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CAVE CREEK SEDIMENT TRANSPORT STUDY

FINAL REPORT
DRAFT

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CAVE CREEK SEDIMENT TRANSPORT STUDY

FINAL REPORT
DRAFT

Prepared for:

City of Phoenix
Water Services Department
200 West Washington St., Eighth Floor
Phoenix, AZ 85003-1697

Prepared by:

WEST
CONSULTANTS, INC.

WEST Consultants, Inc.
2151 East Broadway Road, Suite 116
Tempe, AZ 85282-1705

October 2001

City of Phoenix
WATER SERVICES DEPARTMENT
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TRANSMITTAL LETTER

To: Mr. Russ Miracle
Maricopa County Flood Control Dist.
2801 West Durango Street
Phoenix AZ 85009

Date: April 4, 2002

Subject: Cave Creek Sediment Transport Study
WS90300007

The following items are:
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1	Cave Creek Sediment Transport Study – Draft Final Report

These items are submitted:
- [x] At your request
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Russ,

Here is a DRAFT copy of the sediment transport study. We are still in the process of reviewing the report ourselves, so please be aware that the finding and conclusions may change.

db:H:worddocs:Transmittal Letters:Corps Doc Dist

Andy Terry

200 West Washington Street, Eighth Floor, Phoenix, Arizona 85003-1697 602-534-5813 FAX: 602-495-5843

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Sediment Transport Study & Monitoring Well Design for the Cave Creek Water Reclamation Plant Reclaimed Water Discharge

Project No.: WS90300007

1 Introduction

The City of Phoenix (the City) plans to discharge excess reclaimed water from the Cave Creek Water Reclamation Plant (WRP) to an unnamed tributary to Cave Creek Wash. Phoenix selected WEST Consultants, Inc. (WEST) to study the sediment transport impacts produced as a result of these discharges and to provide design services for a groundwater quality monitoring well. WEST provided the sediment transport study services and overall project management while sub-consultant Knight Piesold provided the monitoring well design services. Project Engineering Consultants provided the surveying services for the project.

The City of Phoenix Cave Creek WRP was scheduled to begin operation in early 2000 but completion has been delayed into the late summer of 2001. The WRP will provide reclaimed water for reuse with the first priority of reuse being turf irrigation and the second priority being groundwater recharge. The City recognized that there could be periods in the future when the amount of reclaimed water available may exceed the irrigation and recharge demands. The City installed a pipeline to discharge the excess reclaimed water to an unnamed tributary of Cave Creek Wash north of the Cave Creek Dam flood pool ("the wash"). The discharge site is in Township 5 North, Range 3 East, Section 26, north and west of the intersection of Cave Creek Rd and Dynamite Rd. The location is shown in Figure 1. The estimated quantity of discharge could be as much as 250,000 gallons per day when the WRP begins operation and could increase to 24 million gallons per day (MGD) in the future when the WRP is expanded. However the City projects the discharges will not be continuous nor will they always be at the maximum discharge.

1.1 Sediment Transport Study

The reclaimed water from the plant that does not either evaporate or percolate into the subsurface aquifer will pool behind the old Cave Creek Dam. In an agreement with the Flood Control District of Maricopa County (FCD), the City agreed to conduct a sediment transport study of the wash during the first year of operation of the reclamation plant. The sediment transport study was to estimate the amount of sediment that may be transported to the pool behind Cave Creek Dam for flow rates varying between 250,000 and 24,000,000 gallons per day (0.4 to 37.1 cubic feet per second). The study reach of the wash extends from the release point to the limits of FCD property, a distance of approximately 5600 feet. The Cave Creek Dam is located approximately 16,000 ft from the release point (WRP outfall).

1.2 Monitoring Well Design

Groundwater quality regulations require a monitoring well be installed down gradient of the reclaimed water discharge point. The areas of investigation for locating the monitoring well consisted of those portions of T5N, R3E, Sections 26, 27, 34, and 35

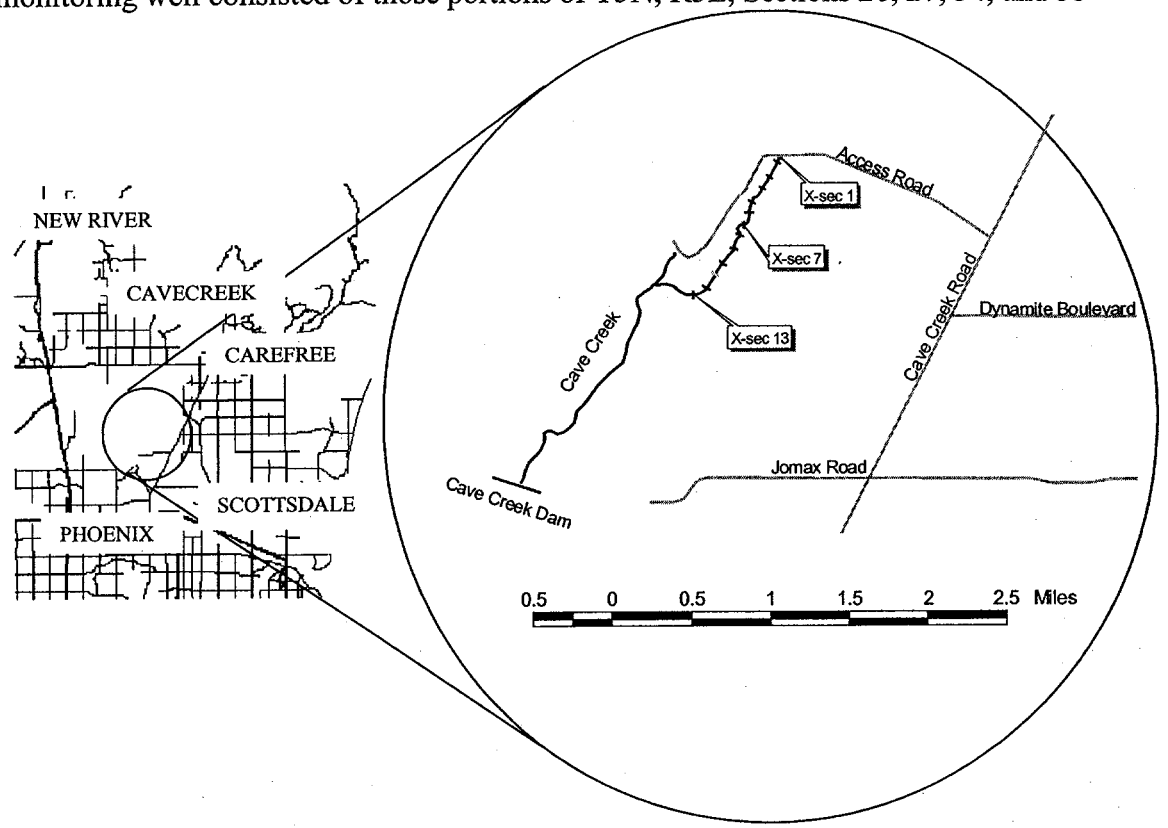


Figure 1. Location Map for Sediment Study showing Wash and Cave Creek Dam.

down gradient from the reclaimed water discharge point. The monitoring well will be used to collect groundwater samples for analysis to determine if the recharge has an impact on the native groundwater.

The City's preference was that the monitoring well be located within the lands owned by the Flood Control District of Maricopa County (FCD). The monitoring well facilities may be subjected to periodic inundation and the designs were to take this into account. The City also indicated that they may elect to use the monitoring well not only as a groundwater quality sample collection well but also as a designated recovery well for the reclaimed water recharged. This decision impacted the design parameters of the well since a recovery/monitoring well must be deeper and have a larger diameter than a groundwater quality monitoring well.

The design of the monitoring / recovery well will not be addressed in this report which deals only with the sediment testing portion of the project. The report describing the monitoring or recovery well will be provided at a later date.

2 DATA COLLECTION AND REVIEW

Earlier studies were collected and reviewed to obtain background and data for the current study. A review was completed of the pertinent reports and available data were incorporated into this study. Studies included a previous analysis of the hydraulic impacts of releases during flood events performed by CH2M-Hill in 1999.

A recording rain gage and a sonic distance meter were installed to measure rainfall and flow in the wash. The rain gage was installed in the outfall control valve enclosure and the flow meter was installed upstream from a weir approximately 500 ft below the outfall. The flow meter measured the distance to the water surface from the meter which was then converted to a depth of water flowing over the weir crest.

2.1 Sediment Samples and Wash Properties

Soil samples were obtained at 8 locations and sieve analyses were performed to determine bed gradations. The samples were very similar as shown in Figure 2. The exception is a sample of the sub-bed layer which has relatively high clay content and is resistant to erosion. The mobile portion of the wash bed consists almost uniformly of sand and gravel size classes with occasional cobble and boulder size material. The sand and gravel sizes are nearly all sharp angular quartz. It was observed that the material moves as bedload for the flows considered in this study. No waves or dunes form for these flows and the bed appears to behave as a plane bed.

One sample was taken from the sub-bed material which consists of a much less erosive material. The analysis of the material shows that it consists of approximately 20% fines. The analysis of this sub-bed material with the fines removed shows that the sub-bed material is identical to the bed material. This indicates that the material being transported along the bottom of the washes results from the removal of fines during higher flows. The fines are rapidly transported down the wash to the reservoir. This removal of fines leaves the sand and gravel sizes to move much more slowly as bed load during a wide range of flow conditions. The gradations of the samples can be found in Appendix 1. The sediment samples were obtained over the full study reach and compare very closely with those obtained by CH2M-Hill.

2.2 Study Cross Sections

A series of 13 cross sections were established by CH2M-Hill during their earlier study in order to develop an HEC-RAS model of flows in the area. Their study dealt primarily with the impact of WRP releases during high flows in the wash and concluded that no significant impact could be found due to the comparatively small WRP releases during flood events. The 13 cross sections consisted of surveyed cross sections along the west branch of the wash where WRP releases are made and elevations for over bank areas and the east channel of the wash obtained from USGS Quads. (CH2M-Hill 1999)

The original 13 cross sections were located and resurveyed for the current study. Some changes were noted in the two years between the CH2M-Hill survey and the initial

survey for this study and provided a basis for natural changes in the wash over time. The location of cross sections is shown in Figure 3. Subsequent to the installation of the weir additional cross sections were added to better characterize sediment transport based on sediment retention upstream and scour downstream of the weir.

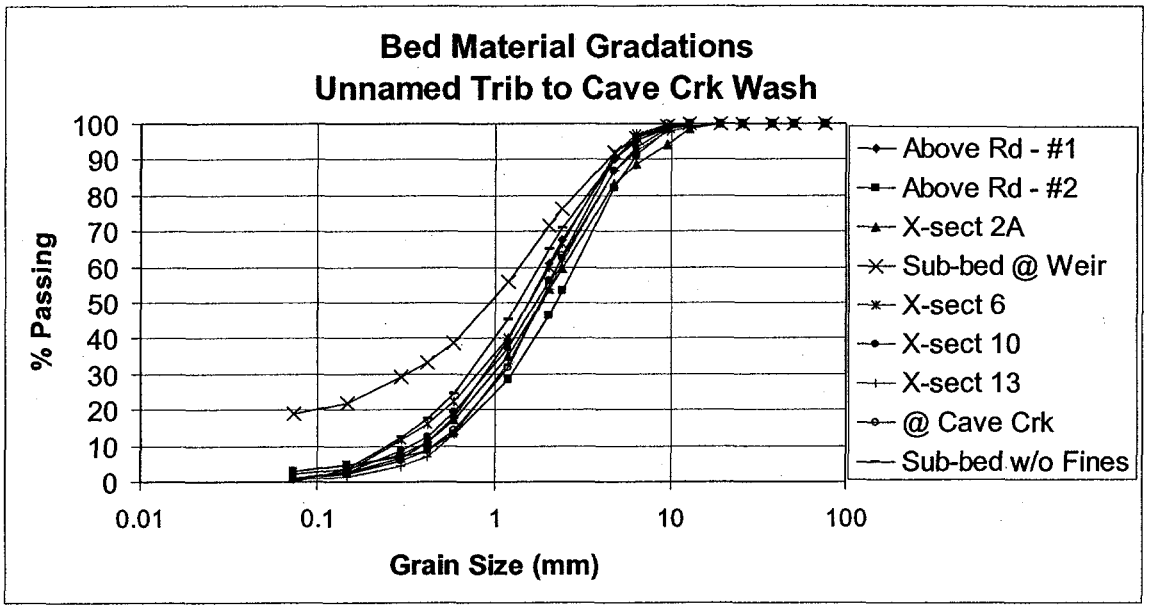


Figure 2. Bed Gradations for the Unnamed Tributary to Cave Creek Wash.

The added cross sections were numbered as 2A, 2B, 2C, 3A, 4A, and 5A to differentiate them from the original 13 while retaining their relative position on the wash. The weir was located between cross section 2A and 2B as shown in Figure 3.

3 Weir Installation and Calibration

In order to measure flows resulting from rainfall in the area as well as to provide an additional measurement of flow in the wash during test releases, a weir was installed approximately 500 ft downstream from the outfall structure. In order to insure a stable rating curve and eliminate conjugate depth problems it was necessary to raise the floor of the weir above the bed of the channel by approximately 1.0 ft. This unavoidably led to the trapping of sediment upstream of the weir and the erosion of bed sediments downstream from the weir.

The crest of the weir was 13 ft wide with side slopes of 1V:1H. Upstream of the weir a concrete faced contraction was placed in the channel to insure that the flow concentration at the weir did not erode the contracting dike and bypass the weir. Some flow bypassed the weir during the 22.6 MGD rainfall event but the upstream end of the dike was repaired by raising the height of the damaged section and sandbagging to insure flow did not bypass the weir during subsequent high flows. After repairs were performed no further flow bypassed the weir. The weir is shown in Figure 4 during the 10.5 MGD release and the rating curve is shown in Figure 5 along with the measured flows.

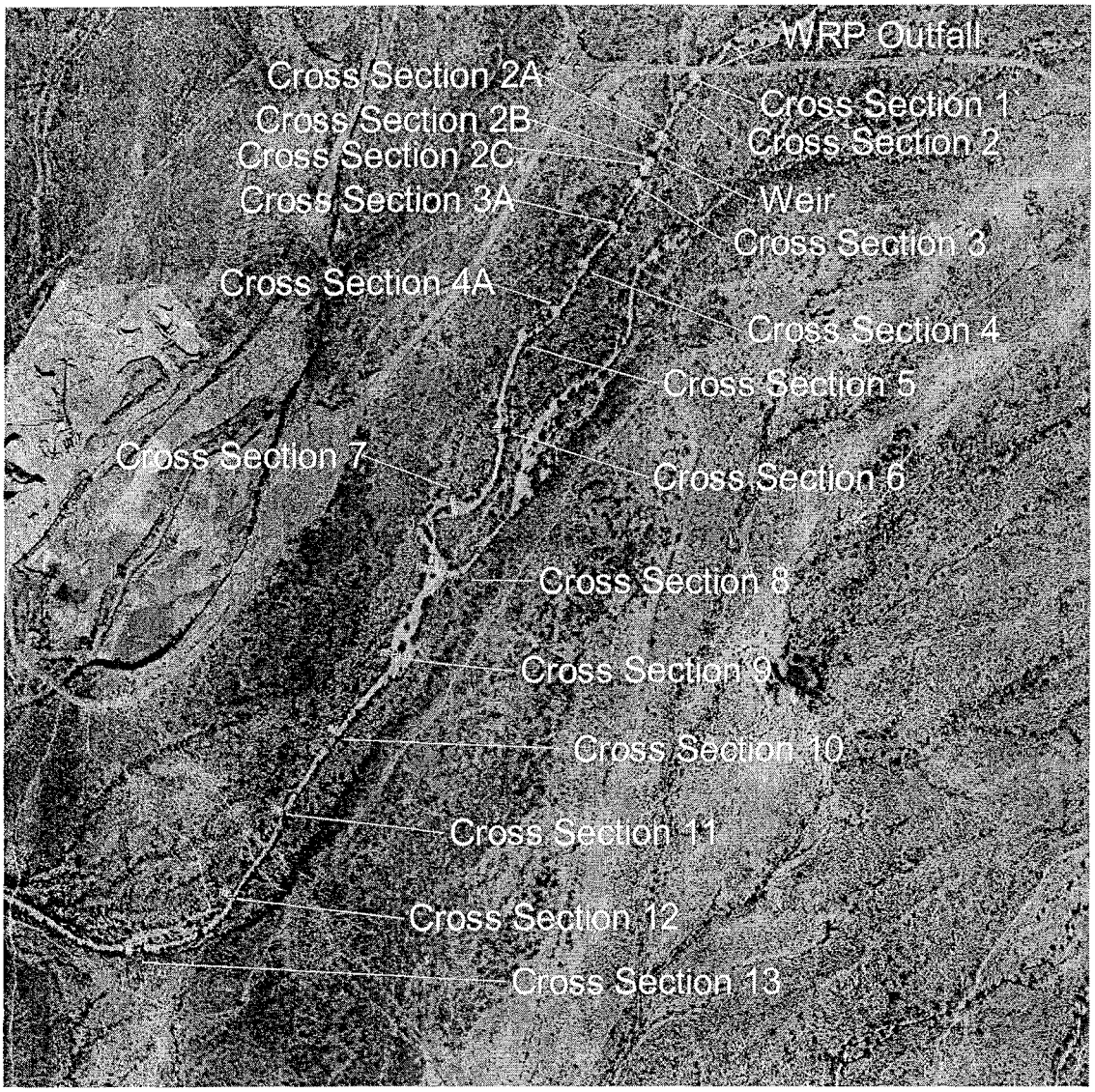


Figure 3. Cross Section Locations for Sediment Transport Study.

During the initial low flow events significant differences were discovered in flow measurements at the weir and at flow meters associated with the WRP. These differences were analyzed and flow measurements taken to calibrate the weir. It was concluded that the errors were due to poor accuracy of the weir at low flows. In order to insure that the weir would pass the largest expected events (24 MGD) without overtopping the banks of the wash it was necessary to widen the weir crest to 13 ft. The requirement to pass large flows resulted in errors for low flows on the order of ±13%. Errors for the higher flows were on the order of ±4%.

It was hoped that the weir pool could be used to calculate infiltration rates based on the reduction in pool elevation over time. The reduction in pool elevation, however, corresponded very closely with the amount of seepage under and around the weir. This

value, while small, was sufficient to account for all of the reduction in pool volume. Infiltration in the wash upstream from Cave Creek thus appears to be low and given the dense structure of the sub-bed material, little infiltration would be expected. The material in the sub-bed has relatively high clay content and appears to be highly impermeable.

Figure 4. Measurment Weir near the Conclusion of the 10.5 MGD Release. Note that the Weir Pool has Filled wth Sediment and Bed Material is being Transported Across the Weir by the Flow.

An additional problem that occurred during prolonged periods of high flow was that the weir pool would fill with sediment. During the shorter rainfall events the bottom of the weir pool was covered with a layer of very fine sand and a surface veneer of fine material (clays) that came for the natural soils in the area. This material tended to seal the weir pool and reduced seepage under and around the weir below the amounts that were originally observed. The upstream side of the weir was also excavated and refilled with sand bags which were covered with material from the sub-bed. This reduced seepage losses to approximately 1 gallon per minute.

The primary losses associated with initial flows in the wash are due to the filling of the void space in the sand and gravel bed material. This volume accounts for a significant portion of the early flows in the wash. Assuming 30% voids which is typical for coarse sands and gravels, an average bed width of 25 ft, a bed sediment depth of 0.5 ft and a length of 7,000 ft to the confluence with Cave Creek; the volume required to fill the bed is 26,250 cubic ft or 196,000 gallons. (0.196 million gallons).

