Application of the cdg1-D model in the lower Athabasca River basin to estimate high flows during open-water season.

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APPLICATION OF THE cdg1-D MODEL IN THE LOWER ATHABASCA RIVER BASIN TO ESTIMATE HIGH FLOWS DURING OPEN-WATER SEASON

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Table of Contents

List of Tables

LIST OF FIGURES

1.0 Introduction

Floods have been and still are one of the biggest threats to life, the infrastructure and the environment of any community. That is why it is important to know in advance the amount of damage a flood will cause to the surrounding community near the river. This can be achieved by using either a hydrologic or hydraulic flood routing technique. Hydrologic techniques are based on the conservation of mass and an empirical approximation of the longitudinal momentum. This technique is less expensive and easier to use than the hydraulic method. However, the output obtained by these models is limited only to the calibration points where stream gauge exists. Conversely, hydraulic models are deterministic based on physical laws and data. These types of models are not as simple since a complicated mathematical model of the mass and momentum conservation along with the geometry and resistance of the reach is needed (Hicks 1996). Even though hydraulic models are complicated, outputs for intermediate points or nodes can be obtained by routing floods.

In this project, the development of a hydraulic routing model for a section of the northern part of the Athabasca River from Fort McMurray to Embarras was performed. The reach was modeled by using the cdg1-D methodology developed at the Civil Engineering Department, University of Alberta (Hicks and Steffler, 1995). A limited geometry model was used involving rectangular approximations of the channel geometry obtained from National Topographic Series (NTS) maps. This was done because of the lack of natural geometry available for the reach. The main purpose of the project was to investigate the performance of the model using this limited geometrical information.

2.0 The cdg1-D Model

The cdg1-D model solves the St. Venant equations by the finite element method using the characteristic dissipative Galerkin (cdg) scheme (Hicks and Steffler 1995). The finite element scheme solves a conservation form of the equations resulting in optimal accuracy and is also quite stable numerically for dynamic events (Hicks and Steffler 1990). For detailed solution methodology refer to the papers by Hicks (1995, 1996, 1997).

3.0 Description of the Study Reach

The study reach is located in northeastern Alberta, at a longitude of approximately 111° 29' *W* and a latitude of 58 ° 43' *N* (Figure 1). The study reach is part of the Athabasca River which flows northward to the Peace Athabasca Delta. The study reach from Fort McMurray to Embarras Airport extends a total length of approximately 189.5 km. At Fort McMurray the Athabasca River meets with the Clearwater River, a major tributary. The catchment basin is comprised mainly of moderately forested lowlands, some muskeg and virtually no cultivation. Some sections of the river can be considered straight, but most of the reach is irregular meandering with occasional islands, point bars and mid-channel bars. The channel is entrenched and possesses good lateral stability. The river bed for the reach is composed of shallow sand with local gravel over limestone (Kellerhals, Neil and Bray 1972).

4.0 Available Geometry (Cross-sections)

All of the geometry utilized to model the study reach was derived from 1:50,000 scale NTS maps. As mentioned earlier the model was developed using a rectangular channel approximation. With this simplification the only channel geometry requirements at a particular section were the width of the channel and the mean bed level. The natural cross-sections were used to obtain an approximation of the actual position of the river bed and water levels with respect to geodetic datum. The natural cross-sections were obtained in digital form from Dr. Faye Hicks from the University of Alberta; they were based on cross-section surveys conducted by the Alberta Research Council (ARC) in 1970. The availability of natural cross-sections were limited and were available for the reach from Fort McMurray to Ings Island (Figure 2). Most of the cross-sections used were never actually tied-in to geodetic elevations, ARC estimated geodetic elevation based on water surface profiles measured at the time of surveys. The earlier studies (Blackburn and Hicks, 2001) have shown that excellent flood routing results could be obtained by using limited channel geometry. It appears that, for accurate discharge predictions, the model only requires accurate information regarding channel gradient. Accurate channel geometry, e.g., cross-section shape was only required for obtaining accurate water level predictions (Blackburn and Hicks, 2001), which would be further investigated in the following sections.

5.0 Development of the data base (for Limited Geometry)

The study reach extends from the Water Survey of Canada (WSC) gauge station 07DA001 below Fort McMurray to Lake Athabasca 07MD001, approximately 139.3 km past the intermediate gauged site at Embarras Airport - 07DD001. The boundary conditions were considered at gauge sites 07DA007 below Fort McMurray and 07MD001 Athabasca Lake at Fort Chipewyan. The intermediate gauged site 07DD001 at Embarras Airport was used for calibration purposes. Refer to Appendix B for detailed information to prepare input data for running the cdg1-D model.

5.1 Channel Discretization

The study reach was discretized on the NTS maps creating node points at a spacing of one kilometer intervals. Earlier investigations (Hicks, 1996) have determined that this was an optimal spacing for moderately sized rivers such as Athabasca River. The sections were laid out on 1:50,000 NTS maps at one kilometer intervals. The starting point was arbitrarily chosen to be near the town of Fort McMurray at the (WSC) gauge station 07DA001 located 17.5km downstream.

The channel distances were measured along the center line of the channel, since little natural geometry data was available. The gauged tributaries and significant site such as Ings island (change in water surface slope) were identified on the NTS maps and all linear distances were measured from the town of Fort McMurray (Table 1).

Table 1: Gauging stations located in the study area of the Athabasca River Basin.

WSC-ID	Name of Gauging Stations	Record Length (years)	Linear Distances (km)	Remarks
07DA001	Athabasca River below Fort McMurray	1957 - 2000	17.5	Gauge station (boundary Upstream)
07DA007	Poplar Creak Near Fort McMurray	1972 - 1986	33.8	Gauge station (inflow tributary)
07DA006	Steepbank River Near McMurray	1972 - 2000	44.8	Gauge station (inflow tributary)
07DA008	Muskeg River Near Fort MacKay	1974 - 2000	59.8	Gauge station (inflow tributary)
07DB001	MacKay River Near Fort MacKay	1972 - 2000	63.8	Gauge station (inflow tributary)
07DA017	Ells River Near the Mouth	1974 - 1986	80.8	Gauge station (inflow tributary)
07DC001	Firebag River Near the Mouth	1971 - 2000	135.8	Gauge station (inflow tributary)
07DD001	Athabasca River at Embarras Airp.	1971 - 1984	189.5	Gauge station (needed for outputs)
07MD001	Lake Athabasca at Fort Chipewyan	1930 - 2000	328.8	Gauge station (boundary Downstream)
----------	Ings Island	-------	73.2	Important location (change in slope)

Note: All distances are measured from the town of Fort McMurray.

5.2 Bed Profile

The profile of the bed was obtained by using the same 1:50,000 NTS maps. Specific points on the map where contour lines intersected the river channel were identified in order to obtain water surface elevation. Once the elevation of two or more points were known, slopes which represent the reach were calculated. The slopes were obtained by using the elevations and distance positioning from the starting point to the intersection of the topographic contour with the river channel. For this study reach only three elevation points were identified, which signified that for the entire study reach only two changes in slope could be observed from the NTS maps (Table 2). The water surface slopes obtained were further compared with the water surface slopes obtained by Kellerhals, Neil and Bray (1972). Figure 3 depicts that the two water surface profiles were tied-in to two different datum, however, the slopes are observed fairly consistent with each other. This was done only to make sure that the results obtained from the NTS maps does actually represent the slope of the reach, consequently, the profile obtained by the NTS maps would be used in this study.

Table 2: Comparison of gradient of the study reach of the Athabasca River.

Hydraulic and Geomorphic Characteristics of Rivers in Alberta (1972)			National Topographic Series (NTS) Maps		
Distance (km)	Geodetic Elevation (m)	Gradient %	Distance (km)	Geodetic Elevation (m)	Gradient %
10	235.97	0.019	17.3	240	0.018
67.5	225.22	0.008	73.2	230	0.008
132.5	219.85		183.25	221	

Note: All distances are taken from the arbitrarily origin at Fort McMurray.

The profile obtained from the NTS maps represents the slope of the reach, however, it does not represent the actual bed elevation at each node identified on the NTS maps at one kilometer intervals. In order to obtain the actual bed elevation, a few available natural cross-sections were used. For each surveyed cross-section, the surface width and the hydraulic mean depth were determined. The mean depth was obtained by dividing the effective flow area by the top width at bankfull condition. Once the mean channel depths were estimated an effective bed profile was determined. This was achieved by aligning this bed profile parallel to the water surface slope obtained from the NTS maps (Figure 4). The bed elevation for each node marked on the NTS maps were obtained by interpolation along the length of the study reach. These steps are performed in order to obtain a better estimate of the actual bed elevation of the reach and it should be checked against any cross-section data collected in the future.

5.3 Channel Widths

For the preparation of the input data in this section, 1:50,000 scale NTS maps were utilized. The widths of the channel in the study reach were measured at the discretized sections located at one kilometer intervals. Figure 5 illustrates the apparently random nature of the width variation presenting no clear pattern. For some models widths should be smoothed by using a four point moving average. However, the cdg1-D model, is numerically stable, so smoothing is not required (Hicks 1996).

5.4 Channel Resistance

For the cdg1-D model the channel resistance or roughness is the only parameter used in the calibration. In this particular project the resistance used for the hydraulic routing model was Manning's '*n*'. An initial estimate of '*n*' utilized for calibration was obtained directly from the work done by Kellerhals, Neill and Bray (1972), which recommended '*n*' equals to 0.026 extending throughout the entire study reach. This value of '*n*' was then adjusted, calibrating the 1983 flood event (hourly interval event). The results of the calibration process and final value of '*n*' will be presented and discussed later in this report.

6.0 Hydrometric Data

6.1 Hydrologic data

In order to route the hydrologic event downstream, hydrological data of the tributary inflows in the study area were required. The major inflow tributaries were identified on the 1:50,000 scale NTS maps. Only gauged inflow tributaries were used in the study for which, hydrological data was obtained from "HYDAT" database. The site characteristics for the gauging stations have been presented in Table 1.

6.2 WSC Gauging Data Available on the Reach Study

Only two gauges exist along the Athabasca River in the study reach, and only one has an active status, i.e., gauging station 07DA001 below Fort McMurray located 17.5 km downstream of the town. On the other hand, the station 07DD001, located 189.5 km downstream of the town of Fort McMurray at Embarras Airport was discontinued in 1984, which limited the number of years available for modeling the reach (Table 1). Since a major inflow tributary (Ells River) was not gauged until 1974, only 11 years on the record were modeled along the study reach.

7.0 Simulation Procedures

For all of the simulation runs, an ASCII text file was created as an input for the cdg1-D model. The file created contains four different components; header, node, element and boundary information. These files contain critical information describing the reach to be modeled and particularly for the node points. A detailed procedure for the preparation of the data input file is provided in the manual presented in appendix B.

In order to run the program, the boundary conditions must be specified for both upstream and downstream extremities of the study reach. In addition, initial conditions must be specified at every node point for any unsteady flow simulation. Gauge station 07DA001 below Fort McMurray was utilized as the upstream boundary, and Lake Athabasca was used as the downstream boundary. Water levels for station 07MD001 (Lake Athabasca at Fort Chipewyan) were used to define the downstream boundary condition, and discharge data from station 07DA001 below Fort McMurray provided the inflows for upstream boundary conditions.

The initial conditions were obtained by using the cdg1-D model with boundary conditions and inflow hydrographs prior to the flood event, which were obtained by running a simulation for steady flow conditions. In order to achieve this, theta (θ) and delta (δt) had to be adjusted in the heading of the input file. For a steady run a value of 1 for 'θ' was utilized and for 'δt' large steps of 100 seconds interval were used. For this steady run large values of delta 'δt' were used since the tolerance level for error is grater than for the unsteady simulation. For more information to manipulate 'θ' and 'δt' to increase accuracy of the model refer to Appendix B.

Once the steady flow and initial conditions were established, an unsteady event was conducted and hydrographs for the Embarras station were obtained. The outputs obtained from the model were then compared with the observed data at the Embarras Station provided by WSC. The results for these runs are discussed below.

8.0 Results

Three months of open-water season for each 11 recorded years were investigated. The length of time to model was selected in order to accommodate the limitations of the cdg1-D model. The program only accepts 100 value points which describe the inflow hydrographs for boundaries and inflow tributaries. In other words only 100 values are allowed for each hydrograph in the boundary component of the input file. For some years (e.g. 1975 and 1983) geodetic water levels were obtained and compared with WSC data provided, which will be discussed in details in the following sections of this report.

8.1 Calibration

An hourly event for the year of 1983 was utilized to initially calibrate the channel roughness (Manning's *n*). This year was selected since hourly data for all of the inflow tributaries was available from WSC. For this event 96 hydrograph points spaced hourly (four days) were used for the inflow tributaries and boundary conditions of the unsteady run. The calibration is done for hourly data to capture the peak of the event with a better resolution than by using mean daily values. Three different runs with values of '*n*' equals to 0.024, 0.025 and 0.026 were used to calibrate the model. Figures 6 to 8 present the results of the model compared with the WSC observed data, which show that the estimated peak flood for this event is always over estimated, and the rising and falling limbs of the hydrograph are always underestimated. This could be due to the limited geometrical approximations of the study reach and not due to Manning's '*n*' as shown by these figures. The best value of Manning's '*n*' was selected based on the minimum absolute percentage-error, consequently, the cdg1-D model with Manning's '*n*' value of 0.024 was chosen and used in the validation process.

