SPAWNING GRAVEL FLUSHING DURING TRIAL RESERVOIR RELEASES ON THE TRINITY RIVER: FIELD OBSERVATIONS AND RECOMMENDATIONS FOR SEDIMENT MAINTENANCE FLUSHING FLOWS

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1. ABSTRACT

Runoff from the uppermost 720 mi^2 (1860 km^2) of the Trinity River basin was impounded by Trinity Dam (and its re-regulating reservoir, Lewiston Dam) beginning in 1961, as part of the US Bureau of Reclamation Central Valley project. Beginning in 1963, about 75 percent of the average natural runoff from the upper basin has been exported from the Trinity River basin to the Sacramento River basin. One effect of the reduced flow regime has been encroachment of riparian vegetation within the pre-dam active channel. Once established, this vegetation traps fine-grained sediment during high flow events, causing deposition of steep banks along a narrow low flow channel. The width of much of the active channel of the Trinity River is now 20 to 60% of its pre-dam condition along a 15 mile (24 km) reach downstream of Lewiston Dam. A second effect of the reduced flow regime of the river, in combination with high sediment yields from tributary watersheds, is the deposition of tributary-derived sediment within the stream channel, which has filled pools, buried cobble substrate, and infiltrated spawning gravels. Both the morphologic and sedimentologic changes in the post-dam channel represent a degraded aquatic habitat for anadromous salmonids, which were formerly abundant in this reach.

Since 1975, the Trinity River Basin Fish and Wildlife Task Force has been engaged in a Comprehensive Action Plan to arrive at alternatives to restore fish habitat. Among these activities are three trial flushing flow releases made in 1991, 1992, and 1993 with the purpose of evaluating the effectiveness of high flushing releases in restoring fish habitat. The objectives of this project are to document the effectiveness of these releases in removing fine-grained sediments from the channel bed and to develop recommendations for future flushing releases to clean and maintain the potential spawning gravels on the Trinity River below Lewiston Dam.

A 5.0 mi (8.0 km) reach of the Trinity River was investigated. The upstream end is the confluence of Grass Valley Creek, a tributary located 7.9 mi (12.7 km) downstream of Lewiston Dam and the major source of the sediment deposited within the bed of the Trinity River. The downstream end of the study reach is an island (a mid-channel bar in the pre-dam regime) just upstream of Steelbridge, site of a recently discontinued USGS gaging station, located 12.9 mi (20.8 km) downstream of Lewiston Dam. Most of our work focused on two representative reaches located 9.3 mi (15.0 km) and 12.9 mi (20.8 km) downstream of Lewiston Dam. Both study reaches contain spawning gravels representative of those requiring flushing. Our goal at both sites is to determine the discharge necessary to entrain (initiate motion of) the spawning

gravels and permit interstitial fine sediment to be flushed from the bed. Because gravel recruitment is limited on the Trinity River, such a sediment maintenance flow should involve minimal downstream gravel transport, but sufficient gravel movement to permit flushing of subsurface fine material. Such a release is also likely to minimize water use. Thus, one of the primary objectives of our work is to determine the magnitude of a release that will just mobilize the gravel surface, thereby permitting subsurface flushing while minimizing gravel transport and water use, and maximizing sand transport at the two study sites.

We measured the sediment bed before and after the releases and made direct measurements of flow velocity and discharge during the releases. The bed observations included pebble counts and visual observations of the bed size distribution throughout both study reaches. Gravel motion was observed using tracer gravels and scour chains placed in potential spawning gravels along cross sections at both study sites. The flow observations permitted calculation of total discharge during each release and observations of local velocity acting directly on the spawning gravels. The combined observations of local flow and gravel entrainment permit the accuracy of our observations to be evaluated in terms of general sediment transport relations, thereby providing a basis for estimating entrainment and flushing at flows other than those we directly observed. Together with the measurement of discharge, these results provide the basis for specifying the discharge magnitude and duration that can produce flushing of the spawning gravels.

Two trial releases at a discharge of roughly 2800 cfs (80 m^3/s) produced negligible entrainment of the gravels. The flushing effectiveness of these discharges is limited to removal of sand from the bed surface; the bed cannot be flushed at any depth. A release of 5800 cfs (164 m^3/s) for 5 days in 1992 was just sufficient to mobilize the surface gravel layer and entrain underlying finer sediment. Although the gravel transport rate was quite small, the gravel on the bed surface was almost completely entrained over the course of the five-day peak flow: the combination of flow strength and duration was just sufficient to dislodge nearly all of the gravel grains present on the bed surface, although many of these grains moved only a small distance downstream. In the presence of a sand concentration in the river bed that is much smaller than presently found, such a release could provide flushing to a depth of 15 cm to 20 cm within the bed.

We find that the onset of gravel entrainment occurs at a value of the dimensionless shear stress $\tau^* = 0.040$ and that entrainment to an average depth of the largest grain on the bed surface occurs at $0.05 < \tau^* < 0.065$. These values are consistent with those observed in theoretical and laboratory studies and provide independent support for extrapolating our conclusions to other locations along the river. The values of bed shear stress used to calculate τ^* were obtained from local flow observations over the spawning gravels; values of shear stress calculated using section- or reach-averaged flow differed substantially from the local stress values and also varied considerably between the two study sites. The fact that the local flow values of τ^* were similar for a similar degree of bed motion at both study sites provides further support for the general applicability of our methods and results.

The entrainment and flow observations lead us to recommend that a release of 6000 cfs (170 m^3/s) for 5 days is a minimum for entraining the bed gravels of the Trinity River to achieve flushing below the bed surface. Other discharges may produce a similar degree of gravel mobilization, but will require a different release duration to achieve the same result. Because the frequency of gravel entrainment increases very rapidly with discharge, larger discharges will mobilize the gravel more efficiently. The most efficient release for gravel entrainment would be the largest possible. For example, a discharge of 8500 cfs (240 m^3/s) for one day would achieve the same degree of gravel entrainment as a discharge of 6000 cfs for five days, but would use approximately 70% less water. High discharges do not, however, provide an optimum combination of maximum sand removal and minimum gravel loss because the amount of gravel transport is very large.

The overall quantity of sand in the study reach is large. None of the trial releases produced a substantial reduction in the proportion of fine materials in the bed. In the presence of a high sand concentration, a discharge sufficient to entrain the bed gravels and flush sand at depth will not produce a markedly cleaner bed because sand will be redeposited with the gravel. To achieve successful flushing at depth, the total volume of sand in the reach must be reduced.

The rate at which sand can be removed by a flushing release may be increased by dredging pools along the reach so that they act as sediment traps. The pools act to decrease the effective reach length from which sediment must be removed by the flow. By increasing the quantity of sand that may be removed from the river with a given volume of water, dredged pools offer the potential of decreasing both the water used in a flushing release and the downstream loss of gravel.

The efficiency of a flushing release depends on the rate at which water is released, or the discharge, the volume of water used, and the extent of pool dredging. We evaluate the effectiveness of different flushing options by calculating the sand removal and downstream gravel transport for different combinations of water volume, discharge, pool dredging. To do this, we estimate the quantity of sand presently in the study reach and develop relations between water discharge and sand transport, gravel transport, and pool trapping. These are then used in a sediment routing formulation to calculate sand removal as a function of water volume, discharge, and pool dredging. The sand-removal efficiency can then be evaluated in terms of water use, gravel loss, and dredging volume.

The optimum magnitude of a sand removal discharge is a compromise: higher discharges produce more efficient sand transport, but also reduce the trap efficiency of the pools and cause a greater loss of gravel. We find that a discharge between 5,000 cfs (142 m^3/s) and 6,000 cfs (170 m^3/s) provides the greatest efficiency in sand removal, while keeping gravel loss to the minimum required to mobilize the bed. Dredged pools greatly increase the amount of sand that can be removed from the reach and do so at a small cost relative to that of the released water.

Two different concepts have been proposed to specify the timing of releases for sediment maintenance. The first is to time the release relative to periods of spawning, incubation, and migration of the anadromous salmonids. A release can be timed to avoid scouring active redds during periods when salmonid eggs and alevin are resident in the bed. A release can also be timed to assist the downstream migration of juveniles in May or June. The second approach is to time reservoir releases to coincide with high flows on the tributaries, which can provide a savings in the water volume released and act to immediately flush tributary-derived sediments. Because tributary floods typically occur during the winter months (January through March), such timed releases run the risk of scouring active redds.

We find that a further problem with such timed releases is that they are likely to accelerate the deposition of the steep, fine-grained banks that have deposited within the former active river channel. Although such banks are common along alluvial rivers, they pose important management problems for the Trinity River. As the banks grow in height, flows are confined at higher and higher discharges. The result is concentration of flow into a deeper channel with higher velocities, providing few refugia from high flows for fish, especially juvenile salmonids.

Analysis of the post-dam water and sediment discharge history of the Trinity River and Grass Valley Creek suggests that these banks have been deposited during relatively short periods (possibly as short as two weeks) with combined high river stage and large fine-grained sediment input from the tributaries. Similar rapid deposition during trial releases would accelerate the bank building process and would also compete against attempts to mechanically restore the channel banks to their pre-dam condition. As a consequence, we do not recommend that flushing releases be timed to coincide with periods of tributary flooding.

Little of the bank-forming fine-grained material is found in the river bed. This suggests that relatively low discharges are capable removing this fine grained material from the river reach and that controlled releases are not necessary to remove the fine-grained bank-building sediment from the river. At the same time, we observe that high discharges producing overbank flow during periods of low tributary inflow do not have a high sediment concentration and, therefore, do not contribute to bank-building. The trial releases observed in this study were made during periods of low tributary inflow. Because very little sediment finer than 1-2 mm is found in the river bed, there was almost no sediment of that size present in the water column during the release. As a result, the trial releases did not produce further deposition of the fine-grained banks and future releases during periods of low tributary inflow will not contribute to further bank building.

Flushing releases in May or June can be scheduled in advance, are unlikely to coincide with tributary floods, assist the downstream migration of juvenile salmonids, and carry essentially no fine-grained material that would contribute to bank building. In contrast, releases timed to coincide with tributary floods provide only a minor potential savings in water, impose additional costs and safety concerns, are likely to scour incubating eggs, will continue or accelerate deposition on the channel banks, and are not likely to produce a net environmental benefit.

2. INTRODUCTION

2.1 Effects of Reservoirs and the Need for Flushing Flows

River channels immediately downstream of reservoirs typically experience a decrease in flood magnitude, sediment transport capacity, and both the load and caliber of sediment supply. When flow diversions are made at the reservoir, the total discharge is also reduced. The adjustments of the downstream channel depend on the relative changes in these variables and on the rate at which unregulated water and sediment is introduced from tributaries downstream of the reservoir.

A common case is one in which coarse sediment is efficiently trapped by the reservoir and excluded from the river downstream, whereas fine sediments are introduced to the downstream channel either from the reservoir or from downstream tributaries. If the transport capacity of the downstream channel is sufficiently reduced, the finer sediment may accumulate on the bed and banks of the river. This sediment may fill pools needed for rearing habitat, bury cobble substrates needed by juvenile salmonids for cover and invertebrate food production, and infiltrate into gravels required for spawning. In the presence of reduced floods, vegetation can colonize higher elevations of the channel. Deposition of fine sediment along channel margins, often abetted by vegetation encroachment, can reduce the hydraulic capacity of the channel, thereby increasing flood hazard if uncontrolled spills occur. Because of the various downstream impacts of impoundment, controlled releases designed to mimic the action of natural floods in removing accumulated fine sediments from the channel, flushing flows, are commonly required by regulatory agencies (Milhous, 1982; Reiser *et al.*, 1989).

Because channel gravels are an important component of the fluvial habitat, the elimination of an upstream supply of coarse sediment by the reservoir represents an important constraint for the ecology of the downstream channel and for flushing releases designed to maintain that channel. Reaches downstream of dams can become deficient in gravel when gravels are transported from these reaches without replenishment from upstream. There is a danger that flushing flow releases may exacerbate gravel supply problems by increasing gravel transport. Thus, the potential loss of spawning gravels must be considered as a potential cost of flushing flows, together with lost power generation and water supply revenues.

A variety of objectives may be addressed with flushing flows. These may be broadly separated into two groups: one based on removing fine sediments from the surface or from within the channel bed and the second based on maintaining the channel size and shape in some desirable form (Milhous, 1990; Reiser et al., 1989). We use the term "flushing" in a general sense to describe any discharge designed to move sediments within a regulated channel. This is consistent with its generic meaning and does not require previous, conflicting usages to be entirely abandoned. Different flushing objectives may be distinguished as either sediment maintenance or channel maintenance flushing flows, based on whether the objective is to modify or maintain the channel sediment or the channel geometry.

The objective of this project is to specify flushing flows for sediment maintenance. In particular, the objective is to develop recommendations for future flushing releases to clean and maintain the potential spawning gravels on the Trinity River below Lewiston Dam.

A wide range of methods have been suggested for estimating flushing flows, as comprehensively reviewed by Reiser *et al.* (1989). These methods can be classified based on their data and field work requirements (Reiser *et al.*, 1985). Flushing methods may also be classified according to their underlying assumptions (Kondolf *et al.*, 1987). One group of methods specifies flushing flows as a discharge with a prescribed frequency calculated from discharge records. A different approach is to specify a discharge that is observed or estimated to produce weak gravel motion (as a surrogate for flushing).

Flushing methods based on an historical discharge frequency may be thought to mimic the natural flow regime in some way that will permit adequate flushing with a minimum amount of water. Examples of this approach include the discharge giving 200% of the mean annual flow (Tennant 1976), the flow exceeded 17% of the time (Hoppe and Finnell 1970), and the flow with a pre-regulation recurrence interval of 1.5 years (Montana Department of Fish, Wildlife, and Parks 1981). Flushing methods of this class depend on a series of assumptions. First, it is assumed that the natural river channel has developed a size and shape that is in a state of adjustment to the natural water and sediment regime. If it is further assumed that the natural range of water discharge may be represented by a particular value of "effective", or "dominant" discharge, then a periodic release of this discharge should maintain the sediments in their natural state. Water regulation and/or consumption is achieved by eliminating all other high discharges that formerly went through the channel. Finally, it a single effective discharge is observed to produce adequate flushing on one river, it is assumed that a discharge of a similar frequency will produce similar flushing on another river, because both river channels will have adjusted their channels to the same effective discharge.

The series of assumptions behind the discharge-frequency flushing methods are often not satisfied on rivers in need of flushing. This is particularly the case for channels downstream of reservoirs that have been in place for a period of more than a few years, as is the case on the Trinity River. If sufficient time has elapsed that the river channel has adjusted substantially from its natural state, a discharge that might have been effective in flushing the natural channel is likely to no longer be appropriate for the adjusted channel. Because of substantial flow diversions from the basin, the present active channel of the Trinity River is considerably smaller than the preregulation channel. Further, development down to a much lower, post-reservoir 100-year flood elevation prohibits discharges more than one-half of the former two-year flood, so a flushing flow based on prescribed frequencies of the preregulation discharge are no longer feasible. The absence of a self-adjusted channel and the severe limits on discharge magnitude make a discharge-frequency flushing method inappropriate for the Trinity River below Lewiston Dam.

Sediment maintenance flows based on the discharge necessary to entrain the river-bed gravel may be calculated from existing channel conditions. Therefore, these estimates may be made independent of channel history. It is well established that flushing of interstitial fine sediment from within a gravel bed requires entrainment of at least the surface layer of coarser grains (Beschta and Jackson, 1979; Diplas and Parker, 1985). Thus, a discharge that entrains the surface gravels can be treated as a surrogate for a sediment maintenance flushing flow. Bed mobility for sediment-maintenance flushing has been predicted using computations of incipient motion (Milhous and Bradley 1986, Milhous 1990), observations of tracer movement (Hey 1981), and computation of effective discharge from flow records and sediment rating curves (O'Brien 1987). The approach used in this study is similar: we use direct observations of sediment entrainment, local flow velocity, and discharge during trial releases to evaluate the relation between flushing effectiveness and release discharge.

2.2 The Trinity River

The Trinity River drains 2,950 mi^2 (7640 km^2) of rugged terrain in the Klamath Mountains of northwestern California, flowing into the Klamath River near Weitchpec on the Hoopa Indian Reservation (Figure 2.2.1). The basin is mostly forested, although much of the basin has been clearcut since 1950. The Trinity River historically supported important anadromous salmonids, including chinook salmon (*Oncorhynchus tschawa sp*), silver salmon (*O. keta*), and steelhead trout (*O. mykiss*). Annual spawning runs of these fish historically provided a principal food source for the Hoopa Indians, and have been responsible for a significant part of the commercial catch (Smith 1976).

Runoff from the uppermost 720 mi^2 (1860 km^2) of the basin was impounded by Trinity Dam (and its re-regulating reservoir, Lewiston Dam) beginning in 1961, as part of the US Bureau of Reclamation Central Valley project. These dams eliminated access to important spawning and juvenile rearing areas upstream. Beginning in 1963, about 75 percent of the average natural runoff from the upper basin of 1800 cfs (53 m^3/s) has been exported from the Trinity River basin to the Sacramento River basin, where it is diverted for irrigation, generating hydroelectric power en route. In the reach directly below the reservoir and above major tributaries, floods have been virtually eliminated. The mean annual flood decreased from 18,500 cfs (525 m^3/s) pre-dam (1911-1960) to 2580 cfs (73 m^3/s) post-dam (1964-1990), as measured at the US Geological Survey gage at Lewiston (Figure 2.2.2). Flood frequency analysis for pre- and post-dam conditions shows that Q_2 (the annual peak flood that occurs, on average, every two years) has decreased from 17,100 cfs (484 m^3/s) to 1,060 cfs (30 m^3/s). These reductions in flow regime have resulted in substantially decreased sediment transport capacity in the Trinity River in the reach immediately below Lewiston Dam, with the effects of flow regulation decreasing with distance downstream from the dam.

One effect of the reduced flood regime on the Trinity River has been encroachment of riparian vegetation and channel narrowing (Frederiksen, Kamine, and Associates 1980). The present channel is flanked with stands of alder (Alnus sp.) that have become established along the low flow channel, within the pre-dam active channel. Aerial photographs show that these alders grew to maturity following closure of Trinity Dam and its elimination of major floods. The alders were able to resist washout by the January 1974 spill, which reached a peak discharge of 14,400 cfs (408 m^3/s) at the Lewiston gage and now form a narrow, dense band along the present channel (Figure 2.2.3). This riparian vegetation stabilizes the banks of the former low-flow channel (the present active channel) (Figure 2.2.4) and induces deposition of suspended sediment within the vegetation (Pitlick 1992).

Concurrent with the reduced flood and sediment transport capacity in the mainstem, sediment yields from tributary watersheds increased as a result of road construction and timber harvest. Most notable among these tributaries is Grass Valley Creek, which flows into the Trinity River about 8 mi (13 km) downstream of Lewiston Dam. Grass Valley Creek drains a 38 mi^2 (98 km^2) basin underlain principally by the Shasta Bally Batholith, which weathers to produce decomposed granitic soils that are readily eroded and produce large yields of sediment in the 1 mm to 8 mm size range. From 1950 to 1960, 90 percent of the Grass Valley Creek basin was logged, resulting in sediment yields estimated at 133,000 yd^3 (102,000 m^3) annually in the 1970s (Frederiksen, Kamine, and Associates 1980). This sediment (mostly decomposed granitic coarse sand and fine gravel) filled the channel of Grass Valley Creek and entered the mainstem Trinity River. The reduced mainstem flood regime has been inadequate to transport these tributary-derived sediments, which have filled pools, buried cobble substrate, and infiltrated spawning gravels (Figure 2.2.5), thereby degrading aquatic habitat for anadromous salmonids, which were formerly abundant in this reach (Smith 1976). The combined effects of high tributary sediment yields and reduced mainstem flows during the large storm of December 1964 resulted in deposition of large deltas at tributary confluences (Ritter 1968).

Since 1975, the Trinity River Basin Fish and Wildlife Task Force has been engaged in a Comprehensive Action Plan to arrive at alternatives to restore fish habitat. The Task force has undertaken periodic dredging of sand from several pools, scarifying of gravel beds, importation of gravel to the mainstem, excavation of side channels, and installation of check dams, groins, and other instream structures, and made trial flushing flow releases.

2.3 Previous Work

Previous studies have recommended flushing flows ranging from 800 to 10,000 cfs, as summarized in Table 2.3.1 and discussed below.

Frederiksen, Kamine, and Associates (1980) collected sediment samples on the Trinity River and tributaries, estimated annual sediment yields, and estimated flows required to entrain gravel in riffles and to transport accumulated sediments downstream. They estimated that discharges of 800 cfs to 1,200 cfs (23 to 34 m^3/s) would "initiate movement of gravel-size material in riffle areas for purposes of cleaning" (p.69)(Table 2.3.1). However, this prediction of gravel mobility is inconsistent with the report's statement that the 1974 flow, with 10 days of flow between 5,000 cfs (142 m^3/s) and 14,000 cfs (396 m^3/s), "...had little impact with respect to moving gravels and cobbles from the [tributary] deltas" (p.69).

They estimated sediment discharge from the Trinity River below Grass Valley Creek for flushing releases of different volume and duration; release magnitude was limited to 900 cfs (25 m^3/s) "...to minimize damage to restored spawning areas" (p.67). They estimated the annual sediment yield of Trinity River below Grass Valley Creek was 91,000 tons (83,000 tonnes).

Annual sediment transport was assumed to be controlled by transport through the pools. Sediment transport with annual releases of 320,000 acre-feet (395 million m^3) was estimated to transport 1,800 to 22,200 tons (1,600 to 22,200 tonnes)(p.67). The lower transport rate was calculated assuming pools were 60-ft (18-m) wide, the higher rate for pools 30-ft (9-m) wide.

Strand (1981) prepared an estimate of the magnitude and duration of flushing flows necessary to remove accumulated sediment from a 5.8-mile (9.2-km) reach of the Trinity River from the confluence of Grass Valley Creek to Steelbridge, which encompasses the study reach of this project. In his study, Strand used observed water surface elevations at low discharges to calibrate a hydraulic model, which was then used to extrapolate flow conditions to much higher discharges. Water surface elevations were measured at 23 cross sections at 300 cfs (8.5 m^3/s) and 600 cfs (17 m^3/s), and at two cross sections at 2,200 cfs (62 m^3/s). Calibrated with these low flows, the model was then run for discharges of 4,000 cfs (113 m^3/s), 6,000 cfs (170 m^3/s), and 10,000 cfs (280 m^3/s).

Transport rates were not measured by Strand (1981). Rather, estimates of transport rate were made using the Modified Einstein Transport Method (Colby and Hembree 1955), which requires as input estimates of the mean channel hydraulics and the grain-size of the bed material and suspended load. These transport estimates were then treated as "measured" values and used to select a second transport model, the Velocity-Xi method (Pemberton 1972). The Velocity-Xi method was used to estimate transport rates at higher flows, for which sediment samples needed to use the modified Einstein method were not available. The amount of sediment in the study reach to be flushed was estimated at 63,700 tons (58,000 tonnes) from measurements of the channel bed area and an estimated depth of sediment based upon field observations (Strand 1981). The extrapolated transport rates were used to estimate the amount of sediment that would be removed over time. Strand concluded that all three modeled flows would remove 50% of the finer sediment, but only the higher releases (6,000 cfs and 10,000 cfs) could remove 90%.

Several factors make the flushing estimates of Strand of limited use in specifying controlled releases for flushing. Because the flows observed by Strand were relatively small, the flushing estimates were necessarily made for flows much larger than those observed, requiring extrapolation of the hydraulic relations, as well as the transport rates computed from the hydraulics. Uncalibrated estimates of sediment transport from observed hydraulics can be considered to provide, at best, order-of-magnitude accuracy (Vanoni 1975; Gomez and Church 1989; Nakato 1990). Strand's transport estimates were based on hydraulics estimated by extrapolation to flows ten times greater than observed at most cross sections and thus can be considered only qualitative. Transport rates were not measured at any flow, and the relation used to extrapolate the transport rates at flushing flows was itself "calibrated" by comparison with results of another transport equation.

Because the goal of a flushing flow is to remove finer sediments while minimizing the removal of coarser grains, Strand calculated the transport removal for eight different size fractions. These calculations do not account for the effect on fractional transport rates of the size of each fraction relative to the remainder of the mixture. These relative size effects have been shown to have a first-order effect on fractional transport rates in poorly sorted sediments such as those found on the Trinity River (Parker *et al.* 1982, Wilcock and Southard 1988, 1989).

Nelson *et al.* 1987 reviewed previous studies and, apparently based on this review, concluded that a flow of 4000 cfs (117 m3/s) was "...large enough to simulate the historical peak flows below Grass Valley Creek that are necessary to move large cobbles, to destroy the annual growth of bank vegetation, and to restore the approximate original channel configuration."

The wide range of flushing flow estimates and the lack of direct field observations of the flushing effectiveness of various discharges contributed to the motivation for a trial release research program in which the potential flushing achieved by different controlled releases is examined. This study and the work of Trinity Restoration Associates (1993) are part of this effort. The latter work was an extensive field study of eleven sites within 35 miles downstream of Lewiston Dam during the experimental flushing flow releases of 1991 (3000 cfs or 85 m^3/s) and 1992 (6000 cfs or 170 m^3/s). The report concluded, "Clearly the 6000 cfs release mobilized the surface of most bar units, while the 3000 cfs flow did not." (p.136).

2.4 Study Objectives

The overall objective of this project is to recommend reservoir releases that will flush and maintain spawning gravels in the reach of the Trinity River between Grass Valley Creek and Steelbridge (the reach defined by Strand, 1981). To do this, we focus on two representative reaches containing spawning gravels and examine the potential flushing produced by trial releases in 1991, 1992, and 1993. Our goal at both sites is to determine the discharge necessary to entrain the spawning gravels and permit interstitial fine sediment to be flushed from the bed. Because gravel recruitment is limited on the Trinity River, a sediment maintenance flow should involve minimal downstream gravel transport, but sufficient gravel movement to permit flushing of subsurface fine material. Such a release is also likely to minimize water use. Thus, our objective was to determine the magnitude of a release that will just entrain the gravel surface, thereby permitting subsurface flushing while minimizing gravel transport and water use, and maximizing sand transport at the two study sites.

We observed bed conditions before and after the releases and made direct measurements of flow velocity and discharge. The bed observations included pebble counts and visual observations of the bed size distribution throughout both study reaches. Gravel entrainment was observed using tracer gravels and scour chains placed in potential spawning gravels along cross sections at both study sites. The flow observations permitted calculation of total discharge during each release and observations of local velocity acting directly on the spawning gravels. The combined observations of local flow and gravel entrainment permit the accuracy of our observations to be evaluated in terms of general sediment transport relations drawn from experiment and theory, which provides a basis for estimating entrainment and flushing at flows other than those we directly observed. Together with the measurement of discharge, these results provide the basis for specifying the discharge magnitude and duration that can produce flushing of the spawning gravels.

In this study, we focus on the discharge required to flush fine sediment from spawning gravels and our field observations focus on flow and transport conditions at two representative study sites to develop a detailed understanding of gravel flushing on the Trinity River. A concurrent study addressing a broader range of flushing objectives was undertaken on behalf of the Hoopa Valley Indian Tribe (Trinity Restoration Associates 1993). This study involved a larger study reach of 35 miles (56-km) and many more sites at which channel change, tracer gravel movement, and riparian scour were observed before and after the releases, with limited observations of water surface elevation made during the releases.

The overall quantity of sand in the study reaches is large. In the presence of a high sand concentration, a discharge sufficient to entrain the bed gravels and flush sand at depth will not produce a markedly cleaner bed because sand can be redeposited with the gravel. To achieve successful flushing, the total volume of sand in the reaches must be reduced. The discharge and water volume required for sand removal will depend on the volume present in the study reach and the rate at which it is transported by the release discharge.

To provide some guidance for selecting a discharge for sand removal on the Trinity River, we include in this report an estimate of the volume of sand in the study reach and the rate at this sand is transported at different discharges. This information is used in a calculation of sand routing through the study reach to estimate the volume of sand that may be removed by discharges of different magnitude and duration. Because pools may act as sand traps, we also examine changes in sand storage during the trial releases in the primary pools in the study reach. We develop a relation for pool trapping as a function of sand transport rate, pool geometry, and water discharge and use this relation with the sand routing algorithm to evaluate the additional sand removal that can be achieved with pools. Because gravel recruitment on the study reach is largely eliminated by upstream dams, we also develop a relation for gravel movement as a function of discharge and use this relation to estimate the volume of gravel that may be lost in a flushing flow.

The relations for sediment transport and pool trapping are used with the routing algorithm to evaluate the tradeoffs among sand removal, gravel loss, pool dredging, and water use during a flushing release. An optimum flushing release is one that maximizes sand removal, while minimizing water use and downstream loss of the gravel resource. Our analysis provides a starting point for identifying such an optimum flushing release, although the analysis has several limitations. The modeling results must be evaluated in the context of the simplifications and assumptions required to compute sand and gravel movement throughout a large river reach, which limit the accuracy with which the total sand and gravel removal can be estimated. A more fundamental limitation is that the selection of an optimum flushing release depends on many factors and it is likely that no single release plan can satisfy all the objectives that might be identified. Many of these factors were not considered in this study, including the value of the water used, the reservoir operating rules, the legal obligations associated with the water, the variation in time of both the supply and demand on the reservoir water, the cost of dredging, and the environmental requirements of the fishery. Because of the large number of factors and objectives requiring consideration, and because some of the objectives cannot be mutually satisfied, it is not likely that a single optimum flushing release can be identified. Rather, it is necessary to develop a rational basis for evaluating the tradeoffs among the different objectives, so that a compromise may be found that is acceptable to all concerned parties. Further work is needed to develop a decision-making tool that may be used to evaluate these tradeoffs in an objective fashion.

3. DESCRIPTION OF STUDY SITES

3.1 Rationale for Study Site Selection

We selected our two detailed study sites in consultation with USFWS to provide representative spawning gravels, hydraulic characteristics favorable for flow modeling, and good access. Another consideration was the availability of hydraulic data at the sites from aquatic habitat studies by USFWS, data which permitted us to predict water depths and velocities during releases to plan study logistics. Our study sites were located downstream of the Grass Valley Creek confluence and upstream of the next major tributary, Indian Creek.

3.2 Poker Bar Study Site

The Poker Bar study site is located about 9 mi (15 km) downstream of Lewiston Dam, about 6 mi (10 km) southeast of Weaverville at latitude 40°41'N and longitude 122°53'W (Figure 2.2.1). The overall study reach from Grass Valley Creek to Steelbridge is a sinuous pool-and-riffle channel, but at the Poker Bar study site the river flows in a straight, single-thread channel for a distance of 2000 ft (600 m; Figure 3.2.1). Bankfull width is 115 ft (35 m) and the average channel gradient is 0.3%. We established 11 cross sections over the 700-ft (200-m) reach. The site contains a large gravel bar that occupies most of the width of the channel. In cross section, the channel resembles a bowling alley along much of the reach, with the bar in the center flanked by deep trenches on either side. The trench running along the base of the right bank is the most pronounced; we designated it the "gutter". A side channel exits the left bank between cross sections PB1B and PB2 and reenters the main channel between sections PB3A and PB4. The base elevation of the secondary channel is above the main-channel water level at discharges below 1000 cfs. At higher flows, the side channel floods, but carries little discharge. The primary study section, PB2, crosses near the highest elevation of the mid-channel bar and has a very deep gutter on the right side (Figure 3.2.2).

The stream gravels are derived principally from schists of various ages, mostly Paleozoic and Mesozoic. Potential spawning gravels are most abundant between cross sections PB1B and PB2. A grain size distribution based on 8 bulk samples taken from PB2 and ranging in size from 135 to 280 kg (total sample size 1776 kg) is presented in Figure 3.2.3. The bed material is weakly bimodal, with a primary mode between 16 mm to 64 mm and a secondary mode between 1 mm and 8 mm. Median grain size of the bed material is 22 mm. 30% of the sediment is finer than 8 mm, three-quarters of which falls between 1 mm and 8 mm. Although a small amount of material finer than 8 mm would be found in the bed under natural conditions, to a first approximation, the material finer than 8 mm represents the sediment requiring flushing. Figure 3.2.4 presents the cumulative grain-size distributions for the bed material segregated into parts finer and coarser than 8 mm. The median grain size of the coarser portion is 36 mm.

The channel is flanked by linear stands of alders typically 1-2 ft (0.3-0.6 m) in diameter. These alders have become established within the active channel since flow diversions began in 1963. The formerly active gravel bar (Poker Bar) has now been filled and reworked by heavy equipment to create a level surface for housing, whose development was encouraged by flood protection offered by Trinity dam.

3.3 Steelbridge Study Site

The Steelbridge study site is located 12 mi (20 km) downstream of Lewiston Dam and about 10 mi (16 km) southeast of Weaverville at latitude 40°40'N and longitude 122°55'W (Figure 2.2.1). The study site is located adjacent to a Bureau of Land management (BLM) campground about 0.25 mi (0.4 km) upstream of the remains of a bridge known as "Steelbridge".

The Trinity River in this reach is split by an island. Our study site is located on the 700-ft (200-m) long right channel. We established 10 cross sections along this reach (Figures 3.3.1). This side channel carried roughly 30% of the total river discharge during the experimental releases. Bankfull width of the side channel is 65 ft (20 m) and the average gradient is 0.2%. Spawning gravels are heavily used by fish in the downstream portion of the reach. Potential spawning gravels are most abundant between cross sections SB3B and SB4. The primary study section, SB3C is 23 m wide and nearly rectangular in shape (Figure 3.3.2).

A grain size distribution based on 10 bulk samples taken from sections SB3C and SB3D and ranging in size from 112 to 228 kg (total sample size 1490 kg) is presented in Figure 3.2.3. The gravels are weakly bimodal with a predominant mode between 32 mm and 180 mm and a wekaer mode between 1 0 mm and 8 mm (Figure 3.2.3). The median grain size of the bed material is 36 mm. 25% of the bed material is finer than 8 mm, three-quarters of which falls between 1 mm and 8 mm. The median grain size of the portion of the bed sediment coarser than 8 mm is 56 mm (Figure 3.2.4). The gravels are coarser than, but lithologically similar to, those at Poker Bar.

The channel at this study site has undergone substantial changes since closure of Trinity and Lewiston Dams. In 1960, the active channel extended across the full river width and the island was a mid-channel bar. With elimination of scouring floods, vegetation established on the bar, grew to maturity, and can now survive virtually any high discharge. The vegetation trapped fine sediment, building the island, especially along its downstream half. The island is flanked by a nearly continuous stand of alders. Most of the island's center is unvegetated (or sparsely colonized by xeric plants) because the water table is too low for riparian vegetation to establish in such coarse alluvium.

4. METHODS

4.1 Trial Releases

Previous estimate of flushing flows on the Trinity River were based on observations at low flows. To provide an opportunity to directly observe the effect of higher discharges on sediment maintenance and aquatic habitat, controlled high-flow releases were made from Lewiston Dam by the Bureau of Reclamation in 1991, 1992, and 1993. Each of the flows was preceded and followed by several days of ramping (intermediate) flows between the release discharge and the normal release of 300 cfs (8 m^3/s). Figure 4.1.1 and Table 4.1.1 present the daily mean discharge recorded at the USGS gage at Lewiston, just downstream of the Lewiston dam. Discharge values for the entire period between April and June in each of the three years are plotted in Figure 4.1.1. The discharge measured at Lewiston represents the release input into the River and is usually slightly different from the discharge values measured at our study sites, because of discharge additions or subtractions between the USGS gage at Lewiston and our study sites.

The 1991 release took place over six days from 28 May to 2 June, with a maximum release between 2,600 cfs and 2,800 cfs (74 m^3/s to 79 m^3/s) for four days from 29 May to 1 June. A relatively constant discharge of 2,670 cfs to 2,685 cfs (75.6 m^3/s to 76.0 m^3/s) was observed at our study sites for three days from 30 May to 1 June.

The 1992 release took place over ten days from 10 June to 19 June, with a release in excess of 6,000 cfs (170 m^3/s) for five days from 12 June to 16 June. A relatively constant discharge of 5,800 cfs (164 m^3/s) was observed at our study sites for four days from 13 June to 16 June.

The 1993 release took place over 22 days from 13 April to 4 May, with a release close to 3,000 cfs (85 m^3/s) for 17 days from 14 April to 30 April. A relatively constant discharge of 2990 cfs (80 m^3/s) was observed at our study sites on 27 April and 28 April.

We were able to make velocity and transport observations on almost every release day in 1991 and 1992, including some ramping days, at both Poker Bar and Steelbridge study sites (Table 4.1.1). Because the 1993 release discharge was nearly the same as that in 1991, we limited our during-release observations to two days, 27 April and 28 April at the Poker Bar site only.

4.2 Logistics of Observations During Releases

Our field work consisted of (1) cross-section surveys and bed sediment sampling before and after the experimental releases, and (2) water surface elevation, velocity, and flow depth observations during the releases. Some limited suspended-load and bed-load sampling was also conducted during the releases. The pre- and post-release field work (described in later parts of this section) was conducted during releases of about 300 cfs (8.5 m^3/s) when the channel could be easily waded.

Our measurements during the releases required substantial logistics. We required a platform from which to lower current meters, sediment samplers, and an acrylic box through which we could observe bed conditions. The channel was too wide to build a temporary footbridge, so we operated from catarafts and custom-designed rafts equipped with a crane and reel for raising and lowering the 170-lb (77-kg) bed-load sampler.

The rafts were held in position against the current and moved across the channel with a network of climbing ropes (Figure 4.2.1). We use climbing rope instead of the steel cables traditionally used in flow measurement work because of the strength and flexibility of climbing rope now available, because climbing rope is more visible than steel cable, and because steel poses a significant safety risk if it fails. Climbing rope has been used for fishery studies elsewhere in California (Li and Holton 1986) and in the Trinity River by the USFWS Lewiston staff, who provided substantial assistance in logistical design and execution.

At each cross section measured during the experimental releases, we established a main line extending across the channel and anchored to large trees on either bank (Figure 4.2.1). The raft was held against the current by a bowline extending through a pulley attached to the main line. When the raft was near the center of the channel in stronger currents, it would tend to be displaced farther downstream. To compensate for this and thus keep the raft traversing a straight cross section, we pulled the raft upstream by reeling in the bowline.

The cross-channel position of the raft was adjusted by lateral lines passing through pulleys on either bank. The arrangement of lateral lines at Poker Bar is shown on Figure 4.2.1. At Steelbridge, our access was from the left bank instead of the right bank, so the arrangement of lateral lines was reversed. We could move the raft towards the left bank by pulling the left bank lateral line into the raft, thereby shortening this line (which passed from our pulley on the mainline through a pulley on the bank, thence to the raft; Figure 4.2.1). The raft was moved toward the right bank by a person on the right bank pulling in the right bank lateral line. This avoiding having too many lines on the raft and took advantage of human resources available on the right bank. When the raft was pulled towards one bank, the opposite lateral line was loosened gradually to provide the needed slack. To prevent the tail of the raft from swaying from side-to-side, we tightened the rear stabilizing line from the raft to the rear line. This line required frequent adjustments to maintain a steady tension.

We typically measured two cross sections from each rope set-up. After the upstream section was measured, we loosened the bowline (and lateral lines) and allowed the raft to drop downstream to the second cross section, tightening the rear stabilizer as we went.

4.3. Hydraulics

The objectives of the flow observations during each trial release were to determine the mean water surface elevation and slope throughout both study reaches and to determine the distribution of velocity across the primary study sections and, in some cases, adjacent sections. The water surface elevations are used to calculate water surface slope, to provide a check on the steadiness of the flow during velocity observations, and to provide calibration observations for hydraulic modeling of the water surface elevations at discharges other than those used for the trial releases. The velocity observations are used to calculate total discharge, the lateral variation of velocity across a section, and to estimate the lateral variation of bed shear stress acting on spawning gravels at each primary study section. This latter information is then used together with the gravel mobilization observations to assess the flushing capability of the trial releases.

Water surface elevations (WSEL) were measured using staff plates attached to trees or stakes along one bank of each study reach. Staff plates were installed on the right bank of all 11 sections at Poker Bar, covering a downstream distance of 210 m. Staff plates were installed on the left bank of all 10 sections along the Steelbridge study channel (i.e. the right channel passing the island at the Steelbridge campground). In addition, two gages were located on the left bank of the main channel and a third gage was located on the right bank downstream of the island. These three gages permit calibration of WSEL modeling of the total discharge through both channels, which provides a basis for extrapolating discharge-WSEL relations to flows in excess of 6,000 cfs, which inundate the channel island.

Flow velocities were typically small to negligible in the vicinity of the staff plates. Surface waves were typically smaller than one or two cm and the mean water surface elevation could generally be read within ±0.5 cm. All staff plates were read on a regular basis during velocity observations at each study site. In addition, the staff plate for a particular section was read immediately before and after each velocity traverse.

A total of 477 vertical profiles of downstream velocity were measured in conducting 38 transects along eight different cross sections. The largest number of profiles and traverses were made at the two principal study sections: PB2 and SB3B. Velocity transects were made at the immediate adjacent sections at both study sites in 1992, and at the Poker Bar study site in 1993. Table 4.3.1 provides a summary of all velocity transects, including the number of locations, or stations, in each traverse, the typical and maximum spacing between stations, and the typical number of individual velocity observations taken in a vertical profile.

With the exception of six transects at Poker Bar in 1992, all velocity observations were made with the current meters mounted on rigid rods, along which the meter could be moved in the vertical. The rods permit more accurate reading of flow depth than possible with cable-suspended meters, and permit the relative vertical distance between meter positions to be determined very accurately. Once in place at a station, the rods were not moved until the vertical profile was complete, so that the accuracy of relative placements of the meter was maintained. Unusually long wading rods 6-ft, 8-ft, and 10-ft (1.8-, 2.4-, and 3-m) in length were used to permit a maximum number of velocity profiles to be made using rods. In 1992, depths were too great to use the wading rods along six transects, so at these sites we deployed the current meters from a crane-and-reel assembly installed on the raft.

All velocity observations were made using Price AA current meters. The meters were regularly inspected during each transect and were cleaned and testing for free rotation before and after each transect. Velocity observations at each point were conducted for a minimum of 40 seconds. In general, the rotations was counted from audible clicks and the sample duration read from a stop watch, with an assistant on the boat to record the information. In some cases, an automated sample counter was used.

4.4 Substrate Modification

The goal of our substrate observations was to characterize the sediment composition of the streambed and evaluate changes as a result of the trial reservoir releases. This was accomplished using repeated cross section surveys to evaluate scour and deposition, characterizing the bed surface visually and with pebble counts, and bulk sampling bed sediment.

4.4.a Cross-Section Surveys

A series of cross sections was established to monitor scour and deposition and perform hydraulic modeling at each of the two flushing study sites (11 at Poker Bar, 10 at Steelbridge). Cross sections were spaced one-half to one channel width apart and were oriented perpendicular to high flow (Fig. 3.2.1 and 3.3.1). Existing U.S. and Fish and Wildlife transects were incorporated to use historical water surface data in our initial hydraulic modeling. All cross sections were monumented with either rebar or spikes in the base of trees. All surveys were done with an automatic level. The maximum spacing between stations was one meter.

Tables 4.4.1 and 4.4.2 show the date each cross section was surveyed at Poker Bar and Steelbridge, respectively. Existing U.S. Fish and Wildlife transects are designated by a single number (e.g., PB1) and our new sections have an alphabetical suffix (e.g., PB1A). All cross sections were surveyed before and after the 1991 release. Cross sections were later resurveyed whenever we suspected that scour or deposition may have occurred; such as after the 1992 release. If we were uncertain that a resurvey was necessary, we resurveyed the most critical sections (over the spawning gravels). If those sections did not show scour or deposition, we did not resurvey additional sections.

4.4.b Visual Characterization of the Bed Surface

Visually substrate characterization is widely employed by fisheries biologists to quickly describe substrate without disturbing the bed. Visual estimates are more subjective than the physical sampling of sediment, but they permit rapid assessments over a wide area. Visual substrate estimates also highlight the local variability within a reach.

Initially (1991), visual characterization was done primarily to estimate bed roughness variations for hydraulic modeling and consisted of classifying the sediment as sand, gravel, cobbles, or boulders. In 1992, we expanded our visual observations of the bed surface with the goal of detecting more subtle changes in bed texture, especially with respect to the abundance of fine sediments on the bed surface. To that end, we estimated the proportion of the bed covered by fine sediments (%finer than 8 mm, hereafter referred to as percent embedded), as well as the median grain size (D_{50}) and the grain size for which 90% of the sediment is finer (D_{90}) for the coarse proportion of the bed (>8 mm), hereafter referred to as the gravel fraction. Grain sizes were estimated using phi size classes. The phi size scale is a logarithmic scale commonly used in earth science and engineering, where phi = $-\log_2 D$, where D is the grain size in mm. A phi difference of one corresponds to a grain size difference of a factor of two.

For standardization, we used the same operator (Barta) to make visual estimates at each survey point before and after each trial release. Errors for each observation are estimated to be ± 1 phi unit for D_{50} and D_{90} and ±10% for percentage embedded. When many observations are averaged over an area with similar bed texture, some individual errors are likely to cancel, yielding composite errors on the order of ±1/2 phi unit for D_{50} and D_{90} and ±5% for percentage embedded.

Tables 4.4.3 and 4.4.4 show the dates the substrate was visually described at each cross section for Poker Bar and Steelbridge, respectively. In each case, we described the substrate prior to other work in the study reach, so that the visual estimates would not be influenced by subsequent bed disturbance. In total, over 2000 visual observations describing the substrate were made over the course of this study.

4.4.c Pebble Counts

The goal of our pebble counting was to characterize the surficial sediment in the flushing study reaches. Pebble counts are performed by randomly selecting rocks within a defined region and recording their grain size. A sample size of at least 100 rocks is typically used. Pebble counts are less subjective than visual characterizations of the bed (Kondolf, 1993). All of our pebble counts were performed along established cross sections. If the bed texture differed laterally across the cross section, the section was sub-divided into continuous regions of similar bed texture. Pebble size was determined by passing each clast through a template with square holes on 1/2 phi intervals. Pebble size was then recorded as the 1/2 phi interval within which each clast fell. Pebble counting is also known as Wolman sampling (Wolman, 1954) and grid-by-number sampling (Kellerhals and Bray, 1971).

An important constraint for pebble counting is that a lower limit exists below which clast size cannot be effectively sampled. This limit is typically between 2 and 8 mm (Church et al., 1987). In our field work, clast sizes down to 2 mm were distinguished. With this lower limit, our samples include nearly all of the grain sizes present in the river bed, including the light-colored decomposed granitic sediments that have embedded the spawning gravels in the post-dam period.

Kellerhals and Bray (1971) have shown, using geometric arguments and field data, that grid sampling by number and volumetric sampling by weight (bulk sampling) produce comparable results with no conversion required. Controlled experiments by Church and others (1987) also suggest that grid by number and bulk samples are directly comparable.

Tables 4.4.5 and 4.4.6 show the dates, locations, and number of particles measured for Poker Bar and Steelbridge, respectively. During 1991 we counted 100 particles per cross section. This was increased in 1992 and 1993 to 200 particles for most cross sections. In total, over ten thousand particles were measured over the course of this study.

4.4.d Bulk Sediment Samples

Bulk sediment samples permit the direct measurement of the grain size distribution of the bed sediment. All bulk samples were taken in the cross sections where spawning gravels were located. After surveying the bed elevation at the sample point, a metal cylinder was inserted into the bed as deep as possible (similar to the method of McNeil and Ahnell, 1960). All sediment down to the bottom of the sampler was excavated and sieved. Three types of bulk samples were taken: 1) pre-release, 2) post-release at pre-release locations, and 3) new post-release samples next to the original samples. The pre and post-release samples at repeated locations were made to determine the sediment composition before and after the release. The additional post-release samples provided a control on the effect of pre-release sampling on the post-release sediment composition. Tracer gravels were installed at all pre-release sample sites as described in section 4.5.

In 1991 we used a sampler with a diameter of 30 cm and sample depths up to 30 cm, giving sample sizes between 13 and 30 kg. All 1991 samples were dried on a camp stove prior to sieving. The coarse fractions (>8 mm) were sieved in their entirety. The finer fractions were weighed and split and a representative sample was sieved. In 1991 we took 15 samples at Poker Bar and 18 samples at Steelbridge, with a total sample weight of 701 kg.

We enlarged our sample size in 1992 by using a sampler with a diameter of 59 cm and sampling as deep as 40 cm. This resulted in sample sizes between 112 and 281 kg. Samples were wet-sieved on the river. Coarse particles (>8 mm) were counted and converted to a mass based upon the average mass for each particle size measured in 1991. The volume of the finer particles was measured and a volume-to-mass conversion was made. In 1992 we took 8 samples at Poker Bar and 10 samples at Steelbridge, with a total sample weight of 3267 kg.

Table 4.4.7 and 4.4.8 list all bulk samples taken at Poker Bar and Steelbridge, respectively. Bulk samples were not taken during 1993 because the 1993 discharge was the same as the 1991 release, which produced little gravel movement. In total, nearly 4000 kg of sediment was bulk sampled over the course of this study.

4.5 Sediment Movement

Assessing gravel movement during high flows is central to determining if gravels are mobilized sufficiently to permit flushing. We monitored the mobility of gravels at Poker Bar and Steelbridge using tracer gravels. Additionally, because the restoration of the Trinity River depends upon the removal of accumulated fine sediments, we conducted an auxiliary bed-load sampling program to estimate the sand and gravel transport through the Poker Bar and Steelbridge study sites.

Monitoring gravel movement with tracer gravels has two advantages over measuring the bed-load transport during a high flow event. First, it provides a direct observation of the motion of the particular sediment of interest, spawning gravels. Second, it provides a direct measurement of the depth of scour, to which the degree of flushing is directly related. The depth of scour, or exchange depth d_{ex}, was estimated as the proportion of tracers removed over the flush multiplied times the installed depth of the tracers. Scour depth was independently estimated using scour chains, which are metal link chains installed vertically in the streambed. If the bed scours, the chain bends downstream to indicate the maximum depth of scour. Subsequent deposition over the chain may also be measured. Repeat cross section surveys before and after a release provided a less precise measure of bed scour over much larger portions of the bed.

The tracer gravels were installed after bulk sampling the sediment as previously described (Section 4.4.d). At each sample location we replaced the sampled gravel with distinctly marked tracer gravels. During the 1991 release we replaced the native sediment particles with pure white quartz particles with similar sizes and shapes. During the 1992 release, we replaced the sampled sediment with brightly painted sediment with similar sizes and shapes. No tracers were used during the 1993 release because the intended discharge had the same magnitude as the 1991 release. After the trial releases we inspected the tracer gravels and recorded the distance traveled for displaced particles and resampled the original location to determine the number and size of particles remaining in place. Tables 4.5.1 and 4.5.2 give the locations of all tracer gravels installed at Poker Bar and Steelbridge, respectively.

A Helley-Smith sampler (Helley and Smith, 1971; Emmett, 1980) was used to estimate the sediment transport through the full-width study reach at Poker Bar. Helley-Smith samplers have a square orifice and are placed on the bed to collect the bed load in a mesh bag. This sample is then weighed and representative samples are saved for grain size analysis. Although these samplers are designed to minimize the disturbance to the flow and transport on the streambed, their efficiency is highly variable. Common problems include oversampling due to scooping of the bed sediment and undersampling when the sampler is lodged on a large clast so that its bottom edge is above the surrounding bed and substantial transport occurs below the sampler. To minimize these errors, we directly observed the placement of the samplers on the streambed through a face mask. Obviously unrepresentative samples were discarded. Table 4.5.3 lists the details of our Helley-Smith bedload sampling at Poker Bar.

Prior the 1993 release, we buried five wooden boxes flush with the streambed across section PB2. The boxes were 80 cm long by 12 cm wide by 10.5 cm deep and were located at Stations 18.1, 23.5, 27, 30, and 33.3 on cross section 2. The purpose of the boxes was to trap the coarser portion of the bedload. During the release we were able to observe these boxes from our measurement platform using a diving mask or a streamlined acrylic box. Additionally, a USFWS SCUBA diver inspected and photographed the sediment traps during the release. Because the release proceeded for 14 days prior to our observations, the amount and caliber of the sediment in the boxes was estimated on the first day of the field work and the boxes were then cleaned out using a garden hoe, so that two separate sample periods were achieved. The primary sample period was taken to be that from the time the boxes were cleaned until the end of the 3000 cfs release.

4.6 Pool Surveys and Estimates of Surficial Fine Sediment

With guidance from USFWS personnel, we identified major pools in our study reach between Grass Valley Creek and Steelbridge (Figure 4.6.1). We initially selected three pools in 1991. One of these was abandoned due to access problems, and three additional pools with good access for study were included in 1992. Surveys were conducted in 1991, 1992, and 1993, as indicated in Table 4.6.1. Pools that were dredged before or during the study period were of particular interest for their potential to act as sediment traps.

Of the six pools measured at least once, Montana Pool was abandoned after the first year due to difficulties collecting data, access problems, and apparent lack of change in bed morphology. Neither maps nor analyses of storage changes in Montana Pool are included in this report. Of the remaining five pools, two (Reo Stott and Society) were surveyed in all three study seasons (1991, 1992 and 1993), while the other three were surveyed in 1992 and 1993 only. Of the five pools described in this report, four were dredged either shortly before or during the study. Tom Lang Pool and SP/Ponderosa were added in 1992 because they had been dredged after the 1991 release.

Each pool was surveyed before and after the trial releases. Precisely the same points were surveyed each time using grid networks consisting either of parallel cross sections (for straight pools such as Society Pool) or multiple rays (transects) extending from monumented points along the banks (see Appendix A for typical survey grids). Survey ropes were stretched along each transect and depths were measured to the nearest 0.1 feet (0.025 m) with a fiberglass survey rod at 5-ft (1.5-m) intervals. The water surface along each pool was assumed to be essentially flat, a reasonable assumption at the low discharges when the surveys were made. In addition, surveys were made before each pool measurement to verify the water surface elevation and insure that it was comparable to previous surveys. Following field measurements, water depth was subtracted from the water surface elevation to obtain bed topography. Illustrative transects are presented in Appendix A. In addition, at each survey point, the substrate was qualitatively identified (as sand, gravel, cobbles, boulders, silt, aquatic vegetation) based on visual appearance and the feel of the rod as it touched the bed.

The survey data were analyzed by determining x-y coordinates for all transect endpoints and converting individual survey points into x-y-z coordinates. These data files were input into digital terrain model and earthworks software from Softdesk, Inc. to produce contour maps of the bed before and after each release and to compute net change in sediment storage. These maps are presented in Appendix A.

To provide a qualitative estimate of fine sediment (< 8 mm) storage over the entire 5.8-mile (9-km) study reach, we floated the reach at 300 cfs (8.5 m^3/s) in two rafts and estimated the percentage of fine sediment on the bed. Areas of uniform fine sediment content were mapped onto enlarged aerial photographs (scale 1:1200) and later planimetered. We made these observations prior to the 1992 release and immediately following the 1993 release. For computation of sediment storage between the pools, we defined six subreaches between Grass

Valley Creek and the Steelbridge study site. The study reaches were typically separated by the major pools. For each of these discrete subreaches, we computed a weighted average fine sediment percentage from these visual estimates. These subreaches are also used to compute the sediment routing discussed in Section 6 of this report.

To compute fine sediment volumes in the study reaches, we assumed that the surface layer was 0.25-ft (0.08-m) thick (approximately equal to one D90 of the bed framework gravel) and that the active bed (which presumably could be flushed) was 0.5-ft (0.15-m) thick. For the top 0.25 ft, we used the visually estimated surficial fine sediment percentage; for the underlying 0.25 ft, we used a constant value of 25%, based on the percent finer than 8 mm in the Poker Bar and Steelbridge bulk sediment samples. The total volume of fine sediment within the study reach was estimated to provide a starting value for the routing calculations discussed in Section 6 of this report.

5. FIELD OBSERVATIONS

5.1 Hydraulics

This section presents the basic hydraulic observations made during the three trial releases. The methods used are reported in Section 4.3 and Table 4.3.1 provides a summary of all velocity observations made during the study. The basic velocity observations are presented in Appendix B.

5.1.a Discharge and Water Surface Elevation

A value of river discharge may be calculated from the results of each velocity traverse. These measurements are a direct estimate of the discharge passing through the study sections, whereas discharge values reported at Lewiston Dam or the USGS gaging station at Lewiston can be somewhat different because of flow additions or subtractions between the dam and the study sites. Hence, our local discharge estimates are used in the later analysis to evaluate the relation between discharge and gravel entrainment at the study sections. Discharge calculations for consecutive days with similar water surface elevation (WSEL) also permit an evaluation of the precision of our velocity measurements.

The discharge values calculated from each velocity traverse are given in part a of Table 5.1.1. A mean value of discharge (marked "estimated" on Table 5.1.1) was calculated from all measurements made during periods when the river was at a constant stage. This value of discharge is carried forward in the later analyses. Part b of Table 5.1.1 presents the statistics describing the range in measured discharge for periods of nearly constant stage. In all cases, the standard deviation of the discharge measurements is less than 2.5% of the mean discharge for that period, suggesting that the discharge estimates, and the supporting velocity observations, are accurate within a comparable range.

The discharge through the Steelbridge study channel was observed to be a nearly constant proportion of the total river discharge. At a discharge of 2700 cfs (76 m^3/s) in 1991, the study channel carried just under 31% of the total discharge. At a discharge of 5800 cfs (164 m^3/s) in 1992, the study channel carried 33% of the total discharge.

On May 31, June 1, and June 2, 1991, velocity observations were made at only the Steelbridge study site. An estimate of the total discharge on those three days is made from the gage height observations at the USGS gage at Limekiln, just downstream from the study reach. A comparison our total discharge observations at Poker Bar, three discharge observations made by USGS personnel at the Limekiln gage in 1991, and the rating curve for the Limekiln gage show that the gage height readings may be used to estimate the total river discharge with good accuracy (Figure 5.1.1).

An important advantage of the field program was that discharge was held constant during periods of observations. This permits the velocity and entrainment observations to be unambiguously correlated with a river discharge. We monitored discharge throughout the study periods by observing WSEL at fixed stations. Figure 5.1.2 presents a WSEL record during the observation periods. In 1991 and 1992, a staff plate was maintained at our base camp in the Steelbridge campground. The trace of this record shows that only minor variations in stage (less than 2 or 3 cm) occurred over the entire period in which discharge was to be held constant. Most of the variation observed occurred at night when no velocity observations were made.

While velocity observations were made at a particular study site, water levels were monitored on staff plates throughout the study reach and with greatest frequency at the two primary study sections: PB2 and SB3C. A trace of WSEL at each study section is given in Figure 5.1.2. A summary of the minimum and maximum WSEL at the study sections for each daily observation period is given in Table 5.1.2. The largest WSEL deviation for an observation period was 3 cm on 5/29/91. On two other days, 6/14/92 and 4/27/93, a total WSEL variation of 2 cm was observed over the period during which measurements were made. All other variations were 1 cm or less, which is also approximately the accuracy with which we could read the staff plates. Each of the three days with a total WSEL variation of 2 cm or 3 cm were followed by days with similar stage and discharge and a smaller WSEL variation during observations. Although all of the variations in WSEL are relatively small, preference is given in the subsequent analyses to days with a total WSEL variation of 1 cm or less.

5.1.b Velocity Observations

Values of depth-averaged velocity U were calculated from multiple observations of velocity along a vertical profile at each station along a cross-section. Each value of U was calculated by fitting a least-squares line of the form

$$u = a + b \ln(z) \tag{5.1}$$

to the set of individual observations of point velocity u and elevation above the bed z at each station. The depth-averaged velocity is then calculated as

$$U = \frac{1}{h}\int_0^h u\,dz \qquad (5.2)$$

where h is the flow depth and the elevation z = 0 is taken to be the elevation where velocity goes to zero. For u defined by Eq. 5.1, the resulting relation for U is

$$U = a + b\ln(h) - b \qquad (5.3)$$

All values of U and h are given in Appendix B.

Plots of U for the principal Poker Bar study section PB2 are given in Figure 5.1.3. Also shown on the plot is the cross-section topography and the WSEL at the different discharges. Because of the large number of velocity traverses made on PB XS2, average values of the velocity observations for the peak flow in 1992, 164 m^3/s, and the 1993 discharge, 80 m^3/s, are given in Figure 5.1.3. The velocities contributing to these averages are shown in Figures 5.1.4 and 5.1.5. The plot of velocity for 4/27/93 and 4/28/93 with Q = 80 m^3/s (Figure 5.1.4) demonstrates the high degree of precision with which velocity observations may be made from wading rods. The velocity observations made from a cable in 1992 (Q = 164 m^3/s; Figure 5.1.5) show more variability, although the mean trend in U across the section may be reliably determined.

Plots of U for each Steelbridge cross section are given in Figure 5.1.6. Also shown on the plot is the cross-section topography and the WSEL at the different discharges. The plot of velocity for 5/31/91 and 6/1/91 with Q = 76 m^3/s shows very little scatter. The velocity observations in 1992 with Q = 164 m^3/s show more variability, although the mean trend in U across the section may be reliably determined. All velocity observations at Steelbridge were made with the current meter mounted on a wading rod.

The main purpose of the velocity observations was to calculate the local bed shear stress acting on the bed. These calculations are discussed in Section 6.2.b.

5.2 Substrate Modification

The goal of our substrate observations was to determine the effect of the trial reservoir release on the streambed sediment composition in potential spawning areas. In this section, we focus on changes in the proportion of fine sediments on the bed surface. This is the sediment that the reservoir releases are intended to flush.

Our observations of substrate modification are summarized using two types of data plots. The first is a longitudinal profile of the study reach showing for each cross section the pre and post release percent finer than 8 mm (from pebble counts) and the percent embedded (from an average of up to 30 visual observations along each section). The second plot, made for the 1992 and 1993 releases, is a cross sectional view of the principal study section at each study reach and presents the percent finer than 8 mm from visual observations, pebble counts, and bulk samples before and after the release.

For the visual observations and pebble counts, differences of less than 10% between pre and post-release observations at individual locations fall within the error associated with this type of sampling and should not be considered significant. Differences of less than 10% for the average of multiple samples may be more indicative, but should be considered relatively weak evidence for a true change in the sand content of the bed.

5.2.a Poker Bar

1991 Trial Release. The 1991 trial release did not significantly modify the substrate at the Poker Bar study site. This is indicated by very small changes in the proportion of fine sediments on the bed surface. Figure 5.2.1 plots the proportion of fine sediments from pebble counts versus distance downstream. Observations were made along four cross sections PB0A, PB1B, PB2, and PB2A. The average decrease in the percent of sediment finer than 8 mm is 9%, almost all of which was measured at PB2A (Table 5.2.1).

1992 Trial Release. The 1992 trial release produced a reduction of fine sediment in the Poker Bar study reach. Every cross section showed a decrease in surface fine sediments after the release. Figure 5.2.2 plots the proportion of fine sediments versus distance downstream before and after the 1992 release. Pebble counts were not done downstream of PB2A because the water was too deep. Reach averaged decreases (PB0A-2A) were 12% for pebble count data and 15% for visual estimates (Table 5.2.1).

Figure 5.2.3 summarizes all of the substrate observations at Section PB2 for the 1992 release. The bed elevation changed very little across this section and at other cross sections, demonstrating that large scale scour and deposition did not occur in the study reach during the 1992 release. Changes in bed elevation are generally less than the diameter of one coarse D_{90} clast. Both the pebble counts and the visual observations show a decrease in fine sediments on the bed surface at PB2. The mean proportion of sediments finer than 8 mm decreased from 27% to 13% from pebble counts and decreased from 31% to 25% from visual observations.

Figure 5.2.3 also shows the changes in the proportion of fine sediments in the bulk sediment samples taken before and after the release.. The post-release samples that incorporate pre-release samples show a slight increase in the proportion of material finer than 8 mm after the release. However, the two nearby post-release samples (taken to control for resampling effects) had smaller values of percent finer than 8 mm, suggesting that the initial disturbance created by collecting pre-release samples may be responsible for a slight increase of fine sediment infiltration during the release, perhaps because the sampled gravel has a looser texture. Also shown on Figure 5.2.3 are the locations and amount of scour produced at each of the three tracer gravel sites. This information will be described in Section 5.3 in the discussion of gravel movement.

Because the bed surface samples suggest a decrease in material finer than 8 mm during the 1992 release, whereas no decrease in fine material is evident in the bulk samples, which include both surface and subsurface sediments, it appears that the flushing provided by the 1992 release at Poker Bar occurred primarily on the bed surface and that the release was not of sufficient duration to permit detectable flushing at depth.

<u>1993 Trial Release.</u> The 1993 trial release was of the same magnitude as the 1991 release, although its duration was considerably longer. Both visual and pebble count measures at seven sections (Figure 5.2.4) and bulk samples at PB2 (Figure 5.2.5) show that no significant changes in the proportion of fine sediments on the bed surface was observed as a result of this release.

<u>5.2.b Steelbridge</u>

<u>1991 Trial Release.</u> The 1991 trial release did not produce significant substrate changes at the Steelbridge study site. The pebble count observations indicate a substantial decrease in material finer than 8 mm only at SB2, which is a narrow, cobble-bedded channel with relatively high

velocities. Essentially no change in the proportion finer than 8 mm is evident at the spawning gravel sections between SB3B to SB4.

1992 Trial Release. The 1992 trial release produced a reduction of fine sediment through more of the Steelbridge study reach than the 1991 release, although little reduction in fine materials occurred in the spawning gravels at the downstream end of the reach. Nearly every cross section shows a decrease in surface fine sediments in the pebble counts; the visual observations show a decrease in fine content for the upstream seven sections (Figure 5.2.7). The two methods show only a slight changes in fine content for sections SB3C-4, with a small decrease indicated by the pebble counts and a small increase indicated by the visual estimate (Figure 5.2.7). Reach averaged decreases (SB2-4) were 11% for pebble count data and 8% for visual estimates (Table 5.2.1). These results are similar, but slightly smaller than those at Poker Bar.