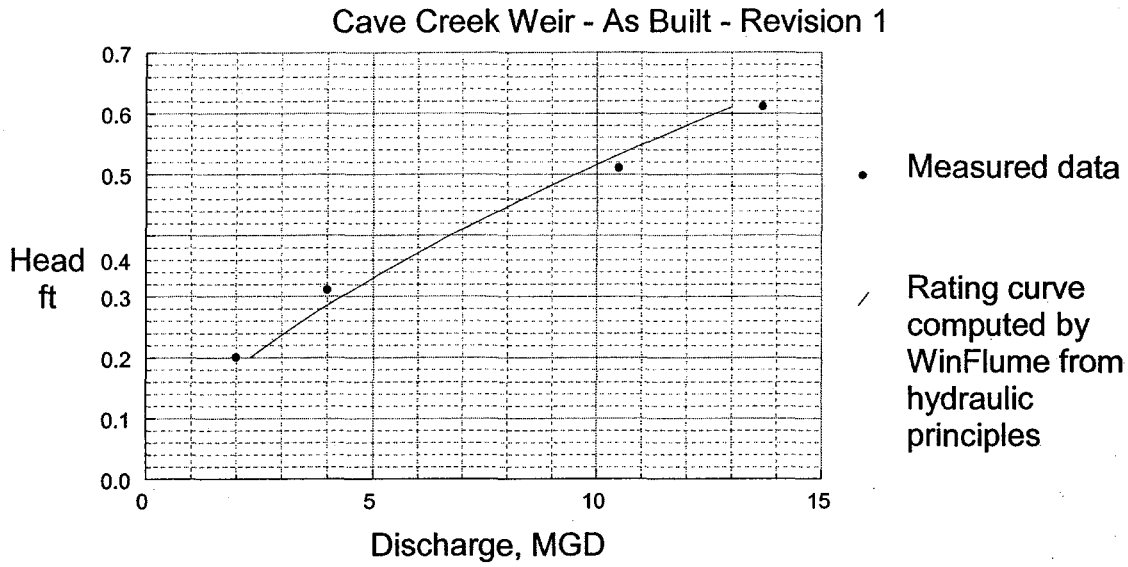


Figure 5. Weir Rating Curve and Measured Flows.

During the releases from the WRP no sediment was carried into the reach by the inflowing water and the only sediment transported (other than an initial flush of debris and some initial fines) was the bed load material. The bed material consisted of sand and gravel size particles and the sediment moving between the outfall and the weir was trapped in the weir pool. This trapping of the bed load required the cleaning of the pool prior to both the 10.5 and 13.7 MGD releases to insure weir accuracy.

The pool filled completely during the 10.5 MGD release and at the completion of the release the weir measurements were approximately 16% low due to the reduction in water surface elevation attributable to velocity head at the meter. The wake from the meter support can be seen in Figure 4 as can the sediment being transported over the weir crest. Normally flow over the crest of the weir was clear with little, if any, suspended load.

4 Development of Flow Rates for Testing Plan

Several rainfall events occurred prior to the initial test release. Natural flows resulting from rainfall on the watershed on January 16 produced flows of approximately 1.0 million gallons per day (MGD). An event that was not captured due to a malfunction of the data logger occurred on January 13 and produced a peak flow of approximately 2.0 MGD. This value was determined from high water marks on the manual gage. Neither of these flows produced discernable changes in the bed of the wash. After the 2.0 MGD natural flow event the position of the low flow channel could be discerned in parts of the channel where flow was concentrated but in other portions of the channel the flow path could not be located. The changes in bed elevations were too small to be detected in a survey of the cross sections and surveys were not performed after these small events.

From these events it was apparent that flows below approximately 2 MGD would not result in any measurable sediment movement in the wash. Based on this data it was determined that the minimum flow release would be 2 MGD and the highest would be limited by the maximum that could be delivered by currently installed pumps at booster pump station 7 (BP-7). The City desired to estimate sediment transport rates up to 24 MGD which is substantially higher than the current plant and booster pump capacity.

The volume for each release was set at 4.0 million gallons which corresponded to the daily storage capacity at the WRP. It was desired to keep the volume constant to be able to compare sediment movement amounts based on a constant volume of water passing down the wash. There was some confusion regarding allowable release volumes to avoid exceeding permit limits that would trigger reporting requirements and requirements for the installation of the observation well. Upon further investigation it was ascertained that the allowable release volumes were based on annual averages rather than monthly averages.

As a result of this confusion the initial two releases were reduced slightly in volume to stay under an average daily release of 0.25 MGD for the month. This reduction may have slightly impacted sediment transport volumes but the reductions were less than 10% of the volume per release. After it was discovered that the reporting and well installation trigger values were not based on monthly releases but on the total annual releases averaged to a monthly basis the release amounts were then increased such that the full 4 million gallons were released for the 10.5 and 13.7 MGD releases. Transport rates were calculated based on the actual time water was released.

4.1 Rainfall Events

Early in the program and prior to any project releases a significant storm occurred on the watershed and produced a flow that peaked at approximately 22.6 MGD. The exact discharge is estimated since a portion of the flow bypassed the weir. The weir measurement indicated a flow rate of 19.1 MGD plus flow in an area that was ultimately 5 inches deep by about 5 ft wide. It is estimated that the bypass flow peaked at approximately 3.5 MGD (5.5 cfs) resulting in a total flow of 22.6 MGD.

The flow from this rainfall event was significantly higher than the flow that was expected to be the maximum that could be supplied by the plant. This flow produced major changes in the wash due to sediment retention upstream from the measurement weir and scour downstream of the weir. Sediment transport volumes for this flow were adjusted to account for the difference in flow volume between the natural flow and the 4 million gallons released during a test release. The peak flow for the rainfall event lasted about 1.5 hours with a second peak occurring approximately 4 hours later. The rainfall and flow data for the weir only are shown in Figure 6.

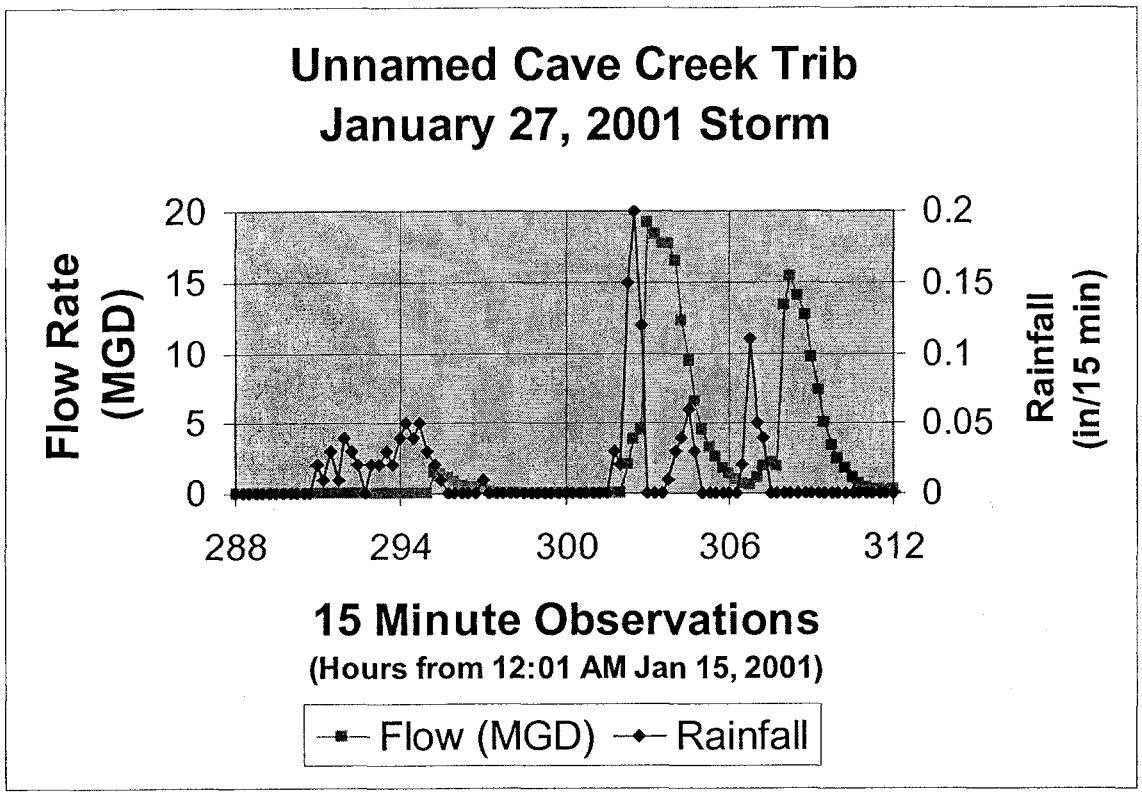


Figure 6. January 27, 2001 Storm Event and Resulting Flow at Project Weir. Flow at Weir was Higher than Shown due to Flow Bypassing the Weir. Bypass flow estimated at 3.5 MGD.

4.2 Cave Creek Water Reclamation Plant Releases

Releases from the WRP started with the 2.0 MGD flow and surveys were performed before and after the release. The decision to perform the lower flow tests first were designed to measure sediment transport rates before any further depletion of the bed sediments or changes in the wash due to high flows.

The second release was set at 4.0 MGD in accordance with the release plan agreed to at the beginning of the project. The remaining flows were planned to be at 8.0 and 16.0 MGD. These flows were selected to give a wide spread of the observation points and allow the construction of a prediction equation for sedimentation in the wash.

WRP releases were planned from lower to higher flow rates with the exception of the proposed 16 MGD release which was planned before the 8 MGD release to allow adjustment of the 8 MGD release if the release of 16 MGD could not be sustained by the plant.

After the 4 MGD release the next planned flow was 12 to 16 MGD or as close to 16 MGD as the installed pumps could deliver. After an initial high flow of approximately 13 MGD lasting an hour or so, pump flow at BP-7 decreased to 10.5 MGD and vibration problems became evident in one of the two booster pumps. The test was then continued

at the 10.5 MGD for enough time to release the full 4 million gallons. The flow data calculated at the weir is shown in Figure 7. The tapering off of flow is due to the filling of the weir with sediment and subsequent accuracy loss by the meter. It also appears that flow may have been nearly lost for a short period at the time initially scheduled for pump shut down. It appears that the WRP pumps were shut down for this period of time.

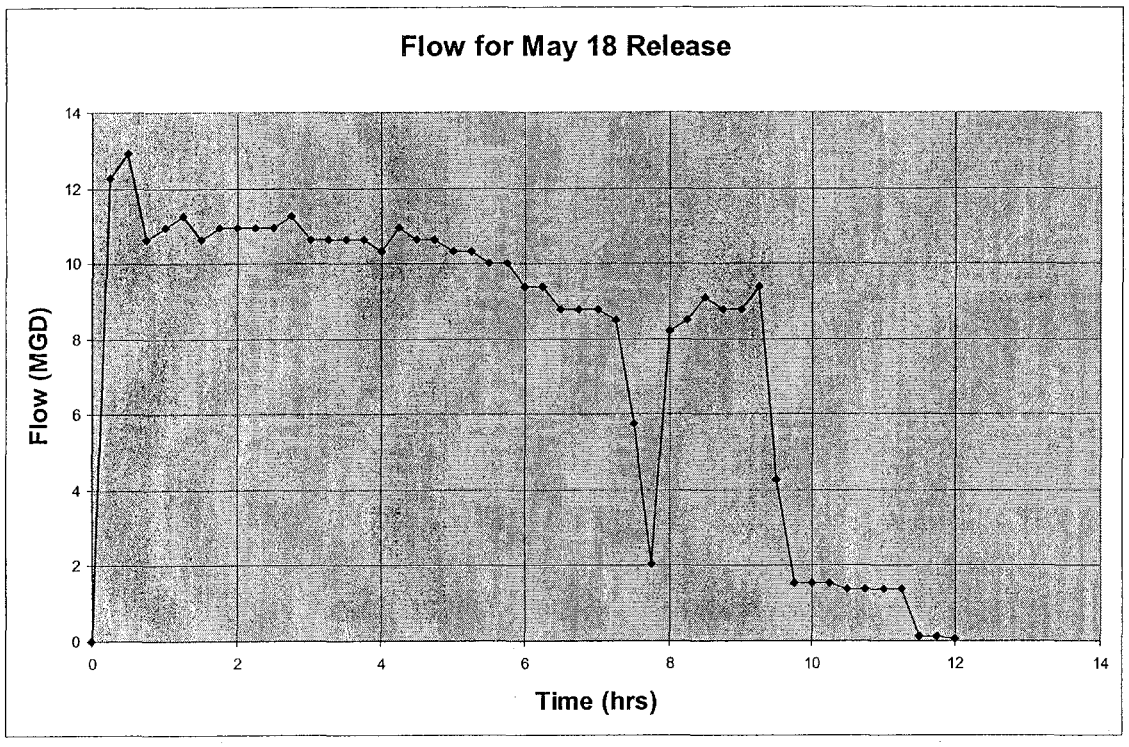


Figure 7. Flow Data For 10.5 MGD Release On May 18, 2001. Weir Pool Filled And Reduced Meter Accuracy After About 5 Hours. Sharp Drop In Flow At Hour 7.5 Was Probably Due To Miscommunication Regarding Total Length Of The Release.

Subsequent to the 10.5 MGD release discussions were held regarding whether the subsequent and final release should be at a higher or lower flow rate since the 10.5 MGD release was not as low nor as high as had been planned.. It was determined that it would be more important to have data at the highest flow rate possible rather than at an intermediate flow rate. The final release was at 13.7 MGD which was the highest volume that could be continuously provided to the outfall with the currently installed pumps at BP-7.

The combination of the rain event at 22.6 MGD (peak) and the four lower test releases gave an adequate spacing of data points and the widest range possible given the current level of plant expansion and available data.

5 Sediment Transport Calculations

The calculation of sediment transport was designed to be accomplished by measurement of changes observed in the field. This provided an opportunity to not only observe sediment transport but to compare observed values with those predicated by sediment

transport equations recommended by such software packages as the SAM model developed by the U.S. Army Corps of Engineers.

Sediment transport rates for the observed flows were calculated based on data from the surveyed cross sections. Data was consistent for the most part and the bed material in the wash had an opportunity to dry prior to the next release or storm event. Releases and surveys were spread over a five month period as shown in Table 1. Two rainfall events and five sets of operational releases occurred during this time period in addition to the test releases. The operational releases were primarily conducted to check the operation of valves, meters and controls in the WRP distribution system.

Table 1. Dates of Surveys, Natural Flows and Operation Flows with Maximum Flows and Time Between Flows.

Flow Event	Survey Date	Natural Flow	Operational Release	Release	Max Flow (MGD)	Drying Time (days)
1	Jan 11	Jan 17			0.95	
2		Jan 27			22.6	
3	Jan 30			Feb 6-8	2	10
4	Feb 9		Feb 23		4.2*	15
5	Mar 5			Mar 5-6	4.0	10
6	Mar 9		Mar 14		13.6	
7			Mar 15		2.3	
8			Mar 19		5.5	
9			Mar 20		6.3	
10	Apr 2					
11	Apr 27			May 18	10.7	59
12	May 22			Jun 6	13.7	20
13	Jun 7					

* Actual maximum was higher but meter was not recording for part of this period.

5.1 Sediment Transport

Surveys were taken at each of the dates shown in Table 1 for each of the original 13 cross sections as well as the 5 additional cross sections shown in Figure 2. The cross section data proved difficult to analyze for bed changes since most of the flows resulted in very small changes in bed elevations that represented long (500 ft) reaches of the channel. Further if the survey did not occur immediately after a flow the bed material was rearranged by the action of cattle, motorcycles and four-wheelers. In places where four-wheelers practiced turning cookies, for example, the change in the bed could be significant.

The actions of the cattle and motorized vehicles rearranged the bed and the displaced sediment was higher (i.e. less dense / larger volume) than the surrounding undisturbed sediment. This small change in elevation could change a stable section into a section that showed deposition occurring during an event. For the most part surveys were conducted

within one to four days after a release and were not impacted by these problems. The surveys after operational releases were delayed longer and were more likely subject to this change in elevation. Most of the surveys for the operational releases showed either sediment gains or at best no significant change in sediment volume even though some of the operational releases showed high flow rates for short periods of time and moderate flows for 4-6 hours at a time as shown in Figure 7 for the March operational releases.

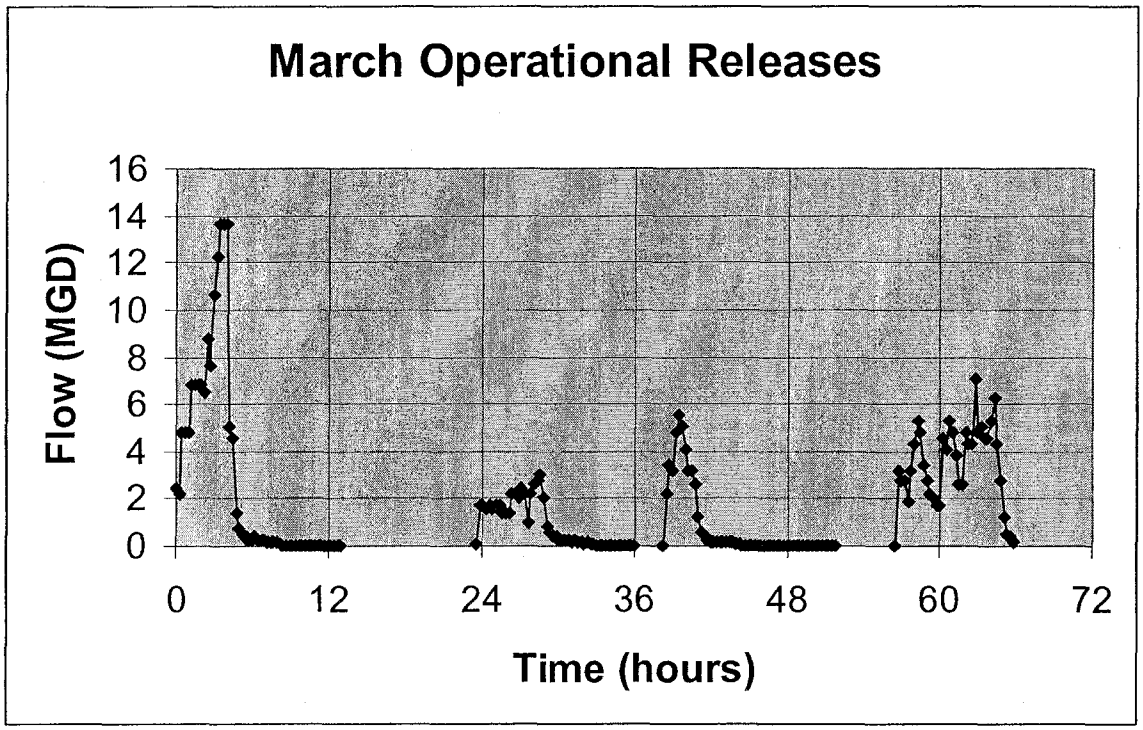


Figure 8. CCWTP Operational Releases for March. The Time Scale for No Flow Events is Shortened to Provide Better Resolution of Release Events.

Downstream of the weir the 22.6 MGD flow removed all of the bed material down to the clayey sub layer for approximately 100 ft downstream after the initial storm event. This bare area persisted throughout the rest of the testing period. The length of the bed with out moveable sediment continued to lengthen with time due to the lack of inflowing sediment and the trapping of upstream sediment in the weir. Sediment was removed from downstream of the weir to beyond cross section 3 prior the 10.7 MGD release and inspection subsequent to the 13.7 MGD release showed removal of bed material reaching to cross section 5.

Sediment moving into the weir pool from upstream moved along the bottom of the channel as bed load and dropped over the edge of the advancing delta until the pool filled and sediment began to move over the weir. The weir pool required cleaning on two occasions to restore weir accuracy. Sediment removed from the pool was stored locally so that it could be restored to the wash upon the completion of the study. It was expected that natural events would restore the sediment balance in the wash after a few significant

flow events. One significant event occurred during the summer of 2001 but had not, as yet, restored the sediment balance in the wash.

The weir pool first filled with sediment during operational releases in March and was initially cleaned on May 9, 2001. The pool refilled during the 10.5 MGD release and was again cleaned on May 23, 2001. The pool did not completely refill during the 13.7 MGD release indicating a lower sediment transport rate – probably due to a lack of transportable material upstream of the weir. It was readily apparent that all easily transportable material had been removed from cross section 1 and the material remaining at cross section 2 may have been left due to a backwater effect from the weir at the higher flow rates.

Sediment transport rates for flows released in this study are negligible. Sediment carried from the unnamed tributary into Cave Creek wash is deposited in a pool located at the confluence and is not transported down the wash. The sediment delivered to Cave Creek Wash will be transported further into Cave Creek Reservoir during high flows in Cave Creek Wash.

5.2 Calculation Procedure

The survey data was obtained at pre-selected surveys at 13 to 18 locations along the wash as shown in Figure 2. The cross-sections were plotted providing a chart showing the depths as a function of the horizontal distance across the wash as shown in Figure 9.

