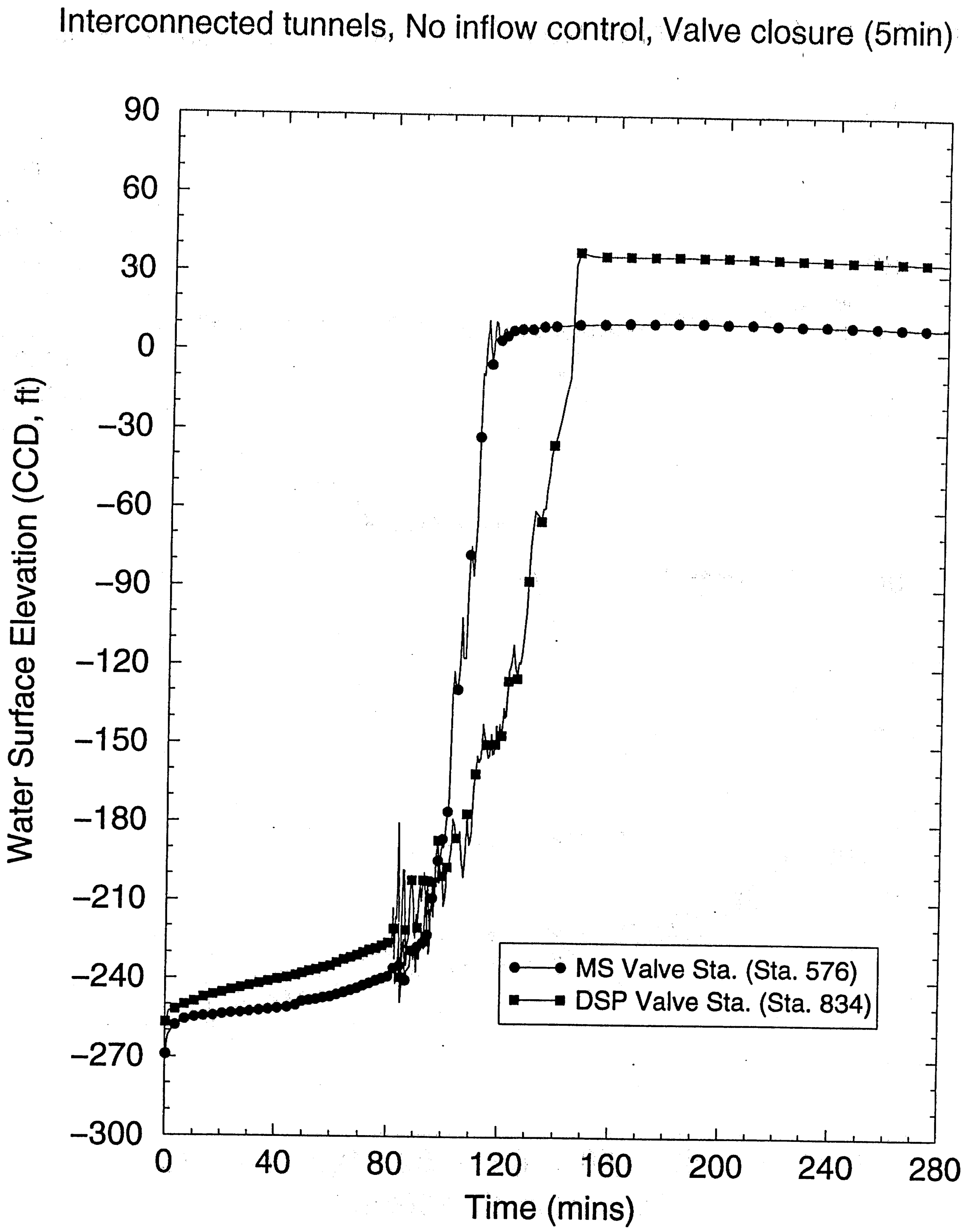

Fig. 41 Water elevation variations with time at the two valve chambers, Case 6: interconnected tunnels, no inflow control, valve closure in 5 mins.