The change in percent finer than 8 mm for both surface and bulk samples at SB3C are shown in Figure 5.2.8. The visual observations show a slight increase in fine material, whereas the point counts show a slight decrease. The bulk samples show negligible changes in the proportion of fine material, suggesting that little surface flushing occurred and that, as at Poker Bar, little or no flushing occurred at depth.

5.3 Gravel Movement

Removal of fine sediment from below the bed surface requires entrainment of gravel clasts forming the surface. Flushing at depth is desirable in that a larger volume of sand may be removed from the reach. The associated gravel transport also represents a problem, however, because gravel recruitment to the study reach is severely limited, so that downstream gravel transport depletes the very resource that the flush is intended to maintain. The frequency of gravel entrainment is determined from the proportion of tracer gravels moved during a flush. The rate of gravel transport rates is determined from Helley-Smith sampling in 1992 and from gravel traps in 1993.

5.3.a Poker Bar

Very little gravel movement occurred at Poker Bar during the 1991 trial release. Virtually all of the tracer gravels remained in place throughout the flush (Table 5.3.1). The depth of scour, calculated as the proportion of tracer gravels removed multiplied by the depth of tracer gravel installation, was less than 4.0 cm for all five tracer gravel sites. This depth corresponds roughly to the median grain size of the gravel fraction and represents only slight movement of the finer clasts on the bed surface. The combination of discharge and duration during the 1991 release is not sufficient to provide flushing at depth.

Substantially more gravel entrainment occurred during the 1992 trial release. Grains from all gravel size classes were entrained (Table 5.3.2). Scour depths for the three tracer installations along PB2 were 10 to 13 cm, which corresponds to the largest few percent of the clasts found in the bed. Entrainment to this depth can permit subsurface flushing to a depth of 15 to 20 cm.

Gravel transport rates were measured during the 1993 release using five boxes set into the bed with the upper edge flush with the bed surface. The boxes acted as efficient traps for grains coarser than 8 mm, although the finer sand grains tended to be swept out of the box. Sediment accumulated in the traps from the start of the release. The volume and grain size of trapped material was estimated after 336 hours and the boxes were then swept clean. Sediment continued to accumulate in the boxes until the end of the release, providing a second sample with a duration of 68 hours. The accumulated sediment was then removed by hand, weighed and sieved. The second sample is more reliable because the quantity of sediment in the traps was directly measured. The transport rates for material coarser than 8 mm is very consistent for all traps and both sample periods (Table 5.3.3). The largest of the ten samples is three times the smallest. A mean gravel transport rate of 0.01 g/ms is used in the later analysis to represent the gravel transport rate at 80 m^3/s.

Transport rates were measured during the 1992 release using a large Helley-Smith sampler with a 6 inch orifice. Because of the weight of the sampler and the greater depths and velocities during this release, the sampler was deployed on a crane-operated cable. These transport samples are not of the same quality as the 1993 sediment trap samples, because the cable deployment prevents precise control of the sample location and visual observation of the sampler was not possible for some of the samples. We focus here on the samples made across a 15 m portion of PB2 that incorporates the primary spawning gravels and covers the same range as the trap samples in 1993.

The 1992 samples show considerably more variability in transport rate than the 1993 samples, although the range of observed rates is not unusual for field observations of bedload, which vary substantially in space and time, even under steady flow conditions. Mean unit transport rates were calculated for each station. For a discharge of 103 m^3/s on 11 June, the largest mean transport rate for a station was 30 times that of the smallest mean transport rate

(Table 5.3.4). For the larger number of samples made at a discharge of 164 m^3/s, the range in mean transport rates is much smaller, with the largest mean transport rate for a station being 3.4 times that of the smallest. The mean transport rate is 24.2 g/ms for Q = 103 m^3/s and 1106 g/ms for Q = 164 m^3/s.

Helley-Smith samplers may collect samples that are either larger or smaller than the true transport rate. The former occurs when the sampler scoops immobile bed material; the latter when the base of the sampler is not flush with the bed surface. As a crude correction for both errors, transport rates were calculated using only one-half of the samples, excluding the largest 25% and smallest 25% of the observed transport rates. These values are presented as "censored transport rates" on Table 5.3.4. The censored mean for Q = 103 m^3/s is 3.6 g/ms, which is roughly seven times smaller than the mean calculated using all samples. The censored mean for Q = 164 m^3/s is 106.1 g/ms, which is very close to the mean calculated using all samples. Both the total and censored mean transport rates are carried forward to the flushing analysis given in Section 6 of this report.

5.3.b Steelbridge

Very little gravel movement occurred at Steelbridge during the 1991 trial release. Virtually all of the tracer gravels remained in place throughout the flush (Table 5.3.5). The depth of scour was less than 6.0 cm for all six tracer gravel sites. This depth corresponds roughly to the median grain size of the gravel fraction and represents only slight movement of the finer clasts on the bed surface. The combination of discharge and duration during the 1991 release is not sufficient to provide flushing at depth.

Substantially more gravel entrainment occurred during the 1992 trial release. Grains from all gravel size classes were entrained (Table 5.3.6). Scour depths for three of the four tracer installations along SB3C were 8 to 11 cm, which corresponds to the roughly the 85th percentile of the clasts found in the bed. Entrainment to this depth can permit subsurface flushing to a depth on the order of 15 cm. No entrainment was observed at the fourth tracer gravel location because fresh gravel was deposited on top of the tracers during the flush. Although no entrainment of the initial marked grains occurred at this site, the depth of mobilized gravels at the end of the release was comparable to that at the other three installations.

5.4 Sand Movement

An evaluation of the sand-removal efficiency of different sediment maintenance flow options requires an estimate of the rate at which sand is transported at different discharges. We made Helley-Smith sampling transects at the Poker Bar study section at three different discharges. The sample locations, sample duration and measured unit transport rates are given in Table 5.4.1. Total sand discharge through the cross section was calculated using the point observations of transport rate per unit width multiplied by the appropriate length of section between sampling locations. The resulting sand discharge rates, in tons per day, are 34,400 at $Q = 80\ m^3/s$, 112,400 at $Q = 103\ m^3/s$, and 223,600 at $Q = 164\ m^3/s$.

In a gravel-bed river, the sand transport rate will vary not only with discharge, but also with the proportion of sand present on the bed. In general, the sand discharge must be calculated as $Q_s = P_sQ_{sc}$, where Q_s is the actual sand discharge, P_s is the proportion of sand on the bed, and Q_{sc} is the maximum sand discharge, which occurs when the bed is entirely covered by sand ($P_s = 1$). The rate at which sand can be transported by a flushing release will decrease directly with the proportion of sand on the bed, as the reach becomes flushed of sand. During the 1992 release, some reduction of sand occurred in the reach immediately upstream of the section used for transport sampling. Further variation in the proportion of sand on the bed remained within our observation capabilities during the 1993 flush. The proportion of sand on the bed in the reach immediately upstream of the sampling section generally falls within 10% and 30% (Figures 5.2.2 through 5.2.5). In Section 6, we use a sand proportion of 22% to scale the observed transport rates in developing a sand discharge rating curve.

5.5 Pool Surveys

Table 5.5.1 provides a summary of the computed volume changes in and around each pool for each flow year measured during the study. The summary includes cut and fill volumes, and net volume change for each pool. The maps depicting pool topography pre- and post-release along with a contour map of net volume change are presented in Appendix A. The longitudinal patterns of cut-and-fill changes within each pool survey area (shown in Figures 5.5.1 through 5.5.5) clearly illustrate the filling of dredged reaches and suggest other scour and fill events along the length of the pools.

The following sections provide a summary of field observations of the pools and the reaches surveyed immediately upstream and downstream. In Section 6.4 of this report, the depth of fill and scour within particular pools is analyzed to develop guidelines for the dredging depths needed to effectively trap sediment in the pools.

SP/Ponderosa This study site consists of two pools (the SP and Ponderosa pools) with an intervening run, and is located at the first bend downstream of the confluence of Grass Valley Creek (Figure 4.6.1) (see Appendix A, p. A01 for location of the individual transects). The reach is approximately 1100 feet (335 m) long in total, of which about 400 feet (122 m) is occupied by the run separating the two pools. Bedrock is exposed in places along the right bank in the run area, but is not a dominant feature of the pools. The lower pool (Ponderosa Pool) was dredged between the 1991 and 1992 flow seasons. It is not known what volume of sediment was removed during the dredging operations. Changes observed following the 1992 release included scour at two locations in the SP pool area with some fill in between, relatively minor changes in the run area, and significant fill in the dredged portion of Ponderosa pool (Figure 5.5.1, Appendix A, p. A02-A03). In addition, visual observations of substrate change showed a dramatic reduction in the amount of "muck" and aquatic vegetation that had occupied areas along the left bank (inside of the bend) in the run area. The overall computed net volume change in this study site was a cut of 516 yd^3 (395 m^3; Table 5.5.1). In 1993, with a nominal release of 3000 cfs for 17 days, the measurements showed a net cut of 1095 yd^3 (837 m^3). In this case, the volume change was relatively evenly distributed over the entire site, with slightly more scour at the two pool areas.

It was clear that while overall "flushing" of this site had occurred, the effects were limited to those areas with appropriate local hydraulic conditions. For example, in 1992, a sand bar developed along the left bank in the first 200 ft (60 m) of the site. This deposit was clearly related to the strong eddy present along the left bank which develops as the higher velocity flow moves towards the outside of the bend slightly downstream.

Tom Lang Pool This pool lies about 500 ft (150 m) upstream of the Poker Bar detailed study site (Figure 4.6.1). This pool was dredged prior to the 1992 release, and was added to the study in 1992 to obtain additional information on the response of dredged pools to flushing flows. The pool itself is about 400 ft (120 m) long, while the entire site is about 900 ft (275 m) in length. The pool occupies a relatively straight reach of the channel with no prominent bedrock exposures. When the pool was dredged prior to the 1992 release, a clean gravel/cobble berm was created at the downstream end of the pool. This feature is evident in the pre-1992 topography to the left of middle of Appendix A, p.A05.

The 1992 release caused substantial changes to the post-dredge geometry (Figure 5.5.2), with fill of 1-4 ft (0.3-1.2 m) in the central portion of the pool, the largest deposits just upstream of the gravel berm. The net volume change for the site in 1992 was a fill of 885 yd^3 (677 m^3). Scour occurred at the upstream end of the dredged pool, at places along the pool margin, along the left bank downstream of the pool where the berm deflected high flows towards the opposite bank, and at points along the thalweg in the lower portions of the site. About 1300 yd^3 (1000 m^3) were deposited within the pool during the 5-day release in 1992. The 1993 release of lower magnitude but longer duration (17 days) caused a net cut of about 1038 yd^3 (794 m^3) in the overall survey area, although most of the net change occurred downstream of the pool.

<u>Reo Stott Pool</u> This pool is located about 2000 ft (610 m) downstream of the Poker Bar study site (Figure 4.6.1). A large bedrock obstruction is responsible for this pool, which is small in extent but whose depths exceed 20 ft (6 m) in places. Flow entering the pool passes over a sharp ledge into deep water, and is then directed towards bedrock along the left bank, creating two large eddies to either side (Appendix A, p.A07-A10). Measurements in the main portion of the pool were made difficult by the combination of current and depth.

The 1991 release resulted in a volume change of 129 yd^3 (99 m^3) of net scour. Figure 5.5.3 shows the longitudinal incremental volume change, which indicates that net scour occurred in the deeper parts of the pool with a net fill along the right bank near the downstream end of the pool. This fill was sand deposited in a well-developed eddy along the right side of the channel. Modest amounts of net scour also occurred downstream of the pool.

An unknown amount of material was dredged from the pool between the 1991 and 1992 seasons. In the absence of significant storm flows during the winter, the pre-1992 survey depicts the topography following dredging operations. The 1992 release resulted in a net fill of 487 yd^3 (372 m^3), almost all of which was in the dredged area. Up to 7 ft (2 m) of fill was deposited in the dredged area near the right bank, again in the location of the large eddy.

The longer 1993 release caused 414 yd^3 (317 m^3) of net scour, most of which occurred at the downstream end of the pool and in the run downstream. The thalweg deepened as the channel shifted toward the right bank.

<u>Society Pool</u> This pool, also referred to as the Poker Bar Pool (USFWS 1990), is located at the downstream end of Poker Bar adjacent to a number of riverside residences (Figure 4.6.1). The pool occupies a straight reach of the river, widening slightly as it continues downstream.

USFWS personnel observed significant deposition of sand in this pool between 1986 and 1990, with much of its original volume filled with sand. The Trinity River Restoration Program dredged about 10,780 yd^3 (8243 m^3) from this pool in 1990 (USFWS 1990), leaving a steep scarp about 7 ft (2 m) high between the dredged pool and the riffle downstream (Appendix A, p.A12). This pool was mapped before and after all three study flows. Its linear nature allowed its geometry to be well defined with a simple grid of parallel transects perpendicular to the flow direction (Appendix A, p. A11).

The 1991 release was the first significant flow following pool dredging, and thus the pool had its highest trap efficiency. The 1991 release resulted in a number of changes to the pool (Figure 5.5.4). The deepest dredged areas filled 1 to 4 ft (0.3-1.2 m) (Appendix A, p.A12) creating a large uniform surface between elevations 91 and 92 ft (arbitrary datum), while the transition to the downstream riffle degraded from 1 to 3 ft (0.3-0.9 m). Much of the sand fill over the cobble riffle downstream was removed during the release. Although there was net fill within the pool, the 1991 flow resulted in a net cut of 160 yd^3 (122 m^3) over the entire survey area, because the downstream scour in the easily moved sand exceeded the fill from sand moving downstream (Figure 5.5.4).

In contrast, the 1992 flow produced a net cut of 1874 yd^3 (1433 m^3). All of the 1991 deposit was removed and considerably more of the bed and sides of the pool were scoured (Appendix A, p.A13). A deep area at the downstream end of the pool that was not filled in 1991, filled in 1992, despite net scour almost everywhere else, presumably filled with sand scoured from upstream.

The 1993 study flow resulted in only minor changes to the pool topography: a net cut of 77 yd^3 (59 m^3). Net cut occurred in the upper portions of the pool and in the transition to the riffle, while net fill occurred in the central areas (Figure 5.5.4).

Upper Steelbridge Pool Between Society Pool and Steelbridge there are no large pools and few pools in general. Access is difficult for most of the reach. Upper Steelbridge pool is located at the upstream end of the Steelbridge campground, and immediately upstream from the Steelbridge study site. The pool was selected principally because it has good access, because it was the last significant pool in the study reach, and because it is a natural (undredged) pool. Due to difficult currents in the upper part, only the lower two-thirds of the pool were mapped, using fifteen ray transects (Appendix A, p.A15). The pool was mapped in 1992 and 1993.

The 1992 flow resulted in a net removal of 167 yd^3 (128 m^3). As shown in the longitudinal distribution of net volume change (computed in 50 ft increments), the largest net volume change occurred in the lower third of the pool (Figure 5.5.5). Appendix A, p.A16 shows that 1 to 3 ft of cut occurred near the right bank, while between 1 and 2 ft of fill was deposited in areas along the left side of the pool.

In contrast, the 1993 flow of lower magnitude and longer duration, caused a net cut of 551 yd^3 (421 m^3), mostly towards the downstream end of the pool (Figure 5.5.5). The topographic mapping indicates that a channel was scoured from the deepest part of the pool along the left side and that the deepest part of the pool (depths up to 10 ft or 3 m)) was enlarged in area.

5.6 Surveys of Surficial Fine Sediment in the Study Reach and Sand Budget

Sand in the Channel of the Study Reach. Estimates of fine sediment within the study reach from Grass Valley Creek to Steelbridge Campground were made prior to the 1992 study flow and following the 1993 study flow. Reach weighted values for the six sub-areas were computed by expressing individual areas as a fraction of the total reach area and summing. Prior to the 1992 flow, weighted reach values of percentage coverage by fine sediment (< 8 mm) varied from 13.6% to 43.5%. Reach estimates following the 1993 release showed a substantial reduction in the reach weighted percentage fine sediment coverage (Table 5.6.1). In general, the reaches with lower amounts of fine sediment prior to the study flows showed less reduction of surficial fine sediment than the areas with initially high percentages of surficial fine sediment. This occurs because the sand discharge rate at any section depends directly on the proportion of sand present on the bed surface, so reaches with less sand will have a correspondingly smaller sand discharge rate.

The percentage of channel area falling within each class of estimated fine sediment percentage is plotted in Figure 5.6.1 for 1992 (pre-release) and 1993 (post-release). This plot shows an increase in area of channel bed with only 10 percent surficial fine sediment, and a reduction in the area with 50 percent covered by fine sediment.

The overall change can also be viewed on a plot of the cumulative frequency of occurrence (in areal extent) of various classes of fine sediment percentage (Figure 5.6.2). Note that this is not a standard cumulative size distribution plot, in which coarser sediment mixtures would plot to the right of finer sediments. On Figure 5.6.2, the 1992 data plot to the right of the

1993 data, indicating that for a given class of fine sediment coverage (such as 30 percent fine sediment coverage), more of the bed area was that clean or cleaner in 1993. Another way to look at the plot is to read the fine sediment content associated with a given percentile of bed area. For example, in 1992, 50% of the bed had 20 percent or less fine sediment. The bed was cleaner in 1993, with 50 percent of the bed having 15 percent or less fine sediment.

Sand Budget. An estimate of the volume of sand in each subreach before the 1992 release and after the 1993 release is given in Table 5.6.2. Also shown on the table are the net change in sand volume in both channels and pools for the same period. This sand budget (computed for the period from before the 1992 release to after the 1993 release) shows the net effect of both releases was removal of about 6300 cubic yards (9400 tons) from the study reach.

Based on the visual estimates of sand in the channels in the study reach, we estimate that there was approximately 4,000 yd^3 of sand in the surface layer of the bed (assumed to be 0.25 ft., or 76 mm thick; Table 5.6.2) following the 1993 release. This is the sand quantity used in the investigation of sand routing developed in Section 6.5 of this report. For the purposes of determining a flushing release that will also remove sand from the subsurface, a quantity of sand in the subsurface of the study reach must also be estimated. A mean value of 25% sand in the subsurface is assumed, based on the bulk samples at Poker Bar. Using a subsurface layer with a thickness of 0.25 ft., (76 mm), giving a total flushing depth of 0.5 ft. (152 mm), the volume of sand in the subsurface is 4840 yd^3, and the total quantity of sand requiring flushing from the study reach of 8850 yd^3.

6. ANALYSIS

6.1 Overview of Objectives and Approach

Trial flushing flow releases on the Trinity River below Lewiston Dam were made in 1991, 1992, and 1993 to evaluate the effectiveness of high flows in restoring fish habitat. The objectives of this project were to document the effectiveness of these releases in removing fine-grained sediments from the channel bed and to develop recommendations for future flushing releases to clean and maintain the potential spawning gravels along a reach 13 mi (21 km) below Lewiston Dam.

In this section of the report, we present the analysis leading to our recommendations for flushing releases. To begin, it is useful to define the objectives as clearly as possible and to present the different options available to achieve those objectives.

Objectives

The general objectives of cleaning and maintaining potential spawning gravels in the Trinity River may be defined in terms of specific objectives concerning the fine and coarse portions of the sediment bed.

Sand Objectives: Maximize sand removal, using the smallest amount of water. Fine-grained sediment has deposited within the river gravels under the decreased post-dam flow regime on the Trinity River. The fine-grained sediment has a negative impact on salmonid spawning success, primarily by blocking the emergence of alevins from the bed. The sand objective of a flushing release is to use a portion of the available water to remove as much sand as possible from the river bed. Because of the value of the water used for flushing, a related objective is to use the smallest volume of water that will provide adequate cleaning and maintenance of the gravels.

Gravel Objectives: Mobilize the gravel bed surface, while minimizing gravel transport. Mobilization of the gravels composing the river bed occurred on a regular basis under pre-dam conditions, but occurs only rarely with post-dam regulated flow. Some mobilization of the gravels on and below the river bed surface is desirable in order to permit removal of sand from below the bed surface and to maintain some looseness in the bed structure, which aids in the spawning process. One gravel objective, therefore, is to achieve a minimum threshold of gravel movement. At the same time, however, recruitment of new gravel to the river reach is largely eliminated through sediment trapping in Trinity Lake. To avoid excessive loss of the gravel resource, a flushing release should involve a minimum of downstream gravel transport. The net result is that the flushing release for achieving the gravel objectives is tightly defined: it should be no larger than that just sufficient to permit flushing of sand from the bed subsurface and to maintain the gravels in a loose state.

Options

The primary variables that can be managed to address the flushing objectives are (1) the total volume of water used in flushing, (2) the discharge, or rate at which reservoir water is released, and (3) the volume of sediment that is dredged from pools (to act as sediment traps) along the river reach.

Release Volume. It may be expected that the amount of sand that can be removed by a flushing release will increase directly with the volume of water used. The amount of water available for flushing depends on a number of issues beyond the scope of this report, and our goal here is to primarily to identify the relative volumes of sand that can be removed at different discharges and to evaluate the efficiency with which different volumes of water should remove sand.

Release Discharge. The rate at which both sand and gravel are transported increases strongly with discharge. Higher discharges will remove sand more efficiently, but at the expense of more rapid gravel loss. Our goal here is to investigate the trade-off between these two transport rates in the context of finding an optimum combination giving sufficient sand removal with a minimum of gravel loss and water use.

Dredged Pools. Removal of sediment via a flushing release is one-dimensional: removal occurs only at the downstream end of a river reach. The transport rate at this section represents a bottleneck that limits the rate at which sand can be removed from a reach. By providing intermediate traps for transported sediment, pools act to decrease the effective reach length from which sediment must be removed by the flow. If the pools are dredged following a flushing release, multiple exits for the sand are introduced, thereby increasing the quantity of sand that may be removed from the river with a given volume of water. By increasing the sand removal efficiency, pools offer the potential of decreasing both the water used in a flushing release and the downstream loss of gravels, assuming that the gravels dredged from the pools are returned to the river channel. The volume of sand that may be removed through pool trapping and dredging will depend on the number, depth, and plan area of the dredged pools. When dredged pools are used to assist in sand removal, the river reach that will be the most difficult to clean will typically be that with the greatest length between pools. By placing pools in the longest reaches currently without pools, the sand removal benefit of a volume of flushing water will be distributed most evenly. We evaluate the effectiveness of the number, depth, and placement of pools in providing additional sand removal for a given volume of water.

Release Timing. In addition to the water volume and discharge, the timing of a release must be specified. The timing of a flushing release has little effect on the relations among discharge, gravel entrainment, and sand removal from the river bed, but may have important impacts on the viability of active Salmonid redds and on the continued deposition of fine-grained banks along the river. The choice of a release period is considered separately from the specification of a discharge volume and magnitude and is based primarily on our analysis of the post-dam history of water and sediment discharge on the Trinity River and Grass Valley Creek.

Approach

There are two basic tasks required to evaluate the different flushing options. The first is to determine the discharge required to produce a minimum threshold of motion for typical spawning gravels in the study reach. This analysis is based primarily on our observations of local flow and entrainment of potential spawning gravels at the detailed study sites at Poker Bar and Steelbridge. These observations are compared with general empirical and theoretical relations for gravel motion in developing recommended values of discharge to achieve a gravel motion threshold. The amount of gravel motion depends on both the flow rate, or discharge, and the duration of the release. Because the volume of released water and the discharge determine the release duration, the result of the analysis is a recommended threshold for gravel motion as a function of discharge and water volume.

The second task is to calculate the sand removal and downstream gravel transport for different combinations of water volume, discharge, pool dredging. To do this, we divide the study reach into subreaches and use a sand routing formulation to calculate sand removal as a function of water volume, discharge, and pool dredging. The sand-removal efficiency can then be evaluated in terms of water use, gravel loss, and dredging volume. The starting point is the quantity of sand presently in the study reach. Our estimate of this quantity is developed in Section 5 of this report. The routing algorithm and subsequent evaluation require relations among bed sediment composition, water discharge, sand and gravel transport rates, and sediment trapping by pools. The transport and pool trapping relations are developed in separate sections, followed by a presentation of the sand routing model and results.

Defining the bed sediment

The flushing objectives are stated explicitly in terms of two-part bed size distribution: a release is to remove fine-grained sediment while just achieving a threshold transport rate for coarse-grained sediment in the river bed. Calculation of sediment transport rates and evaluation of the various flushing objectives requires a clear definition of these sediment sizes. Further, because the size distribution of the sediment in the river bed is not, in reality, a mixture of two different grain sizes, but rather a continuous distribution of sediments ranging in size from a small fraction of a millimeter to greater than 200 mm, it is necessary to consider the rationale and feasibility of simplifying the bed sediment into a two-component distribution.

The definition used here divides the bed sediment at a grain size of 8 mm. Sediment finer than 8 mm is generally light colored and derived from decomposed granitic terrain, particularly in the Grass Valley Creek watershed. This material comprises roughly 30% of the bed at Poker bar and 23% of the bed at Steelbridge. The median grain size of this material is roughly 2 mm at both sites; approximately 75% of the fine-grained sediment is in the 1 mm to 8 mm range and 90% of the fine-grained material is coarser than 0.5 mm at both sites. The sediment coarser than 8 mm is predominantly dark-colored rock fragments of metamorphic and volcanic origin. The median grain size of the coarse fraction is 36 mm at Poker Bar at PB and 56 mm at Steelbridge.

The two size fractions are distinct not only in color, but also in their roles in salmonid spawning success. The coarser grains are the gravels used by salmonids for spawning, while the fine-grained sediments, when intruded into the river bed, can pose two problems. A large concentration of fine sediments within the spawning gravels can decrease intragravel flow, thereby reducing the removal of metabolic wastes and lowering dissolved oxygen within the gravels, causing the incubating eggs to suffocate. This problem is commonly associated with sediments finer than 1 mm. Fine sediments can also block the emergence of alevin from within the spawning gravel beds. This problem is commonly associated with grains between 1-8mm. The relatively coarse size distribution of the intruded sediment in the Trinity River suggests that decreased salmonid spawning success is due primarily to impeded alevin emergence. This is supported by the observation of dead alevin, but no suffocated eggs, in excavated redds on the Trinity River.

The fine and coarse grained sediments described here are described locally, and, on occasion, in the fishery literature, as sand and gravel, respectively. For consistency, we will maintain this usage when discussing flushing objectives in a general context. We will not, however, use these definitions when discussing details of the transport processes, because the sand-gravel size transition is almost universally taken to be 2 mm in the earth science and engineering communities.

The objective of removing fine-grained sediment from the river bed makes the purpose for dividing the sediment into two fractions clear. That such flushing may be feasible on the Trinity River is supported by the field observations at lower discharges, when measurable transport of sand occurs in the presence of no detectable movement of gravel. The feasibility of flushing at higher discharges, particularly if gravel transport is to be minimized, is a function of the sand and gravel transport rating curves to be developed in later sections. The feasibility of flushing finer fractions is also supported by laboratory observations of incipient motion in sediments with a very wide and bimodal size distributions, which show that substantial transport of the finer portions in the bed may occur in the presence of very little transport of the coarser fractions (Wilcock, 1992a, 1993).

Although the motivation for treating the bed sediment as a two-part size distribution is clear, some consideration is needed of the utility of even smaller divisions of the bed sediment. The actual grains in the river bed fall along a continuous scale from a tiny fraction of a millimeter to greater than 200 mm. Grain entrainment and transport is known to vary strongly with grain size. Transport modeling in cases where differential transport rates are an objective, as in the flushing case, can be based on many size fractions (e.g. Stand, 1981). In this case, however, such an approach is neither practical nor necessary.

Calculation of fractional transport rates requires specification of the complete grain-size distribution on the bed surface and in the subsurface. Such grain-size information is not available for nearly all of the reach under consideration. The information available is visual observations of the proportion of the bed surface finer than 8mm throughout the reach and detailed grain-size information at sample locations along four particular cross-sections. To calculate fractional transport rates, the size distributions of sediments throughout the study reach would have to be assumed and the values chosen would have a strong influence on the results. The uncertainty associated with both the grain-size estimates and the calculated transport rates increases with the number of fractions used. The most reliable estimates are likely to be those that use the smallest useful number of fractions which, in this case, is two.

The transport rate of the fine material may be considered to be supply limited: if the gravel matrix of the bed is entrained only sporadically, the transport rate of the fine material will be controlled by the amount available for transport on the bed surface. The transport rate of the fine material is not particularly sensitive to the details of the size-distribution of material finer than 8mm. At flows capable of moving the gravel, the bed shear stress is sufficient to produce general motion of the finer fractions and it is likely that all of the sand fractions are transported at comparable rates, when scaled by their proportion on the bed surface. This is supported by the lack of change in the grain-size distribution of the sand fractions over the 1992 flush (Figures 3.2.3 and 3.2.4). It is also consistent with flume and field observations of mixed-size transport, which show that the proportion of sand in transport approaches its proportion on the bed surface for shear stresses equal to roughly twice that necessary to initiate motion (Wilcock and McArdell, 1993). If the transport rates of the finer sizes are comparable, the total transport rate of the fine materials can be computed based on their total proportion in the bed, allowing direct evaluation of the flushing objective of removing material finer than a cutoff size.

6.2 Hydraulics

6.2.a Stage-Discharge Relations

Water surface elevations were observed at both study sites during the trial releases to provide a basis for predicting the change in flow depth with discharge (stage-discharge curves). Water surface elevations were observed at 11 sections at Poker Bar during all three releases and at 10 sections at Steelbridge during the 1991 and 1992 releases. These data were then used to calibrate a one-dimensional flow model for each study site so that water surface elevations could be predicted for discharges other than those directly observed.

The primary utility of the one-dimensional flow modeling arises from the fact that water surface elevations can be predicted with a high degree of accuracy when calibration observations are available over a wide range of flows. Predictions of the height of the water surface provided an interim benefit in planning the logistics of the field work during each flush. More significantly, they permit the correlation of observations of local flow and transport with a property of the flow at a cross-section, the depth, rather than with the total discharge. This is used to develop rating curves for shear stress over the spawning gravels as a function of discharge at the two primary study sites.

One dimensional flow models were developed for both the Poker Bar and Steelbridge study sites using HEC-2, the US Army Corps of Engineers' Water Surface Profiles Model. HEC-2 is a computer program for calculating water surface profiles for steady, gradually varied flow in natural or man made channels. It was developed by the US Army Corps of Engineers, Hydrologic Engineering Center in Davis, CA in 1964 and has been continuously maintained and updated since that time. The computational method used by HEC-2 is based on the Standard Step Method. In this method, water surface elevations and other pertinent hydraulic parameters are calculated by iterative solution of the one-dimensional energy equations with friction losses evaluated by the Manning equation. Complete documentation on the use of HEC-2, including input data preparation and summary of modeling capabilities, is contained in the HEC--2 User's Manual (Hydrologic Engineering Center, 1990).

Running HEC-2 requires preparation of a data set representative of the river reach under examination. The data set contains information on the stream geometry, channel roughness, and flow conditions. After the data set is prepared, it can be calibrated using known water surface elevations to verify that stream roughness and flow conditions have been properly evaluated. Preparation and calibration of the HEC-2 Data sets for the Steelbridge and Poker Bar study sites is detailed in the appendix.

The Poker Bar flow model was calibrated using observations for six different discharges during the three trial releases (Table 6.2.1). A series of HEC-2 runs was then made to estimate water surface profiles for discharges at regular intervals up to 241 cms (8500 cfs). The resulting water surface profiles are depicted in Figure 6.2.1 and tabulated in Table 6.2.2.

Producing rating curves for the Steelbridge study site was more complicated than the Poker Bar site. At discharges above approximately 100 cms, the island at Steelbridge starts to become inundated. During the peak flow of the 1992 release (164 cms), a majority of the island was submerged. At higher discharges, the island becomes completely inundated and the assumptions used to assemble the HEC-2 data set for the Steelbridge site are no longer valid. High flow projections for the site were developed with a second data set, in which the main and side channels were merged into one continuous channel with the island forming a high point in the stream cross sections. This second data set, referred to as the "High Flow" data set, was calibrated using observations made during the peak flow of the 1992 release.

The Steelbridge flow model was calibrated using observations for five different discharges during the 1991 and 1992 trial releases (Table 6.2.3). A series of HEC-2 runs was then made using the original data set to estimate water surface profiles for discharges of 170 cms (6000 cfs) and less. The High Flow Data set was used to project the water surface profiles for

170 cms (6000 cfs) and greater discharges. Both sets of water surface profiles are depicted in Figure 6.2.2 and tabulated in Table 6.2.4.

6.2.b Local Bed Shear Stress

A principal objective of our field work during the flushing releases was to determine the bed shear stress acting on the spawning gravels at the two primary study sections. This information, together with our observations of the transport of these gravels, provide the basis for extrapolating the gravel entrainment observed during the trial releases to other discharges. Because the bed shear stress varies widely across even a single river cross-section, and because gravel transport is very sensitive to shear stress, particularly at low values of transport rate, it is important to correlate the observed gravel entrainment with values of shear stress directly over the spawning gravels. This local coupling between stress and transport also provides an opportunity to compare our field observations to similar observations in other rivers and in flumes, which allows our relations for gravel entrainment to be compared to general relations developed from experiment or theory. This corroboration provides support for our estimates of gravel entrainment at flows other than those directly observed.

The velocity observations presented in Section 5 were used to determine values of local bed shear stress. Based on our evaluation of different methods for estimating the local shear stress τ_{ol}, we found that the most accurate estimate could be obtained using the local flow depth and local depth-averaged velocity in a flow resistance formula (Wilcock and others, 1994). This expression requires a value for the hydraulic roughness of the bed, which was determined by minimizing the difference between the depth-averaged velocity estimates of τ_o and estimates of τ_o made from the slope of the velocity profile in the portion of the flow near the bed. The latter estimate is known to be accurate, but tends to have much higher scatter than that typically obtained using the depth-averaged velocity. Further detail on the computation of τ_{ol} may be found in Wilcock and others (1994).

Because it is the flow discharge, rather than the local flow over the spawning gravels, that can be set independently, an expression relating flow discharge to τ_{ol} and gravel entrainment is needed for the purpose of evaluating different flushing alternatives. A rating curve is developed here for a representative τ_{ol} over the spawning gravels as a function of discharge. We use a two-step process, first relating τ_{ol} to flow depth over the spawning gravels, and then use the relation between the water surface elevation and discharge (the stage-discharge relation) to substitute discharge for flow depth.

The variation with discharge of τ_{ol} is given in Figures 6.2.3 and 6.2.4 for sections PB2 and SB3C, respectively. The location of the spawning gravels is also shown in each figure. It is clear that the shear stress varies strongly across each section and that τ_{ol} is substantially different than the average value calculated for the entire section. The difference between τ_{ol} and the mean bed shear stress for the entire cross-section is also evident in Figures 6.2.5.c and 6.2.6.c, which show the variation with discharge of both τ_{ol} and mean value of τ_o calculated by HEC-2 for the entire cross section. At Poker Bar, a systematic divergence is evident at discharges greater than about 75 m^3/s (Figure 6.2.5.c). At Steelbridge, an inverse relation exists between discharge the HEC-2 estimate of bed shear stress and discharge (Figure 6.2.6.c.) The controls of the overall flow field at the downstream end of the Steelbridge Island are sufficiently complex that simple channel-averaged estimates of bed shear stress are clearly not usable. The differences between local and section-averaged values of shear stress are not unusual. The total downstream shear stress calculated from section-averaged flow depth and velocity need not equal the mean shear stress calculated by integrating τ_{ol} across the section. It is also not necessary that the rate of change of bed shear stress with discharge be the same everywhere in the cross-section, particularly when bed shear stress varies widely across the section.

The development of a τ_{ol} rating curve is given in Figures 6.2.5 and 6.2.6 for Poker Bar and Steelbridge, respectively. For the flows observed at Poker Bar, the variation of τ_{ol} with local flow depth is essentially linear (Figure 6.2.5.b), whereas the variation of τ_{ol} with discharge is weakly curved. To account for this curvature in developing a τ_{ol} rating curve, τ_{ol} is related first to the local flow depth over the spawning gravels (Figure 6.2.5.a) and the HEC-2 relation between water surface elevation and discharge (stage-discharge curve) is then used to relate τ_{ol} to discharge. The relation between τ_{ol} in Pa and local depth in m is given by the fitted line (Figure 6.2.5.b)

$$\tau_o = 15.4\,(h_{local}) - 1.3 \qquad (6.2.1)$$

The HEC-2 estimates for water surface elevation as a function of discharge (Figure 6.2.5.a), together with the mean bed elevation of the spawning gravels, are then used to calculate local flow depth at different discharges. This depth was used in Equation 6.2.1 to calculate bed shear stress. The resulting relation between τ_{ol} and Q is marked as "extrapolated" on Figure 6.2.5.b.

For computational convenience in developing the sediment rating curves, a least-squares power function was fitted between τ_{O1} and Q:

$$\tau_{O1} = 0.8704\ Q^{0.718} \tag{6.2.2}$$

for τ_{O1} in Pa and Q in m^3/s. Equation 6.2.2 is indistinguishable from the curve marked "extrapolated" on Figure 6.2.5.b.

A similar relation between τ_{O1} and Q was developed for section SB3C (Figure 6.2.6). The relation between τ_{Ol} in Pa and local depth in m is (Figure 6.2.6.b)

$$\tau_o = 6.2\ (h_{local}) + 21.7 \tag{6.2.3}$$

The HEC-2 estimates for water surface elevation as a function of discharge (Figure 6.2.6.a) and the mean bed elevation of the spawning gravels were again used to calculate flow depth at higher discharges. Using this depth in Equation 6.2.3 to calculate bed shear stress, the resulting relation between τ_{O1} and Q is marked as "extrapolated" on Figure 6.2.6.b.

6.3 Sediment Transport

6.3.a Gravel Entrainment

A minimum flushing release is one that just entrains the gravel on the surface of the bed, thereby allowing some flushing of the subsurface and a loosening of the gravel texture. The discharge that produces this mobilization depends on the grain size of the gravel, the rate at which gravel is transported by that discharge, and the duration of the flushing release. The latter quantity, together with the discharge, is set by the volume of water released. In this section, we develop a relation between water volume and discharge for conditions that are just sufficient to produce gravel mobilization.

The tracer gravel observations provide our best measure of gravel entrainment at the study sections. To estimate the degree of gravel entrainment, we define the depth of entrainment, or exchange depth, d_{ex} as the total depth of the tracer gravels multiplied by the proportion of the gravels entrained. Because gravel entrainment depends on grain size, which varies from site to site, comparisons among different sites are facilitated by scaling d_{ex} by the grain size of the bed sediment. We use D_{90} of the bed surface for this grain size, so that gravel entrainment depth can be expressed relative to the thickness of the bed surface layer. Figure 6.3.1 shows the values of d_{ex}/D_{90} for tracer gravels at the two primary study sections for the 1991 and 1992 study releases.

Values of d_{ex}/D_{90} close to one represent a minimum threshold for gravel entrainment in a flushing flow. At this threshold, essentially all of the grains on the bed surface are entrained, allowing the surface layer to be loosened and permitting at least some removal of sand from beneath the bed surface.

A predictive capability for d_{ex}/D_{90} requires that the local shear stress τ_{ol} and bed grain size be accounted for. This may be done using the Shields parameter, which expresses the ratio of the force causing entrainment, τ_{ol}, to the weight of the grains, which accounts for the force resisting entrainment. The Shields parameter is defined as $\tau^* = \tau_{ol}/[(s-1)\gamma D]$, where s is the relative density of the gravel grains, taken to be 2.7, γ is the specific weight of water, taken to be 9800 $kgm^{-2}s^{-2}$, and D is a representative grain size. We form the Shields parameter using τ_{ol} because the shear stress varies widely across the section, and a coherent relation between entrainment and flow can only be made if the local flow producing the observed entrainment is used. We use the median size of the tracer gravels (D_{50}) as the representative grain size, because it has been shown to represent the initial motion and transport rates of gravels well (Wilcock, 1992).

The relation between d_{ex}/D_{90} and τ^*_{50} for the tracer gravel data is shown in Figure 6.3.2. Although a straight line would appear to fit the data well, it is likely that the true entrainment function is strongly curved, with a threshold value of τ^*_{50} at which measurable entrainment begins. At values of τ^*_{50} below this threshold, entrainment depth is essentially zero. Entrainment depth is likely to increase rapidly as τ^*_{50} exceeds this threshold and to asymptotically approach a limiting value of d_{ex}/D_{90} at large values of τ^*_{50}. A function that provides a simple representation of this trend is

$$\frac{d_{ex}}{D_{90}} = \beta\left(1 - \frac{\tau^*_c}{\tau^*_{50}}\right) \qquad \text{for} \quad \tau^*_{50} \geq \tau^*_c \tag{6.3.1}$$

where τ^*_c is the value of τ^* at threshold of measurable entrainment, and β is the value of d_{ex}/D_{90} that is approached asymptotically at large values of τ^*_{50}. The tracer gravel results are fit well by $\tau^*_c = 0.039$ and $\beta = 2.8$. The result that significant entrainment begins around $\tau^* \approx 0.040$ is an important one. This value of Shields parameter falls within the range that has been found to represent incipient grain motion in many experimental studies over the past 60 years. This lends support to the general applicability of our field results and extends the general empirical result to spawning gravels. Comparison of the shear stress values shown in Figures 6.2.3 through 6.2.6 shows that this result would not be obtained using section-averaged values of bed shear stress.

Although the function shown in Figure 6.3.2 would appear to provide a strong basis for predicting entrainment, it is incomplete because the effect of flow duration on d_{ex} is not included. The grain motion giving rise to the observed entrainment is not continuous, but occurs sporadically over the entire duration of the release. At the low gravel transport rates desirable for minimizing gravel loss, direct observation of the bed surface would reveal only occasional grain movements, giving the immediate impression that almost no grains were moving. The overall transport results from the cumulative entrainment and motion of grains over the entire duration of the flow. A very short flow will produce less entrainment, and a smaller value of d_{ex}/D_{90}, than a longer flow. To account for the effect of flow duration on the depth of entrainment, the rate of gravel transport must be considered.

Because the gravel motion produced by the 1992 release produced a value of $d_{ex}/D_{90} \approx 1$, which we judge to be a minimum for a flushing release, we seek to define the flow duration, or volume of water, required to produce the same entrainment at different discharges. To do this, we assume that the entrainment depth is proportional to the volume of gravel transported. Based on this assumption, the gravel transport rate formula developed in Section 6.3.b (Eq. 6.3.2) may be used to calculate the duration of flows necessary to produce a specified volume of gravel transport, and therefore comparable entrainment depths, at different discharges.

The water volume required to produce the minimum gravel entrainment threshold is shown on Figure 6.3.3. Because the rate of gravel transport increases rapidly with increasing discharge, a release at a higher discharge will require a smaller water volume to achieve the same degree of entrainment. Based on the assumptions behind Figure 6.3.3, surface mobilization may be accomplished at a discharge of 8000 cfs with less than half of the water required at 6000 cfs, and mobilization at 5000 cfs requires more than twice the water used at 6000 cfs. Although gravel mobilization can evidently be accomplished with less water at higher discharges, the choice of an optimum flushing discharge also depends on the volume of both sand and gravel moved. The combined analyses of both sand and gravel objectives is deferred until Section 6.5, after the development of a sand routing algorithm and the supporting transport formulas.

The estimates developed here of the flow duration necessary for surface mobilization do not assume any particular relation between entrainment depth and transport volume. Such a relation would be nonlinear and difficult to estimate independently. Instead, it is assumed only that a discharge that produces the same volume of gravel transport as the 1992 release will produce a comparable depth of entrainment. As discussed in the following section, a large degree of uncertainty is associated with any individual calculated value of transport rate. However, more confidence may be placed in the *relative* changes in gravel transport at different discharges, because this depends only on the form of the transport relation (i.e. the rate with which transport increases with discharge) and not on the particular values of transport at any particular discharge.

6.3.b Gravel Transport Rates

Evaluation of different flushing alternatives requires an estimate of the rate at which sand and gravel are transported at different discharges. Because a flushing release should maximize sand removal while minimizing gravel transport, separate transport relations are developed for sand and gravel. The gravel relation is developed in this section and the sand relation is developed in the subsequent section.

The transport observations made at the Poker Bar study site are used to develop a gravel rating curve. The best of these observations were made during the 1993 release (Q=80 cms), when five bedload traps were placed across section PB2. Gravel transport rates at higher discharges (Q=103 and 164 cms) in 1992 were calculated from Helley-Smith sampling transects along PB2. These samples are assumed to be less reliable than the gravel trap samples because they are subject to error from both over and undersampling. The gravel transport rates at PB2 are given as a function of bed shear stress over the spawning gravels in Figure 6.3.4. Also shown on Figure 6.3.4 are transport rates calculated for only the central 50% of the Helley-Smith samples (i.e. the mean transport rate was calculated after discarding the smallest 25% and the largest 25% of the individual samples). This was done to evaluate the possible influence of a few extremely small or large samples due to either undersampling (e.g. when the sampler lands on top of a cobble) or oversampling (from sampler scooping). The truncated sample is nearly the same as the total sample at Q=164 cms; the truncated sample is nearly an order of magnitude smaller than the total sample at Q = 103 cms.

Because of the small amount of transport rate data available, it is preferable to develop a relation for predicting the gravel transport rate using a general transport relation that has been found to work well for rivers similar to the Trinity. We use the limited observations of gravel transport to set the particular values of the function. The transport relation used for gravel is that of Parker (1979), which has been shown to hold for large gravel-bed rivers. The only fitted value required is the shear stress producing a small, specified transport rate, which serves as a surrogate for the stress at which motion begins. The Parker transport relation may be expressed as

$$q_{bg} = 0.0563\,\tau_o^{1.5}\left(1 - 0.85\,\frac{\tau_r}{\tau_o}\right)^{4.5} \qquad (6.3.2)$$

where q_{bg} is gravel transport rate per unit channel width in kg/(ms), τ_o is the bed shear stress in Pa, and τ_r is the shear stress corresponding to a small reference transport rate. Only one parameter, τ_r, is needed to fit Equation 6.3.2 to the transport observations. A value of $\tau_r = 22.5$ Pa was used to fit Equation 6.3.2 to the higher quality transport rate determined from the gravel trap in 1993. Equation 6.3.2 and $\tau_r = 22.5$ Pa also give a good fit to the Helley-Smith gravel observations for higher discharges at Poker Bar (Figure 6.3.4). Because the transport rate observed in the gravel trap is much smaller than the reference transport rate and because the Parker relation is very steep at small transport rates, the value of τ_r is constrained to be within a few percent of 22.5 Pa and still produce a calculated transport rate within an order of magnitude of the gravel trap observation.

A value of τ_r of 22.5 Pa corresponds to a Shields parameter value of $\tau^* = 0.039$ when τ^* is formed using the D_{50} of the gravel portion of the bed and $\tau^* = 0.060$ when τ^* is formed using D_{50} of the entire bed size distribution. These values of Shields parameter fall within the range observed to produce the reference transport rate of the median grain size of a wide range of mixed-size sediments in laboratory and field observations (Wilcock, 1993). This correspondence provides independent support for the estimated value of τ_r and applicability of the overall form of Equation 6.3.2 to the Poker Bar data. More importantly, this value of τ^* is consistent with the onset of gravel mobilization determined using tracer gravels at the same site, but in different years (Figure 6.3.2).

Gravel Rating Curve. In order to calculate gravel transport rates as part of the evaluation of flushing alternatives, a rating curve is required that gives gravel transport directly as a function of discharge. The general form of the rating curve used is

$$Q_g = \left(\frac{B}{\alpha}\right)\left(Q - Q_{cg}\right)^{\beta} \qquad (6.3.3)$$

where B is the channel width in ft over which active gravel transport occurs, Q_g is gravel discharge in tons per day, Q_{cg} is the discharge at the onset of substantial gravel transport, α and β are a fitted coefficient and exponent, respectively, and both Q and Q_{cg} are in units of cfs. Eq.

(6.3.3) gives gravel transport as a power function of water discharge and permits a nonloglinear variation of Q_g with Q at low transport rates.

Equation 6.3.2 may be converted into a relation between Q_g and Q by using the Poker Bar shear stress rating curve to obtain Q from τ_o and by multiplying q_{bg} by the active width of gravel transport to obtain Q_g. The shear stress rating curve is Equation 6.2.2

$$\tau_o = 0.8704\, Q^{0.718}$$

for Q in m^3/s and τ_o in Pa. An active transport width of 15 m (49.2 ft) is assumed. The resulting gravel transport curve is shown in Figure 6.3.5. Equation 6.3.3 was fitted to the Parker curve using $Q_{cg} = 2700$ cfs, $\alpha = 4.0 \times 10^9$, and $\beta = 3$ (Figure 6.3.5). This simple function is seen to fit the Parker relation within ±15% over the range of discharge to be modeled in the routing study.

<u>6.3.c Sand Discharge</u>

Sand transport rates at section PB2 are based on three sample transects using the Helley-Smith sampler. The discharges at which sand transport was measured were Q=103 cms and Q=164 cms in 1992 and Q=80 cms in 1993. The sand transport data are shown in Figure 6.3.6.

The relation between water and sand discharge is similar in general form to that used for the gravel,

$$Q_s = P_s \left[(1/a) \left(Q - Q_{cs} \right)^b \right] \tag{6.3.4}$$

where Q_s is in tons per day, P_s is the proportion of sand on the bed surface, a and b are the coefficient and exponent of the sand discharge rating curve, and Q_{cs} is the discharge, in cfs, at which the sand transport goes to zero. The quantity in large brackets represents the transport that would be produced for a bed covered entirely by sand, or the transport capacity of the specified value of Q. The factor P_s reduces the calculated sand discharge based on the amount of sand available for transport on the bed surface, which is particularly important in the case where P_s becomes very small as a reach becomes flushed of sand.

The value of the exponent b in Equation 6.3.4 is commonly observed to be on the order of 1.5 to 3.0 (Vanoni, 1975). In addition to the exponent b in Equation 6.3.4, a discharge at which sand discharge effectively goes to zero is required. The rating curve for sand discharge as a function of water discharge is given in Figure 6.3.6. Values fitted by eye to the Helley-Smith sand discharge observations at Poker Bar are $Q_{cs} = 1000$ cfs, b = 2, and a = 1.5×10^4 for Q in cfs and Q_s in tons per day. These values were set using $P_{ss} = 0.22$, which is a typical value for the Poker Bar sections during the period of sand transport observation.

6.3.d. Alternative Sediment Rating Curves

The primary goal of developing sand and gravel transport relations is not to calculate specific values of transport rate at a given discharge, to which a large degree of uncertainty must be attached, but to calculate relative changes in the gravel and sand transport rates for the purpose of evaluating the ability of different discharges to produce sufficient gravel entrainment and an acceptable value of the transport of both sand and gravel.

Because of the large degree of uncertainty in the value of predicted transport rates, alternative sediment rating curves were developed to evaluate the sensitivity of the flushing calculations to uncertainty in the estimated transport rates. Transport observations available for the USGS gage at Limekiln were used in combination with the transport observations at Poker Bar. All transport observations and sediment rating curves are shown in Figure 6.3.7.

An alternate gravel rating curve that accounts for bed-load transport of material coarser than 8 mm at both the Poker Bar study site and the USGS Limekiln Gulch gaging station uses the following values: $Q_{c_g} = 2700$ cfs, $b = 2.5$, $a = 2\text{x}10^8$ (Figure 6.3.7).

An alternate sand rating curve that also accounts for bed-load transport of material finer than 8 mm at the USGS Limekiln Gulch gaging station (Figure 6.3.7) uses the following values: $Q_{c_s} = 0$, $b = 4$, $a = 5\text{x}10^{11}$ for $P_{ss} = 0.22$, Q in cfs and Q_s in tons per day.

In addition to providing a fit to both the Poker Bar and Limekiln data, the alternate sediment rating curves were selected to provide reasonable alternative relations that can be used to test the sensitivity of the routing results to differences in the estimated transport rates. Among the four transport relations, a maximum difference in the ratio of sand to gravel transport is obtained using the two Poker Bar curves and the two alternate curves. In both cases, the ratio of sand to gravel transport decreases with discharge (Figure 6.3.8). The ratio of sand and gravel discharge using the Poker Bar rating curves (for $P_{SS} = 0.22$ and $B = 15$ m) equals one at $Q \approx 5750$ cfs and decreases to a value of 0.34 at $Q \approx 8500$ cfs. Using the alternative rating curves (and the same values of P_{SS} and B), sand discharge always exceeds gravel discharge and their ratio decreases toward a nearly constant value of 3.6 for $Q \geq 6400$ cfs. Because the optimum flushing discharge is likely to be sensitive to relative magnitude of sand to gravel transport, these two pair of sediment rating curves were selected for use in the routing algorithm.

6.4 Sediment Trapping by Pools

In order to evaluate the sand-removal effectiveness of dredged pools, it is necessary to calculate the rate at which sand is trapped in the pools during a flushing release. The sand-removal rate is conveniently represented by the trap efficiency T, which is the proportion of the sand entering the pools that is effectively trapped. For a volume $\forall$ of sand entering a pool, the volume $T\forall$ is trapped and the volume $(1-T)\forall$ escapes to the channel downstream. Pool trap efficiency depends on both water discharge and pool geometry, which determine the mean velocity and transport capacity of flow through the pools, as well as the rate at which sediment is delivered to the pool. For a given pool, an increase in discharge will decrease trap efficiency by increasing the mean velocity and transport capacity of the flow in the pool. At a given discharge, a pool with a larger cross-sectional area will have a lower velocity and transport capacity and will, therefore, trap sediment more efficiently.

When a pool is trapping sediment, deposition on the pool bottom causes the pool bed to aggrade and the pool depth to decrease. When a pool bed degrades, more sediment is leaving the pool than entering and the trap efficiency is negative. If the sediment supplied to the pool is just balanced by that transported out of the pool, the pool bed neither scours nor fills, and the trap efficiency is zero. We refer to pool depth at this condition as the stable depth, which we will use as a reference in defining the trap efficiency of pools during a flushing release. The stable depth depends on the rate of sand delivery to the pool, as well as the other factors (water discharge and pool geometry) that determine the transport capacity of the pool.

The dredging options we consider in our evaluation of sand removal are the depth to which pools are dredged and the number of dredged pools. We consider existing pools as well as a combination of existing and new pools. The existing pools vary in their geometry and the geometry of new pools is entirely unknown.

The storage volume and trap efficiency of each pool must be specified in order to calculate the sand removal that can be achieved by the different dredging options. One approach would be to specify a constant dredging depth for all pools. Because storage volume and trap efficiency depend on channel width and pool width, in addition to dredge depth, dredging all pools to a constant depth (e.g. 10 feet below the 300 cfs water surface) will result in a variation in storage capacity and trap efficiency from pool to pool. For example, a narrow pool dredged to 10 ft depth may have no storage capacity and a negative trap efficiency at a particular discharge, whereas the stable depth for the same discharge in a very wide pool might be 8 ft., so that two feet of storage are available. A more consistent evaluation of the sand removal potential of pool dredging can be achieved by specifying dredging depth relative to the stable depth for each pool and discharge. This permits a more direct evaluation of, for example, the relative benefit of dredging an additional foot of sediment from all pools. This is the approach we adopt in the flushing modeling developed in Section 6.5.

By specifying the dredging depth relative to a stable pool depth, we remove most of the between-pool variability from the routing problem and permit direct comparison among dredging scenarios. This approach does not, however, eliminate the need to calculate the sediment trapping in individual pools. Rather, it uncouples the details of the trapping calculation from the routing problem and adds the independent requirement that stable depth must be calculated for individual pools for each discharge. The next two subsections develop a method for estimating stable pool depth and test the method against pool observations made during the trial releases. These sections are not necessary to follow the development of our routing analysis, except in that they demonstrate that a stable pool depth can be estimated, thereby demonstrating the feasibility of specifying dredging depths relative to a stable pool depth.

Because little information is available on the flow and geometry of existing channel/pool reaches and because the location and configuration of new pools is completely unknown, we calculate stable pool depth using a relatively simple model of pool flow and transport that attempts to capture the dominant features of pool sedimentation without requiring detailed topographic information for each pool and channel segment. The model is evaluated against estimates of stable depth based on observations of pool cut and fill during the three trial releases.

Estimates of Stable Depth during the Trial Releases. Table 6.4.1 presents a summary of the pool cut and fill during the three trial releases. Of particular interest is the change in elevation of the pool bottom during a release. If the pool elevation does not change appreciably in the presence of active transport, one may assume that the sand input and output is roughly balanced and the pool is close to a stable depth for that release. If a pool is dredged below this depth, it will trap sand until it fills to the stable depth. If the pool bottom is above the stable depth, it will scour until it reaches the stable depth. The stable depth may vary with discharge.

The time required for a pool bottom to aggrade or degrade to the stable depth depends on the volume of sediment that must be moved and the net rate of sand input or output from the pool. If a release is not sufficiently long to move this amount of sediment, the post-release pool depth will not be the stable depth. To account for this in our field observations, a range of possible stable depths was estimated according to whether the pool was filling or scouring during the release. If a pool was filling, the stable depth was taken to fall in a range between the final depth and a shallower depth. If a pool was scouring, the stable depth was assumed to fall in a range between the final depth and a greater depth.

Stable depth estimates could be made from observations on four pools. The clearest evidence of a stable depth was found at Society pool, which was dredged prior to the 1991 release to a depth of approximately 10 ft below the 300 cfs water elevation. Deposition during the 1991 release caused the pool depth to decrease to 8-10 ft. During the higher discharge of the 1992 release, the pool scoured to a depth of approximately 10-12 ft. Stable depth for Society Pool was estimated to be in the range of 7-10 ft. for a discharge of 2800 cfs and in the range of 10-13 ft. for a discharge of 5600 cfs.

Prior to the 1992 release, Tom Lang pool was dredged to 8-13 ft. below the 300 cfs water surface. Deposition during the 1992 release caused the pool depth to decrease to 7-9 ft. There was little change in the pool depth during the 1993 release, suggesting the pool was close to stable depth. The reach upstream of Tom Lang pool has an unusually small proportion of sand, however, so there may not have been sufficient sand influx during the 1993 release to allow the pool to fill to a shallower stable depth. Stable depth for Tom Lang Pool was estimated to be in the range of 5-7 ft. for a discharge of 2800 cfs and in the range of 7-9 ft. for a discharge of 5600 cfs.

Ponderosa pool was dredged to a depth of greater than 12 ft prior to the 1992 release. This pool was surveyed only at its upstream end, so that only a very approximate estimate of stable depth is possible. The pool filled at the upstream end during the 1992 release, suggesting effective sand trapping and a depth greater than the stable depth. At the same time, the elevation of the pool bottom remained essentially unchanged, suggesting that the pool elevation is below the stable depth. Stable depth for Ponderosa Pool was estimated to be in the range of 10-13 ft. for a discharge of 5600 cfs. No survey of the pool was made in 1991 or 1993.

SP pool is a natural pool with a depth of 8-9 ft. prior to the 1992 release. The upstream portion of the pool was not surveyed, so that only a very approximate estimate of stable depth is possible. The surveyed portion of the pool scoured slightly to a depth of 8-10 ft. during the 1992 release and showed essentially no change during the 1993 release. Stable depth for SP Pool was estimated to be in the range of 6-9 ft. for a discharge of 2800 cfs and in the range of 8-11 ft. for a discharge of 5600 cfs.

Calculating Stable Depth in Pools. The calculation of stable depth is based on the requirement that, for a pool at stable depth, the sand transport rate is the same in both channel and pool. Flow in the channel upstream and downstream of the pool is assumed to be steady and uniform. Because flow during the releases is subcritical in both channel and pool, flow in the pool is a gradually varying backwater, with the water surface elevation at the downstream end of the pool set by normal depth in the downstream channel. The sand transport rate in the upstream channel is determined by the channel hydraulics and the proportion of sand present on the bed surface. Continuity in sediment transport, together with the proportion of sand on the pool bed surface, is used to set the bed shear stress in the pool so that the sand transport rates are the same in pool and channel. A flow resistance relation is then used to determine the mean velocity in the pool from the bed shear stress. Water mass conservation is then used to determine the pool depth from the mean velocity and the pool geometry.

To simplify the calculations, the channel and pools are assumed to have a rectangular cross section, which is a reasonable approximation for most of the channel/pool reaches. Figure 6.4.1 provides definition sketches of the various geometric parameters used in the development, including the dredge depth Δh, which is taken to be the depth to which the river bed is dredged below the channel bed.

To calculate the channel flow, discharge Q, bed slope S_o, channel width B_c, and channel hydraulic roughness n_c must be specified. Mean channel velocity V_c, hydraulic radius R_c, and flow depth h_c are then calculated from water continuity

$$Q = B_c h_c V_c \tag{6.4.1}$$

a flow resistance equation, for which Manning's formula is used

$$V_c = \frac{R_c^{2/3} S^{1/2}}{n_c} \tag{6.4.2}$$

and the definition of the hydraulic radius

$$R_c = \frac{B_c h_c}{B_c + 2h_c} \tag{6.4.3}$$

The bed shear stress τ_o is expressed as a shear velocity u_*, where $u_* = [\tau_o/\rho]^{1/2}$ and ρ is the fluid density, and the channel shear velocity u_{*c} is calculated using momentum conservation for steady uniform flow

$$u_{*c} = \sqrt{gR_c S_o} \tag{6.4.4}$$

where g is the acceleration of gravity.

The requirement that the sand transport rate be equal in pool and channel is next used to set the value of shear velocity in the pool u_{*p}. The sand transport rate (per unit channel width) in both channel q_{sc} and pool q_{sp} are assumed to be proportional to the cube of the shear velocity and the proportion of sand on the bed surface in the channel p_{sc} and pool p_{sp}

$$q_{sc} = p_{sc}\left(u_{*c}\right)^3 \tag{6.4.5.a}$$

$$q_{sp} = p_{sp}\left(u_{*p}\right)^3 \tag{6.4.5.b}$$

Sand transport varies with bed shear stress τ_o raised to the 3/2 power in Equation 6.4.5. This is a common value found in many transport relations for transport conditions well above the threshold shear stress for incipient sand motion. The transport stage for sand in both channels and pools is likely to be in this range. Assuming the width of active transport in pool and channel are comparable, the two parts of Equation 6.4.5 may be equated and solved for u_{*p}

$$u_{*p} = \left(\frac{p_{sc}}{p_{sp}}\right)^{1/3} u_{*c} \tag{6.4.6}$$

Next, it is necessary to relate u_{*p} to the flow and geometry in the pool, so that the dredge depth Δh may be determined. Because the flow in the pool is gradually varying, it is not possible to use a form of Equation 6.4.4 to directly calculate the hydraulic radius of the pool R_p. For the same reason, a flow resistance relation cannot be used directly to relate u_{*p} to either the mean velocity V_p or flow depth h_p in the pool. Instead, we recast the Manning formula in the form of a friction factor, or drag, formula

$$\frac{u_{*p}}{V_p} = C_D = \frac{n_p\sqrt{g}}{R_p^{1/6}} \tag{6.4.7}$$

where u_{*p} is given by Equation 6.4.6 and the form of the drag coefficient C_D is taken from Manning's formula. The value of the roughness term n_p in Equation 6.4.7 must be specified and is chosen below to fit the predicted stable depth to values of stable depth observed during the trial releases. Given specified values of Q, B_c, B_p, h_c, and n_p, it is possible to calculate V_p, A_p, R_p, and Δh from Equation 6.4.7, together with continuity

$$Q = A_p V_p \tag{6.4.8}$$

the definition of the hydraulic radius of the pool

$$R_p = \frac{B_c h_c + B_p \Delta h}{B_c + 2\,(h_c + \Delta h)} \tag{6.4.9}$$

and the definition of the pool cross-sectional area

$$A_p = B_c h_c + B_p \Delta h \tag{6.4.10}$$

The sequence of calculations described to this point is complete. The calculation of the value of Δh giving the stable depth h_s is more directly obtained by solving Equation 6.4.10 for Δh, and using Equations 6.4.6, 6.4.7, and 6.4.8 to replace A_p. After rearranging, this becomes

$$h_s = \frac{1}{B_p}\left[\frac{Q}{\left(\frac{p_{sc}}{p_{sp}}\right)^{1/3} u_{*c}\left(\frac{R_p^{1/6}}{n_p\sqrt{g}}\right)} - B_c h_c\right] \tag{6.4.11}$$

where the substitution $h_s = \Delta h$ has been made to indicate that the value of u_{*p} has been given by Equation 6.4.6. Because Δh is found in R_p, Equations 6.4.9 and 6.4.11 must be solved iteratively for R_p and h_s.

Values of h_s calculated using Equation 6.4.11 were compared with observations of stable depth during the trial releases to evaluate the utility of the simplified analysis is estimating stable depth. A useful model is taken to be one in which a credible value, or narrow range of values, of pool roughness n_p is found to match observed and predicted stable depths. The value of channel roughness was taken to be $n_c = 0.03$, which produces the observed stage-discharge modeling at Poker Bar. Observed values of Q, B_c, and B_p were used in the calculations. A bed slope of 0.003 was used in the calculations and the ratio p_{sc}/p_{sp} was taken to be 0.2, corresponding to a channel sand proportion of roughly 25% and a nearly complete sand coverage on the pool bed.