To perform the sediment transport calculations, it is essential to quantify the vertical change in the stream bed elevation. For this purpose, a reference depth was used at every cross-section and the area between the bed cross-section and the horizontal line at the reference depth was calculated. Several of the cross sections had to be shifted horizontally to align properly. The shift was not large but since the banks were stable, it was apparent that the horizontal alignment of the cross sections should not have been shifting and the cross sections were shifted slightly (usually 1-2 ft) to align correctly.

A plot of cross section 1 (immediately below the outfall) is shown in Figure 9 along with the width over which the calculations were made (labeled Eval Range). The elevation of the line marked Eval Range is also the elevation of the datum that was used to calculate changes in cross sectional area. It is assumed that the bed change at the cross-section is applicable over a region extending to a distance half-way towards the two neighboring cross-sections upstream and downstream. Under this assumption, the change in the sediment volume of the stream bed is calculated using the change in area at the cross-section and half the distance to the neighboring cross-sections. It was not possible to include the banks in the calculations since the points at the very edge of the channel were not always the same as can be seen in Figure 9. The lack of exact beginning and ending points and thus the need to shift horizontal points slightly can be seen in Figure 9 by noting the variation in the end point of the survey.

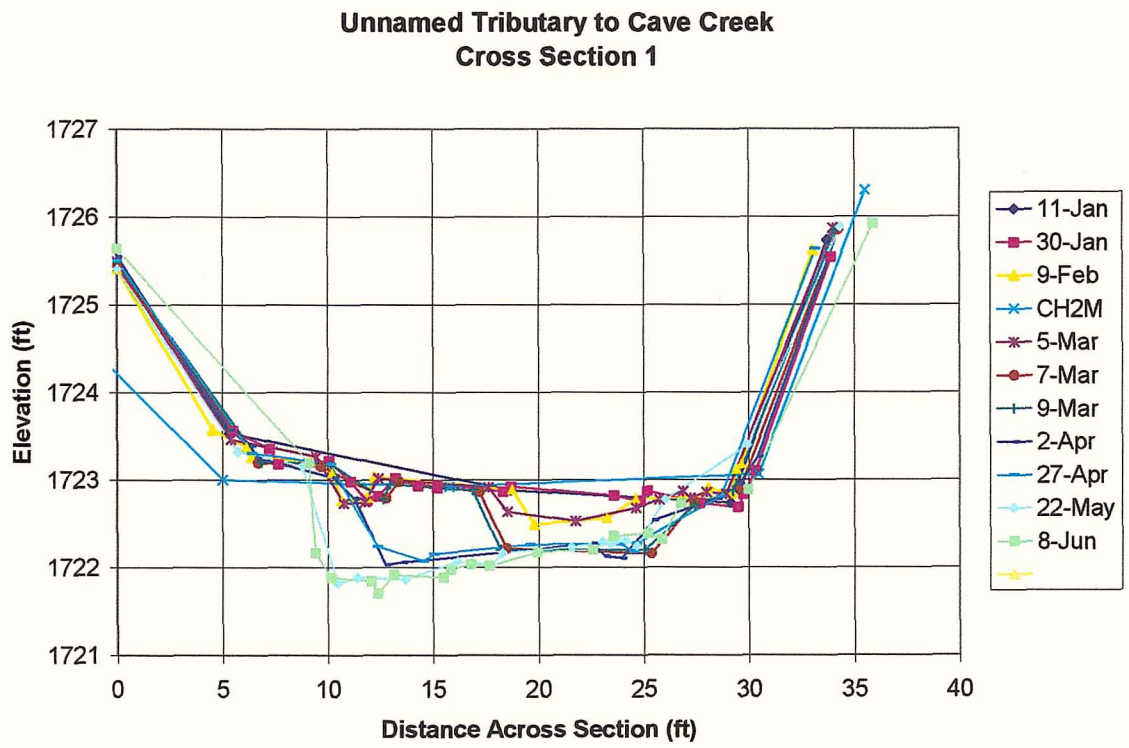


Figure 9. Cross Section 1 Surveys Showing Channel Change with Time. (Looking UPSTREAM)

The effective volume change during each flow event was then obtained by comparing the sediment volume above the datum from the survey data prior to and after the flow event. Assuming 30% voids, a sediment specific density of 2.65 and the time duration of the flow event, the sediment transport rates were calculated in terms of tons per day for each cross section.

The analysis produced a large amount of scatter when analyzed cross section by cross section as shown in Figure 10. This is due in part to the inherent inaccuracy of surveying at a limited number of cross sections. Transport rates were averaged over the entire reach to obtain a better estimate of the actual rate associated with flow in the wash.

5.2.1 Error Associated with Surveying Cross Sections

Two surveys were also taken where no flow occurred between the surveys. These surveys were taken on the 2nd and 27th of April as shown in Table 1. The total volume difference between these two surveys was 86 cubic feet or 5 tons of sediment assuming a specific density of 2.65 and a porosity of 30% (i.e. a weight of 116 lb/cubic ft).

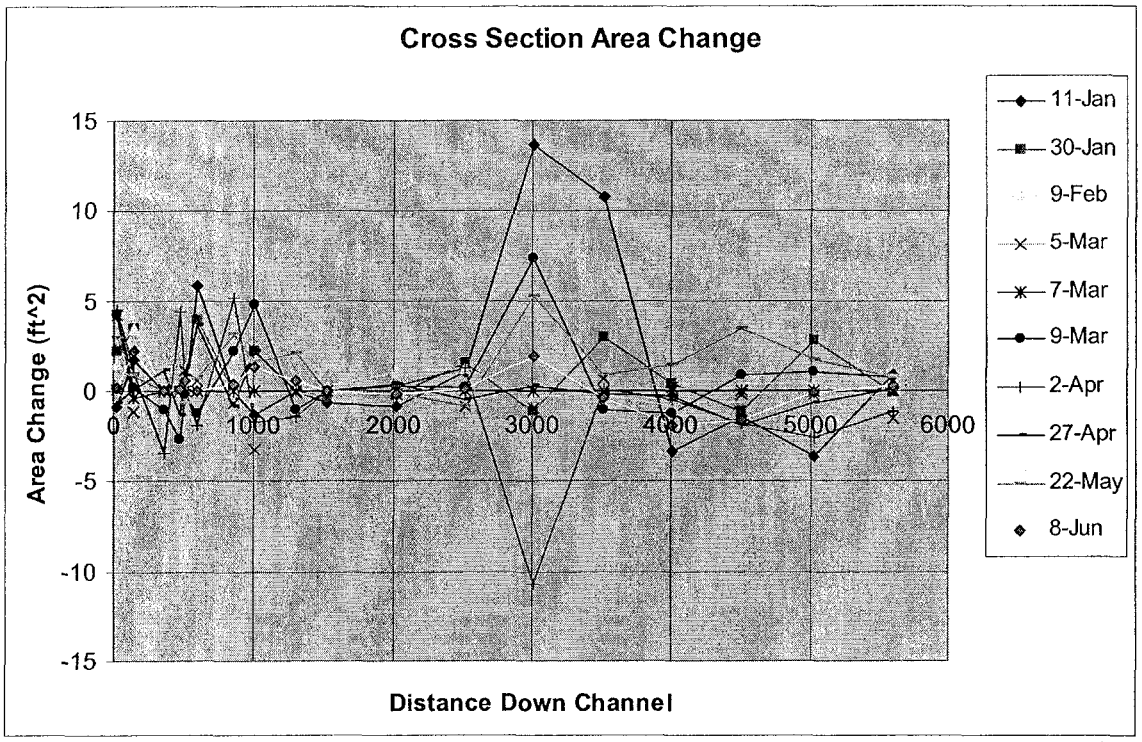


Figure 10. Cross Sectional Area Change Between Study Surveys. Positive Values are Erosional and Largest Values are at Cross Section 8.

The second survey with no flow was the result of being rained out during a survey after a flow event in early March. The first day rain started after only two cross section surveys and surveying was suspended. The crew returned two days later and completed a full survey. No flow occurred as a result of the rain and no releases were made. The difference between the March 7th and March 9th survey was -79.2 cubic feet or a 4.6 ton loss in just two cross sections.

Each cross section was then evaluated as an individual data point. The 17 cross sections contained in the April surveys plus two from the earlier March 7th survey that was rained out were analyzed to obtain a better estimate of survey error. The data indicates that the volume differences in the 19 data points range from -626 to 967 cubic feet with the majority of observations being less than ±300 cubic feet as shown in Figure 10. The average change was 31 cubic feet when averaging positive and negative values. This indicates a slight positive skew which can be seen in Figure 11. The values on the x-axis are the average values for the observation category – i.e. the category marked 0 covers data from -150 to +150 cubic feet, the category marked -300 covers the data from -150 to -450 cubic ft and so forth.

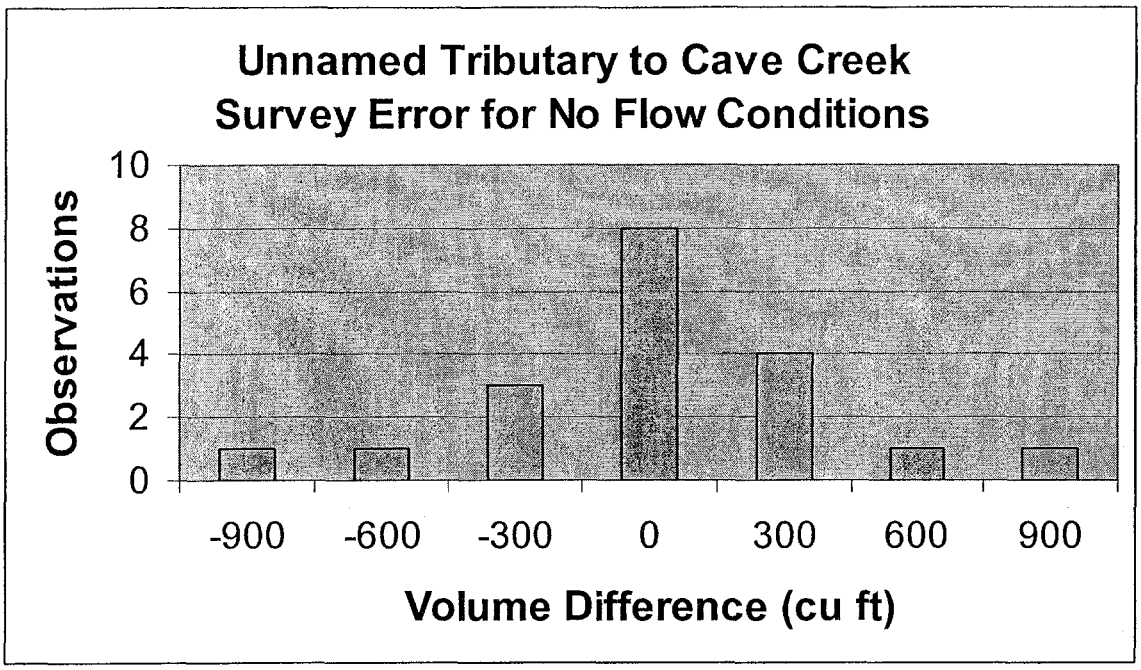


Figure 11. Sediment Volume Error attributable to survey. Surveys were before and after no flow periods.

This data indicates that the error in the surveys is random and low data is as common as high data in the survey errors. This would indicate that there is no significant bias in the data obtained from the surveys. If the change in volume is reduced to the change in area at the cross sections, values vary from 13.6 and -10.7 square feet. It can be noted from Figure 10 that these values are for cross section 8 and that most other points are less than ±5 square feet.

5.2.2 Analysis of Survey Data for Sediment Transport

The survey data was used to calculate the change in area for each cross section between two surveys. The areas were multiplied by the lengths of the reach the cross section represented and this volume was then converted to a transport rate based on the time over which the flow occurred. These data were then weighted to account for reach lengths and averaged for the reach to obtain average rates for each event / release. The observation values are shown as Wtd Average in Figure 12.

The data obtained from the survey computations were then compared to the transport rate predicted by the Meyer-Peter and Mueller Equation (MPM). Other equations were evaluated but none predicted the observed values as well as the MPM equation. Two variations of the equation exist, one as used in HEC-6 and one based on the D_{50} of the bed sediment. Both equations give similar results as shown in Figure 12. The lines marked upper and lower are based on the use of values from the upper portion of the wash (cross sections 1-7) or the lower portion of the wash (cross sections 8-13).

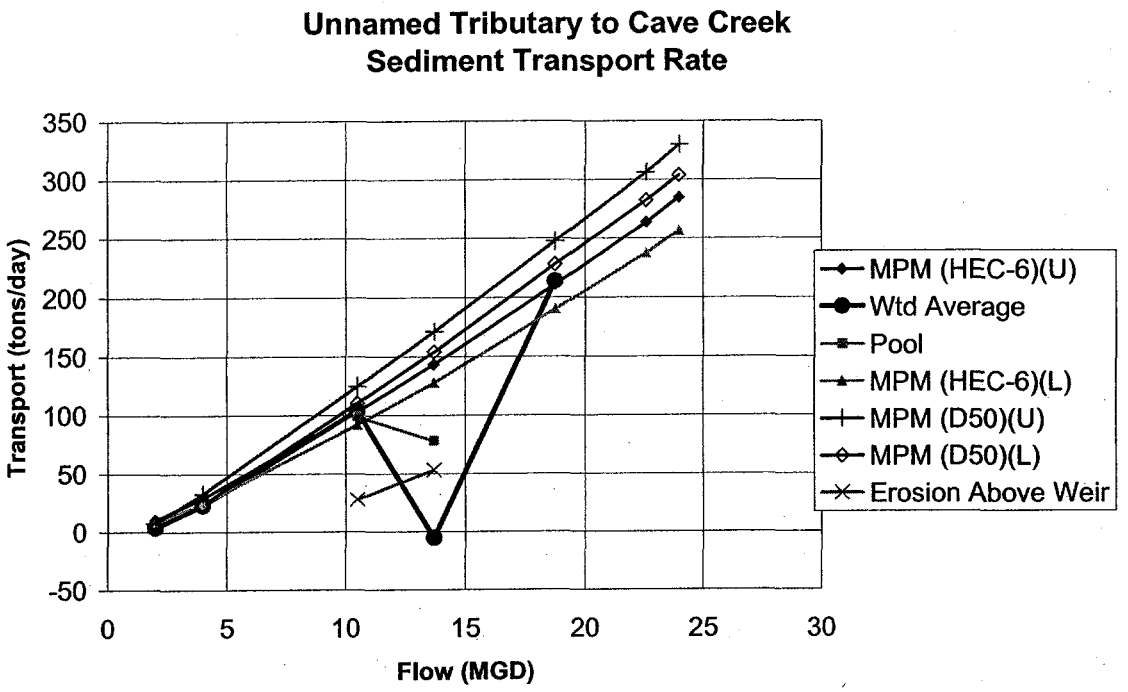


Figure 12. Sediment Transport Rates in Tons per Day for both Observed and Prediction Equations.

The observed data were evaluated using two averaging methods. The use of a straight average produced slightly different results but the reach length average data is more accurate and is presented here.

The MPM equations predictions are very close for both equations but the HEC-6 version appears to better predict the observed values when applied to the general cross section data in the upper reach of the wash (i.e. above cross section 8) while the D_{50} version predicts more closely when used with the cross sectional data for the lower portion (cross sections 10-13) of the wash. Either equation appears to predict within 20% of the observed rate with the exception of the 13.7 MGD release. The transport rate values are shown in Table 2.

5.2.3 Discussion of Transport Data for 13.7 MGD Release

Sediment transport rates for the various releases appear to be approximately what would be predicted by various sediment transport rates with the exception of the data from the 13.7 MGD release. The data indicated that not only did no erosion occur but slight deposition occurred during the release (i.e. sediment was produced). This large reduction in sediment transport produces a problem for analysis as well as for development of a prediction equation. Various causes of error were evaluated and data were carefully reviewed to determine the cause of the problem. The amount of sediment shown to be produced is certainly within the error of surveying described above and can be ignored.

This still leaves a problem of very little sediment transport for the highest controlled release.

Table 2. Observed Transport Rates based on Cross Section Data and Weir Pool Cleaning Data. Negative Values show Negative Transport Rates.

	Transport Rates - Tons / Day	
Flow	Distance Weighted Average	Pool Cleaning Data
2	4	
4	23	
10.5	104	99
13.7	-3	78
18.75*	213	
Operational Releases		
5**	-13	March Releases
4***	-2	February Releases

* Flow Rate Averaged for Two Natural Events. Peak flow was 22.6 MGD.
** Flow Rate Averaged for Three March Releases
*** Average Flow and time were estimated

The full data from the 13.7 MGD release show almost no erosion occurring during the test due to deposition at cross sections 9-12. This deposition was more than the erosion occurring upstream. A first reaction in a situation like this is to examine the survey for errors.

Survey data for the 13.7 MGD release are almost identical to those from the 10.5 MGD release. Some areas show slight erosion or deposition but the cross sections are very similar and appear to be accurate. The survey data appears to be reasonable and shows little if any change over the length of the study area. The surveys appear to contain adequate points upon which to base calculations and no errors are apparent. This leaves the possibilities that 1) the sediment content of the bed has been depleted for a significant portion of the 5,600 ft study reach, 2) that erosion was occurring somewhere that was not accounted for in the cross sections or 3) that some other process reduced the availability of sediment for transport.

The main reduction in transport rates, however, occurred due to deposition in cross sections 9 through 12. This deposition may have been due to widening of the channel at cross section 8 that was not reflected in the measured cross section or due to a sediment wave that is moving slowly down the channel. Cross section 8 is very wide and is impacted by channel instability between cross sections 7 and 8. The surveys indicate that cross section 8 widened significantly during the 10.5 MGD flow and only slightly during the subsequent 13.7 MGD flow. (See Figure 13) An on site review of the cross section showed there is a large rock that is located almost exactly on the line of survey that

appears to have acted as bank protection for the 13.7 MGD flow as can be noted in Figure 14. This rock appears to have collapsed from the bank sometime after the 10.5 MGD flow and probably accounts for the relatively small widening noted in Figure 13 between the 22 May and 8 June surveys. It may have protected the bank against erosion during the 13.7 MGD and reduced the resultant transport rates. It is doubtful that this caused the lack of transport due to the deposition at cross sections 9-12 being greater than the erosion noted at cross section 8. Nearly all transport rates are either lower or negative for the 13.7 MGD release.

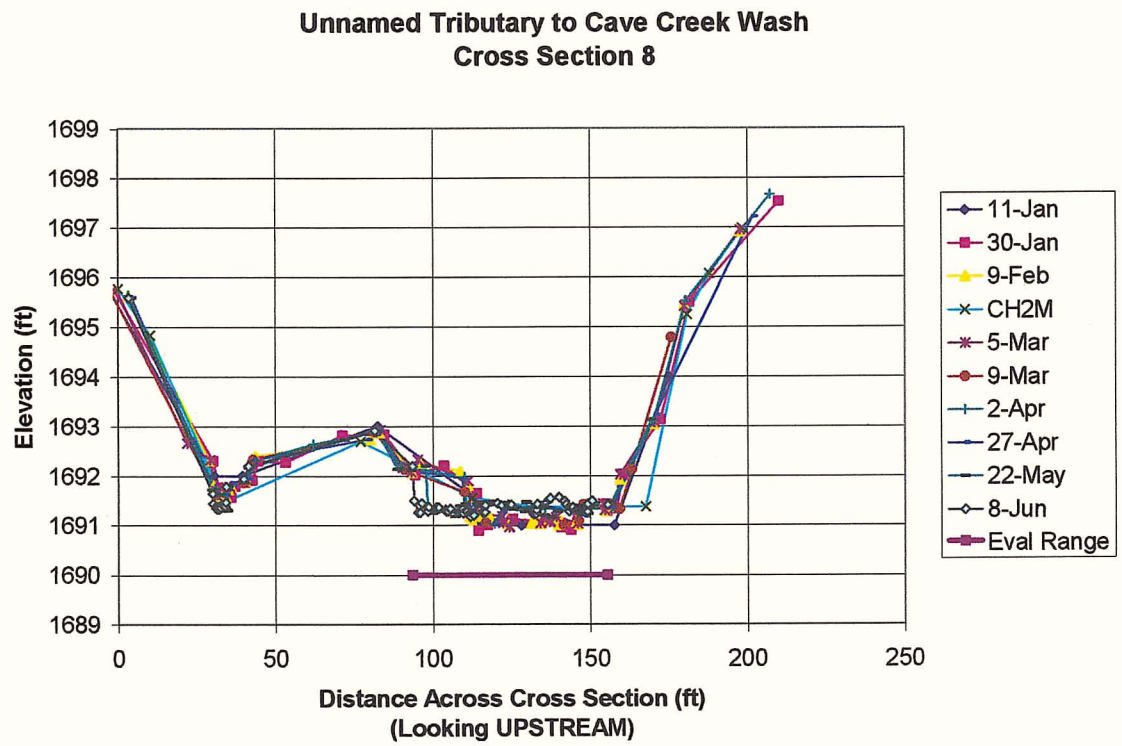


Figure 13. Survey Data for Cross Section 8 showing Evaluation Range and All Surveys Performed. Sudden Widening During 10.5 MGD flow (22 May) can be noted.