8.2 Validation

The year of 1974 was then used with daily mean values for an isolated event in the month of July. The '*n*' value of 0.024 was used to check the performance of the model against observed daily mean values. The results were then plotted and are shown in Figure 9, which illustrates that the model is representative and that '*n*' = 0.024 was adequate for the study reach. Also as part of the validation process all years from 1974 to 1984 with mean daily values were examined and a spring flood event (1980) was used for model validation. For each individual flood event a total of 92 hydrograph points on the study reach were used. All the simulations were conducted with a time step increment of 24 hours for a total of 2208 hours. It took approximately 25 minutes (computer time) with a Pentium 166MHz for each unsteady run. The modeled hydrographs for 1974 - 84 record length were produced, and compared with WSC data at station 07DD001 at Embarras Airport (Figures 10 to 20). It was observed that there was a lag of one day between Fort McMurray and Embarras Airport stations; therefore, figures 10 to 20 started from the second day of the month. In other words, the inflow hydrographs and boundary conditions are all input at the beginning of each month, however, the comparison data used starts always on the second day of each month. Two selected years of record representing a moderate and a extreme event were analyzed below.

8.2.1 1975 Study

The input data for boundary conditions and inflow hydrographs for year 1975 started on August 1st to October 31st. The observed data for Embarras Airport station 07DD001 starts on August 2nd to November 1st and it is compared with the model output on Figure 11. The maximum discharge observed at Embarras Airport 07DD001 for the period of study was 2000m^3/s on September 6th. The 1975 year was observed to be one of the lowest open-water high flow registered on the record, therefore, it is considered to be moderate compared to other years on the record. The maximum peak discharge and the date of occurrence given by the cdg1-D model were 1935 m^3/s on September 6th. The error between the model and WSC observed data for the peak discharge is 3.26%, and the peak discharge was observed the same day (for mean daily values). These were indications that the model predicted the magnitude and time location of the peak event with virtually no error. Note that when using daily mean values the model's estimation can only be approximated to the nearest day. The maximum error on the estimated hydrograph was 16.1% and it was observed towards the end of the simulation; where the flows were the lowest (Figure 11), which indicated the variability of Manning's '*n*' needed to accommodate different discharges. It is known from hydraulic studies that as the discharge increases the effective resistance '*n*' decreases. Conversely, as the discharge decreases the effective roughness of the bed becomes larger increasing the value of '*n*'. The major river crossings and structures are always related to the high water discharges, e.g., the 100-year flood for a bridge. In the future a better approximation for both low and high flows can be achieved by using a roughness height '*k*' which is independent of flow, or by varying Manning's '*n*' with discharge.

8.2.2 1980 Study

The study of year 1980 was done on the maximum water discharges observed during the months of June, July and August. The comparisons at Embarras Airport station 07DD001 starts on June 2nd and ends September 1st. The maximum daily discharge of 3900 m^3/s for year 1980 was observed on June 11th. The cdg1-D model estimated the maximum daily discharge on the same day with a magnitude of 3934 m^3/s, which was 0.86 percent higher than the observed value, which indicated that the model was representative of high flow scenarios on the study reach (Figure 16).

The maximum peak flows for both the observed and estimated are presented in Table 3.

Table 3: Accuracy of the cdg1-D model.

	Maximum Discharge (m^3/s) at Station 07DD001		
Year	**WSC Data observed**	**1 D cdg Model Results**	**% Error**
1974	2890	3091	6.94
1975	2000	1932	3.26
1976	2150	2206	2.61
1977	2860	2882	0.75
1978	2310	2388	3.39
1979	2610	2925	12.09
1980	3900	3934	0.86
1981	1500	1415	5.67
1982	3210	3484	8.66
1983	2310	2209	4.38
1984	2290	2112	7.76

8.3 Water Surface Elevation comparisons

The cdg1-D model not only estimates the discharges at every single node point but it also predicts the water level elevations for each node (marked on the NTS maps). Before any results are presented, it should be kept in mind that the hydraulic routing model is operating with a limited geometrical approximations, which may cause less accurate results. In order to improve the accuracy of the model, natural cross-sections of the entire study reach and particularly the point of interest were required, these were not available, therefore, the same limited model was used to estimate water levels for two years 1975 and 1983. The results obtained from the water level approximations are presented below.

8.3.1 1975 Water Surface Level Results

The flow level data for 1975 was selected from August 2nd extending to November 1st. A peak water level of 221.19 m on September 8th was observed. In comparison, the model estimated a water level 0.45m higher than the observed value (Figure 21). The shape of the hydrographs were almost identical, however, the model over-estimated the levels. This difference in water levels once again was due to the limited channel geometry approximation used in the model, which was illustrated in section 4.0. The model could be used for an approximation of the water levels during a flood event. However, the application of the model for engineering studies requires the use of natural geometry for improved accuracy of the estimation of water levels.

8.3.2 1983 Water Surface Level results

The flow level data for 1983 was selected from June 2nd extending to July 1st. The purpose of the analysis of this month was done in order to observe how the model estimates water levels for events which are considered low. A maximum water level of 220.16 m on July 1st was observed. In comparison the cdg1-D model estimated a water level of 1.09 m higher than the observed value (Figure 22). Even though the difference in elevation was high, the shape of the hydrographs for both observed and estimated remained identical. The large difference between values was affected by the roughness '*n*' and the limited geometry used.

In order to obtain an initial estimate of geodetic water levels, the hydraulic model can be used, however, for more accurate results the surveyed of the area of interest is necessary. In this study only two years were analyzed, however, it is recommended that all the record years be checked also.

9.0 Conclusions

It is important to note that the cdg1-D model was used with limited channel geometrical information assuming a rectangular channel between WSC gauging station 07DA001 below Fort McMurray to WSC gauging station 07DD001 at Embarras Airport. In spite of these approximations, the model was calibrated for a Manning's '*n*' of 0.024 for 1983 hourly data for a high flow event. Further, the model was validated from 1974 to 1984 for high water events during open-water season, giving satisfactory results for most of the years. The results of the model seem to approximate the magnitude and time of occurrence of the peak event accurately. The error was found to range from as low as 0.75 percent to a high of 12.0 percent, which was within an acceptable limits given 328.8 km river length and limited geometrical characteristics of the study reach. The water levels for 1975 and 1983 were also studied in order to observe the behavior of the limited cdg1-D model. It was found that due to the lack of geometrical information the model over-estimates the water levels. This confirms Blackburn and Hicks (2001) finding that the model requires natural geometrical cross-sections to accurately estimate water levels, and that for accurate estimates of discharges only an accurate gradient of the study reach is needed. Based on the analysis and results obtained, it is reasonable to conclude that the calibrated model produces satisfactory results for the Athabasca River between Fort McMurray (WSC gauge station 07DA001) and Embarras Airport (WSC gauge station 07DD001).

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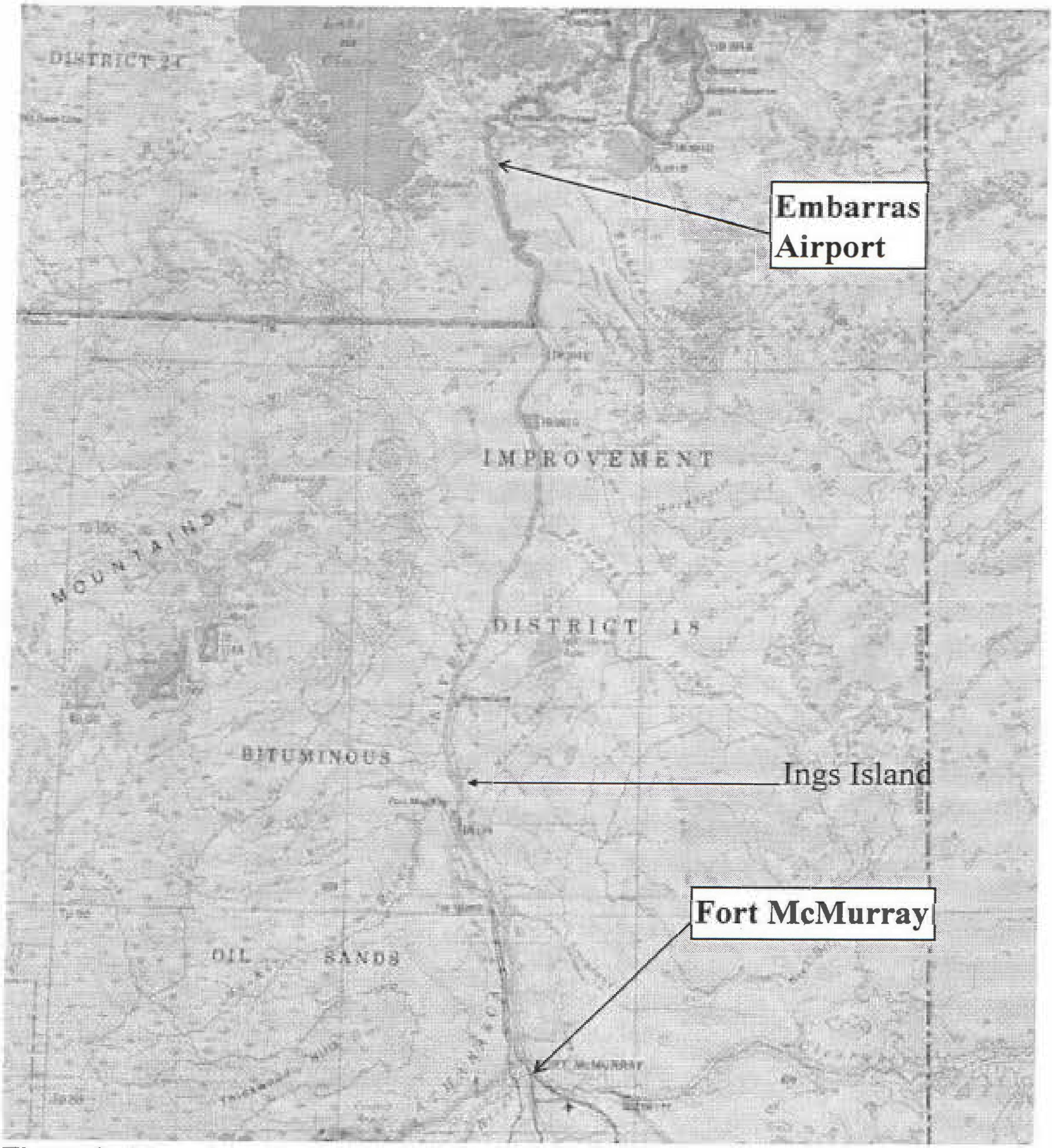

