University of Minnesota
St. Anthony Falls Hydraulic Laboratory

Project Report No. 341

Movable Bed Physical Model of Howard Creek

by

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Conducted for

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1. INTRODUCTION

St. Anthony Falls Hydraulic Laboratory was contracted by the West Virginia office of the U. S. Soil Conservation Service to construct and evaluate a physical scale hydraulic model of Howard Creek as it flows through White Sulphur Springs, West Virginia. The reach in question extends from the Garden Street bridge on the upstream end to just past the Greenbrier Avenue bridge on the downstream end. This reach is subject to periodic flooding and has several complicating features. The complications include the confluence of Dry Creek just upstream of Big Draft Road near midreach, a substantial change in slope from approximately 0.00405 to 0.00151 near this confluence, and a corresponding change in sediment size distribution progressing downstream. In addition, the entire reach is closely surrounded by private residences as well as some commercial and public buildings.

The Soil Conservation Service is proposing channel modifications designed to reduce flooding in this reach. This study primarily evaluates performance of the proposed design with regard to sediment scour and deposition, and bank protection performance under conditions of both sediment deficient and sediment surplus, for each of 2–year, 10–year, and 100–year floods. (Please note that throughout the report the words starved or deficient are used interchangeably.)

This report covers the model design, model construction, the choice and sizing of suitable model sediment and riprap, calibration of the model, testing, modifications, and conclusions.

2. Model Construction

The area modeled, shown in Figure 1, contained the entire reach in question from the Garden Street bridge to about 600 feet downstream of the Greenbrier Avenue bridge. The upstream end of the model was located just downstream of the confluence of Wades Creek. This made it unnecessary to separately model Wades Creek. Dry Creek, on the other hand, was included in the model because its effect was of major importance to the model results.

All horizontal geometries were scaled down by a factor of 50; i.e. the model was a 1:50 horizontal scale model. The model basin used was approximately 60 ft long by 28 ft wide, representing a prototype dimension of 3000 ft by 1400 ft. As shown in Figure 1, water and sediment were supplied at the upstream end of both Howard Creek and Dry Creek in the model. Water was supplied by a pipeline equipped with valves and orifice meters to control and to monitor accurate discharges. Sediment was supplied by two commercial AccuRate auger–type sediment feeders.

The model was constructed of lightweight concrete, using flexible sheet metal templates to allow molding of the correct topography. The entire floodplain shown in Figure 2 was not modeled. The focus of the study was floods in the immediate vicinity of the proposed channel. As a result, the channel was modeled in detail. In order to also allow for study of overbank flow, however, a swath of floodplain extending 200 feet to either side of the channel centerline was modeled. Only major floodplain obstacles were installed in the model; this included the Garden Street, Big Draft Road, and Greenbrier Avenue bridges, and the fire station.

If a 1:50 ratio were also to be used for the vertical scale, the proposed prototype depth of 9 feet would translate into 0.18 ft, or 2.16 inches, in the model. The depth of the riprap key would, in this case, only be 0.09 ft, or 1.08 inches; these values are marginal for accurate placement in the model. As a result, the model required some distortion. It is desirable to keep this distortion as small as possible, however, in order to provide an accurate representation of the flow pattern and bed scour.

With this in mind, a distortion factor of 1.5:1 was chosen. This factor yields a vertical scale ratio of 1:33. Model channel depth would then be 3.2 inches, and the depth of the riprap key would be 1.6 inches. These values were manageable for the purposes intended.

In this case, the appropriate model principle for open channel flows is the distorted Froude principle. Using this principle, discharge is determined according to Eq. (1) below.

$$Q_M = Q_P \lambda_V^{3/2} \lambda_H \qquad (1)$$

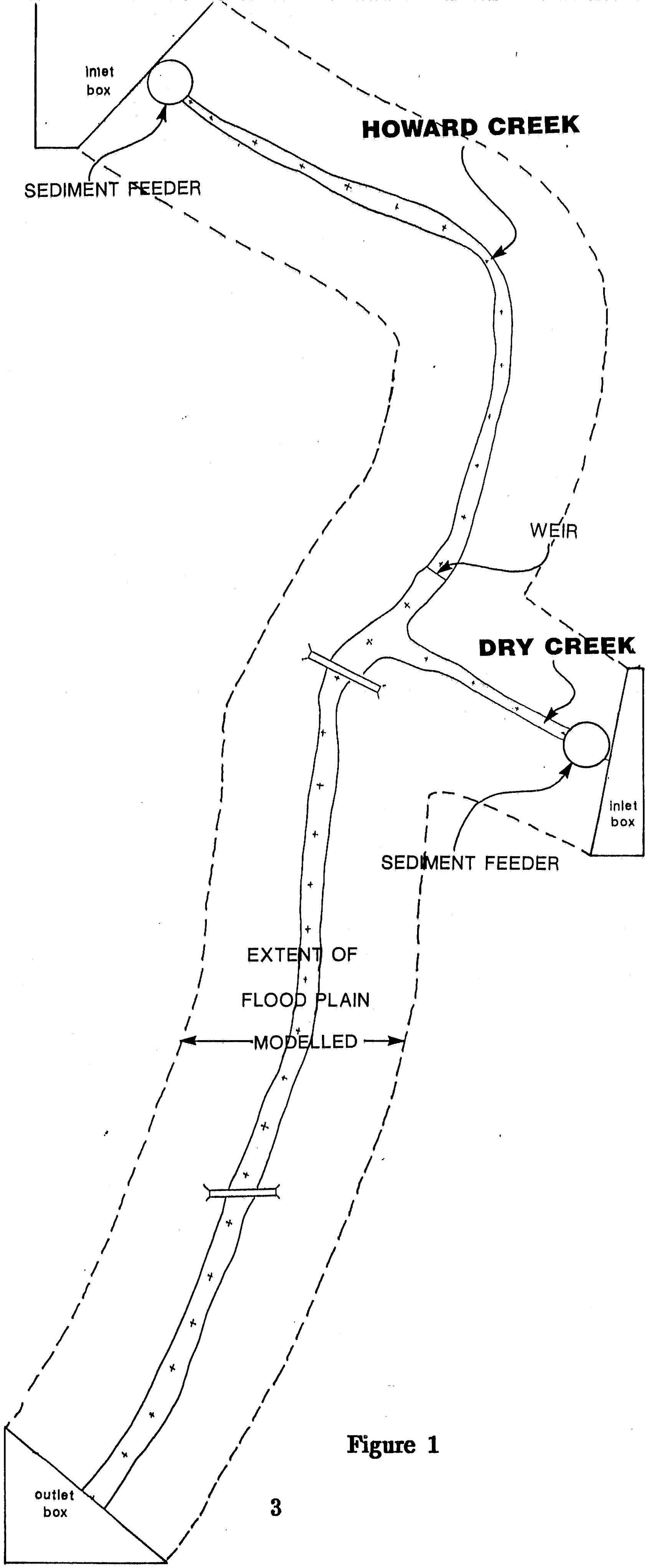
inlet box
HOWARD CREEK
SEDIMENT FEEDER
WEIR
DRY CREEK
inlet box
SEDIMENT FEEDER
EXTENT OF
FLOOD PLAIN
MODELLED
outlet box

Figure 1

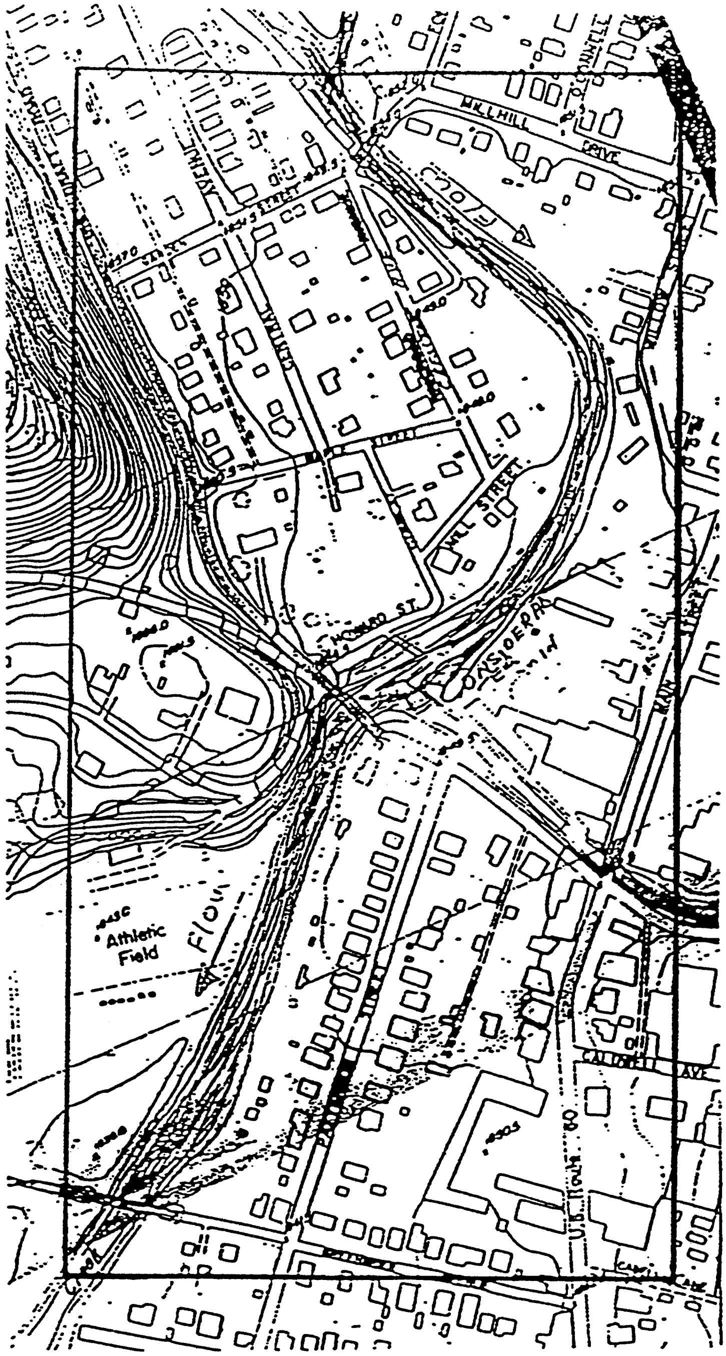

Figure 2 Map of Area to be Modeled

Here Q_P denotes the prototype discharge in cfs, Q_M denotes the model discharge in cfs, λ_H denotes the horizontal scale ratio, and λ_V denotes the vertical scale ratio. In the model, λ_H = 1:50 and λ_V = 1:33.

In this mildly distorted model, geometric scaling for sediment follows the following rule:

$$D_M = D_P \lambda_V^{1/3} \lambda_H^{2/3} \tag{2}$$

Here D_M denotes model sediment size, and D_P denotes prototype sediment size. A realistic prototype bed sediment size of 60 mm would scale to 1.4 mm in the model. The use of uniform sediment in the model, as opposed to mixtures, tends to slightly exaggerate local scour as opposed to mixtures. Results obtained in a model study using uniform sediment would therefore be conservative. Further information about the selection of model sediment and riprap can be found in section 3 of this report.

On a hydraulic basis, flood flows were designed to exhibit fully rough turbulent characteristics in the model. Since the entire floodplain was not being modeled for the 100–year return interval flood, estimated floodplain discharge was prorated for the area modeled before being scaled down to model values. The modeled portion of the floodplain was provided with a dense array of roughness elements in order to approximate field conditions. The roughness elements were formed from metal strips and affixed to the concrete bed of the model floodplain.

Riprap raveling was modeled in the following fashion. The zone of riprap placement was left open in model construction. After construction and during replacement of the bed sediment, scaled riprap was placed on a thin layer of sediment, in accordance with design specifications.

The distortion of the model would suggest that the riprap side slope in the model should be 2(horizontal):1.5(vertical). Local riprap performance is better modeled, however, with the prototype slope of 2:1. As a result, a conservative compromise was selected of 1.5(horizontal):1(vertical). The riprap was allowed to freely settle to a stable configuration during the course of each run.

3. Sediment and Riprap Sizing

Sizing of the bed sediment in the model was based on four large field sediment samples, three taken from the bed of Howard Creek and one taken from the bed of Dry Creek. The field sediment samples were predominantly flat, disc-shaped gravel, with a substantial amount of sand-sized particles in some samples. At present, no reliable and accepted methodology exists for relating the transport of predominantly disc-shaped particles as bedload to an equivalent rate for more rounded particles. After some analysis, the prototype size distribution shown in Figure 3 was adopted as representative of Howard Creek. This size distribution has a median size of about 38 mm.

In a distorted movable-bed model, there are two ways to scale sediment sizes which are not necessarily equivalent. The first and simplest is geometric scaling. According to this method, if D_P denotes prototype sediment size and D_M denotes model sediment size, the two are related as

$$D_M = D_p \lambda_V^{1/3} \lambda_H^{2/3} \tag{3}$$

In the present case, this yields the scaling law

$$D_M = \frac{1}{43.7} D_P \tag{4}$$

The other alternative is the use of Shields stress modeling. The bed shear stress τ_b is assumed to be given by the relation

$$\tau_b = \rho C_f U^2 \tag{5}$$

where ρ denotes water density, U denotes cross-sectionally averaged flow velocity, and C_f denotes a (constant) bed friction factor. The dimensionless Shields stress $\tau^\star$ is given as

$$\tau^\star = \frac{\sqrt{\tau_b}}{\sqrt{(\rho_s - \rho) g D}} \tag{6}$$

where ρ_s denotes sediment density and g denotes the acceleration of gravity. It provides a dimensionless measure of the ratio of the impelling drag force due to the flow on a grain of size D to its force of resistance to movement. Imposing the condition that $\tau^\star$ must be the same in the model and the prototype for a given sediment size D, and further assuming that the friction

ADOPTED PROTOTYPE GRAIN SIZE DISTRIBUTION

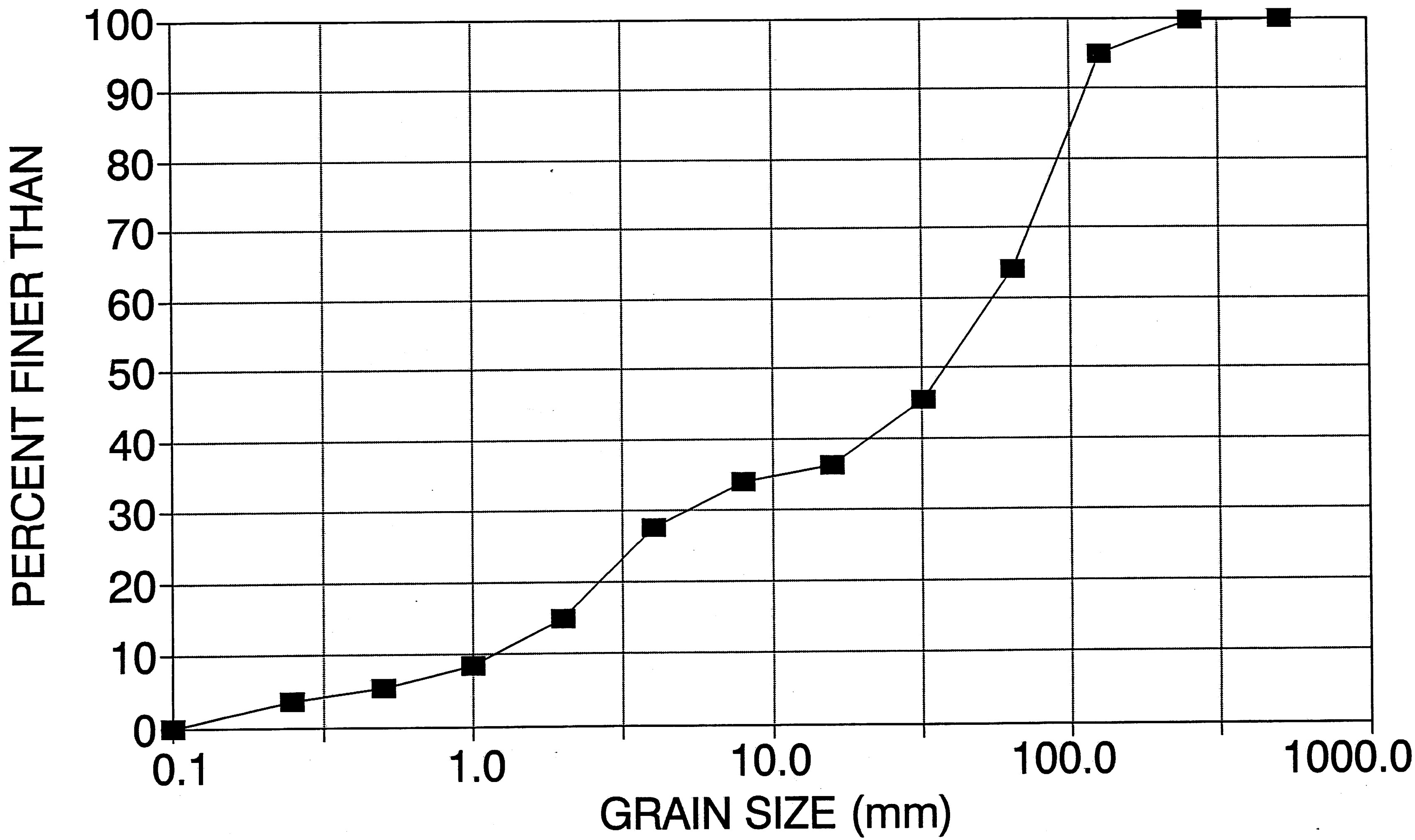

Figure 3

factor is the same in the model as it is in the prototype, the following scale relation is obtained:

$$D_M = D_P \lambda_V \tag{7}$$

For the present model, this yields the result

$$D_M = \frac{1}{33.3} D_P \tag{8}$$

Conditions (4) and (8) would be identical if there were no distortion. Because the distortion is mild in the present case, the two conditions give quite similar results. Due to the importance of modeling sediment movability correctly in the present case, (8) is adopted for the purposes of scaling bed sediment. A direct application of (8) to the size distribution of Figure 3 gives a model value of median size D_{50} of 1.1 mm. The resulting model size distribution is such that 65 percent is finer than 2 mm and 37 percent is finer that 0.5 mm. A compromise was made between the utility of having a wide range of sizes and the simplicity of having a narrow one. The model sediment was selected as follows: 100 percent was finer than 2 mm, 40 percent was finer than 1 mm, and 0 percent was finer than 0.5 mm. This yields a D_{50} of 1.1 mm, as desired, but also supplies a fairly wide range of grain sizes corresponding to the prototype range 17 ~ 67 mm.

Riprap gradation in the prototype was specified by the Soil Conservation Service as ranging from 8 to 18 inches. This riprap is here referred to as Type I riprap. Since riprap should be designed so that it is not mobilized by the flow, Shields modeling was selected in preference to geometric modeling for the riprap as well. The field range thus scales down to 4.6 to 13.7 mm in the model. This range was adjusted slightly to accommodate available sieves. The resulting model Type I riprap gradation was as follows: 100 percent finer than 16 mm, 50 percent finer than 8 mm, and 0 percent finer than 4 mm.

As noted subsequently in this report, the results of model testing indicated that a coarser grade of riprap would be required at some locations. The *Guide to Bridge Hydraulics* (Neill, 1973) was used as a guide in order to compute the new gradation. The calculations indicated that 12 to 30 inch riprap would be adequate. This material is referred to as Type II riprap. Using the same principles as above, the scaled model riprap ranged from 9.1 mm to 23 mm. Using available sieves, this was adjusted to the range 7.9 to 22 mm for use in the model.

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[1]Neill, C. R., ed. 1973. *Guide to Bridge Hydraulics*, prepared by Project Common Bridge Hydraulics, Roads, and Transportation, Assoc. of Canada.

4. Hydrograph Scaling

An important feature of the model study was the modeling of the complete hydrographs for the 2–year, 10–year, and 100–year floods. Discharges and times were provided by the Soil Conservation Service for Howard Creek both upstream and downstream of Dry Creek. This allowed for the determination of inflow hydrographs to be used at the upstream end of Howard and Dry Creeks in the model.

In designing the present model study, the bed resistance altered by the mobile gravel bed of Howard Creek was taken to be identical to that altered by a static gravel bed. This assumption, which is standard for gravel–bed rivers with low to moderate loads, allowed for the use of clear–water hydrographs provided by the SCS to be directly incorporated into the model study.

In a distorted model, model time Δt_M is related to prototype time Δt_P as follows:

$$\Delta t_M = \Delta t_P \frac{\lambda_H}{\sqrt{\lambda_V}} \tag{9}$$

The relation for scaling discharge Q has been given as equation (1). These relations were used to scale down the hydrographs. These were further approximated into 15 minute steps, each of constant discharge.

In the case of the 100–year flood, significant inundation of the floodplain occurred. As outlined previously, only a part of the floodplain is modeled in the present study. As a result, it was necessary to adjust the prototype hydrographs so as to exclude the floodplain discharge not represented in the model before scaling the hydrographs down. This was done with a standard conveyance calculation, using channel and floodplain values of Manning's "n" supplied by the Soil Conservation Service.

The adopted hydrographs in terms of both model and prototype discharge for both Howard and Dry Creeks are given as Table 1 (2–year flood), Table 2, (10–year flood) and Table 3 (100–year flood).

Table 1

Model Hydrograph
Howard Creek
Two–Year Flood

Model hr	Time min	Howard Creek Model Q cfs	Howard Creek Prototype Q cfs	Howard Creek Model Sediment Load Starved g/min	Howard Creek Model Sediment Load Surplus g/min	Dry Creek Model Q cfs	Dry Creek Prototype Q cfs
0	00	0.03	250	0	0	0.01	90
0	15	0.14	1300	130	510	0.10	990
0	30	0.11	1050	82	330	0.08	780
0	45	0.09	840	50	200	0.07	660
1	00	0.08	730	37	150	0.06	580
1	15	0.06	630	6	24	0.06	530
1	30	0.06	600	6	24	0.05	500
1	45	0.05	490	0	0	0.04	420
2	00	0.01	60	0	0	0.02	220

Table 2

Model Hydrograph
Howard Creek
Ten–Year Flood

Model hr	Time min	Howard Creek Model Q cfs	Howard Creek Prototype Q cfs	Howard Creek Model Sediment Load Starved g/min	Howard Creek Model Sediment Load Surplus g/min	Dry Creek Model Q cfs	Dry Creek Prototype Q cfs
0	00	0.18	1700	195	780	0.03	300
0	15	0.48	4630	700	2800	0.28	2740
0	30	0.28	2720	380	1520	0.18	1720
0	45	0.21	2020	250	1000	0.13	1270
1	00	1.18	1680	195	780	0.11	1060
1	15	0.15	1470	145	580	0.10	930
1	30	0.14	1310	130	510	0.09	870
1	45	0.08	770	37	150	0.08	740
2	00	0.01	110	0	0	0.03	320

Table 3

Model Hydrograph
Howard Creek
100–Year Flood

Model hr	Time min	Howard Creek Model Q cfs	Howard Creek Prototype Q cfs	Howard Creek Model Sediment Load Starved g/min	Howard Creek Model Sediment Load Surplus g/min	Dry Creek Model Q cfs	Dry Creek Prototype Q cfs
0	00	0.031	300	0	0	0.016	150
0	15	0.540	5200	775	3100	0.135	1300
0	30	0.960*	9600	1350	5400	0.303*	5500
0	45	0.561	5400	800	3200	0.322	3100
1	00	0.385	3700	535	2140	0.213	2050
1	15	0.307	3000	400	2140	0.177	1700
1	30	0.270	2600	350	1600	0.156	1500
1	45	0.239	2300	290	1400	0.140	1350
2	00	0.156	1500	155	1160	0.120	1150
2	15	0.031	300	0	620	0.042	400

*Flood plain proportional discharge.

5. Model Calibration

Model calibration started on December 2, 1992, and was completed January 7, 1993. In any movable bed model, particularly those where little field information is available, controlled pretesting of the model is necessary to "calibrate" sediment feed rates versus river discharge and to develop a relationship between tailgate position and tailwater level. This information is then used for testing various flood hydrographs.

Three discharges were chosen for model calibration: one–half peak discharge of the two–year event, peak discharge for the two–year event, and peak discharge for the 10–year event. These discharges were chosen because they provided sufficient variation to develop the sediment discharge versus water discharge relationship shown in Figure A–1 of the Appendix. For each of these conditions, steady discharges were evaluated in conjunction with various sediment feed rates until equilibrium or near equilibrium conditions were reached which were representative of the field conditions. Calibration of this particular model was complicated by the confluence of Dry Creek, which had its own discharge and sediment load, and the discontinuity in slopes.

6. Testing

Upon completion of the model calibration, testing of the 2–, 10–, and 100–year events commenced. The model was evaluated by using each hydrograph twice, once under a sediment deficient condition consisting of one–half of the "calibrated" sediment loading for a specific discharge and a second time, using sediment surplus conditions consisting of twice the amount of the "calibrated" sediment loading.

In summary, a total of eight runs were performed as listed in the following table. Station number and time refer to directly to the position on the video tape supplement to this report.

TABLE 4

Run No.	Station No.	Time/min.	Discharge Condition	Sediment Loading Condition	Riprap Condition
1	582	09:00	2 yr	Surplus	Design
2	1240	22:00	2 yr	Starved	Design
3	1482	27:20	10 yr	Surplus	Design
4	1771	34:20	10 yr	Starved	Design
5	2183	45:20	100 yr	Starved	Design
6	2368	50:40	100 yr	Surplus	Design
7	3073	72:45	100 yr	Starved	Modified
8	3372	83:00	100 yr	Starved	Modified

While the constant discharges used in the model calibration runs did not accurately represent the hydrographs of typical field conditions, they did provide an indication of potential zones for riprap failure. The early tests, Runs 1 and 2, using a 2–year hydrograph provided a more representative indication of the fact that several zones may be subject to failure of Type I riprap at higher discharges, as small zones were already experiencing failure. Runs 3 and 4 provided further indication of this failure, and selection of a modified Type II riprap was begun while runs 5 and 6 were being performed. Riprap was replaced with the modified Type II size distribution in areas subject to failure prior to Runs 7 and 8. Runs 7 and 8 indicated satisfactory performance.

The following is a summary of the runs evaluated, the techniques used in testing, and a general description of channel performance. Figure 4 shows the location of the ten cross sections used for discrete data sampling.