Another explanation of the lack of sediment transport could be a lack of available sediment for transport within the wash. The lack of sediment could account for some reduction in transport rates since the volumes transported into the weir pool were also reduced for the 13.7 MGD release. The transport rates based on the volumes captured in the weir pool are shown in Table 2 and volumes are shown in Table 4. Even this rate for the 13.7 MGD release, while significantly higher than that for the entire reach is low compared with transport for the 10.5 MGD release. The actual volumes of sediment eroded above the weir and that trapped in the weir area shown in Table 3. It appears that the erosion that contributed to the deposition in the weir pool for the 10.5 MGD release was eroded between cross sections 1 and 2 since the deposition is much higher than the erosion. Additionally, sediment that was previously removed from the pool was redeposited in the pool when the current moved to the west bank of the pool and attacked the stored sediment.

Figure 14. Cross Section 8 Looking to West. Cross Section Passes Through Opening in Bushes Above Rock and to Right of Mr. Sundararaghavan.

Table 3. Volumes Eroded Above and Trapped in the Weir Pool.

Calculations from WRP Outfall to Weir (Volumes in Cubic Feet)			
	Eroded	Trapped	Ratio of Values
1st Weir Cleaning	-954	991	1.038
10.5 Weir Cleaning	-188	651	3.46
13.7 Weir Pool Trapped	-351	395	1.125

Subsequent to the 13.7 MGD release it was noted that the bed material was removed from the weir to cross section 5 during the study. It should be noted that approximately 10% of the channel length lies above the weir and is thus captured by the weir. The distance to cross section 5 is 1580 ft or 28% of the study length. This portion of the study area was thus either trapped by the weir or had the easily erodible sediment removed from the reach prior to the conclusion of the 13.7 MGD release.

This lack of transportable sediment appears to be the most likely cause of the reduced sediment transport rates. The total volume of easily erodible bed sediment is estimated to be approximately 80,730 cubic ft or 2990 cubic yards (4,660 tons). This assumes an average channel width of 23 ft and a sediment depth of 0.5 ft for the entire 7020 ft to the Cave Creek confluence. Estimates of sediment transport in a single release event or a

Table 4. Erosion and Deposition Volumes for Cave Creek Water Reclamation Plant Sediment Studies on Unnamed Tributary to Cave Creek Wash.

Cave Creek Water Reclamation Plant
Sediment Volume Changes (ft^3)
Negative numbers indicate Erosion

Sediment Tests from 2001
WEST Consultants, Inc.
ND = Insufficient Data

	CH2M-Hill - Jan Difference	Big Storm 22 MGD	2 MGD	Small Storms	4 MGD	Misc/Rain	No Releases	10.5 MGD	13.7 MGD	Total* Gain / Loss	Study** Gain / Loss
Dataset	11-Jan	30-Jan	9-Feb	5-Mar	9-Mar	2-Apr	27-Apr	22-May	8-Jun		
xs1	70	-182	-123	-10	-343	-371	57	-242	-13	-1157	-1227
xs2	-595	-1301	-1300	203	-38	-194	-11	-231	-570	-4039	-3443
xs2a	ND	ND	ND	14	178	576	-197	ND	ND	572	572
Weir	ND	ND	ND	ND	200	-326	102	-265	-11	-300	-300
xs2b	ND	ND	ND	-161	15	-9	-56	45	-88	-253	-253
xs3	-1960	-1476	-139	-98	214	328	-626	-117	ND	-3874	-1914
xs3a	ND	ND	ND	130	-446	-1053	178	-643	-113	-1947	-1947
xs4	598	-991	132	727	-1061	387	-478	-367	-293	-1346	-1943
xs4a	ND	ND	ND	14	475	645	-7	-938	-239	-50	-50
xs5	377	204	-167	201	ND	ND	ND	ND	ND	615	238
xs6	428	97	-515	106	-202	-218	-263	39	129	-399	-827
xs7	-536	-773	-616	409	-106	-624	248	413	-39	-1623	-1087
xs8	-6811	592	145	465	-3671	5338	-154	-2622	-939	-7657	-846
xs9	-5377	-1464	94	-311	549	-54	88	-440	189	-6726	-1349
xs10	1673	-201	436	-103	620	255	152	-720	972	3085	1412
xs11	738	589	-208	102	-450	904	967	-1715	837	1764	1027
xs12	1972	-1510	-201	86	-549	1427	367	-922	68	739	-1233
xs13	-508	68	335	869	-420	675	-48	-438	-183	349	857

* Total Gain / Loss – Deposition or Erosion from CH2M-Hill Survey to End of Current Study
** Study Gain / Loss – Deposition from January 11 Survey to End of Current Study

series of releases between large storm events should not exceed this volume unless it can be shown that the banks have become unstable or additional sources of sediment are available for transport.

Results indicate that net erosion through cross-section 4A was 10,506 cubic feet. This represents approximately 13% of the estimated available sediment in the study reach. This value is lower than the reach length since the channel suddenly widens at cross section 8 and is wider for the reach below cross section 8. It is also reduced by the amount of material stored in the weir pool.

It further appears from the data that sediment is being stored in the lower reach of the study area. From Table 4 it can be seen that the lower cross sections (9-13) are fairly consistently storing sediment. This is most likely due to the wider cross sectional areas and lower resulting velocities as compared with the upper reach above cross section 8. Cross section 8 is currently unstable and the reach between cross section 7 and 8 is undergoing large changes in channel alignment and width.

None of this sediment transported down the wash was transported to the pool area of the Cave Creek Dam. Sediment transported down the unnamed tributary was deposited in the first few hundred feet of Cave Creek. The sediment was delivered to Cave Creek, however, and will be delivered to the upper reaches of the reservoir by subsequent large events on Cave Creek. Velocities from the releases were very small in the much larger Cave Creek channel and were not capable of transporting the sediment delivered to the creek from the unnamed tributary.

5.3 Prediction of Sediment Transport Rates

Rather than use the rather complex Meyer-Peter and Mueller equation a set of simple equations based on the flow released into the wash were developed. These equations are as follow:

$$Qs = 5.219Q^{1.25} \quad (1)$$

$$Q_T = 5.219Q^{1.25}T \quad (2)$$

$$Q_V = 5.219Q^{1.25}T \cdot Cv = 3.34Q^{1.25}T \quad (3)$$

Where:

Qs = Sediment Transport in Tons / Day
Q = Discharge Rate in Million Gallons per Day (MGD)
Q_T = Transport in Tons
Q_V = Transport Volume in Cubic Yards
T = Flow Time in Days
Cv = Conversion from Tons to Cubic Yards (2000/(2.65*62.4*.7)/27 = 0.64)

Equation 1 predicts transport in tons/day and is an excellent fit for the MPM equation based on the values from the upper reach of the wash as used in HEC-6. The fit can be seen in Figure 15 where the predicted values overlay the MPM values almost perfectly. The values can be seen in Table 5 and the error between the prediction equation and the

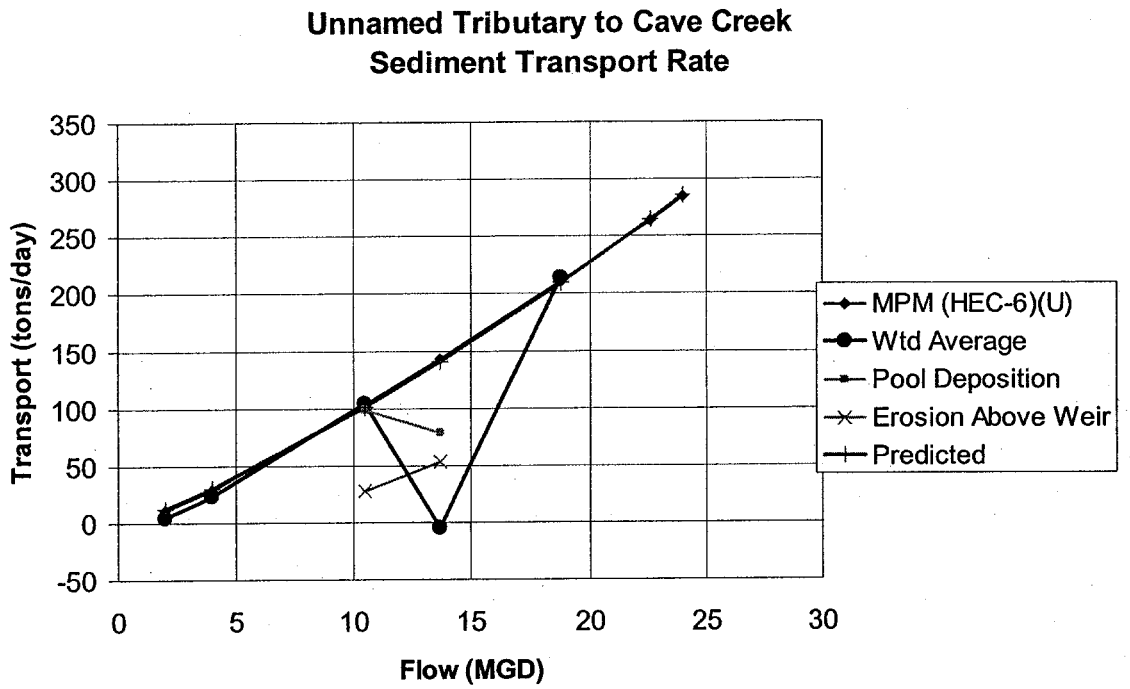


Figure 15. Observed Sediment Transport Rates Compared with MPM Equation Predictions and Proposed Prediction Equation for Cave Creek Water Reclamation Plant.

Table 5. Values Predicted by Meyer-Peter and Mueller as applied in HEC-6 compared with those predicted by the equation developed in this study.

	MPM (HEC-6)	Predictor	
Q (MGD)	Qs (Tons/Day)	Qs Predicted	Error Squared
2	10.5	12.5	3.98
4	28.9	29.9	1.04
10.5	103.1	100.9	4.85
13.7	143.2	141.0	4.58
18.75	210.9	209.4	2.13
22.6	263.7	265.0	1.65
24	284.9	285.8	0.86
		RMS Error	0.236

values predicted by the MPM equation is very low. The values predicted by the above equation (and MPM) appear to be slightly high when to compared with the observed transport rates for low flows but values for higher flows are almost exactly those

predicted by the Meyer-Peter and Mueller Equation (MPM). The differences are well within the error of observation. Equations 2 and 3 predict sediment transport in tons and cubic yards and are simply extensions of Equation 1.

The equation presented above can be used very simply to estimate sediment transport in the wash based on planned release rates and timing. To use the equation one simply needs to know the flow rate and the length of time the flow will last. For example:

If the city plans to release 4 MGD for 14 hours the estimate of sediment transport would simply use Equation 2 and the result would be:

$$
\begin{aligned}
Qs &= 5.219 \times (4)^{1.25} \times 14 \text{ hrs} / 24 \text{ hrs/day} \\
&= 29.52 \times 14/24 \\
&= 17.2 \text{ tons of sediment (Equation 2) or} \\
&= 11 \text{ Cubic Yards of Sediment (Equation 3)}
\end{aligned}
$$

This value would need to be adjusted if the release time was less than the time it takes the water to reach Cave Creek Wash since no sediment delivery would occur. The calculation of time to reach Cave Creek is discussed below.

6 Flow Lengths, Rates of Advance and Timing

A byproduct of the study were data that could be analyzed to evaluate the rate of advance in the wash for the various flow rates and the ultimate distance the flow traveled down the wash as a result of a release. All of the flows reached the confluence with Cave Creek (7,020 ft) and traveled down Cave Creek Wash. Flow lengths ranged from 8,680 ft to 13,560 ft from the outfall with the higher flow rates traveling the farthest. All flows were designed such that a total volume of approximately 4 million gallons were released during the test as discussed previously. The various advance rates and distances traveled are shown in Table 6. The advance rate for the 2 MGD test was complicated by an initial flush of the pipeline of approximately 14 MGD for a short time and continued operational releases to calibrate flow meters at the outfall for approximately 3 hours prior to the start of the scheduled release.

The distance traveled was measured from digital images of the wash as well as stepped off on the ground as a rough check of the digital accuracy. The rates of advance were calculated based on observations of the advancing wave front and distances between surveyed cross sections along the channel. The surveyed sections did not reach the Cave Creek confluence and an abandoned car body in the center of the wash between cross section 13 and Cave Creek served as a convenient location at which to obtain an additional point to calculate the flow advance. The confluence and a tree on the edge of the Cave Creek Wash were other likely points at which to mark the advance. Results of the observations and calculations are shown in Table 6.

Table 6. Advance Rates and Total Distances Traveled for Cave Creek WTP Releases.

			2 MGD	4 MGD	10.5 MGD	13.7 MGD	Comments
Cross Section	Distance (ft)	Total Distance (ft)	Rate of Advance (ft/sec)	(ft/sec)	(ft/sec)	(ft/sec)	
1	20	20		5.0	6.7	9.7	
2	125	145		62.5		147.1	
2a	230	375		46.0	46.0	82.1	
Overtop Weir	102	477	31.8	18.5	40.8	33.8	1
2b	46	523				69.0	
2c	46	569		73.6		78.9	
3	85	654		68.0	193.1	102.0	
3a	256	910		85.3	133.6	120.0	
4	151	1061		30.2	64.7	73.7	
4a	293	1354		58.6		128.3	
5	227	1581		45.4	97.5	104.0	
6	486	2067		44.2	88.4	85.3	
7	496	2563		38.2	90.2	81.8	
8	493	3056	17.3	24.7	46.2	70.4	
9	505	3561	6.9	15.3	37.0	42.0	
10	495	4056	9.0	13.0	25.9	38.3	
11	502	4558	9.1		51.1	52.8	
12	509	5067			41.6	59.9	
13	585	5652			34.2	47.6	
Car Body	636	6288			33.5	54.8	2
Confluence Tree	659	6947			13.9	30.2	3
Confluence	76	7023			13.6	13.7	4
1st 2 mgd	1437	8460	3.2				
End 2 mgd	220	8680	0.1				
1st 4 mgd	853	9533		7.3			
End 4 mgd	312	9845		0.8			
End 10.5 mgd	1532	11377			10.0		
End 14 mgd	2181	13558				13.6	5
Cave Creek Dam	2460	16018					
Total Length of Flow			8680.0	9845.0	11377.0	13558.0	

1. 2 MGD flow was Started an Approximately 14 MGD Initial Flushing Flow then reduced to 2 MGD flows later. – No Data Obtained Prior to Cross Section 8 Except Sonic Meter on 15 Minute Measurement Cycle
2. Car Body in Wash Channel
3. Tree at Northeast Corner of Cave Creek Wash – Unnamed Wash Confluence
4. At Edge of Main Cave Creek Channel
5. Estimated Time to End of 13.7 MGD is 7 hours (advance ranges between 10.5 to 14 ft/min)

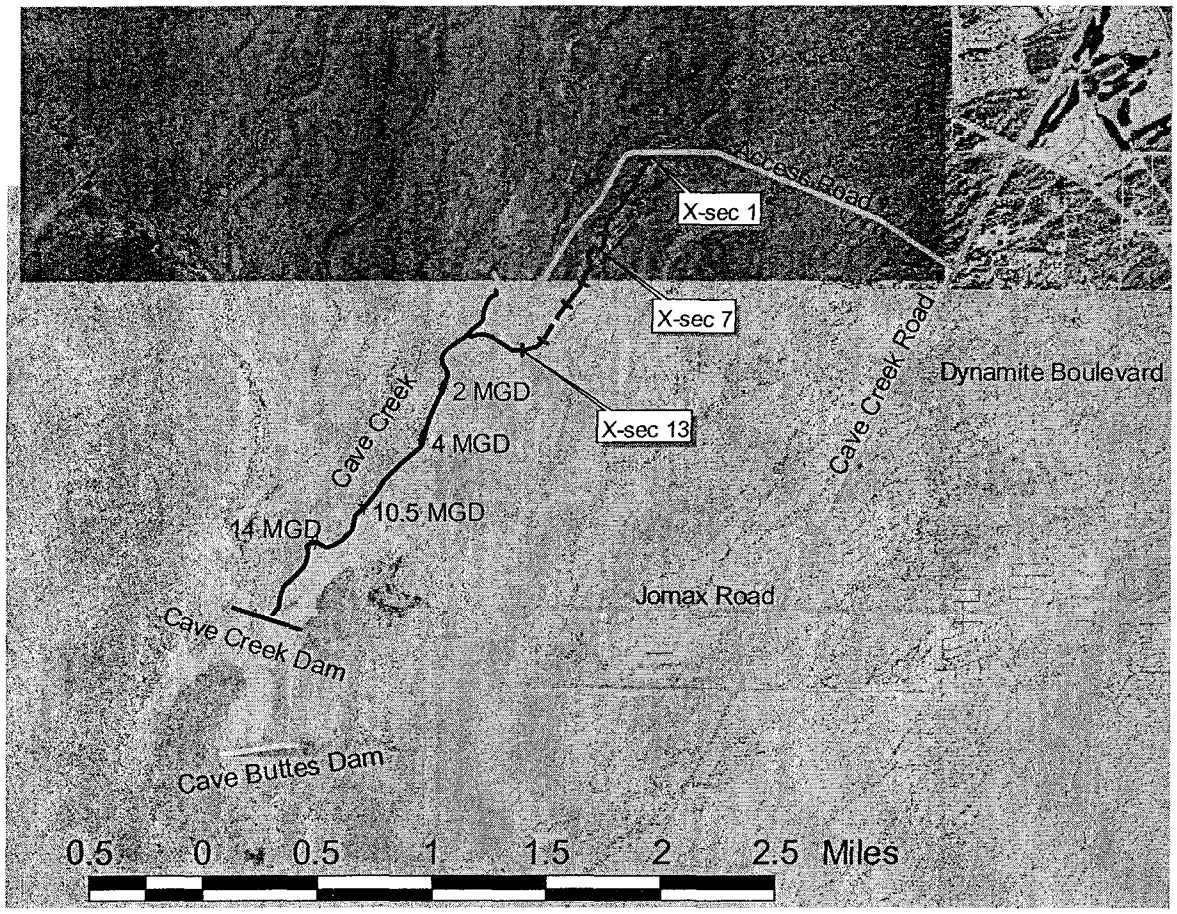


Figure 16. Distance Water Traveled Down Wash for Various Flow Rates.

The total flow distance increased with the increasing rate of flow for the set release volume as would be expected (See Figure 16.. The lowest flow of 2 MGD, while reaching the Cave Creek Wash, stopped in a pool area approximately 1400 ft downstream from the confluence. In approximately 24 hours the pool had filled to very nearly its maximum level. With additional time the flow may have filled the pool and advanced further down Cave Creek but flow was advancing very slowly before reaching the pool and would not be expected to advance much farther down the wash. The bed material in Cave Creek Wash is primarily cobbles and boulders with nearly the only sands and gravels being those contributed by the unnamed tributary being studied here.

It was impossible to tell if the flow would have eventually made it farther down the channel or not without a longer release period. It appears, however, that the channel in combination with this pool has the capability to infiltrate a flow of approximately 2 MGD for an extended period of time. At the conclusion of this release the channel and pool had absorbed the 2 MGD flow for nearly 24 hours. The total flow time was 45 hours as shown in Table 6.

The 4 MGD flow rate traveled down the wash approximately 1200 ft beyond the maximum distance for the 2 MGD flow before stopping in a cobble and boulder riffle

area. This flow would have continued to advance down the channel given additional time however the rate of advance for the last five hours was less than 1 ft/min. Total release time was 23 hours.