Figure 6.4.2 shows a comparison of calculated and observed pool cross-sectional area. In the upper panel, values of $n_p = 0.044$ for $Q = 80$ m^3/s and $n_p = 0.040$ for $Q = 164$ m^3/s were found to give a good fit between predicted and observed. In the lower panel, a constant value of $n = 0.0415$ is used for both flows and produces a fit that is still within the minimum and maximum estimates for observed stable depth, although the fit is not as good as the case with variable n. It is common to observe the Manning roughness n to decrease slightly with increasing discharge. The pool roughness values are somewhat greater than that for the channel, which is consistent with the fact that flow in the pools is a backwater, with the water surface elevation set by normal depth in the downstream channel. A large value of roughness produces a larger pool flow depth than would be obtained in steady, uniform flow in an infinite channel.

Figure 6.4.3 gives the variation of calculated stable depth with channel width, pool width, and water discharge. Stable depth depends primarily on discharge and pool width, and varies only weakly with channel width. An increase in discharge and a decrease in pool width both act to increase the mean velocity and the transport capacity of the pool flow, thereby requiring a larger depth for zero net transport. The weak influence of channel width on stable depth reflects a rough balance between competing influences: increasing channel width increases flow cross sectional area (thereby decreasing flow velocity and permitting a shallower stable depth), but also acts to decrease flow depth in the channel (decreasing the water surface elevation in the pool, requiring a deeper stable depth) and to decrease the bed shear velocity and transport rate in the upstream channel (thereby forcing a deeper stable depth to provide a smaller shear velocity).

Figure 6.4.4 presents calculated stable depth in a form usable for estimating stable depth as a function of discharge, channel width, and pool width. Each panel represents a different channel width and shows the variation of stable depth with discharge and pool width.

Estimating Pool Trap Efficiency. The development presented above for estimating stable pool depth can also be used to estimate the trap efficiency of pools as a function of the depth of dredging below stable depth. The trap efficiency T may be calculated as

$$T = \frac{q_{sc} - q_{sp}}{q_{sc}} = 1 - \frac{q_{sp}}{q_{sc}} \tag{6.4.12}$$

Replacing the transport terms using Equation 6.4.5, Equation 6.4.12 becomes

$$T = 1 - \left(\frac{p_{sp}}{p_{sc}}\right)\left(\frac{u_{*p}}{u_{*c}}\right)^3 \tag{6.4.13}$$

The channel shear velocity u_{*c} may be found from Equations 6.4.1 though 6.4.4 for specified values of Q, S_o, B_c, and n_c. If Δh, B_p, and n_p are also specified, the value of u_{*p} may be found using Equations 6.4.7, 6.4.8, and 6.4.9 to solve for u_{*p}, V_p, and R_p. With values of u_{*p} and u_{*c}, T may be calculated from Equation 6.4.13.

Figure 6.4.5 presents the variation of pool trap efficiency as a function of dredge depth below stable depth, discharge, (p_{sc}/p_{sp}), channel roughness n_c, pool roughness n_p, and the exponent in the sand transport relation Equation 6.4.5. Trap efficiency generally decreases from values in the range 0.6 to 0.8 for dredge depths 2 m below h_s ($\Delta h = h_s + 2m$) to zero at the stable depth ($\Delta h = h_s$). The estimate of T shows only a weak dependence on the choice of roughness values, and shows less than a ±20% variation with discharge and (p_{sc}/p_{sp}) at large dredge depths.

The relatively small variation of T is a result of the fact that the primary influence of these variables has been accounted for in the computation of the stable depth.

6.5 Sand Routing

A sand routing algorithm has been developed to provide a means of quantifying the rate at which sand and gravel are removed from the study reach. These results form the basis for evaluating different flushing options, including release volume, release rate, or discharge, the number of dredged pools , and the depth of dredging in pools.

In general, two options exist for increasing the efficiency with which sand may be removed from the study reach with a given amount of water. One is to use the largest possible water discharge, because sand is transported at a higher efficiency at larger discharges. Even though higher discharges use water at a greater rate, the increased efficiency in transporting sand permits shorter releases and a net savings in water volume. The other option is to dredge sand from pools along the study reach, thereby shortening the length over which sand must be transported. Because the sand transported from the upstream end must pass through the entire reach, shortening the reach by providing intermediate sinks results in a direct reduction in the water used. These two options—higher discharges and dredged pools—are, to some extent, incompatible because higher discharges will make the pools trap sand less efficiently. The problem is further complicated by the fact that higher discharges will also increase the rate of gravel transport, which will increase the rate at which sand can be removed from the subsurface, thereby increasing the efficiency of the flush, while also increasing the amount of gravel deposited in the pools, thereby decreasing their storage capacity for sand.

The routing exercise is characterized by a high degree of uncertainty. The quantity of sand requiring removal is only approximately known. The initial proportion of sand on the bed surface is specified only as a spatial average estimated visually over large subreaches. The proportion of sand in the subsurface has been sampled at only a few cross-sections within the entire study reach. Sand discharge has been measured at only two cross sections and considerable scatter exists in these observations at one location. Both the rate of gravel loss and the sand removal rate depend on the gravel transport rate, which has been sampled at three discharges at only one cross-section.

The routing efforts are also characterized by a high degree of simplification. Channel geometry is known at only a few cross sections covering a small portion of the reach. The study reach is divided into only six subreaches to calculate the sand routing. The variation with discharge of the sand and gravel transport rates is assumed to be the same at the outlet for each subreach. The sediment trapping characteristics of the pools are assumed to be completely represented by the depth dredged below stable depth.

The absence of local observations and the level of simplification necessary for our analysis of the Trinity River are typical of many applications in fluvial sediment transport, particularly those involving reaches tens of kms or more in length. Use of a detailed computational model is difficult to justify in the absence of the required input of channel geometry and sediment composition throughout the study reach and without local relations between discharge and sediment transport. The final computed result is likely to be sensitivity to the choice of initial values and local rating curves.

The approach taken here is to use simple formulations intended to capture the dominant physical processes occurring in the study reach during a flushing flow. The general mathematical form used to represent each process is drawn from theory or general empirical relations known to hold for gravel-bed rivers. In each case, the available data on transport, bed sediment, and pool trapping are used to set particular values of the function. To make the functions directly applicable to the flushing question, they are formulated using river discharge as the independent variable. Because the results of the routing exercise may depend on the general form and particular values of the functions used to represent the transport processes, alternate forms of the transport functions are used to explore the sensitivity of the flushing conclusions the models used to represent transport.

6.5.a Processes Controlling the Rate of Sand Removal

The rate at which sand and gravel are removed from a reach of a gravel-bed river varies with discharge in a complex and nonlinear fashion. We incorporate the following four processes in our sand routing algorithm:

(1) The rate of sand discharge Q_s depends on both the proportion of sand present on the bed surface P_s (which may decrease during a flushing flow), and the rate at which the available sand is transported by a given discharge Q. These two factors are represented by a sand rating curve of the form

$$Q_s = P_s\left[(1/a)\left(Q - Q_c\right)^b\right]$$

In general, the sand discharge rate increases linearly with P_s and nonlinearly with Q. The exponent b is larger than one, meaning that larger Q will transport sand more efficiently. Two different sand rating curves were developed in Section 6.3 to allow an evaluation of the sensitivity of the routing result to uncertainty in the estimated transport rates.

(2) The efficiency with which sand is trapped in dredged pools will depend on the depth and cross-sectional area of the pool, the water discharge, and the rate at which sand is delivered to the pool. We uncouple the details of calculating pool trap efficiency from the routing problem by specifying the dredging depth relative to a stable pool depth with no net scour or fill and, therefore, zero trap efficiency. A method for calculating stable pool depth as a function of pool geometry, sand input, and water discharge is developed in Section 6.4 of this report. In the routing exercise, we take the trap efficiency to be zero when the pool depth is at the stable depth. For greater depths, implying available sand storage, pools may still not trap sand with perfect efficiency and this efficiency can decrease with discharge, although we assume the main effect of discharge on trap efficiency is incorporated in the calculated value of stable depth.

(3) The gravel transport rate increases with water discharge. Significant gravel movement begins at a higher discharge than for sand, but the rate of increase of gravel transport with discharge is more rapid than for sand, so that the gravel transport rate may exceed the sand transport rate for discharges within the available range. Dredged pools will trap essentially all of the transported gravel. As discharge and gravel transport increase, a larger fraction of the pool storage volume will be filled with gravel, leaving less storage for sand. Two different gravel rating curves were developed in Section 6.3 to allow an evaluation of the sensitivity of the routing result to uncertainty in the estimated transport rates.

(4) As discharge increases, the coarse grains on the bed surface are entrained more frequently. When surface grains are entrained, grains in the subsurface are exposed to the flow and may be entrained from the bed. Thus, the rate at which sand is supplied from the subsurface to the bed surface will increase with discharge. By supplying sand to the bed surface, this upward sand transport supplements P_s and the rate of sand discharge.

These flushing processes are evidently interrelated in a complex and nonlinear fashion. Two of them (the sand discharge rating curve and the rate of sand supply from the subsurface) cause sand removal efficiency to increase with water discharge, whereas the other two (pool trap efficiency and pool filling with gravel) cause the sand removal efficiency to decrease with increasing discharge. The sand routing algorithm is used to evaluate the balance between these different processes.

6.5.b. Routing Algorithm

To perform the sand routing, the entire study reach is divided into subreaches and a simple mass conservation model is used to route sand from one reach to the next. Sand present in the bed surface layer and the immediate bed subsurface is treated separately. The nominal thickness of these two layers is taken to 15 cm, which is roughly 1.5 D_{90} for the Poker Bar study reach.

The quantity of sand present in the surface layer S_s at the end of each time step is determined as the starting quantity of sand plus sand inputs from the upstream reach and the bed subsurface and minus the sand discharge out of the reach

$$S_{s,i+1} = S_{s,i} + I_i - O_i$$

where the subscript *i* refers to a particular time step. The output from each reach is calculated using the sand discharge rating curve

$$O_i = P_{si}\left(\frac{1}{a}\right)\left(Q - Q_c\right)^b \Delta t$$

where P_s is the proportion of sand on the bed surface, a and b are a specified coefficient and exponent, respectively, Q_c is the discharge at which the sand transport becomes negligible, and Δt is the time step, which is taken to be 1 hr in all cases. P_s changes as the routing proceeds. As a reach becomes flushed, the value of P_s will decrease and the sand discharge will decrease proportionately. The value of P_s is calculated after each time step as

$$P_{s,i+1} = \left(\frac{S_{s,i+1}}{S_{s,init}}\right) P_{s,init}$$

where the subscript *init* refers to the initial conditions given in Table 6.5.1.

The input of sand to a reach is the sum of sand output from the next reach upstream, reduced by any sand trapped in an intervening pool, and the input of sand winnowed from the bed subsurface when coarse surface grains are entrained. Reach input is calculated as

$$I_{i+1,\,j} = O_{i,j-1}\,(1 - T) + U_{i,j}$$

where T is the trap efficiency of the pool, U is the quantity of sand supplied from the subsurface, the subscript *j* refers to reaches in increasing downstream order and, as before, the subscript *i* refers to the time step. Input from Grass Valley Creek to the upstream reach is unknown and is taken to be one-quarter of the output from the upstream reach at each time step.

The pool trap efficiency is defined in terms of a stable depth below which sand deposition is possible. Trap efficiency is set to zero when the pool depth is less than the stable depth. For greater depths, implying available sand storage, trap efficiency is taken to equal 0.8 in most runs.

The upward movement of sand from the subsurface to the bed surface will depend on the concentration of sand in both layers, as well as the rate at which gravel on the bed surface is entrained, thereby exposing subsurface sand to the flow. The mass of sand that is moved from the subsurface to the surface in time step Δt is calculated as:

$$U = \frac{1}{2}\left(\frac{P_{ss} - P_s}{P_{ss}}\right) M_{ss} \left(\frac{\Delta t}{t_{ex}}\right)$$

where P_{ss} is the proportion of sand in the subsurface, M_{ss} is the total mass of sand in the subsurface of a reach. t_{ex} is an exchange time that determines the quantity of sand that can be moved in one time step. It is taken to be the time required to entrain all of the gravel clasts on the bed surface. The exchange time will vary with both the rate and duration of gravel entrainment and, therefore, depends on both the rate and duration of water discharge. One value of t_{ex} was set using the tracer gravel observation that essentially the entire bed surface was entrained over a 5 day period with Q = 5800 cfs during the 1992 trial release. If the frequency of gravel entrainment is taken to be proportional to the gravel transport rate, values of t_{ex} in days for other discharges can be determined using a ratio of gravel transport rates

$$t_{ex} = 5\left(\frac{q_{g6}}{q_g}\right)$$

where the gravel rating curve is used to calculate q_g, and q_{g6} is the gravel transport rate at Q = 5800 cfs. The variation of t_{ex} with discharge is shown for the Poker Bar and alternate gravel rating curves in Figure 6.5.1, which shows essentially the same information as Figure 6.3.3. The exchange time is seen to vary inversely with the gravel transport rate and takes a value of five days when Q = 5800 cfs. For a flow duration equal to the exchange time, the value of U is seen to vary with P_{ss} and P_s from a value of zero when the concentration of sand in the subsurface and surface is identical, $P_{ss} = P_s$, to a value of 1/2 M_{ss}, when the surface is entirely free of sand. Thus, the upward transport of sand over one exchange time is given as one-half the excess sand mass found in the subsurface.

The quantity of sand present in the bed subsurface layer S_{ss} at the end of each time step is calculated as

$$S_{ss,i+1} = S_{ss,i} - U_i$$

The proportion of sand in the bed subsurface at the end of each time step is calculated as

$$P_{ss,i+1} = \left(\frac{S_{ss,i+1}}{S_{ss,init}} \right) P_{ss,init}$$

The pool storage volume is reduced in each time step by the sand and gravel deposition in the pool. The sand deposition is given by ($O_{i,j-1}$) multiplied by the pool trap efficiency. A trap efficiency of 100% is assumed for the gravel, so the gravel deposition in pools is given by the gravel discharge calculated by the gravel rating curve.

A large degree of uncertainty is associated with each computed value of sand storage and discharge. Some of the principle sources of this uncertainty are errors in the estimate of sand volume in each reach, variation along the reach in the sand and gravel rating curves, and the uncertainty in specifying the trap efficiency of the pools. A greater degree of reliability exists for the trends between the variables and for the *relative* changes in sand storage and sediment transport among different discharges. Fortunately, it is these relative differences that are needed to evaluate the relative merits of different flushing alternatives.

6.5.c. Routing results

The quantity of sand requiring removal is taken to be that estimated to be in the surface and subsurface of the study reach in the visual survey made after the 1993 release (Table 6.5.1). Note that the last two reaches are not currently separated by a pool.

Sand and Gravel Removal as a function of Q and Pool Depth.

Figure 6.5.2 gives the amount of sand removed (open symbols) using the existing pools and a release of 100,000 acre-feet. The left panel uses the Poker Bar sediment rating curves. The right panel uses the alternate sediment rating curves. Five cases are shown: no dredging (solid line; assumes pools trap no sediment), dredging to 1 ft., 2 ft., 3 ft., and ∞ ft. below stable depth. An infinite dredging depth is used as a reference for sand removal if the pools never fill. Also shown on Figure 6.5.2 is the rate at which gravel is transported past the downstream section. At the lower discharges, the gravel transport goes to zero. As Q increases, the relative rate of gravel transport increases and, for the Poker Bar rating curves, surpasses the total sand removal.

The value of discharge that, for 100,000 acre-feet of water, is just sufficient to mobilize the bed surface is 5185 cfs for the Poker Bar sediment rating curves and 5055 cfs for the alternate rating curves. This limit is shown on Figure 6.5.2 as a vertical dashed line. Discharges smaller than this limit will not provide adequate mobilization of the bed surface with 100,000 acre-ft of water. A larger Q will generate more gravel transport than that needed to mobilize all the grains on the bed surface. The value of this minimum Q for full surface mobilization will decrease with increasing water volume, but the volume of water required for discharges less than 5,000 cfs becomes very large (Figure 6.3.3).

The flushing trade-offs that occur with increasing discharge may be described by following one of the sand removal curves on Figure 6.5.2 (e.g., the sand removal produced with one ft of dredging, represented by the open squares). Beginning at Q = 3000 cfs, the amount of sand removed increases with Q because sand is moved more efficiently by larger discharges. The sand removal curve for one ft of dredging begins to deviate from the other sand removal curves at $Q \approx 4700$. It is at this Q that the sediment transport is large enough to begin to filling some of the pools. At higher Q, more of the pools fill and those that fill do so earlier in the release. Once all pools are filled, the pools trap no additional sediment and sand removal occurs only at the downstream end of the entire study reach. At this point, the sand removal curves for dredged pools asymptotically approach a curve parallel to the no dredging case. The difference between the curves is equal to the total storage capacity of the dredged pools. An important point is that, at discharges greater than that sufficient to fill the pools, the amount of sand trapped in the pools does not remain constant for any higher discharge, but decreases with discharge, because the rate of gravel transport relative to sand transport increases with Q so that the proportion of gravel in the pools (taking up storage volume) increases with Q. This point is illustrated by the plot of percent sand in the pool deposits (represented with an "x" on Figure 6.5.2).

A similar decrease in sand removal efficiency is evident with both sets of sediment rating curves (shown in the two panels of Figure 6.5.2), although the decrease with discharge in sand removal efficiency is larger for the Poker Bar rating curves.

Figure 6.5.2 may be used to evaluate the flushing tradeoffs. To minimize gravel loss, it is desirable to use the smallest possible Q. The minimum value of Q to be used is that required for full surface mobilization, which is represented by the vertical dashed line on the figure. If this discharge produces adequate sand removal for the water volume used, the argument for minimizing gravel loss leads to the mobilization discharge as the desired result. For a given water volume, the amount of sand removal will then depend on the number and depth of pools. Our estimate of the total sand available for removal in the reach is 13,275 tons. At Q = 5185 cfs for the Poker Bar sediment rating curves, between 4000 and 5000 tons of sand are removed with 100,000 ac-ft of water, depending on the dredge depth. At Q = 5055 cfs for the alternate sediment rating curves, between 5000 and 5400 tons of sand are removed, depending on the dredge depth. Because this sand removal represents roughly 40% of the sand available, a discharge of 5000 cfs to 6000 cfs accomplishes significant sand removal while also minimizing the gravel loss.

For water volumes less than 125,000 acre-feet, the discharge threshold for gravel mobilization is on the order of 5,000 to 6,000 cfs. Smaller discharges will not produce sufficient flushing at depth or gravel loosening. Larger discharges will involve a lower efficiency of sand removal because pools will begin to fill and an increasing proportion of the trapped sediment is gravel. Therefore, we recommend using a discharge of 5,000 cfs to 6,000 cfs for release volumes comparable to those that have been used in the past.

<u>Sand and Gravel Removal as a function of number and depth of pools and water volume.</u>
Figure 6.5.3 gives the variation of total sand removed as a function of the number of dredged pools and the depth of dredging below stable depth. The pool sets examined are (1) the existing set of pools, including dredging of both SP and Ponderosa, (2) adding Montana pool, and (3) adding a third pool between Montana and Upper Steelbridge. The calculations are made for both sets of sediment rating curves, for Q = 5,000 cfs and 6,000 cfs, and for water volume = 50,000, 100,000, 150,000, and 200,000 acre-ft.

For Q = 5,000 cfs, there is little increase in sand removal for dredge depths greater than two ft below stable depth, because the pools do not fill, or fill only toward the end of the release. At Q = 6,000 cfs, pools fill more rapidly, and dredging deeper than two ft provides an increase in sand removal. For all water volumes and both discharges, there is an increase in sand removal and removal efficiency achieved by dredging to a depth of two ft.

Dredging two additional pools provides an increase in total sand removal, although the increase is smaller than that provided by dredging. The importance of dredging additional pools rests in the fact that the amount of sand removal in an individual subreach will depend on the total amount of sand in the reach and, therefore, its length. A more even distribution of pools throughout the study reach will produce a corresponding distribution of sand removal. If the habitat limitations are set by the reach segment having the most sand, a larger number of pools is needed to produce an even distribution of sand removal.

We conclude that pools increase the efficiency of sand removal directly and significantly. The small cost of dredging relative to the cost of water makes dredging a definite benefit in a restoration program. For most values of water volume and a discharge in the range of 5,000 cfs to 6,000 cfs, two feet of dredging is sufficient, because the pools tend to not fill over the flush, or fill only toward the end of the flush. Dredging deeper than two feet does not produce additional sand removal, except at larger values of discharge and water volume.

The variation of sand removal with water volume is more directly evident in Figure 6.5.4, which gives sand removal as a function of water volume for three of the pool options in displayed in Figure 6.5.3. The amount of sand removed increases with water volume, although the efficiency of sand removal (sand removed divided by water volume used) decreases with water volume, as shown in the lower panels of Figure 6.5.3.

The choice of the water volume to be used for a sediment maintenance release is a tradeoff among a number of factors, some of which are not considered in this study. These include the period of time over which sand flushing should be accomplished and, therefore, the number of releases required, the availability and cost of water and their variation from year to year, and the cost of dredging. The sand routing calculations and Figure 6.5.3 illustrate how the volume of sand removed varies with the water volume and provide a reference for evaluating the effect of release volume on sediment maintenance.

Distribution of Sand Removal Along the Study Reach. Figures 6.5.5 and 6.5.6 show the proportion of sand in the surface and subsurface of all subreaches for different dredging combinations and water volumes. The results for Q = 5,000 cfs are given in Figure 6.5.5 and the results for Q = 6,000 cfs are given in Figure 6.5.6.

Sand removal occurs through the downstream end of a reach, which acts as a bottleneck for reach cleaning. Dredged pools shorten the length of the reach and provide multiple exits from the system. The left panel of Figures 6.5.5 and 6.5.6 represents the case for no pool trapping, so that sand removal occurs only at the downstream end of the entire study reach. The bottleneck effect on sand removal is quite evident in these figures, which show a substantial decrease in sand removal (increase in percent sand) in the downstream direction. The middle panel of these figures represents the case where the existing dredged pools are dredged to a depth of two ft. These pools are all in the upper half of the study reach. The relative benefit of dredging these pools is evident in that the decrease in sand content from a flushing release extends further downstream than in the no-dredge case. The proportion of sand remaining in the lower half of the study reach, however, remains large under this scenario. To decrease the sand content in the lower half of the study reach, it is necessary to dredge pools in this portion of the reach. The right panel of Figures 6.5.5 and 6.5.6 represents the case where two new pools are dredged. In this scenario, a reduction of sand in the lower part of the study reach becomes possible. There is still an general trend of increasing sand content with distance downstream, which is related to the fact that the pools are not completely efficient in trapping sediment, so that a weak bottleneck persists at the downstream end of the study reach.

A release using 100,000 to 150,000 acre-feet of water will provide substantial sand removal in only the upstream portion of the study reach if no pools are dredged. The extent to which the release accomplishes sand removal further downstream depends directly on the number of pools dredged and the length of river reach between the pools. Without new pools in the downstream half of the study reach, the sand removal that can be accomplished in this reach is limited and much larger volumes will be needed to clean the bed sediment.

<u>Evaluation of a Two-part Release with a short, large discharge followed by a long, lower discharge</u>. A sediment maintenance release need not use a constant discharge. One alternative is to use a short, large discharge to efficiently accomplish full bed surface mobilization, followed by a longer release at a low discharge to accomplish additional sand removal with little additional gravel loss.

Figure 6.5.7 presents the sand and gravel removed using 100,000 acre-feet of water divided between a 36-hour, 8,000 cfs spike and a longer, low discharge period. The spike is sufficient to fully mobilize the bed surface. The analysis uses the Poker Bar rating curves, the existing pools, and dredge depths of zero and 2 ft. below stable depth. The post-spike discharge is varied from 3,000 cfs to 6,000 cfs. Also shown on the figure are the sand and gravel removed by a release of 100,000 acre-ft at a constant discharge. The sand removal accomplished by the spike release varies between values comparable to the constant discharge release and values somewhat larger. The volume of gravel removed is, in all cases, greater with the spike release. For example, for a post-spike discharge of 3,500 cfs, the spike release removes roughly 1,000 tons more sand than the constant release, but also removes nearly 3,000 tons more gravel. A 100,000 acre-ft release at 3,500 cfs is not sufficient to mobilize the gravel bed, so a constant

release at a discharge of 6,000 cfs provides a more apt comparison with the spike release using 3,500 cfs. The constant rate release at 6,000 cfs removes more sand than any of the spike releases, together with a smaller loss of gravel. Of all the cases shown in Figure 6.5.7, the most favorable combination of large sand removal and minimum gravel loss is accomplished with a constant rate discharge of about 5,200 cfs, which corresponds to the minimum discharge required to fully mobilize the gravel bed surface. Because spike releases do not remove significantly more sand than constant releases, but move considerably more gravel, we do not recommend their use for sediment maintenance flows on the Trinity River. Should short, high-discharge releases be desirable for other purposes, the analysis presented here provides a basis for evaluating the related sediment impact.

Release Requirements for Sediment Maintenance. If a release program is carried out that successfully decreases the amount of sand in the river bed, a longer term sediment maintenance release plan would then be required. It is likely that the volume of water needed to *maintain* a low sand concentration in the reach will be comparable to that required to *decrease* the sand concentration in the reach, because the efficiency of sand removal decreases directly with the proportion of sand found in the river bed. More water is needed to remove a specified volume of sand from a clean river bed relative to an embedded river bed. A long-term sediment maintenance flow would also be indicated to maintain looseness of the bed material. A minimum maintenance flow would be that required to produce full mobilization of the bed surface, as demonstrated in Figures 6.3.3 and 6.5.1.

6.6 Channel Sedimentation History and Release Timing

The primary post-dam changes to the Trinity River channel have resulted from deposition of tributary-derived sediments within the pre-dam channel. This deposition occurs in two places: within the channel bed and on the steep, fine-grained banks that line the present river channel. Although the sediments in both deposits come from the same tributary source, there is little overlap in their grain size: the fine sediment within the channel bed is predominantly 1 mm to 8 mm in size, whereas the bank sediments are predominantly finer than 1 mm. This is a common sorting process wherein finer grained sediments cannot deposit in the swifter current over the river bed, but are carried higher in the flow and deposit in low velocity regions along the channel margins.

The coarser grained 1 mm-8 mm sediment found in the river bed is characteristically light colored and originates primarily from the decomposed granitic soils in the Grass Valley Creek basin. Once delivered to the Trinity River, this sediment is dispersed downstream by subsequent high flows on the main stem. Little transport of this sediment occurs at discharges less than 3000 cfs and essentially no transport occurs at the typical post-dam in-stream minimum flows of 150 cfs to 300 cfs.

Deposition of the finer-grained (<1 mm) material on the banks of the Trinity River follows a different pattern. Very little sediment finer than 1 mm is found in the channel bed, which suggests that most, if not all, main-stem flows with high sediment concentrations are capable of preventing deposition on the bed. Deposition of this finer sediment within the study reach requires the simultaneous occurrence of high river stage and high sediment concentration, so that sediment-laden water can deposit in low velocity regions above the banks on the channel margin. In contrast to the coarser 1 mm to 8 mm sediment found within the channel bed, the finer-grained material will not remain in the study reach if the main-stem discharge is low. Therefore, deposition of this sediment within the study reach occurs only in the overbank areas and requires an associated high stage on the Trinity River.

The post-dam history of water and sediment input to the Trinity River provides insight regarding the controls and timing of fine sediment deposition that supports an evaluation of channel restoration alternatives. Because the post-dam channel change is primarily a result of sedimentation on the bed and banks of the pre-dam channel, the history of channel change depends not only on the discharge on the main-stem Trinity River, but also on the water and sediment delivered from tributaries downstream of the dam. Of particular importance for the fine material in the channel bed is the sediment input from Grass Valley Creek. The fine-grained material in the channel banks likely derives from both Grass Valley Creek and Rush Creek, the other principal tributary to the study reach.

Information on post-dam water discharge is available from three USGS gages (Trinity River at Lewiston, Grass Valley Creek at Fawn Lodge, Trinity River below Limekiln Gulch). A complete post-dam discharge record is available only for the Lewiston gage. The periods of record for the gages are Lewiston: 1911 - present; Fawn Lodge: 1975 - present; Limekiln Gulch: 1981 - 1991. Measurements of suspended sediment were made at the Fawn Lodge and Limekiln Gulch gages. These samples are taken throughout the water depth and capture suspended sediment almost entirely finer than 2 mm. Insufficient measurements are available of the transport of coarser grained material along the bed to establish a history or rating curve for Grass Valley Creek. It may be presumed that a positive correlation exists between the transport rates of the coarser and finer grained sediments, although the different controls of bed-load and suspended-load transport make a direct correlation not possible.

The large reduction in river discharge from reservoir control and interbasin diversion is at the root of the post-dam channel change. All post-dam floods are small compared to the pre-dam floods. For example, the largest single daily discharge in the 34-year post-dam period is 13,800 cfs (USGS gage, Trinity River at Lewiston , 1/18/74), which is less than both the mean annual flood and the flood with a two-year return period under pre-dam conditions. During this same 34-year period, there have been only 10 days with discharge exceeding 8500 cfs, which is one-half the pre-dam 2 year flood.

Because channel change requires enough water discharge to transport sediment, most of the post-dam channel changes have occurred during the remaining high flow events. Figure 6.6.1 presents a record of all discharge greater than 3000 cfs entering the upstream end of the study reach (Lewiston gage). The most important observation that may be made with these data is that episodes of high flow are relatively rare in the post-dam period. All of the discharge greater than 3000 cfs occurred in eight of the 34 years; 76% of the discharge greater than 3000 cfs occurred in three years: 1963, 1974, and 1983. Discharge has exceeded 3,000 cfs for 215 days in the post-dam period. All of the discharge greater than 6000 cfs occurred in six of the 34 years; 71% of the discharge greater than 6000 cfs occurred in two years: 1963 and 1983. Discharge has exceeded 6,000 cfs for 76 days in the post-dam period.

Because the records at Fawn Lodge and Limekiln Gulch do not cover the entire post-dam period, this discussion focuses on the particular events involving a large sediment discharge from Grass Valley Creek, rather than an accumulated accounting of the sediment moving through the two rivers during the post-dam period. The period of record considered is 1975 to 1990 (15 water years), which extends from the start of the Fawn Lodge record to the installation of a sediment trap upstream of the Fawn Lodge gage in 1990. During this period, an estimated 493,000 tons of suspended sediment were delivered to the Trinity River from Grass Valley Creek. 90% of this sediment was delivered during three years; nearly all of it was delivered during winter storms in January through March (Table 6.6.1).

The fate of sediment delivered to the Trinity River by Grass Valley Creek depends on the magnitude of water discharge in the Trinity River during and after the primary sediment loading.

Figures 6.6.2, 6.6.3, 6.6.4 present the water and suspended sediment records for the three main sediment delivery events in 1978, 1983, and 1986, respectively. The upper panel of each figure presents the water discharge for the Trinity River at Lewiston and Grass Valley Creek at Fawn Lodge. The lower panel presents the estimated suspended sediment discharge for Fawn Lodge and (for 1983 and 1986) the Trinity River at Limekiln Gulch.

January - March, 1978 (Figure 6.6.2). 75,500 tons of suspended sediment is estimated to have been delivered to the Trinity River from Grass Valley Creek during January - March, 1978. Two-thirds of this sediment was delivered over five days: Jan. 14 -18. The flow on the Trinity River was consistently low during this period and in the following two years. The largest subsequent daily mean discharge (Lewiston gage) is 746 cfs for WY1978 and 628 cfs for WY1979. The next larger discharges are 20 days with $2000 < Q < 2600$ cfs in February and March of 1980 and 11 days with $3{,}920 < Q < 4{,}490$ cfs in December, 1981. As a result, it is likely that most of the coarser-grained sediment delivered during the 1978 event remained near the mouth of Grass Valley Creek for a period of four to five years. Downstream dispersal of this sediment is not likely to have occurred until 45 to 60 months following its introduction into the Trinity River. The finer-grained sediment from the 1978 flood on Grass Valley Creek is likely to have been removed from the study reach without substantial overbank deposition because river stage was not high enough to permit deposition in overbank areas.

January - April, 1983 (Figure 6.6.3). 300,000 tons of suspended sediment is estimated to have been delivered to the Trinity River from Grass Valley Creek during January - April, 1983. Over 80% of this sediment was delivered in seven days: Jan. 26-27 and Feb. 28 - March 4. The 1983 sediment load represents 62% of the estimated suspended sediment discharge for the entire 15 year period at the Fawn Lodge gage. This very large sediment load is accompanied by some of the largest post-dam water discharges on the Trinity River. During and following the sediment discharge events on Grass Valley Creek, the Trinity River experienced 73 days with discharge greater than 3,000 cfs and 39 days with discharges in excess of 6,000 cfs. Given the magnitude and duration of very high flows following March 4, it seems unlikely that most of the coarser-grained sediment remained near the mouth of Grass Valley Creek, but was dispersed along the entire Trinity River channel to its confluence with the North Fork. The amount transported past the confluence with the North Fork is unknown.

It is also likely that the period of January - March 1983 was one of the primary episodes of bank building along the margins of the river. The combination of a large influx of fine sediment with roughly two months of high stage suggests that a large portion of these banks may have been constructed at that time.

February - March, 1986 (Figure 6.6.4). 59,000 tons of suspended sediment is estimated to have been delivered to the Trinity River from Grass Valley Creek during February and March, 1986. Three-quarters of this sediment was delivered over six days: Feb. 14-19. This sediment load was accompanied by relatively large discharges on the Trinity River. During and following the sediment discharge events on Grass Valley Creek, the Trinity River experienced 14 days with discharge greater than 3,000 cfs and 9 days with discharges in excess of 6,000 cfs. Discharges in the following years, however, were quite low. The next discharge greater than 1000 cfs occurred more than three years later (25 days with $1520 < Q < 2000$ cfs in May of 1989) and the next discharge greater than 3,000 cfs did not occur until 1992. Thus, some dispersal of the 1986 sediment occurred in the month following its introduction to the Trinity River, but little additional movement of the sediment is likely to have occurred in the following six years.

The degree of bank building in 1986 is likely to be smaller than that in 1983, because less sediment was introduced into the Trinity River and because the subsequent stage on the main stem is lower. The sediment input was immediately followed by a one-day peak at 4,000 cfs, but subsequent discharges greater 3000 cfs did not occur until three weeks after the tributary sediment input, during which time the finer sediment would have been largely removed from the reach.

Earlier High Flows. With the exception of one day with $Q>6000$ cfs in 1970, the only high flow events that precede the beginning of the gaging record at Fawn lodge occurred in 1963 and 1974. The amount of sediment input associated with these events is unknown. In considering the fate of sediment delivered to the main stem during the 1960 to 1975 period, the different transport pattern of sediments finer and coarser than 1 mm is important.

Post Dam Deposition History

The coarser grained (1 mm to 8 mm) decomposed granitic sediment delivered to the main stem by Grass Valley Creek tends to remains in the channel until removed by subsequent high flows. The 1963 and 1974 floods were quite large (15 days $Q>6000$ cfs in 1963; 9 days $Q>6000$ cfs in 1974) and likely carried much of the tributary-derived fine sediment through the entire study reach. At the same time, most of the sediment delivered by Grass Valley Creek over the 12 year period between 1963 and 1974 would have remained in the main stem until the 1974 flood occurred. Therefore, it is likely that the onset of persistent fine-grained deposition in the Trinity River bed dates from Grass Valley Creek sediment delivery events following 1963.

A large quantity of sediment was delivered from Grass Valley Creek in 1978. Because there was negligible main-stem flow during and following this event, this sediment is likely to have remained on the Trinity River bed near the mouth of Grass Valley Creek for a period of four to five years. The sediment delivered by Grass Valley Creek in 1983 and 1986 was accompanied by large discharges on the Trinity River. Despite the very large sediment influx in 1983, the subsequent magnitude and duration of very high flows make it unlikely that a large deposit of sediment remained near the mouth of Grass Valley Creek at the end of the water year, but instead a large portion of this sediment was dispersed down the Trinity River and beyond the confluence with the North Fork.

Deposition of the finer-grained (<1 mm) sediment forming the steep banks of the present day channel requires both a high stage and a high sediment concentration. The very large floods of 1963 and 1974 are likely to have had the necessary high sediment concentrations during at least a part of the flood. Aerial photographs show establishment of some riprarian vegetation by 1965. Channel cross sections were surveyed in 1961 and 1965 by the USGS; unfortunately, most of these sections were disturbed by bulldozers prior to the resurvey. One undisturbed sections about 1500 ft downstream of the Rush Creek confluence clearly shows deposition of a berm along the right bank of the low flow channel and growth of a "profusion of young willows ... along the right bank" was reported by Ritter (1968).

Bank deposition during the 1963 flood may have been limited, however, because only three years had passed since the dam was closed and vegetation that had encroached into the channel would have been small and susceptible to removal by the 1963 flood, which had an instantaneous peak discharge of 12,700 cfs. The low discharges in the subsequent 12 years, on the other hand, would have given the encroaching vegetation time to become well-established, so that the 1974 flood was unable to remove it. Because the vegetation produces a large resistance to the flow along the channel margins, over bank velocities are very low and the fine-grained sediment is able to settle out, thereby building the banks. It is likely that the first major episode of bank building on the Trinity River occurred during the 1974 flood, although the growth of vegetation within the former active channel over the preceding 12 years was necessary to permit this bank building to occur during the 1974 flood.

The only other occurrence of combined high river stage and high sediment concentration during the post-dam period occurred during the early March flood of 1983. Thus, review of the past-dam water and sediment discharge history on the Trinity River suggests that the steep, fine-grain banks lining the present channel may have been deposited during only two events: the floods of January 1974 and March 1983. Because the residence time of fine-grained suspended sediment within the study reach during such floods is well under one day, the duration of the depositional periods with the necessary sediment concentration and river stage is quite limited. This duration in 1974 is unknown, because there was no water and sediment gage operating at that time. The peak sediment concentrations in 1983 occurred over a period of 5 days (2/28 to 3/4). It is likely that the steep, fine-grained banks characteristic of the present day channel of the Trinity River were deposited in two events over a total duration as short as two weeks.

Release Timing

Specification of flush timing must include consideration of the periods of spawning, incubation, and migration of the anadromous salmonids. Releases in May or June will avoid the scour of active redds and can assist the downstream migration of juveniles. A different timing option is to release water simultaneously with high flows on the tributaries, which occur in winter and spring, mostly in response to rain and rain-on-snow runoff. If the release is timed and calibrated to supplement the tributary inflow in order to bring the river discharge up to a specified value, a direct savings in reservoir water is achieved and a high discharge is provided to immediately flush the tributary-derived sediment. Such a timed release also has the advantage of permitting high flows during the winter season, giving a release plan that more closely approximates the river's natural seasonal runoff pattern.

The timing of synchronous flushing releases could be based on weather forecasts and a simple model of basin lag times, or determined more precisely by gage data telemetered from tributary gages.

There are several disadvantages to a timed release plan. The first stems from the relatively small discharges that may be expected from the tributaries. The drainage area of the main stem is nearly 10 times greater than that of the tributaries draining into the reach immediately downstream of Lewiston Dam. The typical flood discharge from these tributaries is much smaller than is required for flushing the main stem. The median annual flood on the largest tributary, Grass Valley Creek, is 357 cfs, using daily mean discharges (Table 6.6.2). The median value of the largest daily mean discharge for five consecutive days, which is a more likely target for release timing is 193 cfs. Assuming that the remainder of the tributaries contribute the same amount of runoff at the same time, the result is that one may expect the tributary supplement to the main-stem flow to exceed 600 cfs in only half the years.

Although substituting tributary water for reservoir water represents a savings, it is a small proportion of the discharge required for flushing the main tributary. This savings is partly balanced by the equipment and personnel costs of a timed release and the potential safety concerns involved with releasing a flood discharge on short notice. Requirements for gradual ramping of release discharges pose an additional constraint on synchronous flushing. Typical ramping schedules would have to begin considerably in advance of the peak precipitation, without reliable knowledge of the actual magnitude of tributary high flows.

A more severe disadvantage of the timed release option is that it is likely to lead to further deposition on the steep, fine-grained bank along the channel margins. The banks represent an important habitat constraint, because the river discharge is concentrated into a deeper channel with higher velocities, providing few refugia from high flows for fish, especially juvenile salmonids. This problem has been recognized along the Trinity River and these levees were mechanically removed at a number of sites in 1994. Bank removal, however, does not remove the problem of fine-grained deposition along the channel margin. The largest release discharge currently under discussion is 8500 cfs, which is roughly one half the mean annual flood for the pre-dam channel. If the banks are removed, restoring the channel to its pre-dam geometry, such a discharge is not likely to prevent channel-margin recolonization of vegetation, which then stabilizes the banks and promotes further deposition of fine-grained banks. The rate of bank sedimentation will be accelerated if high flow releases coincide with delivery of fine sediment from the tributaries, because this will allow sediment-laden water to overtop the banks, flow into riprarian vegetation, and deposit sediment on the banks.

The historical analysis above suggests that these banks have deposited over a very short period, as little as two weeks over the entire post-dam period. We recommend that flushing releases should not be timed to coincide with tributary floods, because the relatively minor savings in water are balanced by monitoring costs and safety constraints, and because timed releases can accelerate the deposition of fine-grained banks, which are an important associated sedimentation problem caused by the post-dam alteration of the Trinity River morphology.

Bank-building is a problem only for flushing releases timed to coincide with tributary discharges, because these are the only releases likely to contain large concentrations of the necessary fine-grained sediment. Flushing flows released without coincident tributary flows contain little suspended sediment. We collected suspended sediment samples during the 5800 cfs release in 1992 and found essentially no suspended load, although there was active near-bed transport of the coarse sand and fine gravel material found in the bed.

If a synchronous release is not used, the interim fate of the tributary-derived sediment and its effect on active redds must be considered. Here, it is important to differentiate the transport and depositional patterns of the sediments deposited in the river bed and on the channel banks. There is almost no overlap in their grain size: the bank sediments are predominantly finer than 1 mm, whereas the fine sediment within the channel bed is predominantly 1 mm to 8 mm. This is a common sorting process wherein finer grained sediments cannot deposit in the swifter current over the river bed, are carried higher in the flow and deposit in low velocity regions along the channel margins.

The absence of sediment finer than 1 mm in the bed of the river suggests that the discharges naturally associated with the influx of tributary-derived sediments are sufficient to remove this fine-grained sediment from the river system and that infiltration of this sediment into redds does not occur. The potential impact of this sediment on incubating eggs is to reduce the intragravel flow, resulting in low levels of dissolved oxygen in redds. Low levels of dissolved oxygen in redds have not been observed in measurements made by USFWS, nor have exhumed redds contained suffocated eggs. These observations further support the conclusion that sediments finer than 1 mm are transported downstream by tributary-derived flows and that their removal does not require augmentation by releases from the dam.

The coarser 1 to 8 mm sediment delivered by tributary storms cannot be transported through the main-stem channel with tributary inflows alone, and will be deposited near the confluence until a high discharge on the main-stem disperses the sediment downstream. These sediments have an important impact on the salmonid population. Although they are coarse enough permit adequate intragravel flow (thereby maintaining dissolved oxygen for incubation and hatching), these sediments do fill gravel interstices and prevent alevins from emerging from the bed. Redds exhumed in the Trinity River by USFWS have contained successfully developed alevins that were unable to emerge due to the presence of these sediments. During most tributary inflows and without a reservoir release, these sediments pose a risk of infiltrating active redds only in the immediate vicinity of a confluence. A large synchronous flushing release can remove some of the coarser fine sediments from the study reach, but at the expense of distributing the sediment influx throughout the reach while redds are occupied, while also increasing the risk of scouring active redds and accelerating the rate at which fine sediments are deposited on the river banks.

7. CONCLUSIONS AND RECOMMENDATIONS

Three trial flushing flow releases on the Trinity River below Lewiston Dam were made in 1991, 1992, and 1993 with the purpose of evaluating the effectiveness of high flushing releases in restoring fish habitat. The objectives of this project were to document the effectiveness of these releases in removing fine-grained sediments from the channel bed and to develop recommendations for future flushing releases to clean and maintain the potential spawning gravels along a reach 13 mi (21 km) below Lewiston Dam.

The following objectives define an optimum sediment maintenance flushing flow:

(1) maximize removal of fine-grained sediment from the river reach

(2) minimize the water consumed in the flushing release

(3) minimize the downstream loss of gravels, while also providing sufficient gravel entrainment to permit flushing of the sediment bed at depth.

Removing Sand from the River Bed

The bed of the study reach is composed of gravel and cobble clasts with up to 30% finer grained material (grain size less than 8 mm) embedded within the coarser grains. The fine-grained sediments, when intruded into the river bed, can pose two problems for incubating salmonid eggs and alevin. A large concentration of fine sediments within the spawning gravels can decrease intragravel flow, thereby reducing the removal of metabolic wastes and lowering dissolved oxygen within the gravels, causing the incubating eggs to suffocate. This problem is commonly associated with sediments finer than 1 mm. Fine sediments can also block the emergence of alevin from within the spawning gravel beds. This problem is commonly associated with grains between 1-8mm. Most of the intruded sediment in the Trinity River falls between 1-8mm, which suggests that decreased salmonid spawning success is due primarily to impeded alevin emergence. This is supported by the observation of dead alevin, but no suffocated eggs, in excavated redds on the Trinity River. A successful sediment maintenance flow on this portion of the Trinity River should remove most of the sediment in this size range.

A range of small discharges exists over which the finer grained sediment can be transported over the bed surface at a relatively small rate without any measurable transport of the gravel. The volume of sediment that can be removed in such a flow is limited not only by the small transport rate, but also by the fact that no sediment can be removed from beneath the bed surface. Two trial releases at a discharge of roughly 2800 cfs (80 m^3/s) produced negligible entrainment of the gravels. The flushing effectiveness of these discharges is limited to removal of sand from the bed surface; the bed cannot be flushed at any depth.

A release of 5800 cfs (164 m^3/s) for 5 days in 1992 was just sufficient to mobilize the surface gravel layer and entrain underlying finer sediment. Although the gravel transport rate was quite small, the gravel on the bed surface was almost completely entrained over the course of the five-day peak flow. This combination of flow strength and duration was just sufficient to dislodge nearly all of the gravel grains present on the bed surface, although many of these grains moved only a small distance downstream. In the presence of a sand concentration in the river bed that is much smaller than presently found, such a release could provide flushing to a depth of 15 cm to 20 cm within the bed.

Recommendation: The entrainment and flow observations lead us to recommend that a release of 6000 cfs (170 m^3/s) for 5 days is a minimum for entraining the bed gravels of the Trinity River to achieve flushing below the bed surface. Other discharges may produce a similar degree of gravel mobilization, but will require a different release duration to achieve the same result. Because the frequency of gravel entrainment increases very rapidly with discharge, larger discharges will mobilize the gravel more efficiently. The most efficient release for gravel entrainment would be the largest possible. For example, a discharge of 8500 cfs (240 m^3/s) for one day would achieve the same degree of gravel entrainment as a discharge of 6000 cfs for five days, but would use approximately 70% less water. High discharges do not, however, provide an optimum combination of maximum sand removal and minimum gravel loss because the amount of gravel transported is very large.

<u>Removing Sand from the Study Reach</u>

The overall quantity of sand in the study reach is large. Although the 1992 release reduced the proportion of sand on the bed surface in many places, it did not produce a substantial reduction in the proportion of fine materials in the bed. In the presence of a high sand concentration, a discharge sufficient to entrain the bed gravels and flush sand at depth will not produce a markedly cleaner bed because sand will be redeposited with the gravel. To achieve successful flushing at depth, the total volume of sand in the reach must be reduced.

The rate at which sand can be removed by a flushing release may be increased by dredging pools to act as sediment traps. Dredged pools decrease the effective reach length from which sediment must be removed by the flow. By increasing the quantity of sand that may be removed from the river with a given volume of water, dredged pools offer the potential of decreasing both the water used in a flushing release and the downstream loss of gravel.

The efficiency of a flushing release depends on the discharge and volume of water used, and the extent of pool dredging. We evaluate the effectiveness of different flushing options by calculating the sand removal and downstream gravel transport for different combinations of water volume, discharge, pool dredging. To do this, we estimate the quantity of sand presently in the study reach and develop relations between water discharge and sand transport, gravel transport, and pool trapping. These are then used in a sediment routing formulation to calculate sand removal as a function of water volume, discharge, and pool dredging. The sand-removal efficiency of the various flushing options can then be evaluated in terms of water use, gravel loss, and dredging volume.

The optimum magnitude of a sand removal discharge is a compromise. Larger discharges produce more efficient sand transport and allow finer-grained sediment to be entrained from below the bed surface. But larger releases also reduce the trap efficiency of the pools, increase the transport rate and downstream loss of gravel, and cause more of the pool storage volume to be filled with gravel, rather than sand. We find that a discharge between 5,000 cfs (142 m^3/s) and 6,000 cfs (170 m^3/s) provides the greatest efficiency in sand removal, while keeping gravel loss to the minimum required to mobilize the bed.

Dredged pools greatly increase the amount of sand that can be removed from the reach and do so at a small cost relative to that of the released water. The volume of sand that may be trapped by a pool for a particular discharge will depend on its width, length, and the depth to which it is dredged. For a given water discharge, individual pools will have a stable depth at which the rate of sand input and output is roughly balanced. Dredging below this depth is necessary to trap sand efficiently. The stable depth depends on the rate of sand delivery to the pool, and on the water discharge and pool geometry, which determine the transport capacity of the pool. We develop a method for estimating stable pool depth and test the method against pool observations made during the trial releases. In the sand routing analysis, we specify dredging depth relative to the stable pool depth in order to permit a direct comparison among different dredging options.

For a discharge of 5,000 cfs to 6,000 cfs and a volume of released water less than 125,000 acre-feet, which includes those used in the trial releases, dredging pools to a depth of two feet below stable depth provides sufficient storage to trap most of the sediment delivered to the pools. Deeper dredging depths would be required to efficiently trap the sediment delivered by the release of a larger volume of water.

The location of dredged pools will determine the spatial distribution of sand removal. If an even distribution of sand removal is desired, pools must be located throughout the reach. The existing dredged pools are all located in the upper half of the study reach. If these pools are dredged, a release using 100,000 to 150,000 acre-feet of water can provide substantial sand removal in upstream half of the study reach, but much less sand removal further downstream.

Recommendation. A flushing discharge between 5,000 cfs (142 m^3/s) and 6,000 cfs (170 m^3/s) provides the greatest efficiency in sand removal, while keeping gravel loss to the minimum required to mobilize the bed. We recommend that pools be dredged to act as sediment traps. If possible, two additional pools should be added to the downstream portion of the study reach from Society Pool to Steelbridge. This reach is the longest with no pools and has the largest sand content. Without new pools in the downstream half of the study reach, the sand removal that can be accomplished in this reach is limited and much larger volumes will be needed to clean the bed sediment. With pool dredging, a discharge of 5,000 cfs (142 m^3/s) and 6,000 cfs (170 m^3/s) gives a combination of sand discharge rate and pool trap efficiency that offers the largest sand removal for the smallest volume of water used, while also limiting the downstream loss of gravels from the study reach.

Release Timing

Specification of flush timing must include consideration of the periods of spawning, incubation, and migration of the anadromous salmonids. Releases in May or June will avoid the scour of active redds and can assist the downstream migration of juveniles. A different timing option is to release water simultaneously with high flows on the tributaries, which occur in winter and spring, mostly in response to rain and rain-on-snow runoff. If the release is timed and calibrated to supplement the tributary inflow in order to bring the river discharge up to a specified value, a direct savings in reservoir water is achieved and a high discharge is provided to immediately flush the tributary-derived sediment. Such a timed release also has the advantage of permitting high flows during the winter season, giving a release plan that more closely approximates the river's natural seasonal runoff pattern.

The potential savings in water from synchronous releases are likely to be small. The drainage area of the main stem of the Trinity River is nearly 10 times greater than that of the tributaries draining into the reach immediately downstream of Lewiston Dam. The median annual flood (calculated using the largest daily mean discharge for each year) on the largest tributary, Grass Valley Creek, is 357 cfs. The median value of the largest discharge for five consecutive days, which is a more likely target for release timing, is 193 cfs. Assuming that the remainder of the tributaries contribute the same amount of runoff at the same time, one may expect the tributary supplement to the main-stem flow to exceed 600 cfs in one-half the years.

Although substituting tributary water for reservoir water represents a savings, it is a small proportion of the discharge required for flushing the main tributary. This savings is partly balanced by the equipment and personnel costs of a timed release and the potential safety concerns involved with releasing a flood discharge on short notice. Requirements for gradual ramping of release discharges pose an additional constraint on synchronous flushing. Typical ramping schedules would have to begin considerably in advance of the peak precipitation, without reliable knowledge of the actual magnitude of tributary high flows.

There are more substantial problems with a synchronous release plan. In most years, high flows on the tributaries occur between December and March. Salmonid redds are occupied during this time, so a synchronous release plan introduces the risk of scouring active redds.

A second important disadvantage of the synchronous release option is that it is likely to lead to further deposition on the steep, fine-grained banks along the channel margins. These banks, together with the widespread deposition of fine-grained sediment within the channel bed, are the primary sedimentation problems produced by the altered flow regime of the Trinity River. The fine-grained banks are built when the discharge exceeds the capacity of the smaller, post-reservoir channel, causing the river stage to exceed the bank elevation, which allows slow moving sediment-laden water to deposit fine-grained material within the alder band on the banks. Deposition of these banks requires a combination of high river stage and a high concentration of fine sediment. An analysis of the post-dam discharge history of the Trinity River, together with the record of sediment discharge from Grass Valley Creek, suggests that bank building is a very rapid process, occurring over a very short period of time, perhaps as small as two weeks in the period following the dam closure.

Although such banks, or levees, are common along alluvial rivers, they pose potential management problems for the Trinity River. As the banks grow in height, flows are confined at higher and higher discharges. The result is concentration of flow into a deeper channel with higher velocities, providing few refugia from high flows for fish, especially juvenile salmonids.

This problem has been recognized along the Trinity River and these banks were mechanically removed at a number of sites in 1994. Bank removal, however, does not remove the problem of fine-grained deposition along the channel margin. The largest release discharge currently under discussion is 8500 cfs, which is roughly one half the mean annual flood for the pre-dam channel. If the banks are removed, restoring the channel to its pre-dam geometry, a discharge of 8500 cfs is not likely to not prevent channel-margin recolonization of vegetation, which then stabilizes the banks and promotes further deposition of fine-grained sediment. The rate of bank sedimentation will be accelerated if high-flow releases from the dam coincide with delivery of sediment from the tributaries because this will allow sediment-laden water to overtop the banks, where it is slowed by the riparian vegetation and able to deposit sediment.

Bank-building is a problem only for flushing releases timed to coincide with tributary discharges, because these are the only releases likely to contain significant concentrations of the fine-grained bank-forming material. Flushing flows released without coincident tributary flows contain little suspended sediment. We collected suspended sediment samples during the 5800 cfs release in 1992 and found essentially no suspended load, although there was active near-bed transport of the coarse sand and fine gravel material found in the bed. The absence of the fine-grained bank-forming material in the bed of the river demonstrates that the discharges naturally associated with the influx of tributary-derived sediments are sufficient to remove this fine-grained sediment from the river system.

Recommendation. Flushing releases should not be timed to coincide with tributary floods, because the relatively minor savings in water are balanced by monitoring costs, safety constraints, increased risk of scouring active redds, and accelerated deposition of the fine-grained banks, which are an important habitat limitation produced by the post-dam alteration of the Trinity River hydrology. Flushing releases in May or June can be scheduled in advance, are unlikely to coincide with tributary floods, avoid scour or sedimentation of active redds, assist the downstream migration of juvenile salmonids, and carry essentially no fine-grained material that would contribute to bank building.

Further Work

The selection of a sediment maintenance release plan depends on a large number of factors, many of which are beyond the scope of this study. These include the value of the water used, the reservoir operating rules, the legal obligations associated with the water, the variation in time of both the supply and demand on the reservoir water, the cost of dredging and imported gravel, and the response of the fishery to changes in physical habitat. Because of the large number of factors and objectives requiring consideration, and because some of the objectives cannot be mutually satisfied, it is not likely that a single optimum flushing release can be identified. Rather, it is necessary to develop a rational basis for evaluating the tradeoffs among the different objectives, so that a compromise may be found that is acceptable to all concerned parties. Further work is needed to develop a decision-making tool that may be used to evaluate these tradeoffs in an objective fashion.

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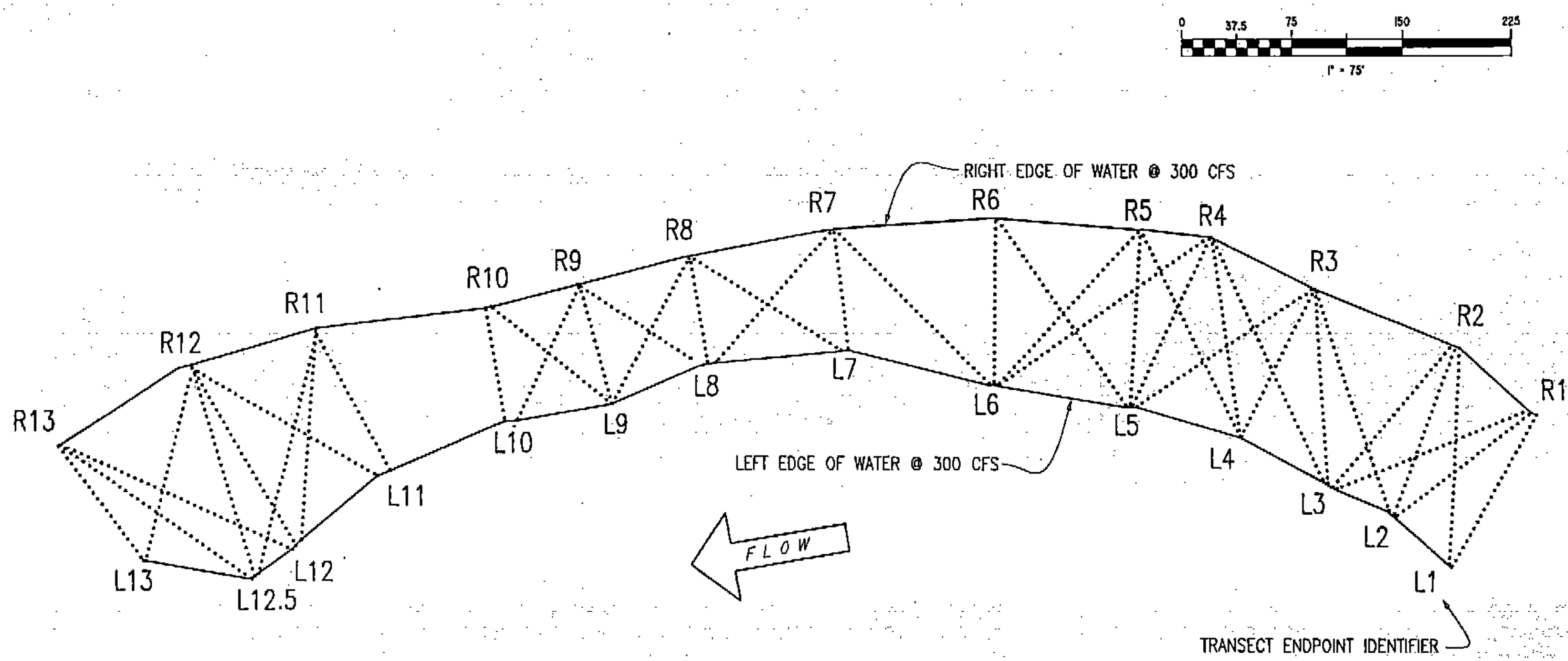
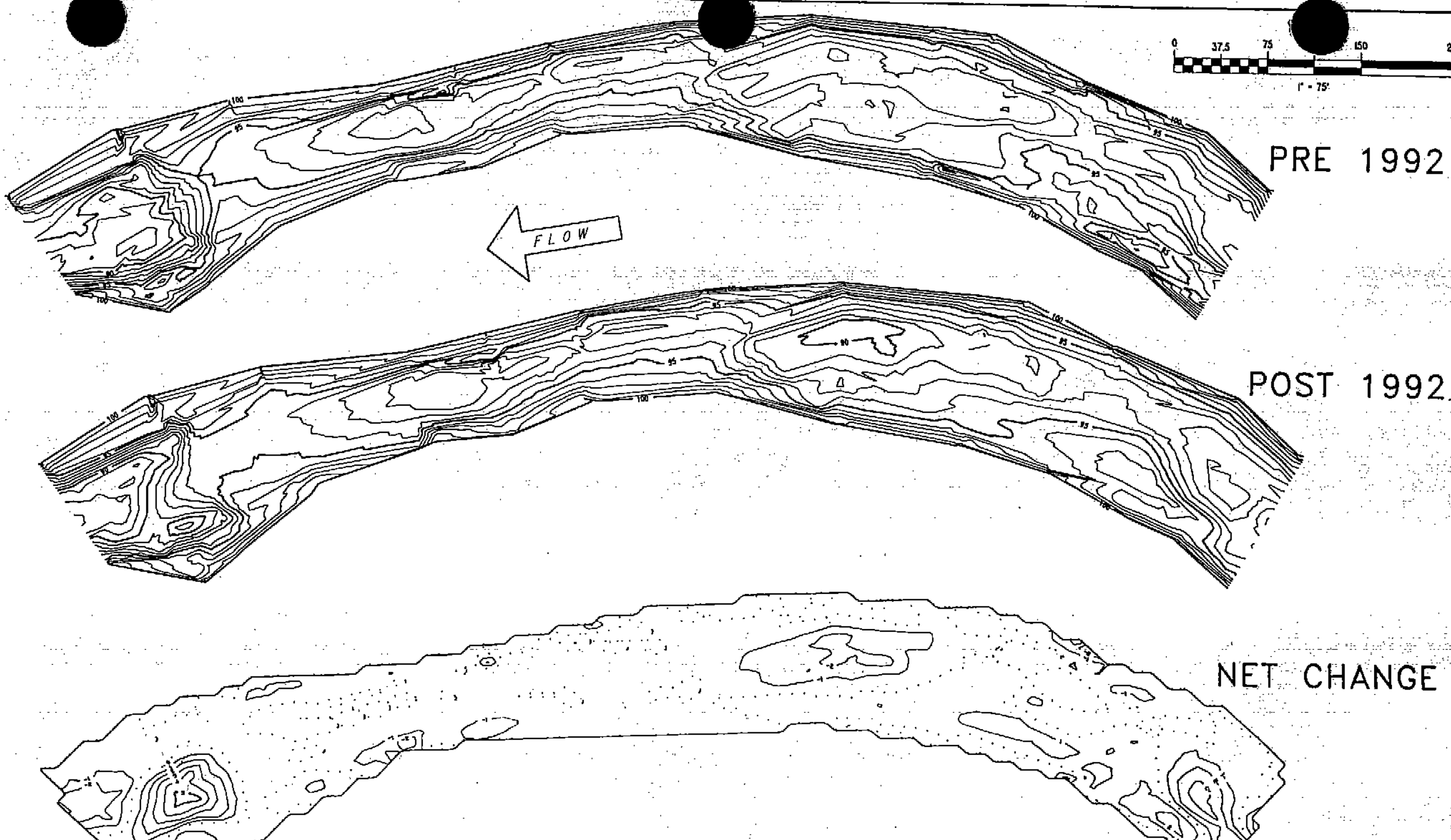
Appendix A. Maps of Pool Topography Pre- and Post-Release, and Net Change During Release, Trinity River.