CROSS SECTION LOCATIONS

AND

BOULDER PLACEMENT

TESTING LOCATIONS

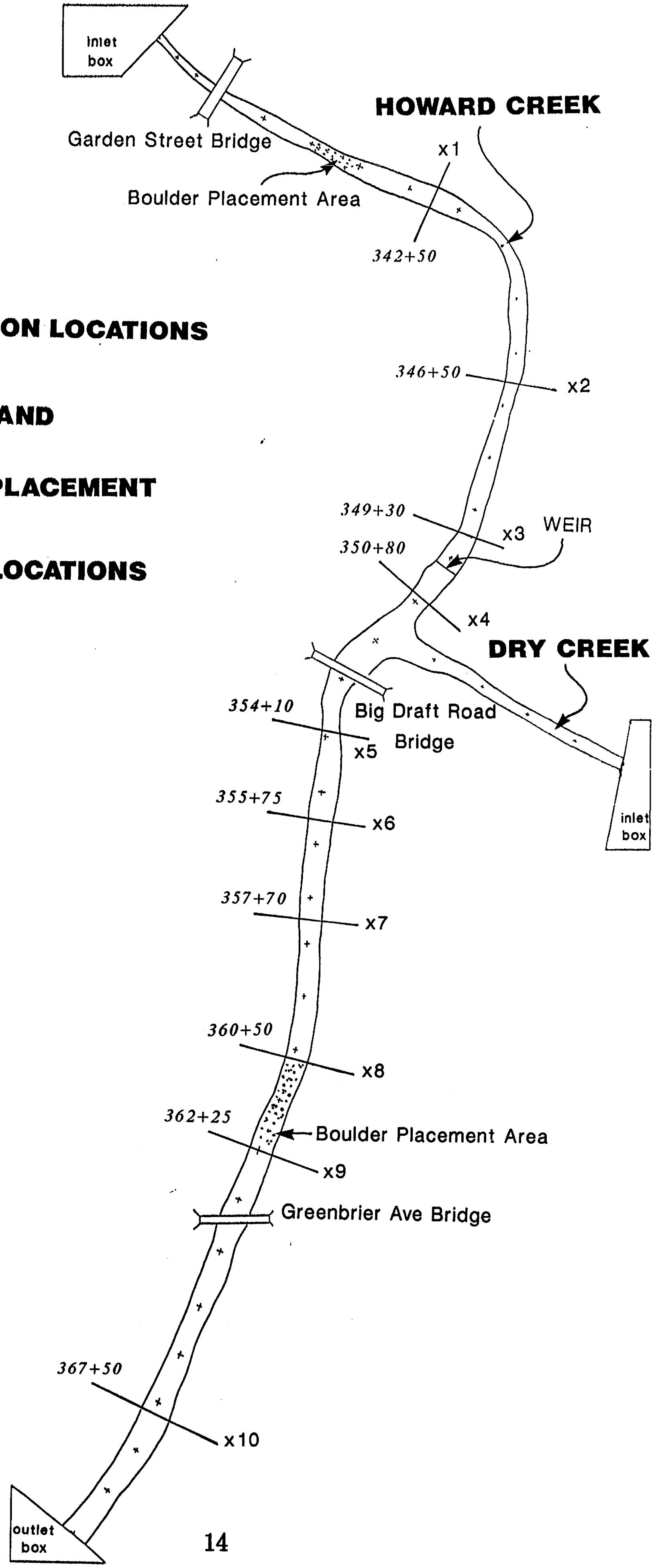


Figure 4

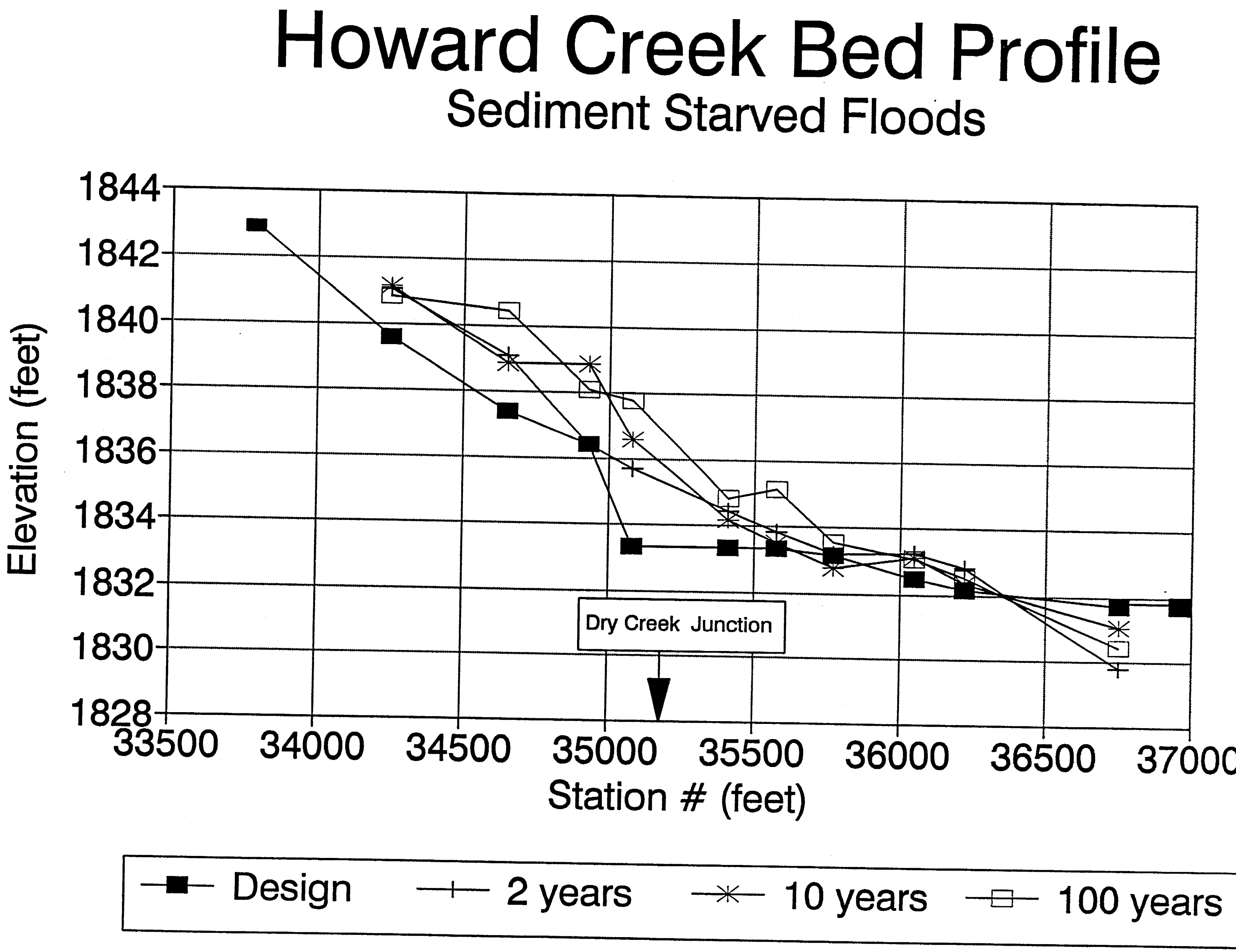
Due to a dam presently under construction, which will drastically reduce the amount of sediment entering Howard Creek via Dry Creek, the sediment feed to Dry Creek was eliminated after model calibration. Hence, no sediment arrived via Dry Creek for the test runs, a condition which should reasonably represent future field conditions.

Several general items regarding the testing procedure need stating but have little if any impact on the overall performance of the reach. First, the flood hydrographs from both Howard and Dry Creeks were initiated at the same time for all test runs. That is, the hydrographs were assumed to be perfectly in phase. No test runs were performed with a lag between the hydrographs of the two streams, as this could lead to endless combinations.

This method allowed assembly of the most general and suitable combinations in the time available for testing. It is likely that if the discharges were significantly offset in time, localized differences in scour might have occurred near the junction of the two streams. For example, the relatively high bed elevation between the weir and Dry Creek seems, in part, due to backwater effects related to the discharge of Dry Creek. Hence, bed elevations in this reach might be somewhat lower if Dry Creek were to have had a reduced discharge. Similarly, the size of the scour hole at the mouth of Dry Creek, and the effect on the right bank of Howard Creek across from the entrance of Dry Creek, might have been greater if the discharge of Howard Creek were reduced.

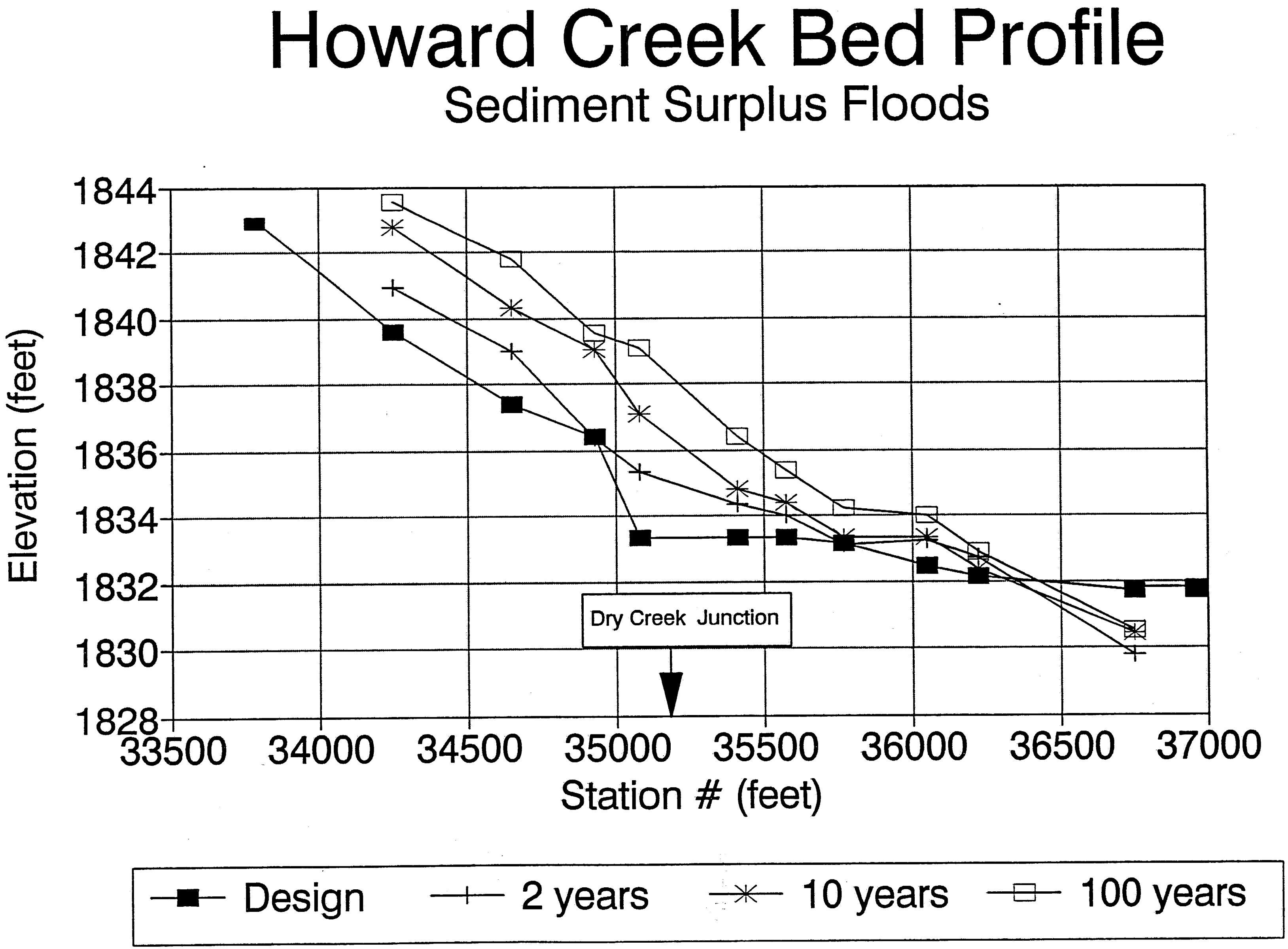
Both the calibration and early test runs indicated that the channel repeatedly formed a pattern very similar to that seen during the site visit conducted at the beginning of the project. With this in mind the reworking of the bed conducted prior to each run was limited to removal of major changes in the bed, removal of the sediment riprap mixture deposited on the bed during riprap failure, replacement of the failed riprap, and general reworking of the bed in selected areas. Little effort was made to assure that the bed was exactly as designed prior to each run, a simplifying procedure dictated by both time constraints and the fact that the bed returned to a repeatable "natural" shape within minutes of starting each run. The channel patterns indicated above were limited to the formation of some in–channel flow meandering and the formation of alternate bars. These deviations were not great and appeared to be produced by relatively lower discharges, utilizing less than the full bed width. Similar patterns appear to be reflected in the prototype. This procedure should have little effect on the test results.

Generally the test runs proceeded as planned and the overall results appear representative. As illustrated in Figures 5, 6, and 7, the bed slope routinely finished slightly steeper than the design slope. As expected, the sediment surplus runs deposited sediment in the upstream reaches of Howard Creek. The sediment starved runs more closely matched the design slope. Occasional anomalies where the bed is higher than normally anticipated are seen in bed topography. This is particularly true for cross sections 2 to 5 and can be explained by the failed riprap mixing with the bedload material thereby creating a relatively immovable bed. These zones are generally described in the following summary of specific test runs.



Howard Creek Bed Profile
Sediment Starved Floods
Elevation (feet)
1844
1842
1840
1838
1836
1834
1832
1830
1828
33500
34000
34500
35000
35500
36000
36500
37000
Station # (feet)
Dry Creek Junction
Design
2 years
10 years
100 years

Figure 5



Howard Creek Bed Profile
Sediment Surplus Floods
Elevation (feet)
1844
1842
1840
1838
1836
1834
1832
1830
1828
33500
34000
34500
35000
35500
36000
36500
37000
Station # (feet)
Dry Creek Junction
Design
2 years
10 years
100 years

Figure 6

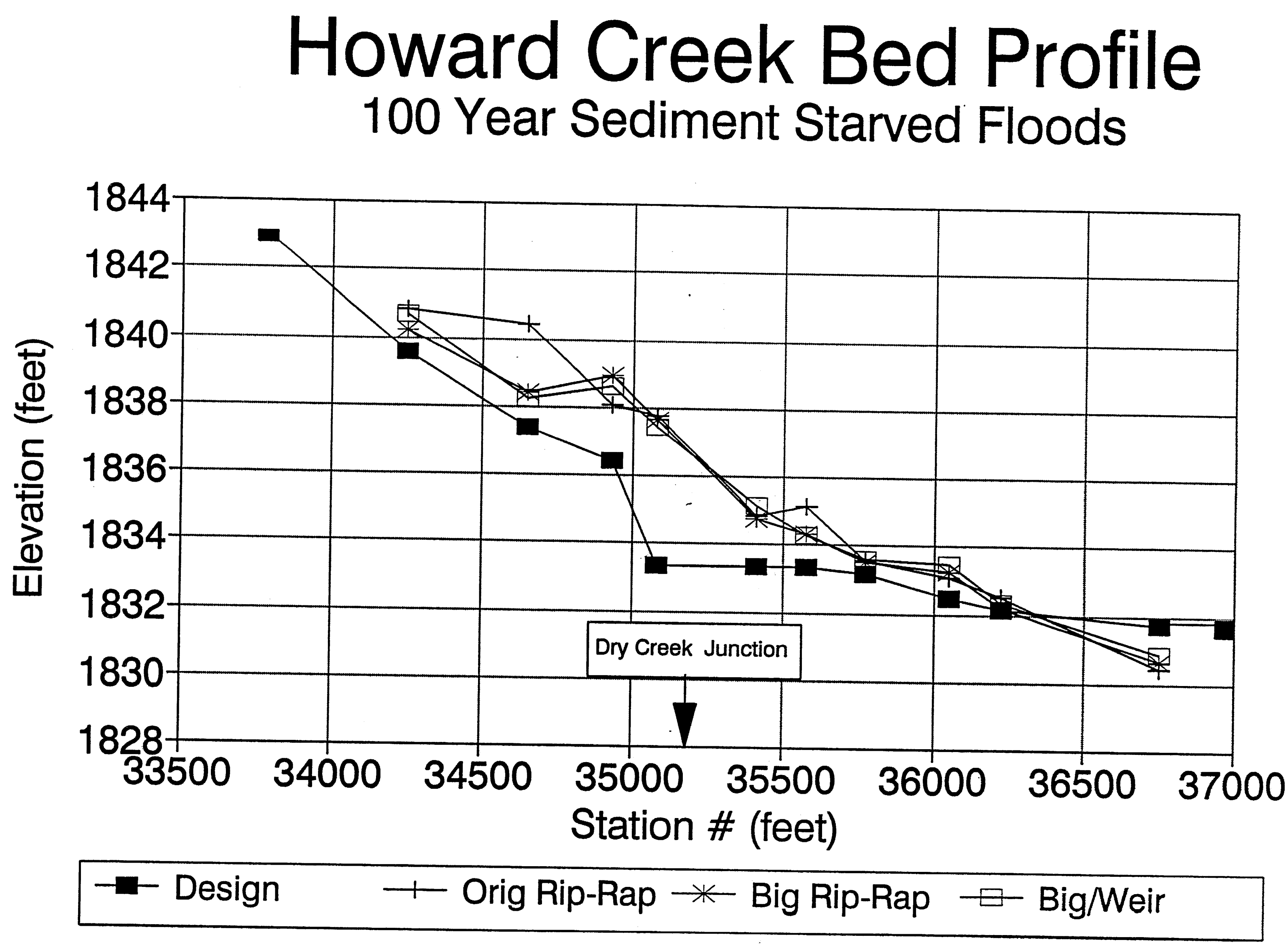

Howard Creek Bed Profile
100 Year Sediment Starved Floods
Elevation (feet)
1844
1842
1840
1838
1836
1834
1832
1830
1828
33500
34000
34500
35000
35500
36000
36500
37000
Station # (feet)
Dry Creek Junction
Design
Orig Rip-Rap
Big Rip-Rap
Big/Weir

Figure 7

All cross sections except cross section 10 exhibit some bed aggradation. The degradation of cross section 10 was noted to be on the order of one foot for the sediment–starved 100–year floods. This behavior may not in fact be reflected in the field. This is because cross section 10, being farthest downstream is most strongly affected by the tailgate, which was positioned by trial and error. In addition, the constriction caused by the Greenbrier Street bridge probably contributed to lower bed elevation during floods near this cross section. This notwithstanding, the performance of the model near cross section 10 does not suggest any reason for concern farther downstream.

For the 100–year return interval runs, floodplain discharge was proportioned according to the previously described technique. The varying slope conditions made separate proportioning necessary for both the upstream and downstream reaches. As Howard Creek is the stream being studied, the discharge from Dry Creek was adjusted so that the overall appropriate proportioned flow was modeled correctly for the downstream reach of Howard Creek.

All of the first series of tests were conducted using a two–foot high weir installed at the pipe crossing approximately 200 ft upstream of the confluence with Dry Creek.

Flood hydrographs and associated sediment loadings used for the testing are given in Tables 1 through 3. Discharges were determined by segmenting the flood hydrographs into time steps corresponding to approximately 2 hours prototype (15 minutes model). Upon completion of each run, video and photographic documentation were recorded, as well as data regarding bed and side slope elevations for 10 selected cross sections. Each of the cross sections are documented for any of the test runs in Figures 8 through 37. The figures are broken into three groups for plotting purposes as follows: the 2–year, 10–year, 100–year sediment deficient runs; the 2–year, 10–year, 100–year sediment surplus runs; 100–year sediment deficient runs using original Type I (8–18") design riprap, modified Type II (12–30") riprap, and modified Type II (12–30") riprap with the weir height reduced to one foot.

In summary, the channel performed as planned for both two–year hydrograph test runs with no major areas of concern. The channel formed a general bed configuration consistent with that observed during the field visit conducted at the beginning of the study. A small zone of riprap raveling was noted during the sediment deficient run just downstream of the main bend (Fig. 38). The data are summarized in figures 8 to 27, and useful information is also contained on the videotape provided.

The 10–year runs proved interesting with more obvious changes in bed contour and with several sections of riprap failure. In particular, the riprap at the toe of the rock masonry wall on the outside of the first bend downstream of the Garden Street bridge, hereafter referred to as the main bend (lefthand bank looking downstream), and the riprap on the bank for a reach of approximately 100 ft just downstream of the same wall, experienced failure during the sediment surplus run (Photo 1). Both 10–year tests experienced failure on the left bank just downstream from the weir (Photo 2). Figures 38 to 39 summarize the extent of the failure in plan view drawings.

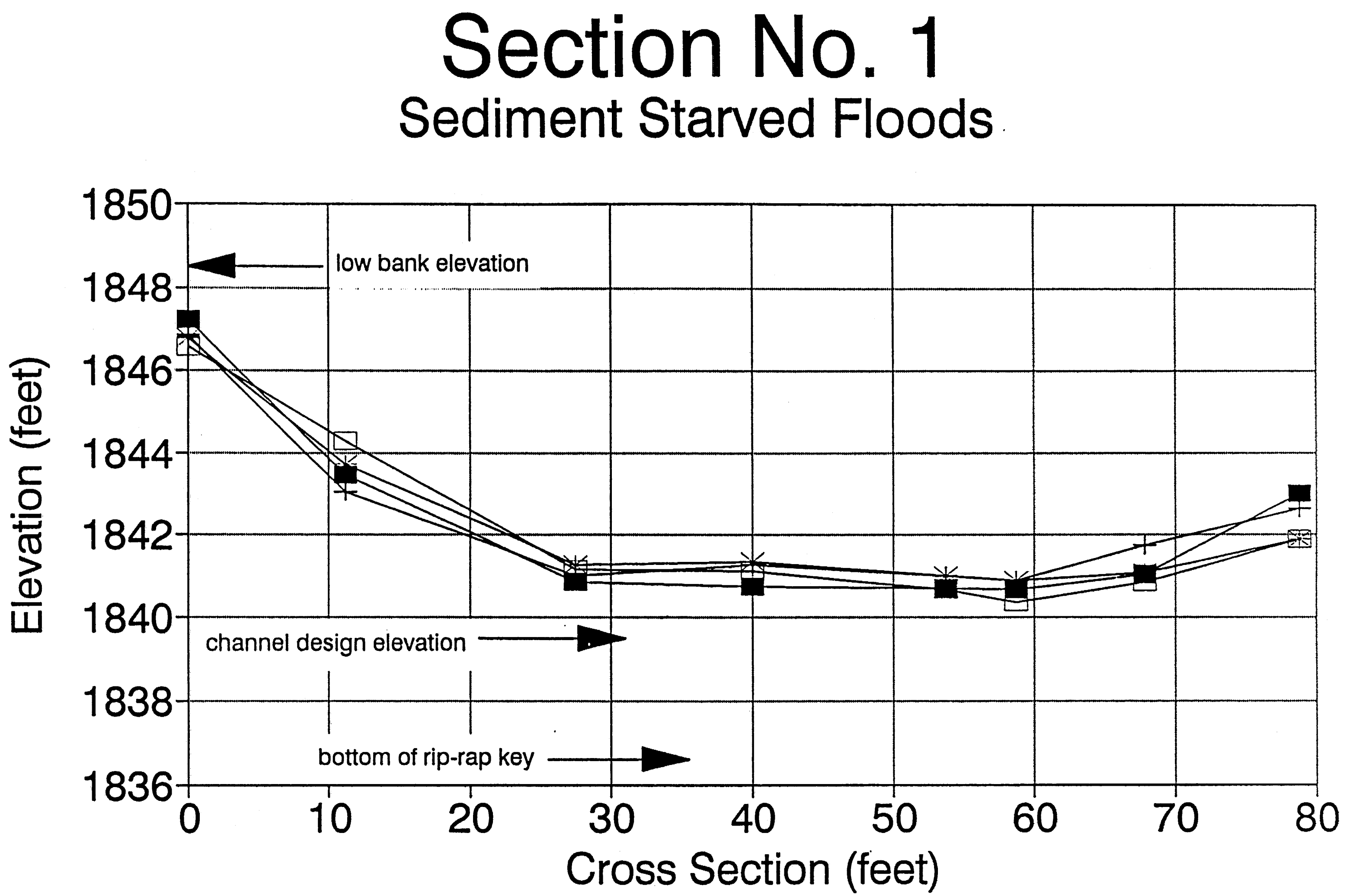
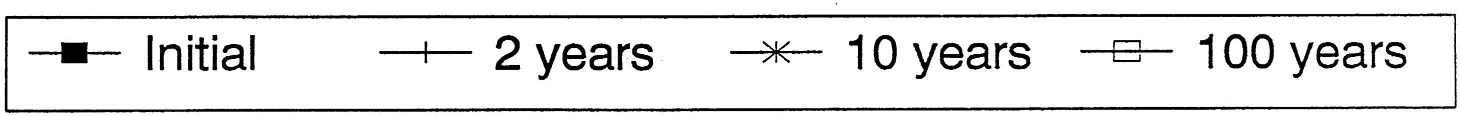