The 10.5 MGD flow traveled a total distance of 11,390 ft and was well into a broad flat area where the channel banks are ill defined and high flows can spill across the valley. The location of the maximum flow lengths are shown in Figure 16. This flow was continuing to move slowly down the channel and would have reached the dam with sufficient time.

The 13.7 MGD release reached a point 13,558 ft from the outfall and given additional time most certainly would have reached Cave Creek Dam. The flow reached a point where Cave Creek turns west to flow around the west side of a small hill. Cave Creek dam is located approximately 16,000 ft along the channels from the outfall.

It can be noted that some very high advance rates existed for cross-sections 3 and 3a for releases later in the test series as shown in Table 1. These areas were below the measurement weir and were void of bed material in the channel. The bed in these cross-sections consisted of the native sub-bed material that was much less erodible and this eliminated water storage in the bed material. This storage of water accounted for slower rates in other parts of the channel and is suspected in the reduced advance rates noted just before the flow reached the confluence as shown in Table 6. This bed storage, when combined with storage in the riffle pool system of Cave Creek Wash, also accounts for the slow advance rates in the portions of Cave Creek immediately downstream from the confluence with the unnamed tributary.

Data was not obtained for the arrival of flow at the confluence with Cave Creek for the 2 MGD and 4 MGD flows. This was primarily due to the slow advance rates. The 4 MGD release was initiated in the late afternoon and by the time flows had reached cross section 10 it was well after dark and prospects were that the flow would not reach Cave Creek until well after midnight based on the time for flow to travel between cross sections 9 and 10. The travel times for the 2 MGD release were also prolonged as shown in Figure 16. It can be noted that there appears to be an almost linear reduction in the rate of advance as the flow moves down the wash. There is also a very pronounced drop in the advance rate when the flow enters Cave Creek with its relatively large pools that must be filled before the flows can advance across the riffle sections.

The reduction in advance rates was estimated for each of the flow releases with the 2 MGD release assumed to follow the same slope as the 4 MGD release. This assumption was made due to the limited data on the 2 MGD release. Slopes for both the 2 MGD and 4 MGD flows were broken at cross section 10 and lower rates used from there to Cave Creek Wash since a continuation of the same rate would result in negative advance rates. A set of linear equations were developed to predict advance rates along the wash ignoring cross sections 8-10 which are lower for all of the releases due to wide channels with additional bed storage. Based on these equations the time to reach Cave Creek was estimated as shown in Table 8. Also presented in Table 8 are the observed values for

arrival times at Cave Creek. The estimates for the 2 and 4 MGD releases should be field checked during future releases to insure accuracy however times are thought to be conservative since the lower advance rates for cross sections 8-10 were ignored in the development of the equations.

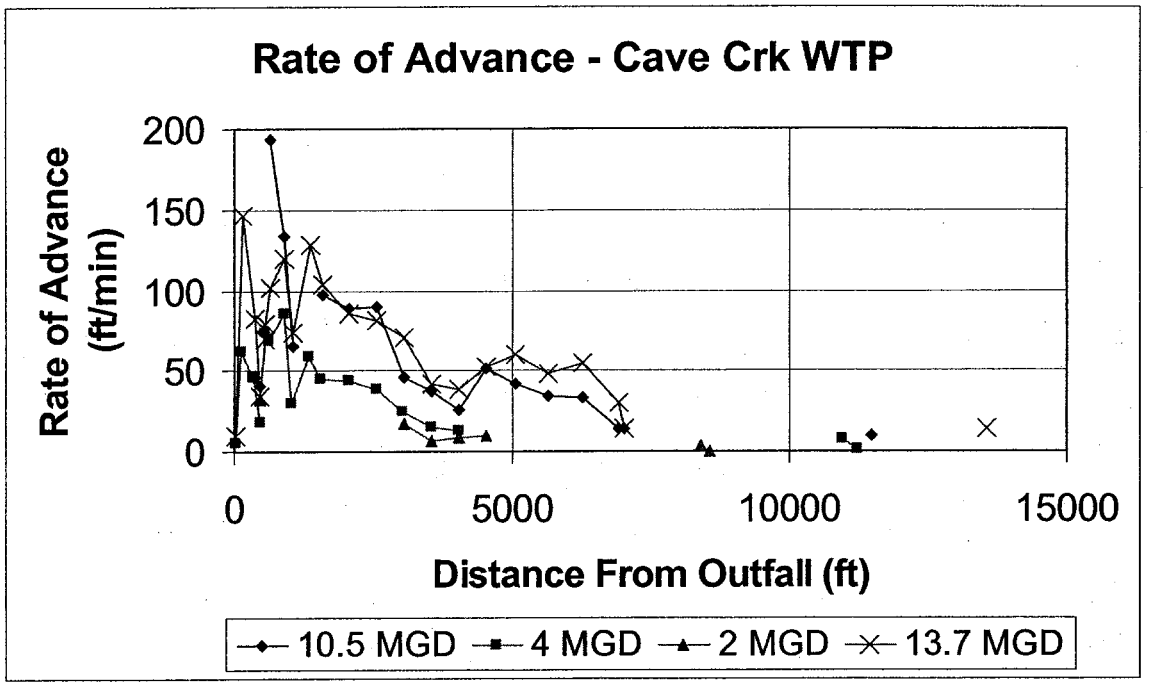


Figure 17. Advance Rates for Release Flows down Unnamed Tributary to Cave Creek.

Table 7. Flow Release Times and Time to reach Cave Creek Wash.

Flow Rate	Total Time	Travel Time (minutes)		Ratio
(MGD)	(hrs)	C-Sect 10	Cave Creek	Xsect10/CaveCrk
2	45	305	-	
4	23	152	-	
10.5	9.1	76	184	.41
13.7	7	63	132	.48

These values are also based on dry wash conditions. If the wash is wet or flowing, the time will obviously be reduced due to reduced storage in the wash. A review of the collected data shown in Table 6 lends credence to the estimated values for the 2 and 4 MGD releases.

Table 8. Observed and Estimated Time for Releases to Reach Cave Creek Wash from Outfall. Time for Release to Reach Outfall from Valve Station is Approximately 30 Minutes.

Flow Rate MGD	Observed Time (Minutes)	Estimated Time (Minutes)
2	-	730
4	-	375
10.5	184	185
13.7	132	119

7 HEC-RAS Model

A new HEC-RAS model was developed using the cross sections developed in this study. The cross sections were merged with data from the Cave Buttes Dam Interpond Area Delineation Study. The model was used to estimate velocities and flow depths in the wash for a series of releases from the WRP. From field observations it was noted that water flowing in the wash did not flow directly down the channel. CH2M-Hill estimated that this meandering pattern increased the sinuosity from 1.1 to 1.25 for flow in the wash. No data was obtained with which to check this estimate. The flow did, however, meander down the channel between the banks for all of the observed flow rates. This phenomenon, while common, will create differences between the HEC-RAS model and actual flows in the field. This may also have some limited impacts on the calculated sediment transport in the channel and can lead to channel instability during medium flow events. Cross section data was from the June 8, 2001 survey with the exception of data for cross section 5 which was taken from the April 2nd survey. Cross section 5 was missed during the June 8th survey.

Manning's n values were estimated to be 0.37 in the channel and 0.045 on the banks. The channel values were estimated by use of Brownlie's Equation and the overbank values were estimated based on field observations. Both were comparable with values used in the CH2M-Hill HEC-RAS model.

The model was initially run with only the cross sections surveyed but analysis of the results indicated that the model was unstable with oscillations between cross sections. Additional cross sections were then interpolated such that no reach length was greater than 200 ft and the model performed much better. This resulted in 2-3 cross sections being added between each surveyed cross section. Survey data exists at midpoints for several of the sections but has not been added to the model due to the uniform nature of the wash.

A series of cross sections were added to the bottom of the model to insure the model was properly calculating solutions by the time it reached cross section 13. These cross sections are labeled as less than 1.0 and were cut from the digital data obtained from the Interpool Study. It should also be noted that the cross sections in the HEC-RAS model

are numbered from the bottom of the reach rather than from the top. The corresponding study cross sections numbers are shown in Table 9.

Average channel velocities are presented in Table 9 for all flows released as well as results for the 18.75 MGD (averaged value) and 24.0 MGD flow rates. The table shows that the velocity reduces at the lower end of the study section. This reduction could be enough to cause deposition at the lower end of the wash as noted in the 13.7 MGD release. The drop in velocities do not correspond exactly to the erosion/deposition data contained it Table 4 however. A part of the difference is likely due to the meandering in the low flow channel and associated changes in velocities.

The HEC-RAS model is provided on diskette and includes the model with the interpolated cross sections as well as the model based only on the surveyed cross sections.

Table 9. Velocities Predicted by HEC-RAS Model. Interpolated Cross Sections not Shown.

		Flow Rate (MGD)					
Study Cross Section	HEC-RAS River Station	2.0	4.0	10.5	13.7	18.8	24.0
1	13	1.01	1.24	1.72	1.9	2.08	2.29
2	12	1.47	1.78	2.26	2.29	2.52	2.77
3	11	1.42	1.81	2.42	2.5	2.56	2.74
4	10	1.3	1.66	2.34	2.59	2.89	3.15
5	9	1.09	1.42	1.94	2.11	2.35	2.56
6	8	1.19	1.45	2.01	2.19	2.4	2.57
7	7	1.08	1.43	2.13	2.39	2.74	3.07
8	6	0.9	1.04	1.43	1.55	1.73	1.87
9	5	0.92	1.19	1.72	1.91	2.16	2.41
10	4	0.94	1.22	1.78	1.98	2.24	2.47
11	3	1.05	1.35	1.9	2.11	2.37	2.59
12	2	0.93	1.19	1.72	1.9	2.14	2.34
13	1	0.93	1.13	1.6	1.75	1.95	2.14

8 Incipient Motion Analysis

An incipient motion analysis was performed to estimate the maximum velocity in the channel that would not move sediment down the wash. This analysis indicated that sediment in the wash begins to move at approximately 0.9 ft/sec. The results from the HEC-RAS model indicate that this occurs in most cross sections at about 2.0 MGD. The HEC-RAS model was developed using the June 2 cross sections so the model shows the currently existing bed in the area. The bed has narrowed and deepened at cross sections 1-2 and 3-5 and they exhibit higher velocities than neighboring cross sections that have not been as impacted by the weir. At several cross sections the 2.0 MGD flow is enough to transport sediment which would be deposited at other locations.

This is very similar to the behavior observed from the natural events although not on a cross section to cross section basis.

9 Findings and Conclusions

Sediment Transport in the Unnamed Wash for flows that can be released by the Cave Creek Water Treatment Plant occurs as bed load transport. Unless flows attack the banks little or no fine materials are present in the wash to be transported as suspended load. Sediment transported to Cave Creek Wash is deposited at or within a short distance of the confluence and is not transported to the pool area. However high flows in Cave Creek will transport this material into the Cave Creek / Cave Buttes storage area.

Sediment transport was best predicted by using the Meyer-Peter and Mueller transport equations. A set of simple equations were developed to match the Meyer-Peter Mueller equation that require less data, no complex calculations and can be easily used by City personnel to predict sediment transport. The equations require only the flow rate and the time over which the flow is to occur. This data can then be used to calculate either the volume or dry weight of the material transported. While the treatment plant releases will not transport this material into the pool area, high flows on Cave Creek will eventually carry this material into the pool.

Transport on the Unnamed Wash is also limited by the availability of sediment in storage on the bed of the wash. The amount of stored sediment is estimated to be approximately 3,000 cubic yards. Once this sediment volume is transported no additional transport will occur until the sediment storage is replenished either from upstream or by replenishing the storage by mechanical means unless stream instability results. It is likely that the depletion of stored sediment could result in increased instability in the wash.

10 References

CH2M-Hill 1999. "A Study on the Probable Effects of Reclaimed Water Discharge (Preliminary Projections) into a Tributary of Cave Creek Wash", Technical Memorandum, August 23, 1999.

Dames & Moore 1999. "Supplemental Cultural Resources Survey for Cave Creek Discharge Pipeline Project, Maricopa Co. AZ", Prepared for CH2M-Hill, July 1999.

APPENDIX 1

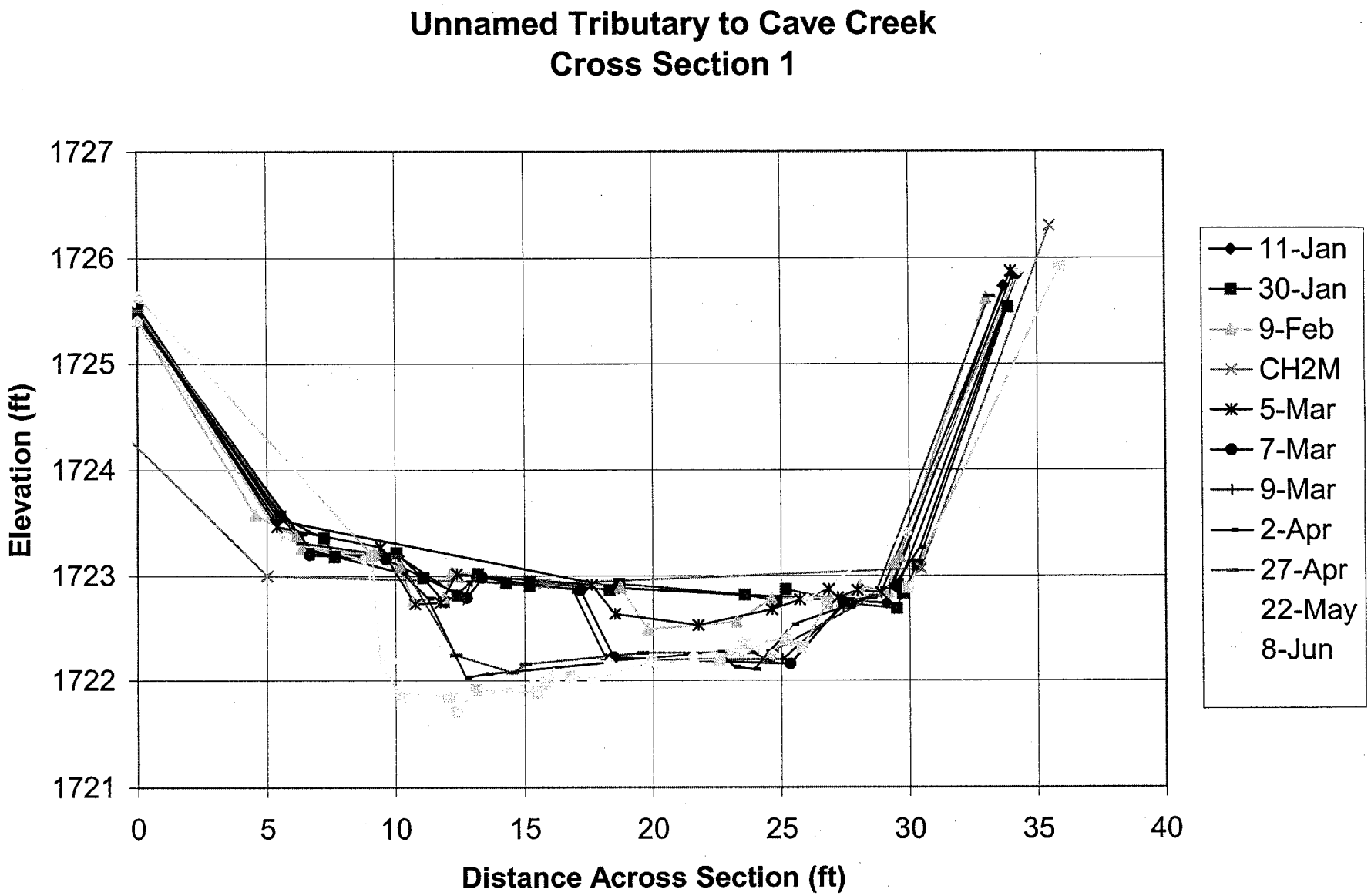
BED GRADATIONS

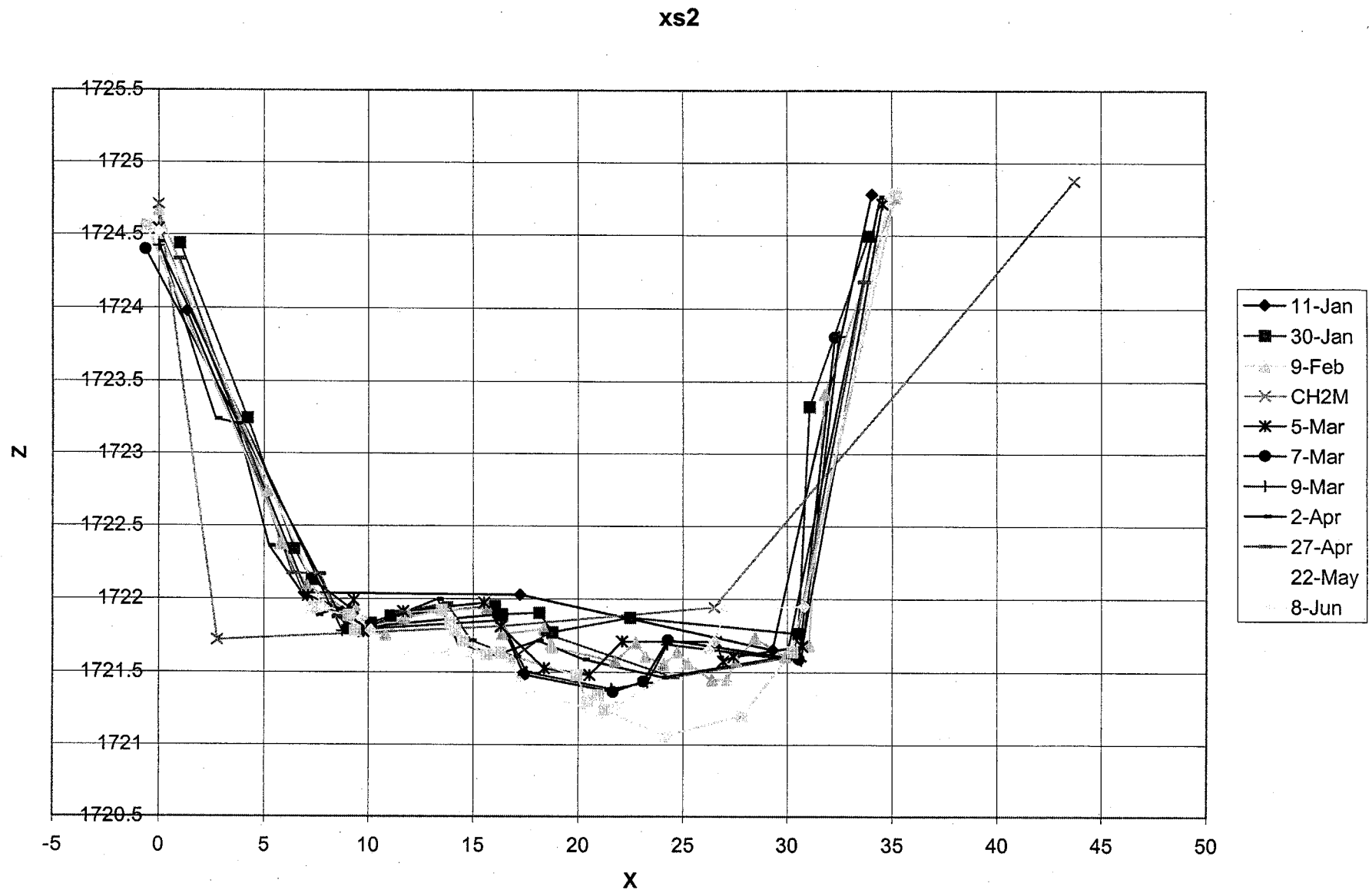
Bed Gradations for Unnamed Tributary to Cave Creek Wash March 2001 (% Passing)