Figure 1: Map of the Study Reach

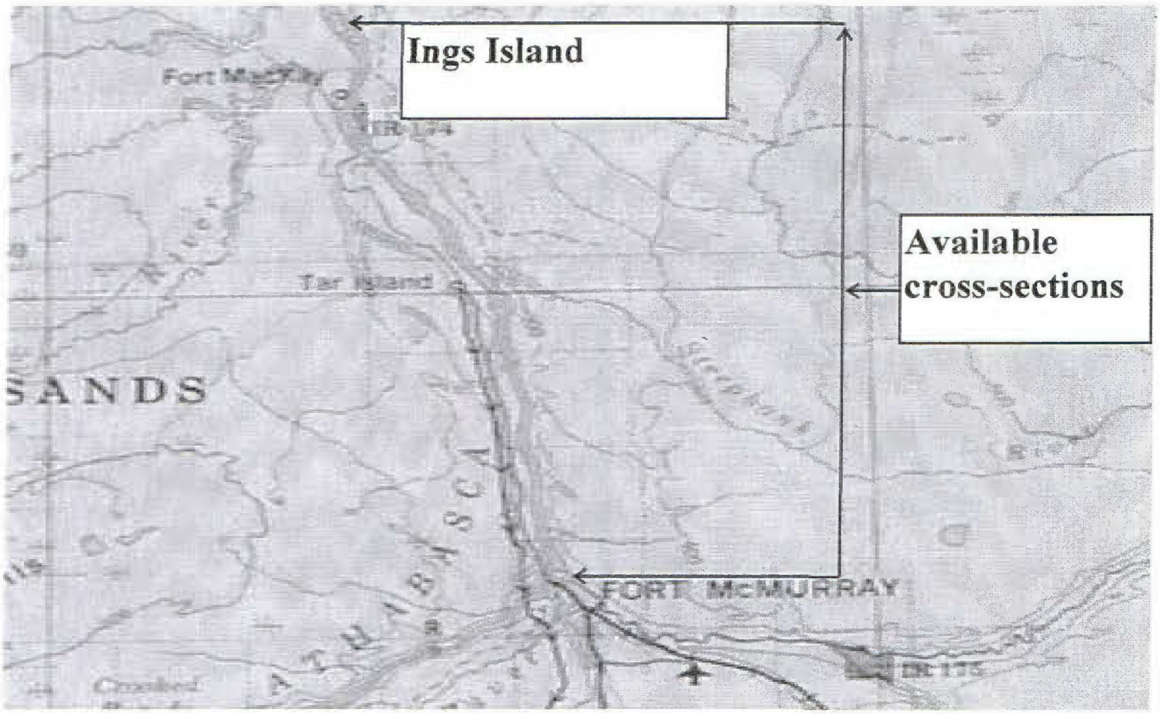

Figure 2: Available Cross-Sections Map

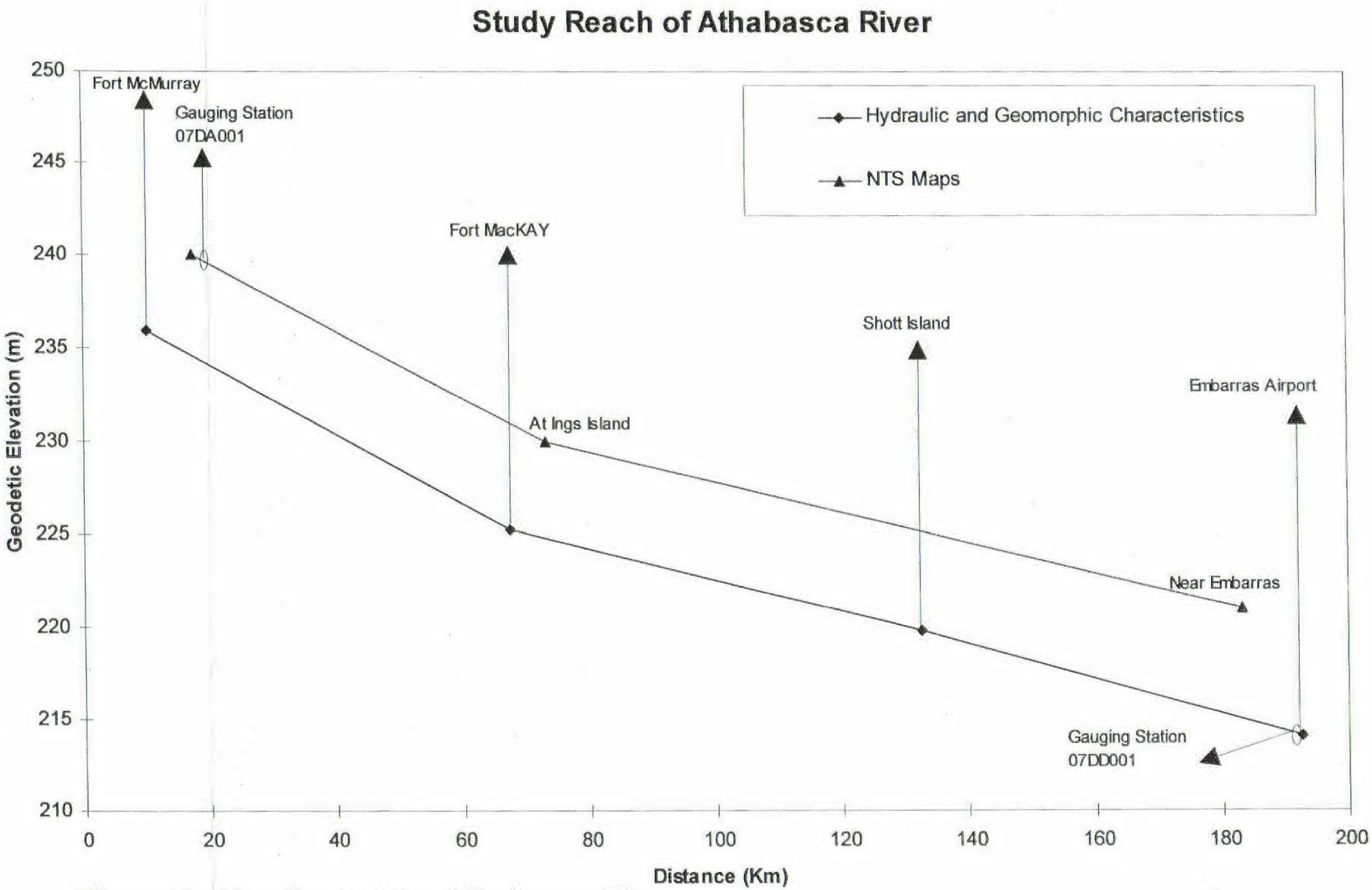

Figure 3: Gradient of the Athabasca River

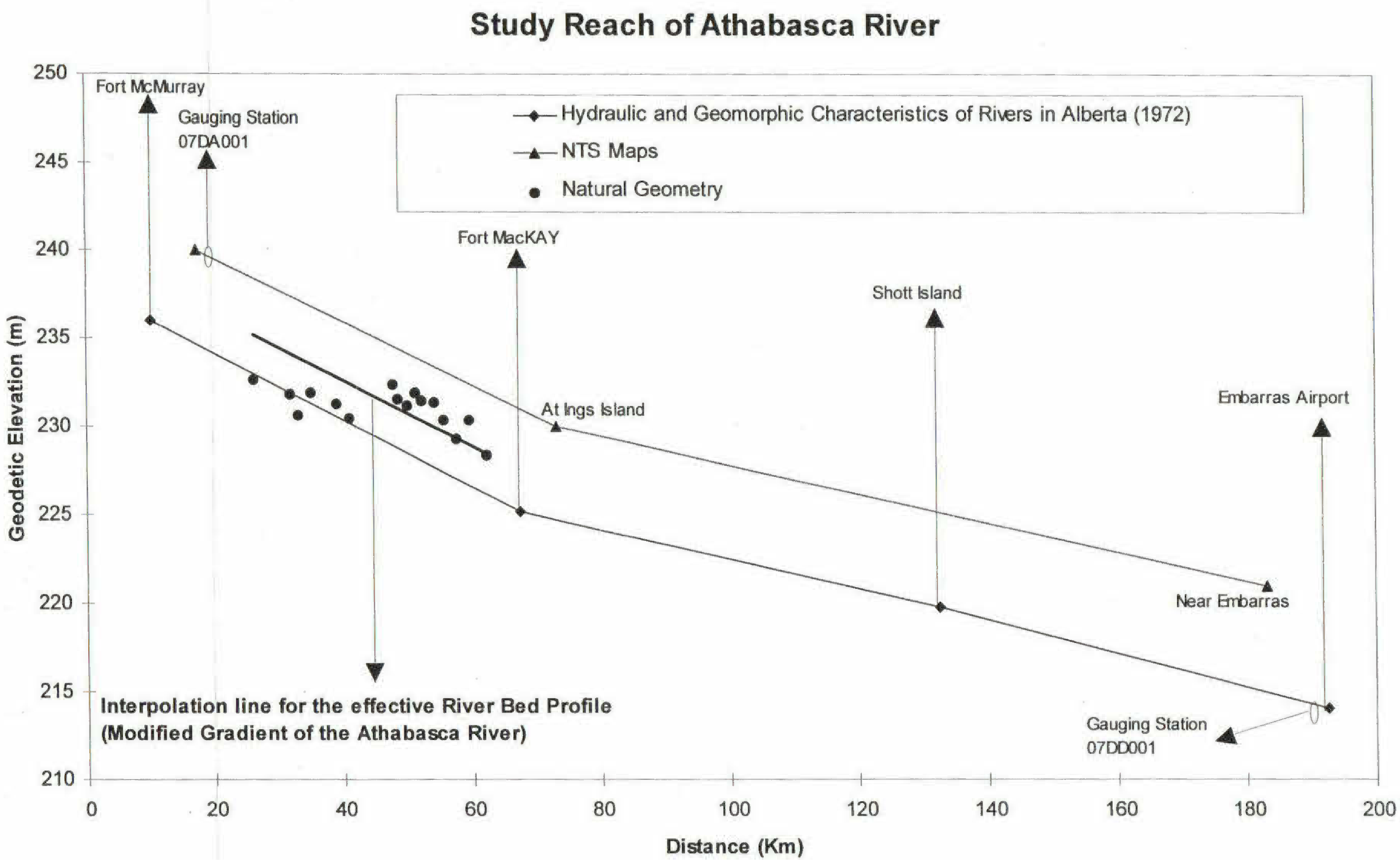

Figure 4: Modified Gradient of the Athabasca River

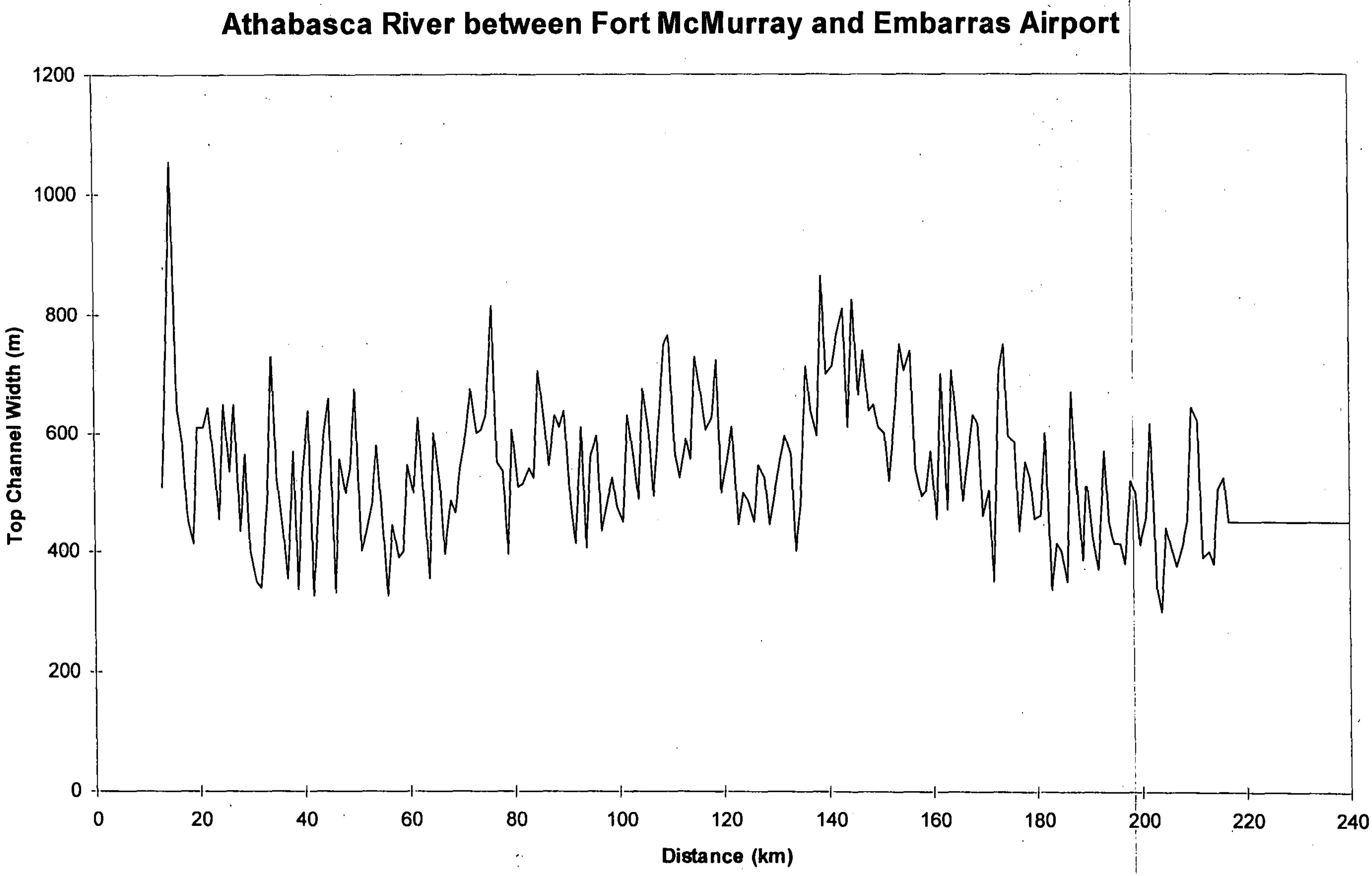