Figure 8

Original Rip-Rap

Station # 342+50



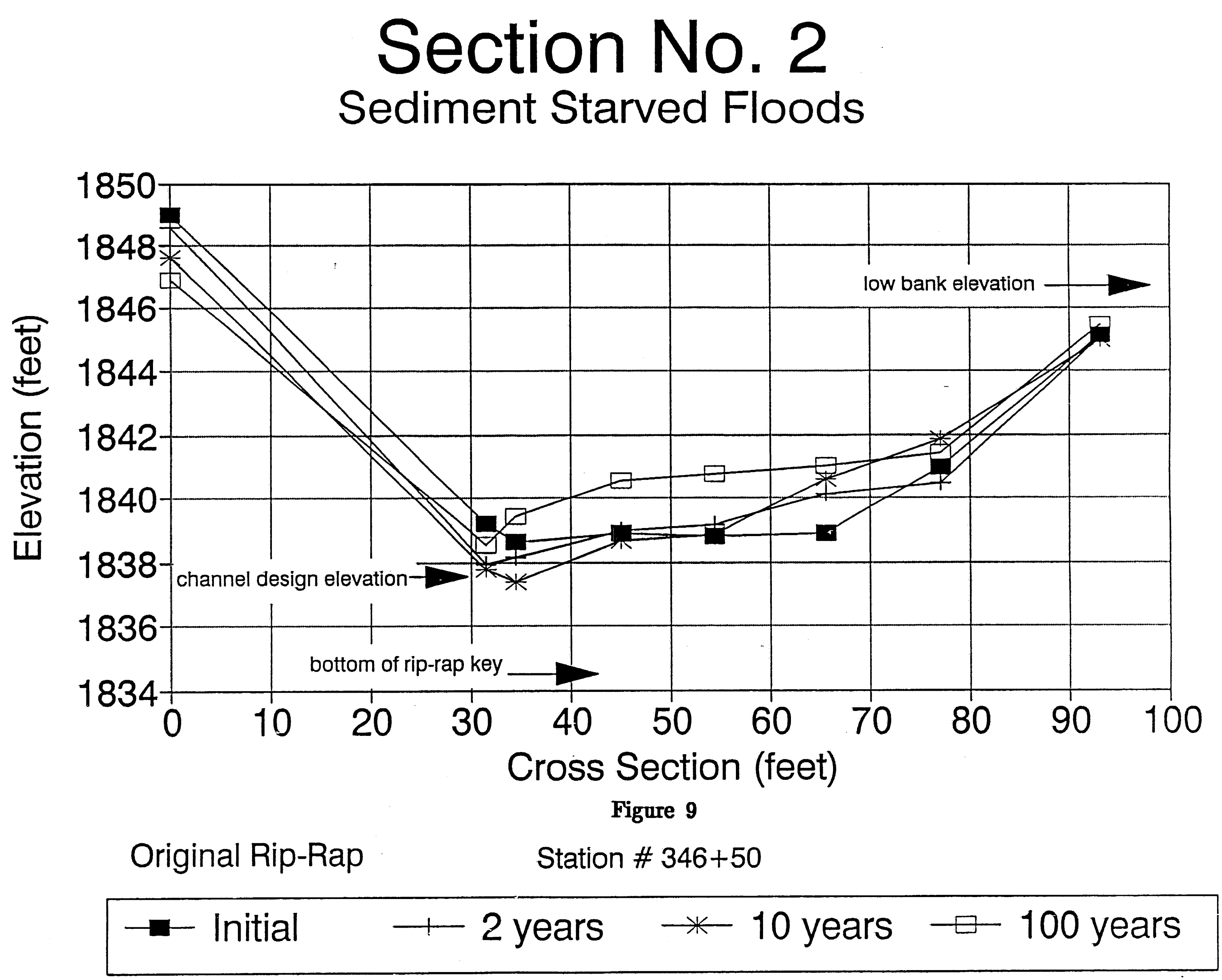

Section No. 2
Sediment Starved Floods
Elevation (feet)
1850
1848
1846
1844
1842
1840
1838
1836
1834
0
10
20
30
40
50
60
70
80
90
100
Cross Section (feet)
low bank elevation
channel design elevation
bottom of rip-rap key
Original Rip-Rap
Station # 346+50
Initial
2 years
10 years
100 years

Figure 9

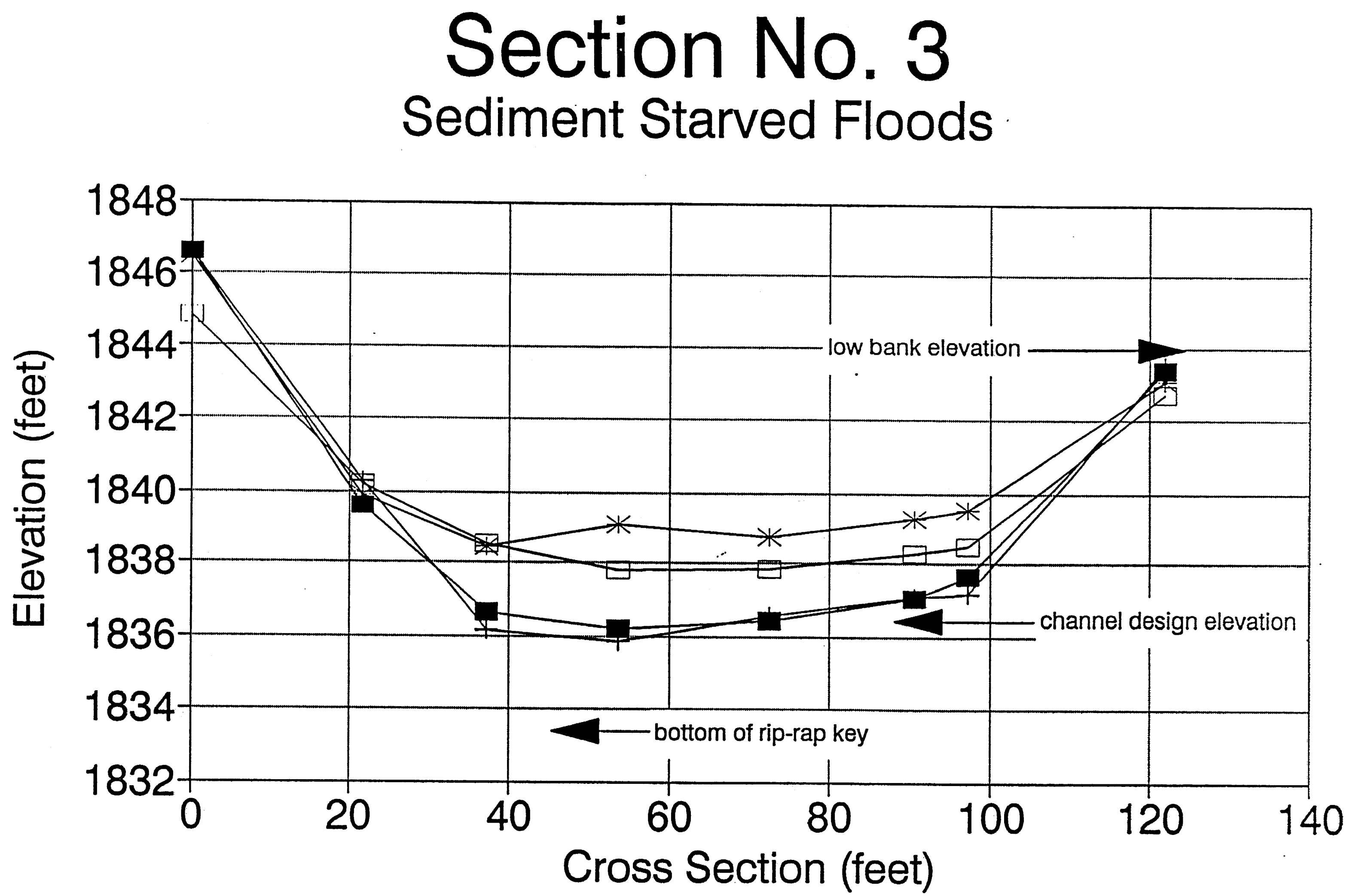


Figure 10

Original Rip-Rap Station # 349+30

Initial 2 years 10 years 100 years

Section No. 4

Sediment Starved Floods

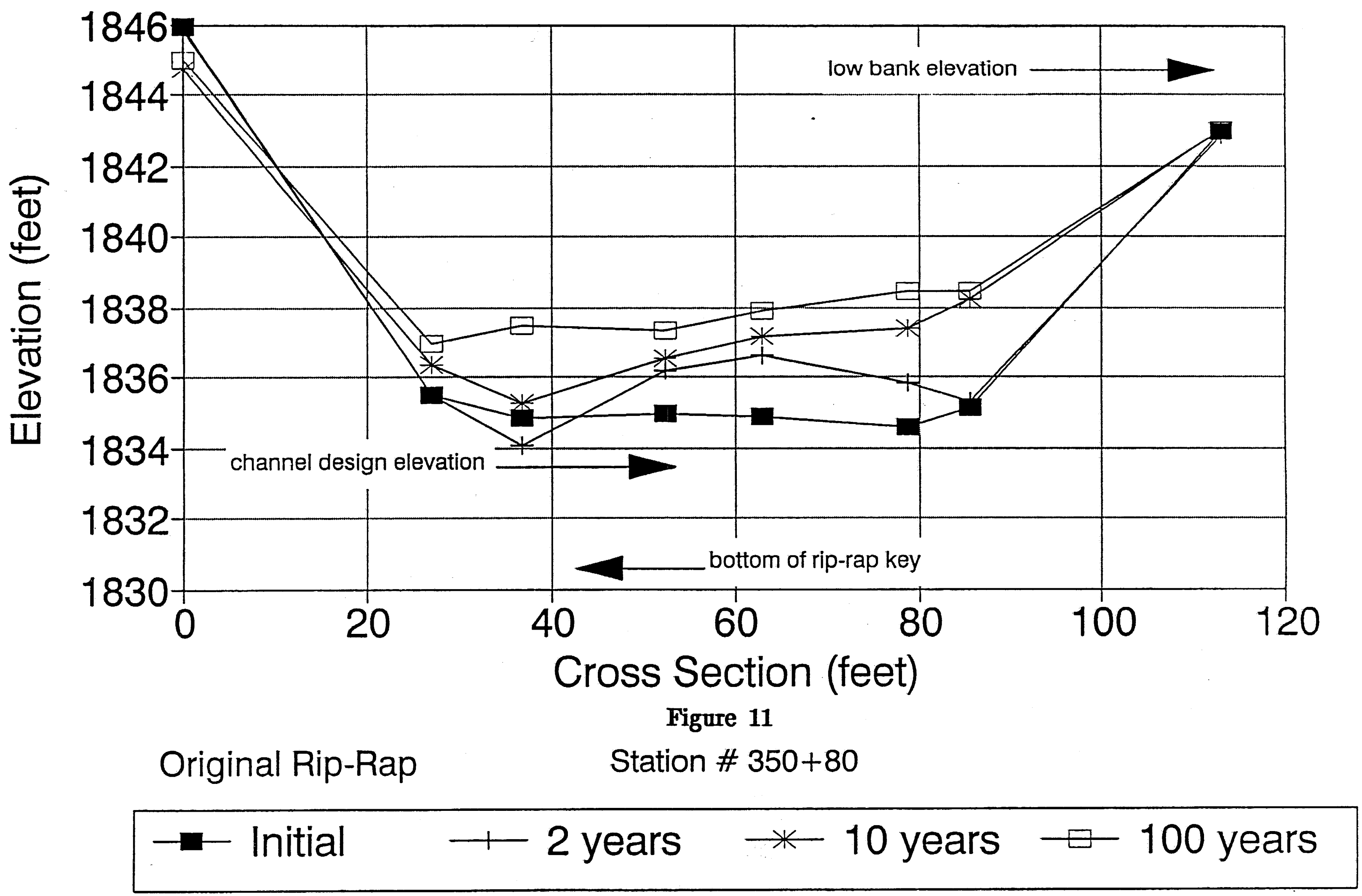


Figure 11

Section No. 5
Sediment Starved Floods

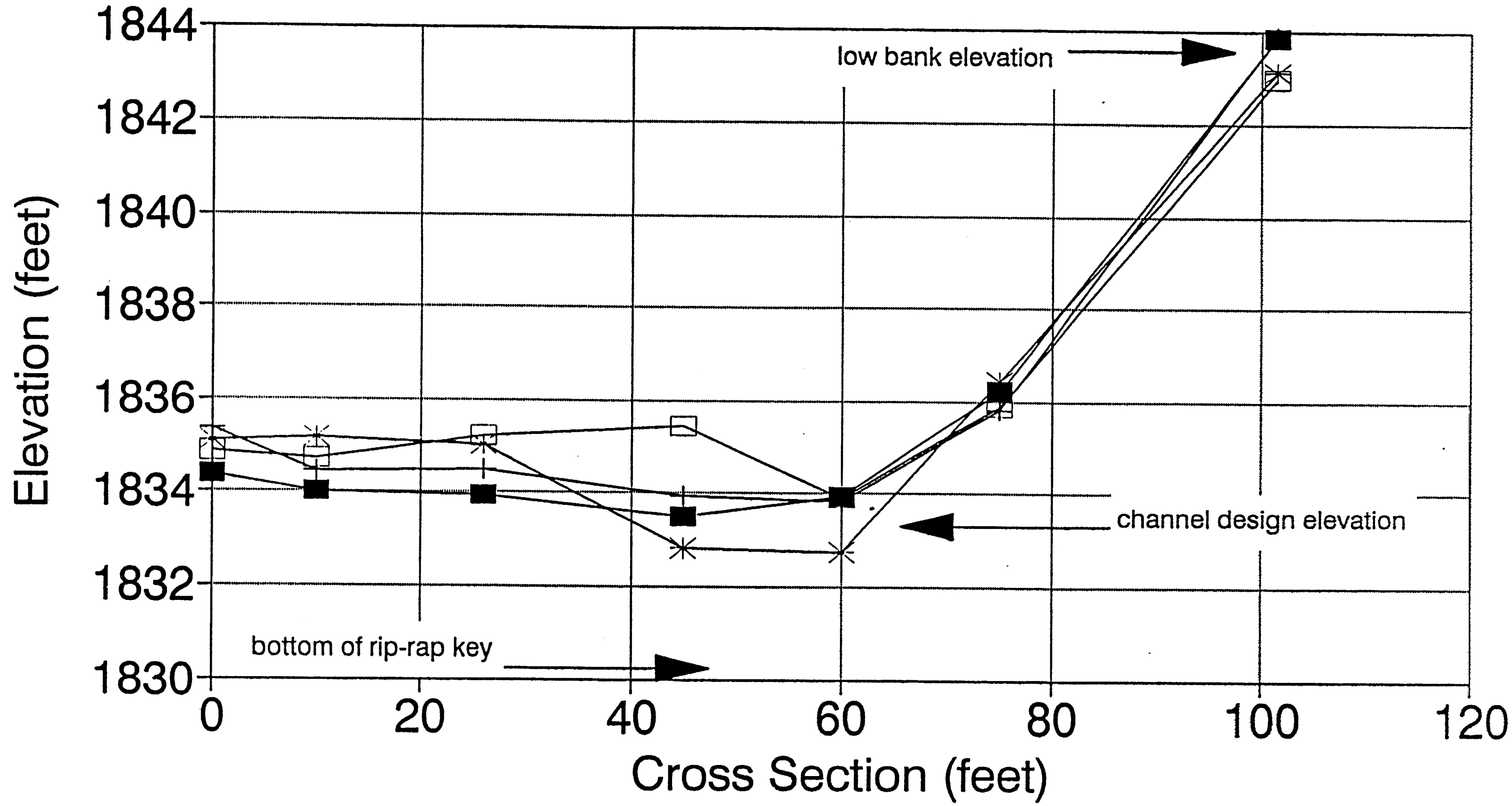
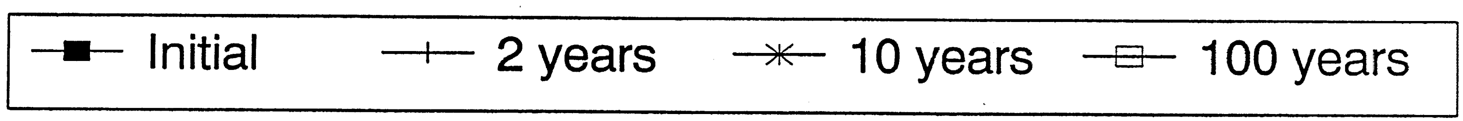