		Sample Location									
Bed		Active Bed								Sub-bed	
Location		Above Road Sample 1	Above Road Sample 2	Cross Section 2A	Cross Section 6	Cross Section 10	Cross Section 13	Cave Ck Confluence	Active Bed Average	60' Above Weir	Sub-bed w/o Fines
Sieve Size	Sample #	6952	6957	6954	6956	6958	6955	6953		6959	
mm	Sieve										
25.4	1"	100	100	100	100	100	100	100	100.0	100	100.0
19.0	3/4"	100	100	100	100	100	100	100	100.0	100	100.0
12.7	1/2"	99	100	98.6	100	100	98.9	100	99.5	100	100.0
9.5	3/8"	98	98	94.2	99.6	98.7	97.7	100	98.1	99.1	98.9
6.3	1/4"	95	91	88.6	96.9	93.5	92.3	96.2	93.4	96.1	95.2
4.8	#4	90	82	83.1	91.5	86.9	86.7	90.2	87.2	92	90.1
2.4	#8	68	53	59.6	66.6	62.5	61.3	63.3	62.0	76.3	70.7
2	#10	61	46	53.5	59.8	56.1	53.8	54.9	55.1	71.7	65.1
1.2	#16	39	29	35.2	40	37.3	32.8	31.8	35.0	55.7	45.3
0.59	#30	18	13	17.1	22.4	19.3	13.2	14.2	16.7	38.7	24.3
0.42	#40	11	9	10.9	16.2	12.5	7	8.7	10.7	33.1	17.4
0.297	#50	8	7	6.7	11.7	8.6	4.3	5.8	7.4	29.1	12.5
0.149	#100	3	5	2.6	2.9	2.7	1.6	2.1	2.9	22	3.7
0.074	#200	2.1	2.9	1.1	1.2	1.1	0.8	1.1	1.5	19	0.0