Figure 5: Variation of Channel Width Along the Reach

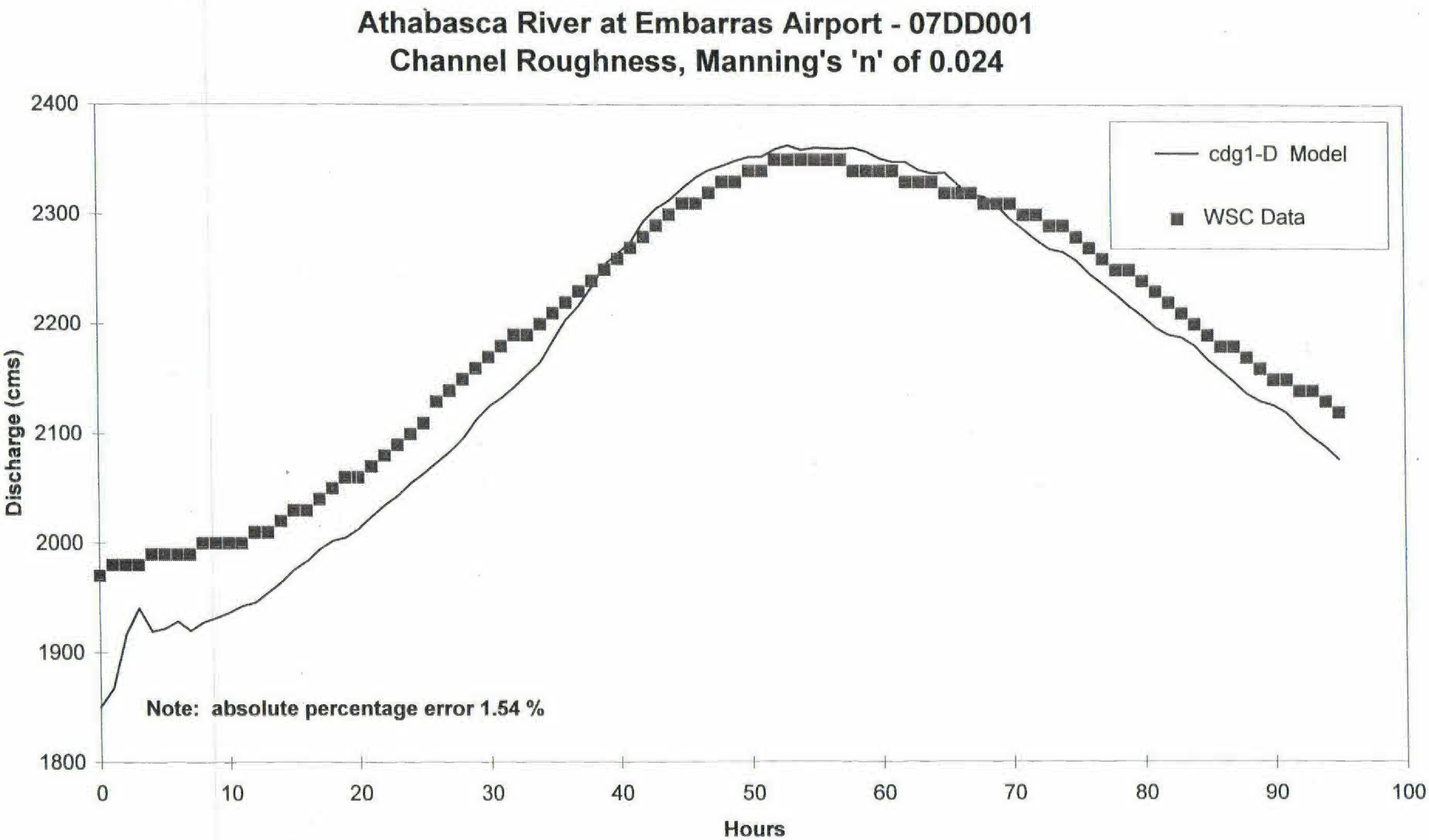

Figure 6: Hourly flow data for July 29 to August 1, 1983

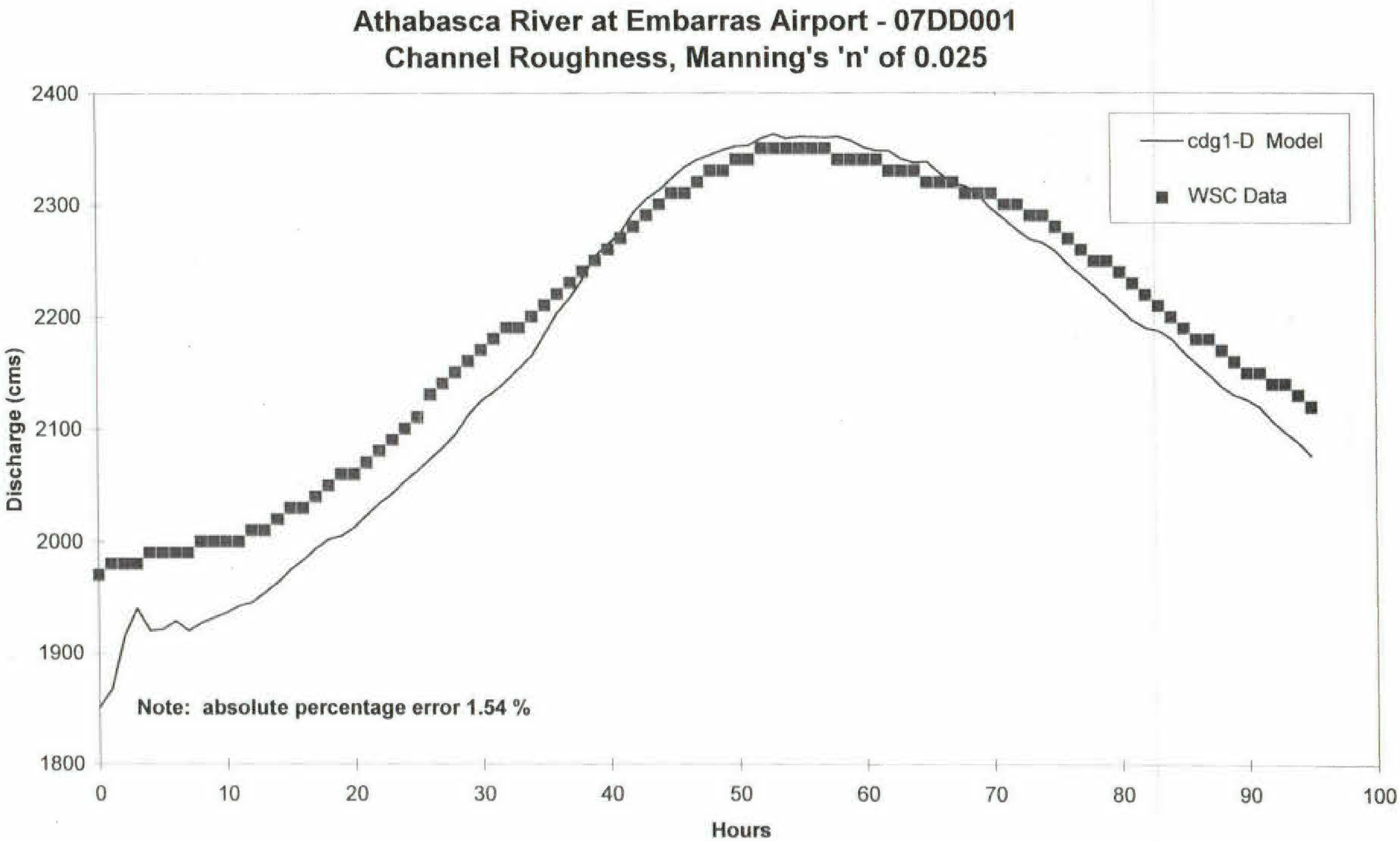

Figure 7: Hourly flow data for July 29 to August 1, 1983

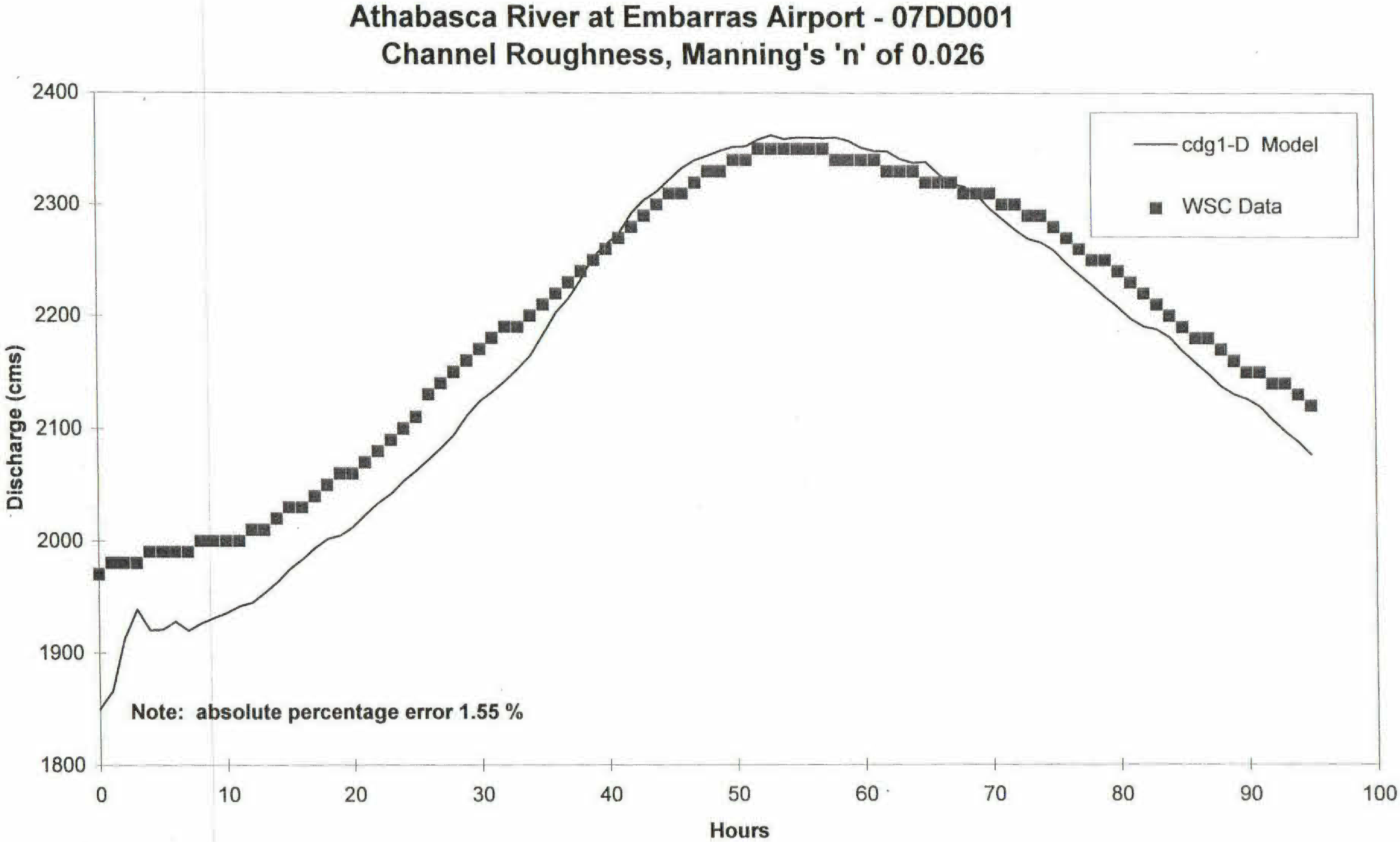

Figure 8: Hourly flow data for July 29 to August 1, 1983