Figure 12

Original Rip-Rap

Station # 354+10



Section No. 6
Sediment Starved Floods

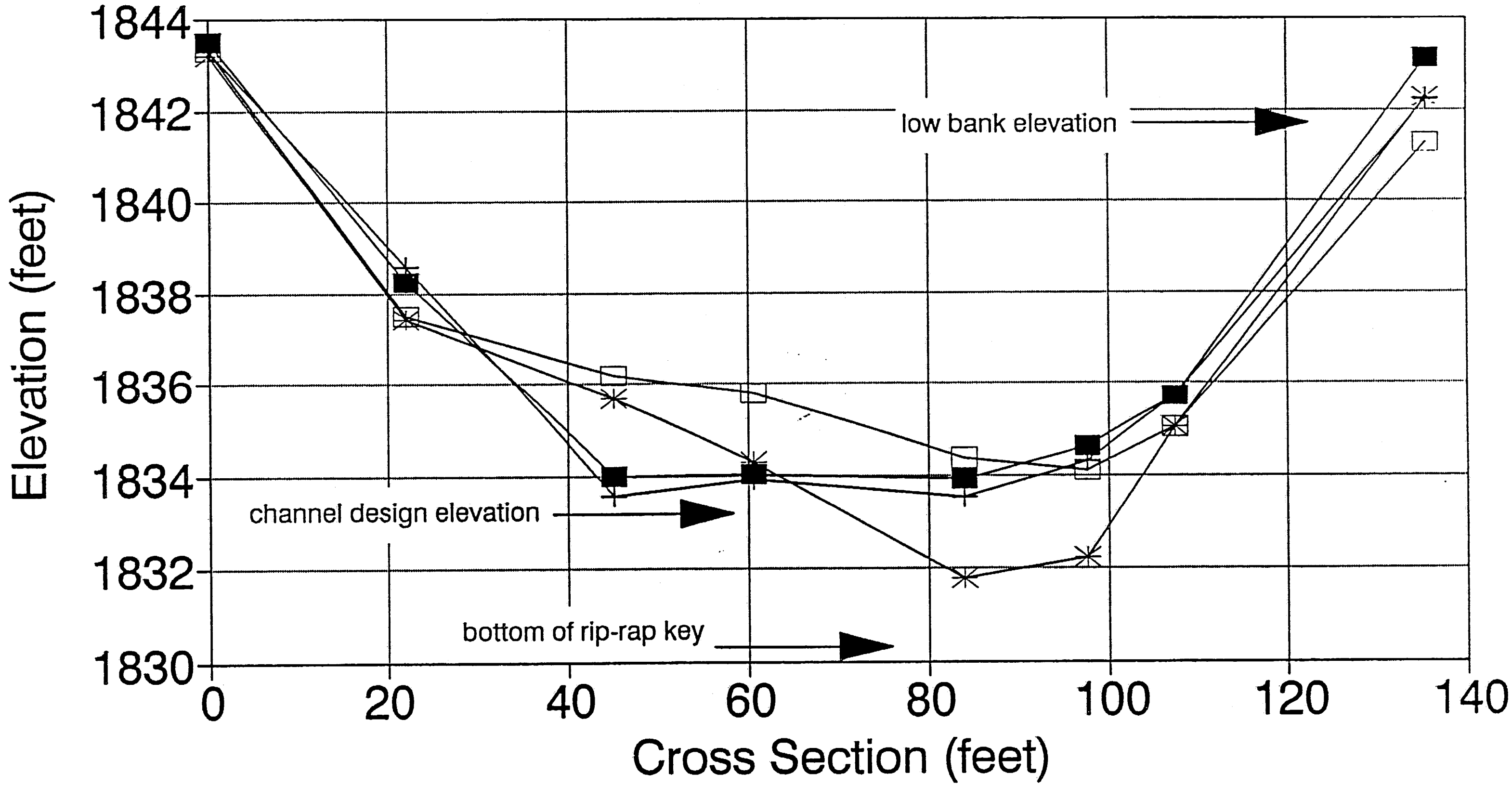


Figure 13

Original Rip-Rap

Station # 355+75

Initial — 2 years — 10 years — 100 years

Section No. 7
Sediment Starved Floods

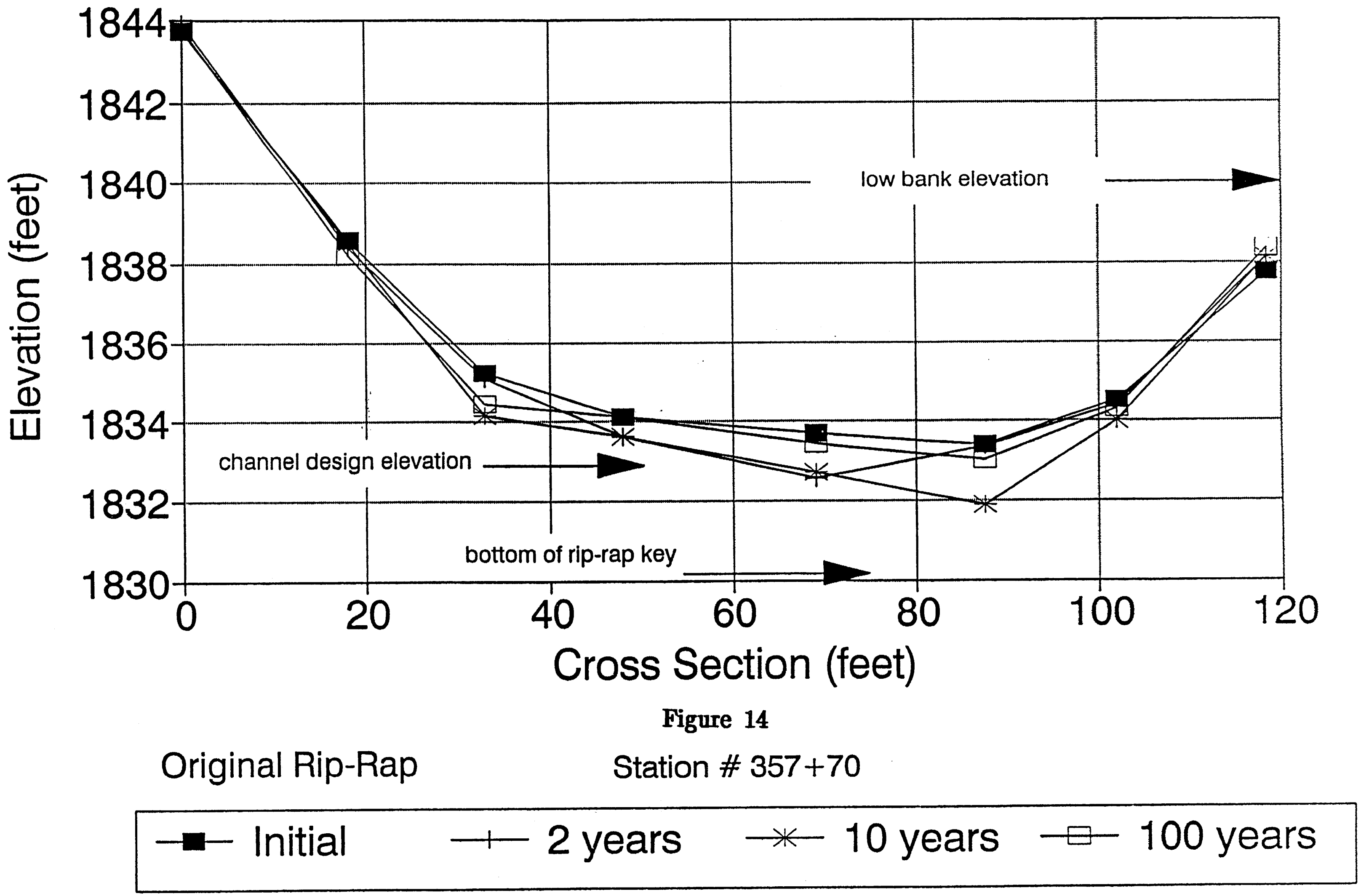


Figure 14

Section No. 8

Sediment Starved Floods

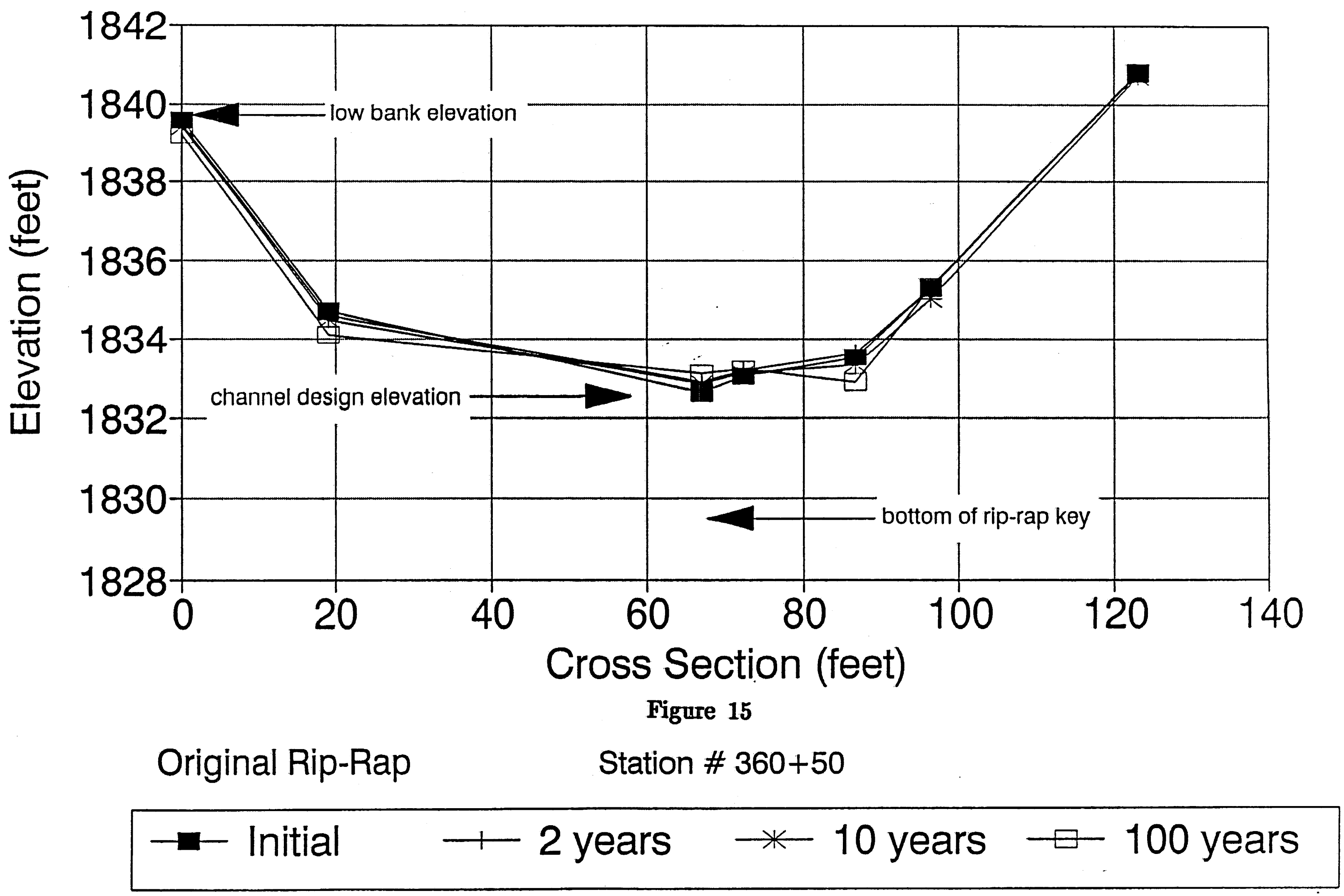


Figure 15

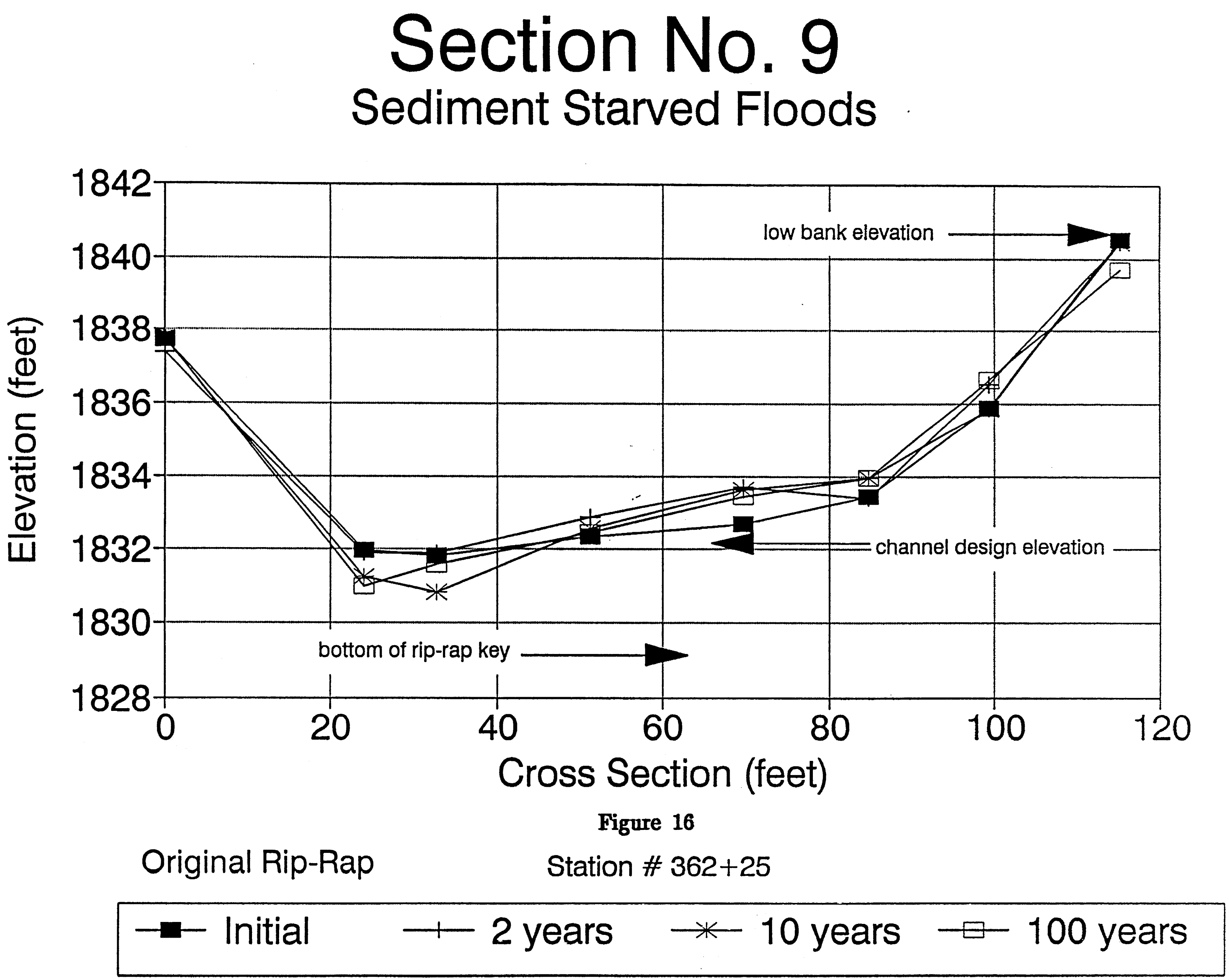

Section No. 9
Sediment Starved Floods
1842
1840
1838
1836
1834
1832
1830
1828
Elevation (feet)
0
20
40
60
80
100
120
Cross Section (feet)
low bank elevation
channel design elevation
bottom of rip-rap key
Original Rip-Rap
Station # 362+25
Initial
2 years
10 years
100 years

Figure 16

Section No. 10

Sediment Starved Floods

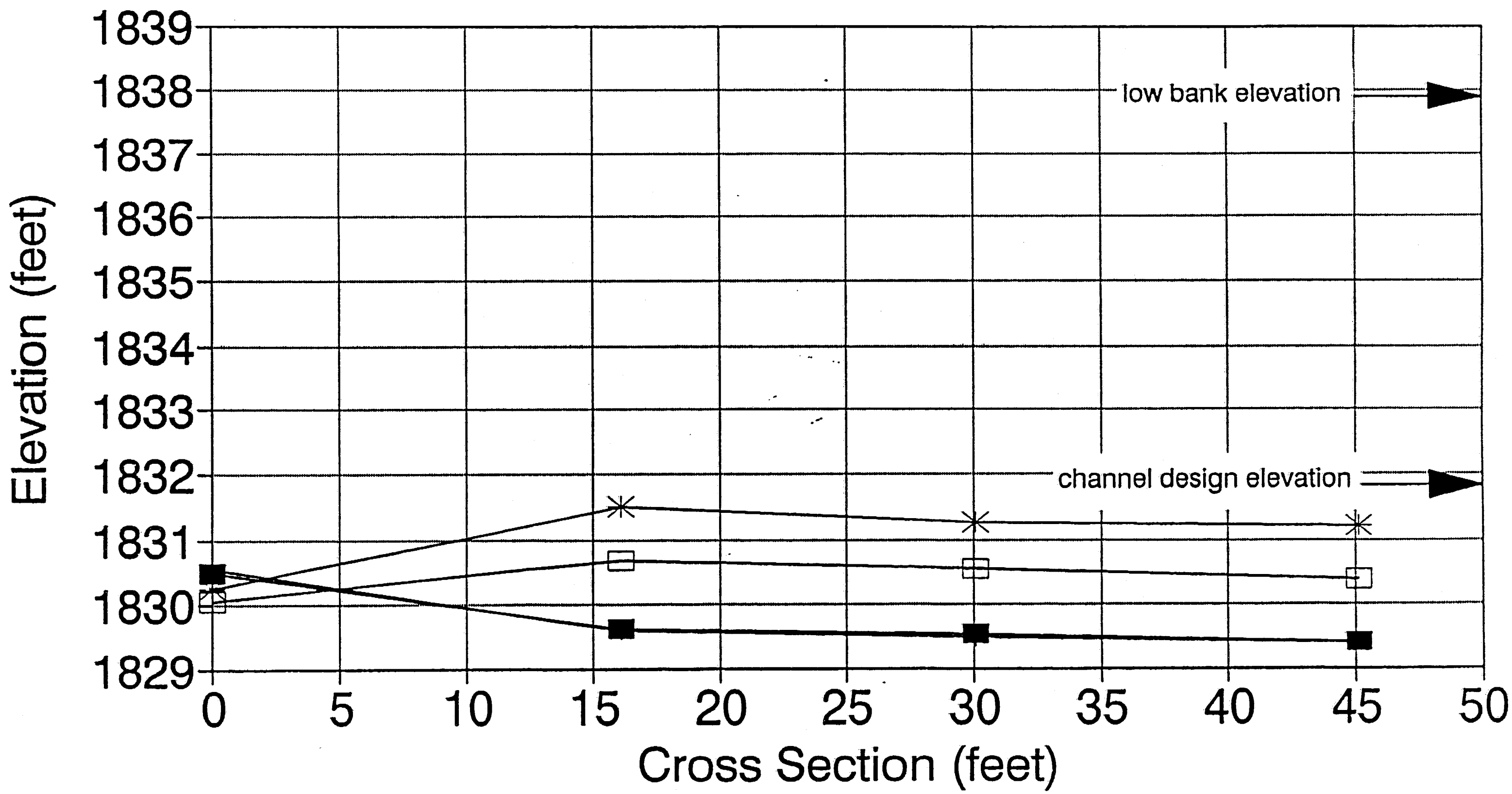


Figure 17

Original Rip-Rap

Station # 367+50

Initial — 2 years — 10 years — 100 years

Section No. 1

Sediment Surplus Floods

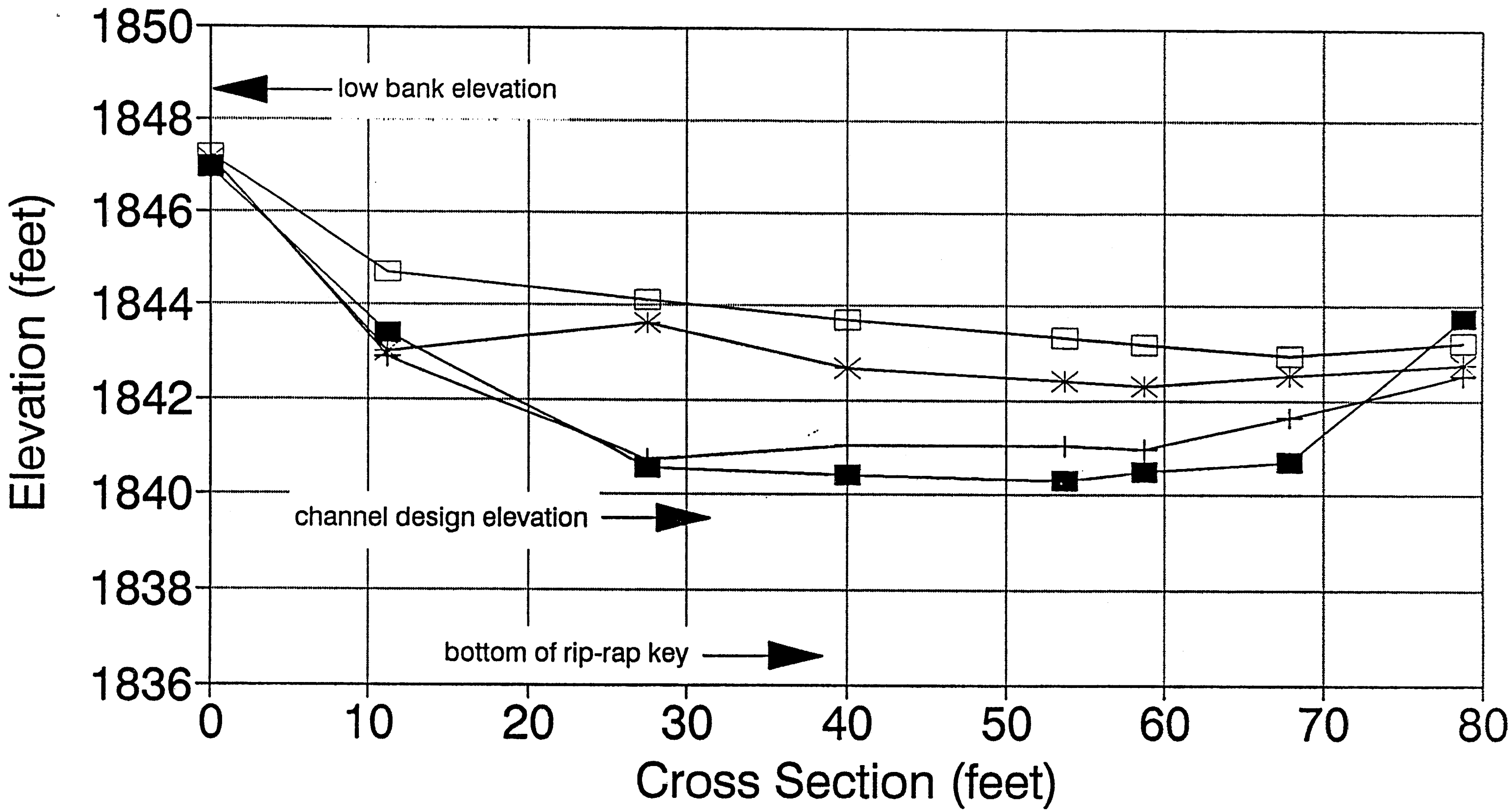
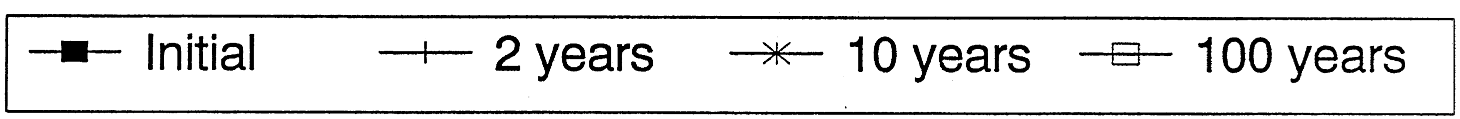