APPENDIX II

CROSS SECTION SURVEYS


Unnamed Tributary to Cave Creek
Cross Section 1
Elevation (ft)
1727
1726
1725
1724
1723
1722
1721
0
5
10
15
20
25
30
35
40
Distance Across Section (ft)
11-Jan
30-Jan
9-Feb
CH2M
5-Mar
7-Mar
9-Mar
2-Apr
27-Apr
22-May
8-Jun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1725
1724.5
1724
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1723
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1721
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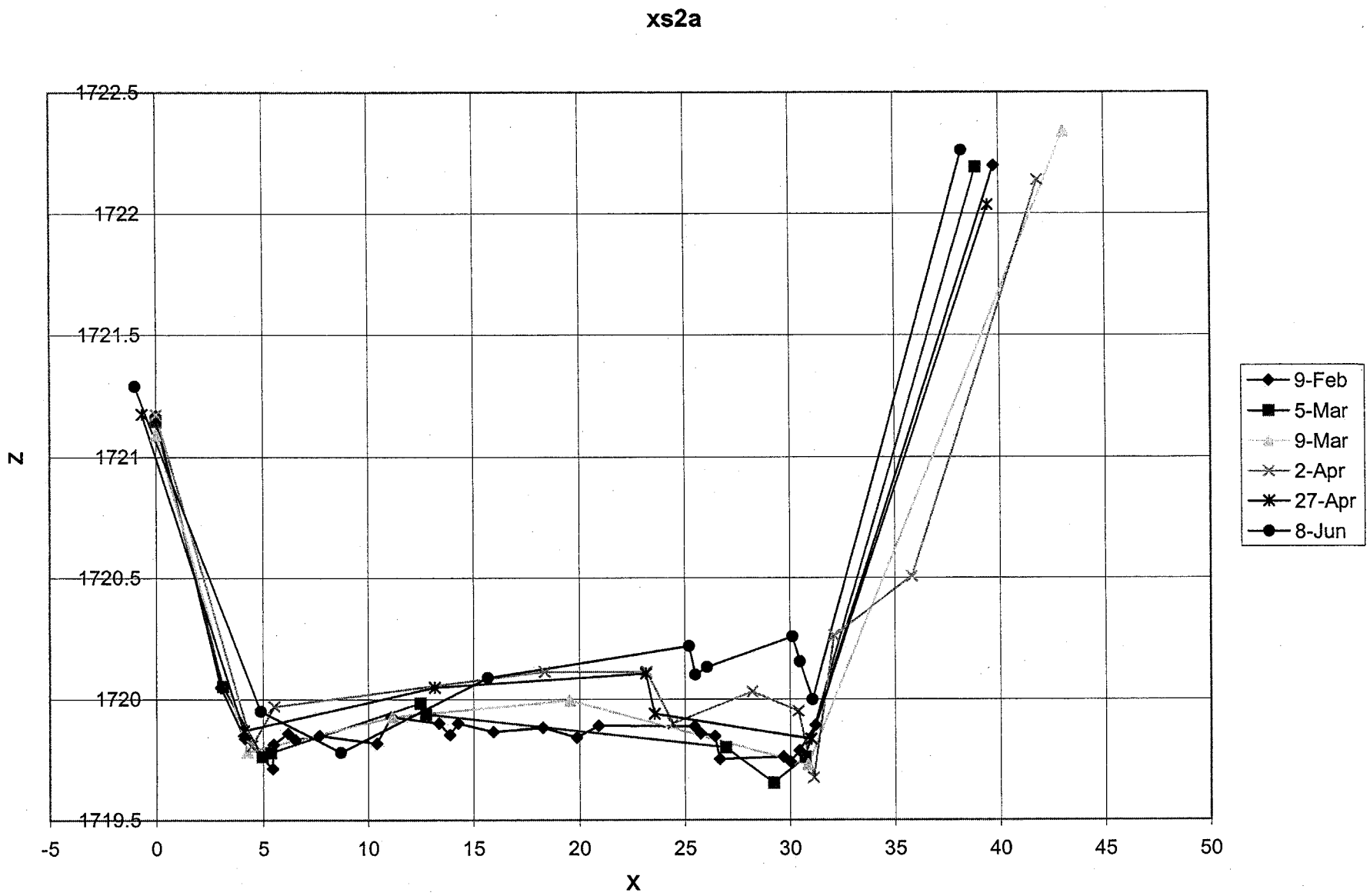
Z
-5
0
5
10
15
20
25
30
35
40
45
50
X
11-Jan
30-Jan
9-Feb
CH2M
5-Mar
7-Mar
9-Mar
2-Apr
27-Apr
22-May
8-Jun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1721
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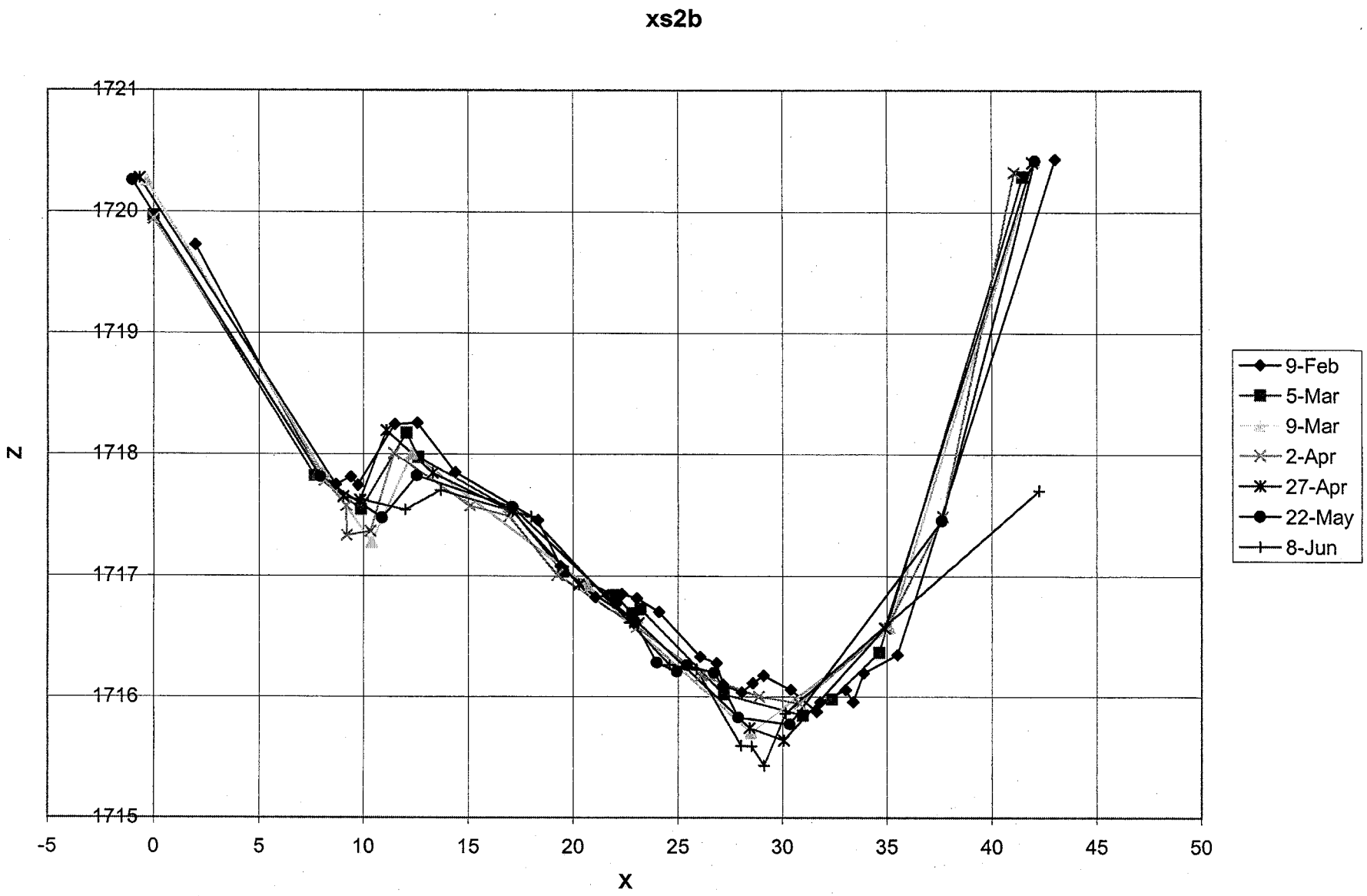
-5
0
5
10
15
20
25
30
35
40
45
50
X
9-Feb
5-Mar
9-Mar
2-Apr
27-Apr
8-Jun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1715
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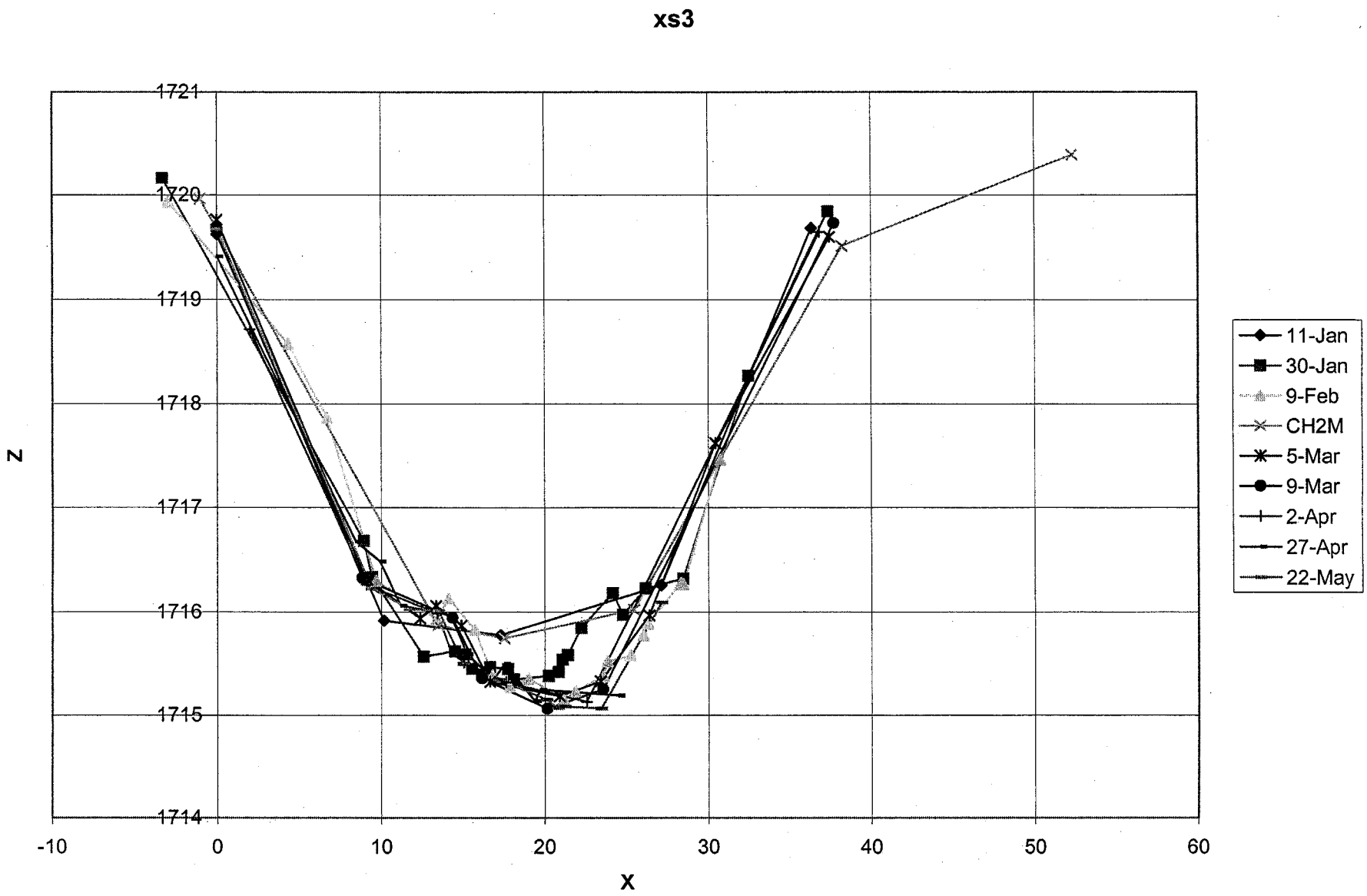
5
10
15
20
25
30
35
40
45
50
X
9-Feb
5-Mar
9-Mar
2-Apr
27-Apr
22-May
8-Jun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1718
1717
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1715
1714
Z
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0
10
20
30
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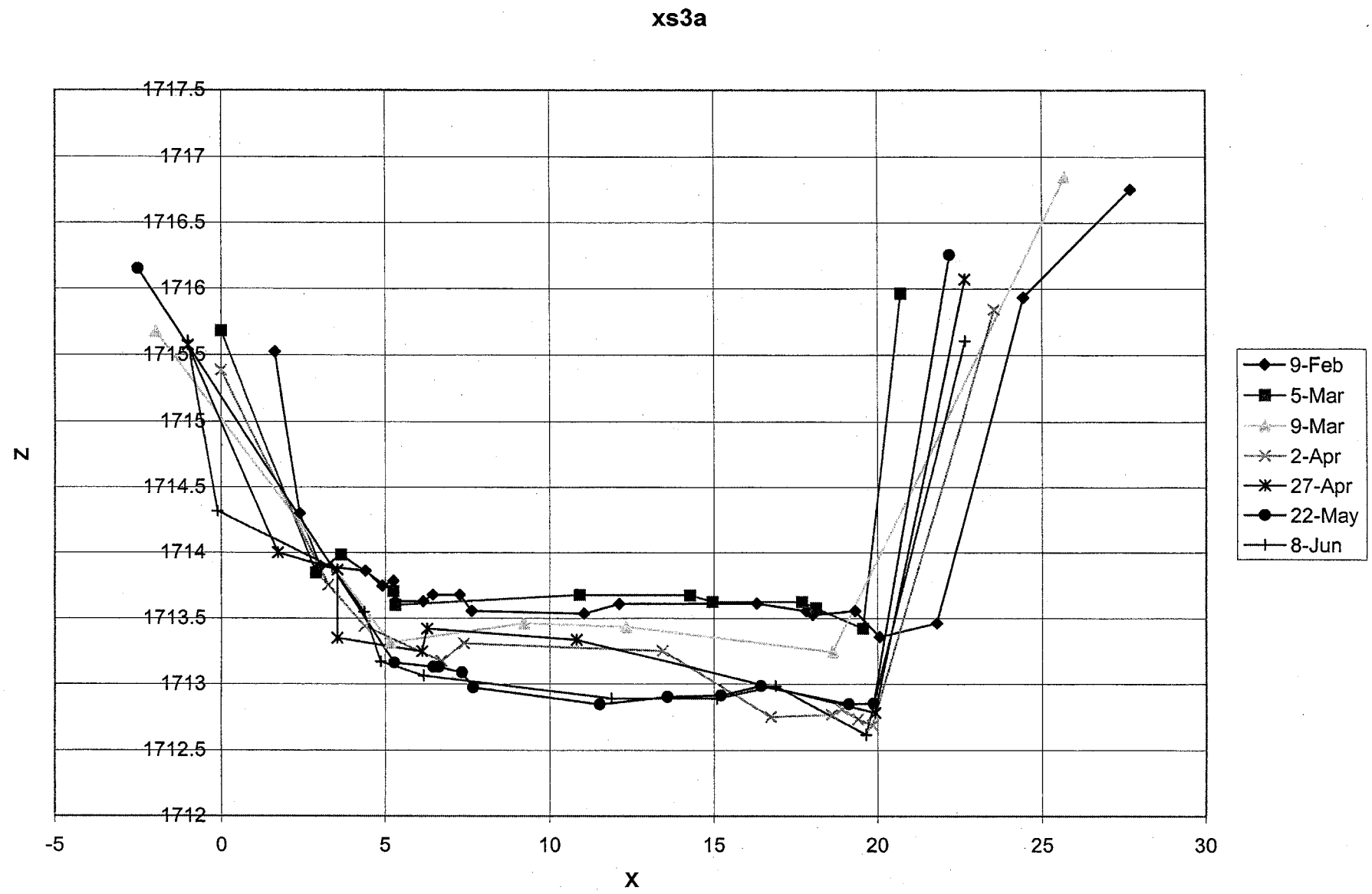
60
X
11-Jan
30-Jan
9-Feb
CH2M
5-Mar
9-Mar
2-Apr
27-Apr
22-May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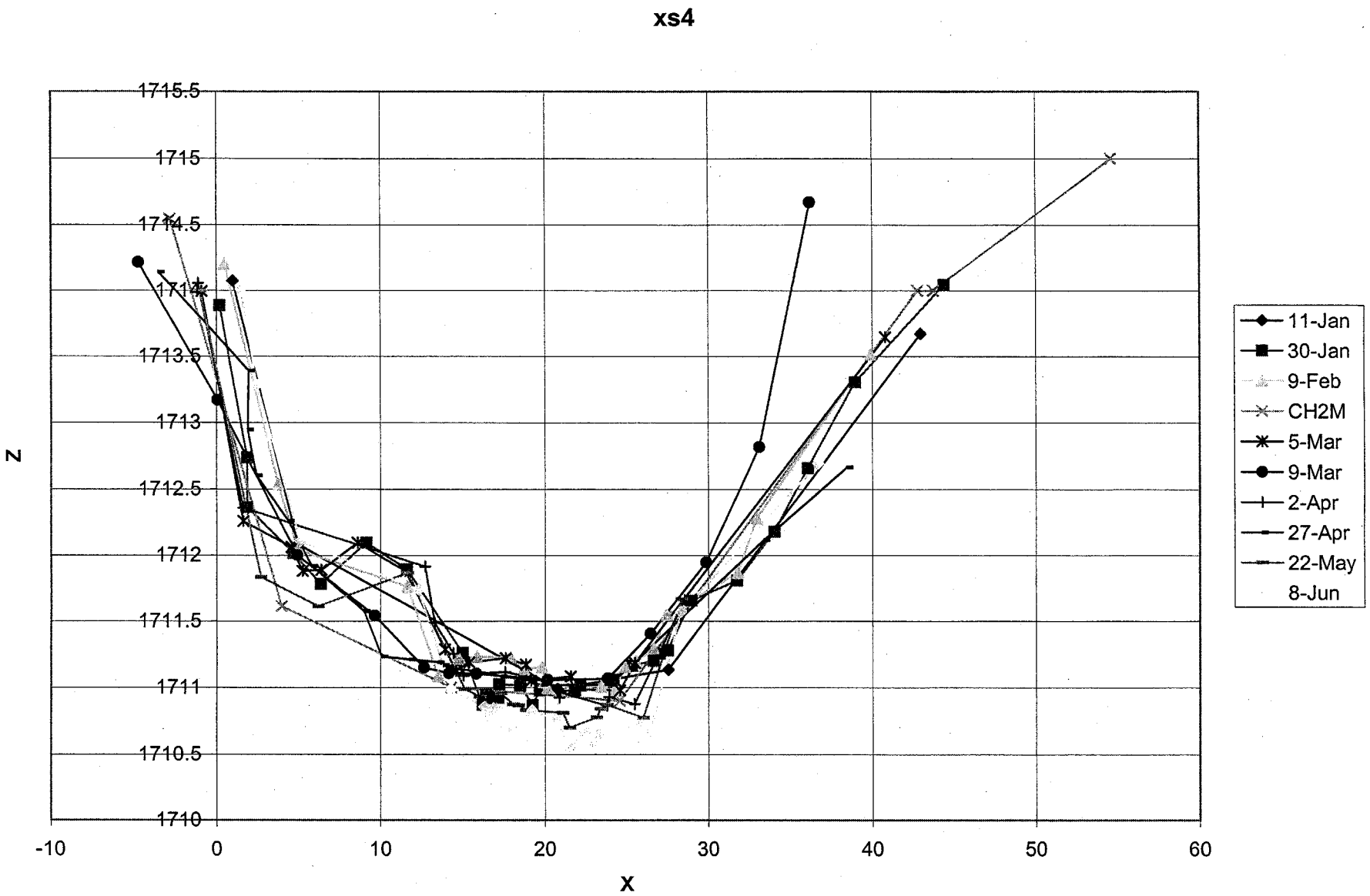
9-Mar
2-Apr
27-Apr
22-May
8-Jun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Z
-10
0
10
20
30
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50
60
X
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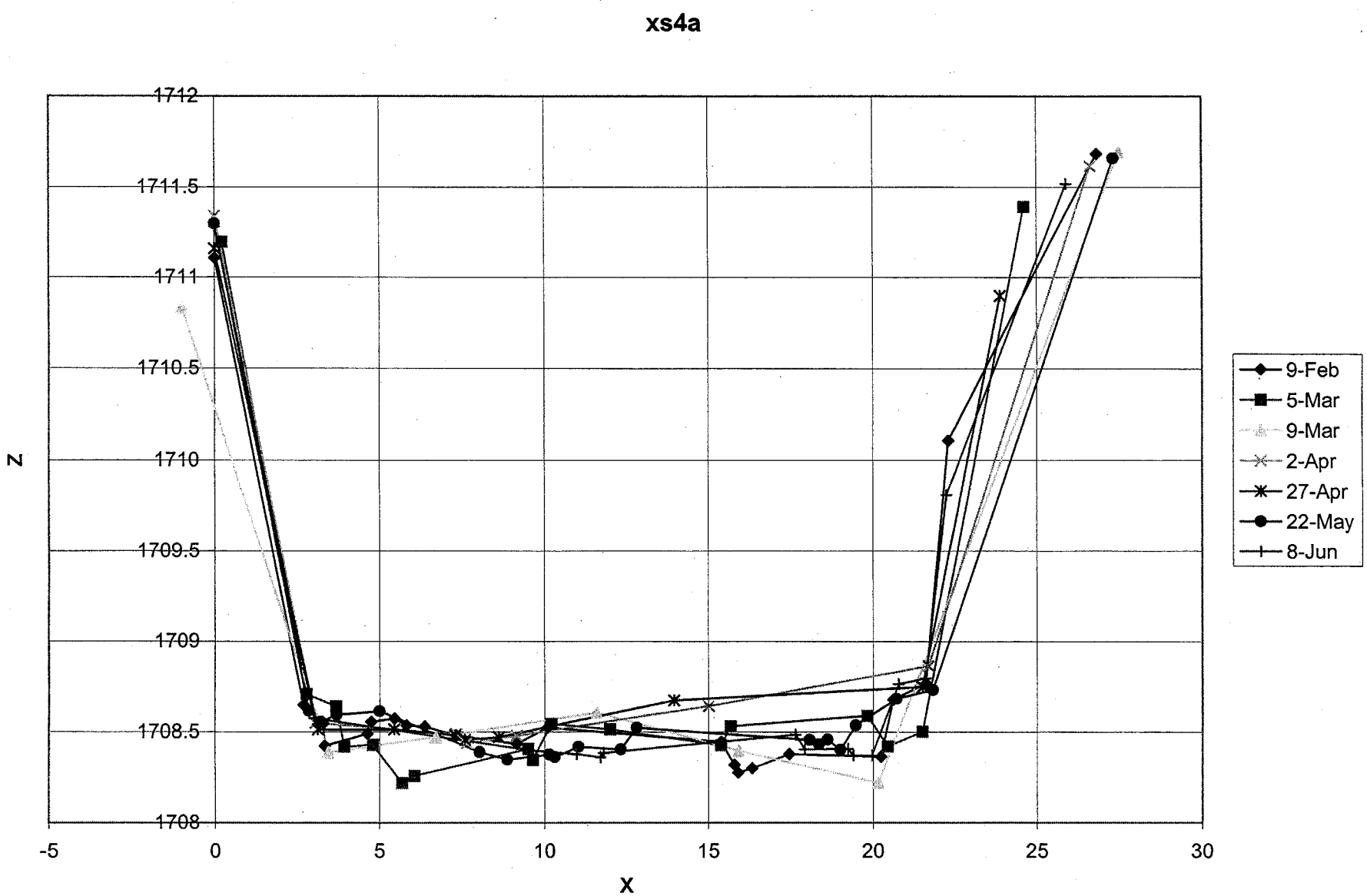
30-Jan
9-Feb
CH2M
5-Mar
9-Mar
2-Apr
27-Apr
22-May
8-Jun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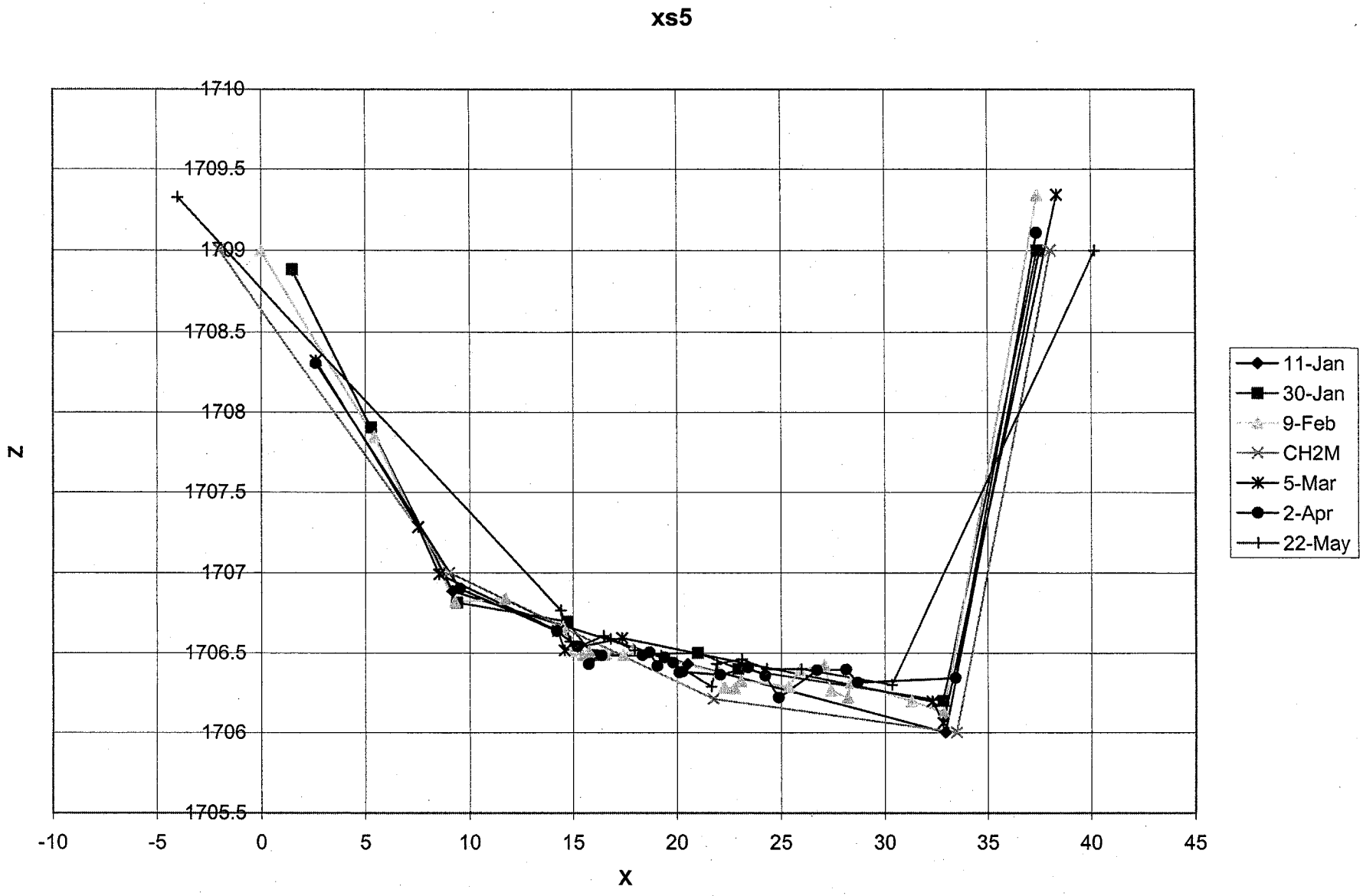
5-Mar
9-Mar
2-Apr
27-Apr
22-May
8-Jun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5
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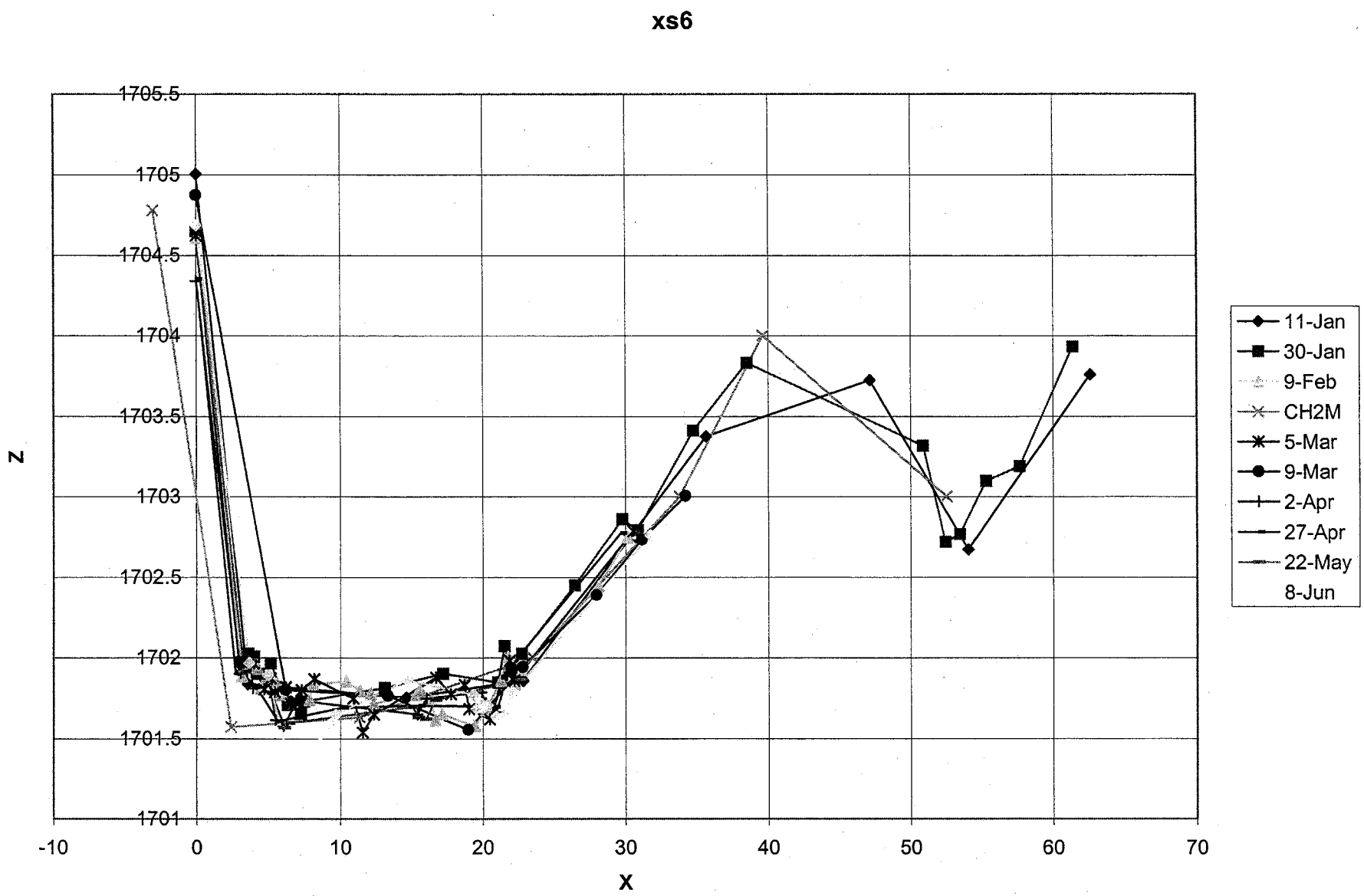
40
45
X
11-Jan
30-Jan
9-Feb
CH2M
5-Mar
2-Apr
22-May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30
40
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5-Mar
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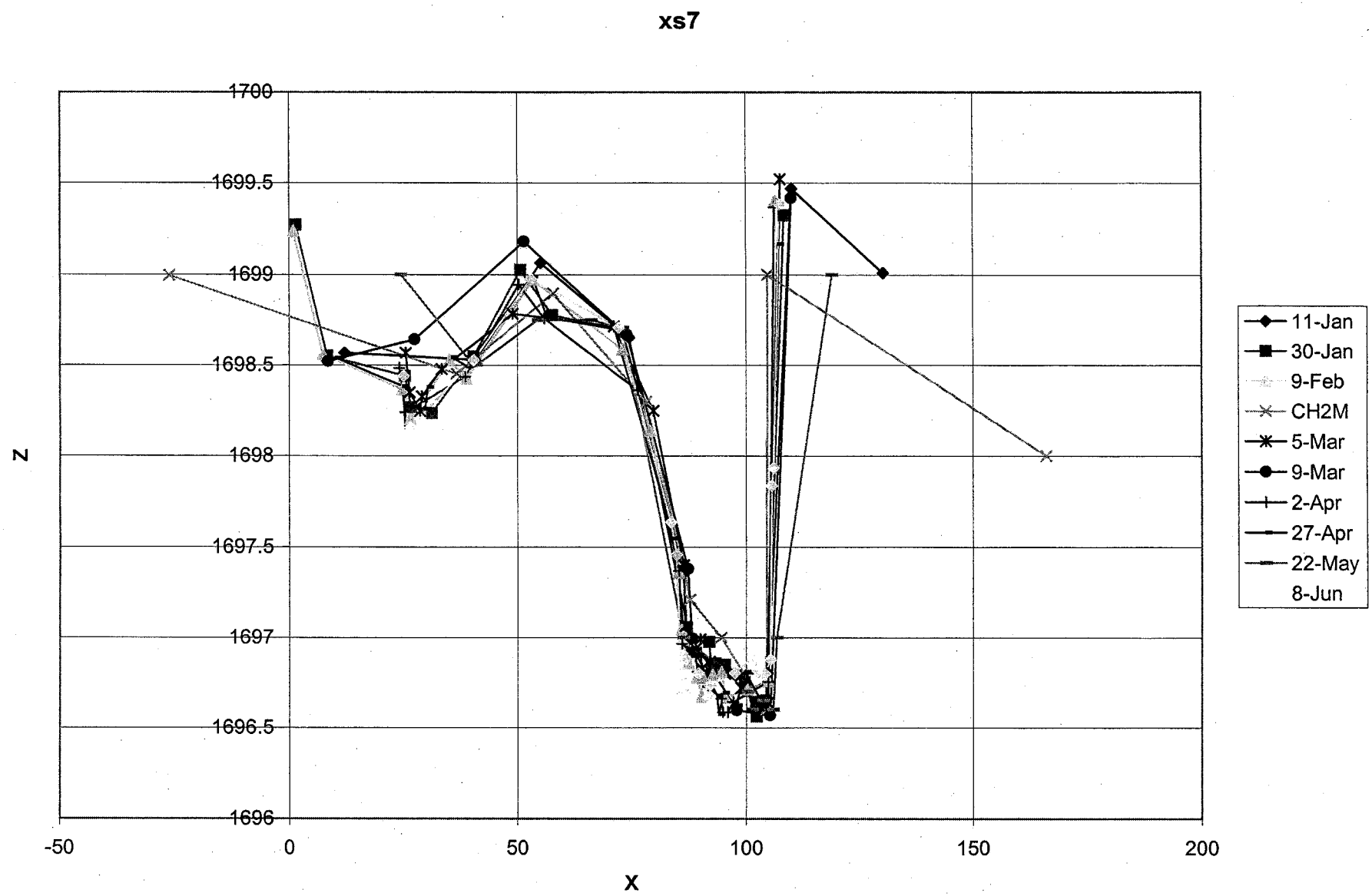
27-Apr
22-May
8-Jun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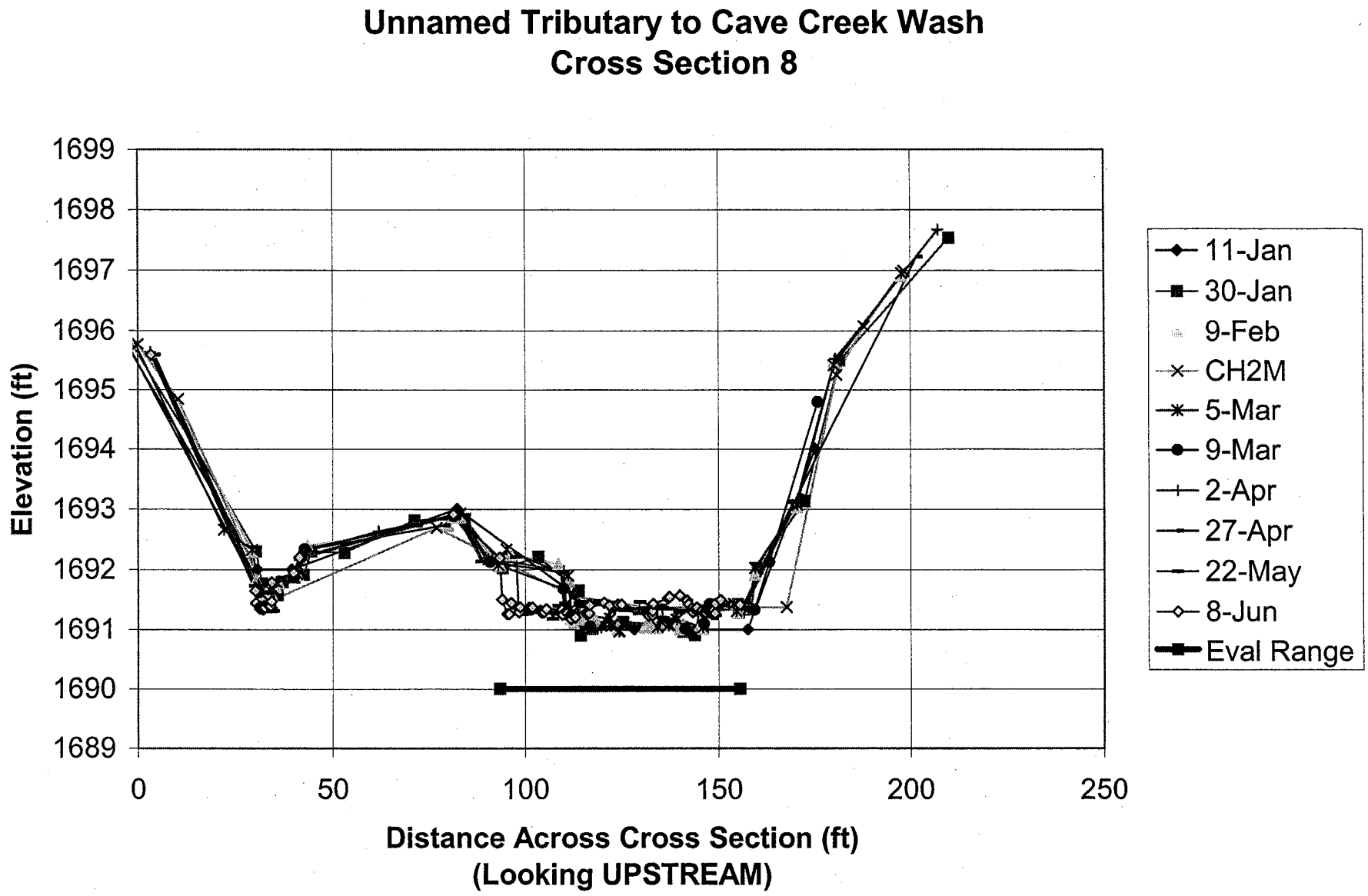
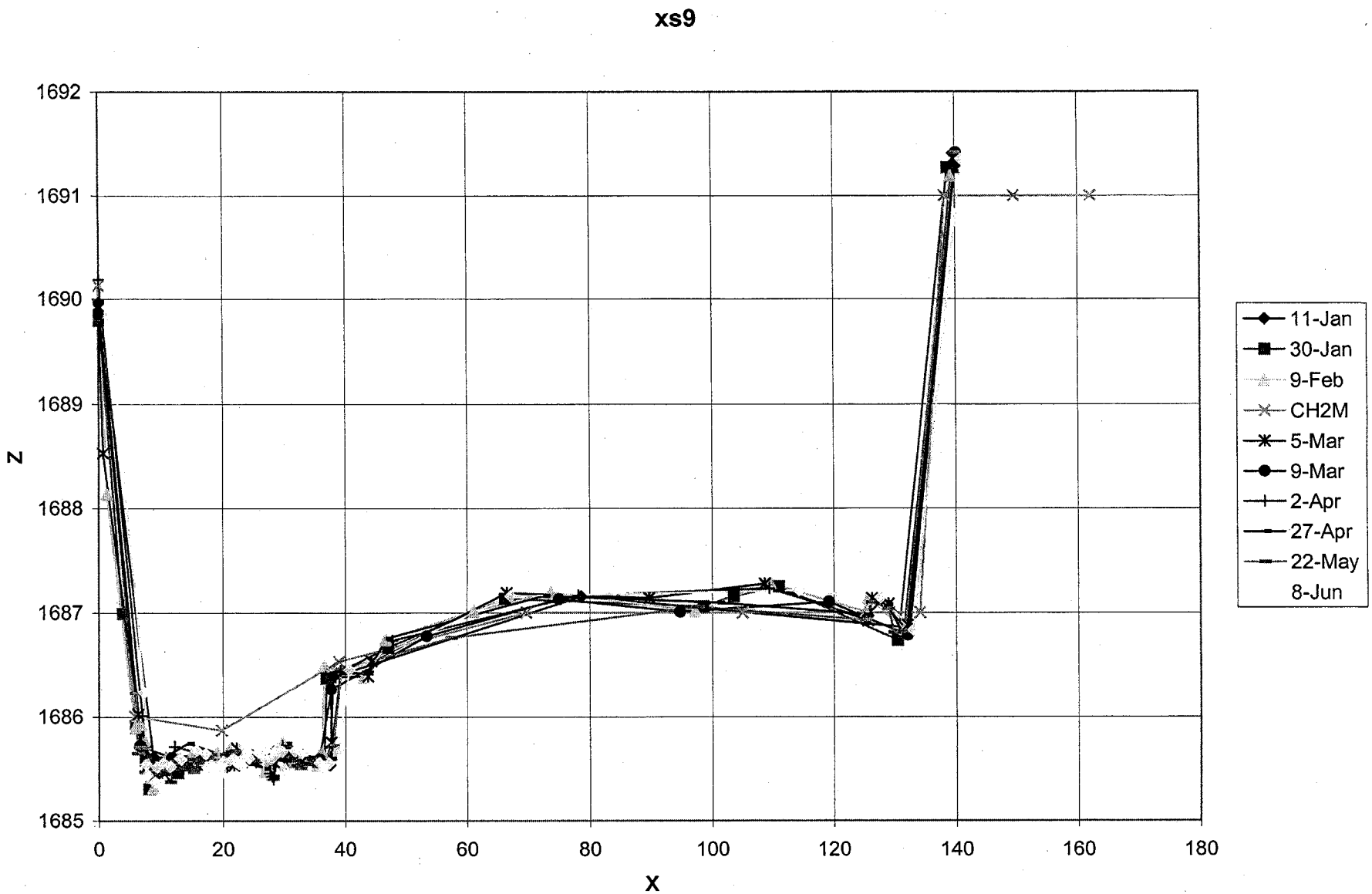
5-Mar
9-Mar
2-Apr
27-Apr
22-May
8-Jun