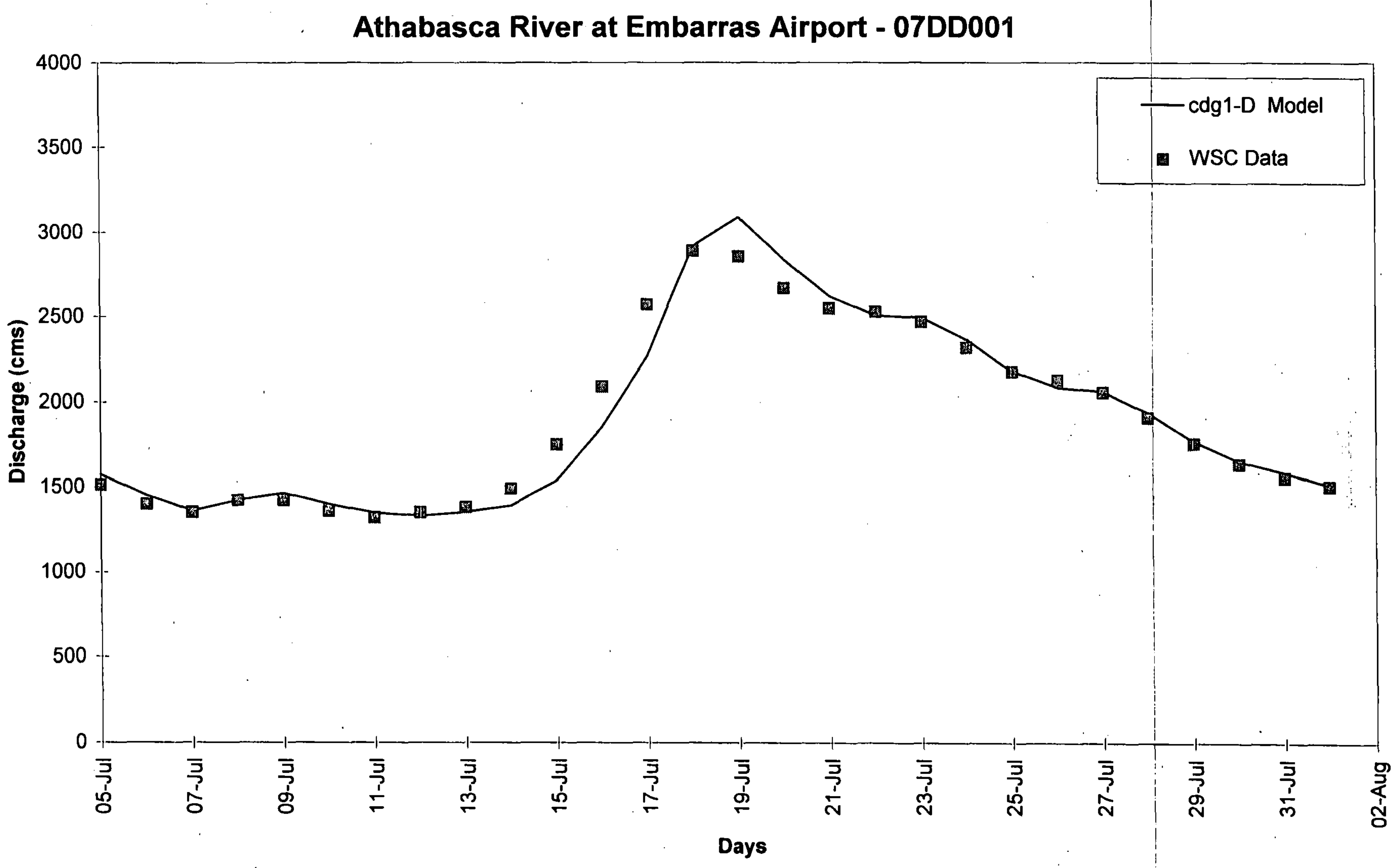

Figure 9: Isolated high flow Event, 1974

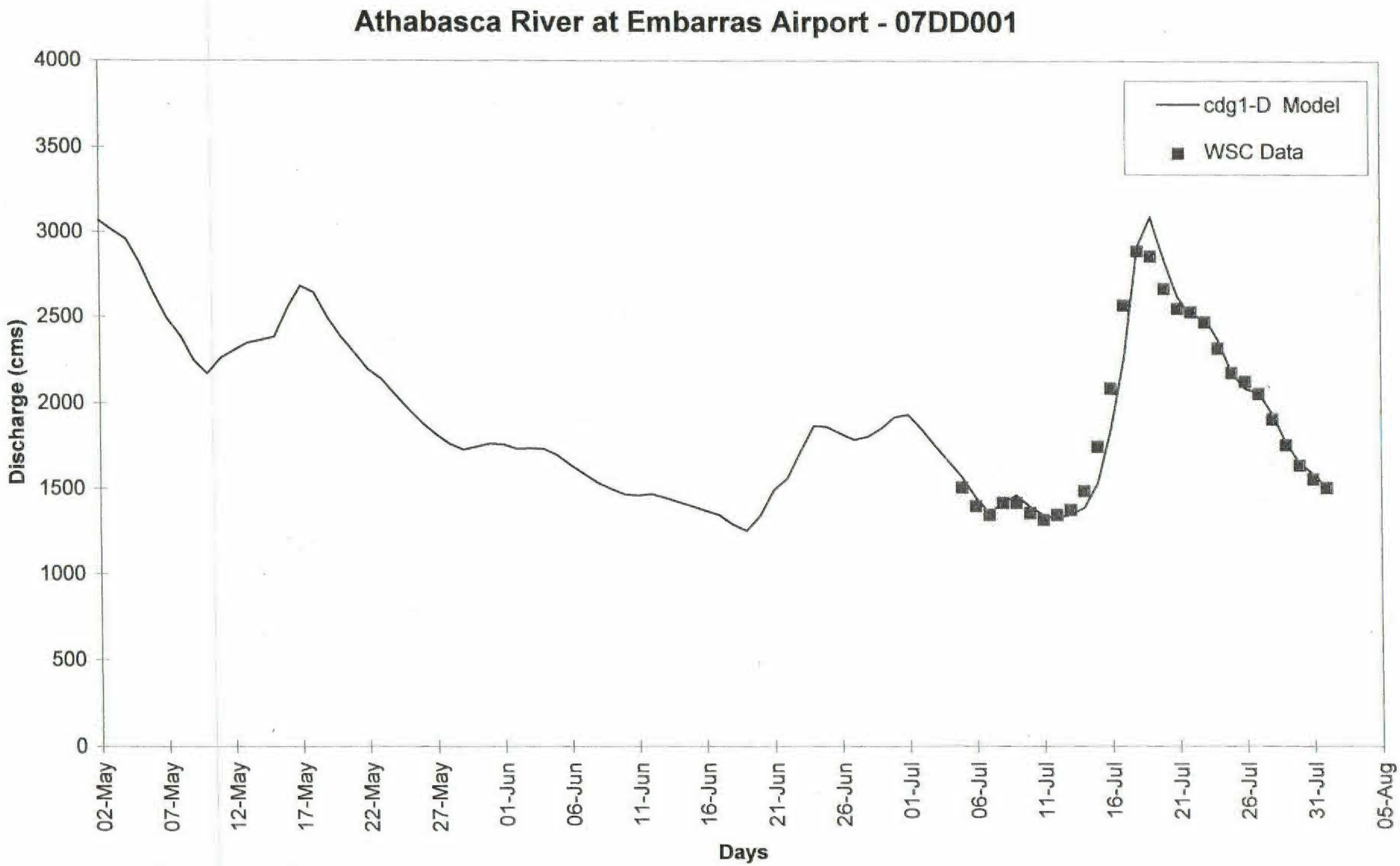

Figure 10: May 2nd to August 1st, 1974

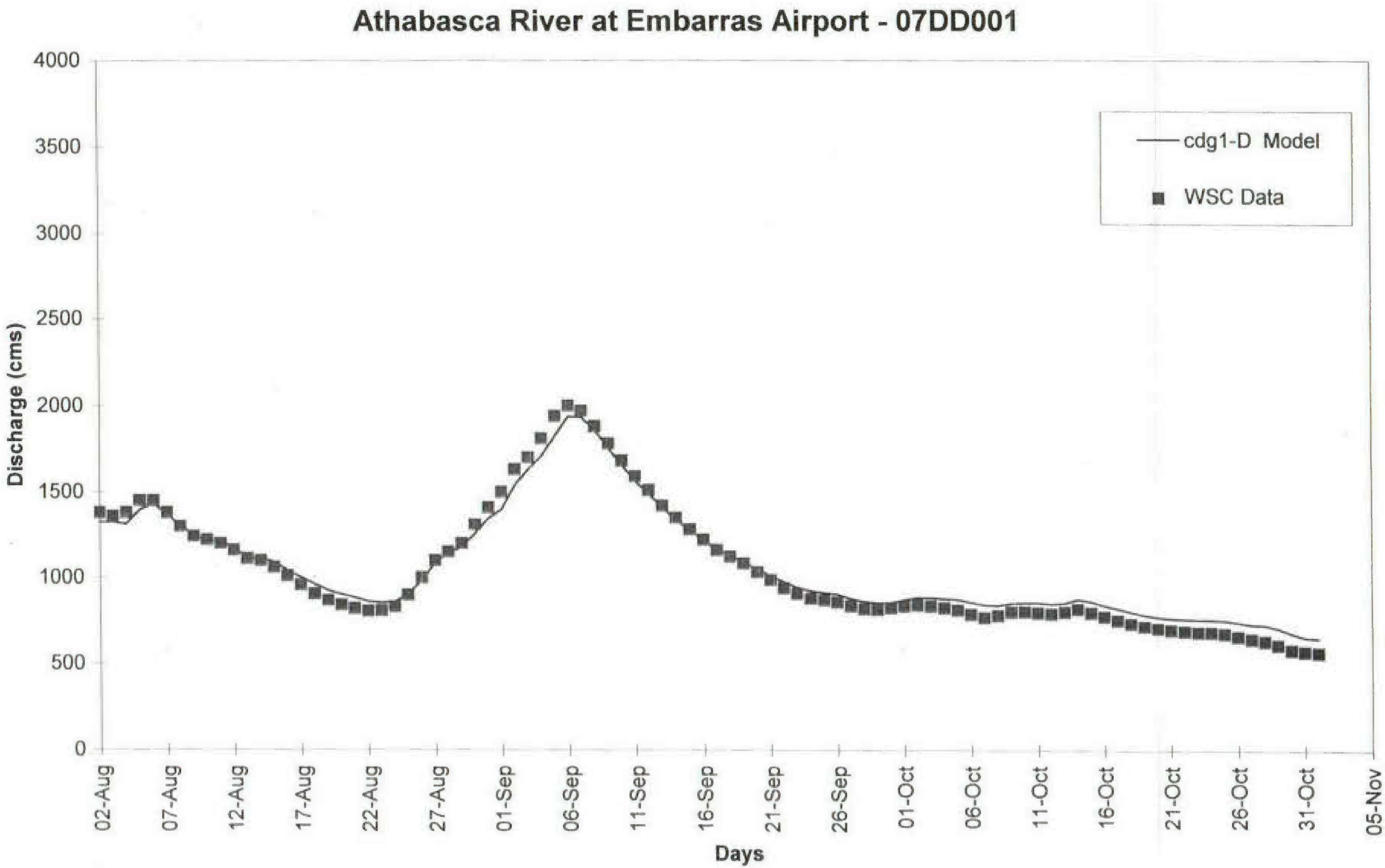

Figure 11: August 2nd to November 1st, 1975

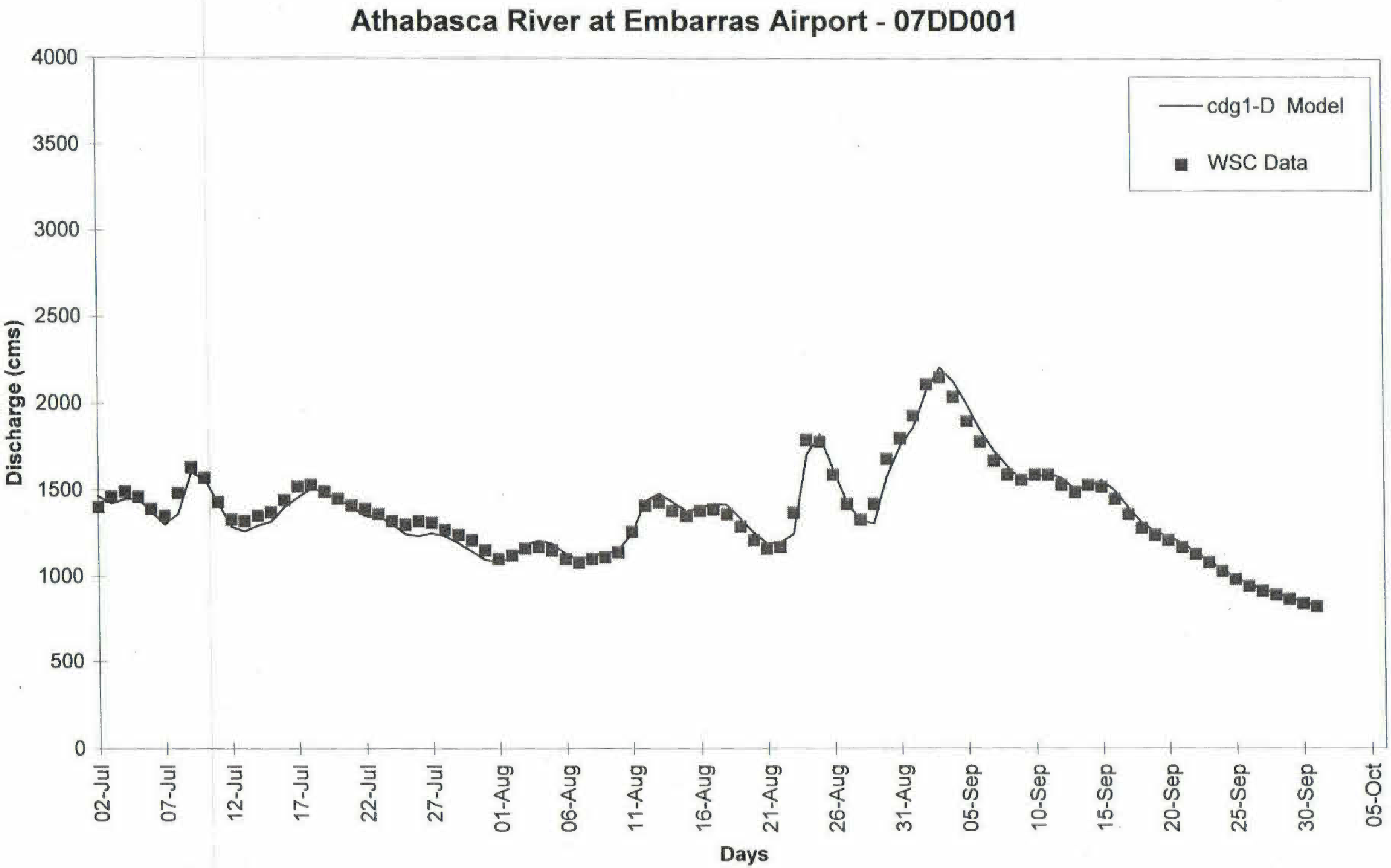

Figure 12: July 2nd to October 1st, 1976