Figure 18

Original Rip-Rap

Station # 342+50



Section No. 2
Sediment Surplus Floods

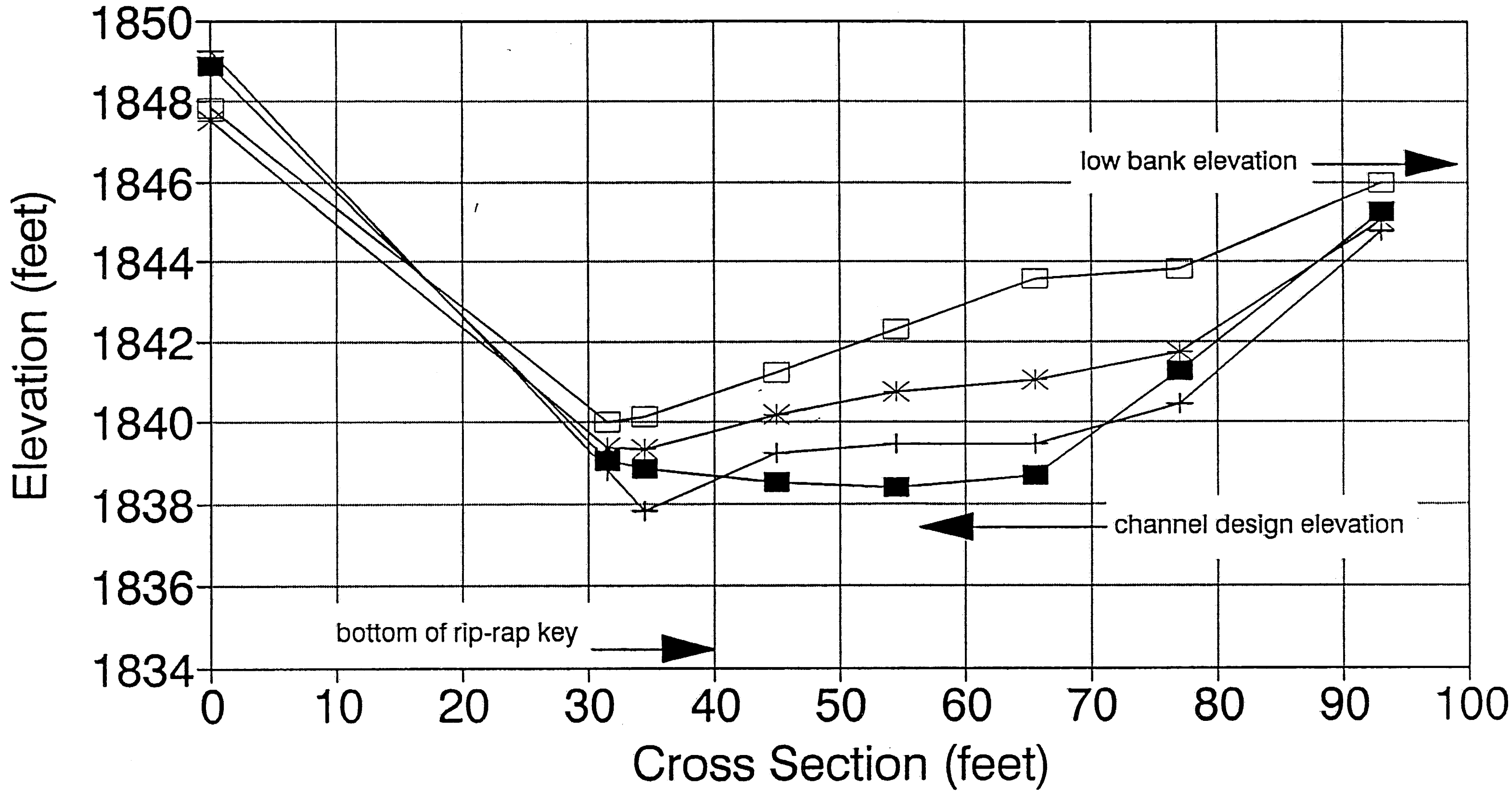


Figure 19

Original Rip-Rap Station # 346+50

Initial — 2 years — 10 years — 100 years

Section No. 3

Sediment Surplus Floods

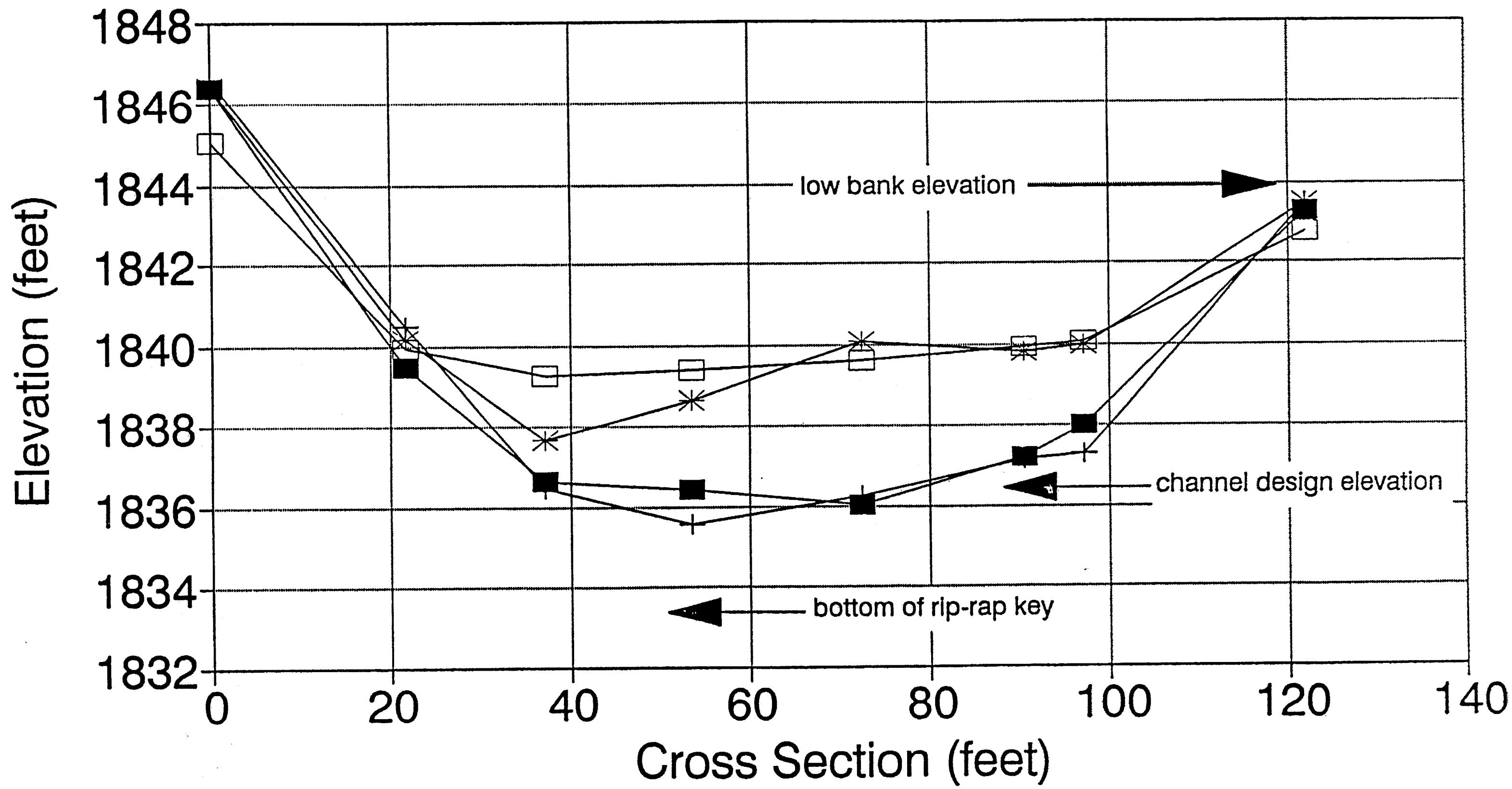


Figure 20

Original Rip-Rap Station # 349+30

Initial 2 years 10 years 100 years

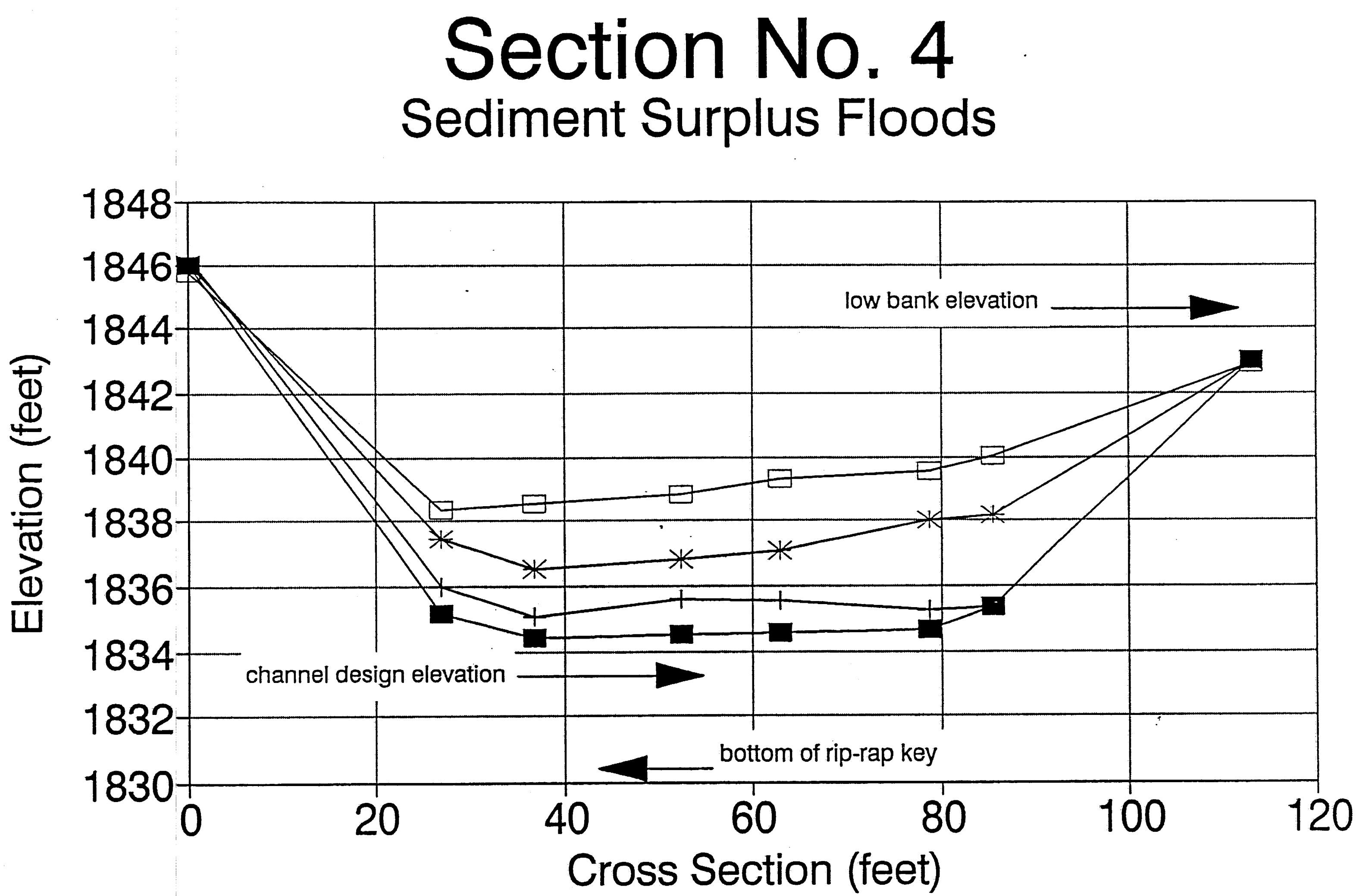


Figure 21

Original Rip-Rap

Station # 350+80

Section No. 5

Sediment Surplus Floods

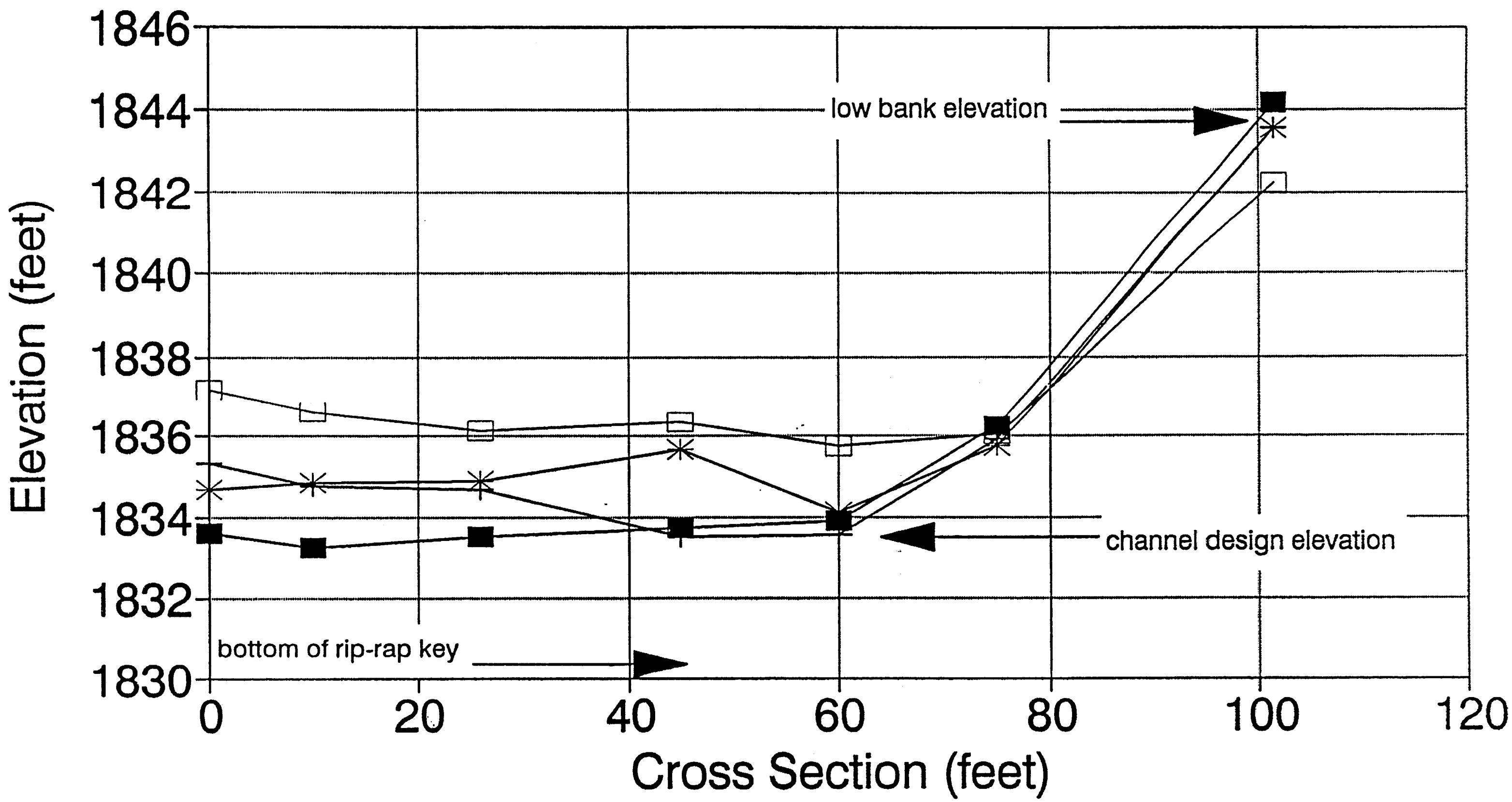


Figure 22

Original Rip-Rap Station # 354+10

Initial — 2 years — 10 years — 100 years

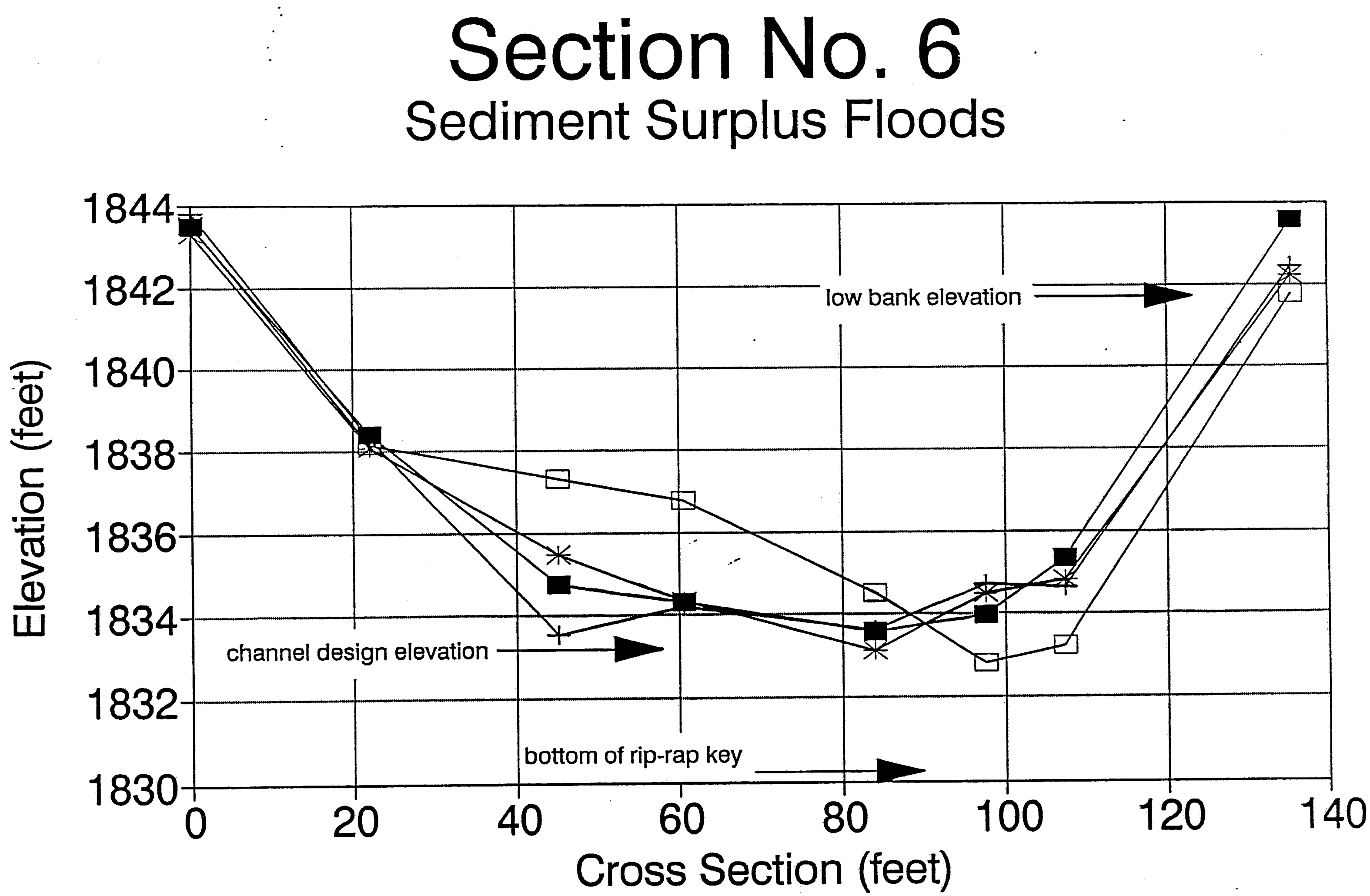


Figure 23

Original Rip-Rap

Station # 355+75

Initial — 2 years — 10 years — 100 years

Section No. 7

Sediment Surplus Floods

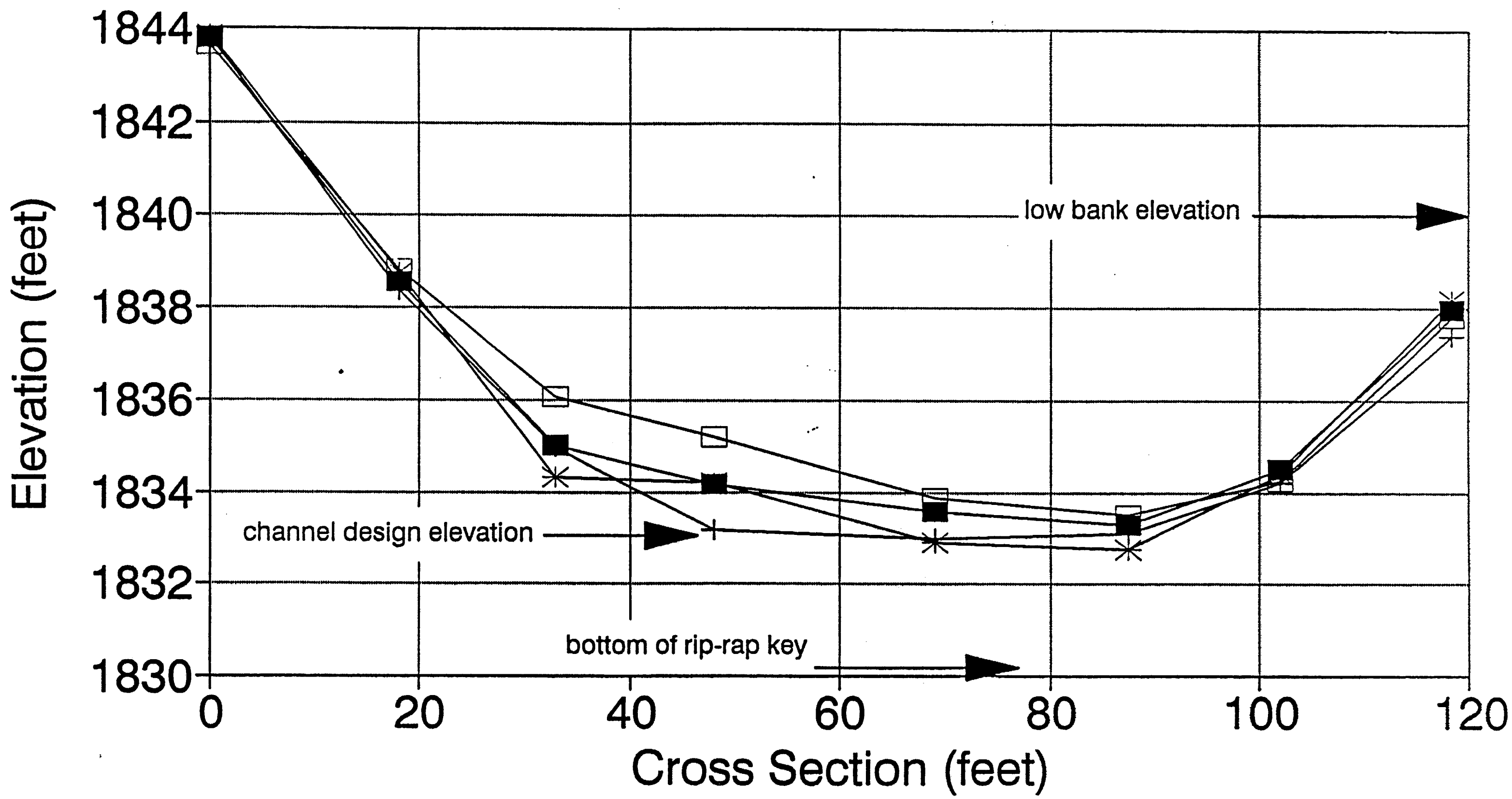


Figure 24

Original Rip-Rap

Station # 357+70

Initial — 2 years — 10 years — 100 years

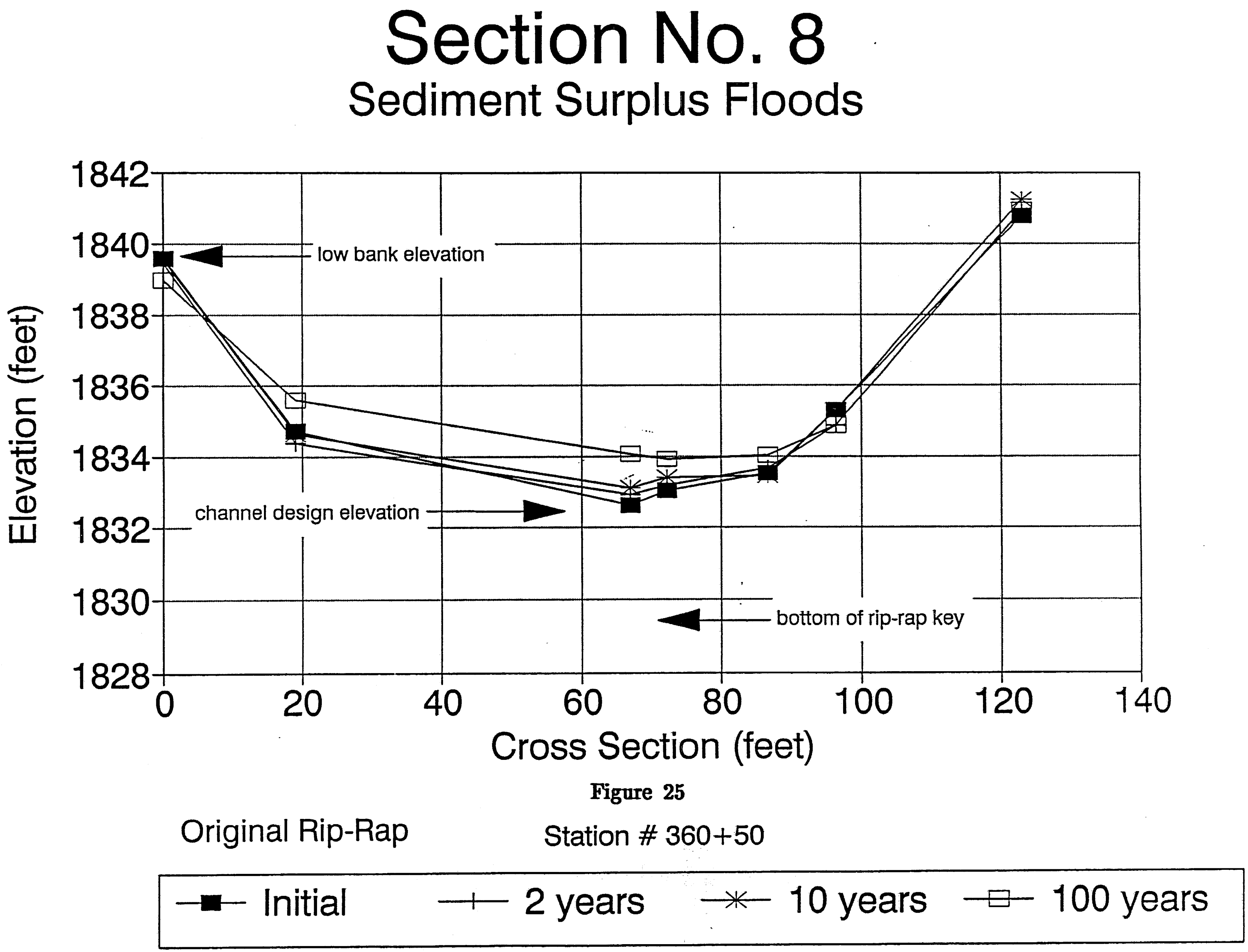

Section No. 8
Sediment Surplus Floods
1842
1840
1838
1836
1834
1832
1830
1828
Elevation (feet)
0
20
40
60
80
100
120
140
Cross Section (feet)
low bank elevation
channel design elevation
bottom of rip-rap key
Original Rip-Rap
Station # 360+50
Initial
2 years
10 years
100 years