Unnamed Tributary to Cave Creek Wash
Cross Section 8
Elevation (ft)
1699
1698
1697
1696
1695
1694
1693
1692
1691
1690
1689
0
50
100
150
200
250
Distance Across Cross Section (ft)
(Looking UPSTREAM)
11-Jan
30-Jan
9-Feb
CH2M
5-Mar
9-Mar
2-Apr
27-Apr
22-May
8-Jun
Eval Range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160
180
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30-Jan
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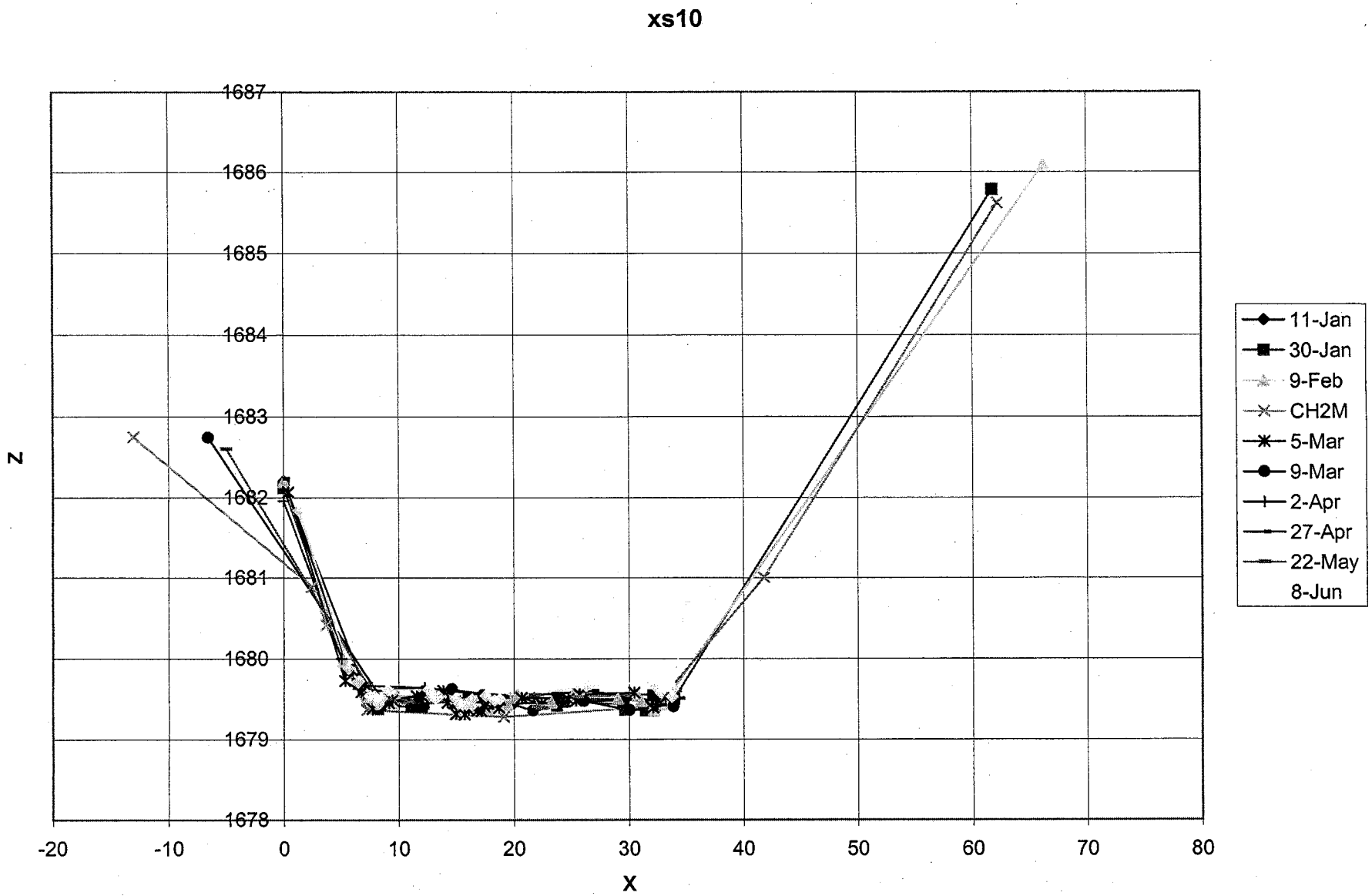
2-Apr
27-Apr
22-May
8-Jun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CH2M
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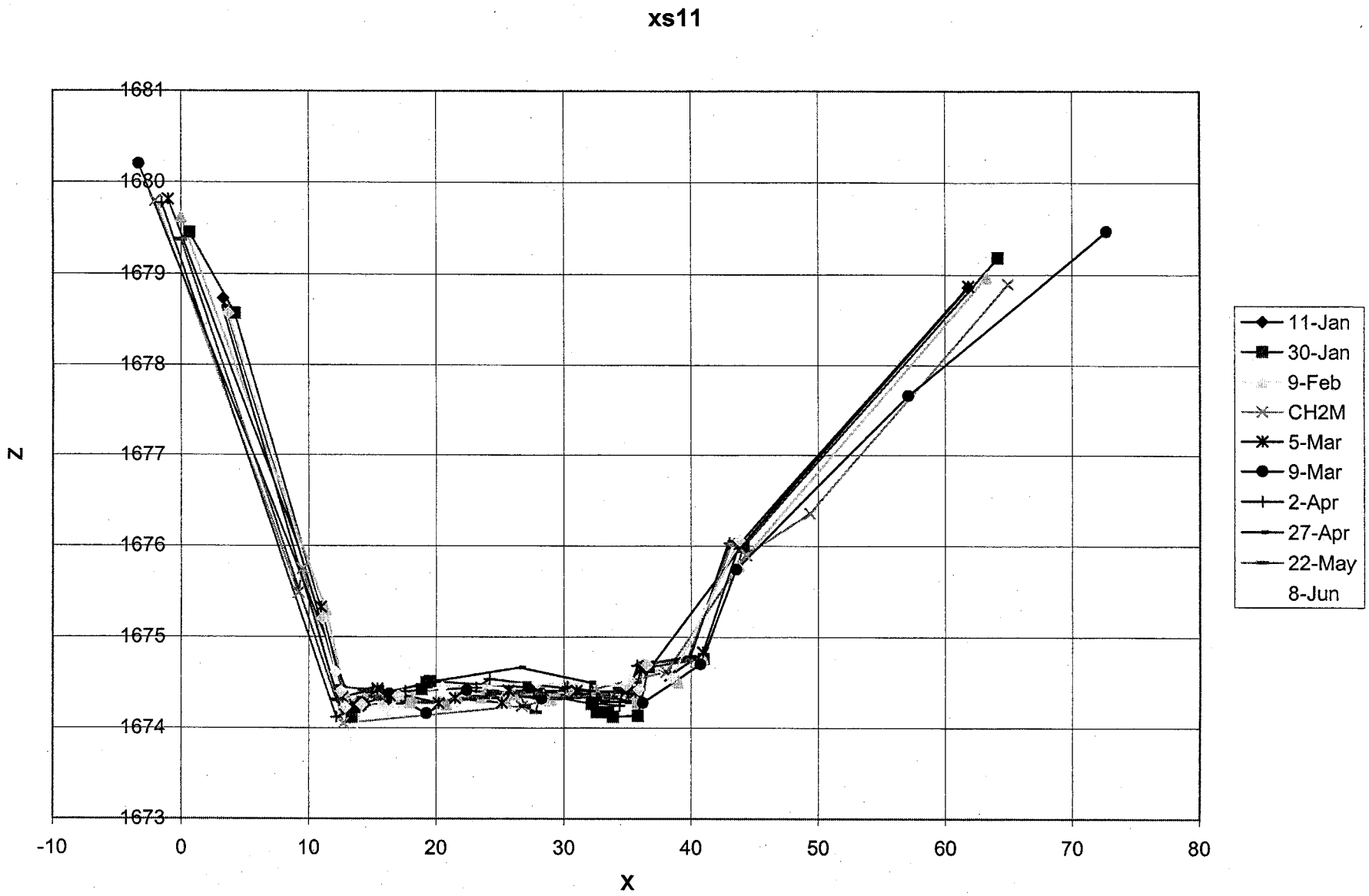
27-Apr
22-May
8-Jun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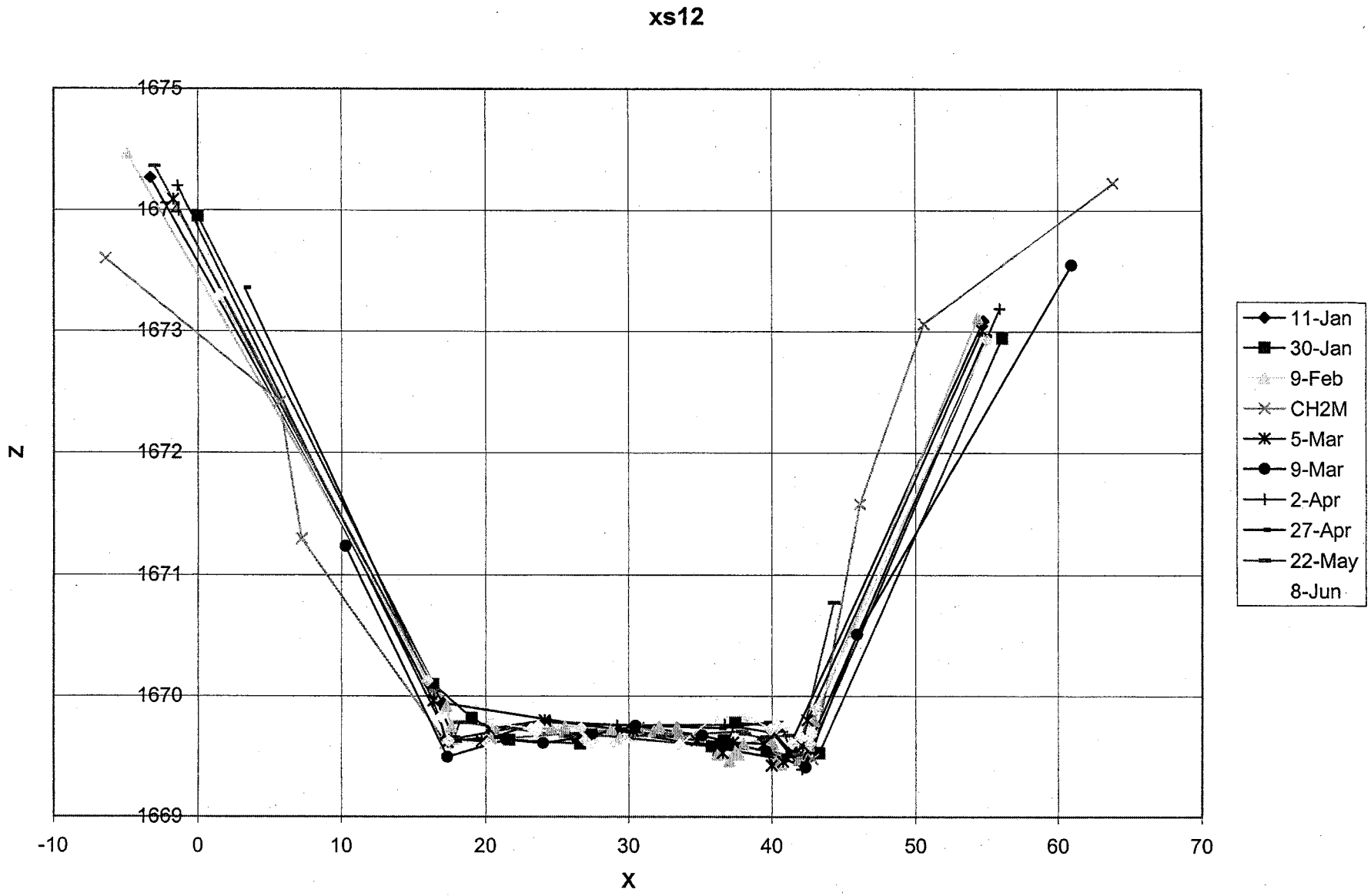
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CH2M
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2-Apr
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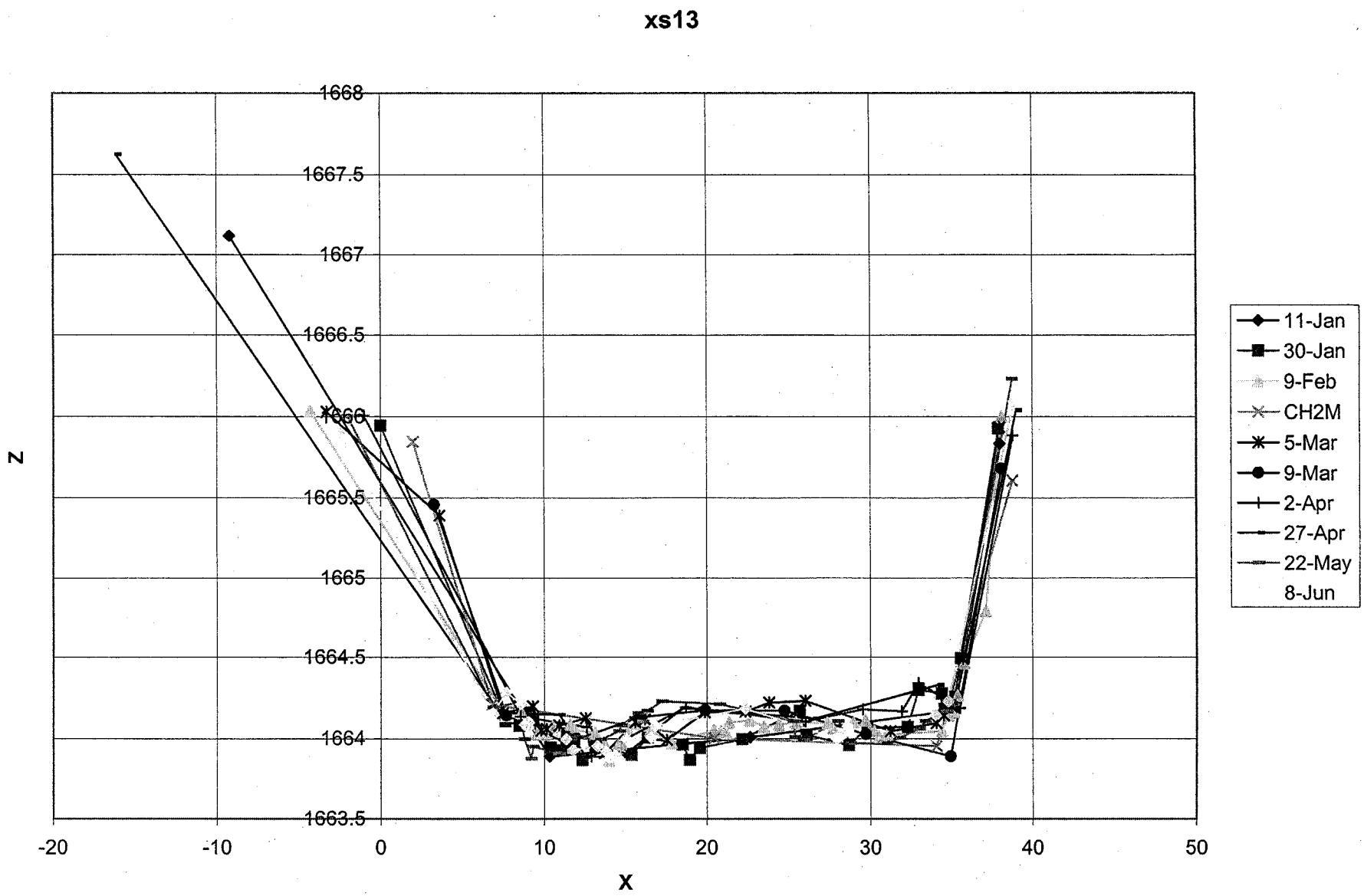
22-May
8-Jun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11-Jan
30-Jan
9-Feb
CH2M
5-Mar
9-Mar
2-Apr
27-Apr
22-May
8-Jun