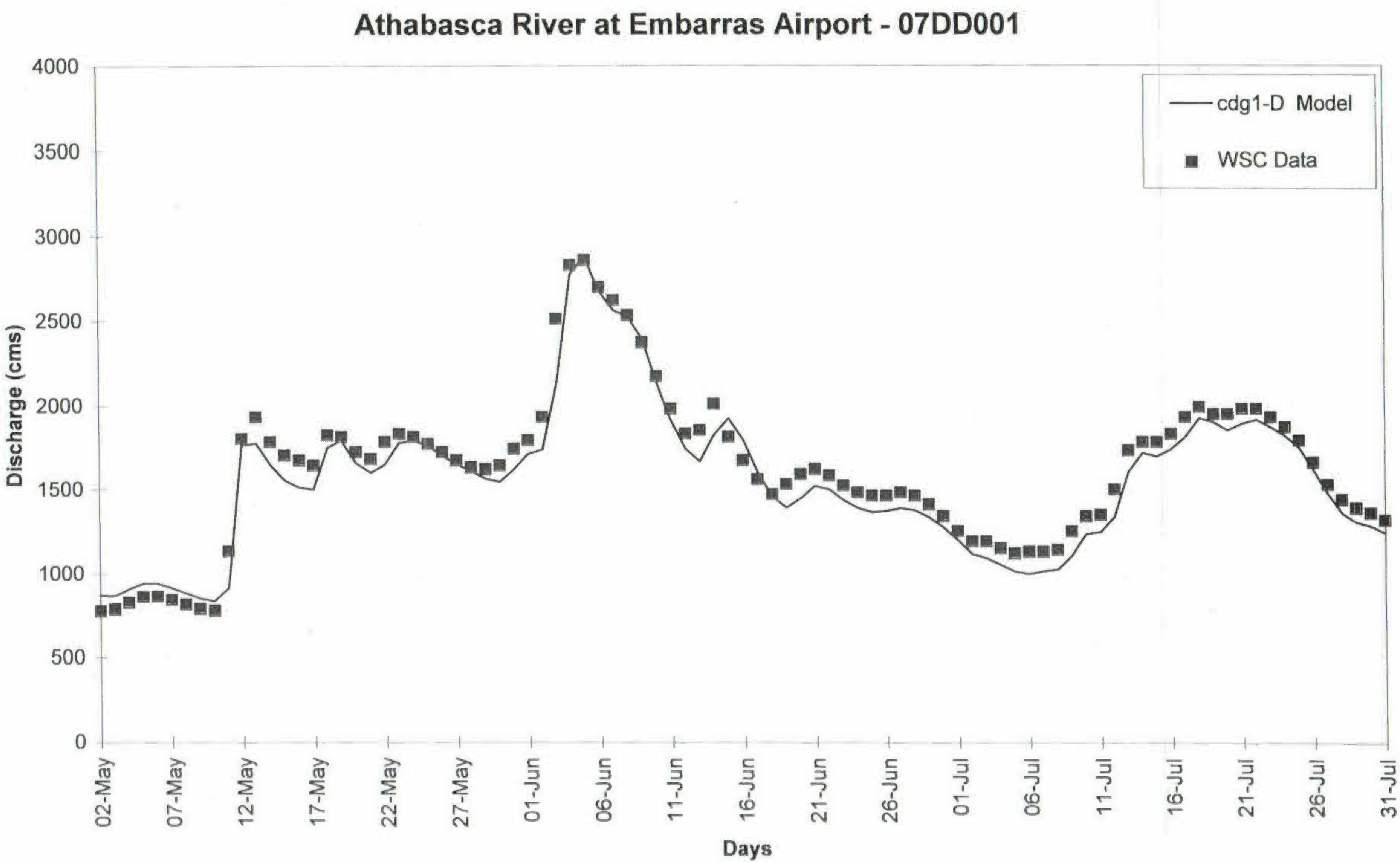

Figure 13: May 2nd to August 1st, 1977

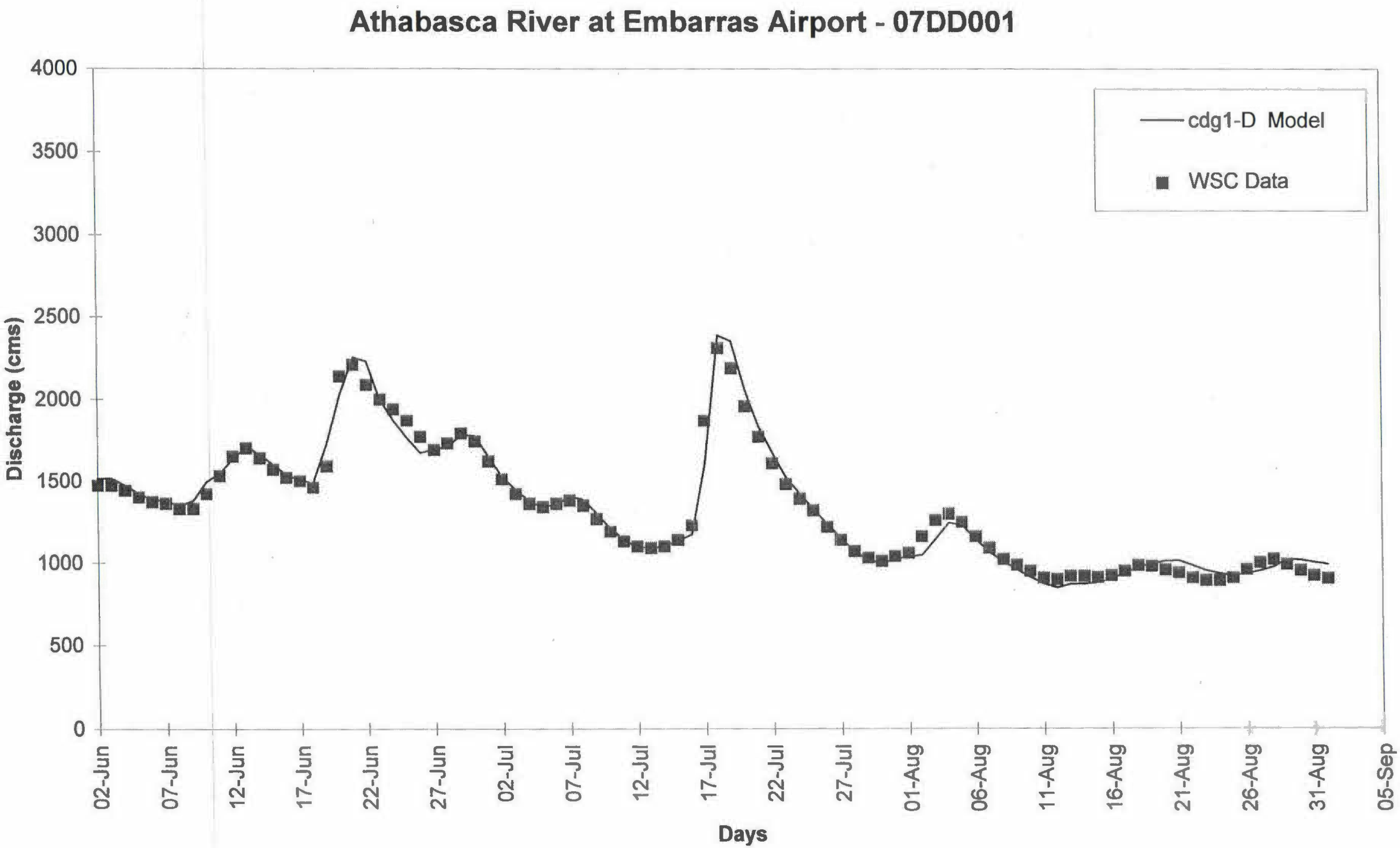

Figure 14: June 2nd to September 1st, 1978

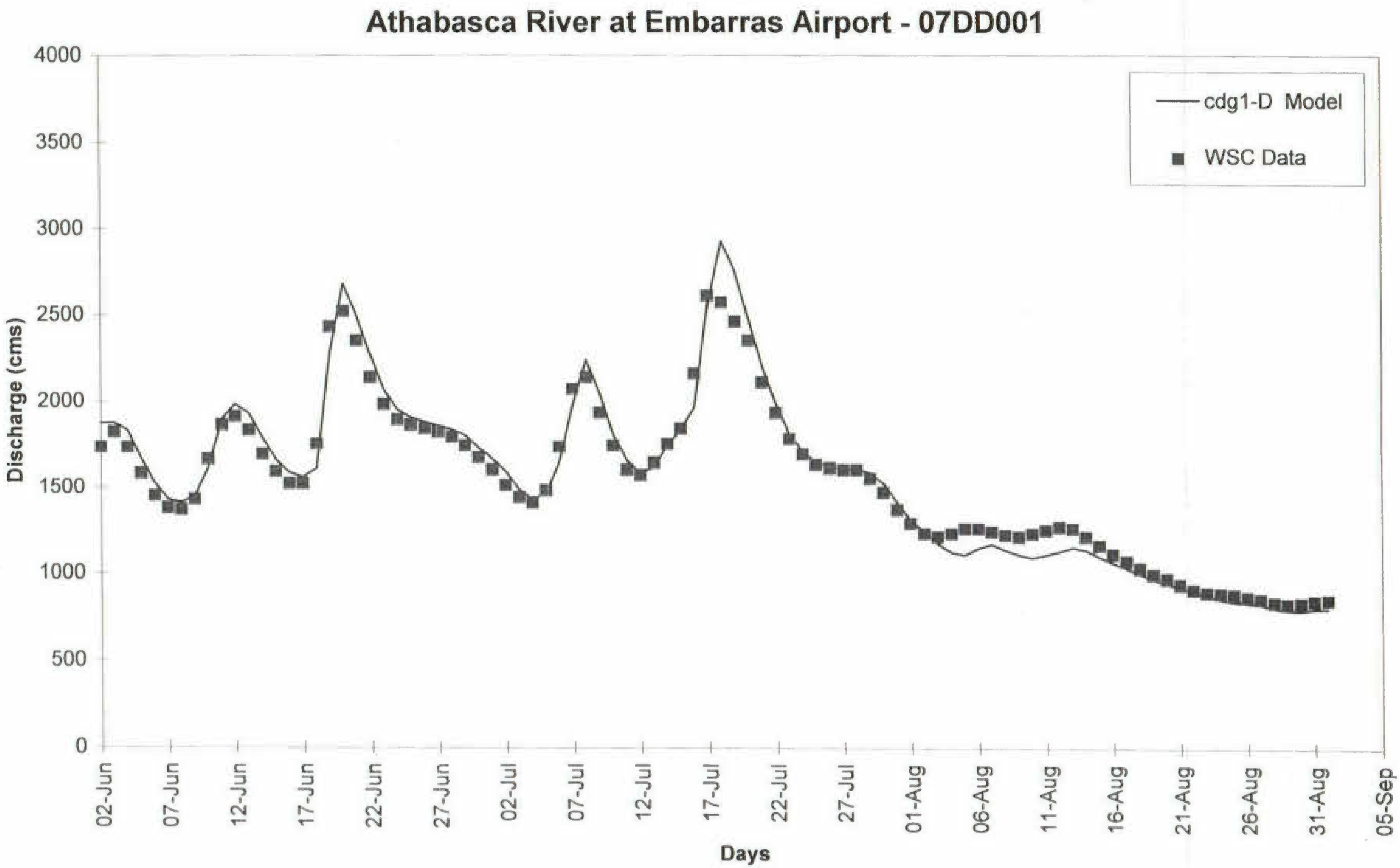

Figure 15: June 2nd to September 1st, 1979

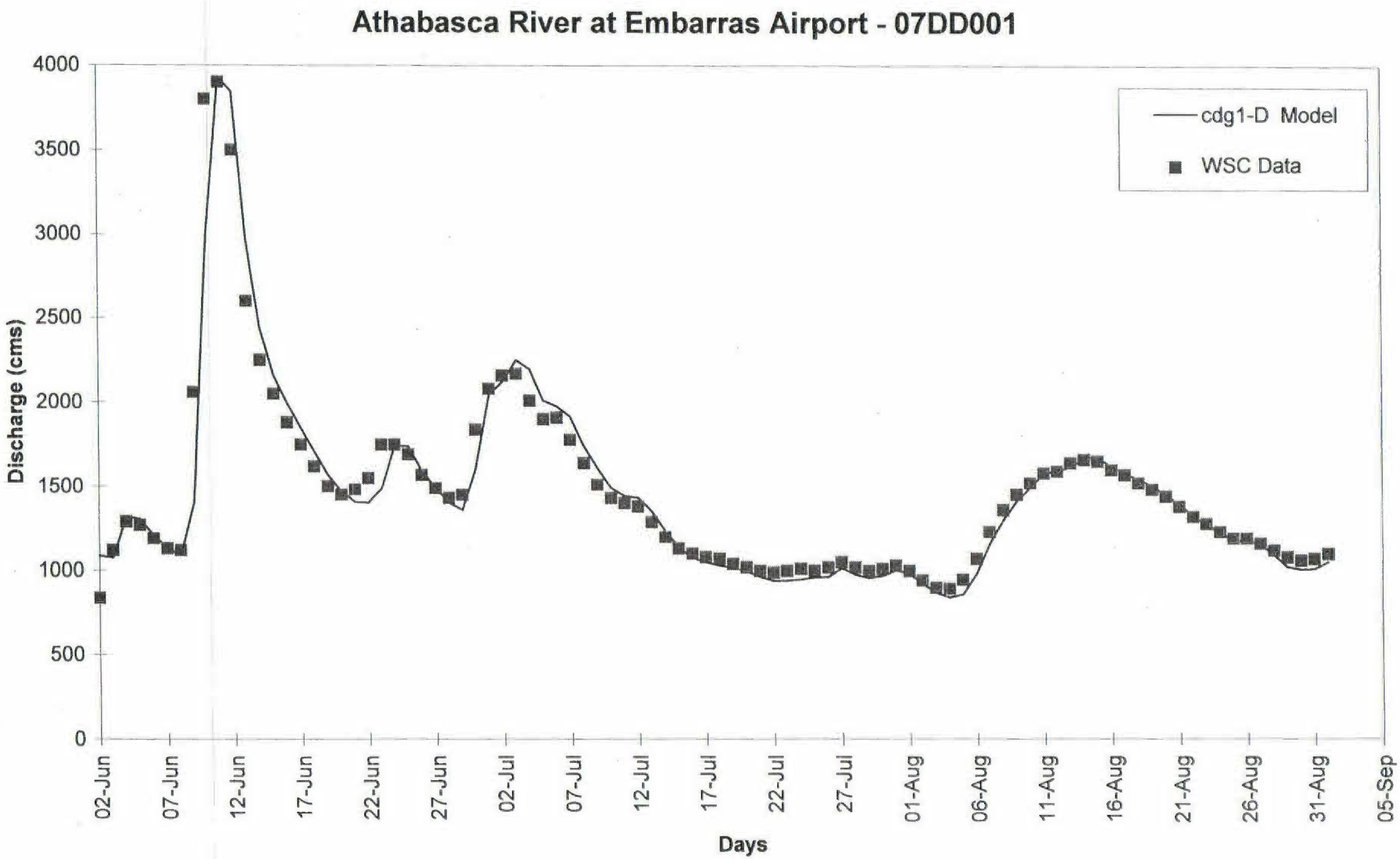

Figure 16: June 2nd to September 1st, 1980