TRINITY RIVER – SP/PONDEROSA POOL



DEPTH MEASUREMENT LOCATIONS



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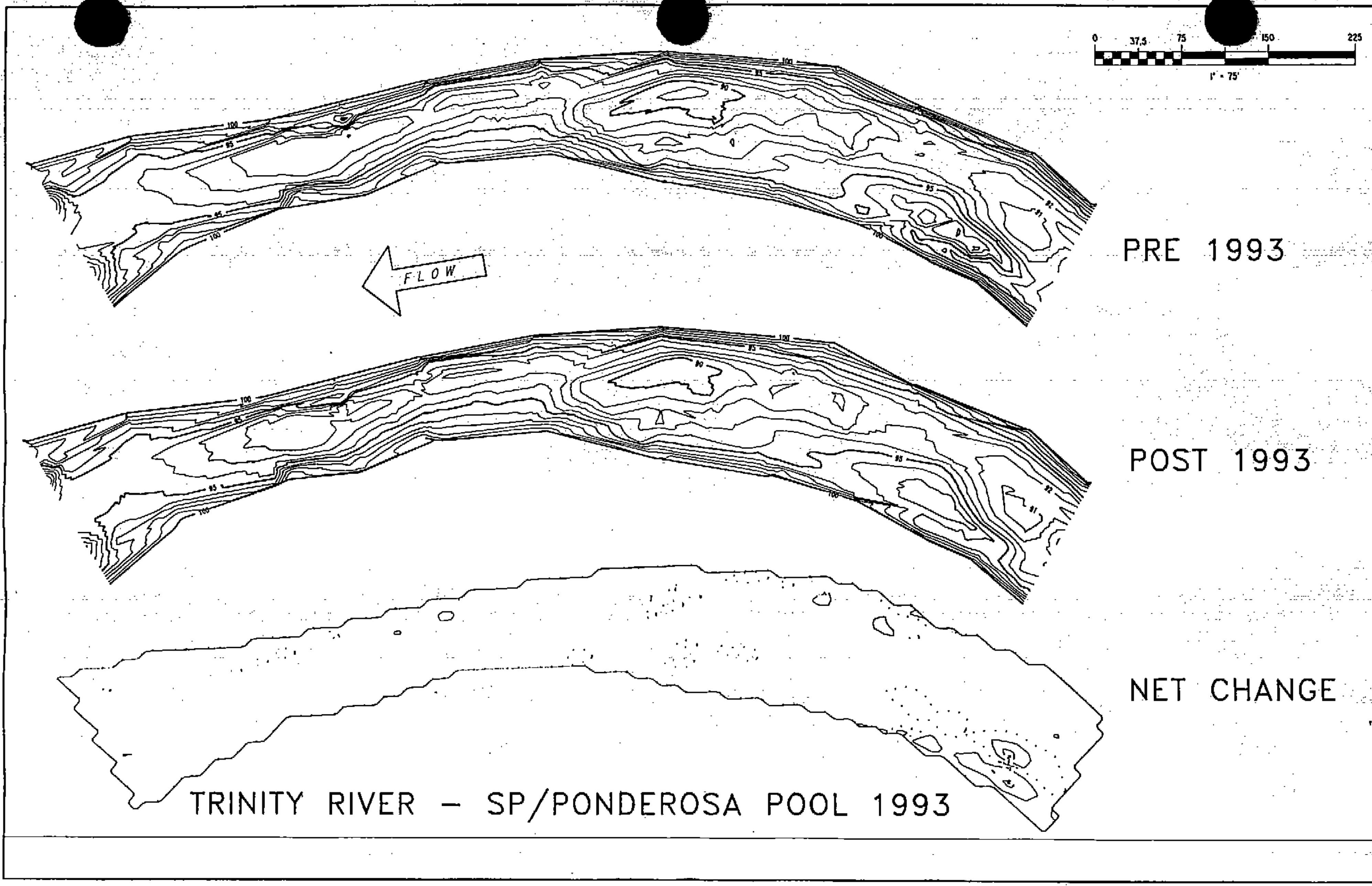
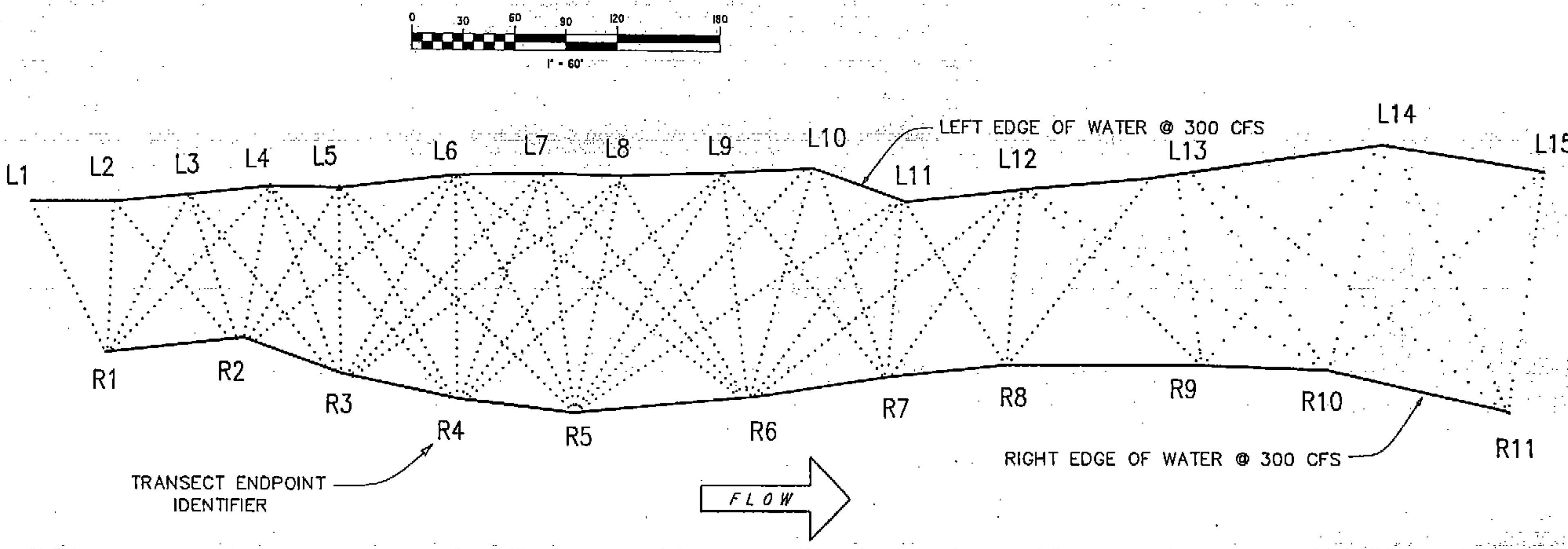
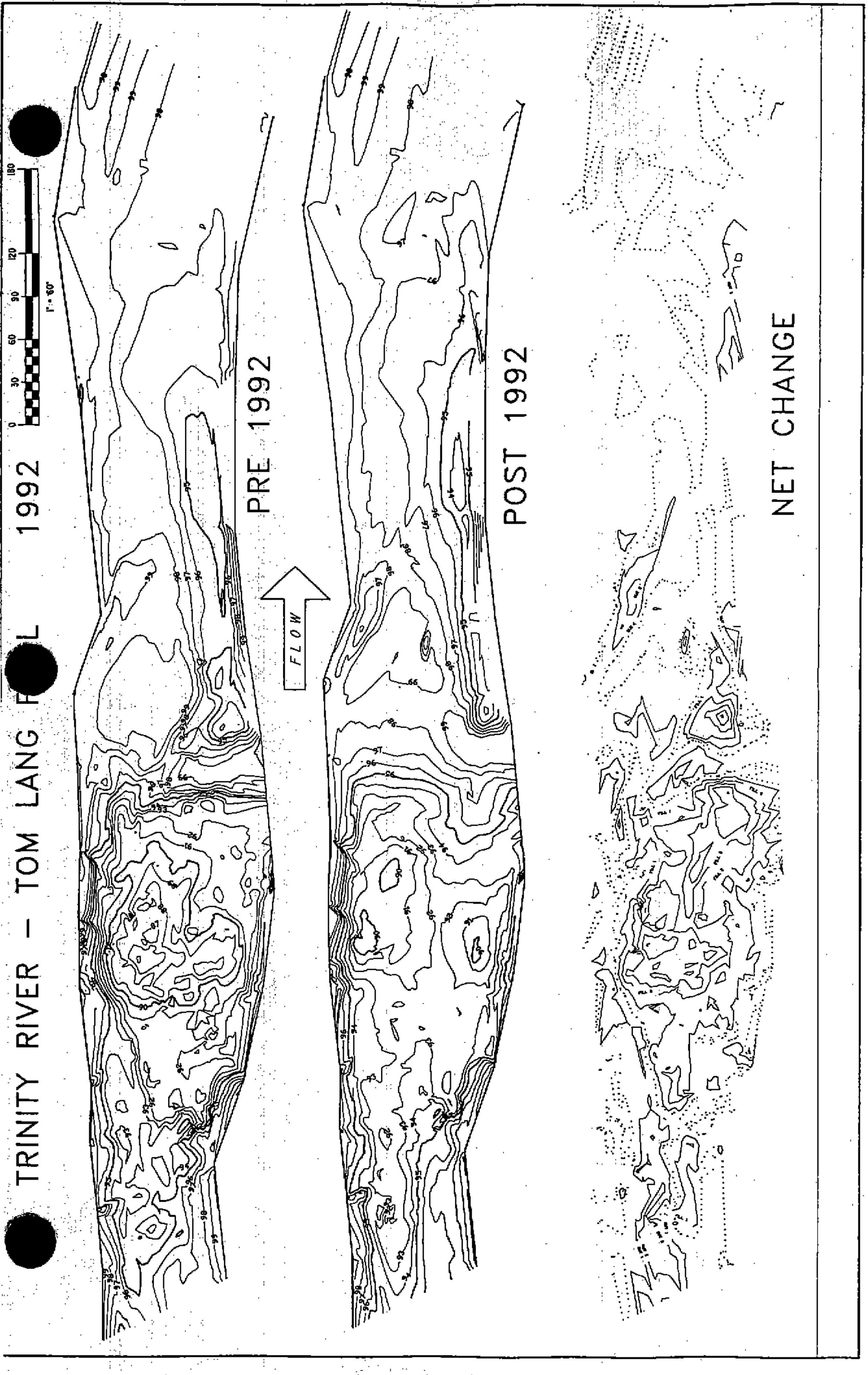

0
37.5
75
150
225
1" = 75'
100
95
90
92
91
FLOW
PRE 1993
POST 1993
NET CHANGE
TRINITY RIVER - SP/PONDEROSA POOL 1993

TRINITY RIVER — TOM LANG POOL



DEPTH MEASUREMENT LOCATIONS


TRINITY RIVER – TOM LANG
1992
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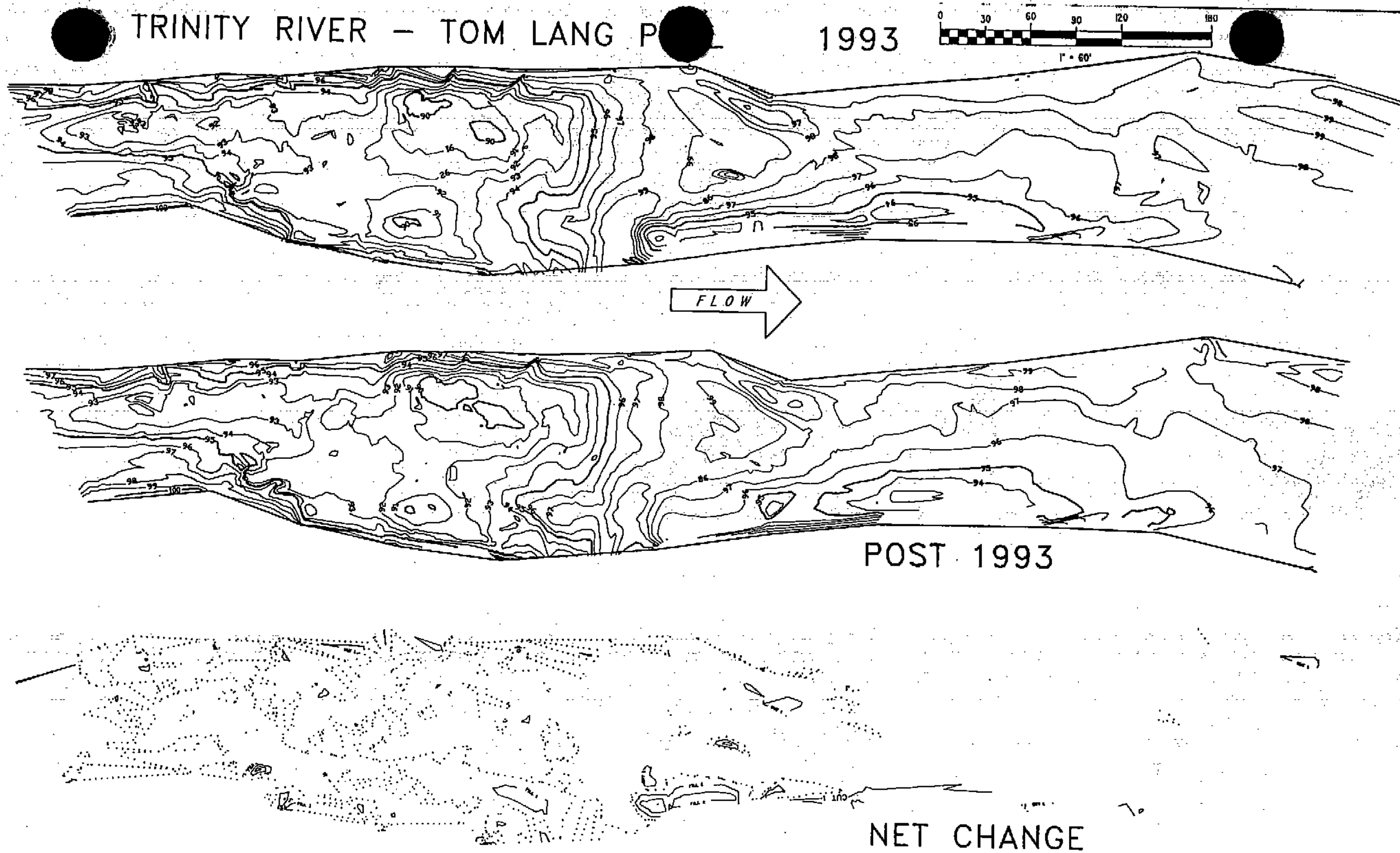
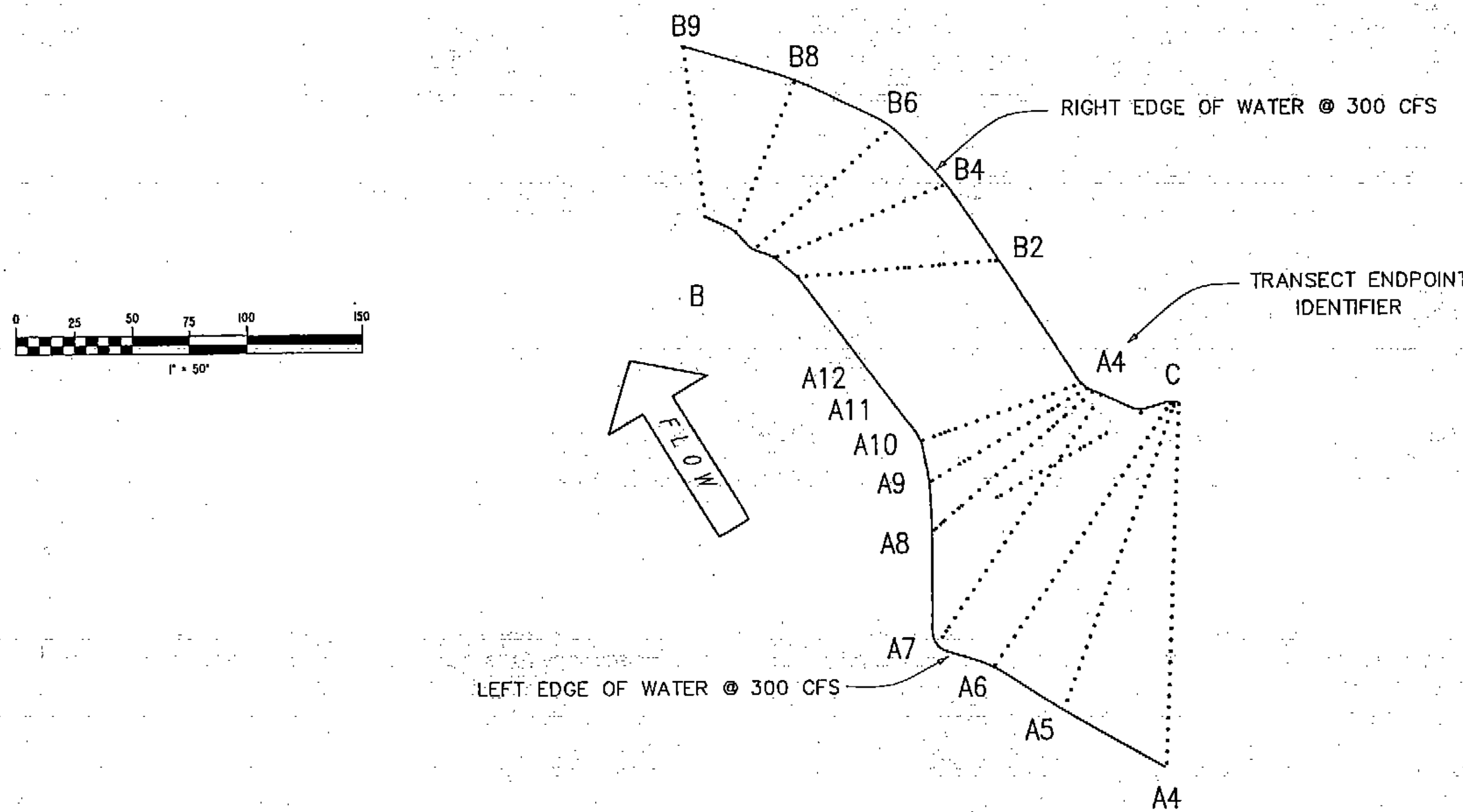
1" = 60'
PRE 1992
FLOW
POST 1992
NET CHANGE

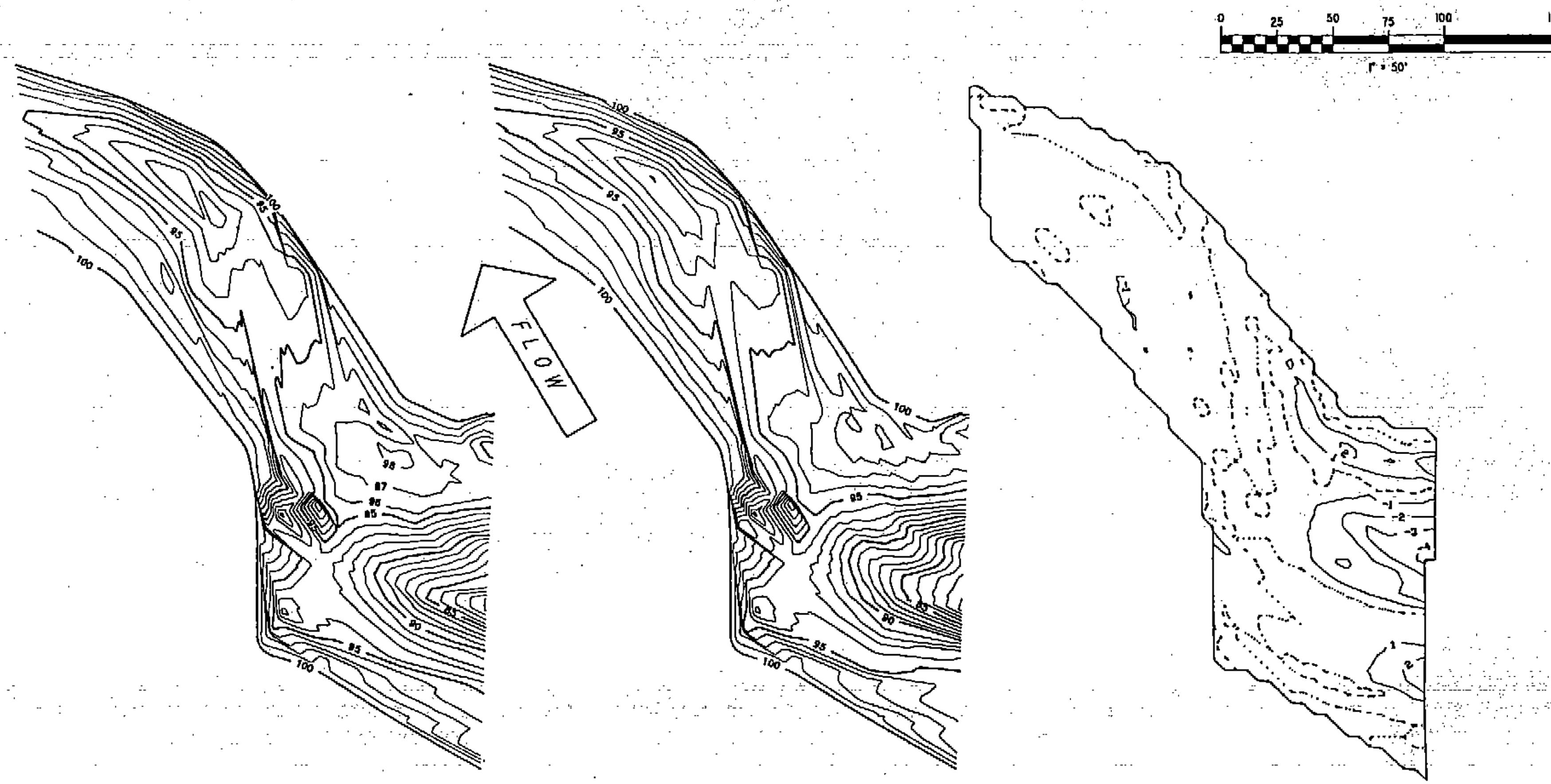

TRINITY RIVER – TOM LANG P
1993
0 30 60 90 120 180
1" = 60'
FLOW
POST 1993
NET CHANGE

TRINITY RIVER – REO STOTT POOL



DEPTH MEASUREMENT LOCATIONS

TRINITY RIVER – REO STOTT POOL 1991

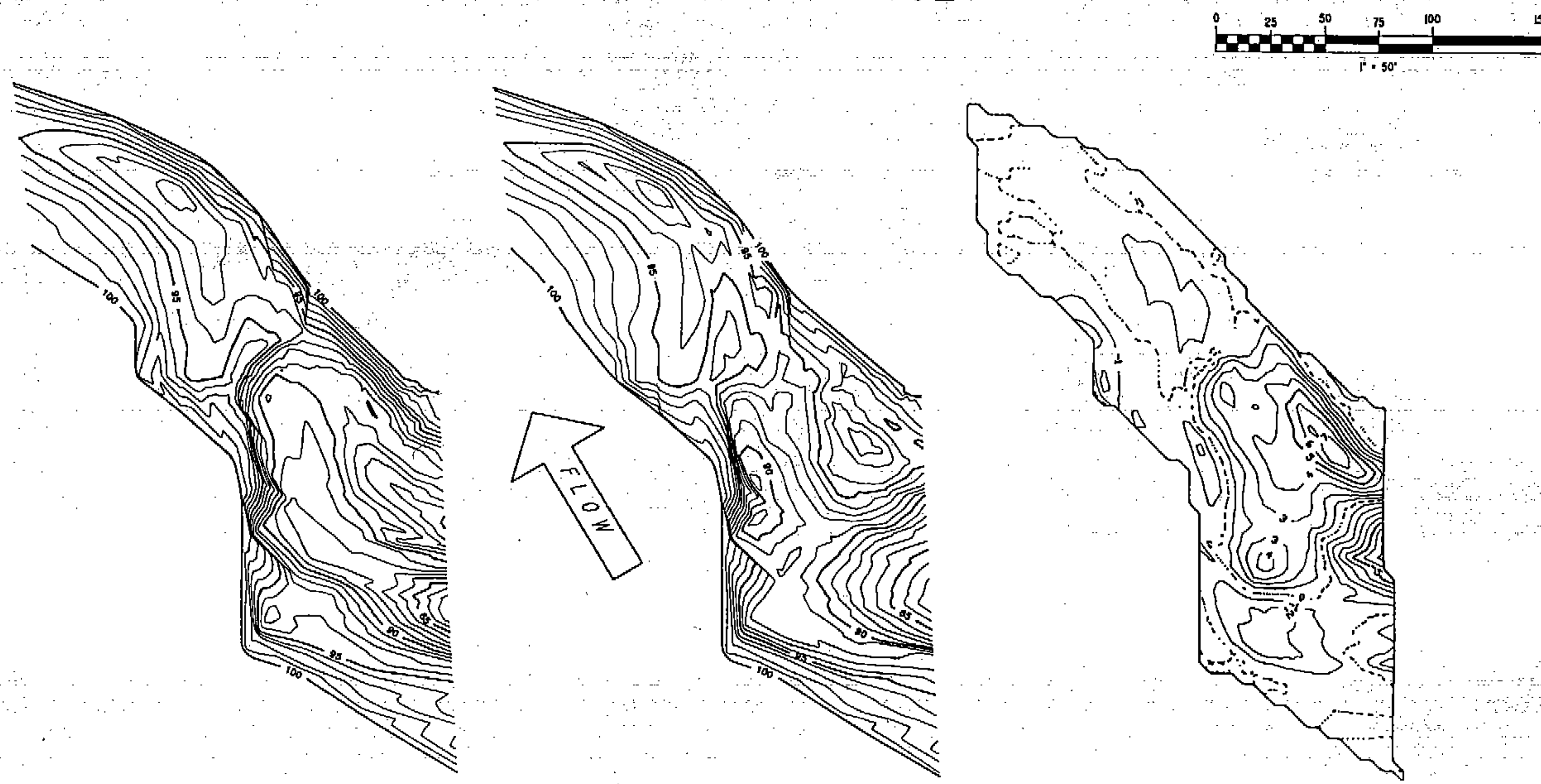


PRE 1991

POST 1991

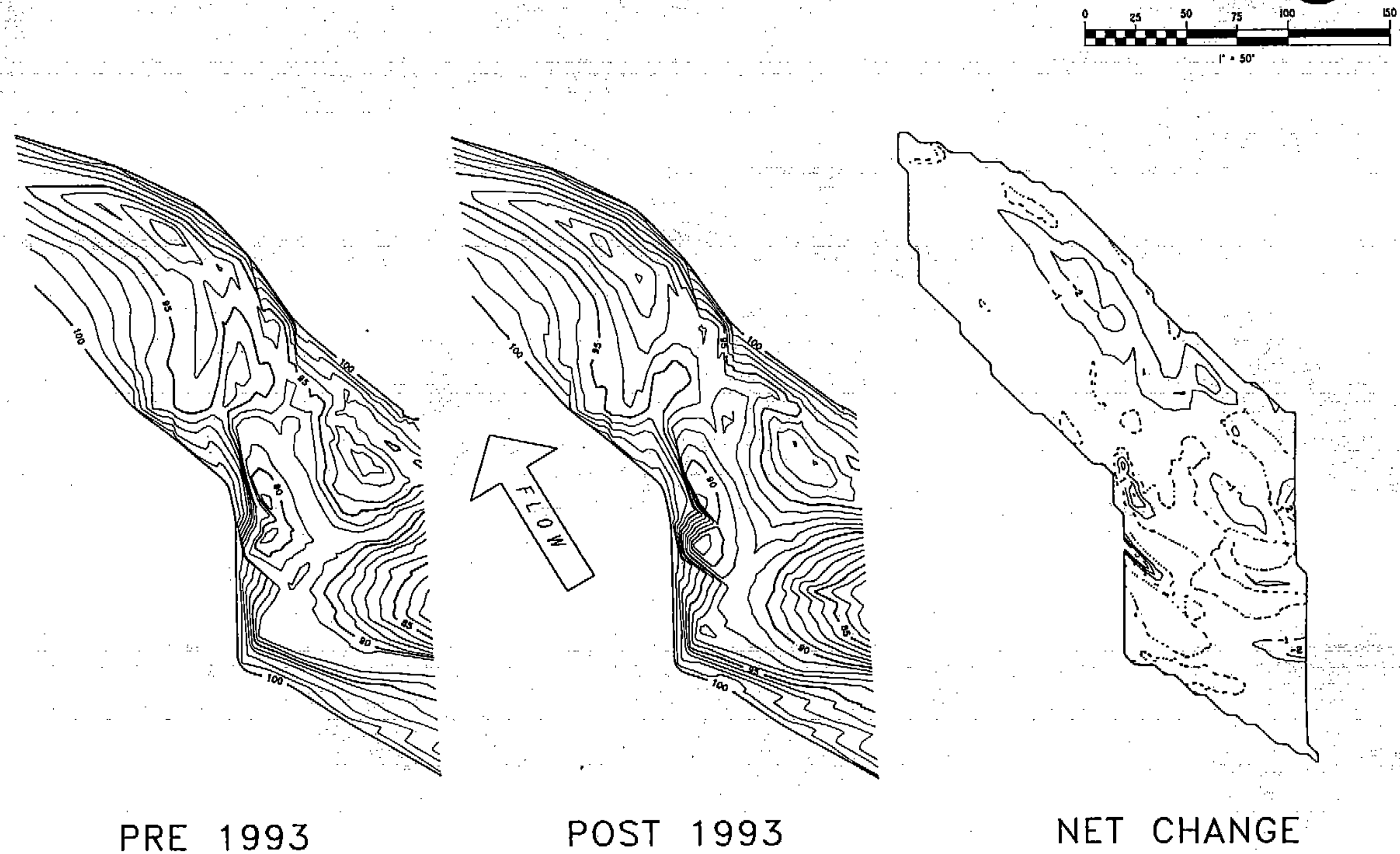
NET CHANGE

TRINITY RIVER – REO STOTT POOL 1992

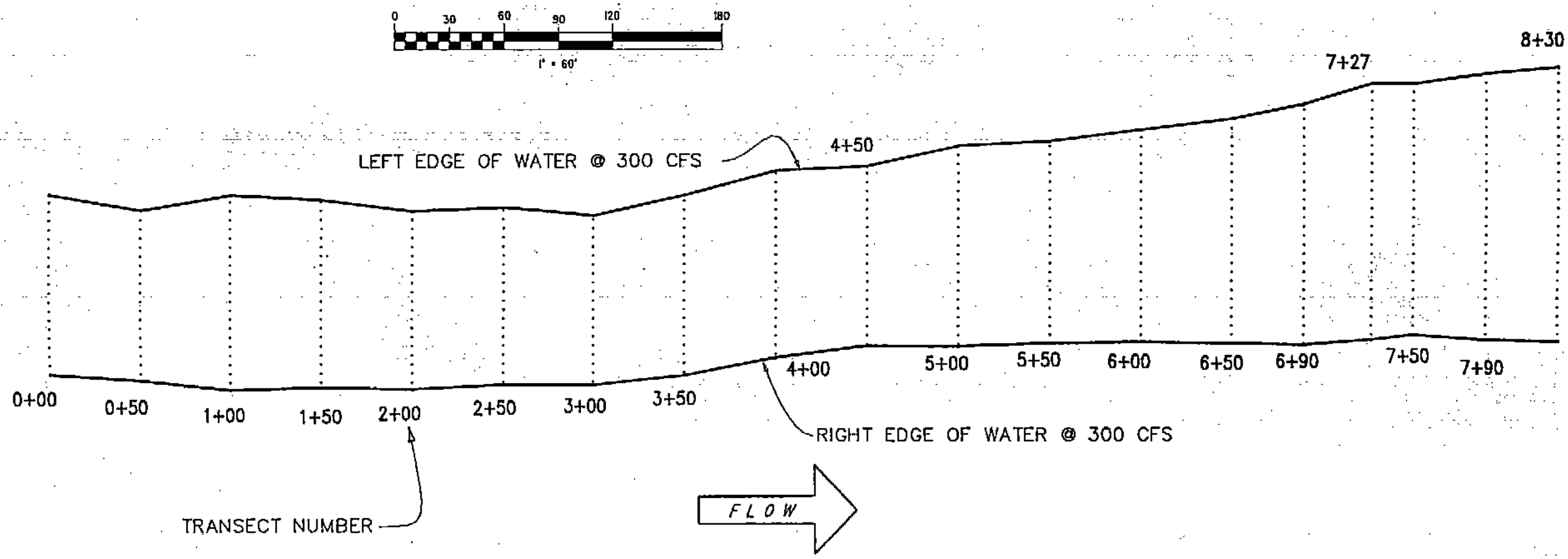


PRE 1992 POST 1992 NET CHANGE

TRINITY RIVER — REO STOTT POOL 1993

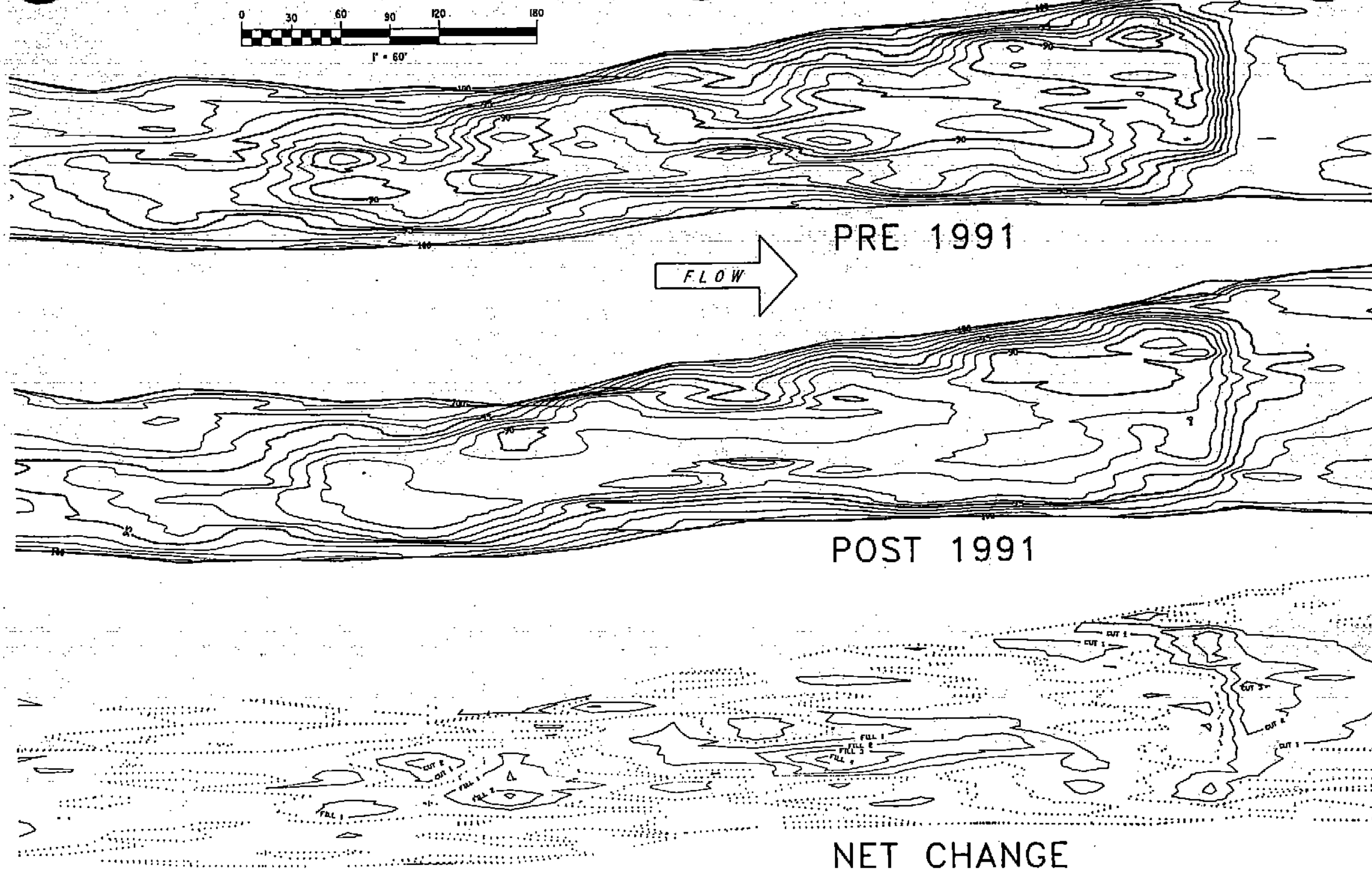


TRINITY RIVER – SOCIETY POOL

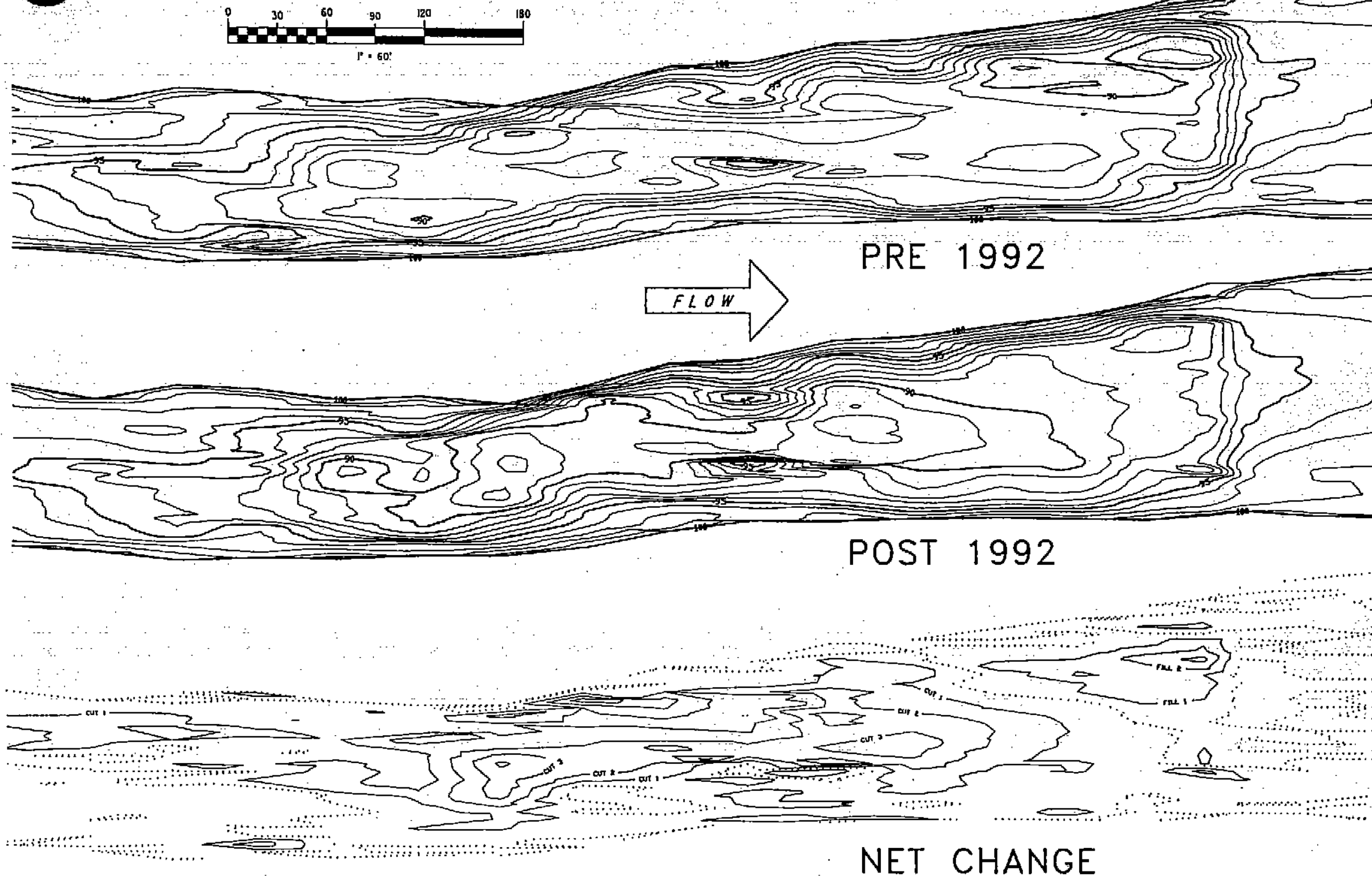


DEPTH MEASUREMENT LOCATIONS

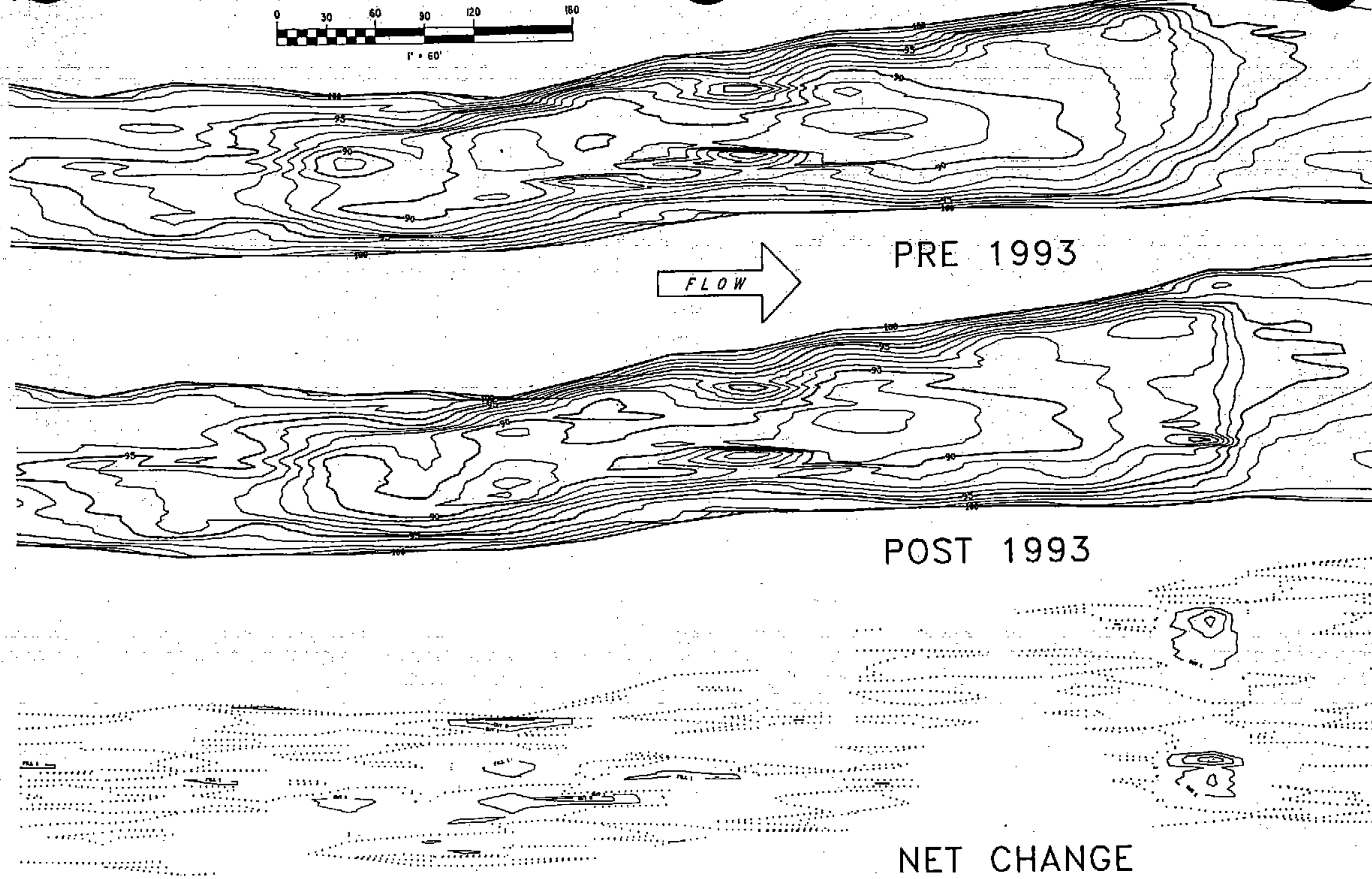
NITY RIVER – SOCIETY POOL 991



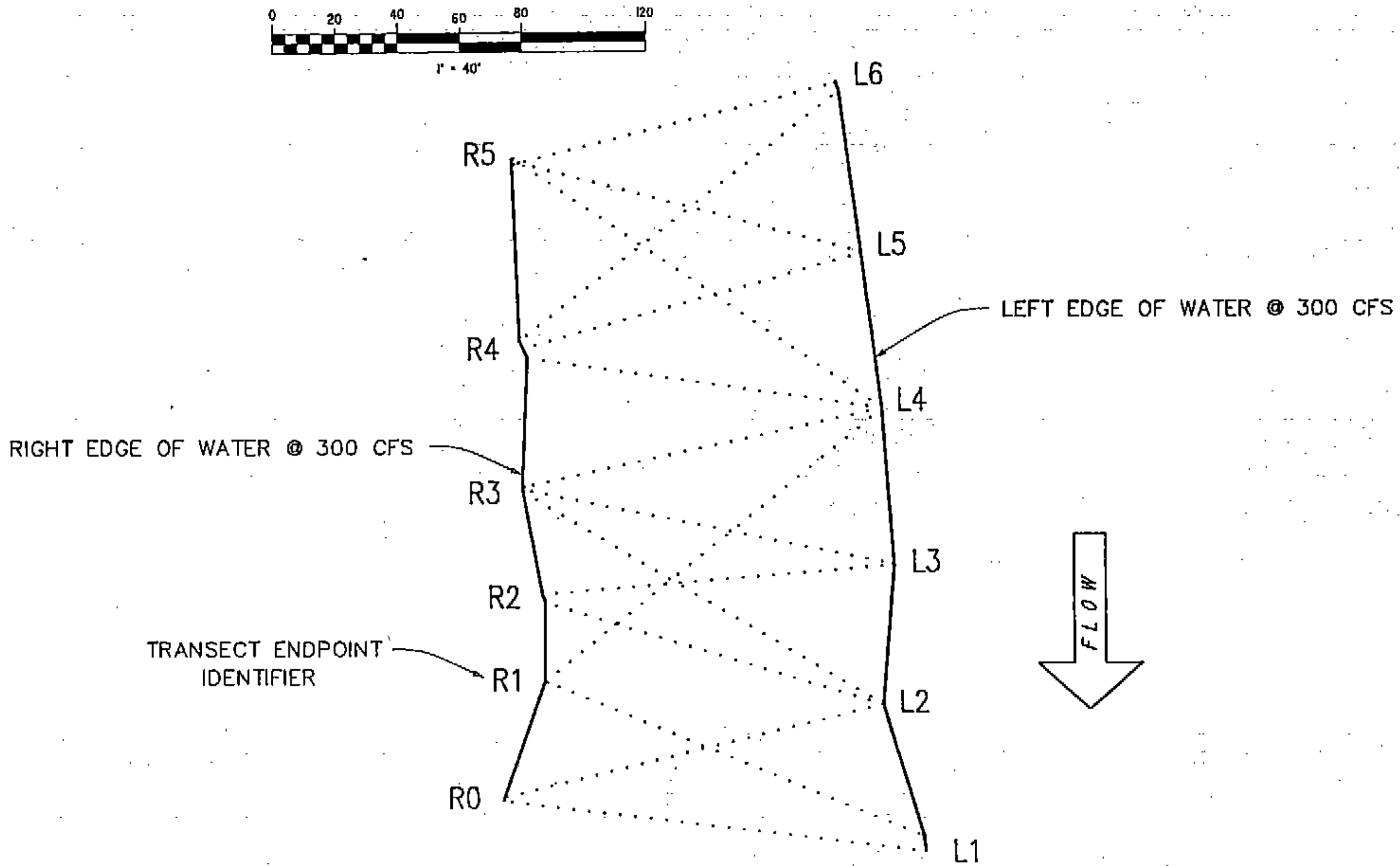
NITY RIVER – SOCIETY POOL 992



T NITY RIVER – SOCIETY POOL 93

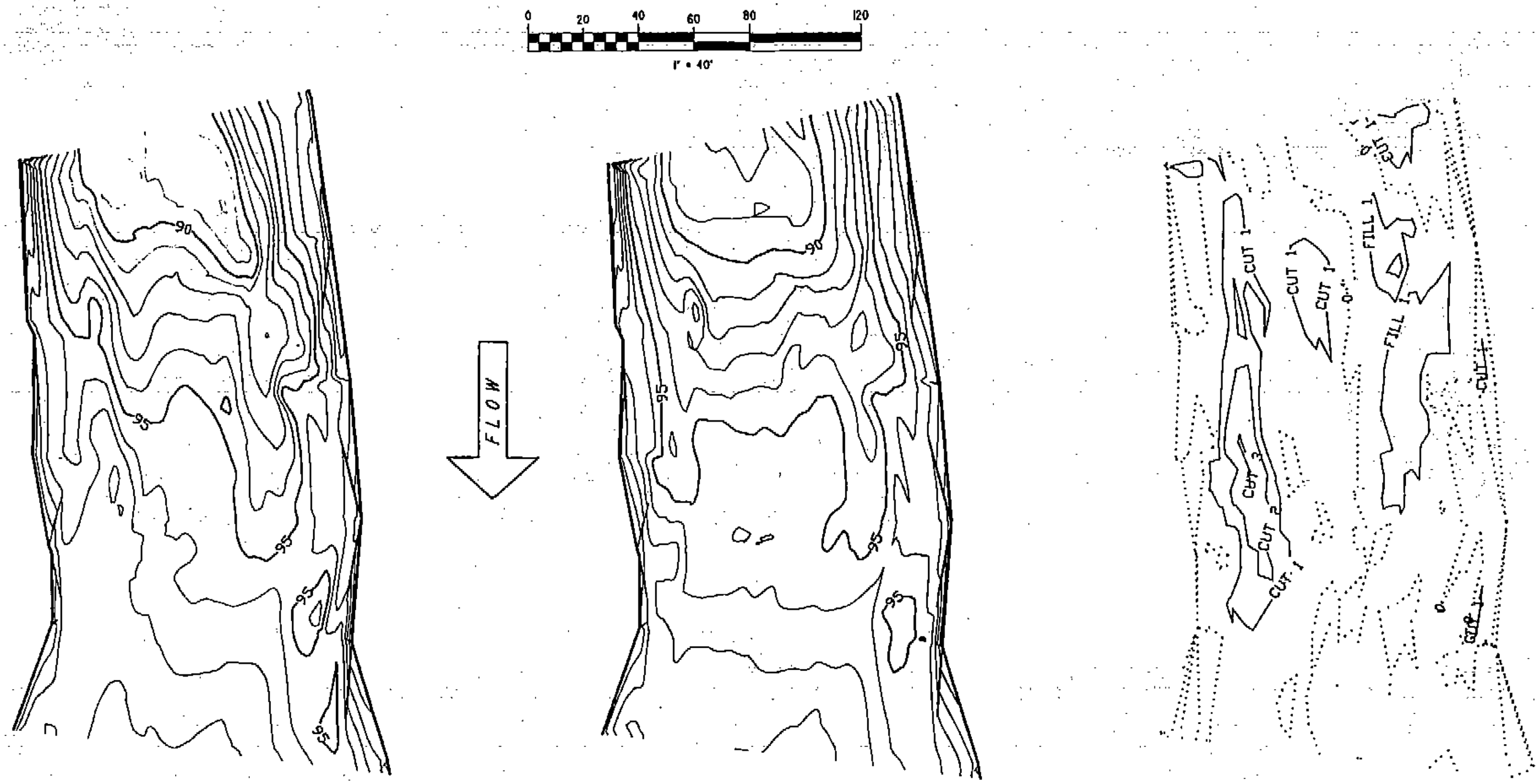


TRINITY RIVER — UPPER STEEL BRIDGE POOL



DEPTH MEASUREMENT LOCATIONS

TRINITY RIVER – UPPER STEEL BRIDGE POOL 1992

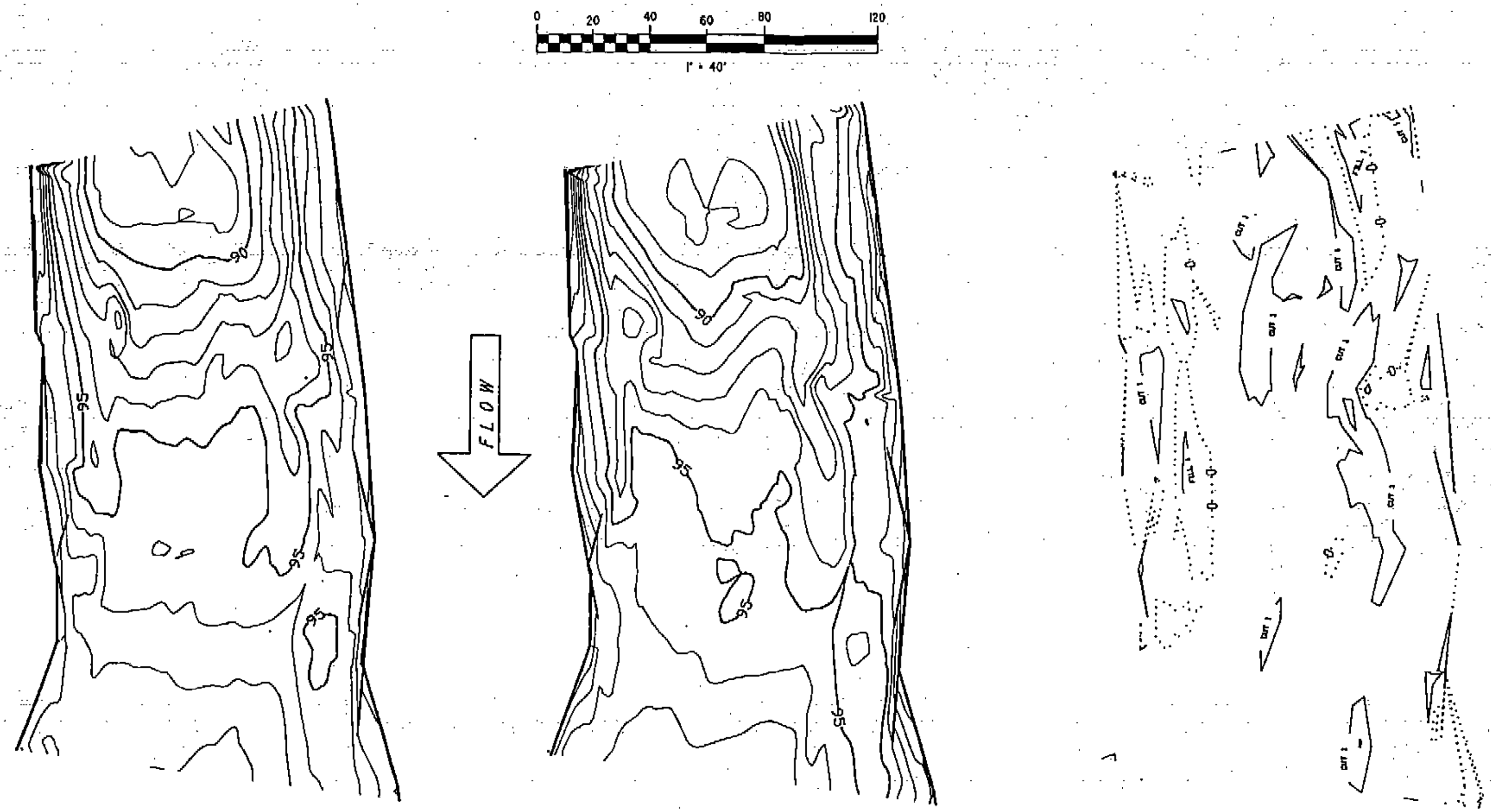


PRE 1992

POST 1992

NET CHANGE

TRINITY RIVER – UPPER STEEL BRIDGE POOL 1993



PRE 1993 POST 1993 NET CHANGE

PB0A 5/28/91			Q =	35.7	PB2 5/28/91			Q =	35.1	
STA	h	U	n		STA	h	U	n		
(m)	(m)	(m/s)			(m)	(m)	(m/s)			
3.0	1.03	0.31	2		14.0	0.85	0.85	14		Notes:
6.0	1.25	0.61	2		16.0	0.85	0.92	14		Q = water discharge (m^3/s)
9.0	1.02	0.66	2		18.0	0.91	0.72	13		STA = Location along cross-section
12.0	0.62	0.69	2		20.0	0.85	0.83	13		h = flow depth
15.0	0.83	0.82	2		22.0	0.85	0.75	13		U = depth-averaged velocity
18.0	1.01	0.86	2		25.0	0.70	0.78	12		n = number observations
21.0	1.08	0.89	2		28.0	0.76	0.90	12		If n=1, reading 0.4h above bed
24.0	1.13	0.90	2		31.0	0.91	0.91	13		
27.0	1.10	0.93	2		34.0	1.16	0.90	13		
30.0	1.07	0.89	2		37.0	1.22	0.92	13		
33.0	1.13	1.06	2		40.0	1.31	0.93	6		
36.0	1.17	1.05	2		43.0	1.68	0.94	7		
39.0	1.08	0.88	2		46.0	2.10	0.91	7		
42.0	1.11	0.74	2		49.0	1.52	0.31	2		
45.0	0.76	0.21	2							
PB0A 5/29/91			Q =	69.3	PB2 5/29/91			Q =	67.1	
STA	h	U	n		STA	h	U	n		
(m)	(m)	(m/s)			(m)	(m)	(m/s)			
2.0	1.26	0.27	2		8.0	0.76	0.17	2		
3.0	1.72	0.46	2		10.0	0.37	0.21	2		
6.0	1.68	0.69	2		12.0	0.55	0.64	2		
9.0	1.42	0.92	2		14.0	1.37	1.35	9		
12.0	1.11	1.01	2		17.0	1.34	1.38	9		
15.0	1.37	1.16	2		20.0	1.37	1.52	13		
18.0	1.48	1.28	2		22.0	1.31	1.15	13		
21.0	1.55	1.29	2		24.0	1.17	1.19	16		
24.0	1.62	1.28	2		26.0	1.13	1.31	13		
27.0	1.60	1.26	2		28.0	1.19	1.31	13		
30.0	1.54	1.18	2		30.0	1.34	1.32	13		
33.0	1.58	1.24	2		32.0	1.46	1.31	14		
36.0	1.63	1.25	2		35.0	1.65	1.22	14		
39.0	1.52	1.18	2		38.0	1.74	1.23	15		
42.0	1.60	0.88	2		41.0	1.89	1.22	14		
44.0	1.49	0.47	2		44.0	2.41	1.12	11		
					47.0	2.53	0.86	8		
					49.0	2.07	0.28	2		
PB0A 5/30/91			Q =	77.0	PB2 5/30/91			Q =	74.3	
STA	h	U	n		STA	h	U	n		
(m)	(m)	(m/s)			(m)	(m)	(m/s)			
2.0	1.36	0.23	2		8.0	0.85	0.16	2		
3.0	1.78	0.44	2		10.0	0.47	0.31	2		
6.0	1.77	0.77	2		12.0	0.61	0.67	2		
9.0	1.52	0.98	2		14.0	1.46	1.07	8		
12.0	1.22	1.05	2		17.0	1.46	1.17	8		
15.0	1.43	1.23	2		20.0	1.46	1.16	8		
18.0	1.55	1.38	2		23.0	1.37	1.25	8		
21.0	1.65	1.37	2		26.0	1.22	1.38	8		
24.0	1.69	1.37	2		28.0	1.28	1.39	8		
27.0	1.66	1.34	2		30.0	1.40	1.38	8		
30.0	1.63	1.24	2		32.0	1.57	1.35	8		
33.0	1.68	1.31	2		35.0	1.74	1.23	9		
36.0	1.72	1.32	2		38.0	1.80	1.29	9		
39.0	1.62	1.28	2		41.0	1.98	1.29	9		
42.0	1.66	0.90	2		44.0	2.47	1.15	9		
45.0	0.99	0.19	1		47.0	2.59	0.85	4		
					49.0	2.13	0.40	3		

Table B.1 Poker Bar Velocity Observations

					PB2 6/11/92			Q =	103	
					STA	h	U	n		
					(m)	(m)	(m/s)			
					15.1	1.62	1.15	3		Notes:
					18.1	1.86	1.24	3		Q = water discharge (m^3/s)
					21.1	1.83	1.43	3		STA = Location along cross-section
					25.1	1.46	1.46	3		h = flow depth
					29.1	1.65	1.68	1		U = depth-averaged velocity
					33.1	1.83	1.53	1		n = number observations
					37.1	2.07	1.47	1		If n=1, reading 0.4h above bed
					41.1	2.26	1.40	1		For n = -6 or -5: reading taken
					44.6	2.90	1.51	1		6 or 5 feet below water surface
					47.8	2.74	1.19	1		
PB1B 6/13/92			Q =	165	PB2 6/13/92			Q =	171	
STA	h	U	n		STA	h	U	n		
(m)	(m)	(m/s)			(m)	(m)	(m/s)			
7.0	1.65	0.34	4							
9.25	2.47	0.82	2		16.1	2.18	1.65	4		
12.0	2.44	1.25	3		19.1	2.26	1.78	4		
15.0	2.35	1.52	3		22.1	2.23	1.98	4		
17.65	2.23	1.73	3		25.1	2.13	2.07	4		
21.0	2.29	1.87	3		28.1	2.18	2.16	4		
24.0	2.38	1.95	3		31.1	2.30	2.17	4		
27.0	2.38	2.02	3		34.1	2.56	2.12	5		
30.0	2.38	1.78	3		37.1	2.62	2.11	4		
33.0	2.50	1.97	3		40.1	2.72	2.00	4		
36.0	2.59	1.89	3		43.1	3.01	1.80	5		
39.0	2.74	1.69	3		46.1	3.30	1.40	5		
41.5	*≈10*	*1.77*	-6		49.1	3.15	0.78	5		
45.0	*>10*	*1.57*	-6							
PB0A 6/14/92			Q =	159	PB2 6/14/92			Q =	164	
STA	h	U	n		STA	h	U	n		
(m)	(m)	(m/s)			(m)	(m)	(m/s)			
6.2	2.62	0.61	3		16.1	2.23	1.54	4		
9.2	2.35	1.03	3		19.1	2.26	1.72	3		
12.2	2.13	1.34	3		22.1	2.21	1.88	3		
15.2	2.38	1.49	3		25.1	2.13	1.94	3		
18.2	2.56	1.54	3		28.1	2.17	1.94	3		
21.2	2.68	1.71	3		31.1	2.28	2.01	3		
24.2	2.74	1.78	3		34.1	2.51	2.02	4		
27.2	>10	1.74	-6		37.1	2.55	1.99	4		
30.2	>10	1.68	-6		40.1	2.72	1.95	4		
33.2	2.74	1.75	3		43.1	2.88	1.78	4		
36.2	*>10*	*1.74*	-6		46.1	3.34	1.54	4		
39.2	2.74	1.68	1		49.1	3.24	0.84	4		
42.2	2.68	1.26	3							
45.2	2.23	0.69	2							

Table B.1 Poker Bar Velocity Observations

Notes:
Q = water discharge (m^3/s)
STA = Location along cross-section
h = flow depth
U = depth-averaged velocity
n = number observations
If n=1, reading 0.4h above bed
For n = -6 or -5: reading taken
6 or 5 feet below water surface

PB2 6/15/92 Q = 163

STA (m)	h (m)	U (m/s)	n
15.6	2.16	1.46	4
17.1	2.19	1.58	4
20.1	2.26	1.80	4
23.1	2.20	1.85	4
26.1	2.01	2.00	4
29.1	2.15	2.11	4
32.1	2.34	2.05	5
35.1	2.58	2.00	5
38.1	2.55	1.95	5
41.1	2.80	1.97	5
44.1	2.99	1.69	5
47.1	3.28	1.27	5
50.1	3.03	0.47	5

PB2A 6/15/92 Q = 164

STA (m)	h (m)	U (m/s)	n
16.0	1.55	0.83	4
19.0	2.21	1.60	4
23.5	2.58	1.76	6
26.0	2.51	1.83	5
29.0	2.46	1.88	5
32.0	2.45	1.99	5
35.0	2.58	2.01	5
38.0	2.70	2.01	5
41.0	2.76	1.88	5
44.0	2.99	1.89	5
47.0	3.22	1.54	5
49.8	3.00	1.03	4

PB2 (Side Channel) 6/16/92

STA (m)	h (m)	U (m/s)	n
2.1	0.79	0.30	3
3.1	0.88	0.17	3
5.1	0.88	0.08	2
7.1	1.52	0.42	2
8.1	1.65	0.61	1
9.1	1.46	0.10	2
10.1	1.34	0.72	3
12.1	1.43	0.82	3

PB2A (Side Channel) 6/16/92

STA (m)	h (m)	U (m/s)	n
2.0	0.88	0.32	3
3.0	0.98	0.04	2
5.0	1.46	0.60	3
6.0	1.40	0.30	3
8.0	1.10	0.09	2
9.0	1.04	0.19	2
10.0	0.94	0.11	2
11.0	1.19	0.14	2
13.0	0.82	0.33	2
15.0	1.11	0.62	3

PB1B 6/17/92 Q = 123

STA (m)	h (m)	U (m/s)	n
7.5	1.58	0.50	3
10.0	2.13	1.02	3
13.0	2.01	1.08	3
17.0	1.77	1.42	3
21.0	1.71	1.53	3
25.0	1.92	1.61	3
29.0	1.92	1.63	3
33.0	1.98	1.75	3
37.0	2.10	1.66	3
41.0	2.29	1.66	3
45.0	*?>10*	*1.64*	-5

PB2 6/17/92 Q = 114

STA (m)	h (m)	U (m/s)	n
17.1	1.78	1.46	3
21.1	1.84	1.55	3
25.1	1.66	1.75	3
29.1	1.72	1.68	3
33.1	1.86	1.79	3
37.1	2.09	1.78	3
41.1	2.30	1.69	3
45.1	2.74	1.52	4
48.1	2.71	0.89	4

PB2A 6/17/92 Q = 117

STA (m)	h (m)	U (m/s)	n
20.2	1.84	1.60	3
24.0	2.13	1.52	3
28.8	1.97	1.69	3
32.8	2.04	1.69	3
36.8	2.19	1.78	4
41.7	2.32	1.73	3
45.0	2.56	1.66	3
47.7	2.71	1.23	4
50.0	1.84	1.17	3

Table B.1 Poker Bar Velocity Observations

PB1B 4/27/93			Q =	78.9	PB2 4/27/93			Q =	82.1	PB2A 4/27/93				
STA	h	U	n		STA	h	U	n		STA	h	U	n	
(m)	(m)	(m/s)			(m)	(m)	(m/s)			(m)	(m)	(m/s)		
14.0	1.49	1.11	4		13.0	0.87	0.96	4		21.5	1.62	1.35	4	
17.0	1.31	1.23	4		16.0	1.46	1.24	4		25.0	1.65	1.25	4	
20.0	1.25	1.24	4		19.0	1.49	1.18	4		30.9	1.62	1.37	4	
23.0	1.34	1.38	4		22.0	1.40	1.29	4		35.5	1.80	1.39	4	
26.0	1.43	1.39	4		25.0	1.28	1.31	4						
29.0	1.43	1.39	4		28.0	1.25	1.45	4						
32.0	1.46	1.43	4		31.0	1.43	1.42	4						
35.0	1.58	1.48	4		34.0	1.65	1.40	4		Notes:				
38.0	1.65	1.48	4		37.0	1.71	1.45	4		Q = water discharge (m^3/s)				
41.0	1.80	1.43	4		40.0	1.83	1.44	4		STA = Location along cross-section				
44.0	2.19	1.53	4		43.0	2.09	1.36	4		h = flow depth				
47.0	1.86	0.54	4		43.0	2.10	1.37	4		U = depth-averaged velocity				
					46.0	2.50	1.39	4		n = number observations				
					48.0	2.53	1.20	4		If n=1, reading 0.4h above bed				
					50.0	1.80	0.55	4						
PB1B 4/28/93			Q =	80.3	PB2 4/28/93			Q =	78.9	PB2A 4/28/93			Q =	79.9
STA	h	U	n		STA	h	U	n		STA	h	U	n	
(m)	(m)	(m/s)			(m)	(m)	(m/s)			(m)	(m)	(m/s)		
8.0	1.28	0.52	4		8.0	0.84	0.40	4		5.0	0.58	0.39	1	
11.0	1.65	1.07	4		9.0	0.58	0.22	2		6.0	0.52	0.21	1	
14.0	1.55	1.08	4		10.5	0.43	0.45	1		7.0	0.27	0.04	3	
17.0	1.34	1.21	4		11.0		0			8.5		0		
20.0	1.25	1.35	4		11.5		0			16.0		0		
23.0	1.34	1.37	4		12.0	0.49	0.21	4		17.0	0.72	0.20	4	
26.0	1.46	1.42	4		13.0	0.76	0.88	4		18.0	1.25	0.77	4	
29.0	1.43	1.43	4		16.0	1.48	1.24	4		21.0	1.52	1.33	4	
32.0	1.48	1.49	4		19.0	1.49	1.15	4		24.0	1.77	1.26	4	
35.0	1.58	1.52	4		22.0	1.40	1.29	4		27.0	1.65	1.31	4	
38.0	1.65	1.43	4		25.0	1.25	1.32	4		30.0	1.58	1.30	4	
41.0	1.81	1.46	4		28.0	1.22	1.49	4		33.0	1.65	1.48	4	
44.0	2.23	1.50	4		31.0	1.39	1.38	4		36.0	1.80	1.41	4	
47.0	1.81	0.54	4		34.0	1.65	1.37	4		39.0	1.86	1.50	4	
					37.0	1.65	1.44	4		42.0	1.95	1.47	4	
					40.0	1.83	1.44	4		44.8	2.07	1.50	4	
					43.0	2.07	1.34	4		47.6	2.26	1.10	4	
					46.0	2.53	1.35	4		50.7	2.07	0.92	4	
					49.0	2.23	0.91	4						