Figure 25

Section No. 9

Sediment Surplus Floods

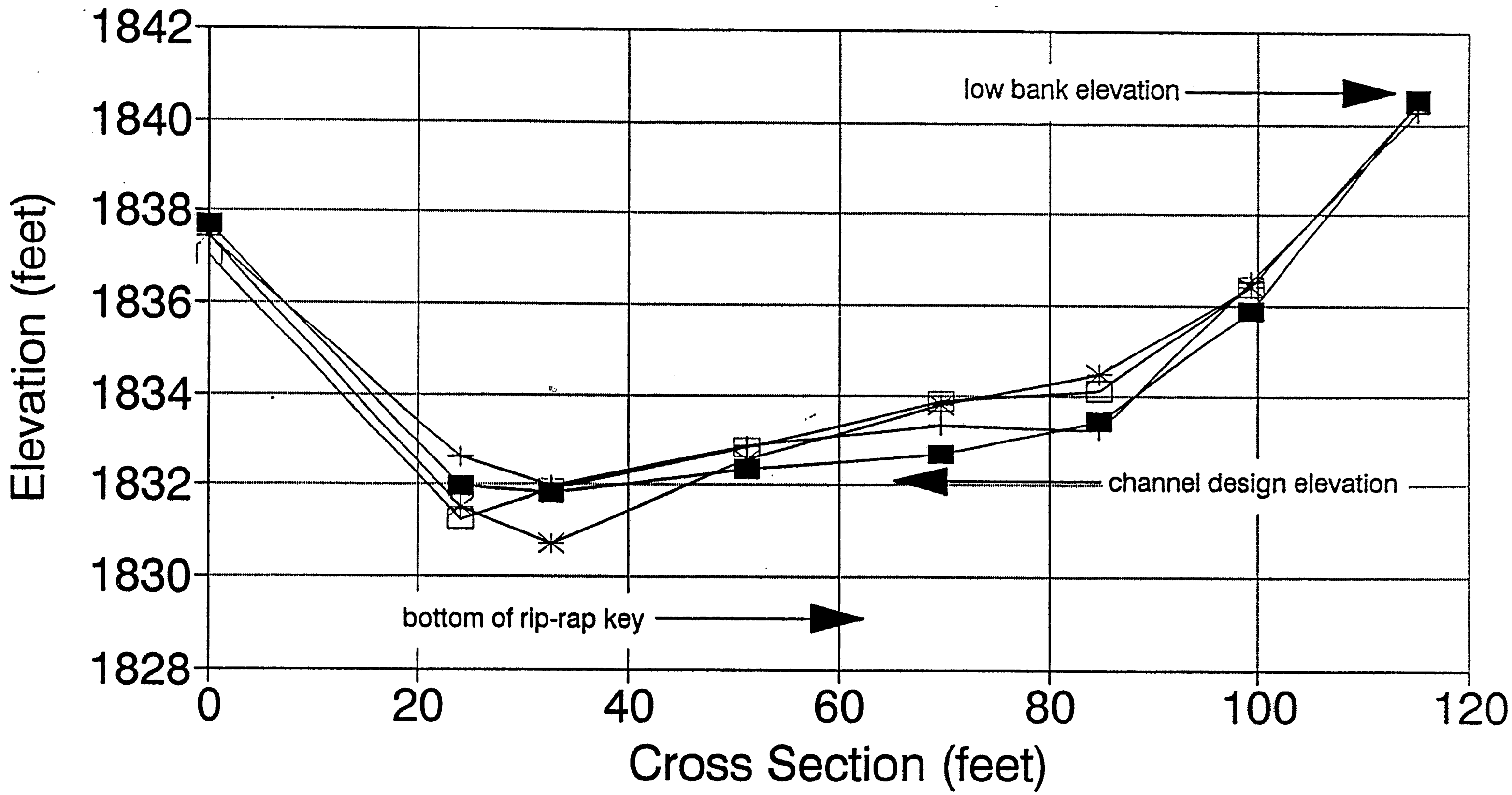


Figure 26

Original Rip-Rap

Station # 362+25

Initial — 2 years — 10 years — 100 years

Section No. 10

Sediment Surplus Floods

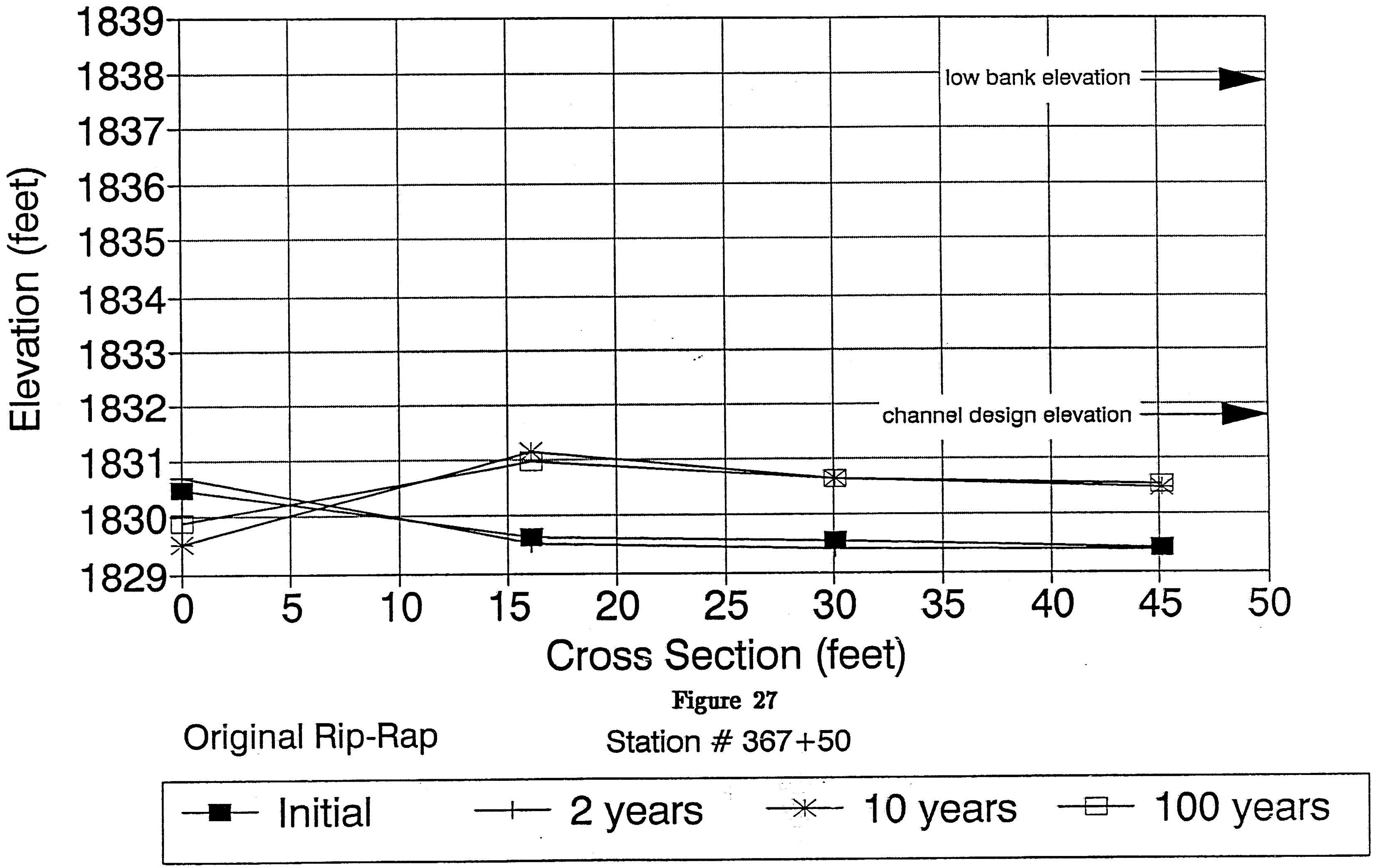


Figure 27

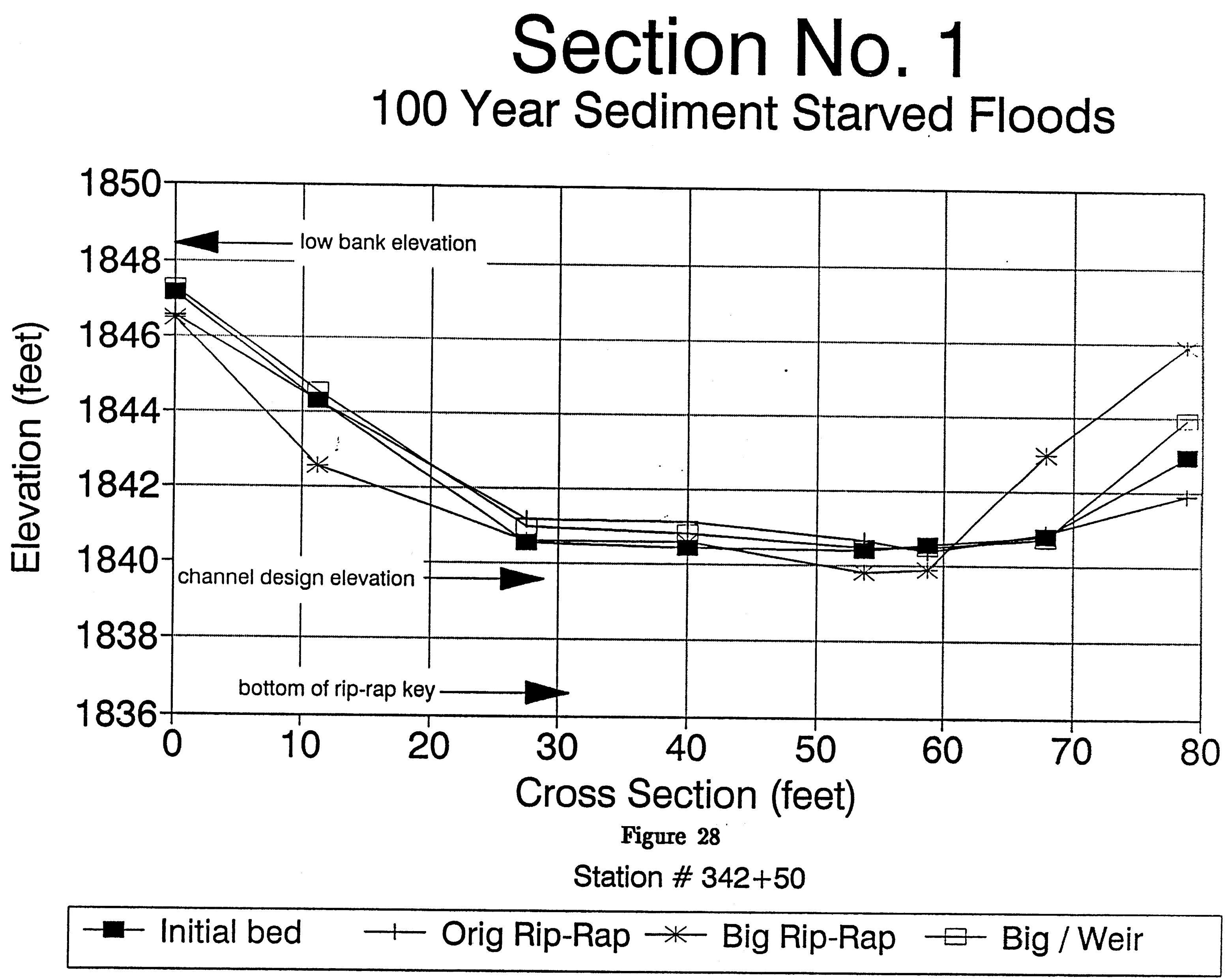


Figure 28

Station # 342+50

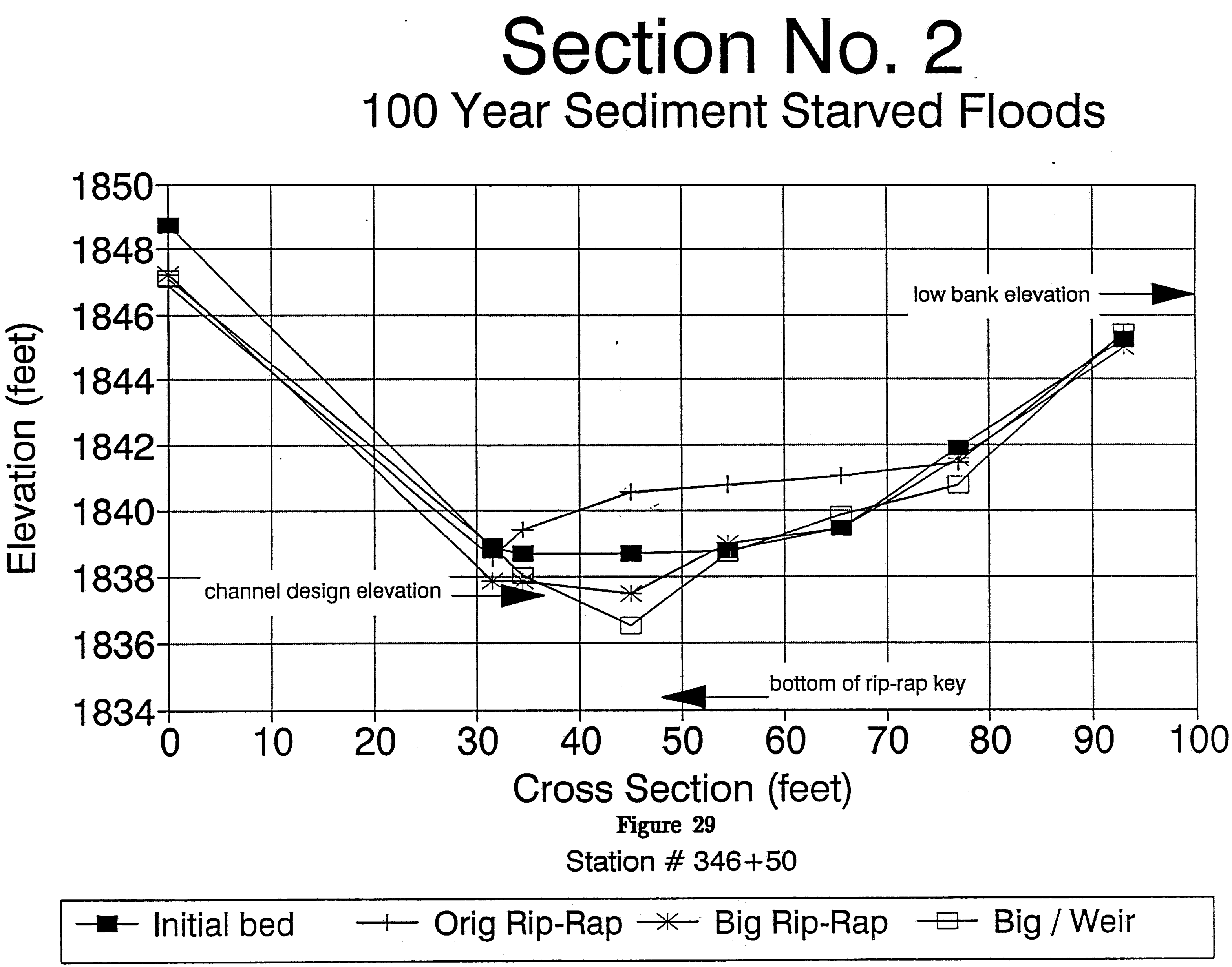


Figure 29

Station # 346+50

Section No. 3

100 Year Sediment Starved Floods

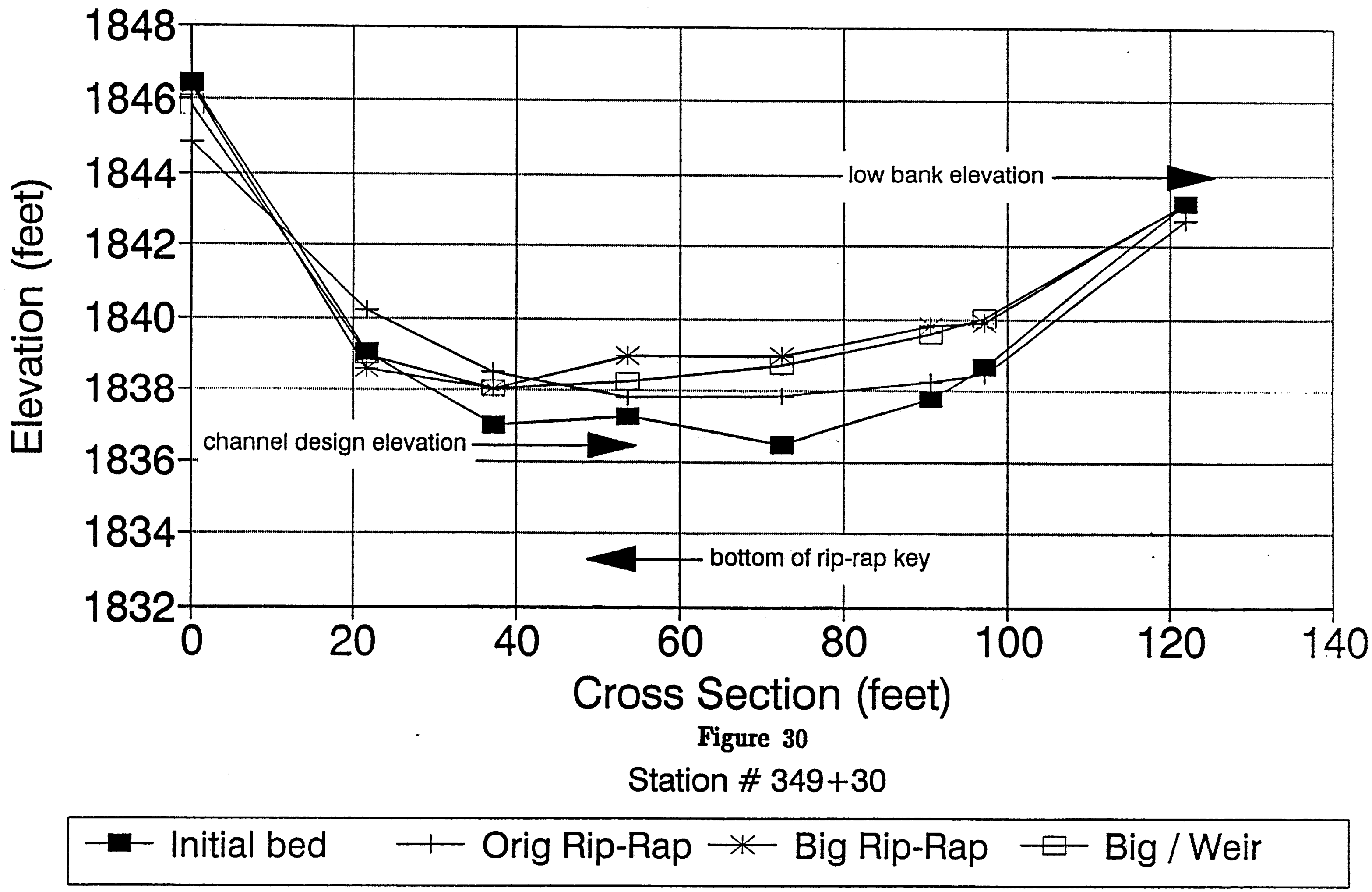


Figure 30

Station # 349+30

Section No. 4

100 Year Sediment Starved Floods

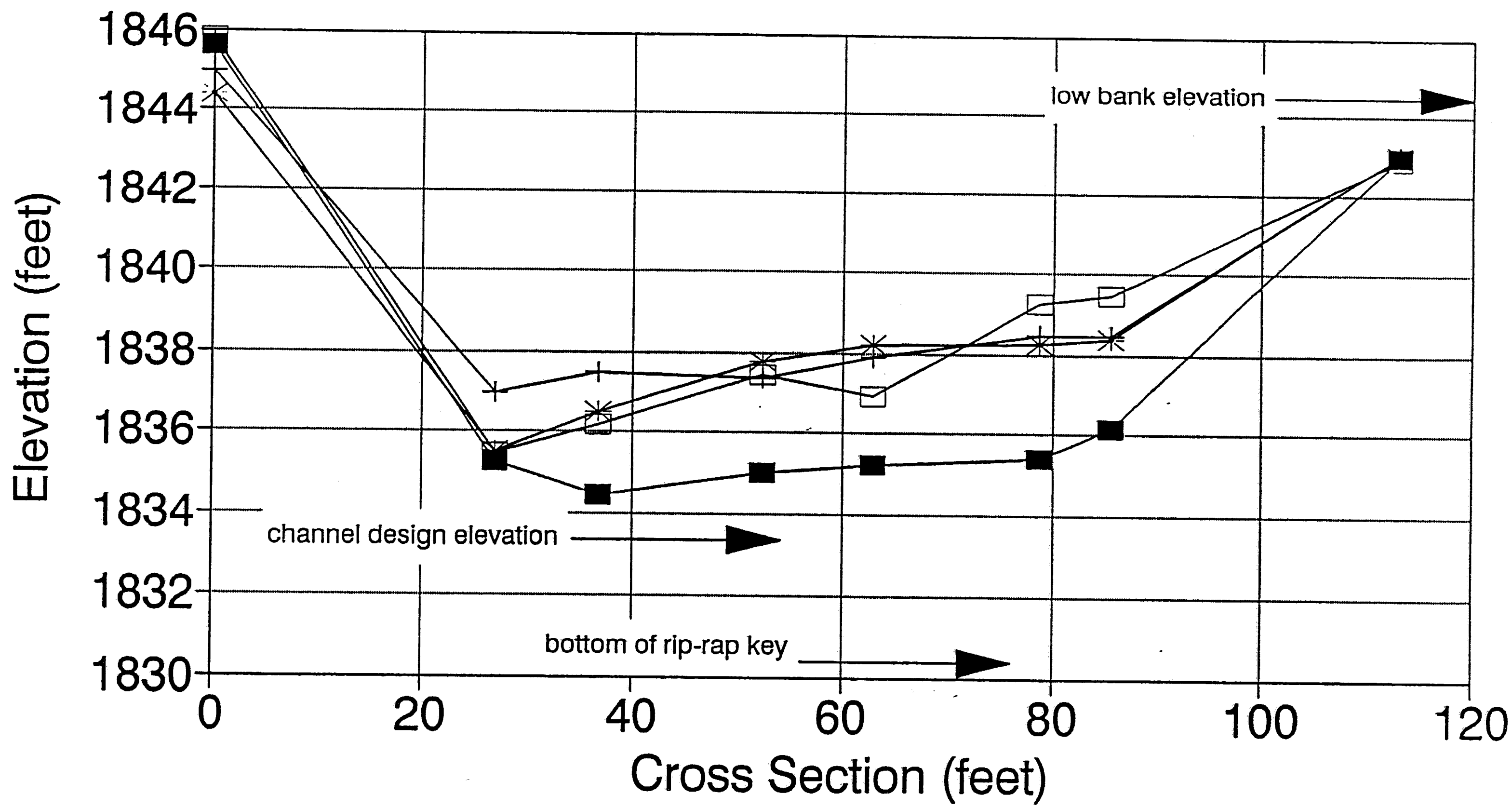


Figure 31

Station # 350+80

Initial bed — Orig Rip-Rap — Big Rip-Rap — Big / Weir

Section No. 5

100 Year Sediment Starved Floods

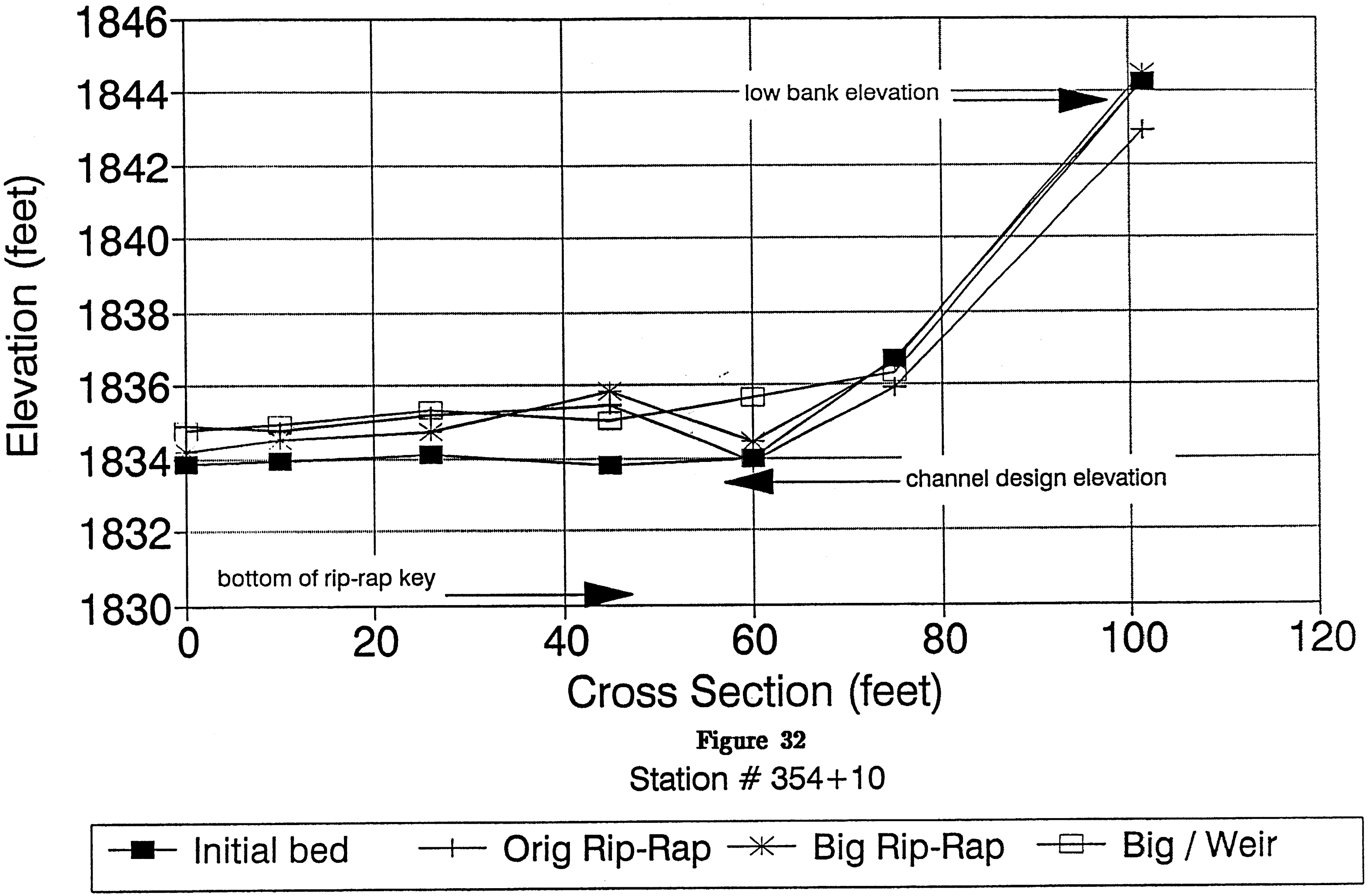


Figure 32

Station # 354+10

Section No. 6

100 Year Sediment Starved Floods

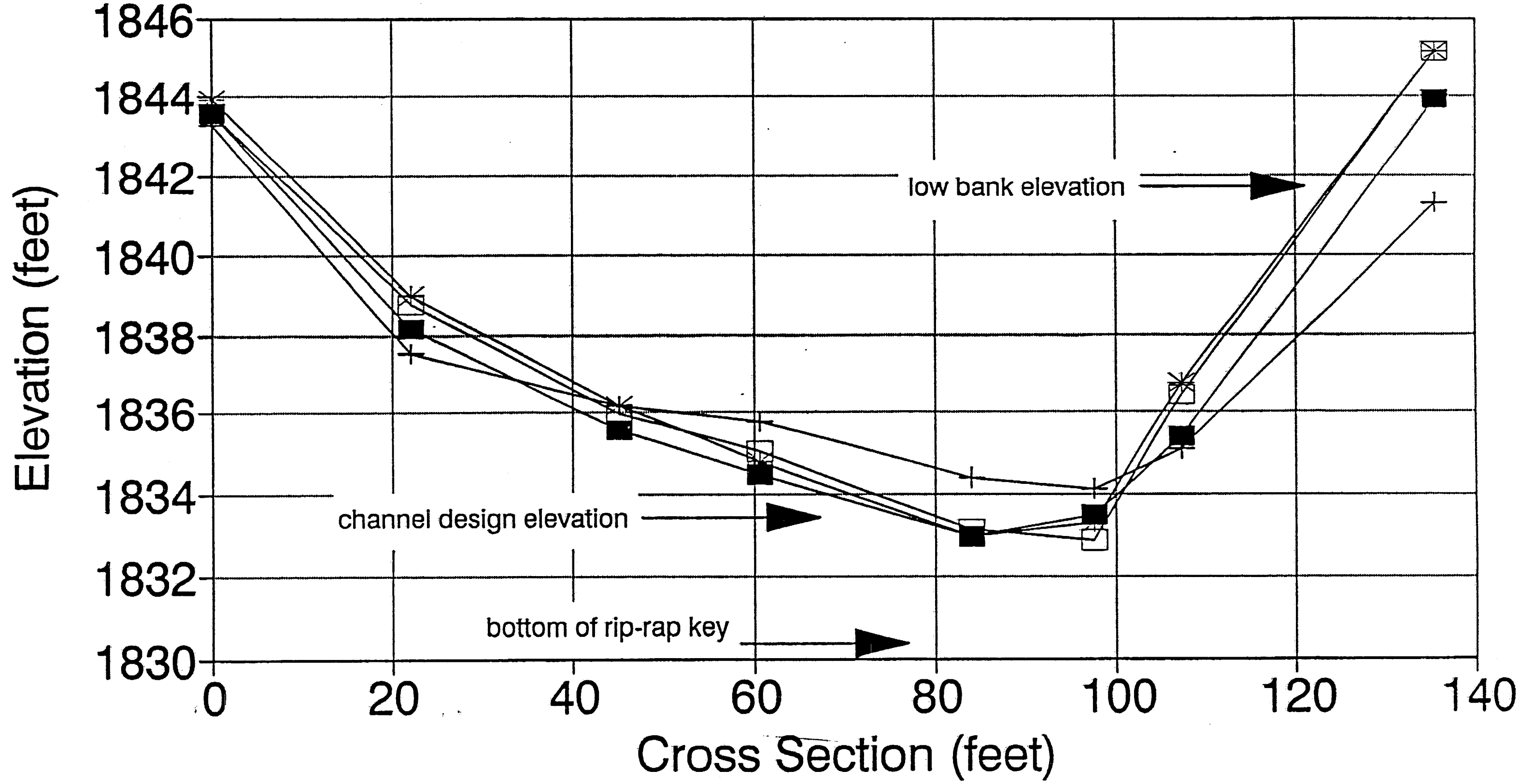


Figure 33

Station # 355+75

Initial bed — Orig Rip-Rap — Big Rip-Rap — Big / Weir

Section No. 7

100 Year Sediment Starved Floods

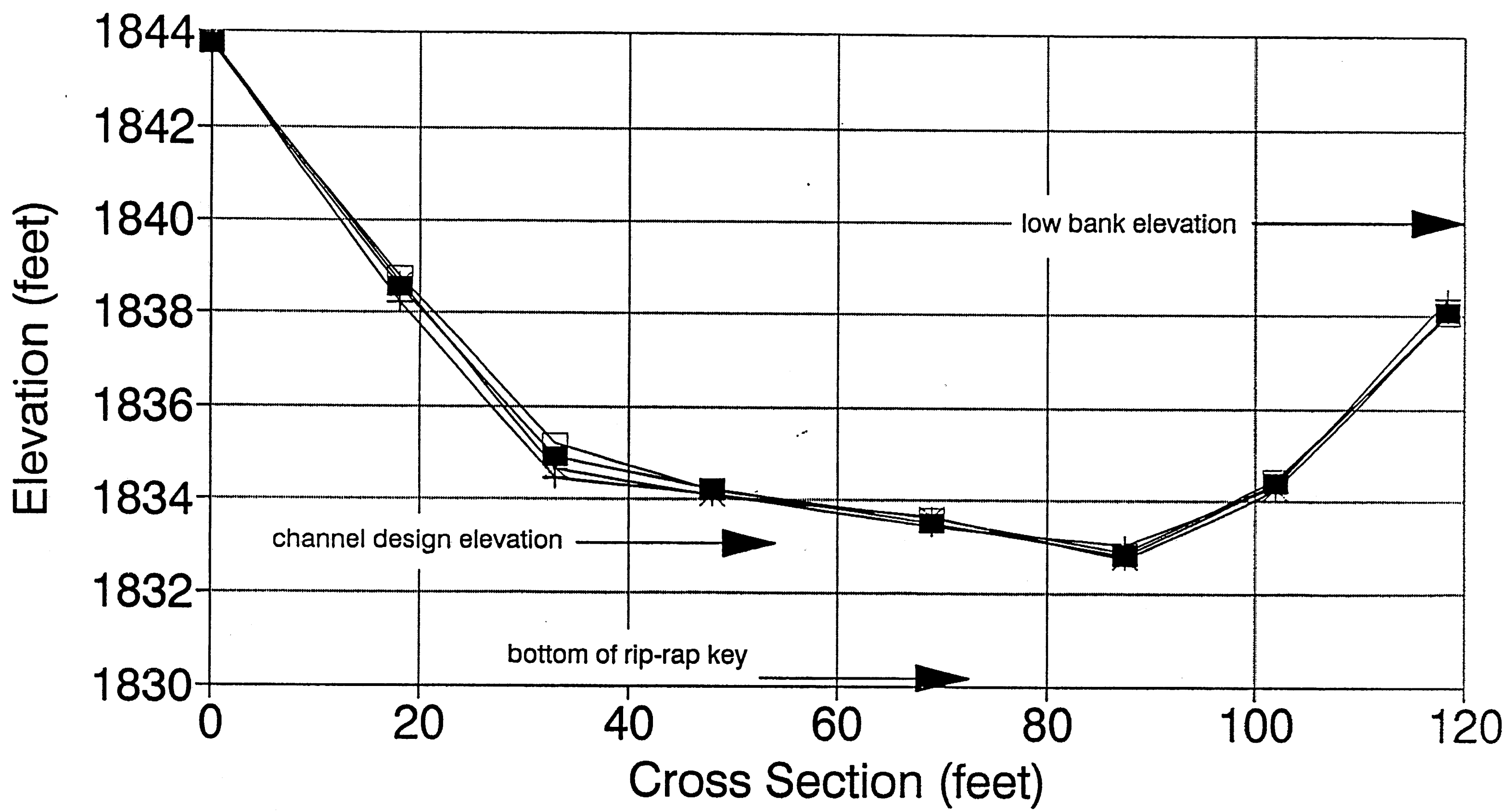


Figure 34

Station # 357+70

Section No. 8

100 Year Sediment Starved Floods

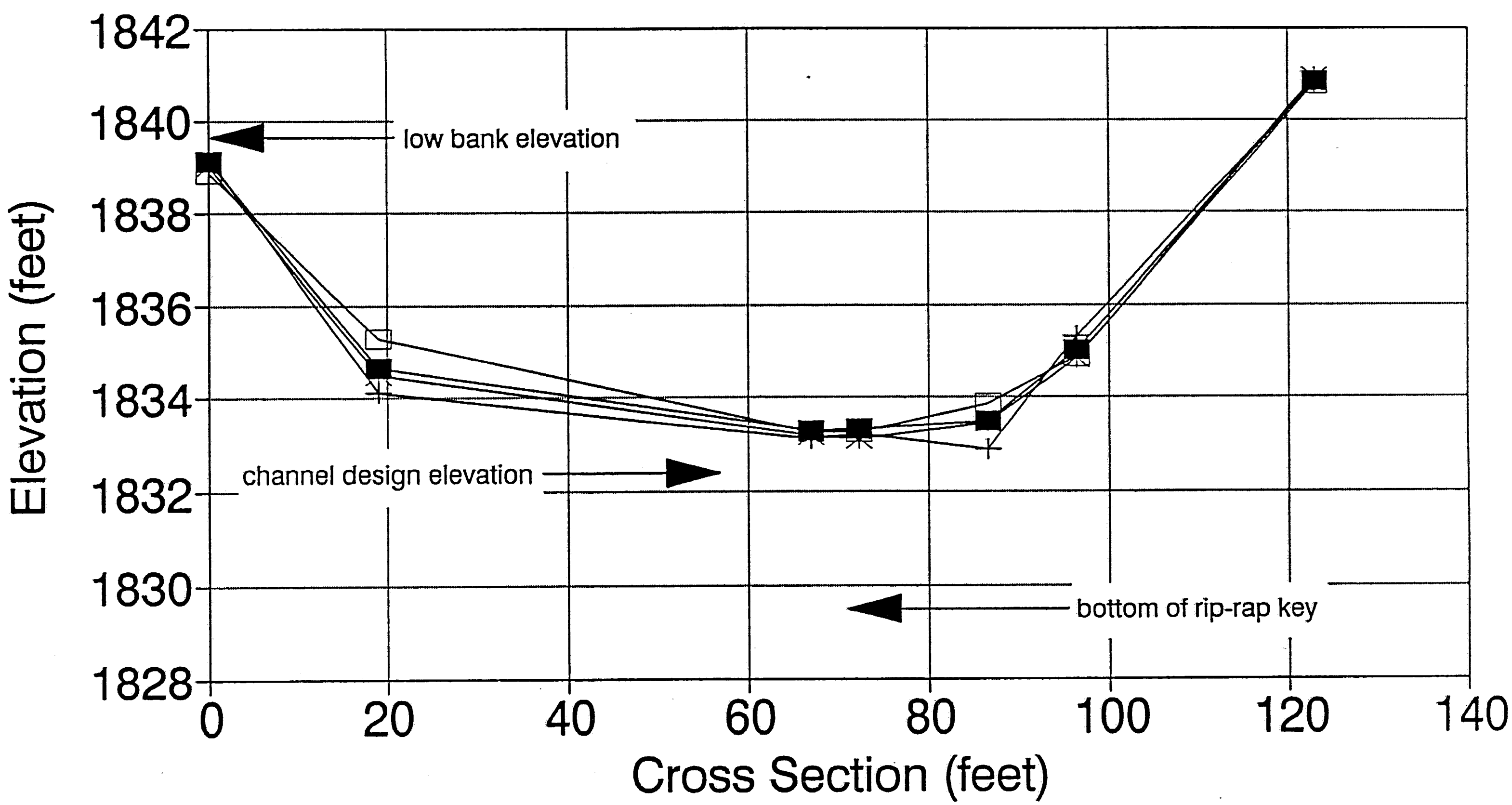


Figure 35

Station # 360+50

Section No. 9

100 Year Sediment Starved Floods

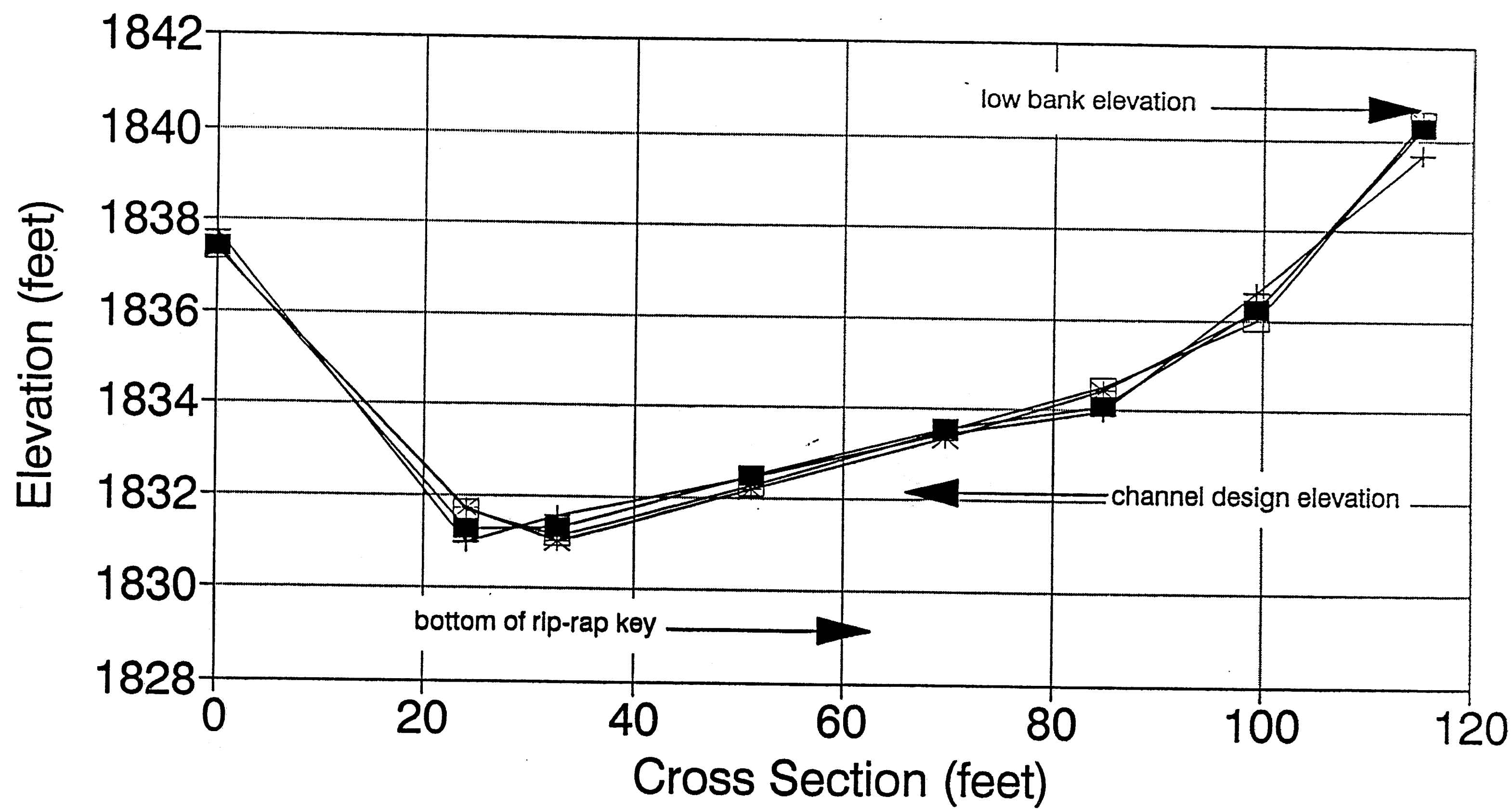


Figure 36

Station # 362+25

Initial bed — Orig Rip-Rap — Big Rip-Rap — Big / Weir

Section No. 10

100 Year Sediment Starved Floods

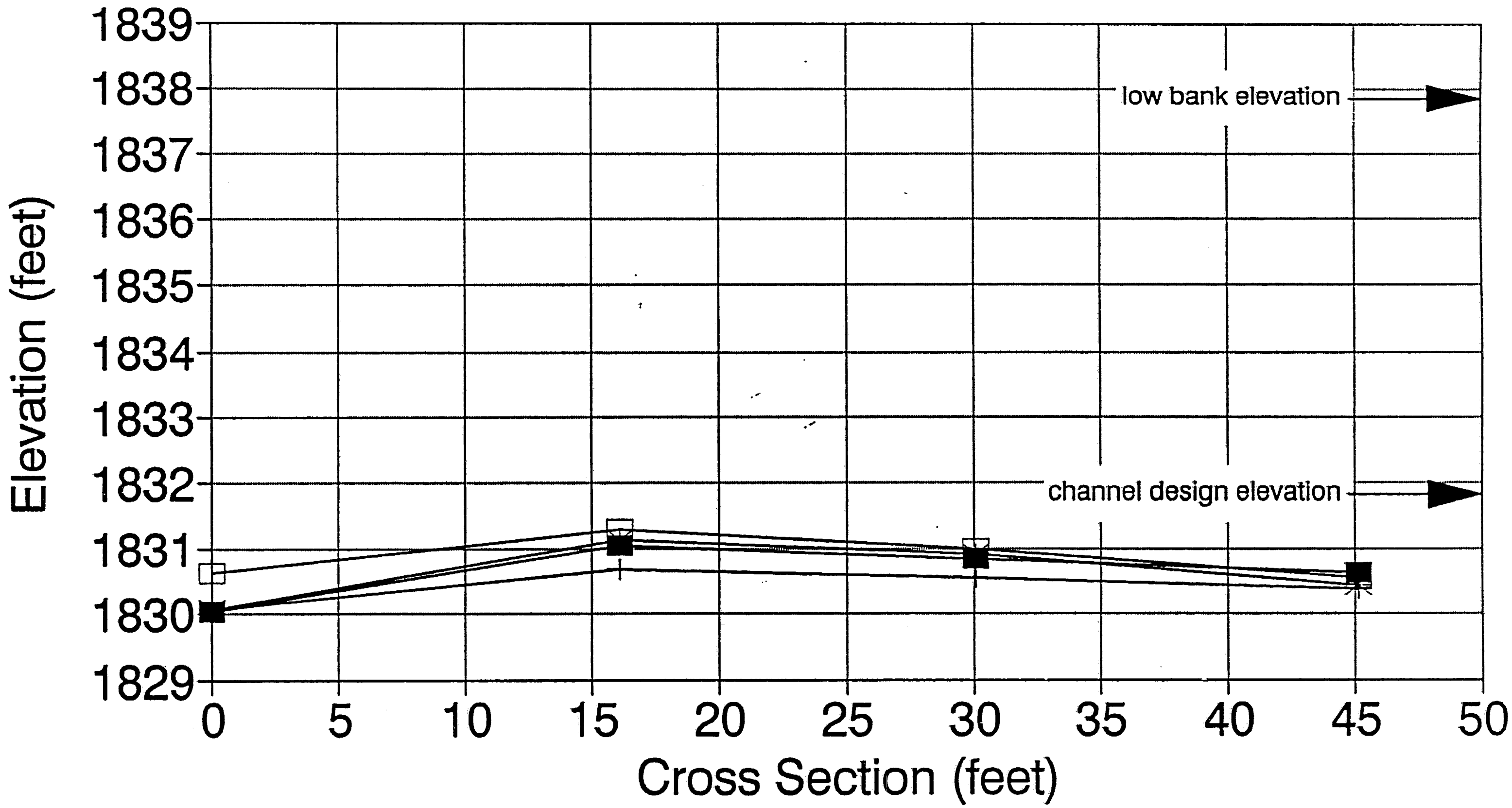


Figure 37

Station # 367+50

Initial bed — Orig Rip-Rap — Big Rip-Rap — Big / Weir

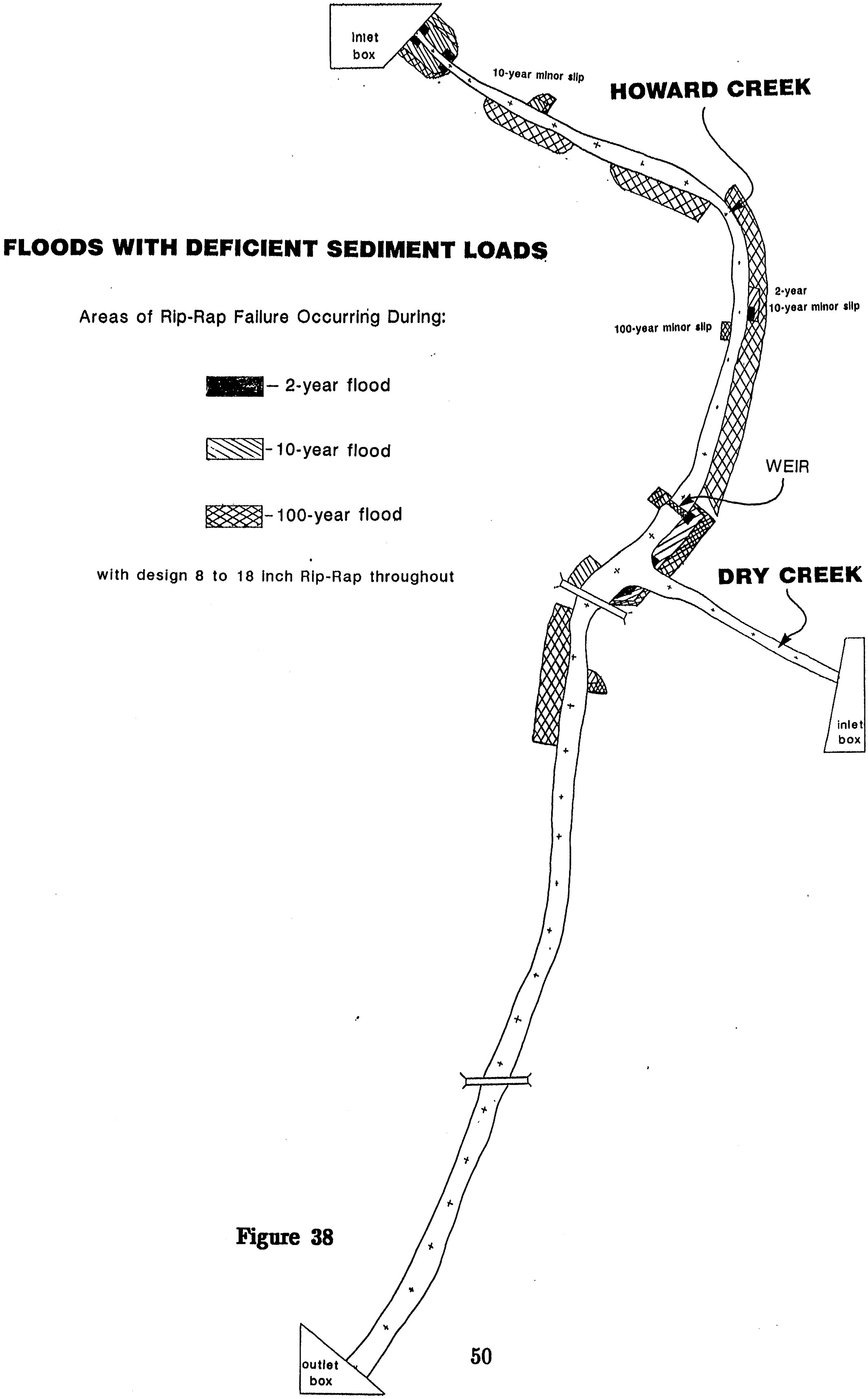

FLOODS WITH DEFICIENT SEDIMENT LOADS
Areas of Rip-Rap Failure Occurring During:
– 2-year flood
- 10-year flood
- 100-year flood
with design 8 to 18 inch Rip-Rap throughout
Inlet box
10-year minor slip
HOWARD CREEK
2-year
10-year minor slip
100-year minor slip
WEIR
DRY CREEK
inlet box
outlet box

Figure 38

As expected, the 100–year flood caused more damage, with riprap failure occurring in isolated areas upstream of the main bend and along the entire left bank from the upstream toe of the wall downstream to the fire house just past the confluence with Dry Creek. Failure also occurred along the righthand bank near Big Draft Road. The reach downstream of about 500 ft from the confluence with Dry Creek performed adequately with little raveling of the riprap. Similar levels of performance were observed with both the sediment deficient and sediment surplus test runs, with slightly worse riprap failure occurring in the case of the sediment deficient run. The reader is again referred to Figures 8 through 27 and 38 and 39. With this in mind, the 100–year sediment deficient run was chosen as the critical test condition to be used in further evaluations.

Upon completion of the first series of test runs (1–6) the bed was reworked to approximate the design bed configuration, and the 8–18" riprap was replaced with 12–30" material in zones of previous failure. The model was then retested (Run 7) for the critical conditions. The revised riprap performed well with the only raveling observed in the short reach between the entrance of Dry Creek into Howard Creek and the Big Draft Road bridge (Photos 3, 4, and 5). This failure may, as mentioned earlier, have been due to model phenomena caused by either the limitation of scour depth as the local scour reached. the fixed bed, or the extremely steep side slopes associated with the transition to the vertical wall below the bridge (a reminder that the model is distorted 1.5V to 1.0H).