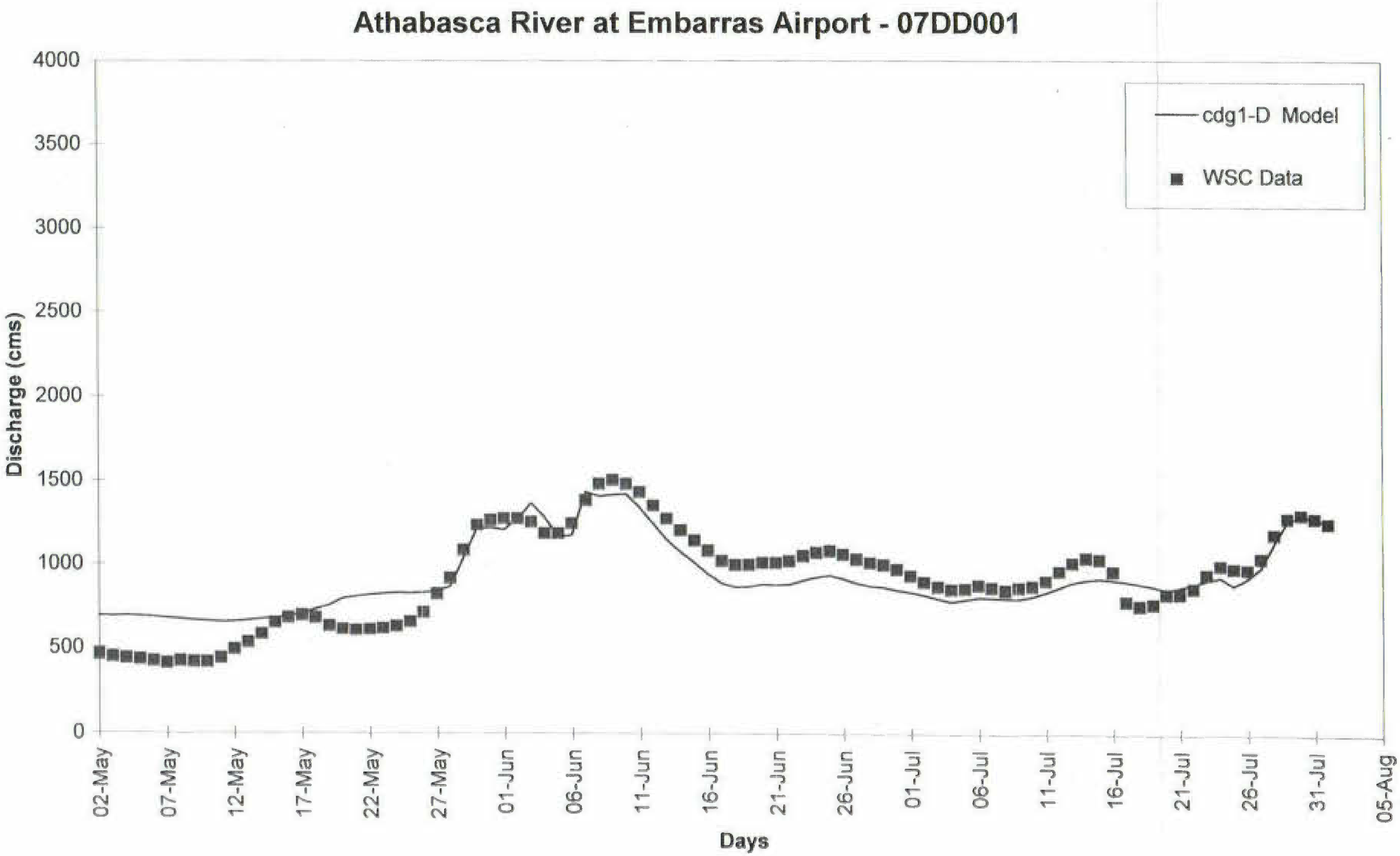

Figure 17: May 2nd to August 1st, 1981

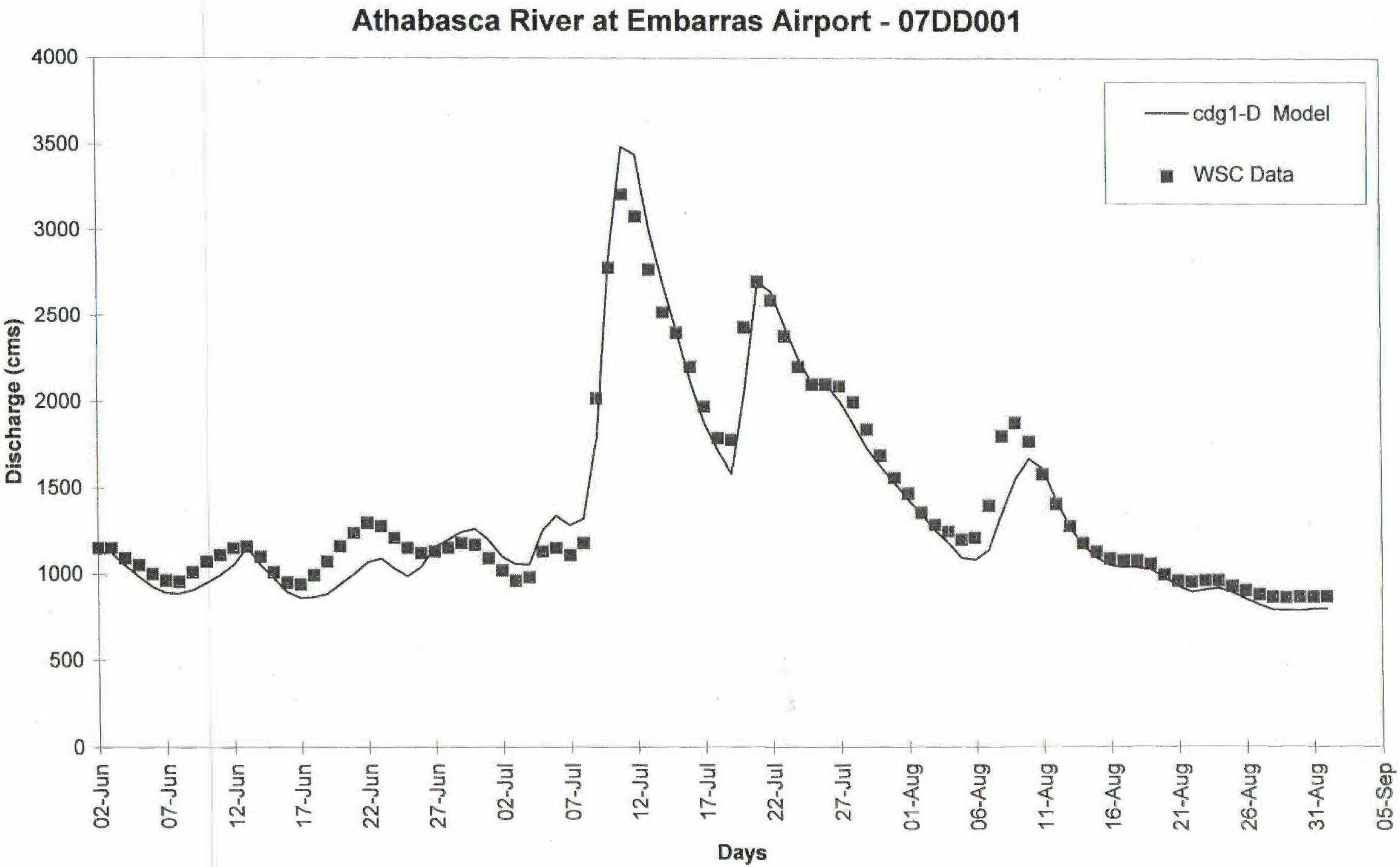

Figure 18: June 2nd to September 1st, 1982

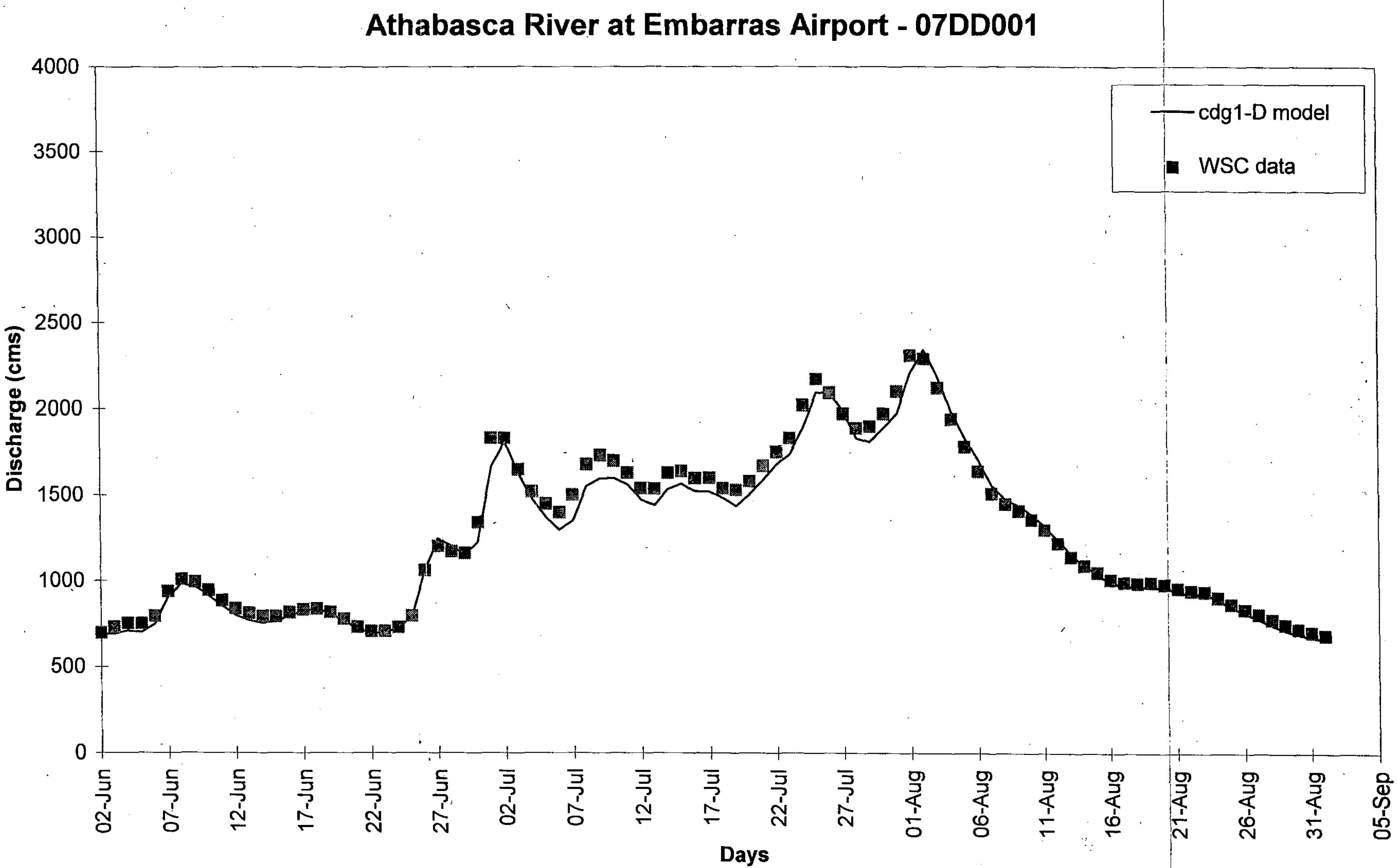

Figure 19: June 2nd to September 1st, 1983

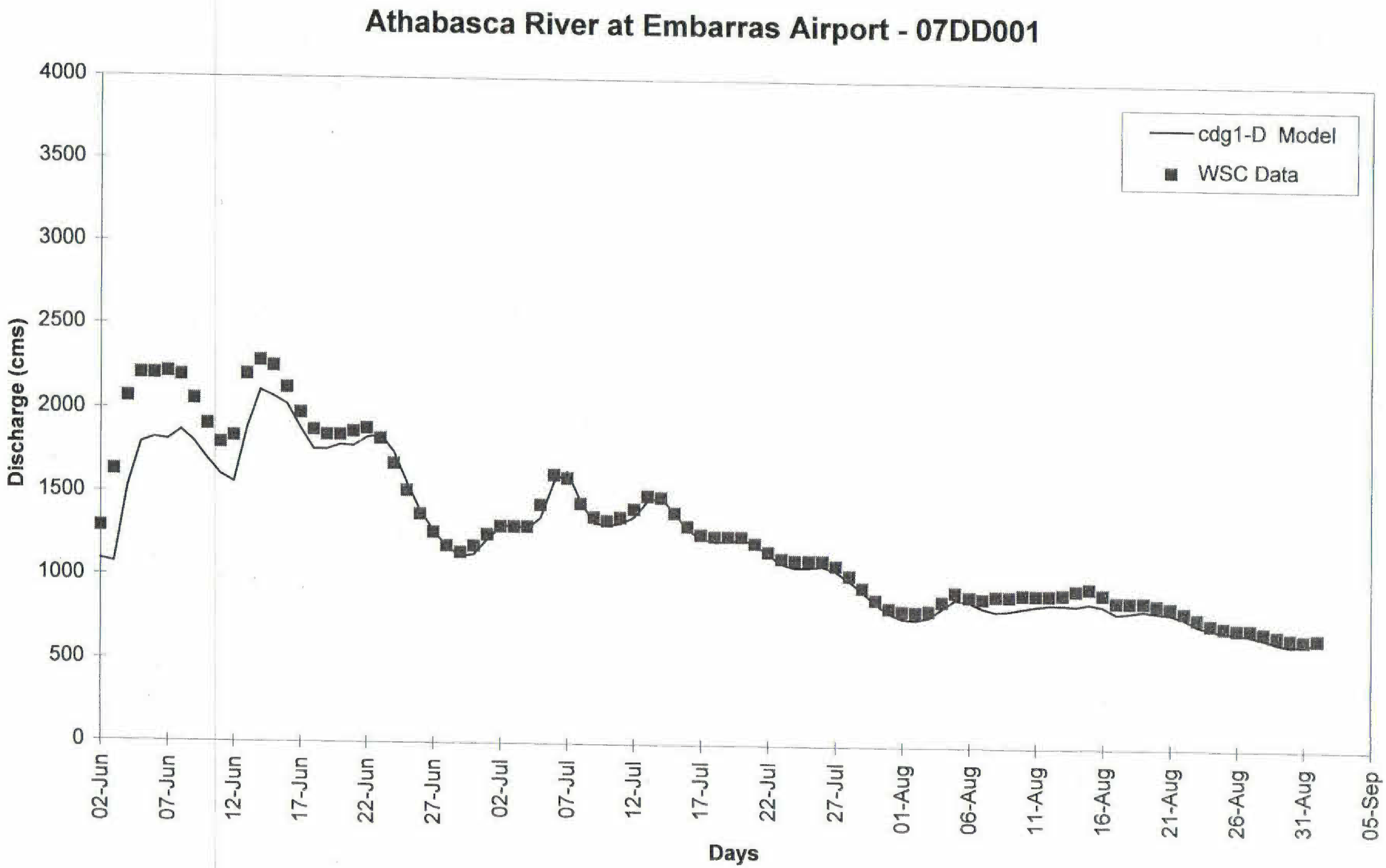

Figure 20: June 2nd to September 1st, 1984