Table B.1 Poker Bar Velocity Observations

SB2 5/31/91			Q =	23.4
STA	h	U	n	
(m)	(m)	(m/s)		
13.0	0.18	0.09	1	
14.0	0.30	0.60	2	
15.0	1.22	1.23	2	
16.0	1.28	1.54	2	
17.0	1.28	1.72	2	
18.0	1.31	1.72	2	
19.0	1.31	1.96	2	
20.0	1.40	1.96	2	
21.0	1.46	1.99	2	
22.0	1.40	1.67	2	
23.0	1.25	1.40	2	
24.0	1.19	1.05	2	
25.0	1.16	0.92	2	
26.0	1.13	0.51	2	
27.0	0.38	0.12	1	

SB3C 5/31/91			Q =	23.4
STA	h	U	n	
(m)	(m)	(m/s)		
4.0	0.88	0.57	8	
6.0	1.01	1.30	8	
8.0	0.94	1.43	8	
10.0	0.99	1.49	8	
12.0	0.98	1.47	8	
13.0	1.01	1.42	8	
15.0	0.99	1.38	8	
16.0	0.91	1.35	8	
18.0	0.91	1.19	8	
20.0	0.88	1.20	8	
22.0	0.76	0.85	7	
24.0	0.79	0.23	7	

Notes:
Q = water discharge (m^3/s)
STA = Location along cross-section
h = flow depth
U = depth-averaged velocity
n = number observations

SB2 6/1/91			Q =	23.6
STA	h	U	n	
(m)	(m)	(m/s)		
13.0	0.18	0.16	1	
14.0	0.34	0.66	2	
15.0	1.19	1.25	2	
16.0	1.30	1.52	2	
17.0	1.30	1.73	2	
17.5	1.31	1.78	2	
18.0	1.30	1.68	2	
18.5	1.31	1.91	2	
19.0	1.30	1.97	2	
19.5	1.37	2.00	2	
20.0	1.40	2.01	2	
20.5	1.40	2.07	2	
21.0	1.43	1.91	2	
21.5	1.48	1.80	2	
22.0	1.43	1.57	2	
22.5	1.33	1.52	2	
23.0	1.28	1.34	2	
24.0	1.20	1.09	2	
25.0	1.16	0.94	2	
26.0	1.14	0.54	2	
27.0	0.40	0.08	1	

SB3C 6/1/91			Q =	23.0
STA	h	U	n	
(m)	(m)	(m/s)		
4.0	0.87	0.60	8	
6.0	0.94	1.23	8	
8.0	0.91	1.43	8	
10.0	0.94	1.45	8	
12.0	0.94	1.51	8	
13.0	0.91	1.49	8	
15.0	0.94	1.43	8	
16.0	0.98	1.35	8	
18.0	0.93	1.17	8	
20.0	0.88	1.19	8	
22.0	0.75	0.88	7	
23.8	0.79	0.34	7	

SB3D 6/1/91			
STA	h	U	n
(m)	(m)	(m/s)	
7.0	0.99	1.29	6
9.0	0.94	1.34	6
11.0	0.98	1.31	6
12.0	0.98	1.26	6
14.0	0.94	1.32	6

Table B.2 Steelbridge Velocity Observations

SB2 6/2/91			Q =	12.9	SB3C 6/2/91			Q =	12.1	SB3D 6/2/91				
STA	h	U	n		STA	h	U	n		STA	h	U	n	
(m)	(m)	(m/s)			(m)	(m)	(m/s)			(m)	(m)	(m/s)		
14.5	0.67	0.70	2		4.0	0.56	0.60	7		7.0	0.65	1.10	6	
15.0	0.87	1.00	2		6.0	0.66	1.13	7		9.0	0.58	1.03	6	
15.5	0.96	1.03	2		8.0	0.58	1.27	7		11.0	0.59	1.08	6	
16.0	0.96	1.13	2		10.0	0.62	1.17	7		12.0	0.63	0.83	7	
16.5	0.96	1.26	2		12.0	0.62	1.14	7		14.0	0.59	1.07	6	
17.0	0.98	1.28	2		13.0	0.56	1.13	6						
17.5	0.94	1.32	2		15.0	0.58	1.12	7						
18.0	0.96	1.32	2		16.0	0.58	1.04	7						
18.5	0.99	1.40	2		18.0	0.59	0.99	7						
19.0	0.98	1.48	2		20.0	0.55	0.89	7						
19.5	1.01	1.39	2		22.0	0.46	0.81	7						
20.0	1.04	1.39	2		23.8	0.47	0.67	7						
20.5	1.07	1.48	2											
21.0	1.07	1.53	2											
21.5	1.13	1.33	2											
22.0	1.13	1.10	2							Notes:				
22.5	1.01	0.98	2							Q = water discharge (m^3/s)				
23.0	0.96	0.97	2							STA = Location along cross-section				
23.5	0.94	0.86	2							h = flow depth				
24.0	0.94	0.76	2							U = depth-averaged velocity				
24.5	0.84	0.75	2							n = number observations				
25.0	0.79	0.66	2											
25.5	0.73	0.60	2											
26.0	0.79	0.47	1											

Table B.2 Steelbridge Velocity Observations

SB3C 6/11/92

STA (m)	h (m)	U (m/s)	n
7.0	1.19	1.69	2
11.0	1.22	1.76	2
13.0	1.20	1.82	2
15.0	1.13	1.75	2
19.0	1.07	1.54	2

Notes:
Q = water discharge (m^3/s)
STA = Location along cross-section
h = flow depth
U = depth-averaged velocity
n = number observations

SB3C 6/14/92 Q = 54.0

STA (m)	h (m)	U (m/s)	n
5.0	1.74	0.66	4
7.0	1.77	1.33	4
9.0	1.83	1.73	4
11.0	1.86	1.87	4
13.0	1.83	1.97	4
15.0	1.80	2.01	4
17.0	1.77	1.73	4
19.0	1.74	1.51	4
21.0	1.68	1.28	4
23.0	1.62	0.75	4

SB3D 6/14/92 Q = 53.5

STA (m)	h (m)	U (m/s)	n
5.0	1.83	0.95	4
7.0	1.77	1.35	4
9.0	1.80	1.63	4
11.0	1.83	1.76	4
13.0	1.83	1.88	4
15.0	1.83	1.91	4
17.0	1.80	1.78	4
19.0	1.75	1.38	4
21.0	1.68	1.19	4
22.4	1.51	0.97	4

SB 3B 6/16/92 Q = 53.4

STA (m)	h (m)	U (m/s)	n
2.0	0.61	0.58	2
4.0	0.79	0.36	2
6.0	0.79	0.32	1
7.5	1.10	0.40	2
8.0	1.68	0.82	3
10.0	1.89	1.46	3
12.0	1.95	1.65	3
14.0	1.98	1.83	3
16.0	1.95	2.01	3
18.0	1.95	1.93	3
20.0	1.83	1.78	3
22.0	1.83	1.50	3
24.0	1.74	0.85	2
26.0	1.22	0.09	1

SB3C 6/16/92 Q = 55.2

STA (m)	h (m)	U (m/s)	n
4.0	1.69	0.42	4
6.0	1.72	1.17	4
8.0	1.86	1.51	4
10.0	1.92	1.65	4
12.0	1.89	1.84	4
14.0	1.83	1.99	4
16.0	1.89	1.86	4
18.0	1.83	1.70	4
20.0	1.77	1.43	4
22.0	1.68	1.16	4
23.0	1.62	0.76	4

SB3D 6/16/92 Q = 54.3

STA (m)	h (m)	U (m/s)	n
4.0	1.89	0.67	4
6.0	1.86	1.13	4
8.0	1.80	1.58	4
10.0	1.80	1.76	4
12.0	1.77	1.92	4
14.0	1.83	1.98	4
16.0	1.80	1.82	4
18.0	1.80	1.75	4
20.0	1.77	1.38	4
21.8	1.65	0.83	4

SB3C 6/17/92 Q = 35.4

STA (m)	h (m)	U (m/s)	n
4.0	1.23	0.35	4
6.0	1.30	1.30	4
8.0	1.34	1.65	4
10.0	1.34	1.57	4
12.0	1.37	1.63	4
14.0	1.31	1.74	4
16.0	1.28	1.62	4
18.0	1.25	1.36	4
20.0	1.26	1.30	4
22.0	1.14	0.92	4
23.8	1.14	0.37	4

SB3D 6/17/92 Q = 36.9

STA (m)	h (m)	U (m/s)	n
5.5	1.40	1.31	3
8.5	1.34	1.59	4
12.0	1.37	1.75	4
15.0	1.23	1.62	4
18.0	1.28	1.51	4
21.0	1.19	1.05	4

Table B.2 Steelbridge Velocity Observations

Table 2.3-1. Previous estimates of flushing flows for the Trinity River

Source	Magnitude	Duration	Stated Objective	Basis
Frederickson, Kamine and Associates (1980)	800-1,200 cfs	not stated	remove sand from gravel riffles	Gessler equation applied to a range of assumed riffle widths
Strand (1981)	6,000 cfs Or 10,000 cfs	36 days* 22 days*	90% removal of total sand deposits	Velocity Xi equation applied to hydraulic model extrapolated from values measured at 300 cfs
Nelson et al. (1987)	4,000 cfs	29-45 days	90% removal of deposits < 4mm	previous studies

*Computed from total volumes of water to be released, assuming constant discharge.

Date	(cfs)	(cms)	Velocity Obs. at Study Sites
28-May-91	1,230	35	*
29-May-91	2,600	74	*
30-May-91	2,790	79	*
31-May-91	2,770	78	*
1-Jun-91	2,690	76	*
2-Jun-91	1,540	44	*
10-Jun-92	1,910	54	
11-Jun-92	3,670	104	*
12-Jun-92	6,050	171	
13-Jun-92	6,450	183	*
14-Jun-92	6,420	182	*
15-Jun-92	6,370	180	*
16-Jun-92	6,250	177	*
17-Jun-92	4,620	131	*
18-Jun-92	2,390	68	
19-Jun-92	1,300	37	
13-Apr-93	2,280	65	
14-Apr-93	3,040	86	
15-Apr-93	3,060	87	
16-Apr-93	3,070	87	
17-Apr-93	3,060	87	
18-Apr-93	3,040	86	
19-Apr-93	3,040	86	
20-Apr-93	3,020	86	
21-Apr-93	3,010	85	
22-Apr-93	2,980	84	
23-Apr-93	2,980	84	
24-Apr-93	2,980	84	
25-Apr-93	2,990	85	
26-Apr-93	2,990	85	
27-Apr-93	2,990	85	*
28-Apr-93	2,990	85	*
29-Apr-93	2,980	84	
30-Apr-93	2,350	67	
1-May-93	1,590	45	
2-May-93	1,580	45	
3-May-93	1,580	45	
4-May-93	1,520	43	

Table 4.1.1. Daily mean discharge during trial reservoir releases: Trinity River @ Lewiston

Cross Section	Date	Discharge (m^3/s)	Water Surface Elevation (m)	Number of Stations	Typical No. Readings per Station	Maximum Spacing Between Stations (m)	Typical Spacing Between Stations (m)	Notes
PB0A	5/28/91	35.4	31.059	15	2	3	3	
	5/29/91	68.2	31.516	16	2	3	3	
	5/30/91	75.6	31.595	16	2	3	3	
	6/14/92	164	32.461	14	3	3	3	
PB1B	6/13/92	164	32.413	14	3	3	3	
	6/17/92	117	31.944	11	3	4	4	
	4/27/93	80.0	31.513	12	4	3	3	left bank not accessible
	4/28/93	80.0	31.523	14	4	3	3	
PB2	5/28/91	35.4	31.015	14	11 - 13	3	2	
	5/29/91	68.2	31.469	15	14 - 16	3	2	
	5/30/91	75.6	31.551	14	8, 9	3	3	
	6/11/92	103	31.846	10	1, 3	4	4	
	6/13/92	164	32.428	12	4, 5	3	3	cable
	6/14/92	164	32.427	12	3, 4	3	3	cable
	6/15/92	164	32.423	13	4, 5	3	3	cable
	6/16/92	164	32.453	8	2, 3	2	1, 2	side channel
	6/17/92	117	31.968	9	3	4	4	cable
	4/27/93	80.0	31.535	14	4	3	3	
	4/28/93	80.0	31.534	17	4	3	3	incl. side channel
PB2A	6/15/92	164	32.423	12	4, 5	4	3	cable
	6/16/92	164	32.455	10	2, 3	2	1, 2	side channel
	6/17/92	117	31.955	9	3, 4	4	4	cable
	4/27/93	80.0	31.500	4	4	-	-	partial transect
	4/28/93	80.0	31.517	16	4	3	3	incl. side channel
SB2	5/31/91	23.4/76.0	30.796	15	2	1	1	
	6/1/91	23.4/76.0	30.796	22	2	1	0.5	
	6/2/91	12.5/41.5	30.473	16	2	0.5	0.5	
SB3B	6/16/92	54.3/164	31.538	14	3	2	2	
SB3C	5/31/91	23.4/76.0	30.589	12	7, 8	2	2	
	6/1/91	23.4/76.0	30.583	12	7, 8	2	2	
	6/2/91	12.5/41.5	30.260	12	7	2	2	
	6/11/92	n.a./103	30.898	5	2	4	2	partial transect
	6/14/92	53.8/164	31.491	10	4	2	2	
	6/16/92	54.3/164	31.509	11	4	2	2	
	6/17/92	35.4/117	31.022	11	4	2	2	
SB3D	6/1/91	23.4/76.0	30.580	5	6	2	2	partial transect
	6/2/91	12.5/41.5	30.244	5	6	2	2	partial transect
	6/14/92	53.8/164	31.491	10	4	2	2	
	6/16/92	54.3/164	31.511	10	4	2	2	
	6/17/92	35.4/117	31.034	6	4	3	3	

NOTES
All velocity readings made with Price AA current meters. Meters positioned using top-setting wading rods, except where noted at sections PB2 and PB2A in 1992, when large flow depth and velocity required the use of cable-mounted meters with a 100 lb. sounding weight.

Two discharge values are given for the Steelbridge sections: the first is the discharge through the study reach on the right side of the island, the second is the total discharge through both channels.

Table 4.3.1 Summary of Velocity Observations

Table 4.4.1 Poker Bar Cross Section Surveys

Cross Section	Pre 1991 Release	Post 1991 Release	Pre 1992 Release	Post 1992 Release	Pre 1993 Release	Post 1993 Release
0A	5/21/91	6/5/91	6/5/92	6/25/92	-	-
0B	5/21/91	6/5/91	-	6/25/92	-	-
1	5/21/91	6/5/91	-	6/27/92	-	-
1A	5/21/91	6/5/91	-	6/27/92	-	-
1B	5/22/91	6/5/91	6/5/92	6/27/92	-	5/6/93
2	5/22/91	6/5/91	6/5/92	6/22/92	4/7/93	5/6/93
2A	5/23/91	6/5/91	6/5/92	6/27/92	-	5/6/93
2B	5/23/91	6/5/91	-	6/27/92	-	-
3	5/23/91	6/5/91	-	6/27/92	-	-
3A	5/23/91	6/5/91	-	6/26/92	-	-
4	5/23/91	6/5/91	-	6/26/92	-	-

Table 4.4.2 Steelbridge Right Channel Cross Section Surveys

Cross Section	Pre 1991 Release	Post 1991 Release	Pre 1992 Release	Post 1992 Release
2	5/24/91	6/3/91	6/4/92	6/29/92
2A	5/24/91	6/3/91	6/4/92	6/29/92
2B	5/25/91	6/3/91	6/4/92	6/28/92
2C	5/25/91	6/3/91	-	6/28/92
3	5/25/91	6/3/91	6/4/92	6/28/92
3A	5/25/91	6/3/91	6/4/92	6/28/92
3B	5/26/91	6/3/91	6/4/92	6/28/92
3C	5/25/91	6/3/91	6/4/92	6/20/92
3D	5/25/91	6/3/91	6/4/92	6/28/92
4	5/26/91	6/3/91	6/4/92	6/28/92

Table 4.4.3 Poker Bar Visual Characterization of Bed Surface

Cross Section	Pre 1991 Release	Post 1991 Release	Pre 1992 Release	Post 1992 Release	Pre 1993 Release	Post 1993 Release
0A	5/21/91	6/5/91	6/5/92	6/25/92	4/7/93	5/8/93
0B	5/21/91	6/5/91	6/6/92	6/25/92	4/7/93	5/8/93
1	5/21/91	6/5/91	6/6/92	6/27/92	4/6/93	5/8/93
1A	5/21/91	6/5/91	6/6/92	6/27/92	4/6/93	5/8/93
1B	5/22/91	6/5/91	6/5/92	6/27/92	4/6/93	5/6/93
2	5/22/91	6/5/91	6/5/92	6/22/92	4/7/93	5/6/93
2A	5/23/91	6/5/91	6/5/92	6/27/92	4/7/93	5/6/93
2B	5/23/91	6/5/91	6/6/92	6/27/92	-	-
3	5/23/91	6/5/91	6/6/92	6/27/92	-	-
3A	5/23/91	6/5/91	6/6/92	6/26/92	-	-
4	5/23/91	6/5/91	6/6/92	6/26/92	-	-

Table 4.4.4 Steelbridge Right Channel Visual Characterization of Bed Surface

Cross Section	Pre 1991 Release	Post 1991 Release	Pre 1992 Release	Post 1992 Release
2	5/24/91	6/3/91	6/4/92	6/30/92
2A	5/24/91	6/4/91	6/4/92	6/29/92
2B	5/25/91	6/4/91	6/7/92	6/28/92
2C	5/25/91	6/4/91	6/7/92	6/28/92
3	5/25/91	6/4/91	6/7/92	6/28/92
3A	5/25/91	6/4/91	6/7/92	6/28/92
3B	5/26/91	6/4/91	6/7/92	6/28/92
3C	5/25/91	6/4/91	6/4/92 & 6/7/92	6/20/92
3D	5/25/91	6/4/91	6/7/92	6/28/92
4	5/26/91	6/4/91	6/7/92	6/28/92

Notes:

1. The bed at cross section 3C was visually characterized twice prior to the 1992 release.

Table 4.4.5 Poker Bar Pebble Counts

Cross	1991 Pre-Release			1991 Post-Release			1992 Pre-Release			1992 Post-Release			1993 Pre-Release			1993 Post-Release		
Section	Stations	Number	Date	Stations	Number	Date	Stations	Number	Date	Stations	Number	Date	Stations	Number	Date	Stations	Number	Date
0A	9 to 43	101	5/26/91	9 to 43	100	6/7/91	9 to 18	101	6/5/92	9 to 18	100	6/25/92		-	-	-	-	-
							18 to 43	100	6/5/92	18 to 43	100	6/25/92						
0B	-	-	-	-	-	-	11 to 19	101	6/6/92	16 to 40	100	6/25/92	-	-	-	-	-	-
							19 to 38	100	6/6/92									
1	-	-	-	-	-	-	11 to 39	201	6/6/92	11 to 39	200	6/27/92	11 to 39	100	4/6/93	11 to 39	100	5/8/93
													11 to 39	101	4/6/93	11 to 39	100	5/8/93
1A	-	-	-	-	-	-	22 to 32	100	6/6/92	22 to 32	201	6/27/92	22 to 32	100	4/6/93	22 to 32	100	5/8/93
							22 to 32	99	6/8/92				22 to 32	100	4/6/93	22 to 32	103	5/8/93
1B	16 to 40	100	5/26/91	16 to 40	100	6/7/91	16 to 40	100	6/5/92	16 to 40	203	6/27/92	16 to 40	100	4/6/93	16 to 40	100	5/6/93
							16 to 40	98	6/8/92				16 to 40	100	4/6/93	16 to 40	101	5/6/93
2	15 to 37	100	5/26/91	15 to 37	103	6/7/91	15 to 24	100	6/5/92	15 to 24	100	6/24/92	24 to 32	100	4/7/93	24 to 32	100	5/6/93
							24 to 32	101	6/5/92	24 to 32	100	6/24/92	24 to 32	100	4/7/93	24 to 32	105	5/6/93
							24 to 32	100	6/5/92	24 to 32	100	6/24/92						
							32 to 37	100	6/5/92	32 to 37	100	6/24/92						
2A	21 to 36	106	5/26/91	21 to 36	99	6/7/91	21 to 36	201	6/2/92	21 to 36	199	6/27/92	21 to 36	100	4/7/93	21 to 36	102	5/6/93
													21 to 36	98	4/7/93	21 to 36	100	5/6/93

Table 4.4.6 Steelbridge Pebble Counts

Cross Section	1991 Pre-Release			1991 Post-Release			1992 Pre-Release			1992 Post-Release		
	Stations	Number	Date	Stations	Number	Date	Stations	Number	Date	Stations	Number	Date
2	All	100	5/26/91	All	100	6/8/91	All	92	6/4/92	All	90	6/30/92
2A	-	-	-	-	-	-	LEW to 22	100	6/4/92	-	-	-
2B	-	-	-	-	-	-	13 to 16	102	6/7/92	-	-	-
2C	-	-	-	-	-	-	9 to 14	101	6/7/92	9 to 14	100	6/29/92
3	-	-	-	-	-	-	All All	100 101	6/7/92 6/8/92	All	201	6/29/92
3A	-	-	-	-	-	-	All All	100 100	6/7/92 6/7/92	All	199	6/28/92
3B	All	106	5/26/91	All	106	6/8/91	All All	100 99	6/7/92 6/7/92	All	198	6/28/92
3C	3 to 18	102	5/26/91	3 to 18	102	6/8/91	3 to 18 3 to 18 11 to 16.5	100 100 101	6/4/92 6/8/92 6/8/92	3 to 18 11 to 16.5	200 100	6/28/92 6/28/92
3D	All	99	5/25/91	All	101	6/8/91	All All	100 100	6/1/92 6/1/92	All	200	6/28/92
4	LEW to 17	101	5/26/91	LEW to 17	100	6/8/91	LEW to 17 LEW to 17	100 100	6/1/92 6/1/92	LEW to 17	200	6/28/92

Table 4.4.7 Poker Bar Bulk Sediment Samples

Cross Section	Station (m)	Date Sampled	Pre or Post Release	Diameter (cm)	Mean Depth (cm)	Mass (kg)
1B	23	6/8/91	Post	30	11	17
1B	25	5/27/91	Pre	30	30	24
1B	25	6/7/91	Post	30	19	29
1B	27	6/8/91	Post	30	15	24
1B	29	5/27/91	Pre	30	19	18
1B	29	6/7/91	Post	30	13	20
1B	31	6/8/91	Post	30	9	21
1B	33	5/27/91	Pre	30	25	25
1B	33	6/7/91	Post	30	19	25
2	29	5/26/91	Pre	30	9	14
2	29	6/7/91	Post	30	11	19
2	30	6/8/91	Post	30	9	16
2	32	6/8/91	Post	30	11	20
2	33	5/27/91	Pre	30	14	28
2	33	6/7/91	Post	30	15	25
2	26	6/3/92	Pre	59	39e	271
2	26	6/22/92	Post	59	40	281
2	27	6/23/92	Post	59	32	214
2	28.5	6/3/92	Pre	59	35	238
2	28.5	6/23/92	Post	59	37	251
2	29.25	6/24/92	Post	59	25	135
2	30	6/2/92	Pre	59	27	196
2	30	6/24/92	Post	59	32	190

Notes:

1. The letter "e" after a quantity designates an estimated value.

Table 4.4.8 Steelbridge Bulk Sediment Samples

Cross Section	Station (m)	Date Sampled	Pre or Post Release	Diameter (cm)	Mean Depth (cm)	Mass (kg)
3C	7	5/26/91	Pre	30	3	22
3C	7	6/4/91	Post	30	12	25
3C	9	6/5/91	Post	30	10	20
3C	12	6/5/91	Post	30	24	26
3C	14	5/25/91	Pre	30	5	15
3C	14	6/4/91	Post	30	13	23
3C	16	6/5/91	Post	30	21	26
3C	17	5/25/91	Pre	30	7	13
3C	17	6/4/91	Post	30	13	21
3D	6	6/5/91	Post	30	17	22
3D	8	5/26/91	Pre	30	14	22
3D	8	6/4/91	Post	30	11	15
3D	9	6/5/91	Post	30	19	30
3D	10	5/26/91	Pre	30	13	23
3D	10	6/4/91	Post	30	14	20
3D	12	6/5/91	Post	30	12	22
3D	13	5/26/91	Pre	30	9	14
3D	13	6/4/91	Post	30	11	17
3C	11	5/30/92	Pre	59	25	146
3C	11	6/20/92	Post	59	25	154
3C	12.5	5/31/92	Pre	59	20	112
3C	12.5	6/21/92	Post	59	25	141
3C	13.5	6/21/92	Post	59	26	140
3C	14.5	6/1/92	Pre	59	21e	132
3C	14.5	6/21/92	Post	59	29	186
3C	15.5	6/22/92	Post	59	26	228
3C	16.5	5/31/92	Pre	59	19	120
3C	16.5	6/22/92	Post	59	22	132

Notes:

1. The letter "e" after a quantity designates an estimated value.

Table 4.5.1 Summary of Poker Bar Tracer Gravel Installation

Year	Section	Station	Sample Depth (cm)
1991	1B	25	30
1991	1B	29	19
1991	1B	33	25
1991	2	29	9
1991	2	33	14
1992	2	26	39
1992	2	28.5	35
1992	2	30	27

Table 4.5.2 Summary of Steelbridge Tracer Gravel Installation

Year	Section	Station	Sample Depth (cm)
1991	3C	7	3
1991	3C	14	5
1991	3C	17	7
1991	3D	8	14
1991	3D	10	13
1991	3D	13	9
1992	3C	11	25
1992	3C	12.5	20
1992	3C	14.5	21
1992	3C	16.5	19

Table 4.5.3	Summary of Poker Bar Sediment Transport Observations (XS PB 2)				
Date	Discharge (cms)	Station	Number of Samples	Sample Duration	Method
6/11/92	103	20.1	6	2 min	6" Helley-Smith
6/11/92	103	25.1	5	2 min	6" Helley-Smith
6/11/92	103	31.1	6	2 min	6" Helley-Smith
6/11/92	103	35.1	3	2 min	6" Helley-Smith
6/11/92	103	40.1	3	2 min	6" Helley-Smith
6/11/92	103	44.1	3	2 min	6" Helley-Smith
6/12/92	164	20.1	4	2 min	6" Helley-Smith
6/12/92	164	25.1	2	2 min	6" Helley-Smith
6/12/92	164	30.1	2	2 min	6" Helley-Smith
6/12/92	164	35.1	2	2 min	6" Helley-Smith
6/12/92	164	40.1	2	2 min	6" Helley-Smith
6/12/92	164	43.9	2	2 min	6" Helley-Smith
6/13/92	164	20.6	4	2 min	6" Helley-Smith
6/13/92	164	25.1	11	2 min	6" Helley-Smith
6/13/92	164	30.1	5	2 min	6" Helley-Smith
6/13/92	164	35.1	5	2 min	6" Helley-Smith
6/13/92	164	40.1	5	2 min	6" Helley-Smith
6/13/92	164	45.1	2	2 min	6" Helley-Smith
6/15/92	164	20.1	4	2 min	6" Helley-Smith
6/15/92	164	25.1	4	2 min	6" Helley-Smith
6/15/92	164	30.1	4	2 min	6" Helley-Smith
6/15/92	164	35.1	4	2 min	6" Helley-Smith
6/16/92	164	30.1	20	2 min	6" Helley-Smith
6/16/92	164	35.1	4	2 min	6" Helley-Smith
6/16/92	164	40.1	4	2 min	6" Helley-Smith
6/16/92	164	43.1	4	2 min	6" Helley-Smith
4/29/93	80	16	2	4 min	3" Helley-Smith
4/29/93	80	19	2	4 min	3" Helley-Smith
4/29/93	80	22	2	2 min	3" Helley-Smith
4/29/93	80	25	2	2 min	3" Helley-Smith
4/29/93	80	28	2	2 min	3" Helley-Smith
4/29/93	80	31.5	2	2 min	3" Helley-Smith
4/29/93	80	34	2	2 min	3" Helley-Smith
4/29/93	80	37	2	2 min	3" Helley-Smith
4/29/93	80	40	2	2 min	3" Helley-Smith
4/29/93	80	43	2	2 min	3" Helley-Smith
4/28/93	80	45.8	2	4 min	3" Helley-Smith
4/28/93	80	46.5	2	4 min	3" Helley-Smith
4/28/93	80	47	2	4 min	3" Helley-Smith
4/28/93	80	47.5	2	4 min	3" Helley-Smith
4/30/93	80	18.1	1	68 hr	Sed. Trap
4/30/93	80	23.5	1	68 hr	Sed. Trap
4/30/93	80	30.0	1	68 hr	Sed. Trap
4/30/93	80	33.3	1	68 hr	Sed. Trap

TRINITY RIVER
LIST OF POOL MEASUREMENTS BY FLOW YEAR

Flow Year

Pool	1991		1992		1993		Notes
	Pre	Post	Pre	Post	Pre	Post	
Reo Stott	Yes	Yes	Yes	Yes	No	Yes	Dredged prior to 1992
Society	Yes	Yes	Yes	Yes	Yes	Yes	Dredged prior to 1991
Tom Lang	No	No	Yes	Yes	Part	Yes	Dredged prior to 1992
Upper Steelbridge	No	No	Yes	Yes	Part	Yes	Natural
SP/Ponderosa	No	No	Yes	Yes	Part	Yes	Dredged prior to 1992
Montana	Yes	Yes	No	No	No	No	Partly dredged, abandoned after 1991 due to access problems

Notes:
- See plots for locations of grid points at each pool
- All transects repeated entirely unless noted as partial resurvey
- Tom Lang and SP/Ponderosa added in 1992 after dredging

4.6.1 List of pool measurements by flow year.

(a) Discharge observations during the 1991, 1992, and 1993 releases

Section:	PB0A	PB1B	PB2	PB2A	SB2	SB3B	SB3C	SB3D	Estimated	Nominal	Estimated	Nominal
	Q	Q	Q	Q	Q	Q	Q	Q	Q	Q	Q	Q
Date	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(ft^3/s)	(ft^3/s)
5/28/91	35.7		35.1						35.4	42.5	1250	1500
5/29/91	69.3		67.2						68.2	85.0	2408	3000
5/30/91	77.0		74.3						75.6	85.0	2670	3000
5/31/91					23.4		23.4		76.0	85.0	2684	3000
6/1/91					23.6		23.0		76.0	85.0	2684	3000
6/2/91					12.9		12.1		41.5	42.5	1466	1500
6/11/92			103						103	113	3637	4000
6/12/92									164	170	5792	6000
6/13/92		165	171						164	170	5792	6000
6/14/92	159		164				54.0	53.5	164	170	5792	6000
6/15/92			163	164					164	170	5792	6000
6/16/92						53.4	55.2	54.3	164	170	5792	6000
6/17/92		123	114	117			35.4	36.9	117	113	4132	4000
4/27/93		78.9	82.1						80.0	85.0	2825	3000
4/28/93		80.3	78.9	79.9					80.0	85.0	2825	3000

(b) Statistics for multiple discharge observations during periods of approximately constant stage

	Mean	St. Dev.	Coef.Var.	Min.	Max.	Number	5% of mean
	(m^3/s)	(m^3/s)	%	(m^3/s)	(m^3/s)	Obs	(m^3/s)
1991: SB	23.3	0.3	1.1	23.0	23.6	4	1.2
1992: SB	54.1	0.7	1.3	53.4	55.2	5	2.7
1992: PB	164	3.9	2.4	159	171	6	8.2
1993: PB	80.0	1.3	1.7	78.9	82.1	5	4.0

(c) Percentage of discharge passing through Steelbridge study channel

	SB2	SB3B	SB3C	SB3D	Mean	Q (m^3/s)
5/31/91	30.7		30.9		30.8	76.0
6/1/91	31.0		30.2		30.6	76.0
6/2/91	31.2		29.2		30.2	41.5
6/14/92			32.9	32.6	32.8	164
6/16/92		32.6	33.7	33.1	33.1	164
6/17/92			30.3	31.5	30.9	117

Table 5.1.1 Summary of discharge calculations

Date	Time	WSEL-PB2
5/28/91	13:40	31.01
5/28/91	19:40	31.01
5/29/91	11:20	31.46
5/29/91	15:20	31.49
5/30/91	10:50	31.55
5/30/91	14:50	31.56
6/13/92	12:30	32.43
6/13/92	17:25	32.43
6/14/92	8:13	32.42
6/14/92	15:28	32.44
6/15/92	11:48	32.42
6/15/92	20:07	32.43
6/17/92	17:16	31.97
6/17/92	20:24	31.97
4/27/93	10:00	31.54
4/27/93	20:20	31.52
4/28/93	9:22	31.54
4/28/93	18:30	31.53

Date	Time	WSEL-SB3C
5/31/91	15:20	30.64
5/31/91	18:20	30.63
6/1/91	11:05	30.64
6/1/91	19:00	30.64
6/2/91	14:48	30.32
6/2/91	18:20	30.32
6/14/96	9:20	31.50
6/14/96	17:00	31.50
6/16/96	9:20	31.50
6/16/96	15:00	31.51
6/17/96	15:00	31.03
6/17/96	19:00	31.02

Table 5.1.2 Variation of water surface elevation during velocity observation periods

Table 5.2.1 Summary of Pre and Post-Release Sediment Proportions <8mm

Location and Release	Cross Sections	Pebble Counts		Visual Estimate		% Reduction	
		Pre-Release <8mm	Post-Release <8mm	Pre-Release Embedded	Post-Release Embedded	Pebble Count <8mm	Visual Embedded
1991 Poker Bar	0A, 1B-2A	26%	17%	-	-	9%	-
1991 Steelbridge	2, 3B-4	17%	14%	-	-	3%	-
1992 Poker Bar	0A-2A	28 %	16 %	40 %	25 %	12%	15%
1992 Steelbridge	2-4	22 %	11 %	31 %	23 %	11%	8%
1993 Poker Bar	1-2A	17%	12%	-	-	5%	-
	0A-2A	-	-	18%	19%	-	-1%

Table 5.3.1 1991 Poker Bar Tracer Gravel Observations

Location	XS 1B Sta. 25	XS 1B Sta. 29	XS 1B Sta. 33	XS 2 Sta. 29	XS 2 Sta. 33
Pebble Count D_{50} (mm) Pre/Post	42/38	42/38	42/38	45/45	45/45
Pebble Count D_{90} (mm) Pre/Post	90/90	90/90	90/90	100/100	100/100
Bulk Sample D_{50} (mm) Pre/Post	30/35	33/29	23/19	57/41	40/41
Bulk Sample D_{90} (mm) Pre/Post	80/100	72/98	130/130	105/105	140/110
Largest Grain Moved (mm)	62	95	40	40	None
Maximum Grain Displacement (m)	3	15	0.2	0.2	0
128 mm Tracers installed/in place/ downstream	0/0/0	0/0/0	1/1/0	0/0/0	2/2/0
90 mm Tracers installed/in place/ downstream	1/1/0	0/0/0	0/0/0	2/2/0	3/3/0
64 mm Tracers installed/in place/ downstream	7/7/0	5/4/1	5/5/0	6/6/0	9/9/0
45 mm Tracers installed/in place/ downstream	19/18/1	14/11/3	10/10/0	10/10/0	6/6/0
32 mm Tracers installed/in place/ downstream	38/37/1	34/27/7	27/9/1	19/18/1	30/29/1
22 mm Tracers installed/in place/ downstream	71/71/0	56/48/8	72/68/0	23/18/0	54/43/1
Depth of Scour from Tracers (cm)	0.6	3.8	3.5	0.2	0.3
Gravel Movement	Negligible	Partial	Negligible	Negligible	Negligible

Table 5.3.2 1992 Poker Bar Tracer Gravel Observations

Location	XS 2 Sta. 26	XS 2 Sta. 28.5	XS 2 Sta. 30
Pebble Count D_{50} (mm) Pre/Post	31/27	31/27	31/27
Pebble Count D_{90} (mm) Pre/Post	85/84	85/84	85/84
Bulk Sample D_{50} (mm) Pre/Post	25/21	23/18	22/21
Bulk Sample D_{90} (mm) Pre/Post	90/78	70/64	86/77
Maximum Grain Displacement (m)	>200	>200	>200
128 mm Tracers installed/in place/ downstream	0/0/0	0/0/0	0/0/0
90 mm Tracers installed/in place/ downstream	2/1/0	1/1/0	4/3/1
64 mm Tracers installed/in place/ downstream	5/2/1	7/6/0	7/6/1
45 mm Tracers installed/in place/ downstream	18/13/1	29/18/1	22/18/3
32 mm Tracers installed/in place/ downstream	48/36/4	65/52/1	65/47/6
22 mm Tracers installed/in place/ downstream	-/41/4	126/70/4	171/92/11
16 mm Tracers installed/in place/ downstream	195/106/7	264/176/2	396/166/23
Depth of Scour from Tracers (cm)	12.9	11.9	10.5
Gravel Movement	General	General	General

Table 5.3.3 Poker Bar Gravel Bedload Rates From Sediment Traps

Date	Discharge (cms)	Station	Number of Samples	Sample Duration	Transport Rate (g/ms)
4/27/93	80	18.1	1	336 hr	0.0101
4/27/93	80	23.5	1	336 hr	0.0112
4/27/93	80	27	1	336 hr	0.0112
4/27/93	80	30.0	1	336 hr	0.0123
4/27/93	80	33.3	1	336 hr	0.0110
4/30/93	80	18.1	1	68 hr	0.0082
4/30/93	80	23.5	1	68 hr	0.0054
4/30/93	80	27	1	44 hr	0.0104
4/30/93	80	30.0	1	68 hr	0.0161
4/30/93	80	33.3	1	68 hr	0.0084

Traps cleaned on 4/27/93;

First sample duration is from start of release at Q = 80 cms to 4/27;

Second sample is from 4/27 to end of release at Q = 80 cms

Trap at Sta. 27 was cleaned a second time on 4/27

Table 5.3.4 Poker Bar Gravel Bedload Rates From Helley-Smith Sampler

Date	Discharge (cms)	Station	Total Number of Samples	Total Sample Duration	Transport Rate (g/ms)	Number of Censored Samples	Censored Transport Rate (g/ms)
6/11/92	103	20.1	6	12 min	63.1	2	32.1
6/11/92	103	25.1	5	10 min	4.0	3	1.8
6/11/92	103	31.1	6	12 min	2.1	2	1.9
				Mean	24.2		3.6
6/12/92	164	20.1	12	24 min	48.7	6	47.9
to	164	25.1	17	34 min	72.4	9	41.1
6/16/92	164	30.1	31	62 min	165.9	15	168.4
				Mean	116.0		106.1

Censored Samples: Calculate mean transport rates after dropping the largest one-quarter and smallest one-quarter of the samples

Gravel taken to be all sediment coarser than 8 mm

Table 5.3.5 1991 Steelbridge Tracer Gravel Observations

Location	XS 3C Sta. 7	XS 3C Sta. 14	XS 3C Sta. 17	XS 3D Sta. 8	XS 3D Sta. 10	XS 3D Sta. 13
Pebble Count D_{50} (mm) Pre/Post	53/45	53/45	53/45	62/47	62/47	62/47
Pebble Count D_{90} (mm) Pre/Post	120/96	120/96	120/96	112/108	112/08	112/108
Bulk Sample D_{50} (mm) Pre/Post	97/102	46/40	80/33	58/39	57/55	68/44
Bulk Sample D_{90} (mm) Pre/Post	162/162	114/138	119/115	90/98	145/145	113/110
Largest Grain Moved (mm)	None	55	66	130	93	125
Maximum Grain Displacement (m)	0	11	8	12	9	16
128 mm Tracers installed/in place/ downstream	1/1/0	0/0/0	0/0/0	0/0/0	1/1/0	0/0/0
90 mm Tracers installed/in place/ downstream	3/3/0	2/2/0	4/4/0	2/1/1	1/0/1	3/2/1
64 mm Tracers installed/in place/ downstream	3/3/0	2/2/0	2/1/1	8/4/3	8/3/5	4/3/1
45 mm Tracers installed/in place/ downstream	9/8/0	5/2/3	1/1/0	17/12/5	9/6/3	8/8/3
32 mm Tracers installed/in place/ downstream	19/19/0	21/17/4	10/9/1	24/18/6	27/20/7	17/7/6
22 mm Tracers installed/in place/ downstream	35/19/0	34/31/3	26/26/0	35/30/2	40/18/9	28/16/2
16 mm Tracers installed/in place/ downstream	-	52/6/0	-	-	-	-
Depth of Scour from Tracers (cm)	0.2	0.8	0.6	5.3	5.5	3.3
Gravel Movement	Negligible	Partial	Negligible	Partial	Partial	Partial

Table 5.3.6 1992 Steelbridge Tracer Gravel Observations

Location	XS 3C Sta. 11	XS 3C Sta. 12.5	XS 3C Sta. 14.5	XS 3C Sta. 16.5
Pebble Count D_{50} (mm) Pre/Post	36/54	36/54	36/54	36/54
Pebble Count D_{90} (mm) Pre/Post	100/119	100/119	100/119	100/119
Bulk Sample D_{50} (mm) Pre/Post	39/33	43/33	43/26	46/35
Bulk Sample D_{90} (mm) Pre/Post	140/140	139/142	140/126	147/125
Maximum Grain Displacement (m)	42	>100	>100	0
128 mm Tracers installed/in place/ downstream	1/1/0	1/1/0	1/1/0	1/1/0
90 mm Tracers installed/in place/ downstream	3/2/1	2/1/1	3/2/1	3/3/0
64 mm Tracers installed/in place/ downstream	3/3/0	5/1/3	3/1/2	3/3/0
45 mm Tracers installed/in place/ downstream	15/5/10	13/6/5	15/5/4	9/9/0
32 mm Tracers installed/in place/ downstream	38/6/21	26/11/9	23/13/4	23/23/0
22 mm Tracers installed/in place/ downstream	62/31/12	35/5/10	41/17/1	52/52/0
16 mm Tracers installed/in place/ downstream	115/63/6	72/20/2	109/63/0	119/-/-
Depth of Scour from Tracers (cm)	11.0	10.4	8.8	0
Gravel Movement	General	General	General	Negligible (buried)

Table 5.4.1	Poker Bar Sand Bedload Rates From Helley-Smith Sampler				
Date	Discharge (cms)	Station	Number of Samples	Sample Duration	Transport Rate (g/ms)
6/11/92	103	20.1	6	12 min	21.8
6/11/92	103	25.1	5	10 min	45.2
6/11/92	103	31.1	6	12 min	75.4
6/11/92	103	35.1	3	6 min	35.1
6/11/92	103	40.1	3	6 min	29.2
6/11/92	103	44.1	3	6 min	31.8
		Total Sand Discharge PB 2 at Q = 103 cms: 112,400 kg/day			
6/12/92	164	20.1	4	8 min	11.7
6/12/92	164	25.1	2	4 min	42.5
6/12/92	164	30.1	2	4 min	1.5
6/12/92	164	35.1	2	4 min	9.1
6/12/92	164	40.1	2	4 min	28.1
6/12/92	164	43.9	2	4 min	4.6
6/13/92	164	20.6	4	8 min	26.8
6/13/92	164	25.1	11	22 min	127.2
6/13/92	164	30.1	5	10 min	322.1
6/13/92	164	35.1	5	10 min	457.2
6/13/92	164	40.1	5	10 min	131.4
6/13/92	164	45.1	2	4 min	22.5
6/15/92	164	20.1	4	8 min	9.1
6/15/92	164	25.1	4	8 min	12.9
6/15/92	164	30.1	4	8 min	45.8
6/15/92	164	35.1	4	8 min	181.2
6/16/92	164	30.1	20	40 min	66.0
6/16/92	164	35.1	4	8 min	187.2
6/16/92	164	40.1	4	8 min	50.7
6/16/92	164	43.1	4	8 min	18.4
		Total Sand Discharge PB 2 at Q = 164 cms: 223,600 kg/day			
4/29/93	80	16	2	8 min	16.0
4/29/93	80	19	2	8 min	2.6
4/29/93	80	22	2	4 min	24.8
4/29/93	80	25	2	4 min	7.2
4/29/93	80	28	2	4 min	11.0
4/29/93	80	31.5	2	4 min	24.9
4/29/93	80	34	2	4 min	6.5
4/29/93	80	37	2	4 min	3.9
4/29/93	80	40	2	4 min	3.9
4/29/93	80	43	2	4 min	19.1
4/28/93	80	45.8	2	8 min	6.5
4/28/93	80	46.5	2	8 min	3.9
4/28/93	80	47	2	8 min	18.3
4/28/93	80	47.5	2	8 min	13.9
		Total Sand Discharge PB 2 at Q = 80 cms: 34,403 kg/day			

TRINITY RIVER

Summary of Pool Volume Changes by Year

Flow Year

Pool	1991			1992			1993		
	Cut	Fill	Net	Cut	Fill	Net	Cut	Fill	Net
Reo Stott	308	179	-129	395	881	487	506	91	-414
Society	1096	936	-160	2311	437	-1874	504	426	-77
Tom Lang	-na-	-na-	-na-	1011	1896	885	1291	254	-1038
Upper Steelbridge	-na-	-na-	-na-	340	174	-167	593	42	-551
SP/Ponderosa	-na-	-na-	-na-	1578	1062	-516	1201	106	-1095
Montana	-na-	-na-	-na-	-na-	-na-	-na-	-na-	-na-	-na-

Notes:
- All volumes expressed in cubic yards
- Volumes computed by earthworks module of Softdesk AdCADD software using pre- and post release digital terrain models from field surveys of depth

5.5.1. Summary of pool volume changes by year.

TRINITY RIVER

Comparison of Pre-1992 and Post-1993 Visual Estimates of Surficial Sediment

WEIGHTED PERCENT SURFICIAL SEDIMENT BY REACH

Year	Grass Valley to SP-Ponderosa	SP-Ponderosa to Tom Lang	Tom Lang to Stott	Stott to Society	Society to Island	Island to Upper Steel Bridge
1992	31.6 %	13.6	43.5	25.1	28.9	32.8
1993	27.2 %	13.4	27.6	18.9	16.8	25.9

Notes:
1. See text for description of methods for visual estimation and computation of results
2. 1992 measurement prior to 6000 cfs release
3. 1993 measurement post 3000 cfs release

5.6.1 Comparison of pre-1992 and post-1993 visual estimates of surficial sediment.

TRINITY RIVER

Estimated Sediment Budget pre-1992 through post-1993

VOLUME ESTIMATES AND NET VOLUME CHANGE BY REACH/POOL (cubic yards)

	Grass Valley to SP-Pondo	SP-Pondo Pool	SP-Pondo to Tom Lang	Tom Lang Pool	Tom Lang to Stott	Reo Stott Pool	Stott to Society	Society Pool	Society to Island	Island to Upper SB	Upper Steelbridge Pool	Total Change
Pre-92	578		335		997		514		1973	1285		
Post-93	499		331		632		388		1145	1013		
Net Change	-79	-1739	-4	-160	-365	88	-126	-2100	-828	-272	-718	-6303

Notes:
1. See text for description of methods for visual estimation and computation of results
2. All values in cubic yards; negative values are net cut, while postive values are net fill
3. Surface Volume is computed as: (reach surficial sediment percent) x (surface thickness of 0.25 feet) x (area)
4. Pool volumes computed by earthworks module of Softdesk AdCADD software using pre-92 and post-93 release digital terrain models from field surveys of depth

5.6.2. Estimated sediment budget from prior to 1992 release to after 1993 release.

1991 Water Surface Elevations (m) ------>

Cross Section	River Station (m)	May 28 Discharge: 41 cms (1460 cfs) Surveyed	HEC-2	Error	May 29 Discharge: 76 cms (2680 cfs) Surveyed	HEC-2	Error	May 30 Discharge: 76 cms (2680 cfs) Surveyed	HEC-2	Error
0A	0.00	31.06	31.10	0.04	31.52	31.56	0.04	31.60	31.65	0.05
0B	14.50		31.08		31.52	31.54	0.02	31.60	31.63	0.03
1	30.00		31.06		31.51	31.51	0.00	31.60	31.60	0.00
1A	45.65		31.04		31.50	31.49	-0.01	31.59	31.58	-0.01
1B	62.95		31.02		31.46	31.47	0.01	31.54	31.55	0.01
2	81.40	31.02	31.00	-0.02	31.47	31.44	-0.03	31.55	31.53	-0.02
2A	98.35		30.99		31.43	31.42	-0.01	31.51	31.50	-0.01
2B	116.45		30.98		31.43	31.41	-0.02	31.51	31.50	-0.01
3	135.75	30.97	30.96	-0.01	31.39	31.38	-0.01	31.47	31.46	-0.01
3A	174.05		30.95		31.38	31.35	-0.03	31.46	31.43	-0.03
4	209.45		30.94		31.34	31.34	0.00	31.41	31.41	0.00

1992 Water Surface Elevations (m) ------>

Cross Section	River Station (m)	June 11 Discharge: 103 cms (3640 cfs) Surveyed	HEC-2	Error	June 12-16 Discharge: 164 cms (5790 cfs) Surveyed	HEC-2	Error	June 17 Discharge: 115 cms (4060 cfs) Surveyed	HEC-2	Error
0A	0.00	31.87	31.88	0.01	32.47	32.49	0.02	32.00	32.01	0.01
0B	14.50	31.87	31.87	0.00	32.48	32.48	0.00	32.00	32.00	0.00
1	30.00	31.87	31.86	-0.01	32.48	32.47	-0.01	32.01	31.99	-0.02
1A	45.65	31.86	31.85	-0.01	32.47	32.45	-0.02	31.99	31.98	-0.01
1B	62.95	31.83	31.83	0.00	32.42	32.44	0.02	31.94	31.96	0.02
2	81.40	31.85	31.83	-0.02	32.44	32.43	-0.01	31.97	31.96	-0.01
2A	98.35	31.82	31.81	-0.01	32.44	32.42	-0.02	31.95	31.94	-0.01
2B	116.45	31.81	31.80	-0.01	32.42	32.41	-0.01	31.94	31.94	0.00
3	135.75	31.77	31.74	-0.03	32.33	32.33	0.00	31.87	31.87	0.00
3A	174.05	31.74	31.73	-0.01	32.33	32.32	-0.01	31.86	31.86	0.00
4	209.45	31.70	31.70	0.00	32.29	32.28	-0.01	31.83	31.83	0.00

1993 Water Surface Elevations (m) ------>

Cross Section	River Station (m)	May 24-25 Discharge: 80 cms (2820 cfs) Surveyed	HEC-2	Error
0A	0.00	31.60	31.59	-0.01
0B	14.50	31.57	31.58	0.01
1	30.00	31.58	31.57	-0.01
1A	45.65	31.56	31.56	0.00
1B	62.95	31.52	31.54	0.02
2	81.40	31.53	31.54	0.01
2A	98.35	31.51	31.52	0.01
2B	116.45	31.51	31.51	0.00
3	135.75	31.46	31.46	0.00
3A	174.05	31.45	31.45	0.00
4	209.45	31.42	31.42	0.00

Table 6.2.1 HEC-2 Calibration Results; Poker Bar

Cross Section	Reach Station (m)	Water Surface Elevations at Discharge of: 28.32 cms (1000 cfs)	56.64 cms (2000 cfs)	84.96 cms (3000 cfs)	113.28 cms (4000 cfs)	141.60 cms (5000 cfs)	169.92 cms (6000 cfs)	198.24 cms (7000 cfs)	226.56 cms (8000 cfs)	240.72 cms (8500 cfs)
0A	0.0	30.74	31.15	31.56	31.93	32.24	32.54	32.82	33.07	33.19
0B	14.5	30.73	31.15	31.56	31.92	32.24	32.54	32.81	33.07	33.19
1	30.0	30.72	31.13	31.55	31.91	32.22	32.53	32.80	33.06	33.18
1A	45.7	30.69	31.11	31.53	31.89	32.21	32.51	32.79	33.04	33.16
1B	63.0	30.66	31.09	31.51	31.88	32.19	32.50	32.77	33.02	33.14
2	81.4	30.65	31.08	31.50	31.87	32.19	32.49	32.77	33.03	33.15
2A	98.4	30.63	31.05	31.48	31.85	32.17	32.48	32.76	33.02	33.14
2B	116.5	30.61	31.04	31.47	31.84	32.17	32.47	32.76	33.01	33.13
3	135.8	30.59	30.99	31.41	31.77	32.08	32.38	32.66	32.90	33.02
3A	174.1	30.58	30.97	31.40	31.76	32.08	32.38	32.66	32.91	33.03
4	209.5	30.55	30.93	31.36	31.72	32.03	32.34	32.61	32.87	32.98

Table 6.2.2 Discharge ratings: Trinity River at Poker Bar.

Cross Section	River Station (m)	1991 Water Surface Elevations -------> May 31 & June 1, Discharge: 76 cms (2680 cfs)			June 2, Discharge: 41 cms (1460 cfs)		
		Surveyed	HEC-2	Error	Surveyed	HEC-2	Error
2	0.0	30.80	30.79	-0.01	30.47	30.47	0.00
2A	19.3	30.72	30.74	0.02	30.39	30.43	0.04
2B	38.1	30.71	30.73	0.02	30.39	30.43	0.04
2C	56.3	30.71	30.72	0.01	30.38	30.42	0.04
3	75.0	30.67	30.69	0.02	30.35	30.40	0.05
3A	89.9		30.67			30.38	
3B	106.5	30.61	30.58	-0.03	30.30	30.31	0.01
3C	122.4	30.59	30.57	-0.02	30.26	30.26	0.00
3D	134.0	30.59	30.55	-0.04	30.25	30.24	-0.01
4	150.0		30.51			30.18	

Cross Section	River Station (m)	1992 Water Surface Elevations -------> June 11, Discharge: 103 cms (3640 cfs)			June 12 -16, Discharge: 164 cms (5790 cfs)			June 17, Discharge: 115 cms (4060 cfs)		
		Surveyed	HEC-2	Error	Surveyed	HEC-2	Error	Surveyed	HEC-2	Error
2	0.0	31.02	31.03	0.01	31.61	31.59	-0.02	31.13	31.15	0.02
2A	19.3	30.95	30.97	0.02	31.60	31.54	-0.06	31.10	31.10	0.00
2B	38.1	30.93	30.98	0.05	31.56	31.55	-0.01	31.07	31.11	0.04
2C	56.3	30.93	30.97	0.04	31.50	31.55	0.05	31.06	31.09	0.03
3	75.0	30.89	30.95	0.06	31.48	31.54	0.06	31.01	31.08	0.07
3A	89.9	30.84	30.93	0.09	31.43	31.53	0.10	30.96	31.06	0.10
3B	106.5	30.89	30.89	0.00	31.53	31.51	-0.02	31.04	31.03	-0.01
3C	122.4	30.90	30.89	-0.01	31.50	31.52	0.02	31.02	31.03	0.01
3D	134.0	30.91	30.88	-0.03	31.50	31.52	0.02	31.03	31.03	0.00
4	150.0		30.86		31.38	31.50	0.12	30.89	31.00	0.11

Table 6.2.3 HEC-2 Calibration Results; Steelbridge

Cross Section	River Station (m)	Water Surface Elevation (m) at Discharge: 28.32 cms (1000 cfs)	56.64 cms (2000 cfs)	84.96 cms (3000 cfs)	113.28 cms (4000 cfs)	141.60 cms (5000 cfs)	169.92 cms (6000 cfs)	169.92 cms (6000 cfs)	198.24 cms (7000 cfs)	226.56 cms (8000 cfs)	240.72 cms (8500 cfs)
							<-- Split Flow	Island Inundated -->			
2	0.0	30.31	30.63	30.88	31.13	31.37	31.64	31.70	31.92	32.13	32.23
2A	19.3	30.27	30.58	30.82	31.08	31.32	31.59				
2B	38.1	30.27	30.58	30.83	31.09	31.33	31.60				
2C	56.3	30.27	30.58	30.82	31.09	31.33	31.60				
3	75.0	30.26	30.55	30.80	31.06	31.32	31.59				
3A	89.9	30.24	30.53	30.78	31.05	31.30	31.58				
3B	106.5	30.19	30.48	30.74	31.02	31.28	31.57	31.58	31.80	32.01	32.11
3C	122.4	30.13	30.46	30.73	31.02	31.29	31.57				
3D	134.0	30.07	30.44	30.71	31.01	31.29	31.57				
4	150.0	29.95	30.34	30.60	30.99	31.27	31.56	31.50	31.72	31.93	32.03

Table 6.2.4 Discharge ratings: Trinity River at Steelbridge

Pool Name	Result of 1991 Release (3000 cfs for 4 days)	Result of 1992 Release (6000 cfs for 5 days)	Result of 1993 Release (3000 cfs for 16 days)	Notes
SP	-	Pool bottom shows slight scour from 8-9' depth to 8-10' depth.	Slight deposition, little change in pool bottom elev. at 8-10 ft.	1992: Stable depth by scour 8-11 ft.; 1993: Stable depth 6-9 ft. Complete extent of sand trapping or deposition unknown; upstream part of SP pool not accessible
Ponderosa	-	**Dredged** to 11-12 ft depth prior to 1992 release. 321 yd^3 fill on upstream end of pool; little change in pool bottom elevation	Little change in pool depth at upstream end, most of pool inaccessible.	1992: Stable depth 10-13 ft. 1993: No estimate of stable depth Complete extent of sand trapping or deposition unknown; downstream part of Ponderosa Pool not accessible
Tom Lang	-	**Dredged** to 8-13 ft depth. Fills to 7-9 ft depth, fill volume = 1273 yd^3	Little change during 1993 flush; pool depth remains at ≈7-10 ft.	1992: Stable depth 7-9 ft 1993: Stable depth 5-7 ft
Stott	-	**Dredged**. Deposition in large eddy on right-bank		Deposition occurs primarily in right-bank eddy. Fill capacity not controlled by stable depth.
Society	**Dredged** to 9-11 ft. Fills to 8-10 ft depth, fill volume = 468 yd^3	Scours to ≈10-12 ft. depth.	Little change in bed elevation.	1991: stable depth: 7-10 ft. 1992: stable depth: 10-13 ft.
Upper Steelbridge	-	Little change in pool depth.; u/s end of pool not surveyed.	Little change in pool depth; u/s end of pool not surveyed.	Stable depth ≤12 ft. Natural pool 11-12 ft. deep. Extent of sand trapping unknown; u/s part of pool not surveyed.

All pool depths referenced to the 300 cfs water surface.

Table 6.4.1 Summary of pool cut and fill during trial releases.

Table 6.5.1 Reach description and initial sand quantity used in sand routing calculations.

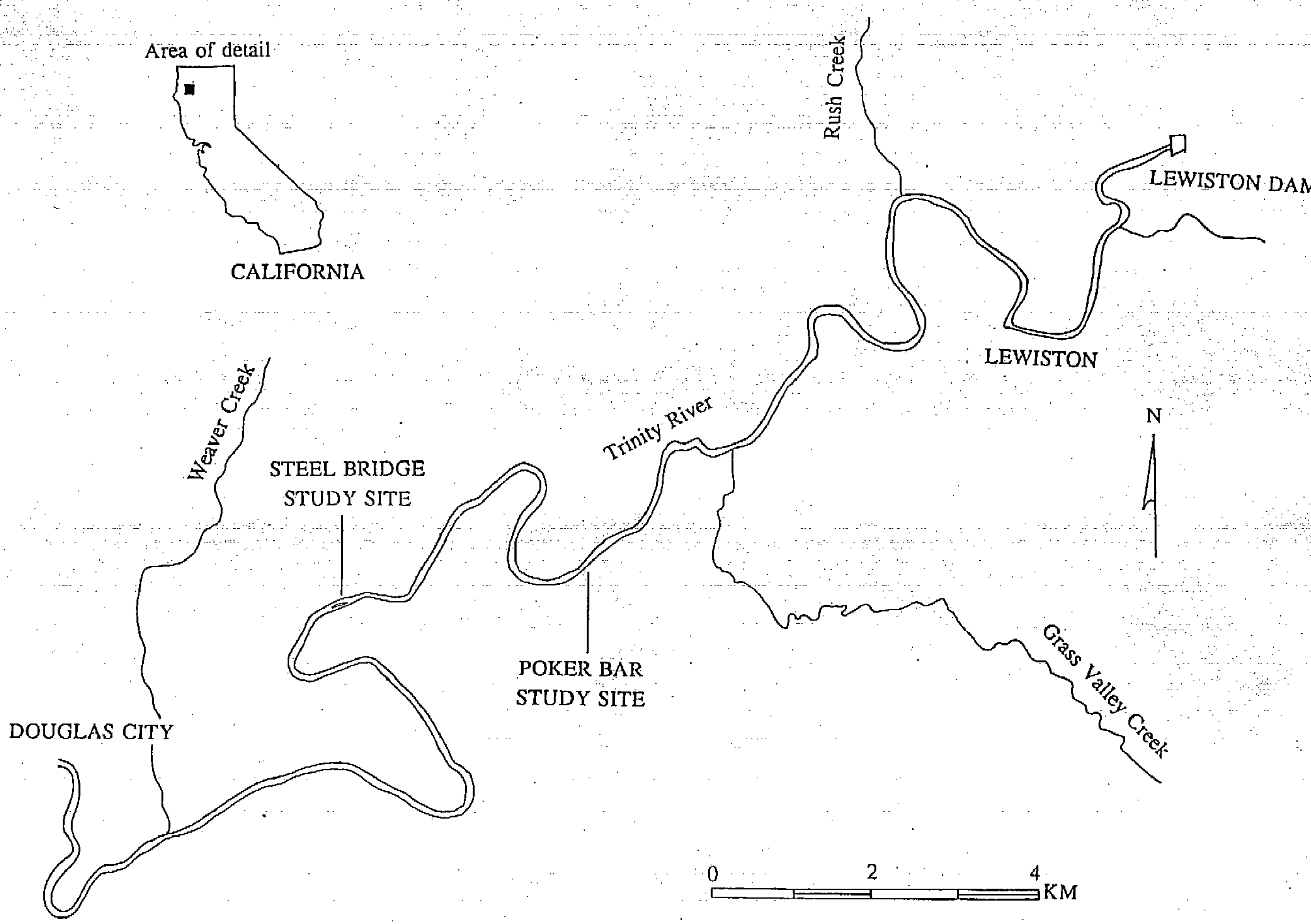
Reach	Reach Area (ft^2)	Reach Length (ft)	Sand Proportion Bed Surface	Sand Proportion Subsurface	Total weight of sand (tons)
Grass Valley Creek to SP/Ponderosa Pool	197,850	2,250	0.272	0.25	1,435
SP/Ponderosa Pool to Lang Pool	266,400	3,100	0.134	0.25	1,421
Lang Pool to Stott Pool	247,350	3,000	0.276	0.25	1,807
Stott Pool to Society Pool	221,400	3,250	0.189	0.25	1,350
Society Pool to Island	736,500	6,750	0.168	0.25	4,275
Island to Upper Steelbridge Pool	422,850	4,250	0.259	0.25	2,988

Period	GVC Suspended Sediment Discharge (tons)	GVC Bedload Discharge (tons)
January-March, 1978	75,500	n/a
January-April, 1983	300,000	34,000
February-March, 1986	59,000	6,600

Table 6.6.1 Major sediment delivery events on Grass Valley Creek

	Annual	Annual	Annual	Monthly	Monthly	Monthly	Monthly	Monthly	Monthly	Monthly	Monthly
All Q in cfs		Mean of 5	Date of	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum
	Maximum	Largest	Maximum	Daily Q	Daily Q	Daily Q	Daily Q	Daily Q	Daily Q	Daily Q	Daily Q
Water Year	Daily Q	Daily Q*	Daily Q	October	November	December	January	February	March	April	May
1976	86	71	4/8/76	-	-	33	20	73	47	86	35
1977	28	21	5/2/77	12	16	11	14	20	17	14	28
1978	857	707	1/16/78	13	23	146	**857**	363	459	145	96
1979	276	168	3/27/79	16	24	19	79	100	276	97	71
1980	519	388	2/18/80	62	65	56	232	**519**	169	110	55
1981	326	150	1/28/81	15	16	116	326	193	102	75	49
1982	557	296	12/19/81	40	239	**557**	89	231	282	170	84
1983	2420	1518	3/2/83	33	76	204	1580	1280	**2420**	360	302
1984	592	437	12/11/83	23	218	**592**	211	97	90	50	45
1985	304	171	11/13/84	20	304	78	37	55	35	40	30
1986	808	664	2/14/86	27	21	62	239	**808**	192	98	64
1987	406	193	3/5/87	21	19	23	38	156	**406**	67	38
1988	208	116	12/6/87	12	23	208	64	40	38	73	49
1989	357	226	3/9/89	11	54	19	29	26	**357**	83	43
1990	171	119	5/27/90	121	19	16	71	25	32	21	171
Maximum	2420	1518		121	304	592	1580	1280	2420	360	302
Mean	528	349		30	80	143	259	266	328	99	77
Median	357	193		21	24	62	79	100	169	83	49
Minimum	28	21		11	16	11	14	20	17	14	28
* 5 consecutive days including annual maximum					Annual maximum daily Q shown in bold for values greater than median (Q>357 cfs)						

Table 6.6.2 Record of large discharges on Grass Valley Creek, 1976-1990.