For the last run, i.e. run 8, the height of the weir upstream of the confluence with Dry Creek was reduced from 2 ft to 1 ft. Testing was again performed with the 100–year sediment deficient condition. In addition, fish habitat boulders and rock clusters were placed in the two selected reaches as shown on Figure 4, which also shows the data measurement cross sections. Placement of the boulders and clusters was based according to Appendix B. Channel performance differed little from that observed during evaluation of the two–foot weir; the scour hole depth downstream from the weir was similar to that observed with the two–foot weir. Photos 6 and 7 taken before and after testing show the general performance of the boulders and rock clusters in the downstream reach. There was some shifting in the upstream reach, particularly with the smaller rocks subject to movement, while the downstream reach performed well. It was also observed that the clusters in the upstream reach were covered with sediment during the peak discharges, even during sediment deficient conditions, but reappeared near the end of the 100–year hydrograph. The boulders and rock clusters on the stream bed did not induce failure of adjacent bank riprap.

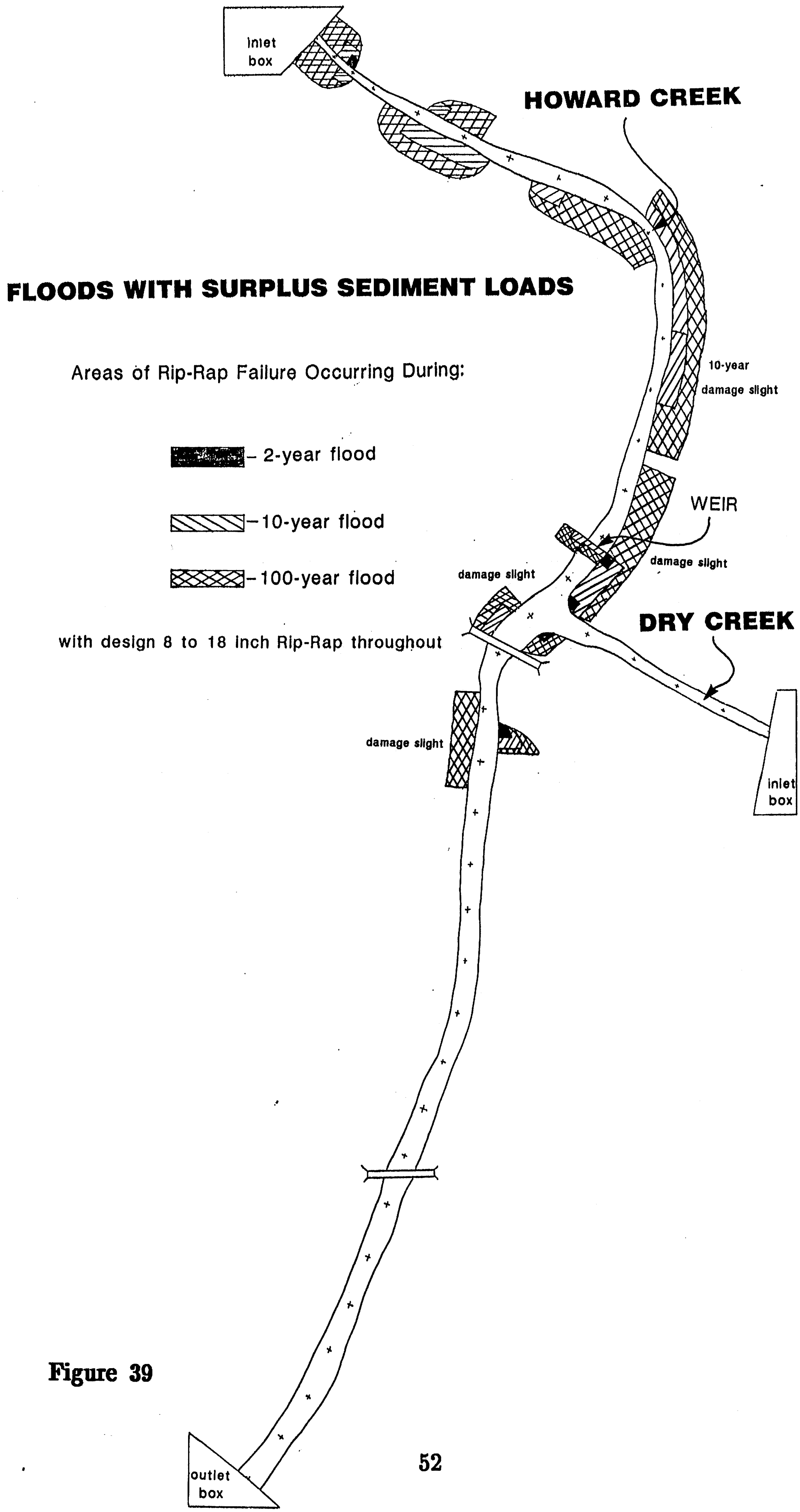

FLOODS WITH SURPLUS SEDIMENT LOADS
Areas of Rip-Rap Failure Occurring During:
– 2-year flood
–10-year flood
–100-year flood
with design 8 to 18 inch Rip-Rap throughout
inlet box
HOWARD CREEK
10-year
damage slight
WEIR
damage slight
damage slight
DRY CREEK
damage slight
inlet box
outlet box

Figure 39

7. Conclusions

The model performed as desired, showing areas subject to riprap failure and the performance of modified riprap. The bed topography repeatedly produced values close to those observed in the field, lending additional confidence to the performance of the model. A modified riprap schedule is shown on Figure 40. Zones of large (12–30 inch) riprap are given, using stationing estimated from the model. In general, the combination of riprap scheduling shown in figure 40 withstood the 100–year flood without significant failure. The effect of the weir height appeared minimal, and the use of the two–foot weir versus the one–foot weir is not critical to overall channel performance.

Suggested riprap modifications are further described by Parker (1993)[2] in his diagnostic study of Howard Creek. Neill (1973)[3] recommends that "the thickness of the stone layer measured normal to the slope should be at least as great as the long dimension of the largest stones in the specified grading." He recommends at least two layers of overlapping stones "so that slight loss of material does not cause massive failure." With this in mind, the 24" thickness originally recommended for the 6" to 18" riprap appears adequate. Where the 18" to 30" riprap is to be used, however, a placement thickness of 35" to 45" is more advisable. This extra thickness should not be attained at the expense of narrowing the resulting channel, but rather by somewhat increased bank excavation.

Figure 41a and 41b illustrate the typical sections for riprap placement. It is expected that with the incorporation of the recommended riprap modifications, the channel will perform as presently designed. It is not realistic to expect a self–formed channel to exactly adhere to design gradelines. As a result, it is necessary to anticipate some local variation within general overall conformance and design, accordingly. The sediment transport rates selected for evaluation during the study are based on very limited field information and subsequent prediction of channel maintenance is difficult. The channel may require occasional dredging in the future after a particular storm event. Fortunately, sediment transport rates on Howard Creek appear to be rather low. The results of the model study did not indicate any serious problems occurring between the Greenbrier Avenue bridge and the Greenbrier Hotel property. This conclusion is somewhat tentative because of tailgate effects at the extreme downstream end of the model.

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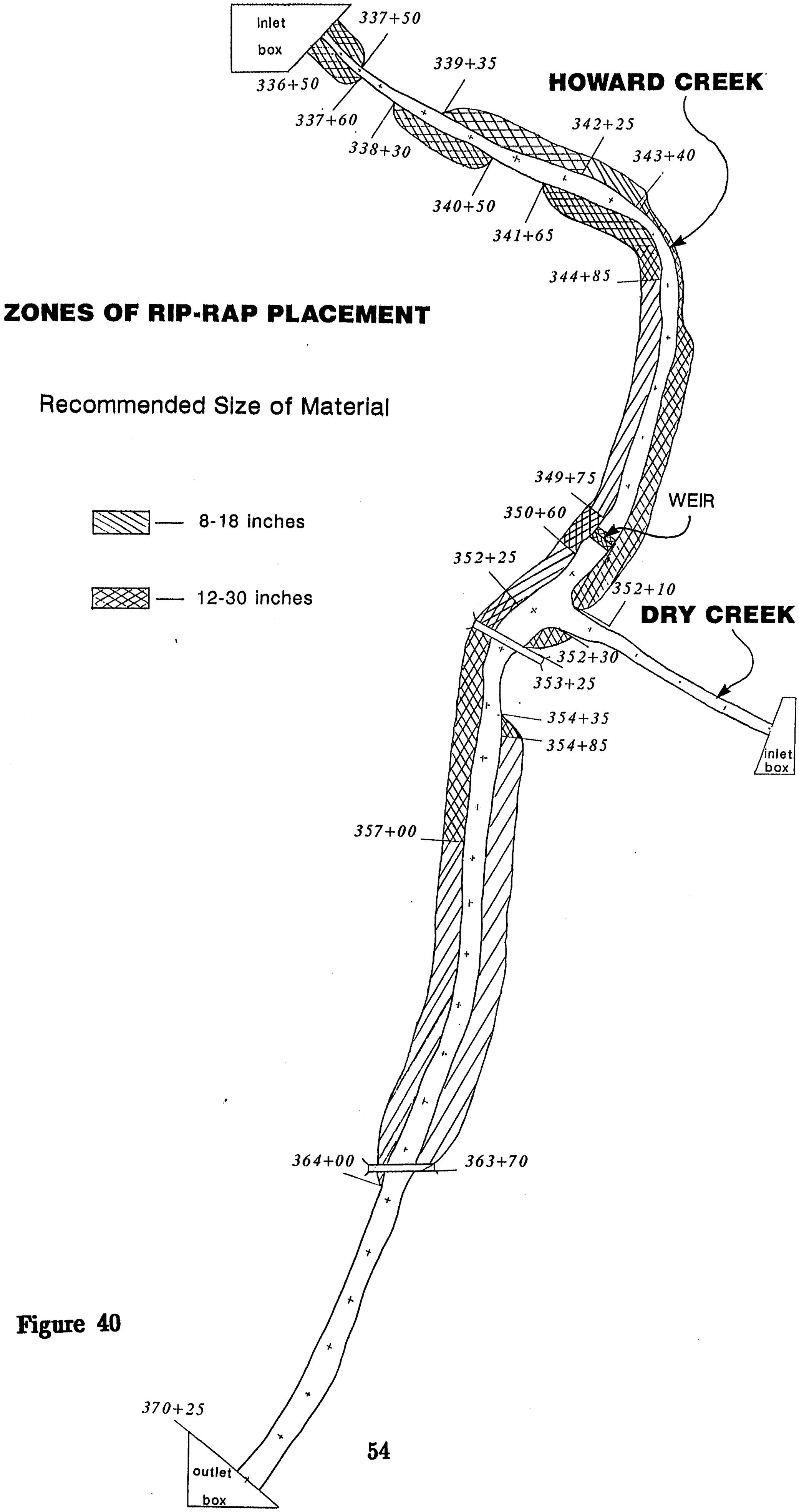
[2]Parker , Gary, 1993. Howard Creek Diagnostic Study, White Sulphur Springs, West Virginia.

[3]Neill, C.R., ed. 1973. *Guide to Bridge Hydraulics,* prepared by Project Common Bridge Hydraulics, Roads, and Transportation, Assoc. of Canada.