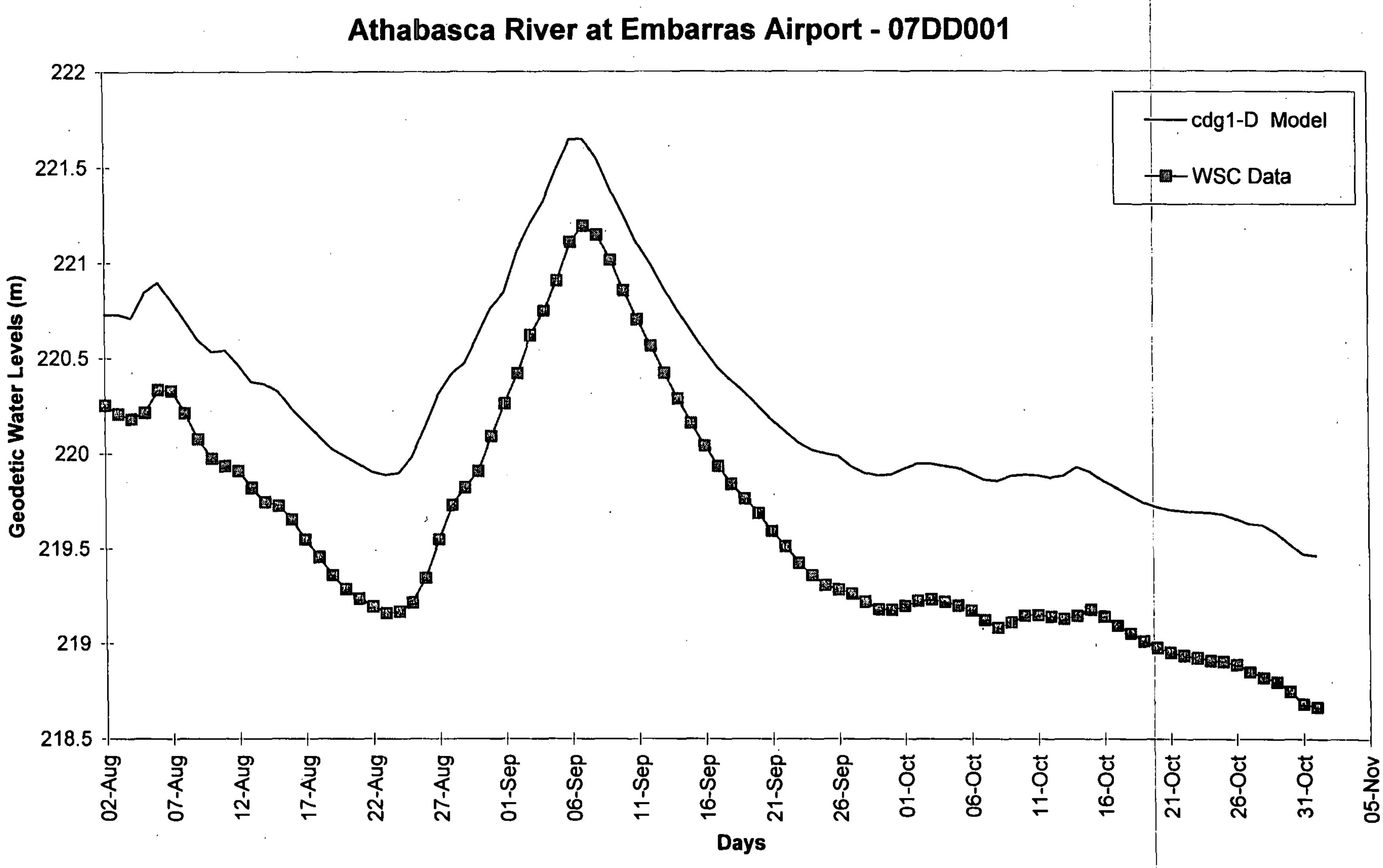

Figure 21: August 2nd to November 1st, 1975 (Geodetic Water Levels)

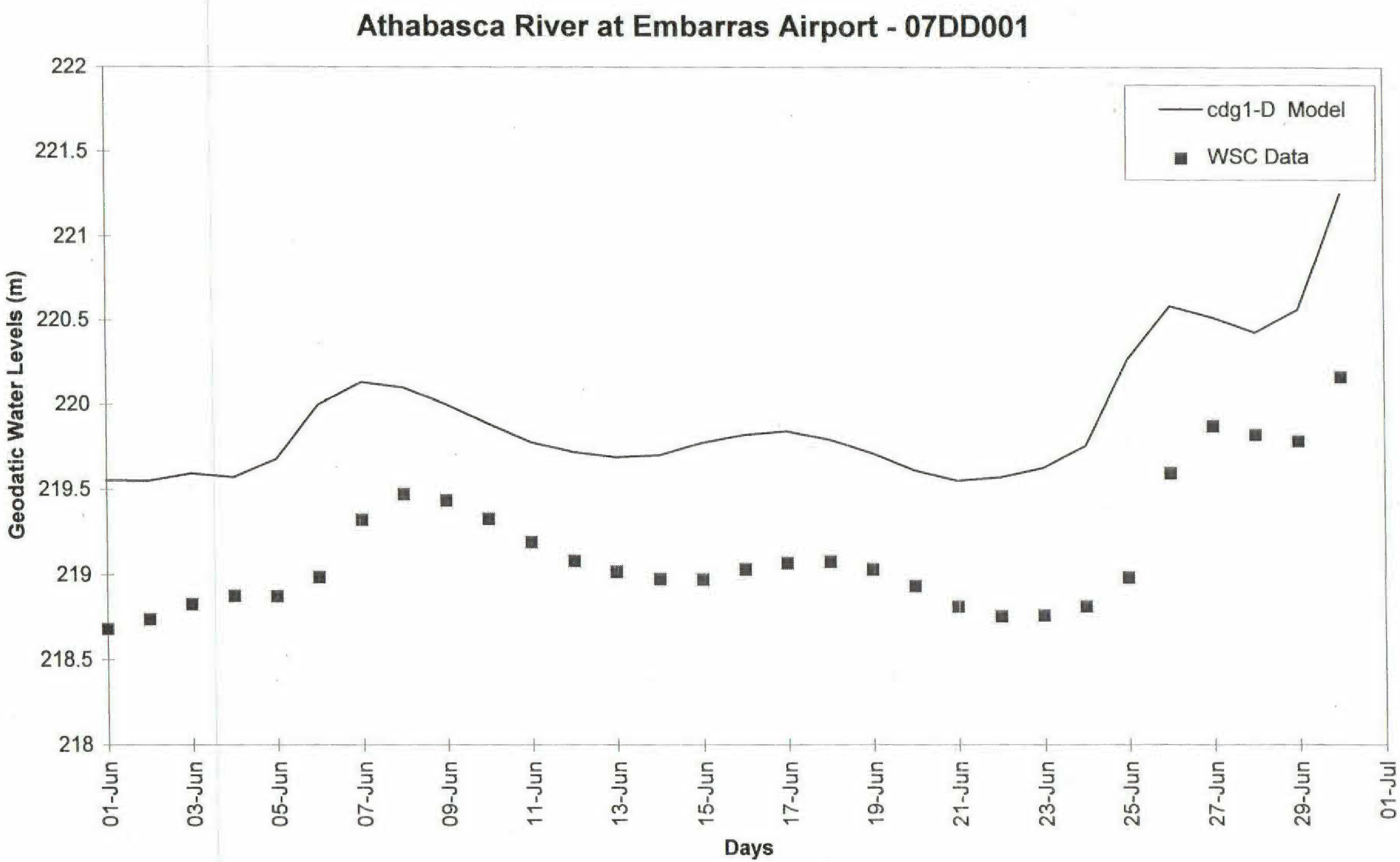

Figure 22: June 2nd to July 1st of 1983 (Geodetic Water Levels)

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Khanna, V.K.

Application of the cdg1-D model in the lower Athabasca River basin...