2.2.1. Location map, Trinity River basin and study area.

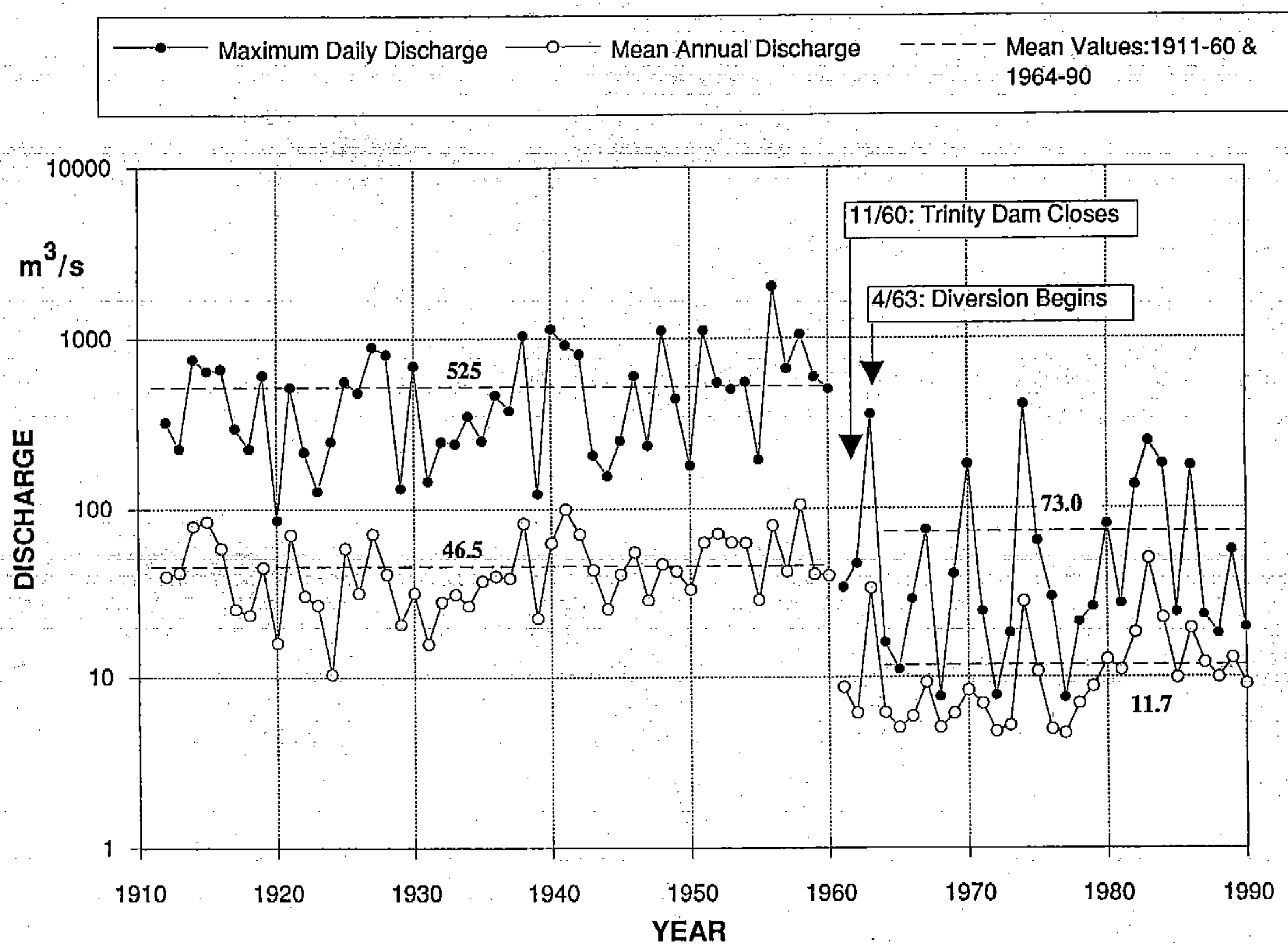


Figure 2.2.2. Mean annual discharge and annual maximum daily discharge, Trinity River at Lewiston, 1911-1990

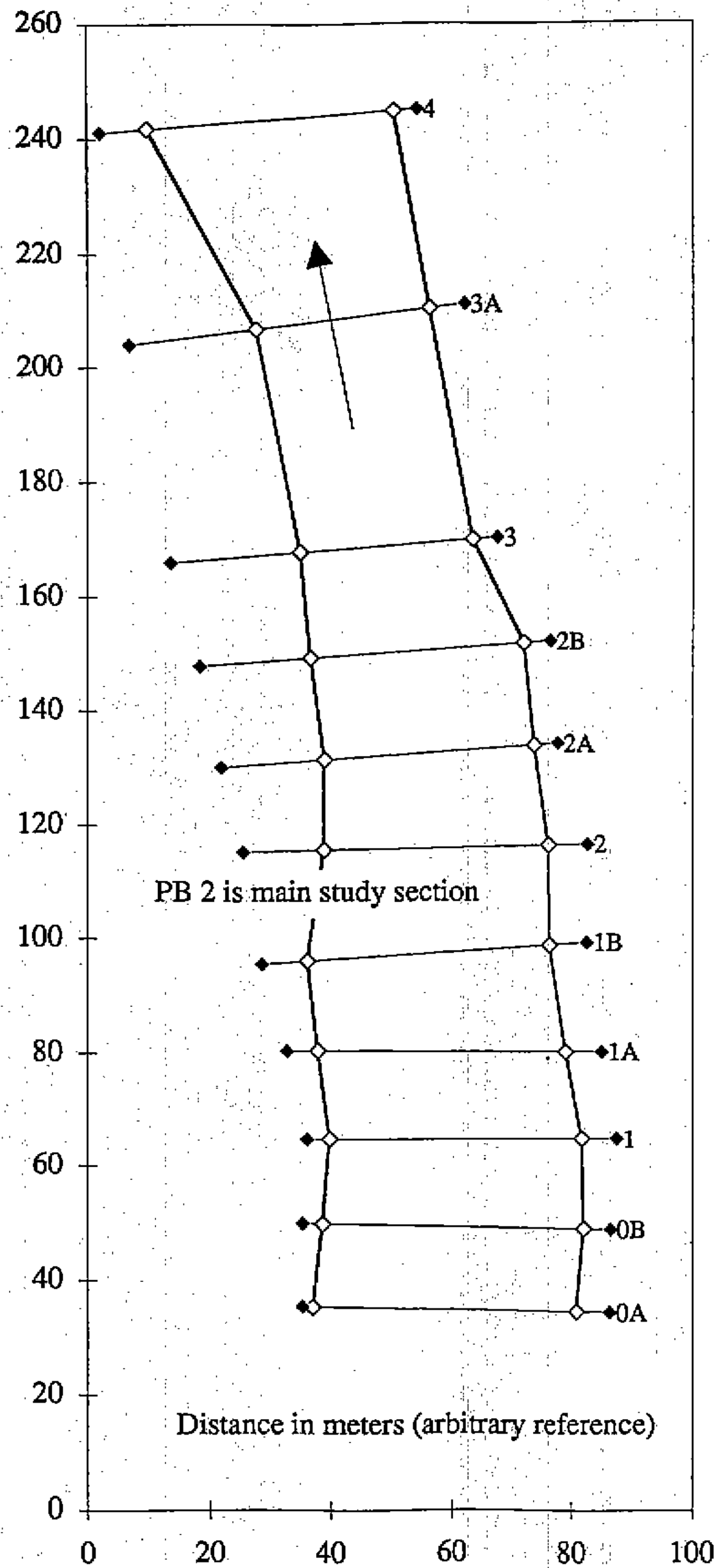


Figure 3.2.1 River bank and cross section locations for Poker Bar Study Reach

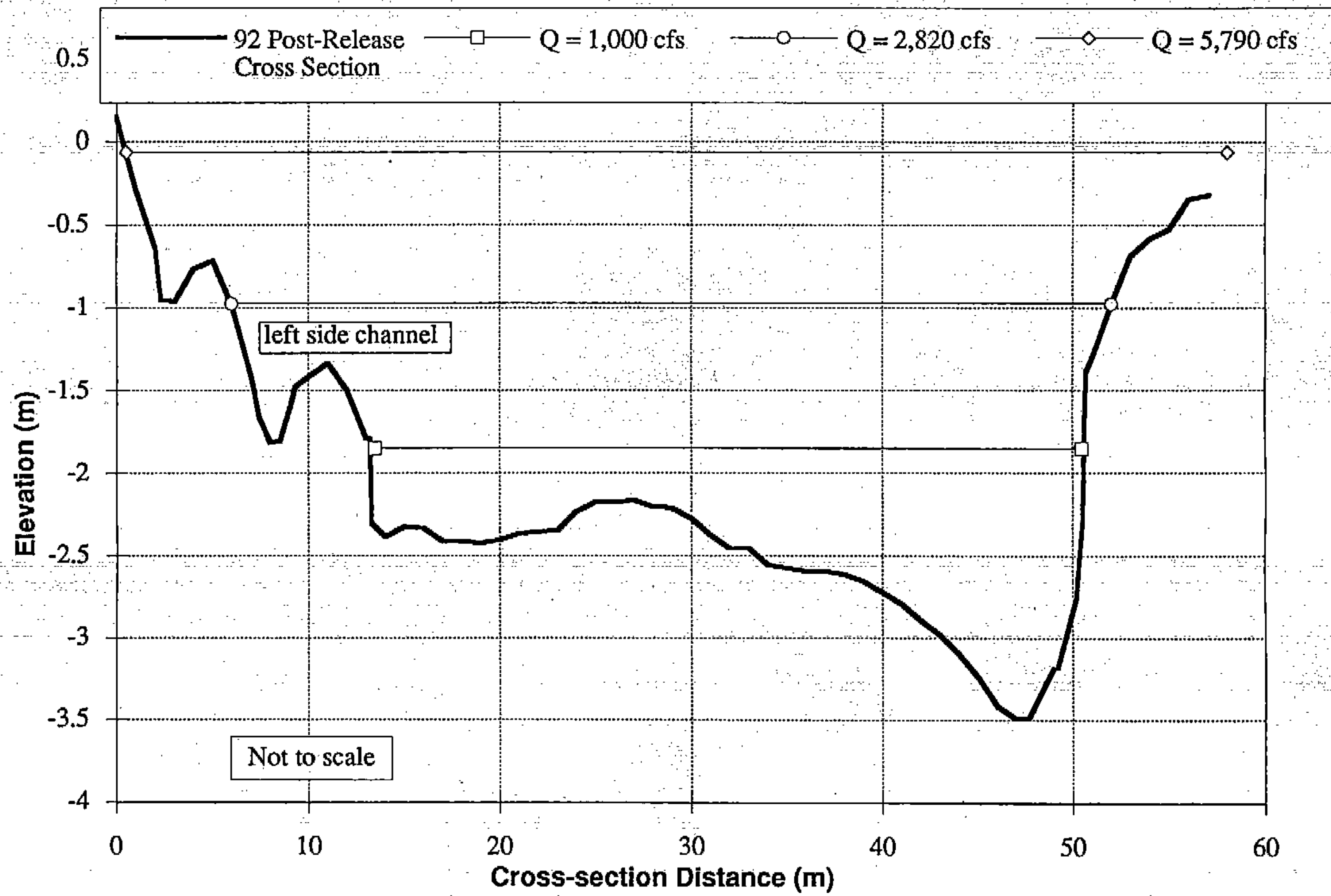


Figure 3.2.2. Cross section elevation of primary study section at Poker Bar, PB 2, looking downstream. Water surface elevation at three different discharges also shown.

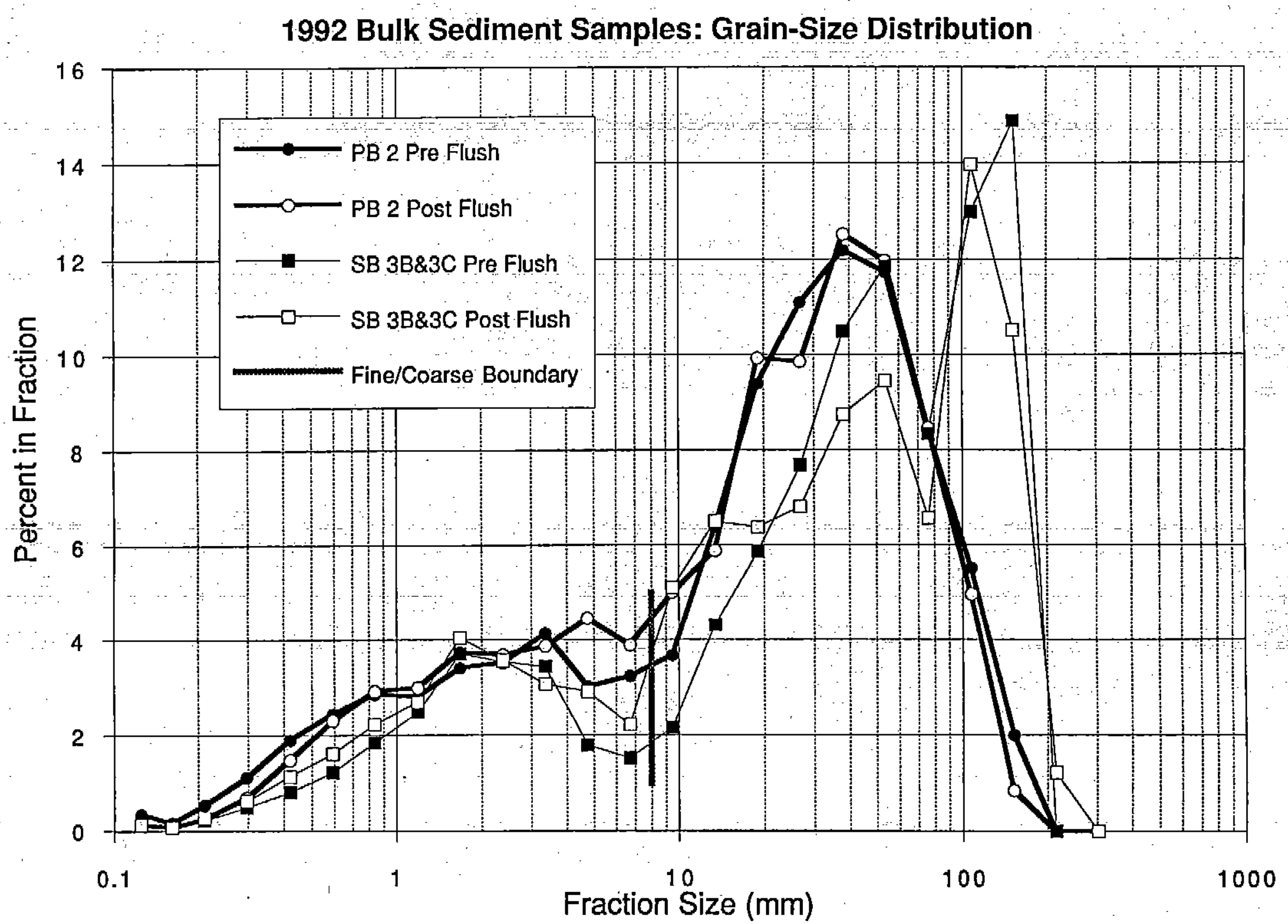


Figure 3.2.3 Size distribution of the bed material at the Poker Bar and Steelbridge study sites

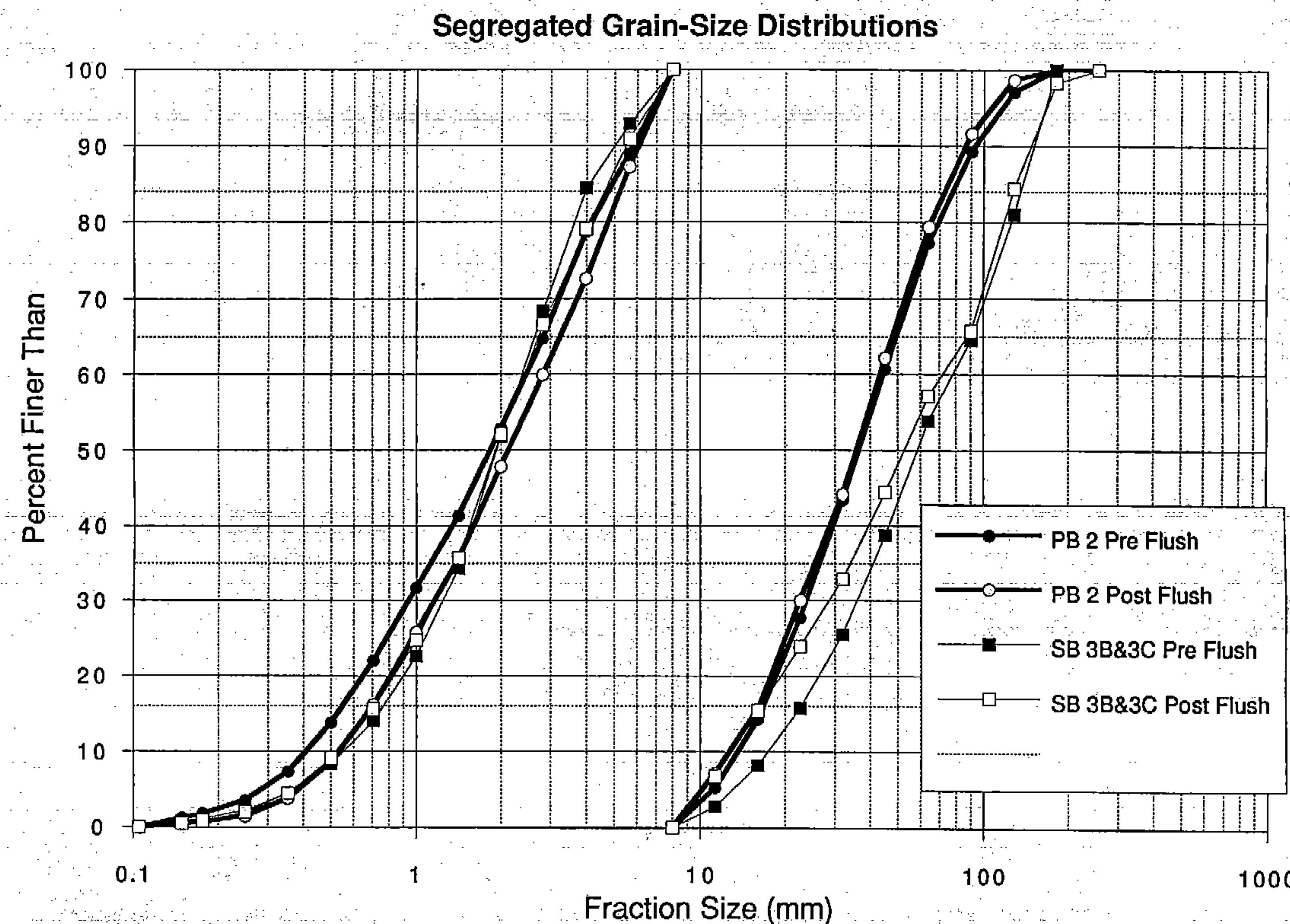


Figure 3.2.4 Cumulative size distribution of the bed material segregated into fine and coarseportions

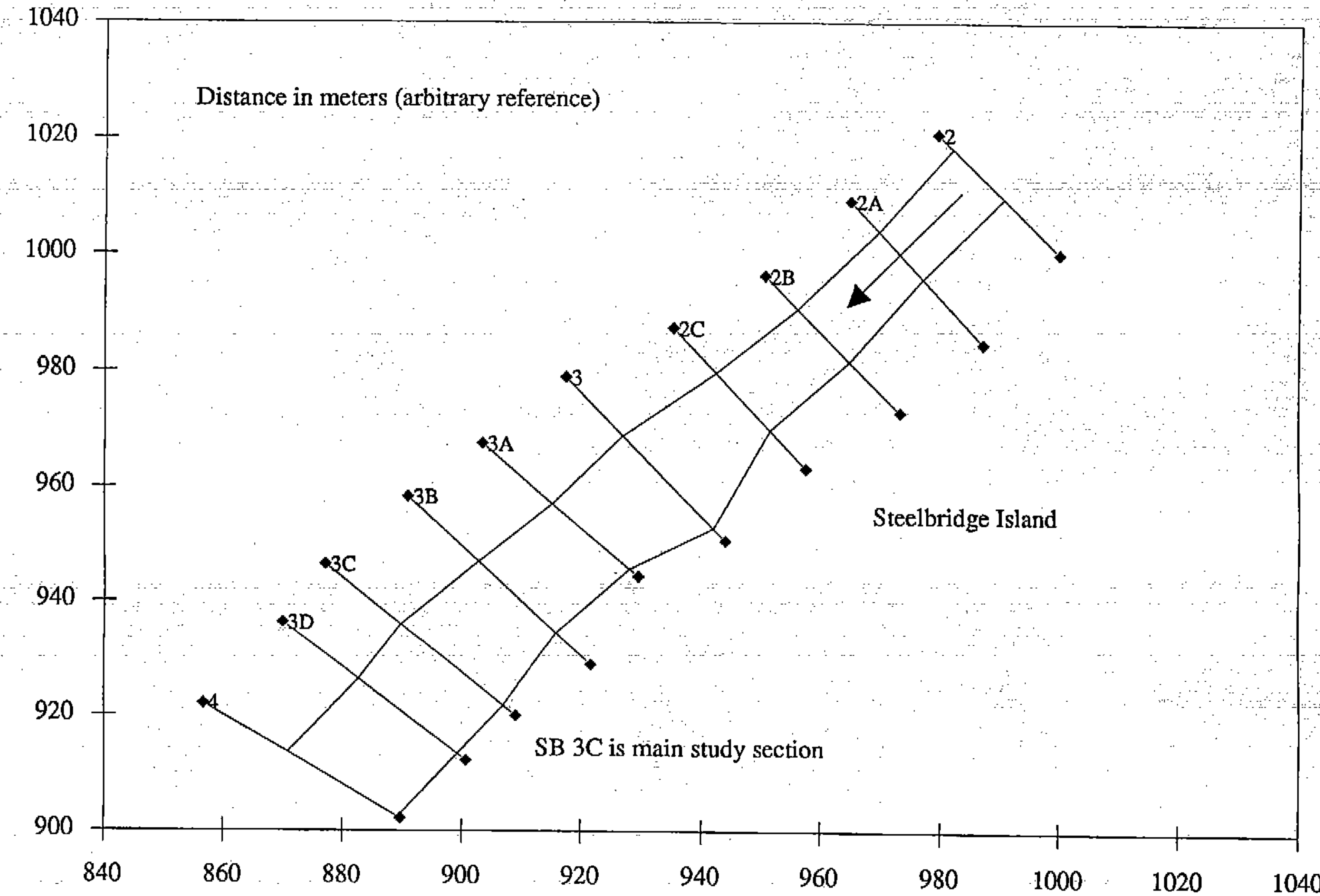


Figure 3.3.1 River bank and cross section locations for Steelbridge study reach

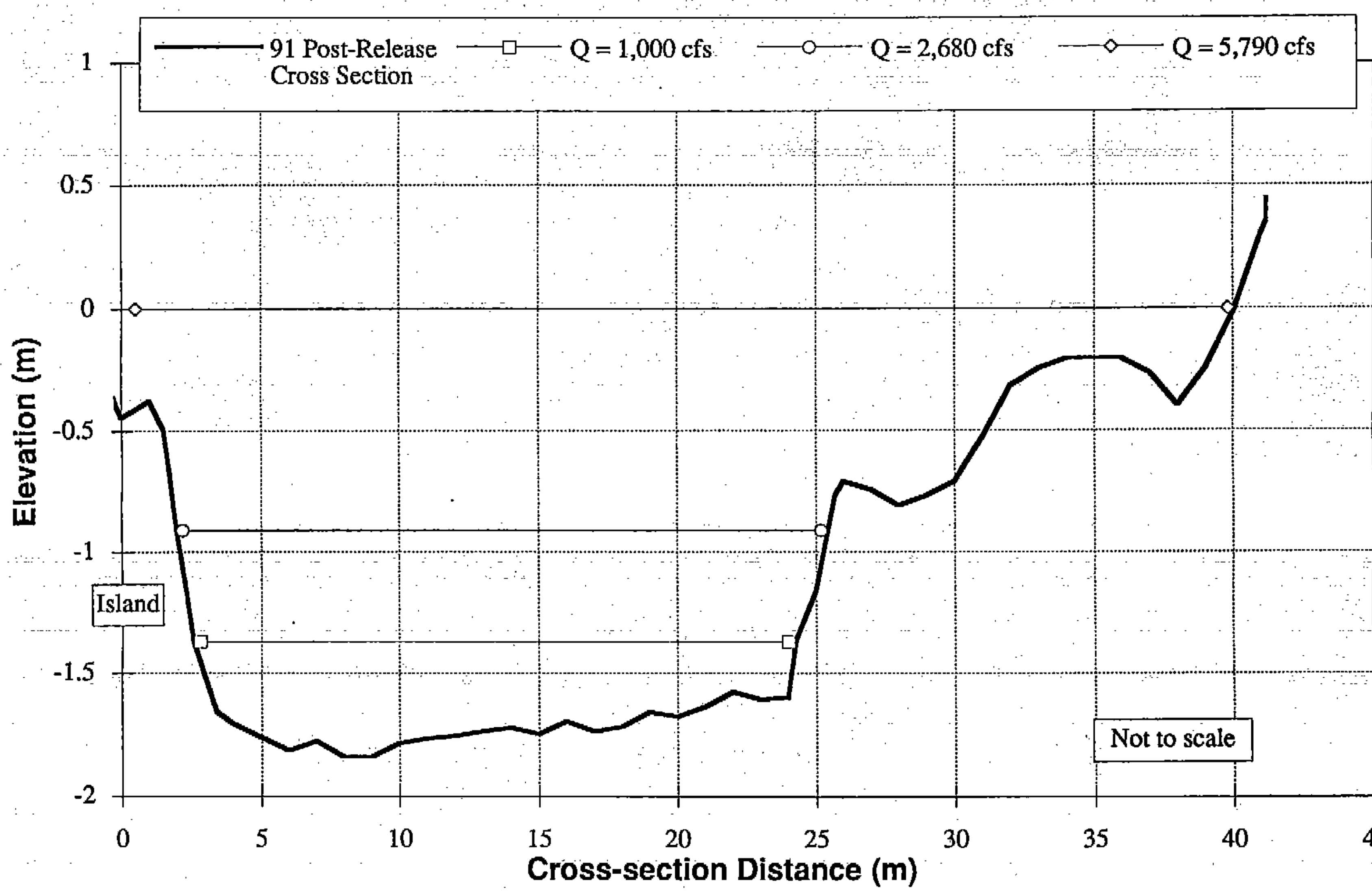


Figure 3.3.2. Cross section elevation of primary study section at Steelbridge, SB 3C, looking downstream. Water surface elevation at three different discharges also shown.

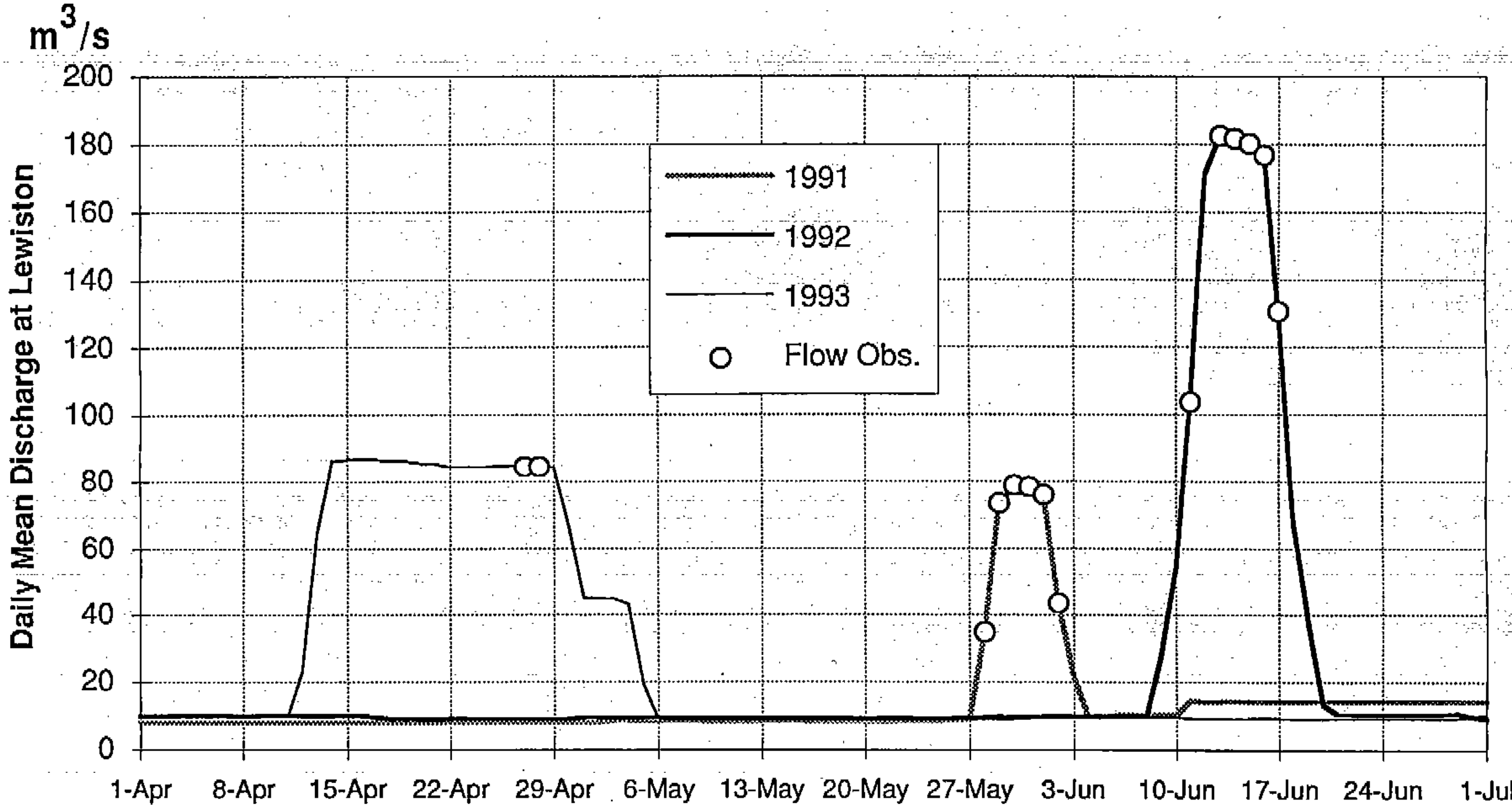
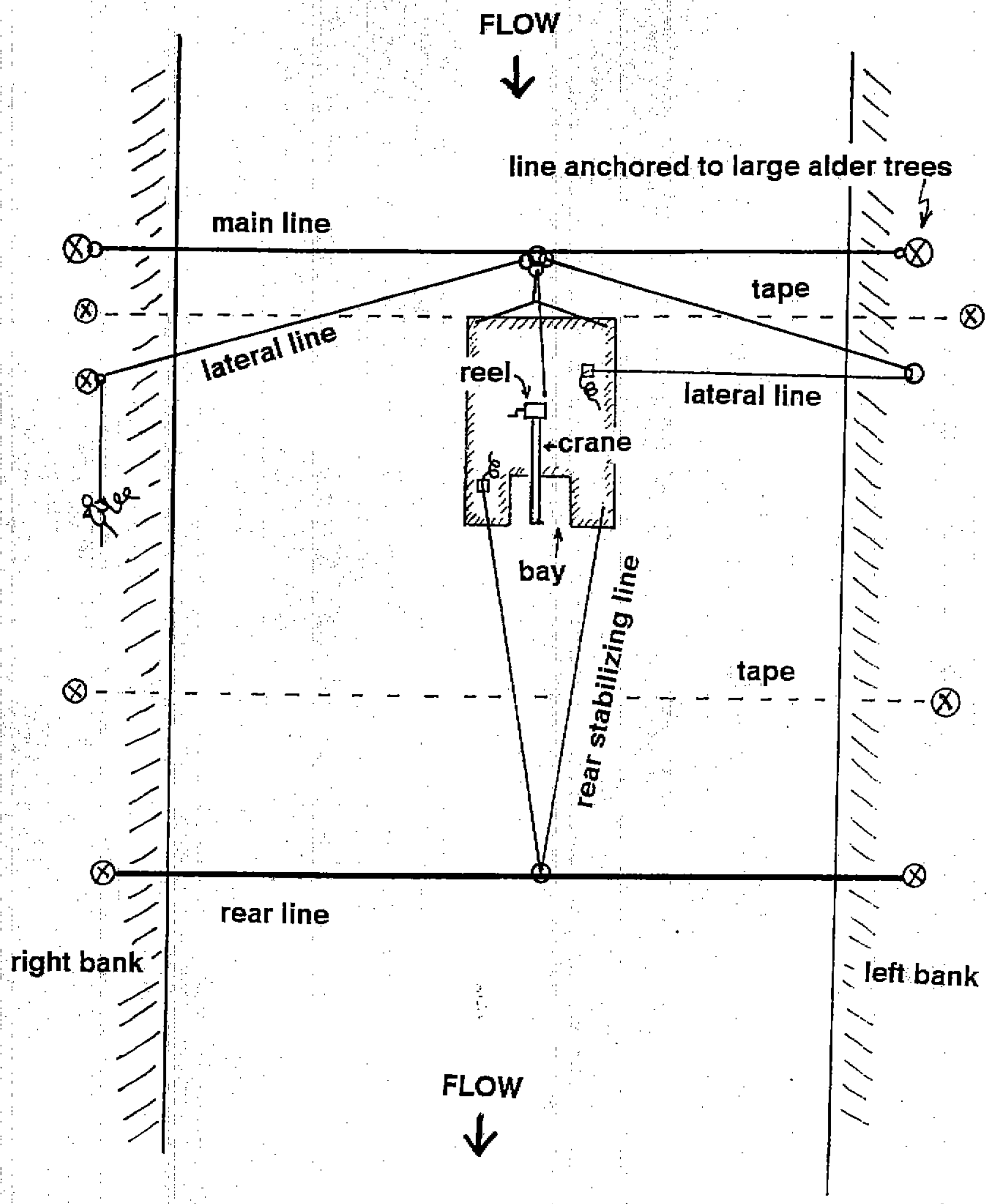
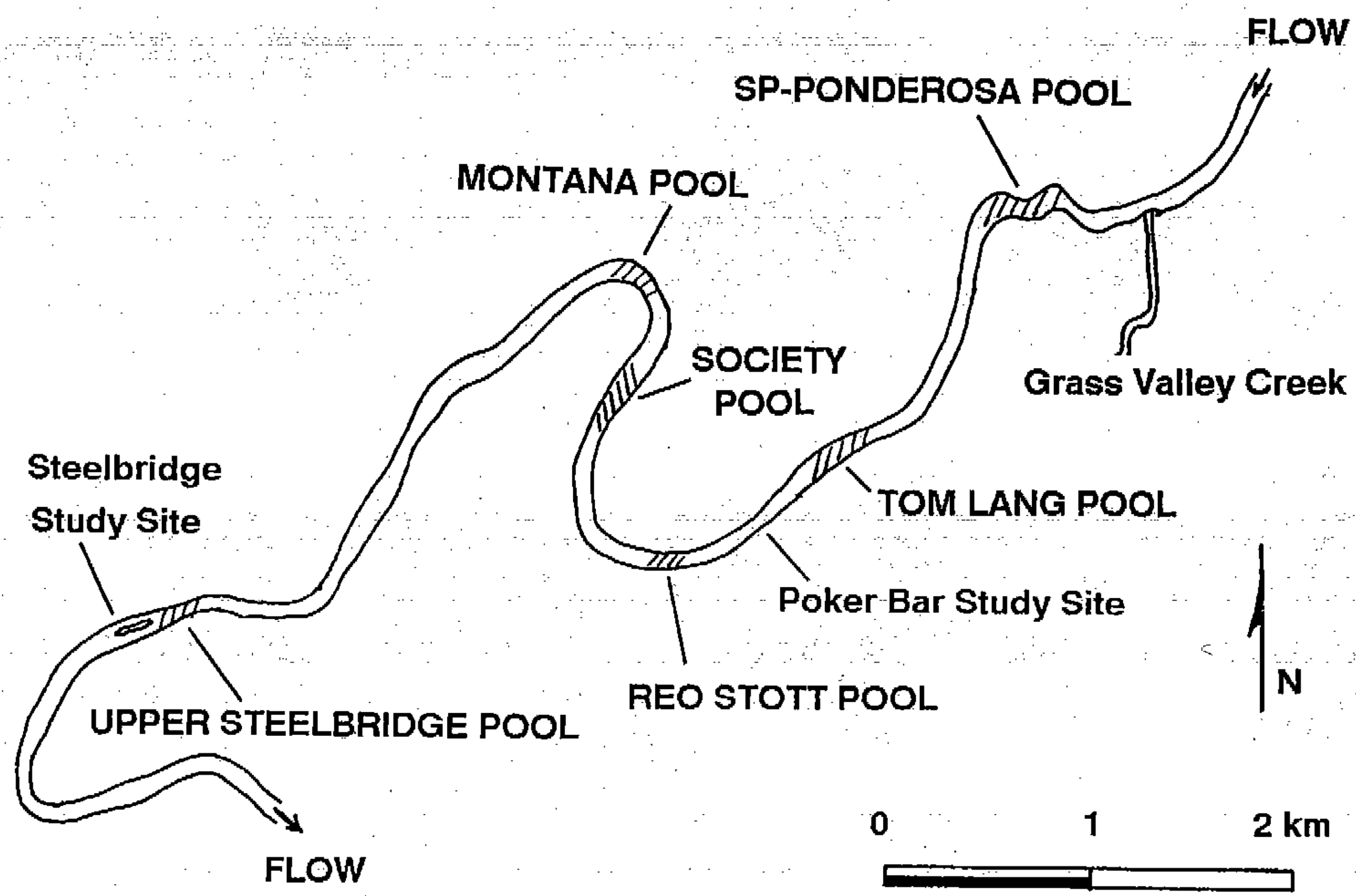


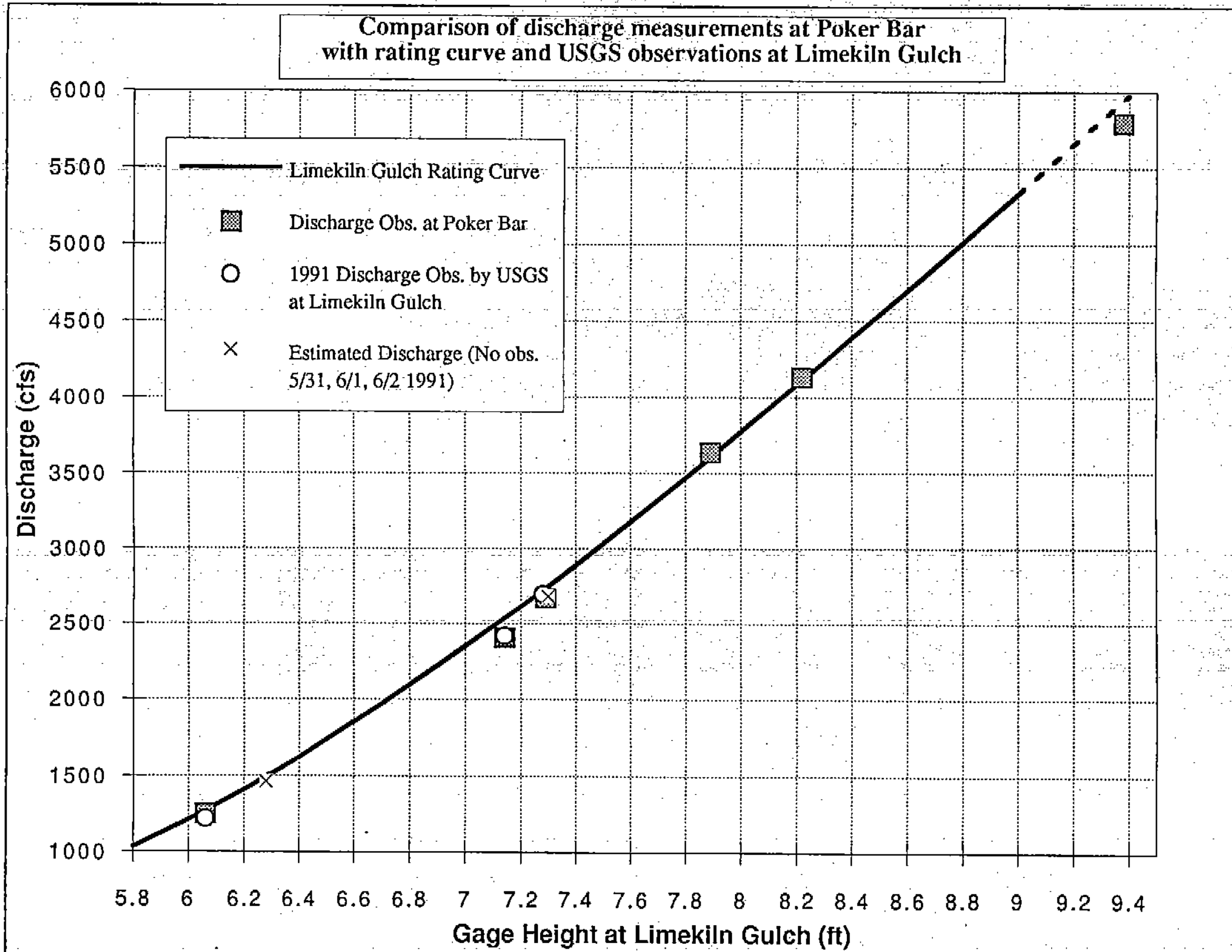
Figure 4.1.1. Daily mean discharge: Trinity River at Lewston, April - June, 1991, 1992, and 1993. Days with flow observations at study sites are indicated.



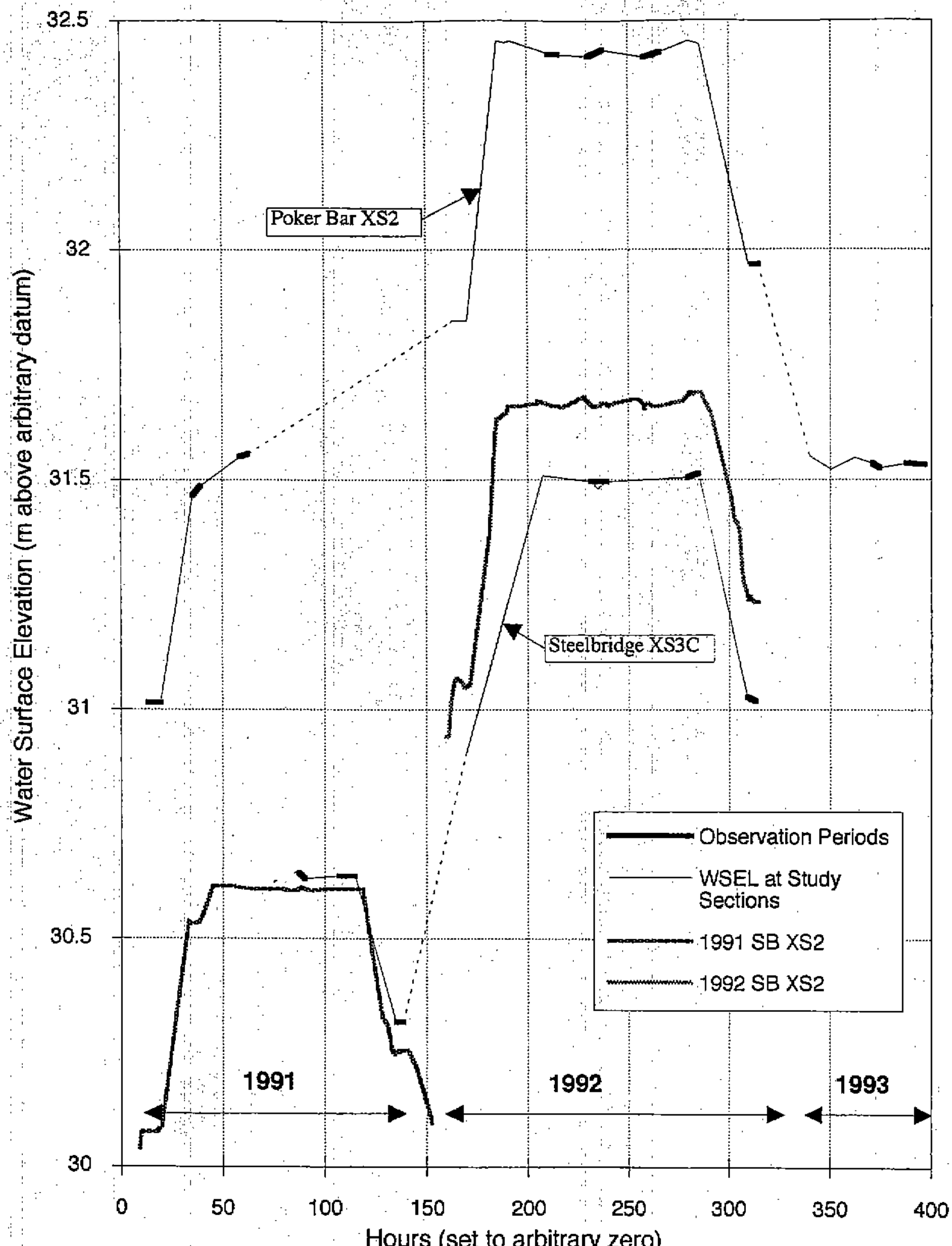
4.2.1. Schematic diagram of rope network for measurements at the Poker Bar study site.



4.6.1. Map of major pools between Grass Valley Creek and Steelbridge.



5.1.1 Comparison of calculated discharge at Poker Bar with observed stage, USGS observations, and rating curve at the Limekiln Gulch USGS gage.



5.1.2 Summary of water surface elevations during the trial releases.

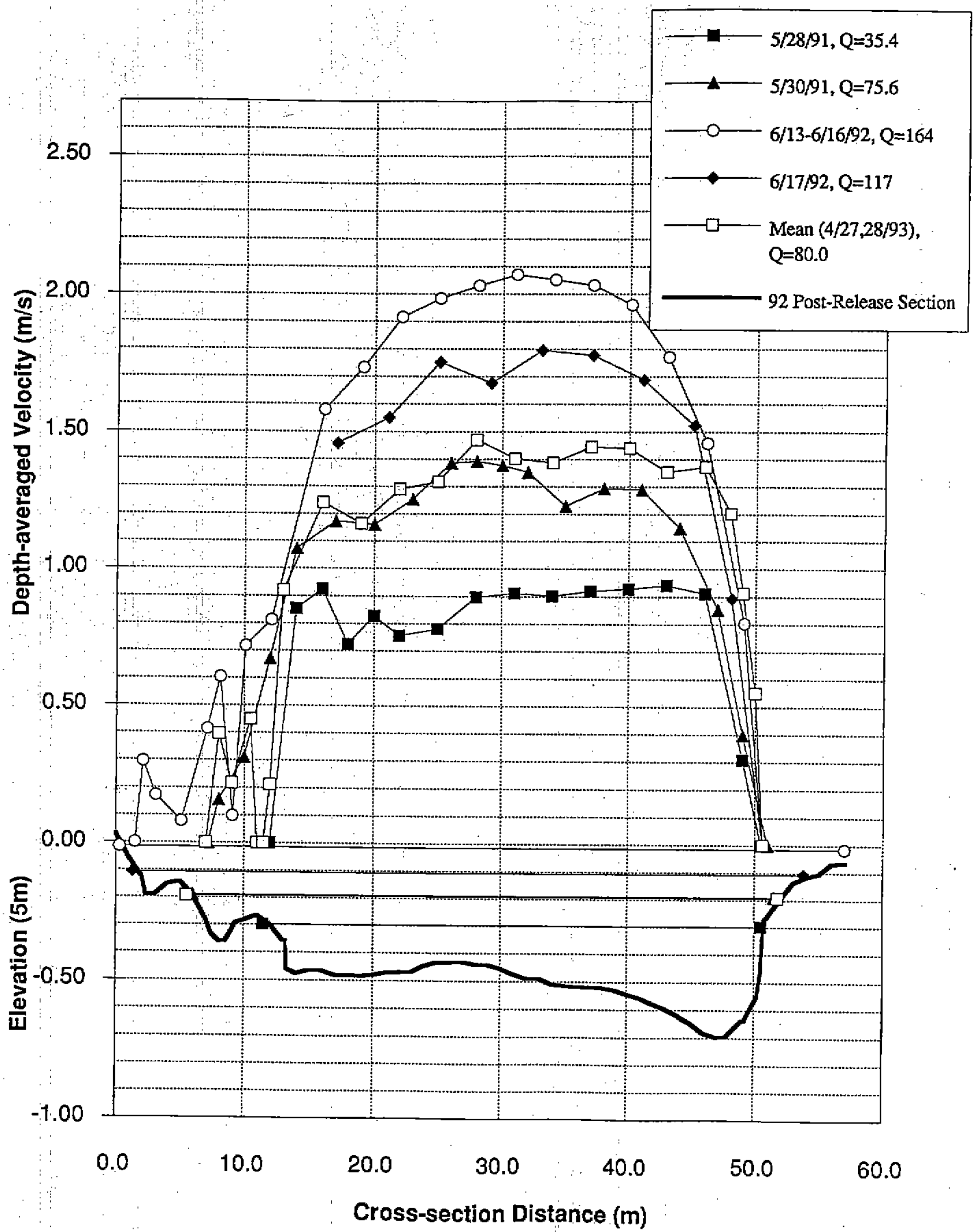


Figure 5.1.3 Depth-averaged velocity as a function of discharge and station along PB2

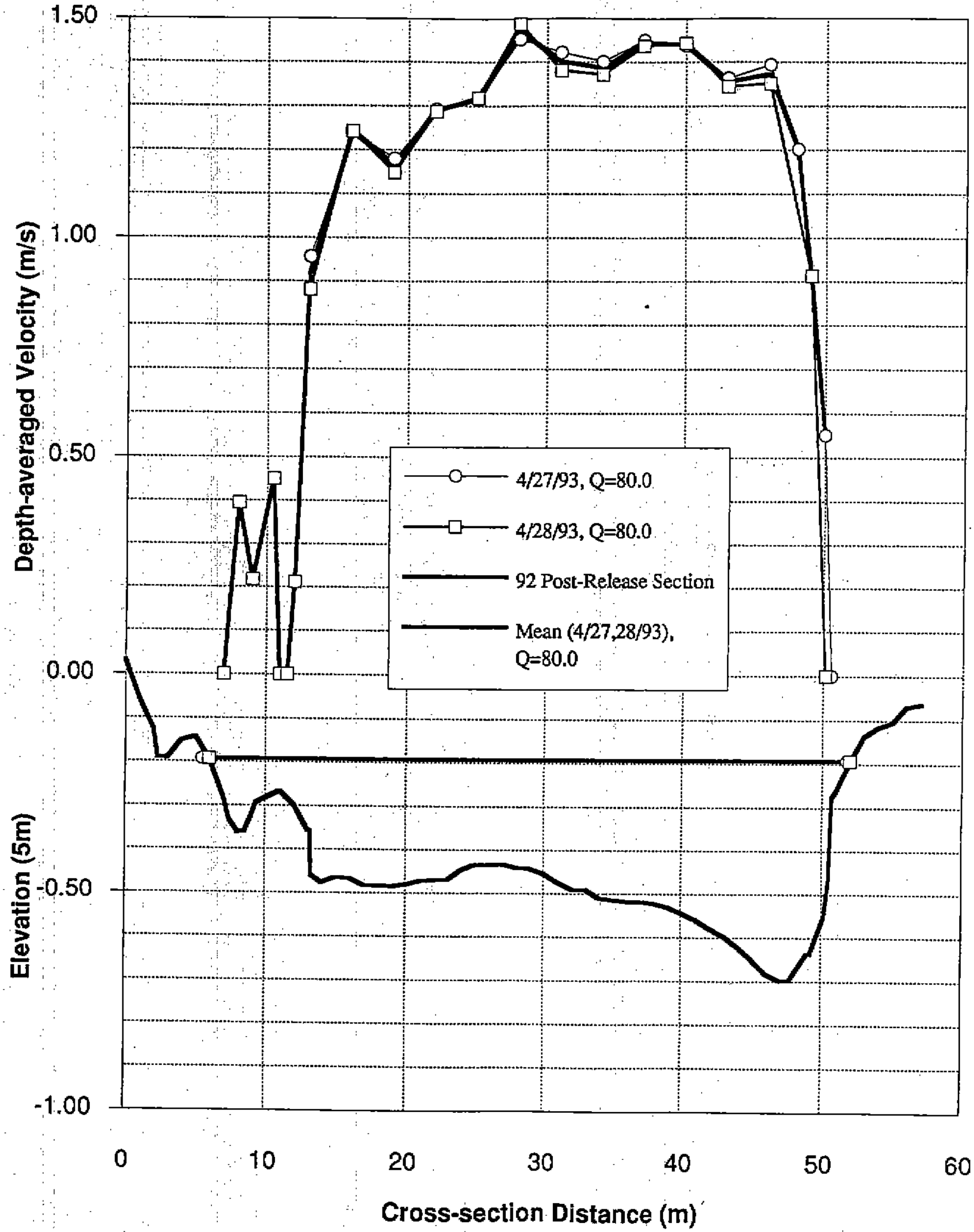


Figure 5.1.4 Depth-averaged velocity as a function of station along PB2 for a discharge of 80 cms

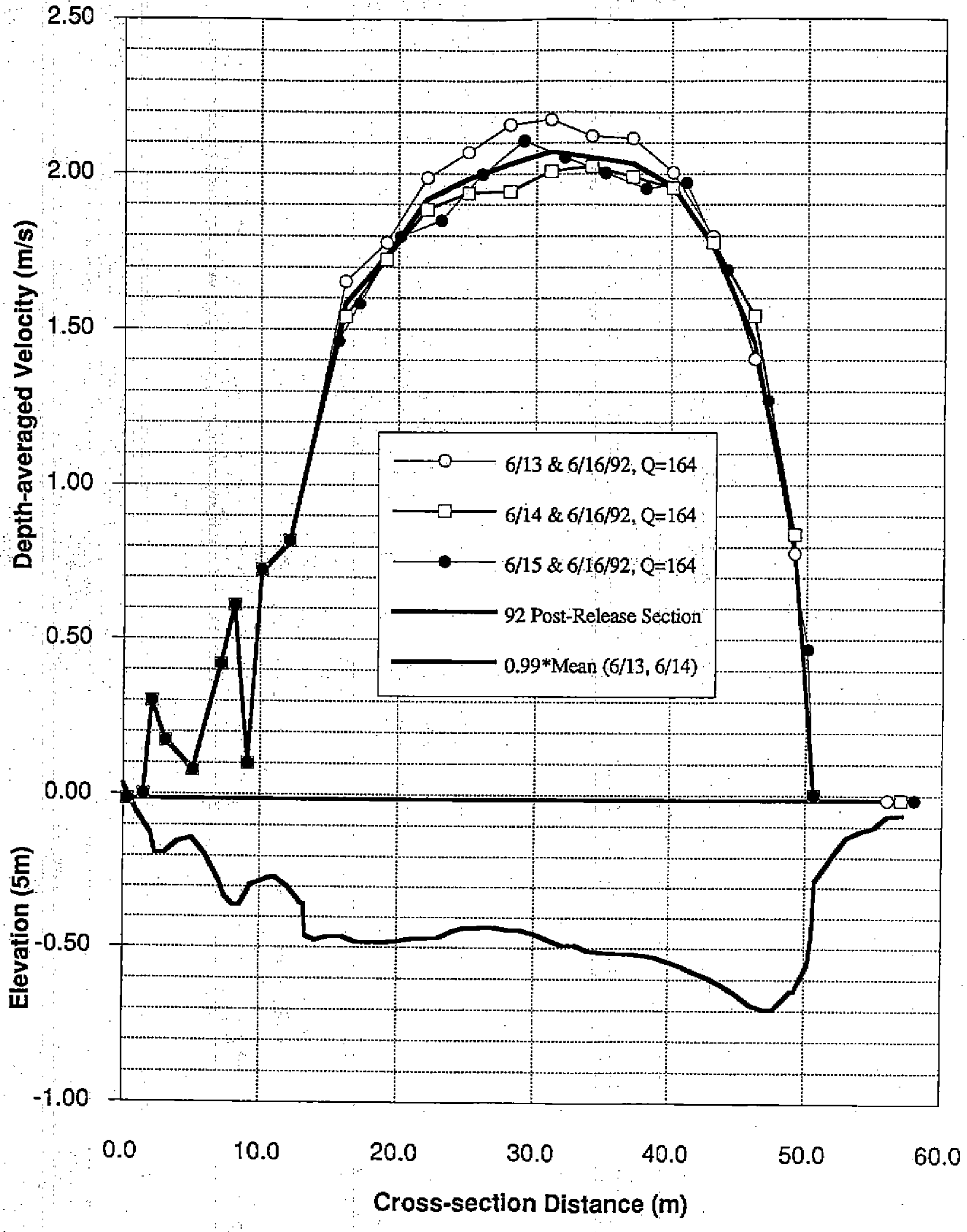


Figure 5.1.5 Depth-averaged velocity as a function of station along PB2 for a discharge of 164 cms

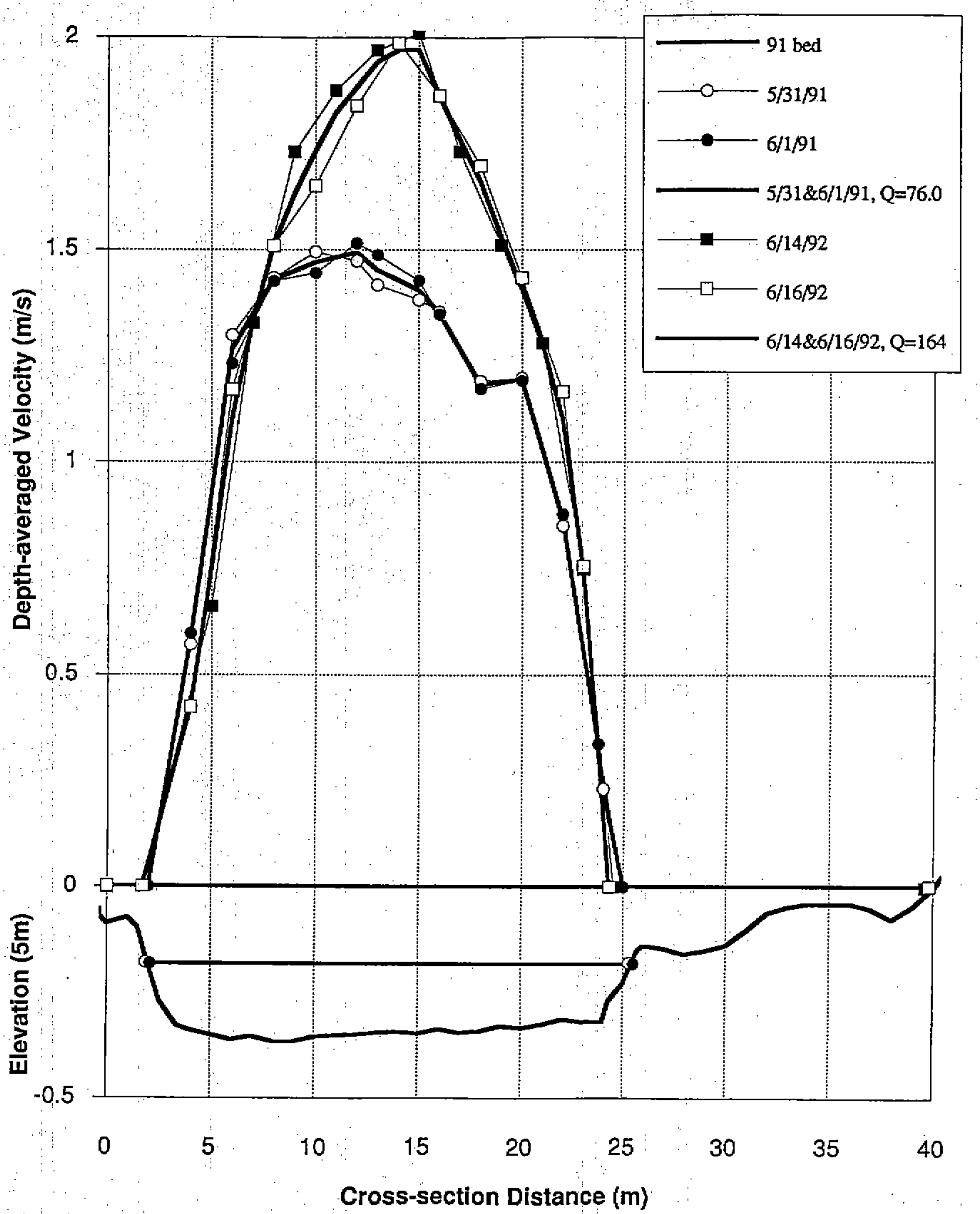


Figure 5.1.6 Depth-averaged velocity as a function of station along SB3C for discharges of 76 cms and 164 cms

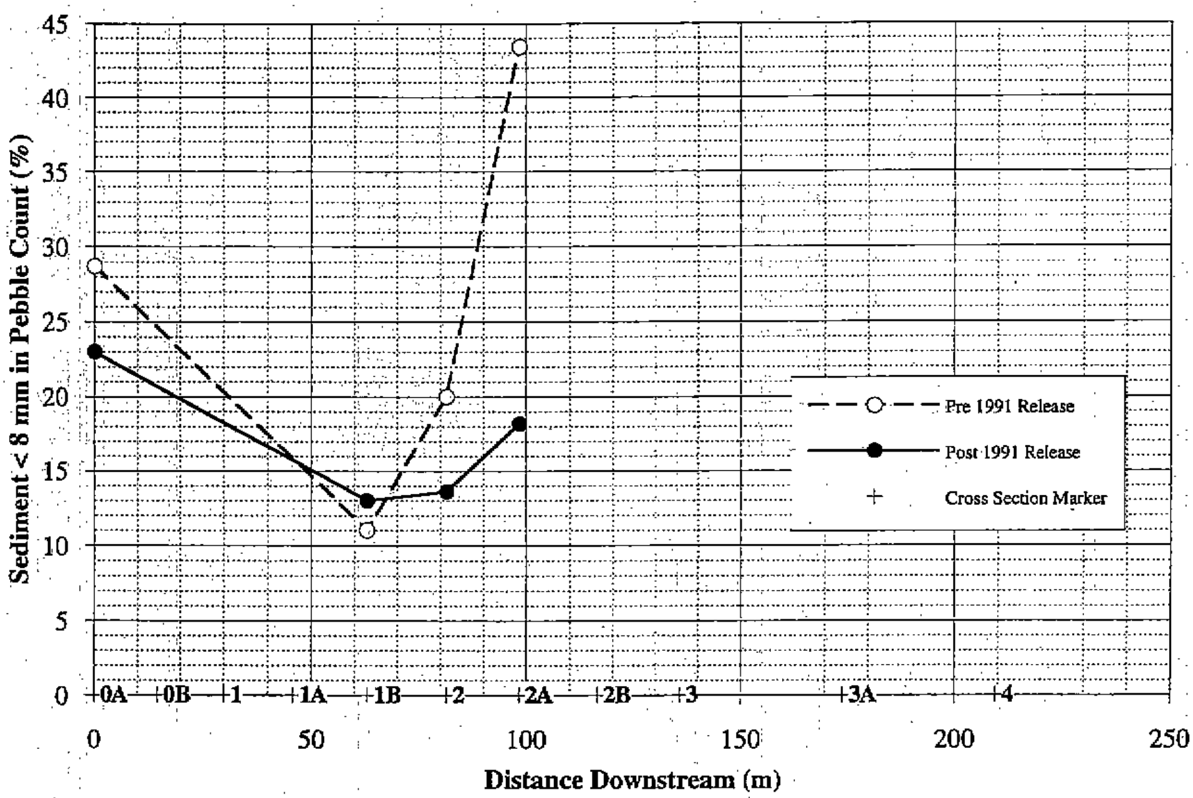


Figure 5.2.1 **1991 Poker Bar proportion of fine sediment vs. downstream distance.** The open symbols and dashed lines represent pre-release conditions and the closed symbols and solid lines represent post-release conditions. No definite trend is discernible in the pebble counts for the reach as a whole except for a significant decrease in fine sediments at cross section 2A.