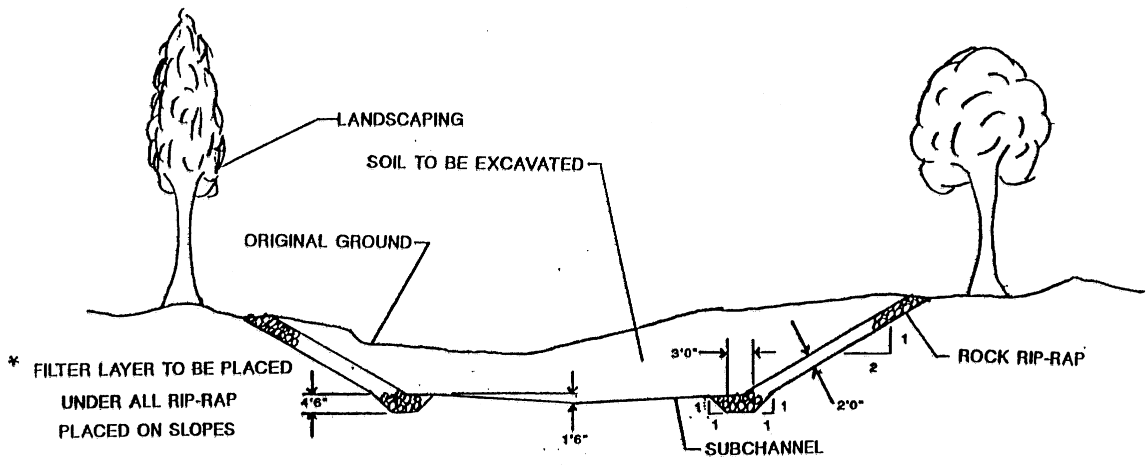
ZONES OF RIP-RAP PLACEMENT
Recommended Size of Material
8-18 inches
12-30 inches
HOWARD CREEK
DRY CREEK
WEIR
inlet box
inlet box
outlet box
336+50
337+50
337+60
338+30
339+35
340+50
341+65
342+25
343+40
344+85
349+75
350+60
352+25
352+10
352+30
353+25
354+35
354+85
357+00
364+00
363+70
370+25

Figure 40

TYPICAL SECTION CHANNEL WORK

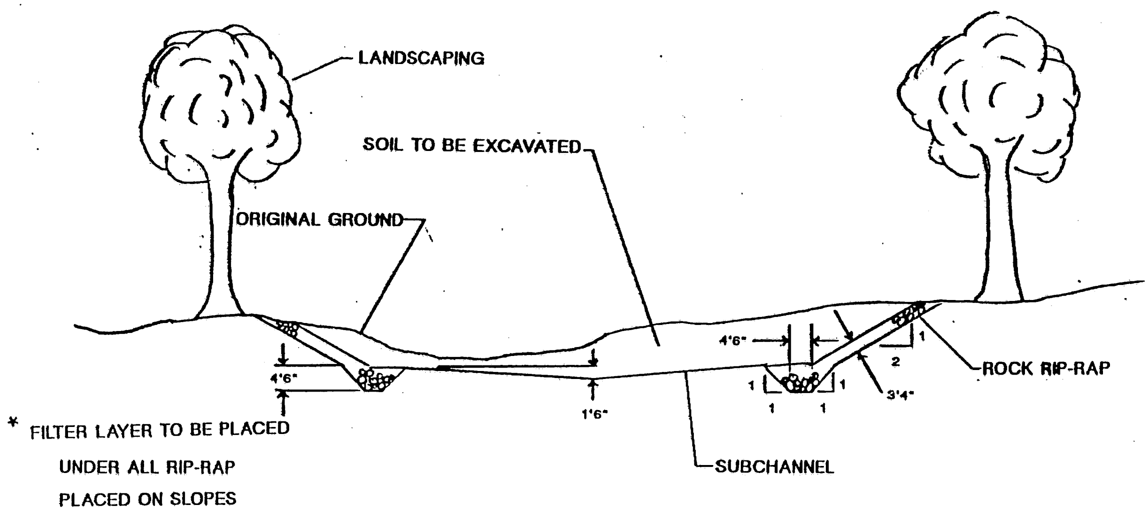
TYPE I RIP-RAP

8-18 INCHES



TYPE II RIP-RAP

12-30 INCHES



TENTATIVE DESIGN MODIFICATION

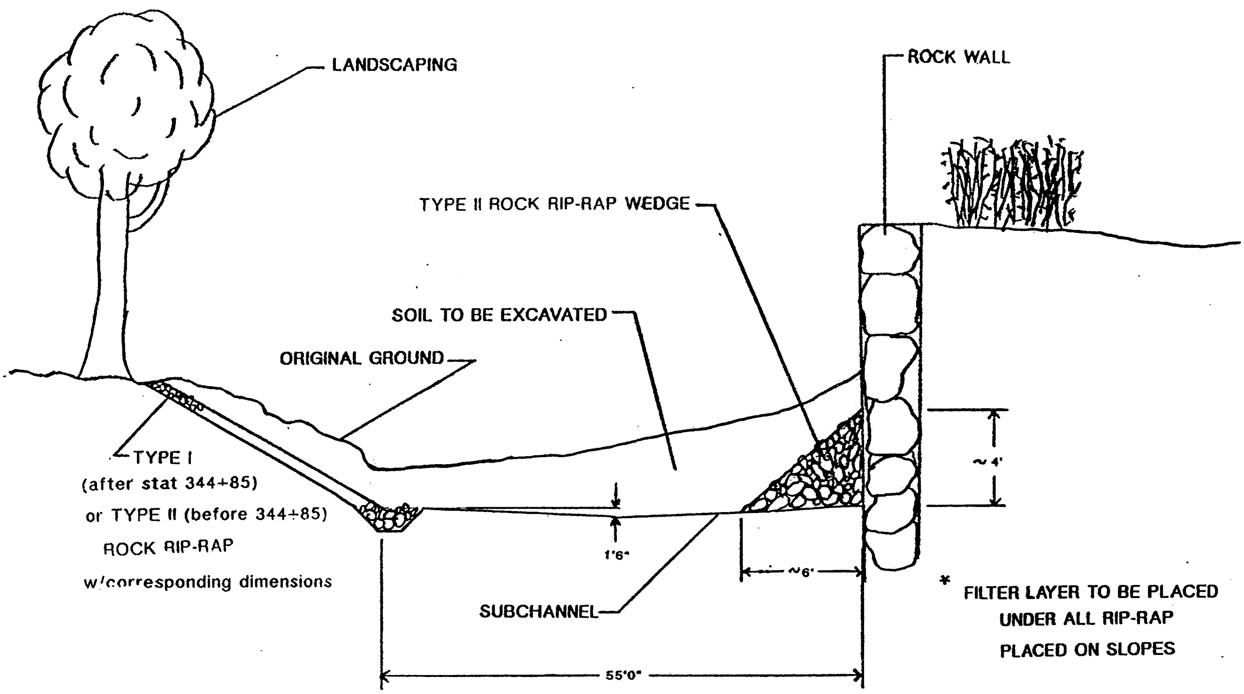
(FINAL DESIGN WILL BE BY SCS)

TYPICAL SECTION CHANNEL WORK

TYPE II RIP-RAP

ROCK WALL

STATION #343+40 TO #345+80



TENTATIVE DESIGN MODIFICATION

(FINAL DESIGN WILL BE BY SCS)

The fish habitat rocks performed better in the flatter, downstream reach. Before being buried in sediment, some of the smaller rocks used in clusters appeared to be close to becoming mobilized during the 100–year hydrograph, while the larger individual rocks remained stable. This may provide some guideline when selecting appropriate sizes for the various reaches.

It is anticipated that this report will contain information regarding proposed modifications which are of value to the Soil Conservation Service and its engineers in their final design of the project.

Acknowledgements

The authors would like to thank Mr. Wes Morrow and his associates at the Soil Conservation Service offices in Morgantown, West Virginia. In addition, the authors would like to thank Messrs. James Stingel and Tom Iivari, whom along with Mr. Morrow provided many helpful comments during a visit to observe the model in operation. The authors would also like to thank the shop staff, who constructed the model, and Ms. Pat Swanson for editing this report.

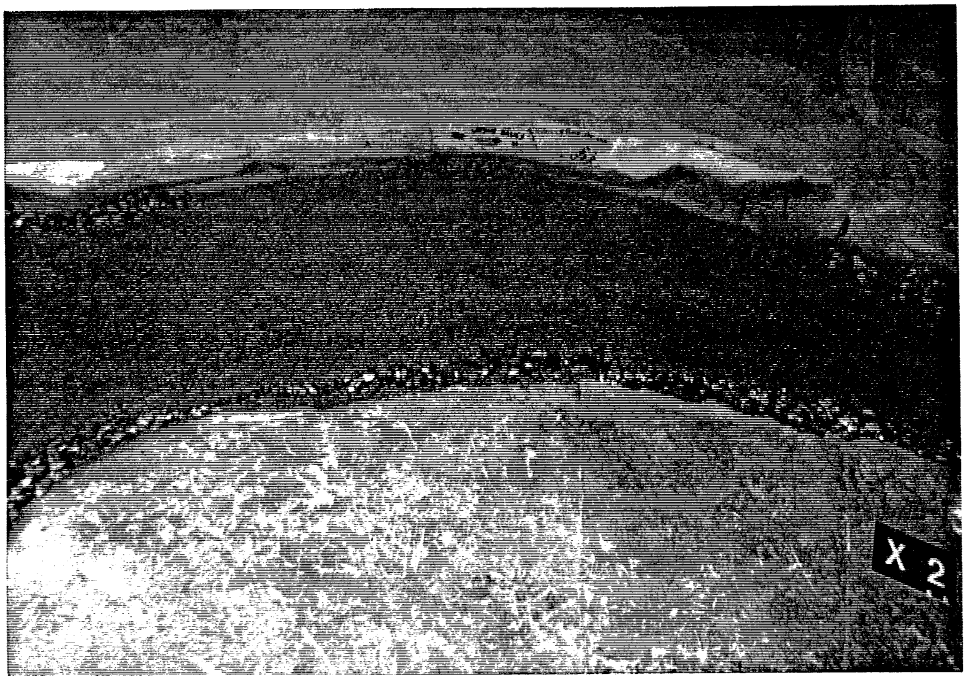


Photo 1.

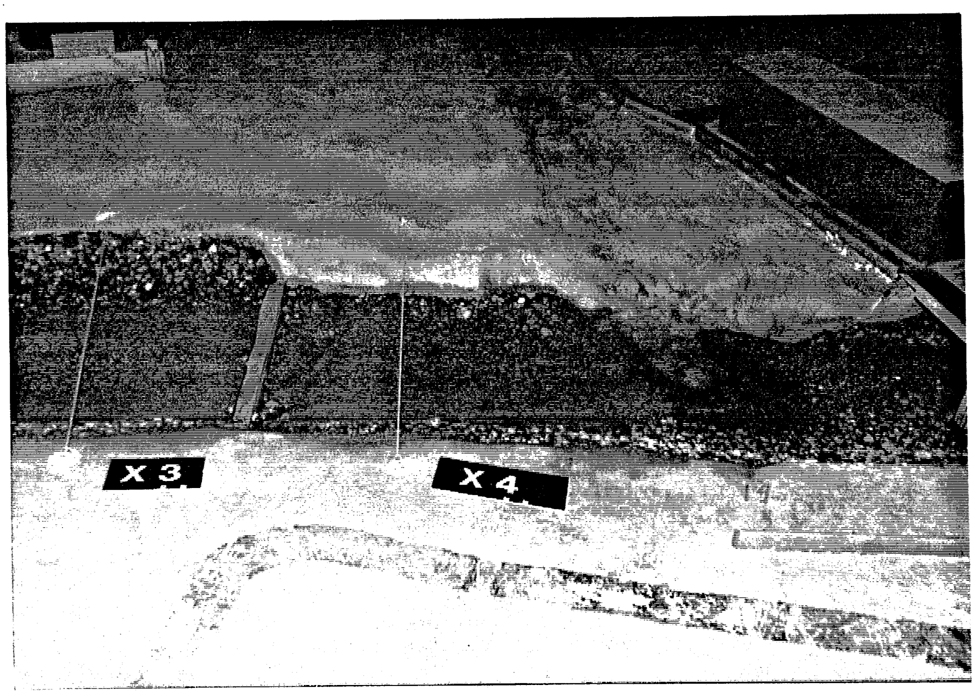


Photo 2

Main bend and weir to fire house reaches following a 10–year sediment surplus run using Type I (8–18") riprap.

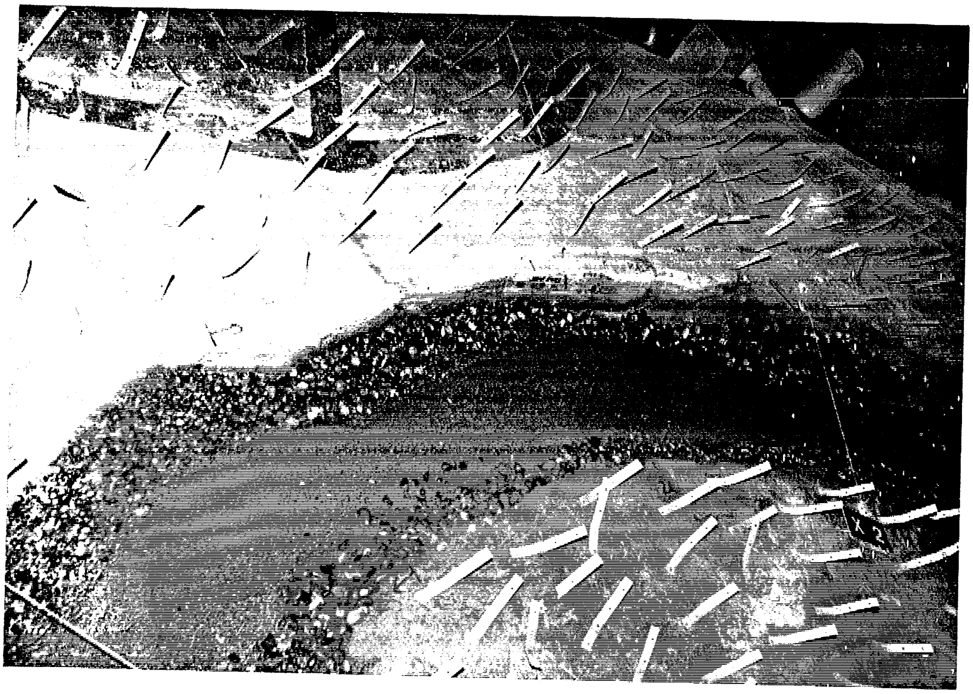


Photo 3

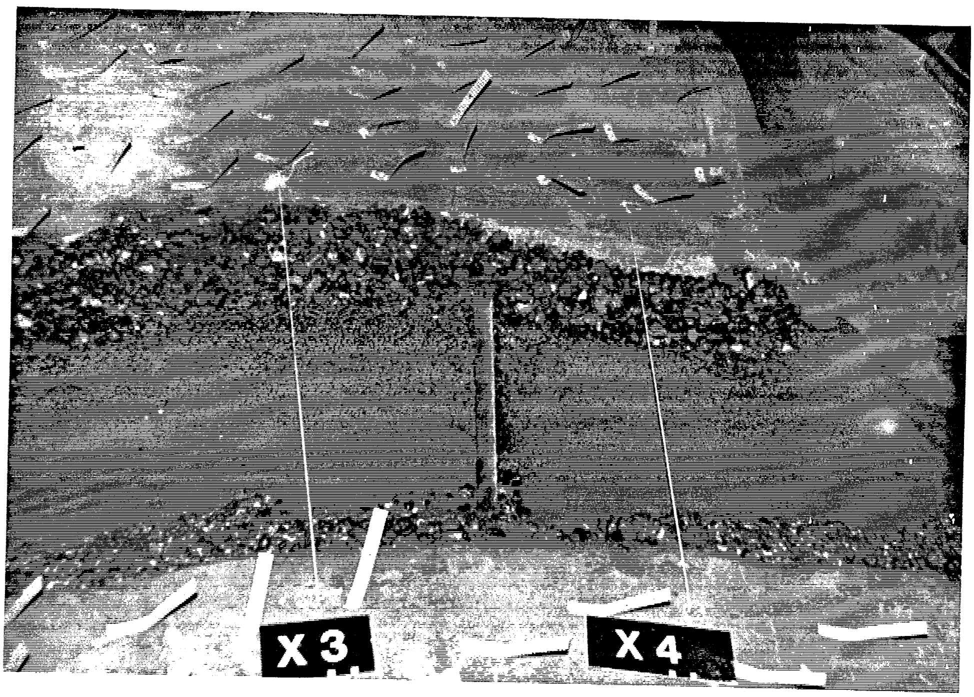


Photo 4

Main bend and weir to Dry Creek reaches following a 100–year sediment starved run using Type II (12–30") riprap.

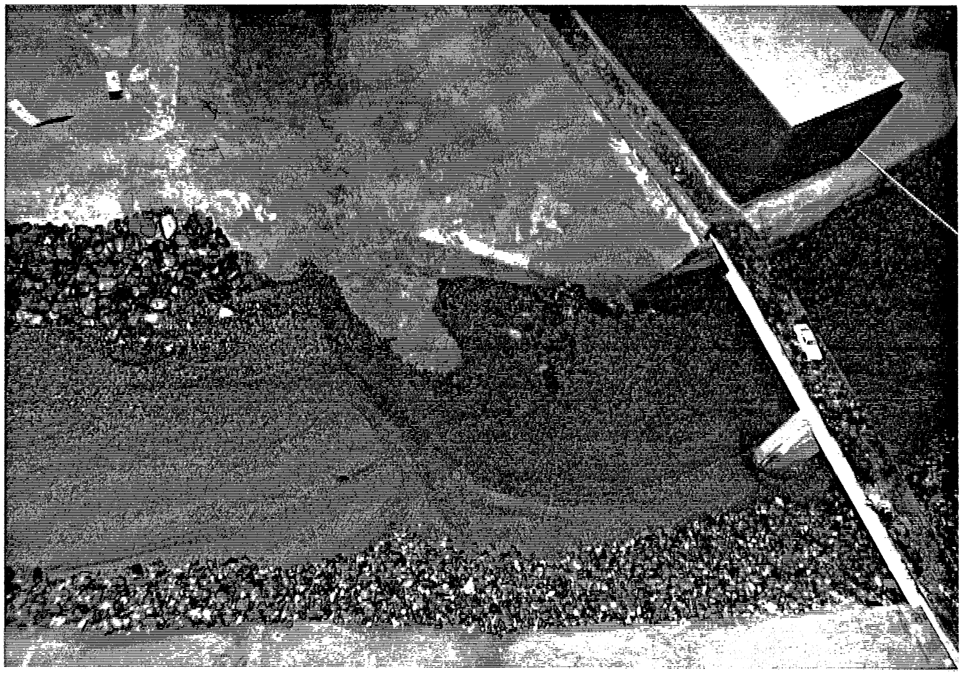


Photo 5

Localized failure at juncture of Dry Creek following a 100–year sediment starved run using Type III (12–30") riprap.

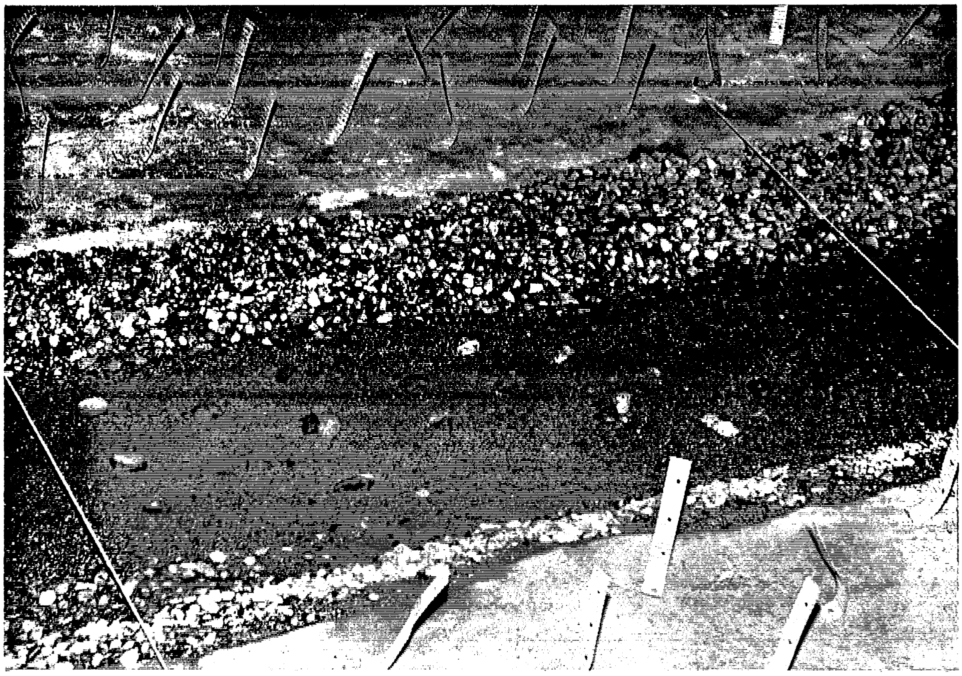


Photo 6

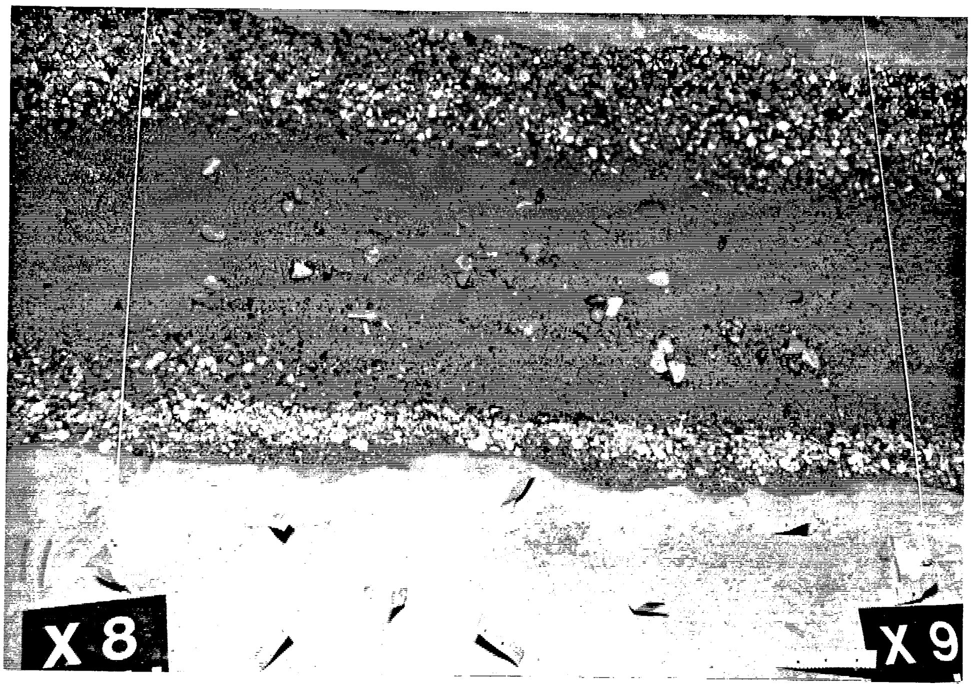


Figure 7

"Fish rocks" before (6) and after (7) for a 100–year sediment starved run.

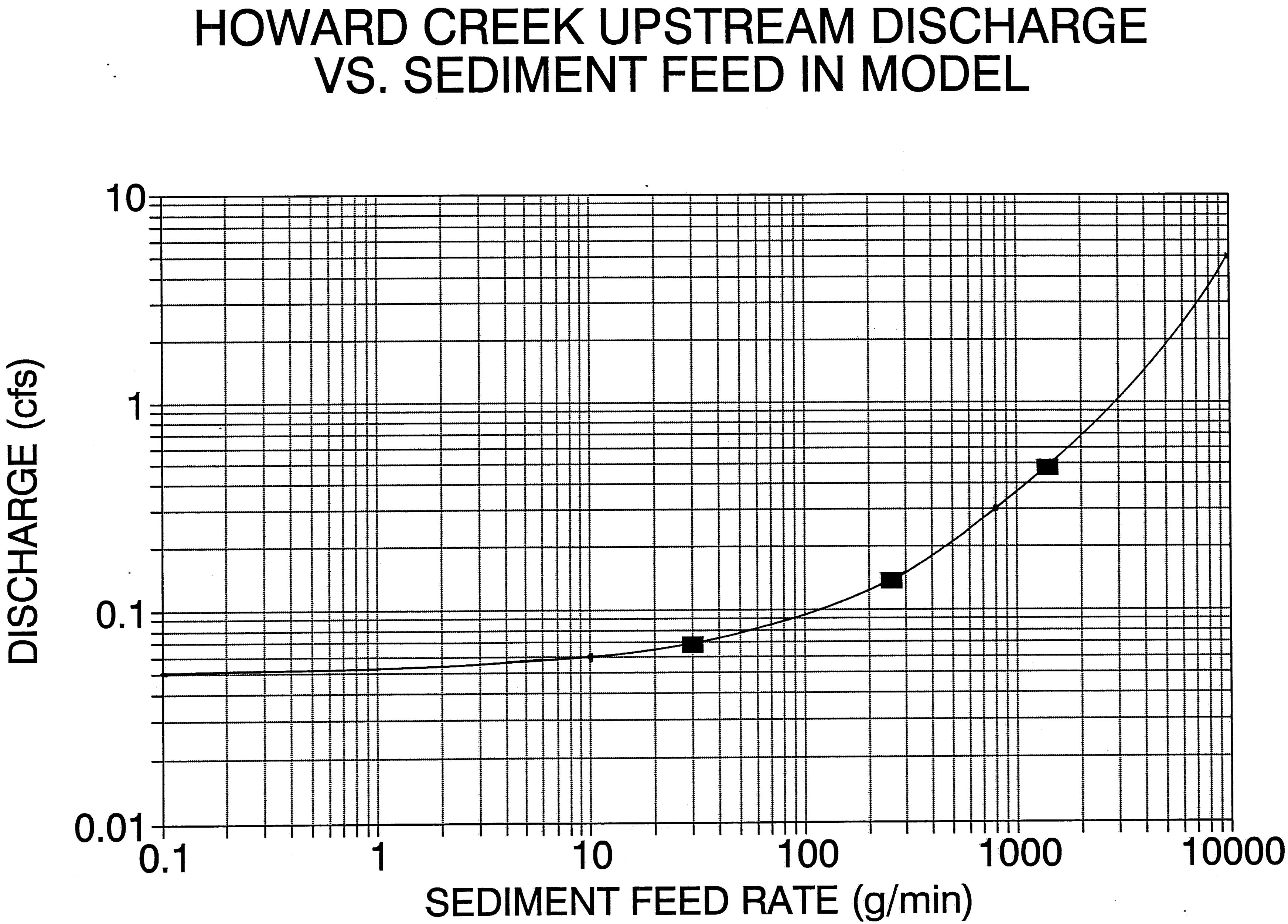

HOWARD CREEK UPSTREAM DISCHARGE
VS. SEDIMENT FEED IN MODEL
DISCHARGE (cfs)
10
1
0.1
0.01
0.1
1
10
100
1000
10000
SEDIMENT FEED RATE (g/min)

Figure A–1