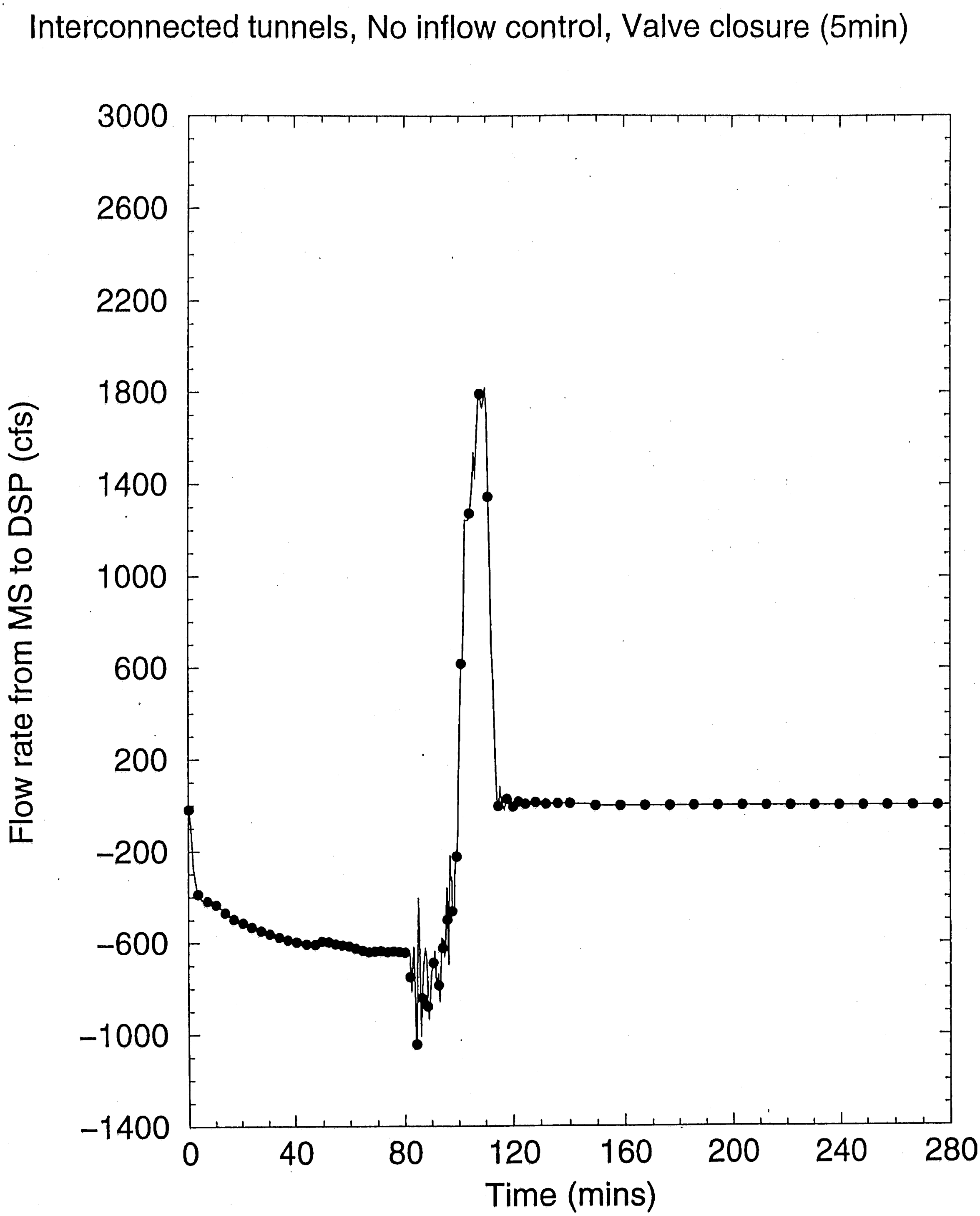

Fig. 42 Flow rate changes between the interconnected tunnels, Case 6: interconnected tunnels, no inflow control, valve closure in 5 mins.