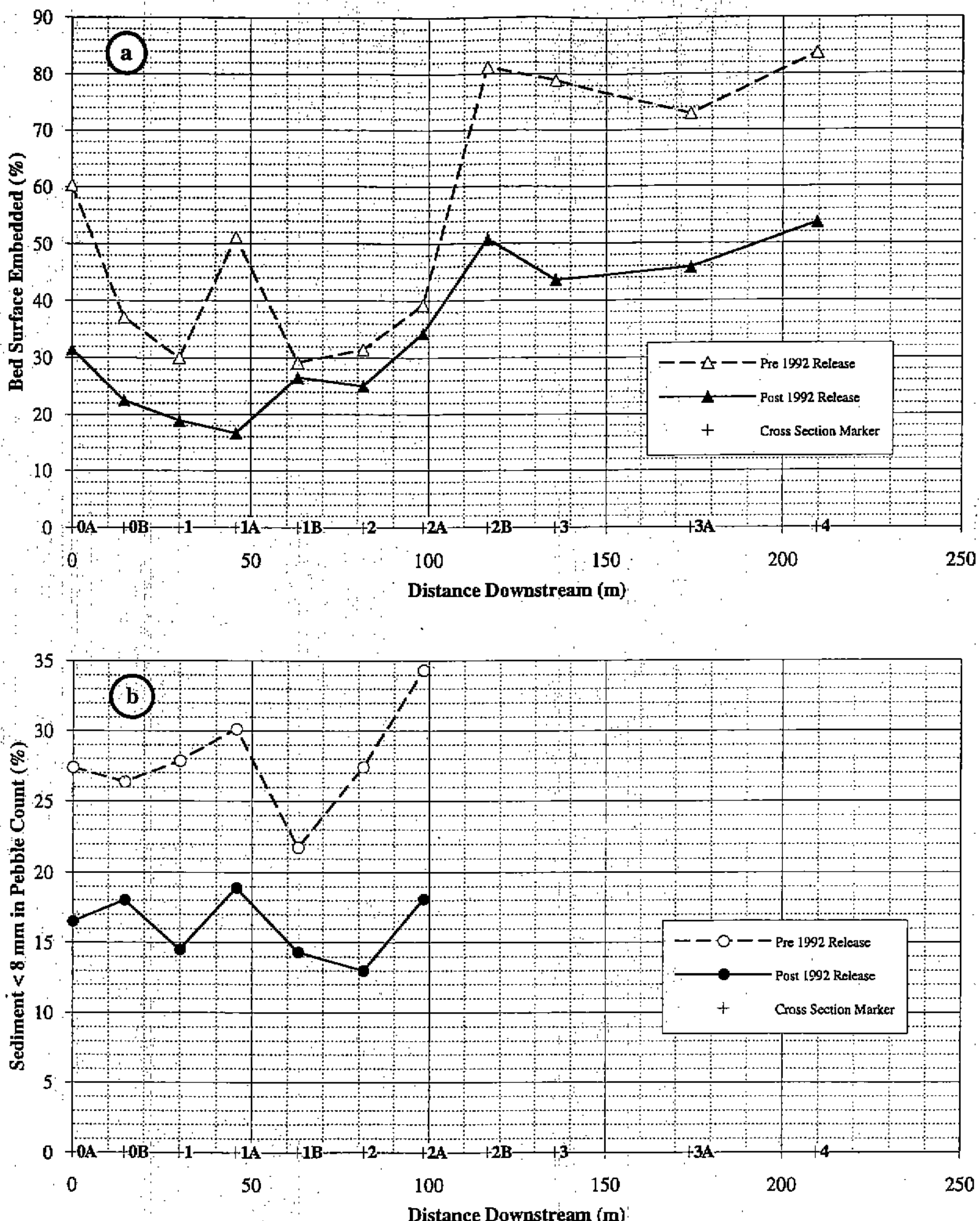


Figure 5.2.2 **1992 Poker Bar proportion of fine sediment vs. downstream distance.** The open symbols and dashed lines represent pre-release conditions and the closed symbols and solid lines represent post-release conditions. Both visual estimates and pebble counts show a decrease in the proportion of fine sediments on the bed surface as a result of the 1992 trial reservoir release.

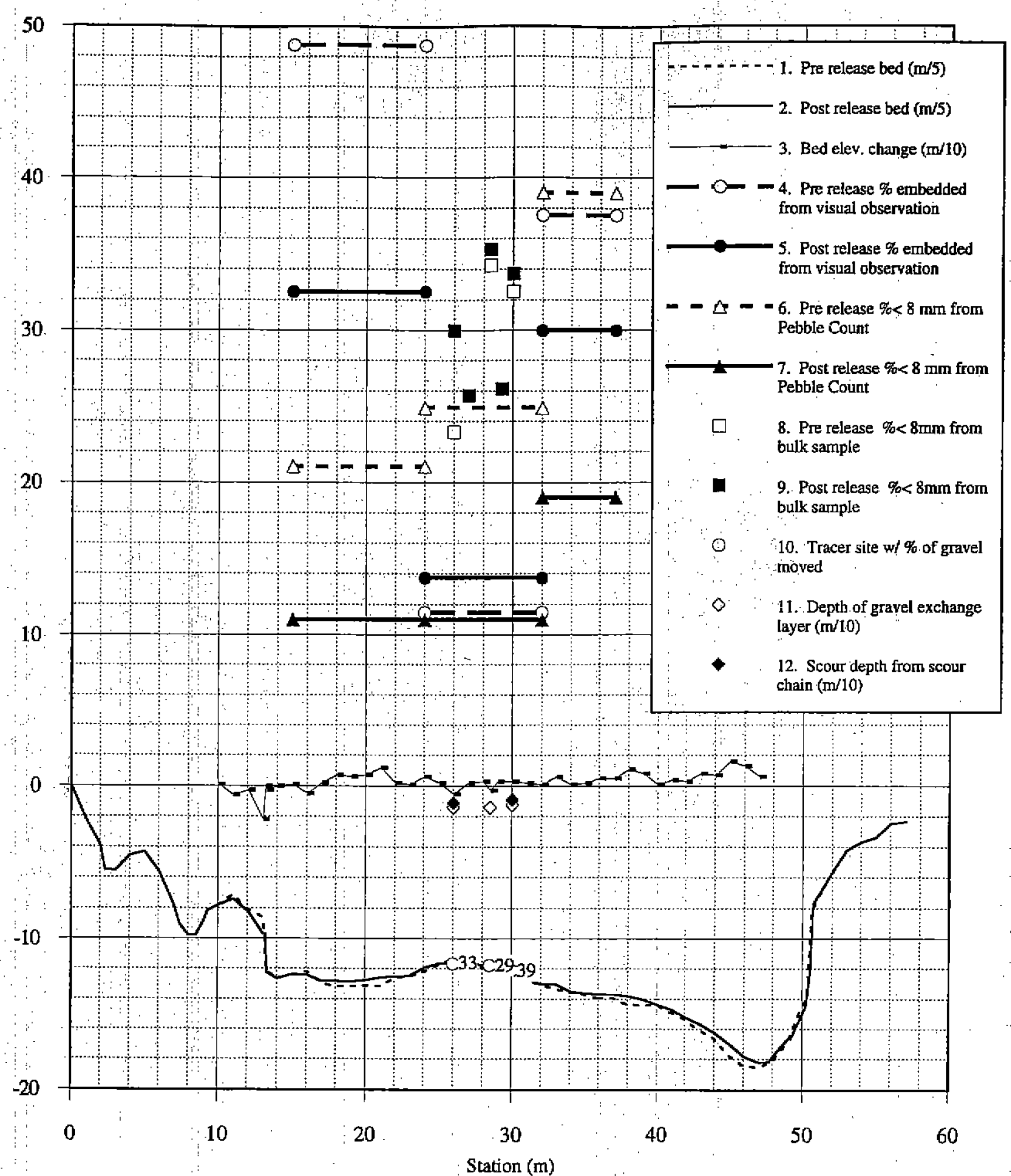


Figure 5.2.3 **Summary of 1992 Bed and Sediment Changes for Poker Bar Cross Section 2.**

Dashed lines and open symbols denote pre release conditions; solid lines and filled symbols denote post release conditions. Pre and post release bed elevations have been vertically exaggerated 5 times (items 1 & 2 in legend). Bed elevation changes have been exaggerated 10 times (item 3). Percent embedded values (items 4 & 5) represent averages of visual estimates for same portion of cross section that pebble counts (items 6 & 7) were done. Percent less than 8 mm for all bulk samples are shown for 3 pre release samples and 5 post release samples (items 8 & 9). The location of the tracer sites are shown with the percentage of tracers (by mass) which moved during the release (item 10). The depth of the gravel exchange layer (item 11) was estimated from the tracer results as described in the text. The scour depths from the scour chains (item 12) have been vertically exaggerated 10 times.

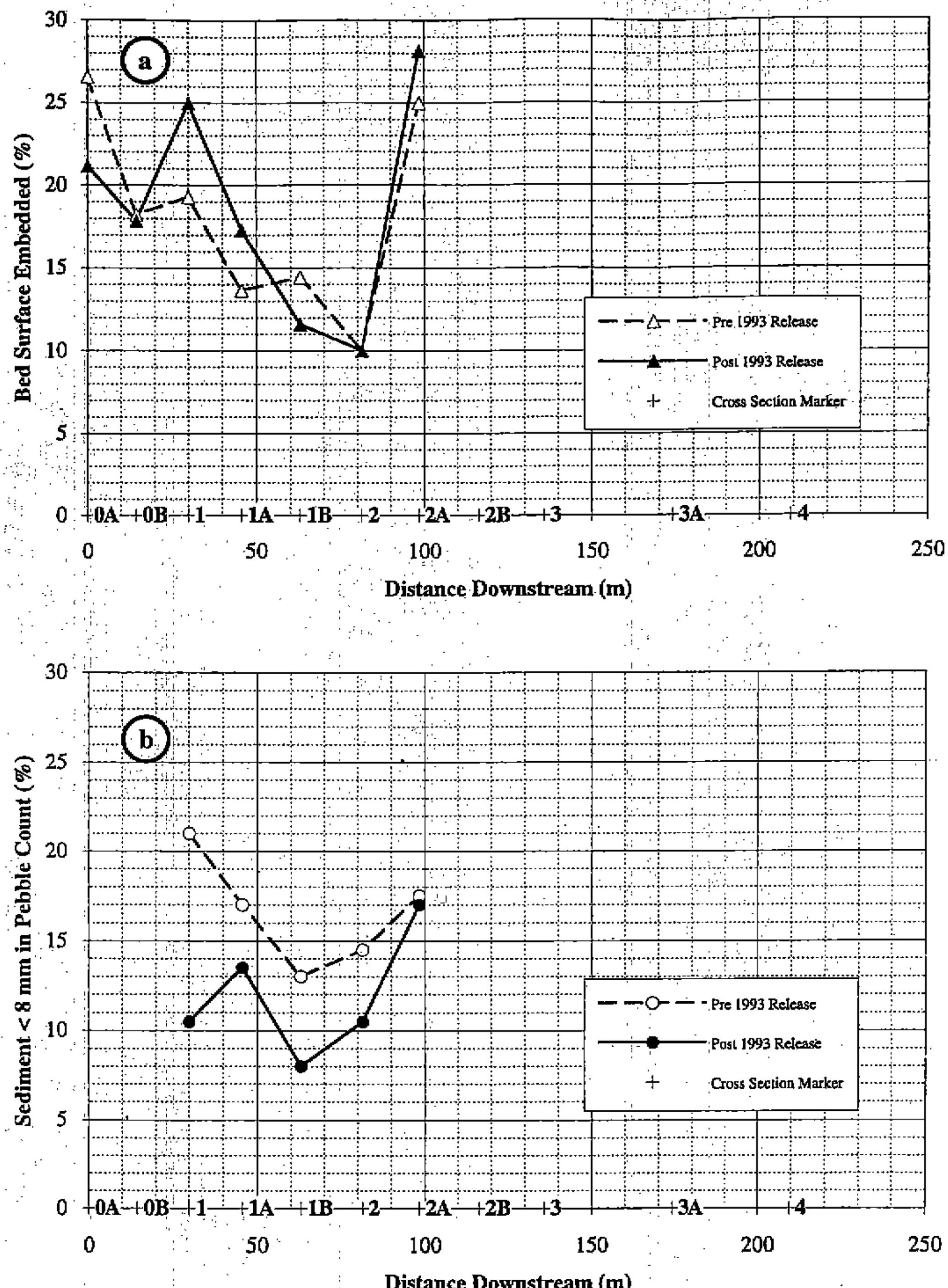


Figure 5.2.4 **1993 Poker Bar proportion of fine sediment vs. downstream distance.** The open symbols and dashed lines represent pre-release conditions and the closed symbols and solid lines represent post-release conditions. The visual estimates (a) show no change and the pebble counts (b) show a slight decrease in the proportion of fine sediments on the bed surface after the 1993 trial reservoir release.

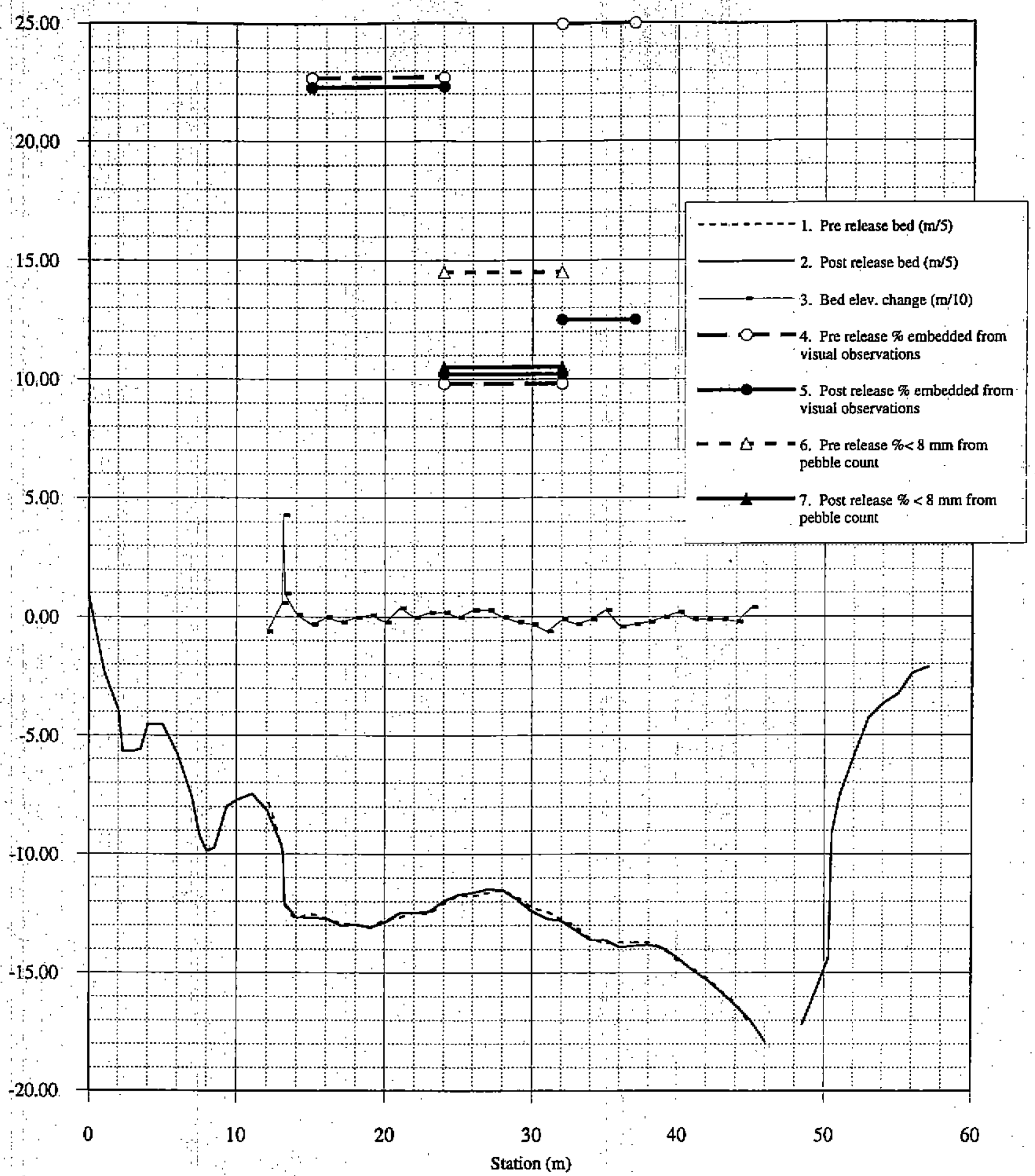


Figure 5.2.5 **Summary of 1993 Bed and Sediment Changes for Poker Bar Cross Section 2.**

Dashed lines and open symbols denote pre release conditions; solid lines and filled symbols denote post release conditions. Pre and post release bed elevations have been vertically exaggerated 5 times (items 1 & 2 in legend). Bed elevation changes have been exaggerated 10 times (item 3). Percent embedded values (items 4 & 5) represent averages of visual estimates for same portion of cross section that pebble counts (items 6 & 7) were done. A reduction in fine sediment occurred between stations 32 and 37 but the other monitored portions of the bed were essentially unchanged.

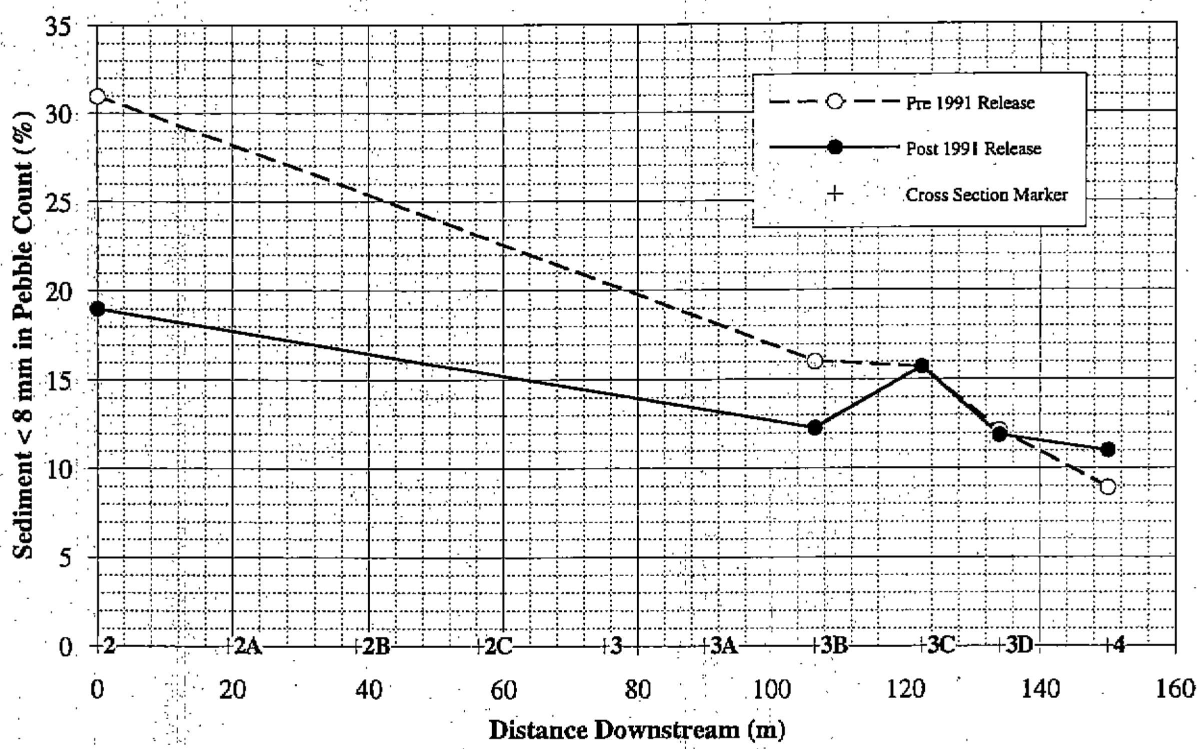


Figure 5.2.6 **1991 Steelbridge proportion of fine sediment vs. downstream distance.** The open symbols and dashed lines represent pre-release conditions and the closed symbols and solid lines represent post-release conditions. The pebble counts over the spawning gravels (XS 3B-3D) show negligible differences as a result of the 1991 trial release. The pebble count at cross section 2 suggests that a slight decrease in fine sediments occurred upstream.

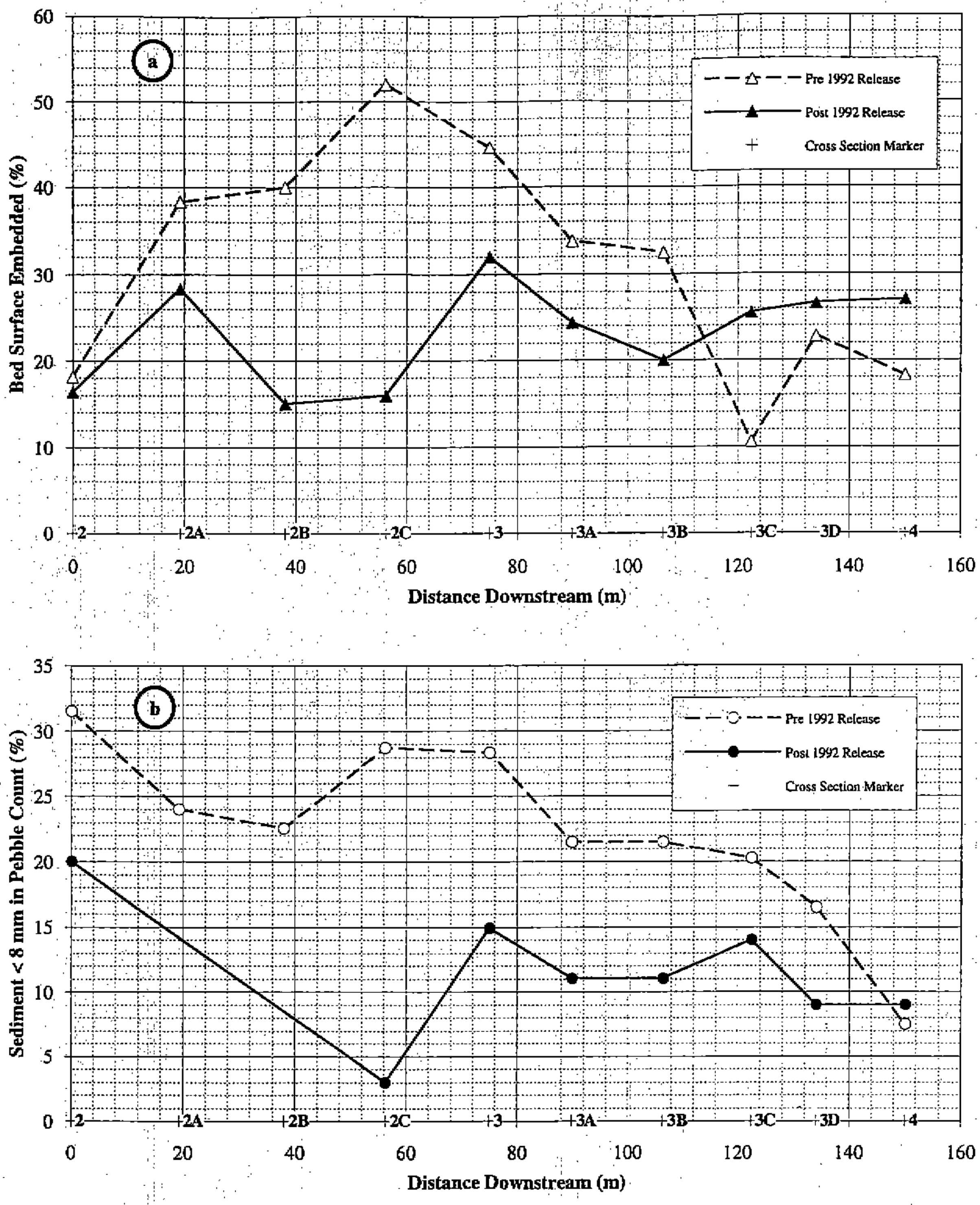


Figure 5.2.7 **1992 Steelbridge proportion of fine sediment vs. downstream distance.** The open symbols and dashed lines represent pre-release conditions and the closed symbols and solid lines represent post-release conditions. The visual estimate (a) shows a reduction in surficial fine sediment at all but the most downstream 3 cross sections. The pebble counts (b) show a decrease in fine sediments throughout virtually the entire study reach.

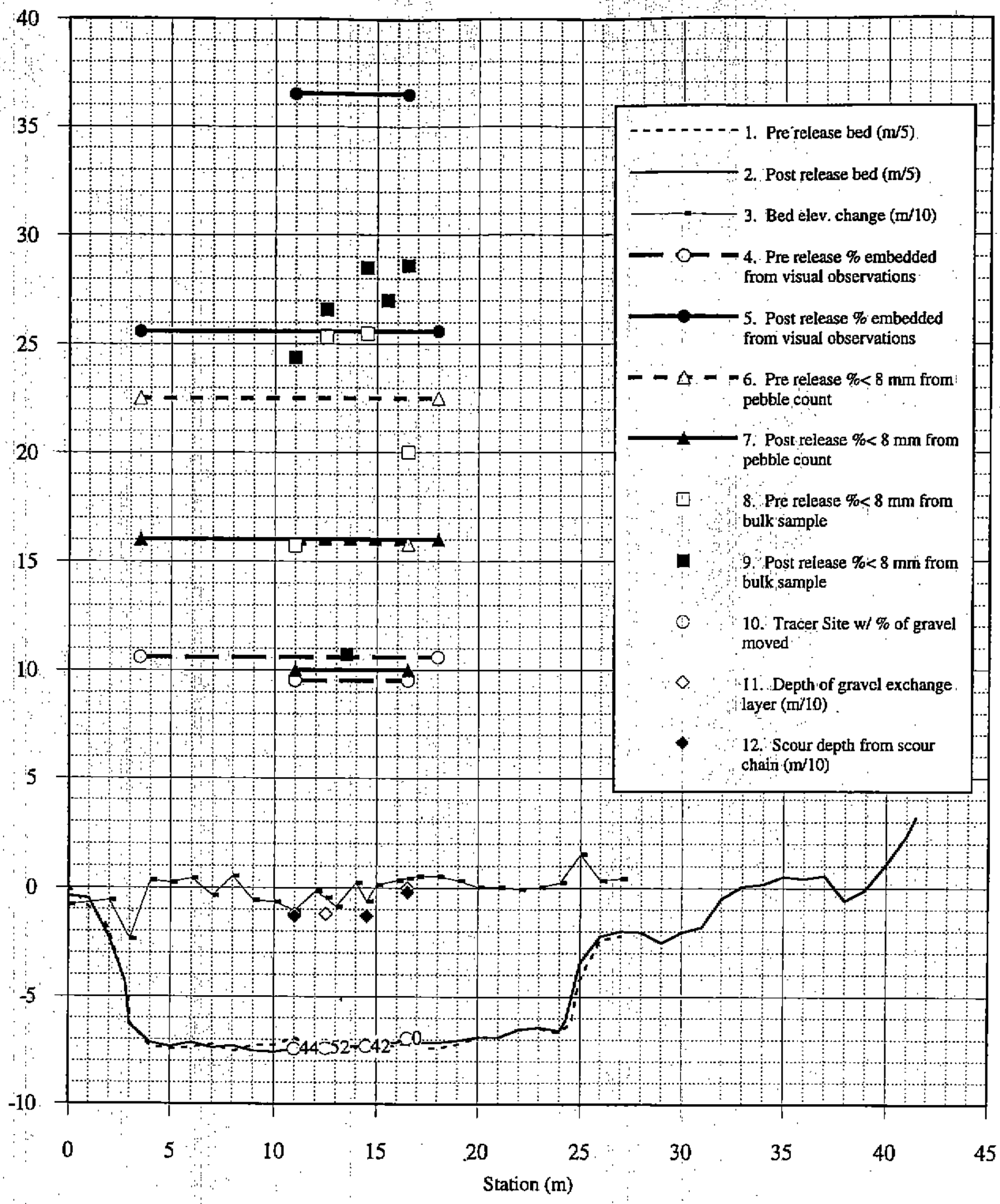
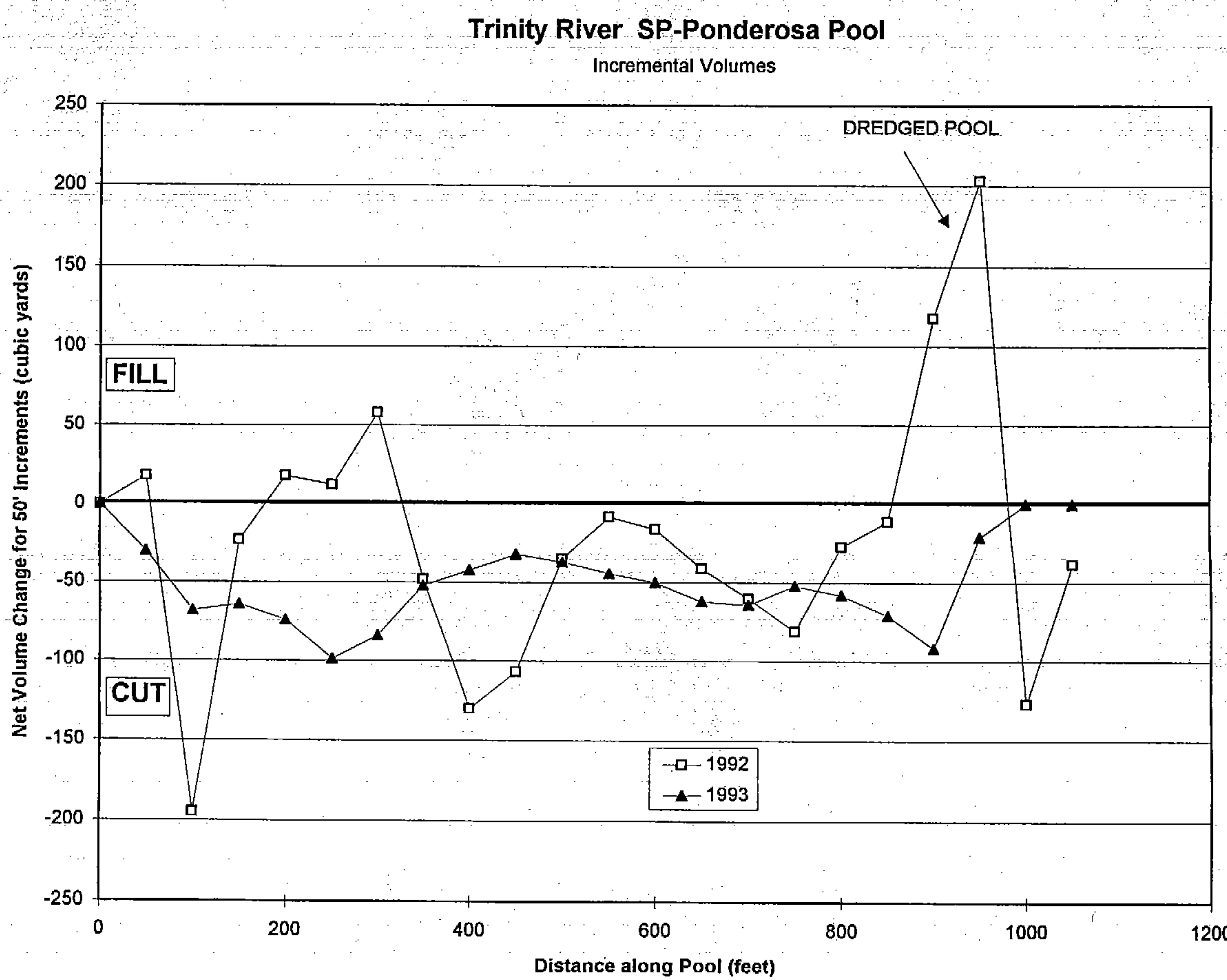
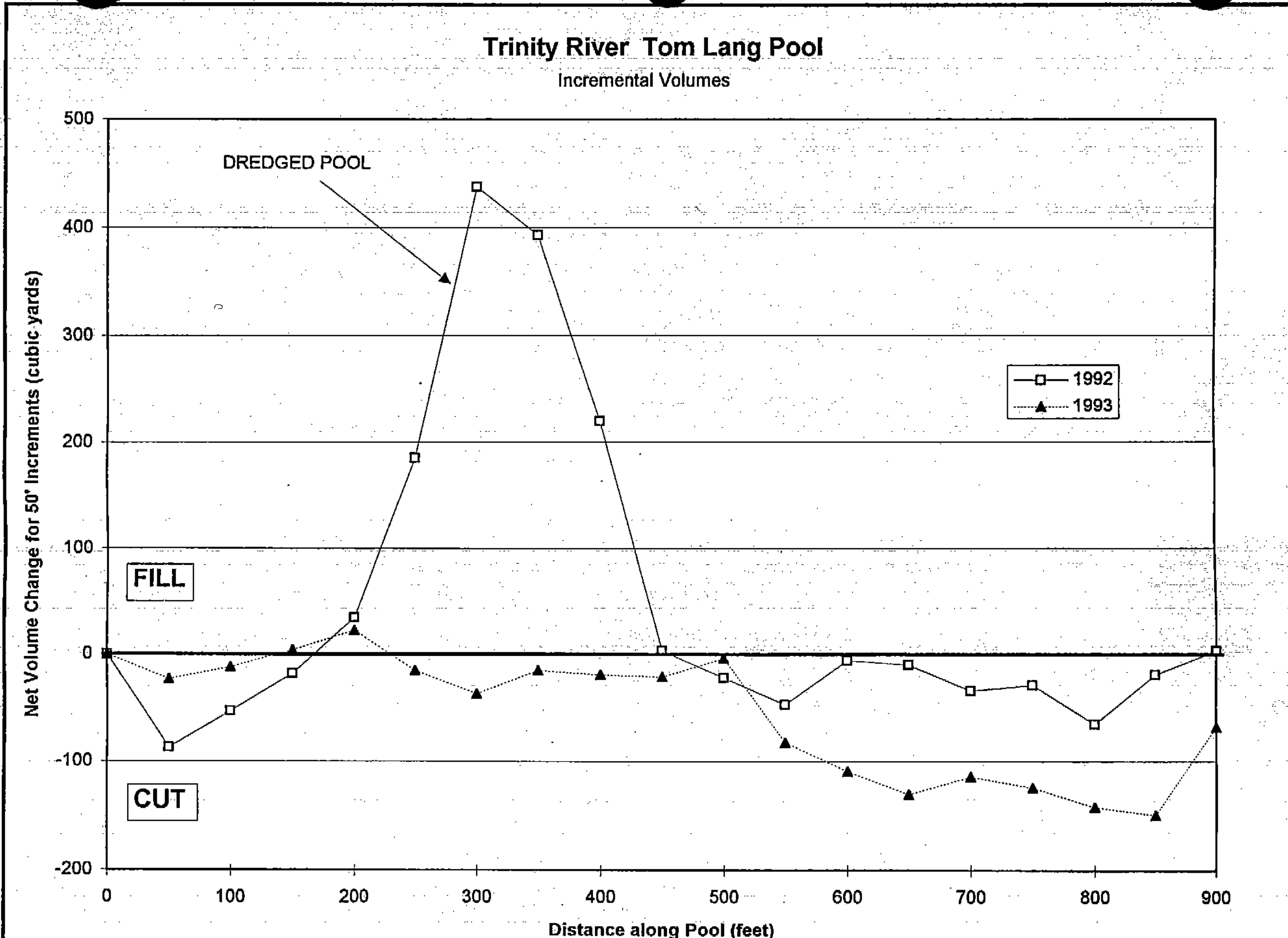


Figure 5.2.8 Summary of 1992 Bed and Sediment Changes for Steelbridge Cross Section 3C.

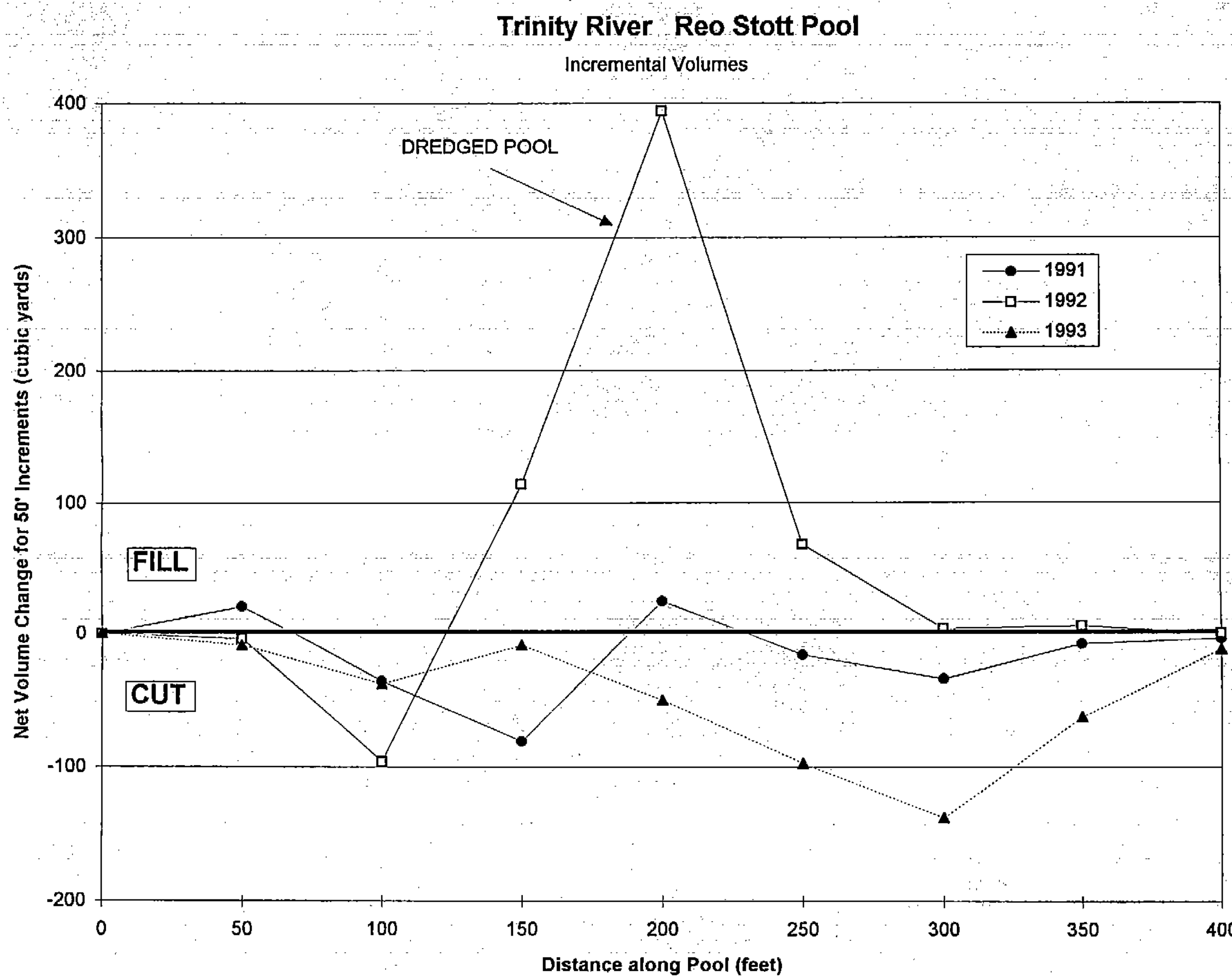
Dashed lines and open symbols denote pre release conditions; solid lines and filled symbols denote post release conditions. Pre and post release bed elevations have been vertically exaggerated 5 times (items 1 & 2 in legend). Bed elevation changes have been exaggerated 10 times (item 3). Percent embedded values (items 4 & 5) represent averages of visual estimates for same portion of cross section that pebble counts (items 6 & 7) were done. Percent less than 8 mm for all bulk samples are shown for 4 pre release samples and 6 post release samples (items 8 & 9). The location of the tracer sites are shown with the percentage of tracers (by mass) which moved during the release (item 10). The depth of the gravel exchange layer (item 11) was estimated from the tracer results as described in the text. The scour depths from the scour chains (item 12) have been vertically exaggerated 10 times.



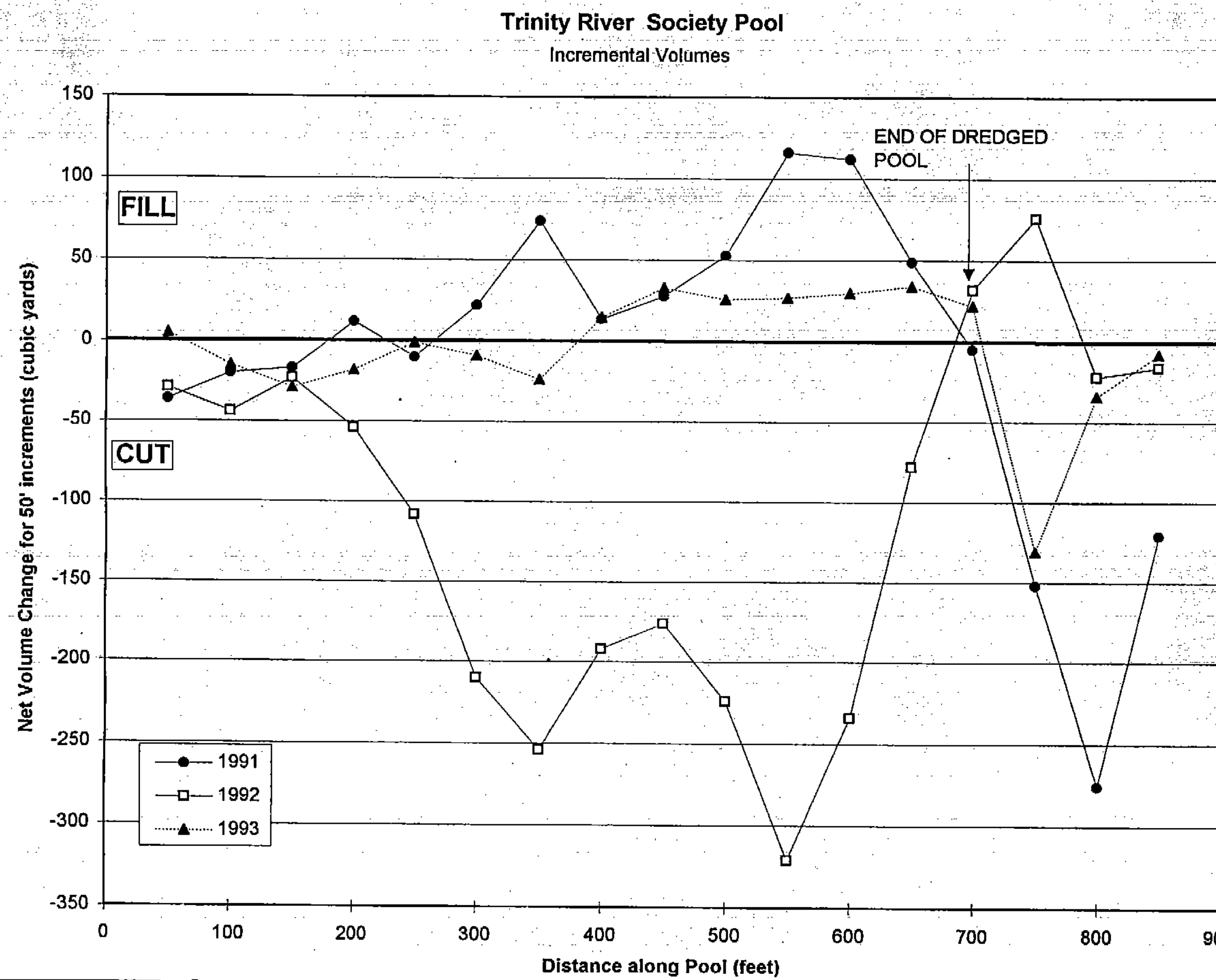
5.5.1. Longitudinal pattern of cut-and-fill in SP/Ponderosa Pool in 1992 and 1993.



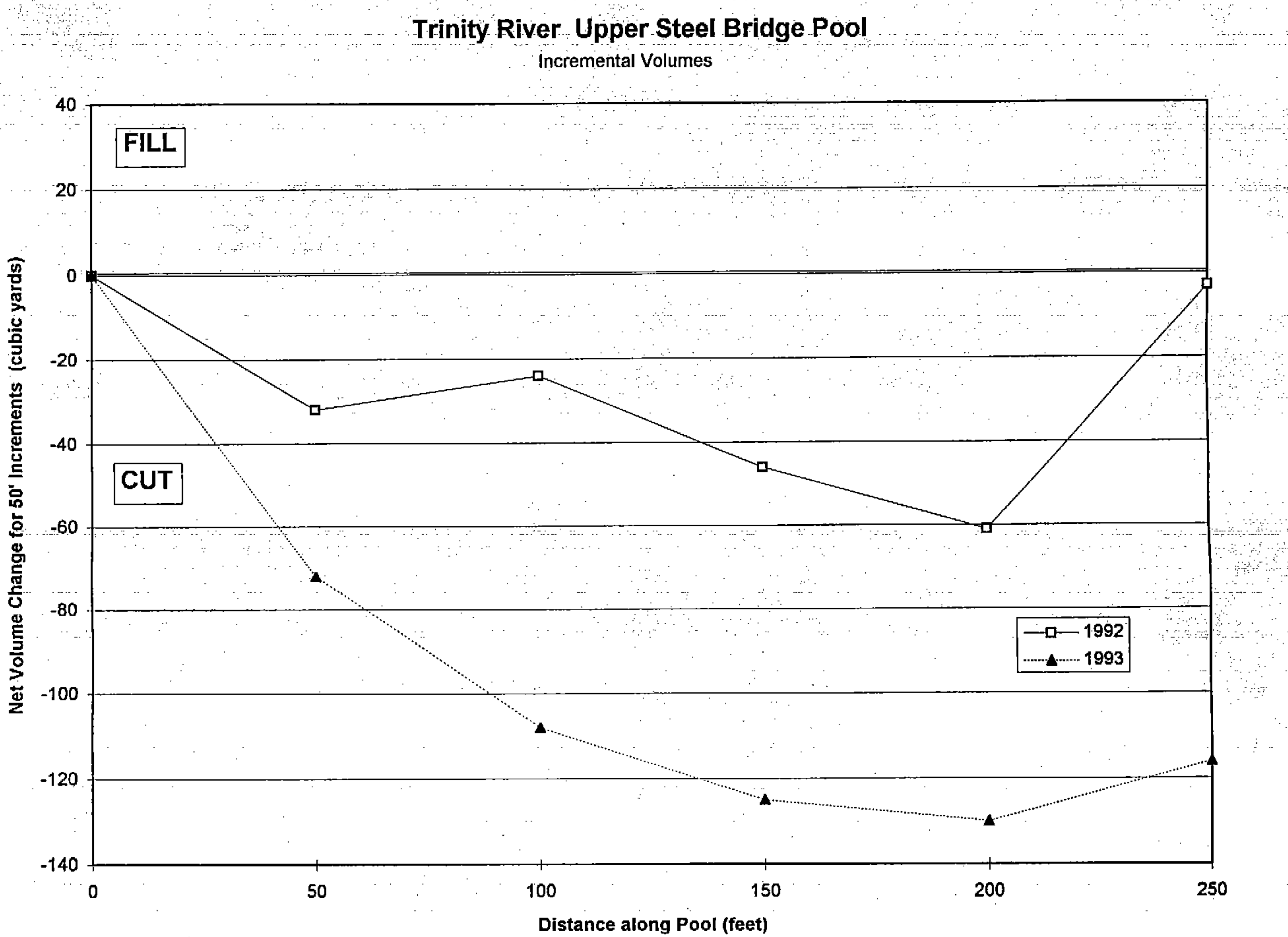
5.5.2. Longitudinal pattern of cut-and-fill in Tom Lang Pool in 1992 and 1993.



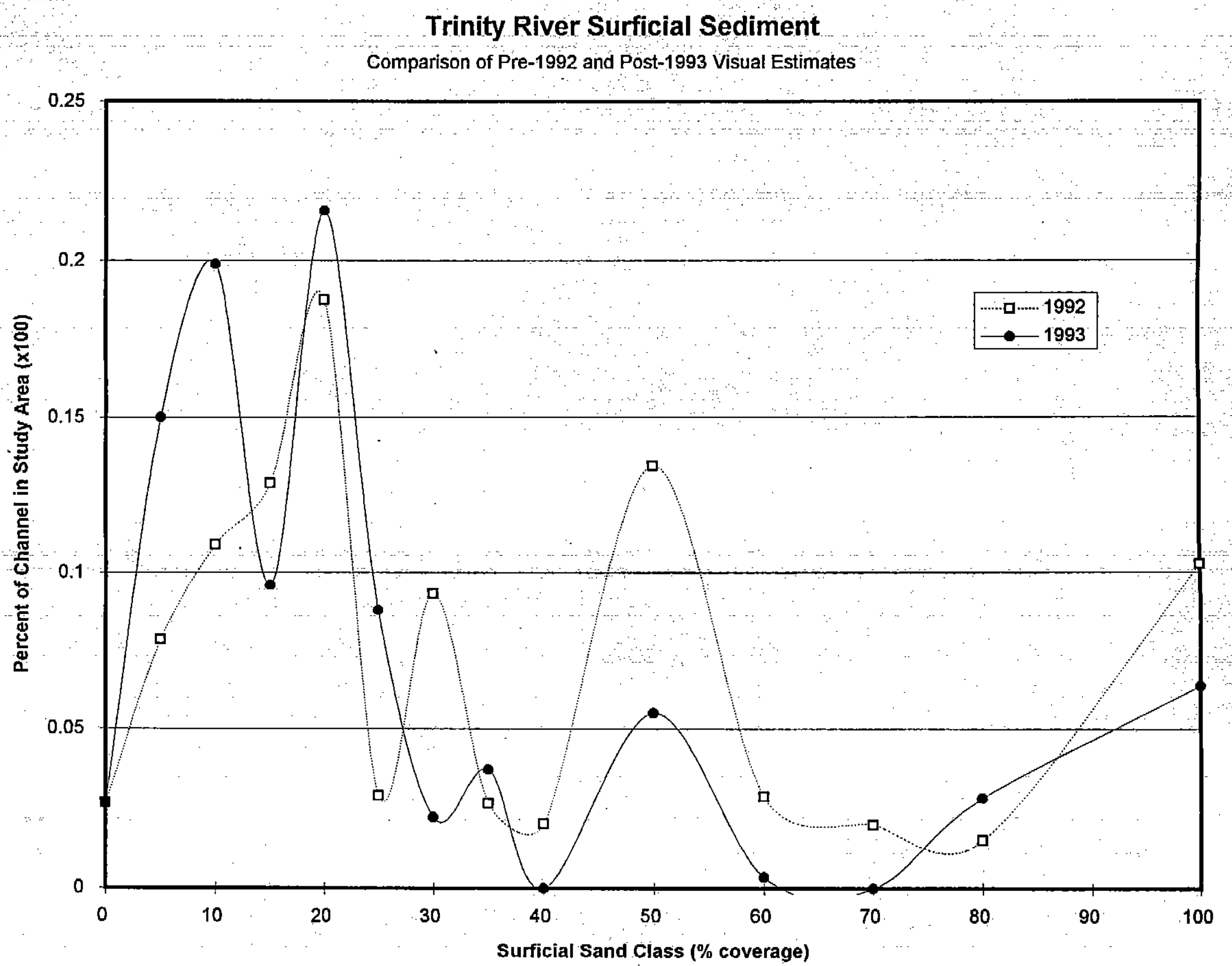
5.5.3. Longitudinal pattern of cut-and-fill in Reo Stott Pool in 1991, 1992, and 1993.



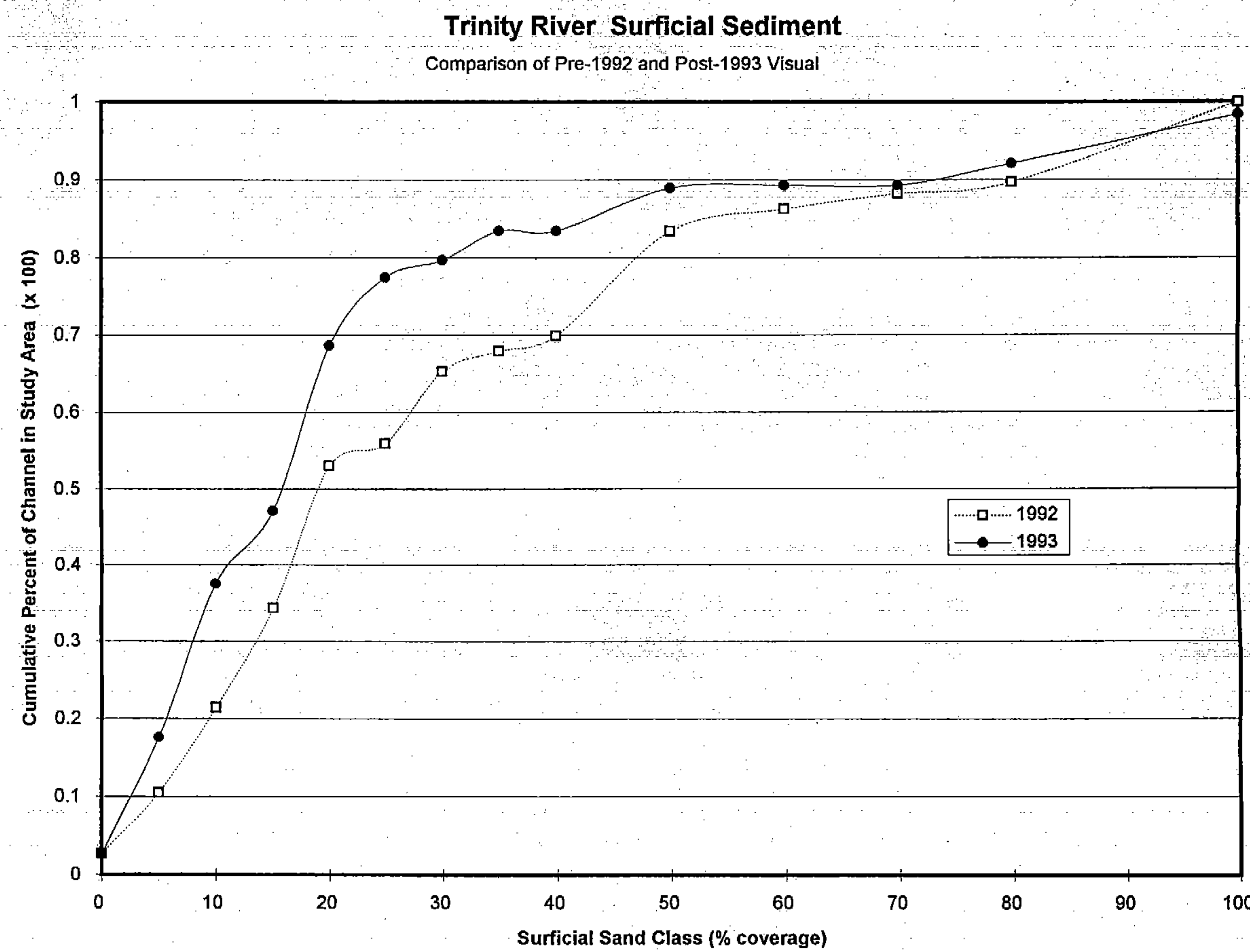
5.5.4. Longitudinal pattern of cut-and-fill in Society Pool in 1991, 1992, and 1993.



5.5.5. Longitudinal pattern of cut-and-fill in Upper Steelbridge Pool in 1992 and 1993.



5.6.1. Frequency of occurrence of classes of fine sediment percentage on the bed of the Trinity River study reach prior to 1992 release and after 1993 release.



5.6.2. Cumulative frequency of occurrence of classes of fine sediment percentages on the bed of the Trinity River prior to the 1992 release and after the 1993 release.

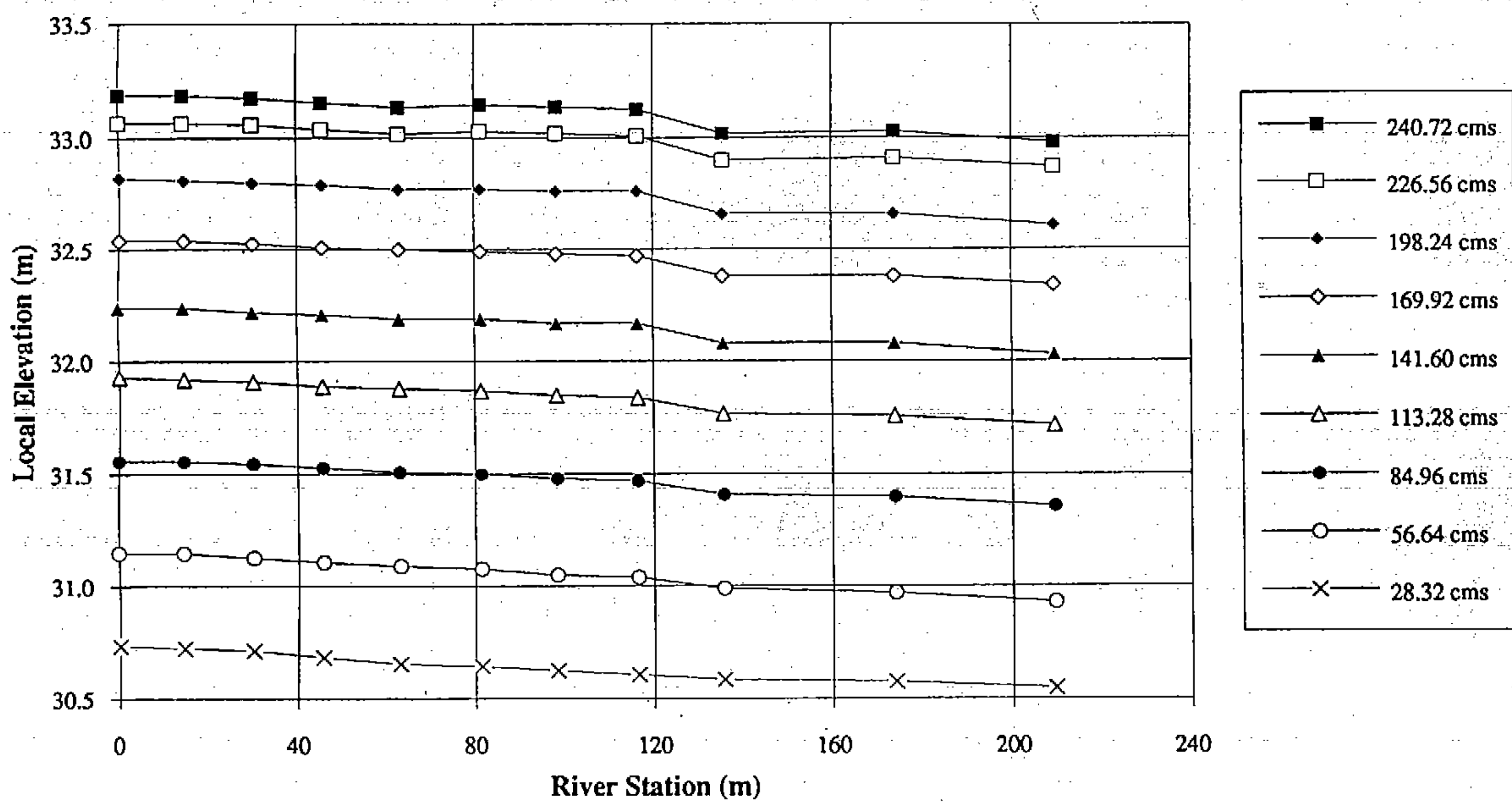


Figure 6.2.1 Water surface profiles as a function of discharge: Poker Bar

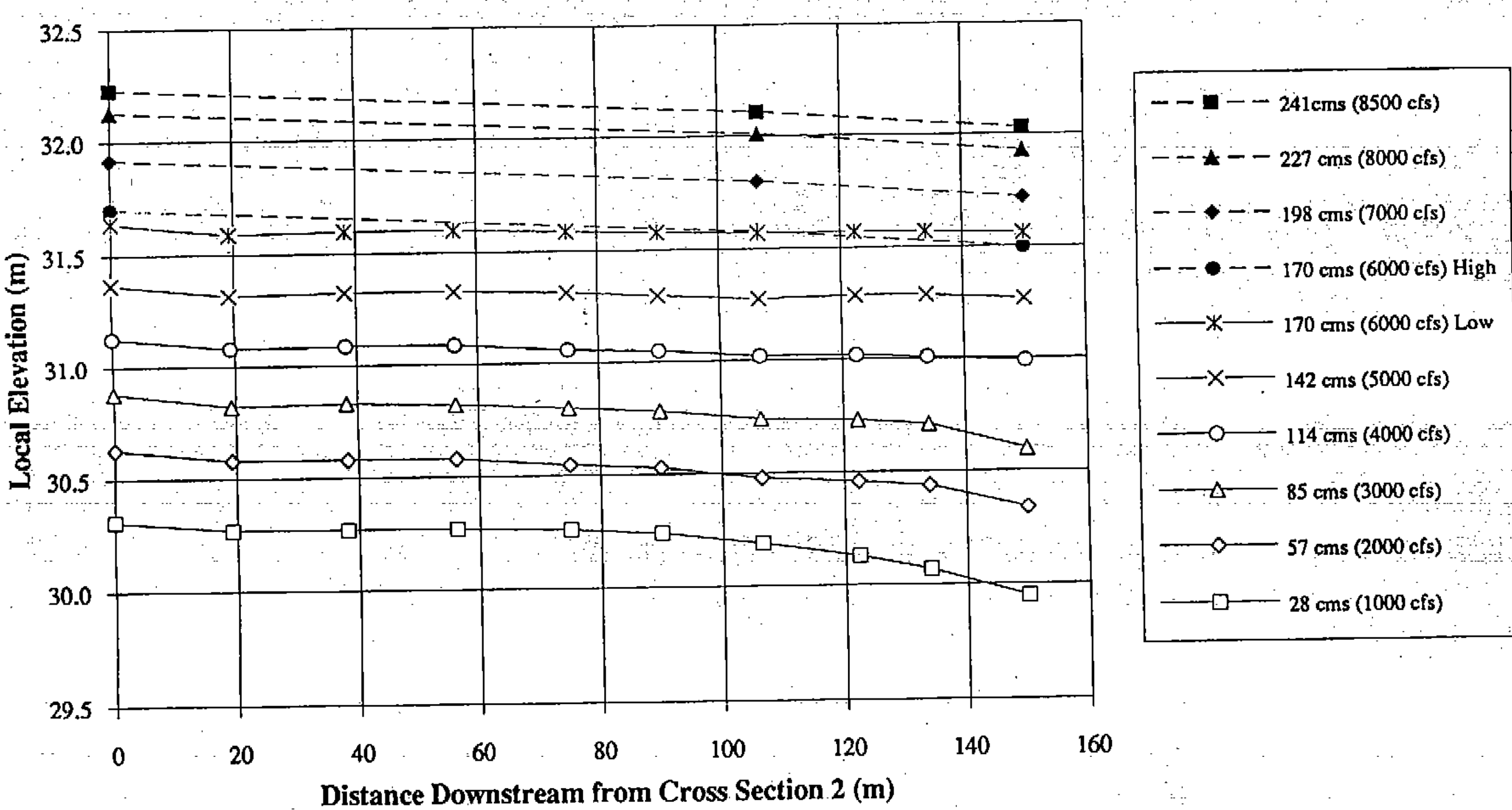


Figure 6.2.2 Water surface profiles as a function of discharge: Steelbridge

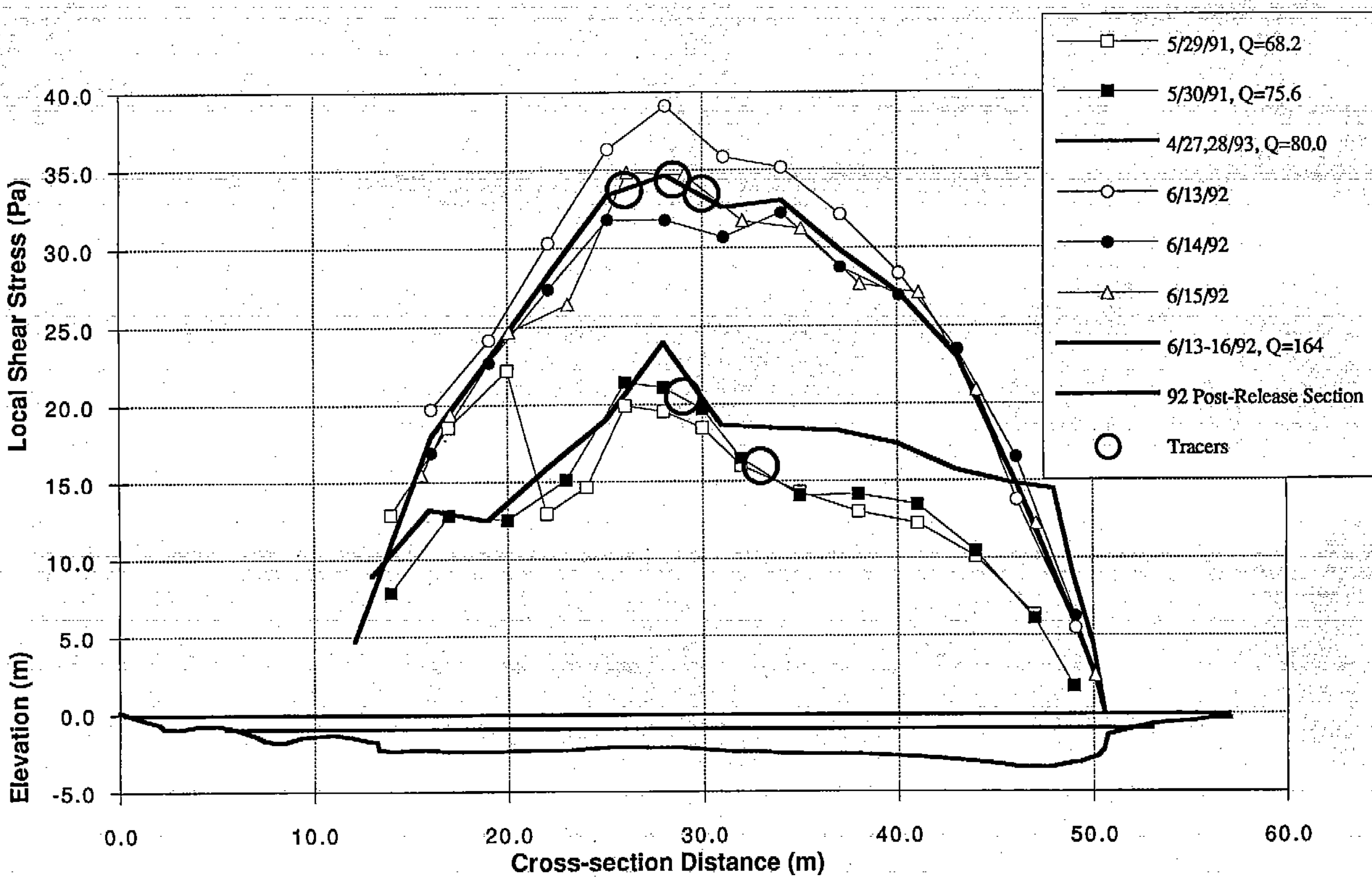


Figure 6.2.3 Local shear stress at Poker Bar Section 2, calculated using the depth averaged velocity, flow depth, and bed grain size in log-law resistance equation

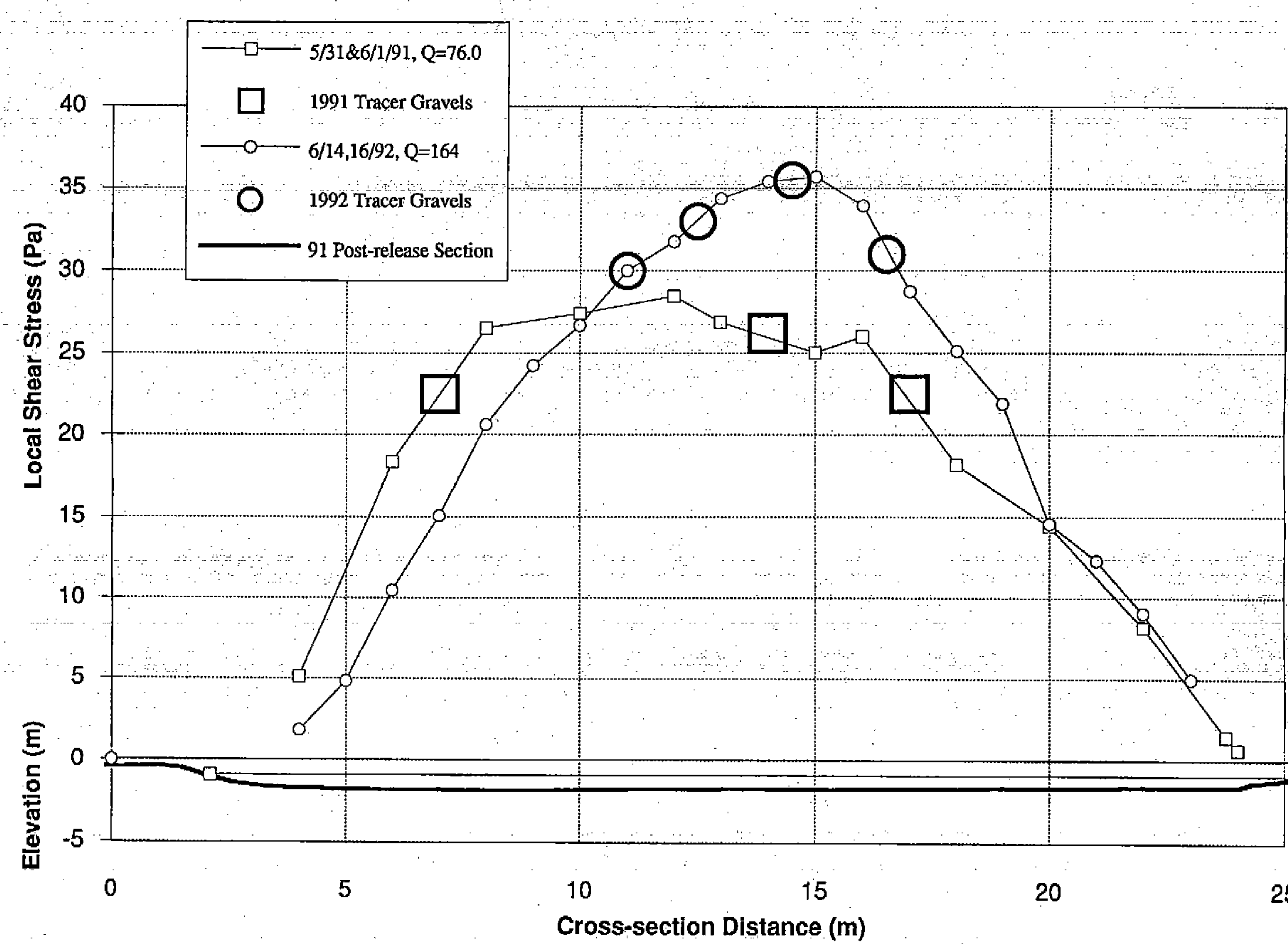


Figure 6.2.4 Local bed shear stress as a function of discharge at Section SB3C

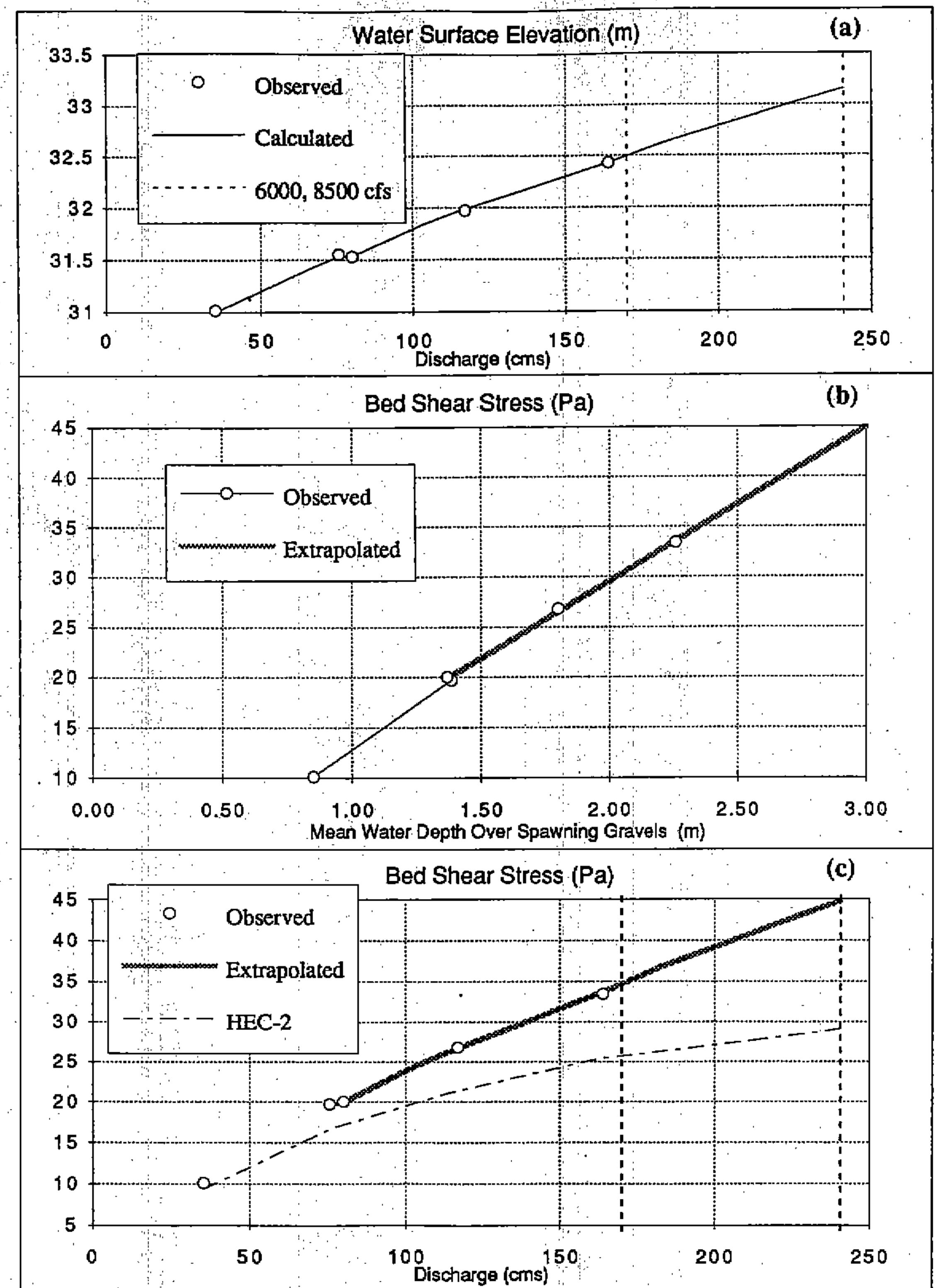


Figure 6.2.5 Rating curve for water surface elevation and bed shear stress over spawning gravels at Poker Bar Section 2. (a) Observed and calculated water surface elevation. (b) Observed bed shear stress over the spawning gravels as a function of observed local flow depth. (c) Observed and predicted bed shear stress over spawning gravels. Extrapolated curve based on depth—shear stress relation in (part (b)) and calculated water surface elevation (part (a)).

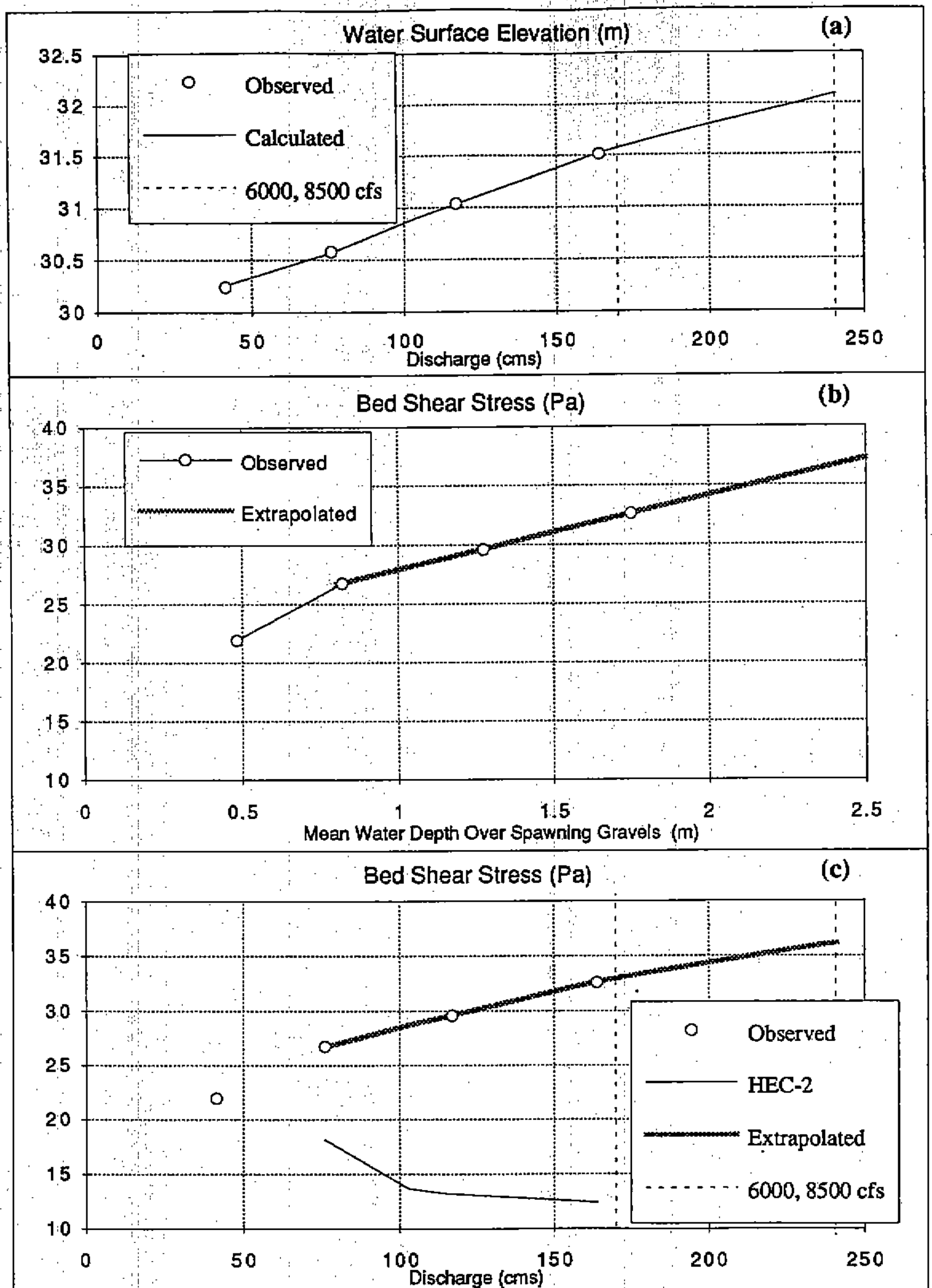


Figure 6.2.6 Rating curve for water surface elevation and bed shear stress over spawning gravels at Steelbridge Section 3C. (a) Observed and calculated water surface elevation. (b) Observed bed shear stress over the spawning gravels as a function of observed local flow depth. (c) Observed and predicted bed shear stress over spawning gravels. Extrapolated curve based on depth—shear stress relation in (part (b)) and calculated water surface elevation (part (a)).

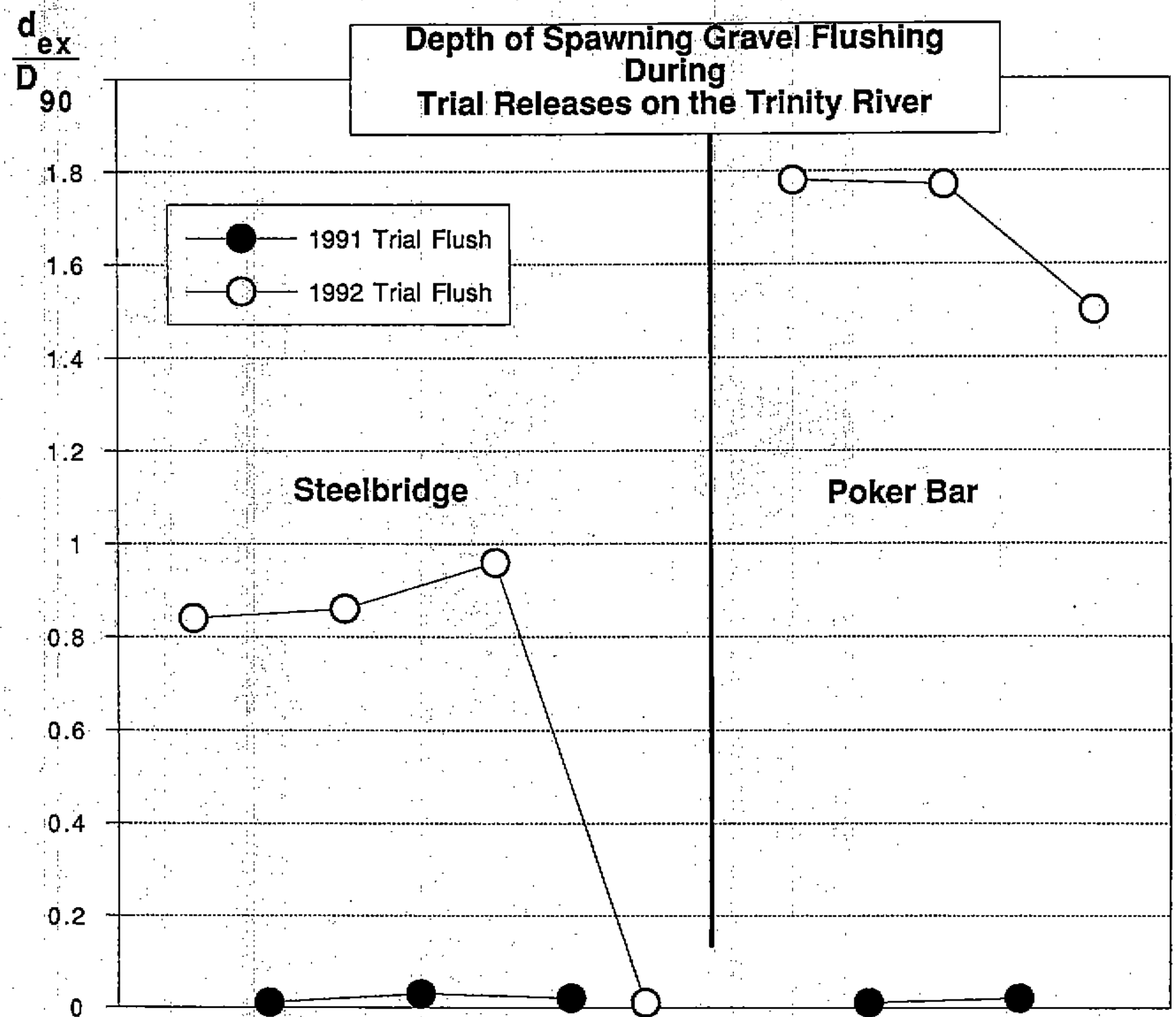


Figure 6.3.1 Depth of Spawning Gravel Flushing During the 1991 and 1992 Trial Releases

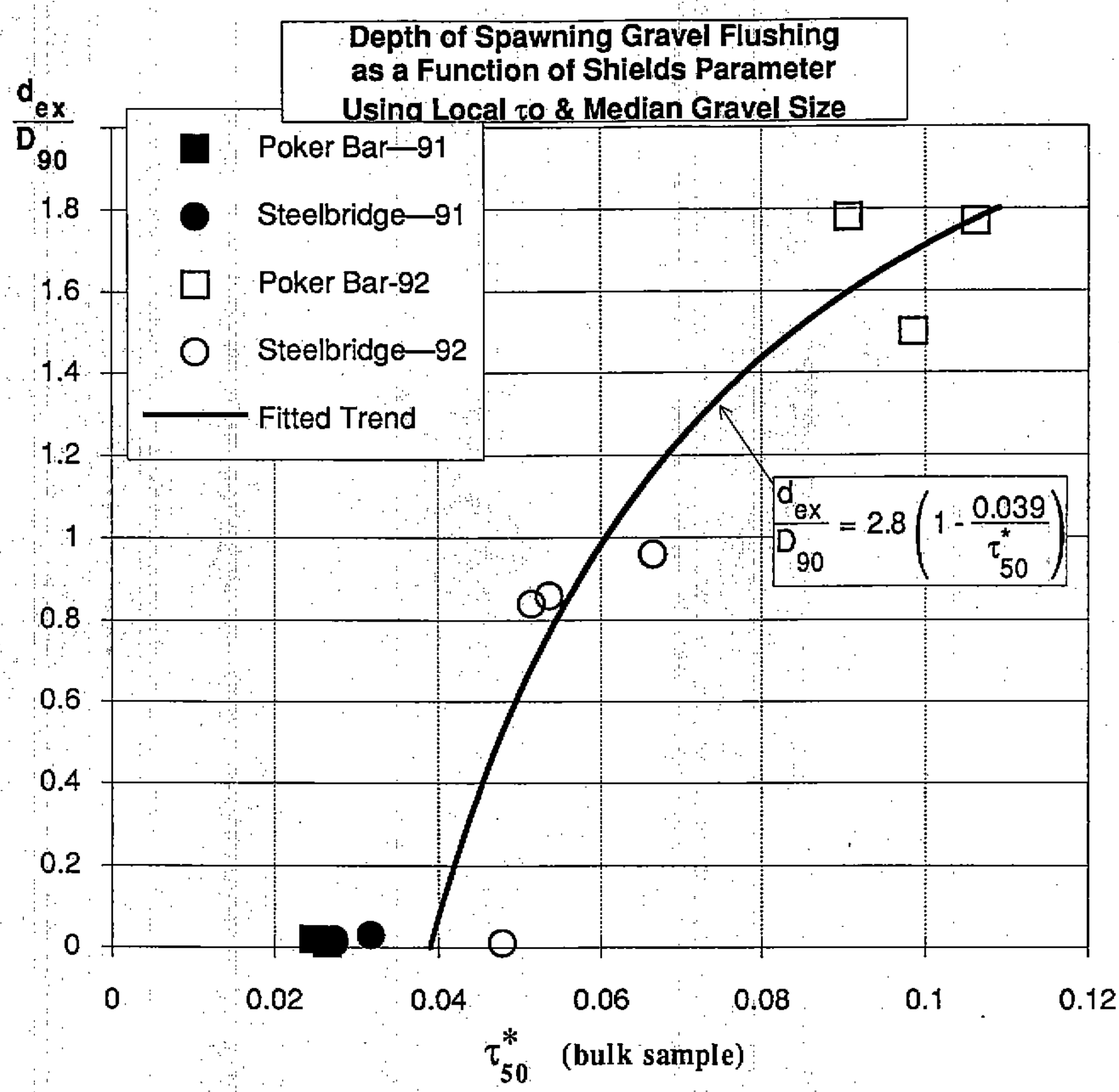


Figure 6.3.2 Depth of Spawning Gravel Flushing as a Function of Shields Parameter

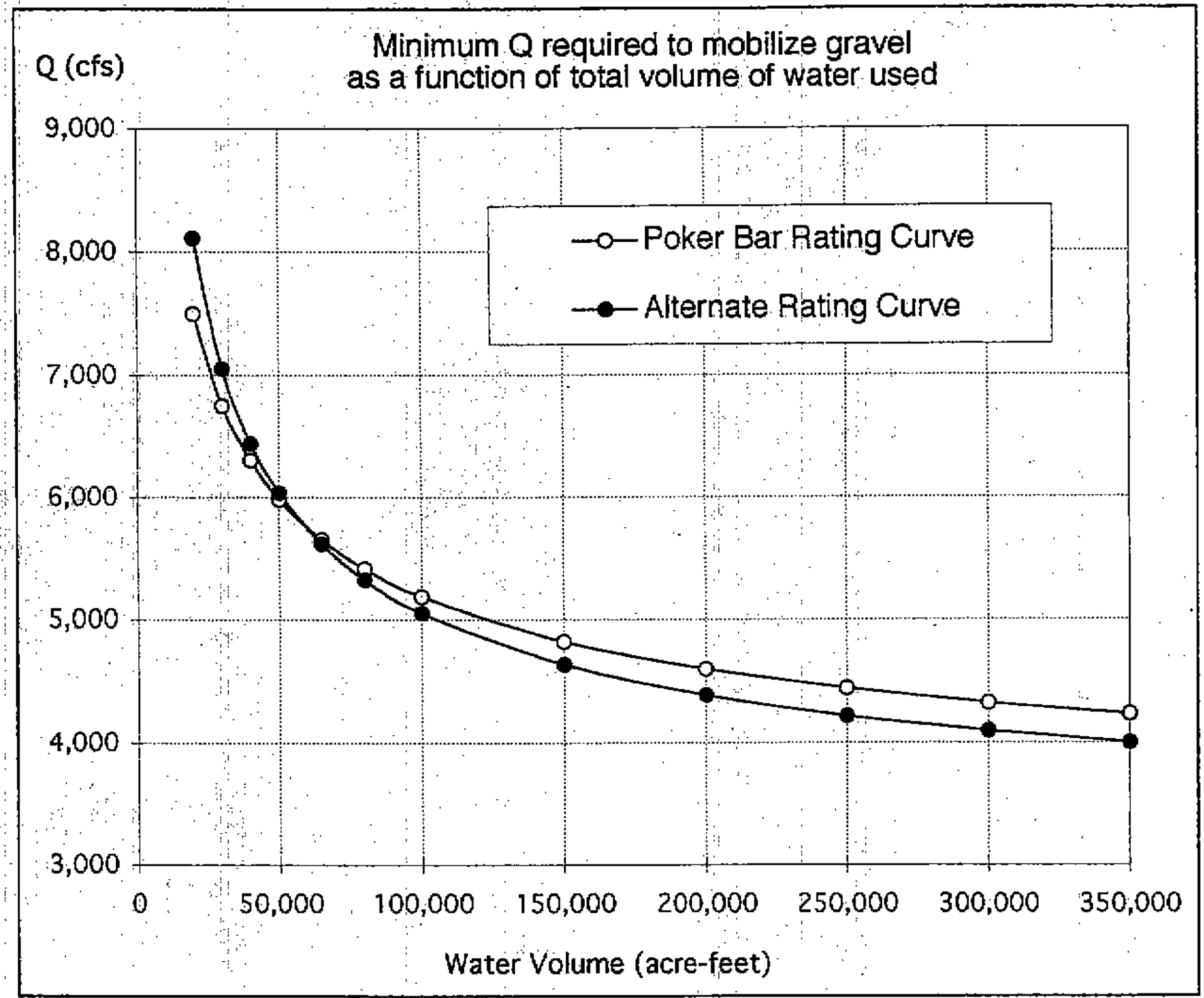


Figure 6.3.3 Minimum discharge required to mobilize gravel surface as a function of water used in a flushing release

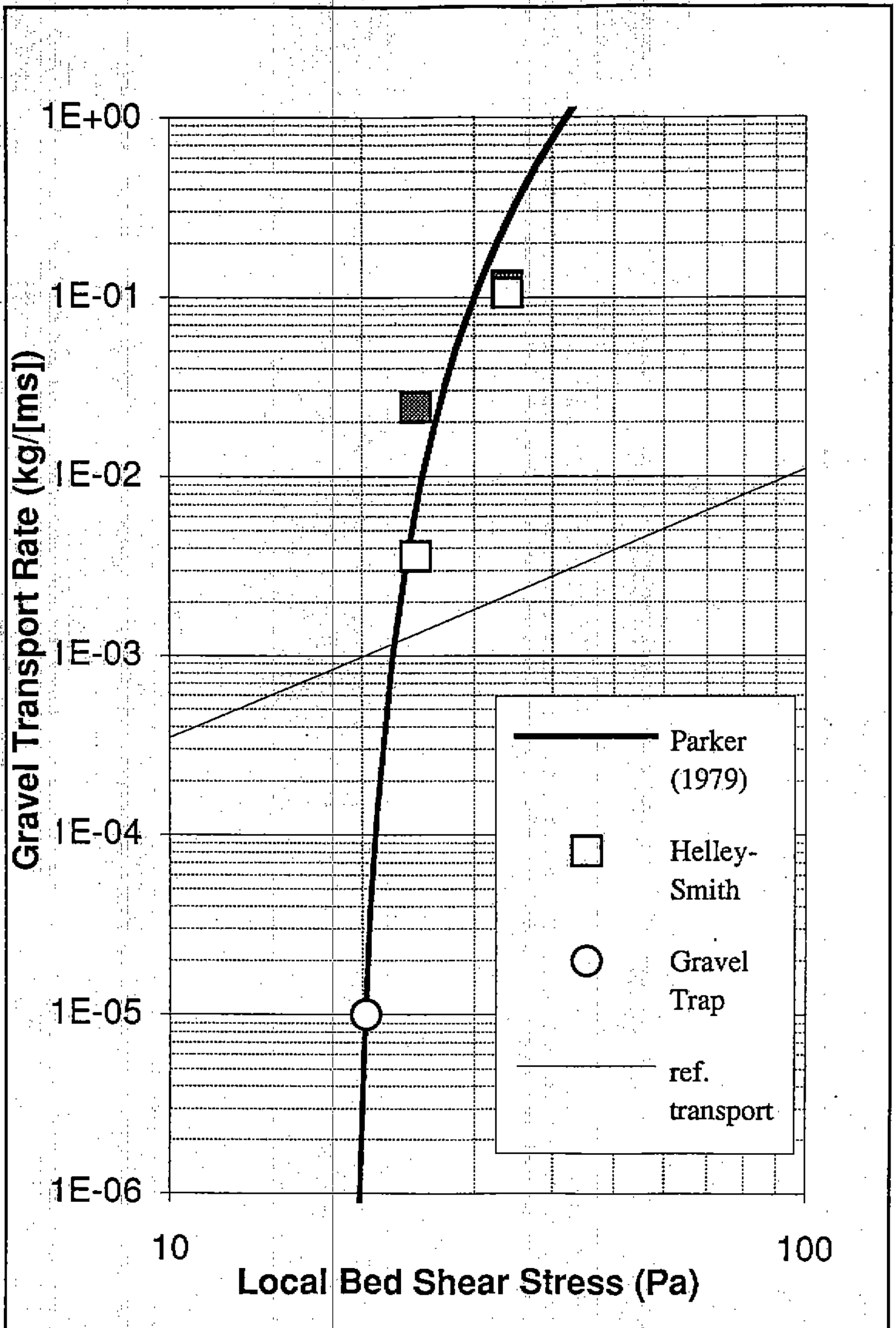


Figure 6.3.4 Gravel transport rate as a function of local bed shear stress at Section PB2

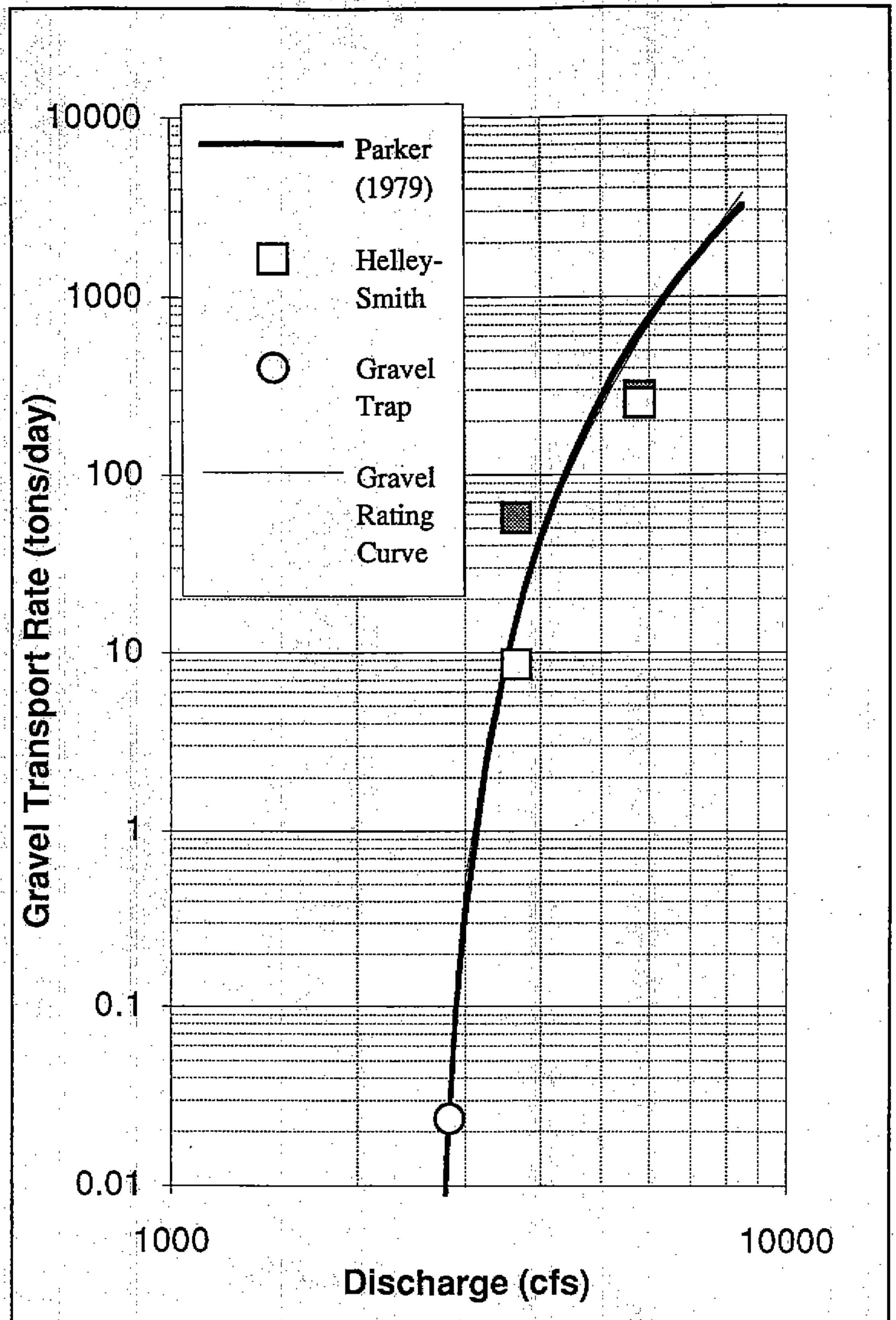


Figure 6.3.5 Gravel transport rate as a function of discharge at Section PB2

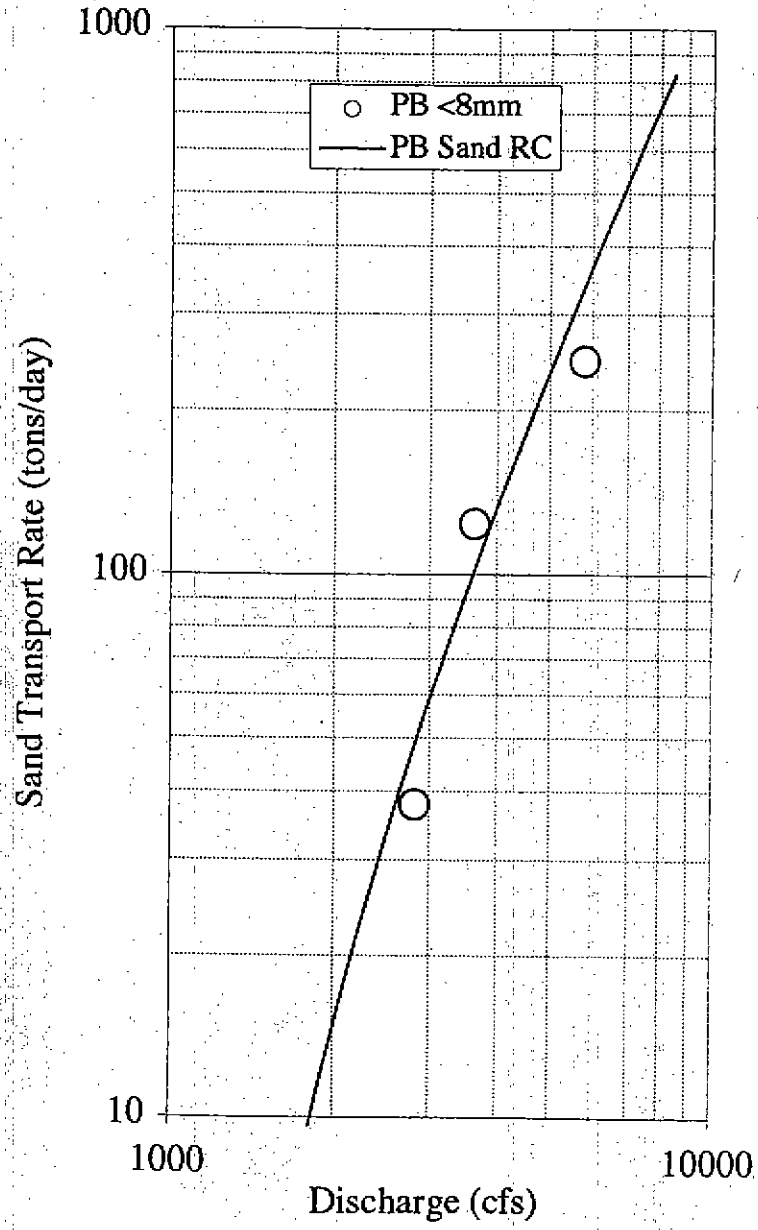


Figure 6.3.6 Sand transport rate as a function of discharge at Section PB2

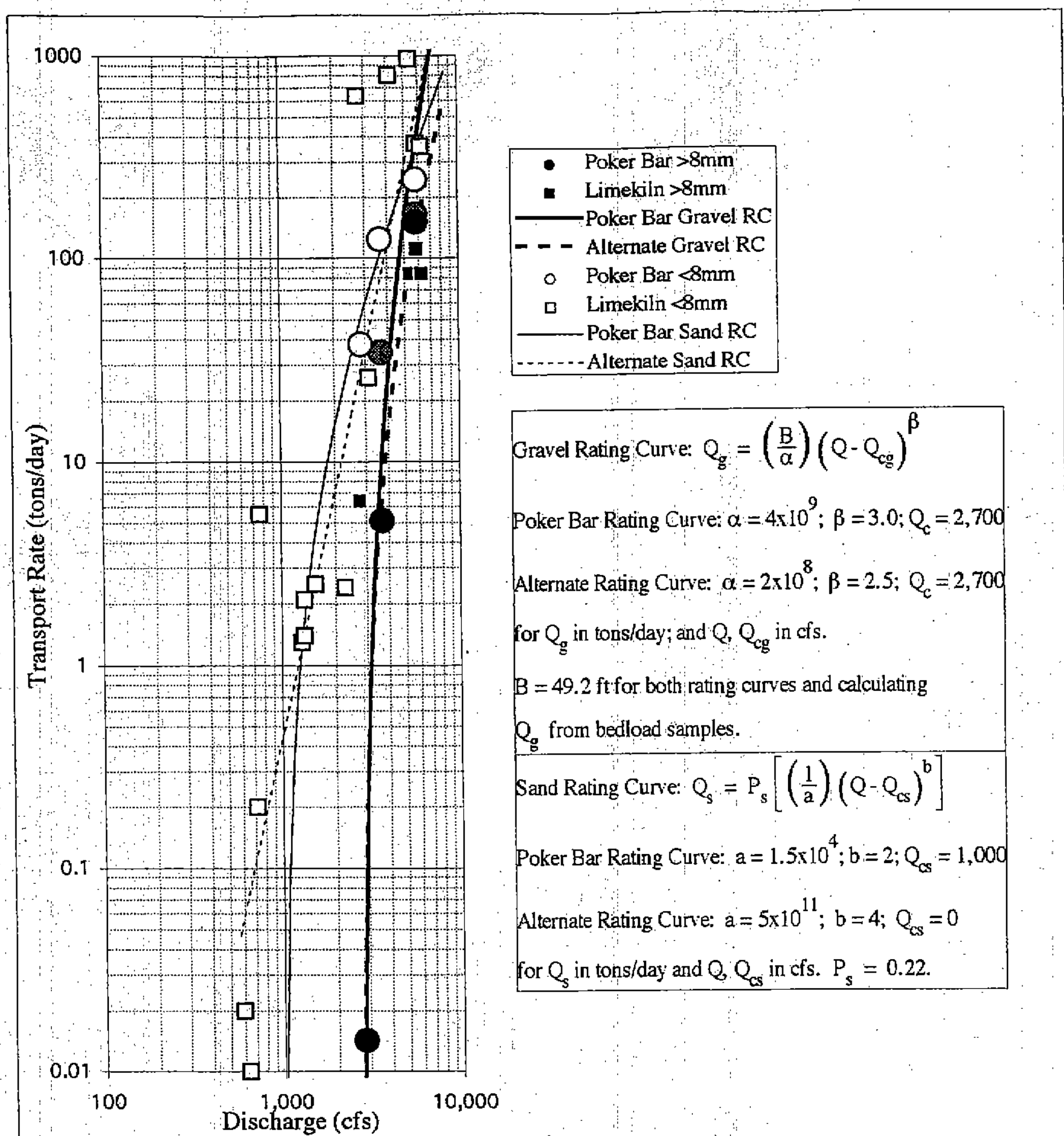


Figure 6.3.7 Sand and gravel transport as a function of discharge at Section PB2 and USGS gage below Limekiln Gulch. Field samples shown with symbols; sediment rating curves (RC) shown with lines.

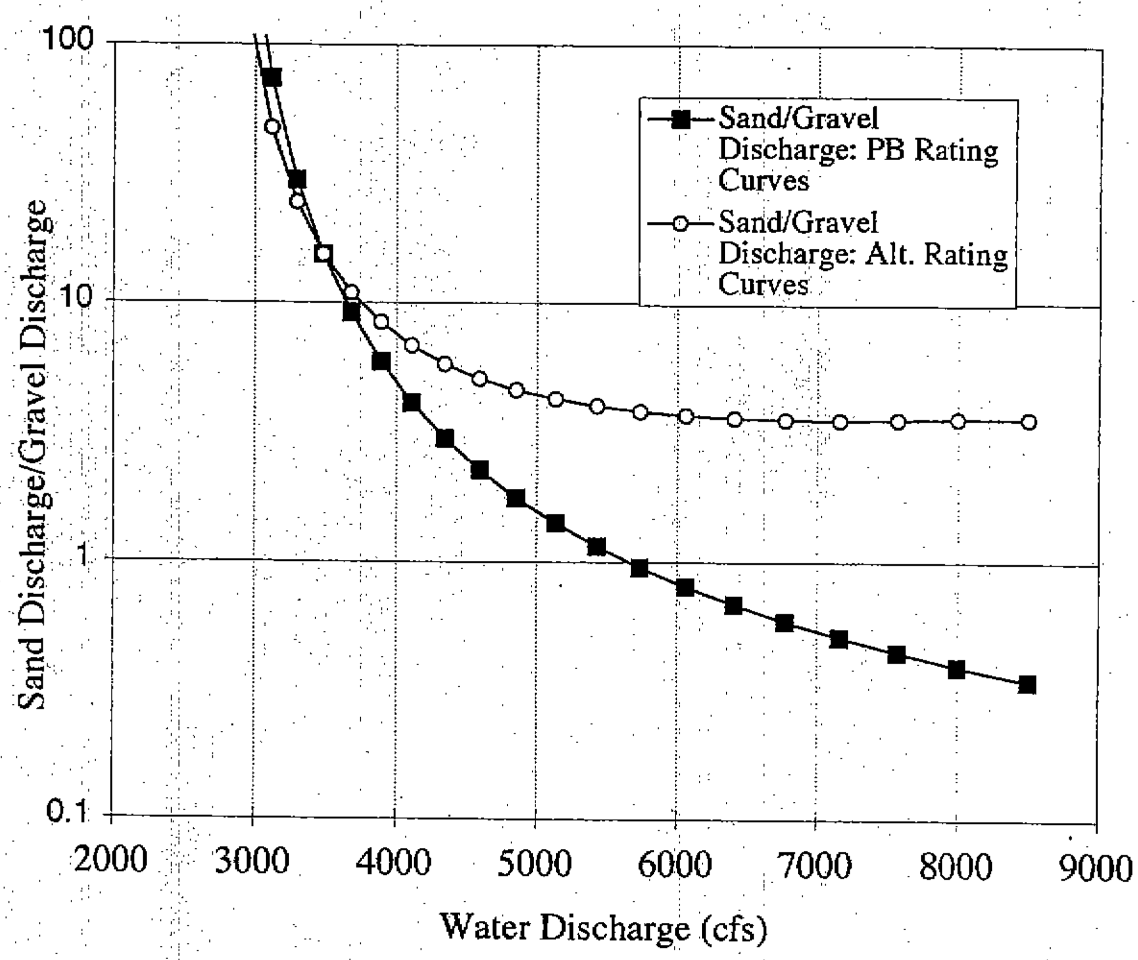


Figure 6.3.8 Ratio of sand to gravel transport rates for Poker Bar and Alternate Rating Curves

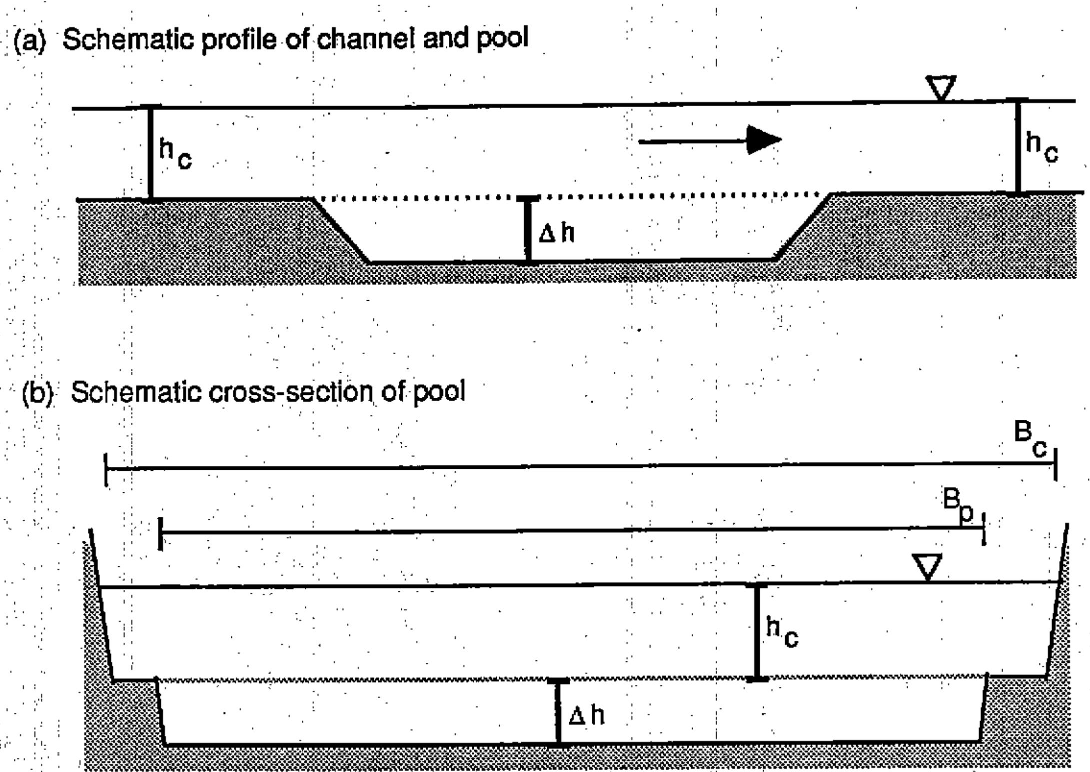


Figure 6.4.1 Definition Sketch for Dredge Pool Geometry

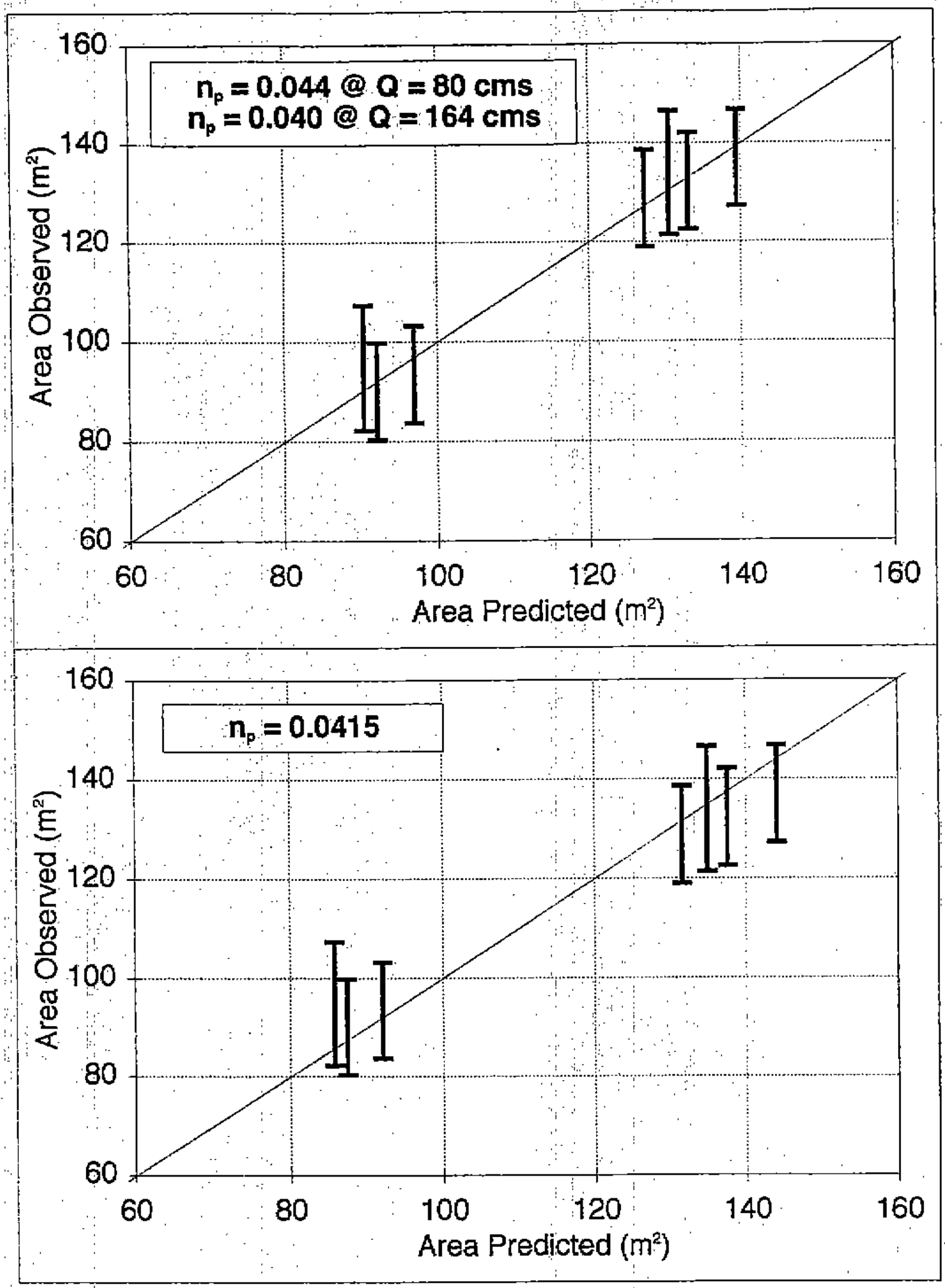


Figure 6.4.2 Pool cross-sectional area: comparison of observed during 1992 and 1993 flushing releases and predicted using the stable depth calculation method

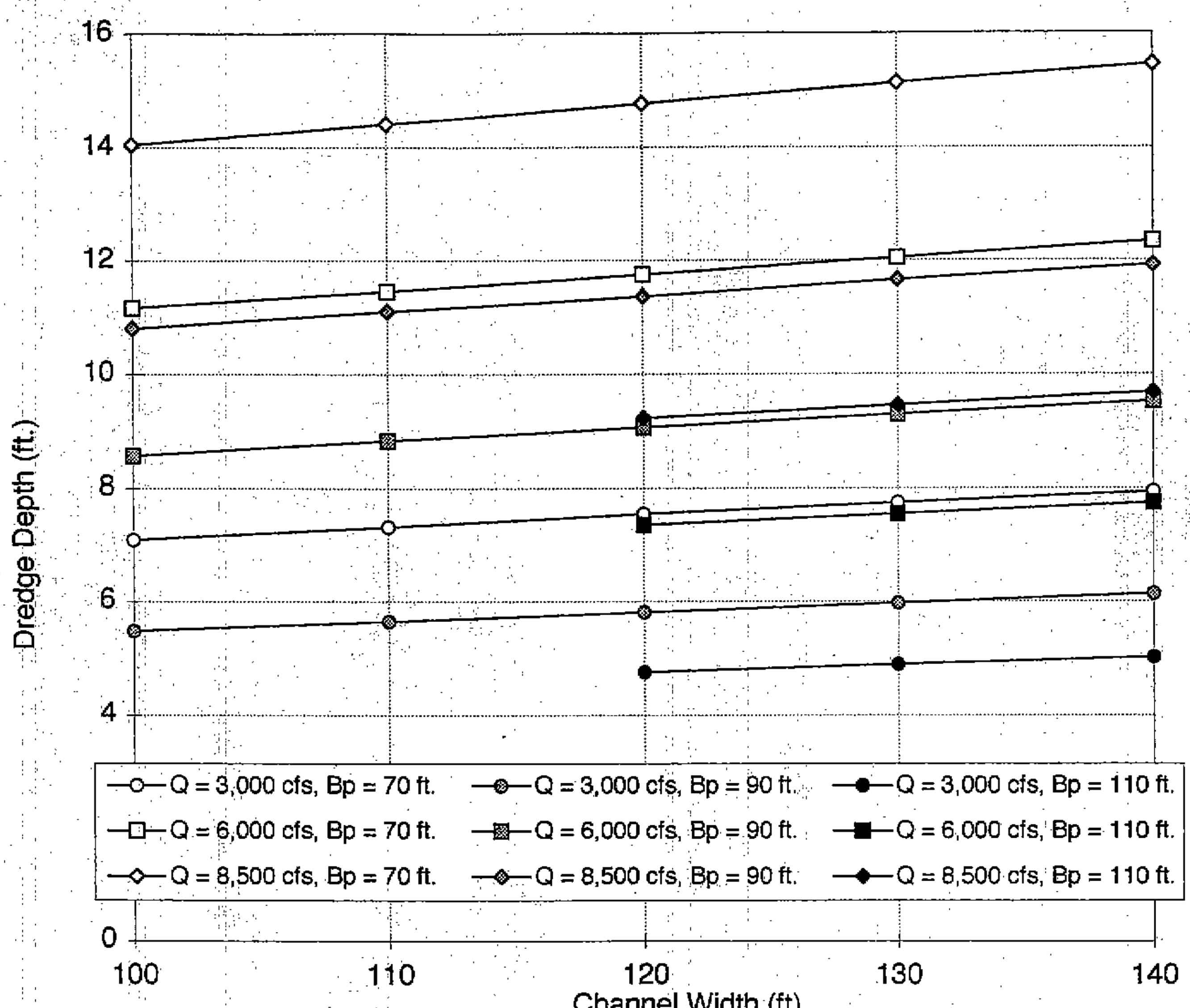


Figure 6.4.3 Variation of stable pool depth with channel width, pool width, and discharge. Calculations use Manning's n = 0.03 for the channel and n = 0.0415 for pool.

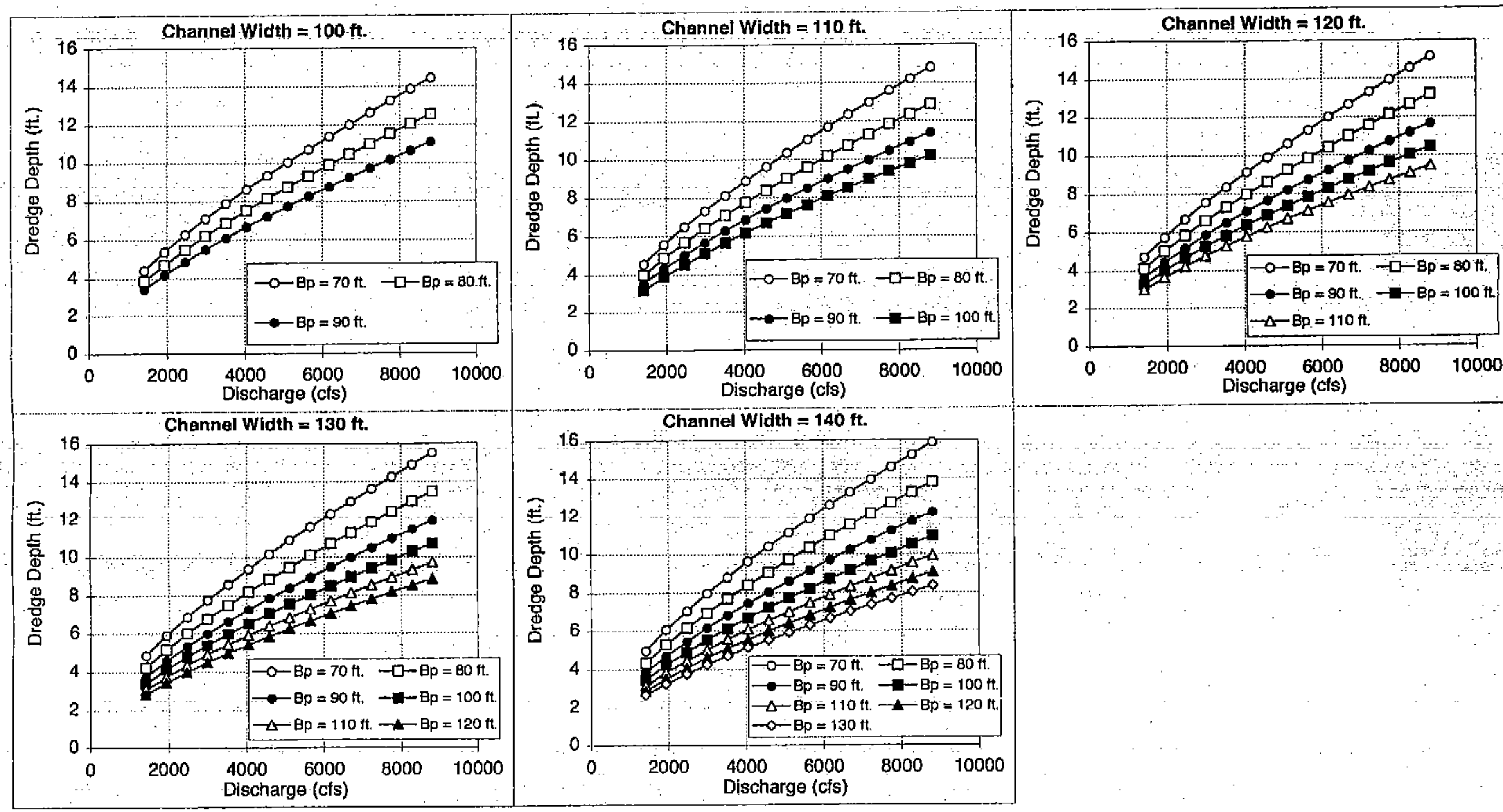


Figure 6.4.4 Variation of stable pool depth with channel width, pool width, and discharge. Calculations use Manning's n = 0.03 for channel and n = 0.0415 for pool

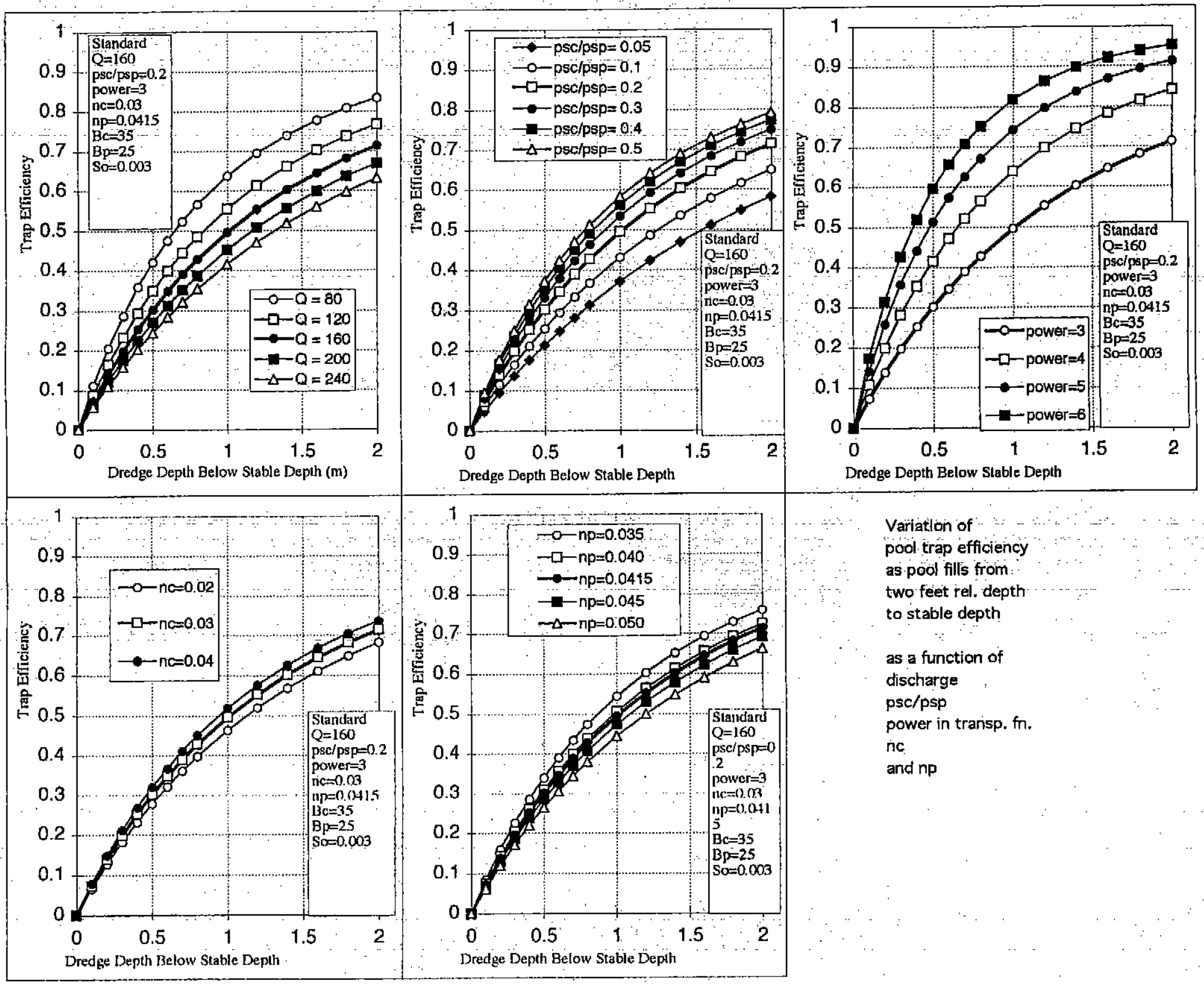


Figure 6.4.5 Variation of pool trap efficiency with dredging depth, discharge, sand concentration, channel roughness and pool roughess.

Q for ∂t=T, 1/29/95

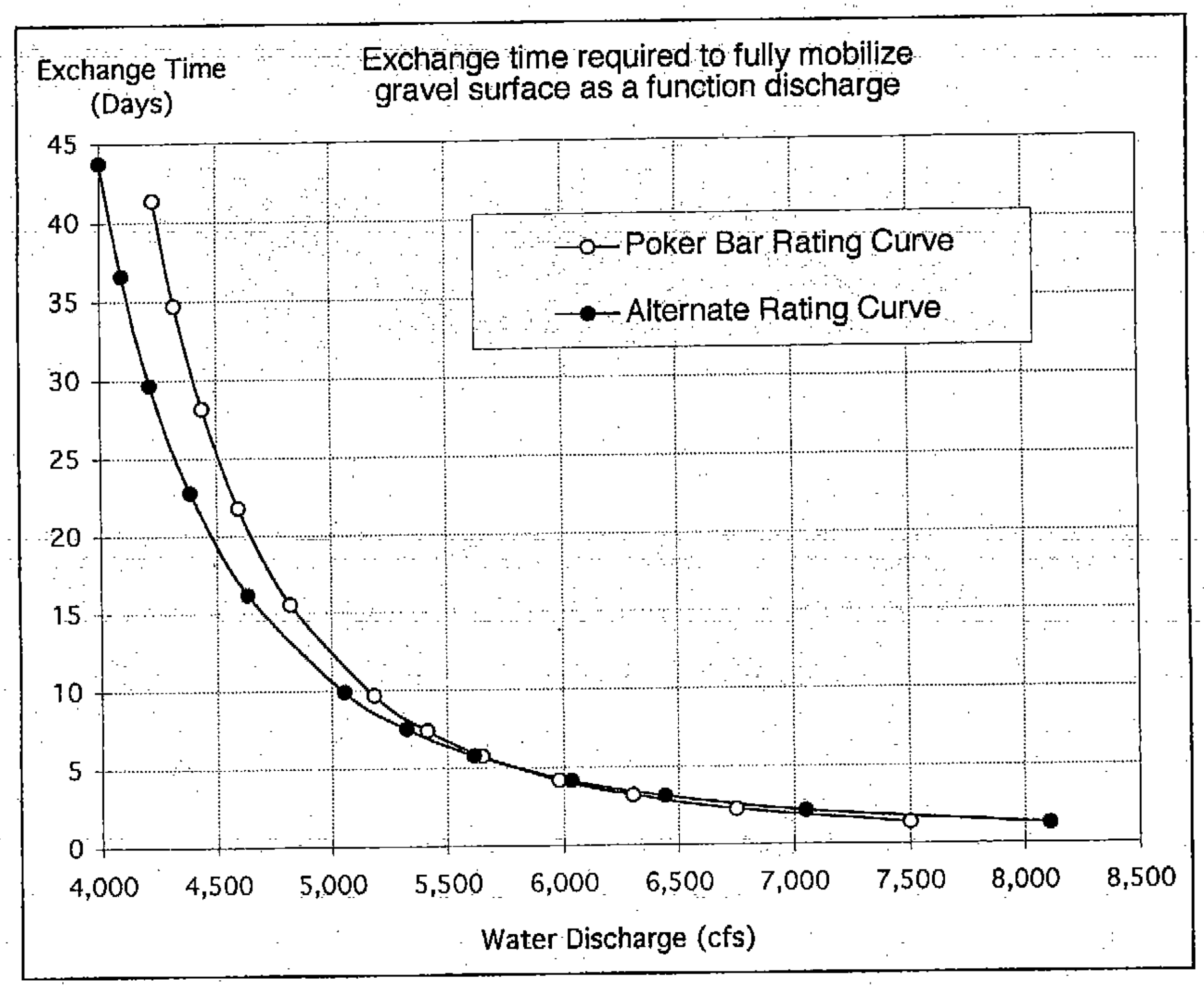


Figure 6.5.1 Exchange time for bed surface mobilization as a function of discharge

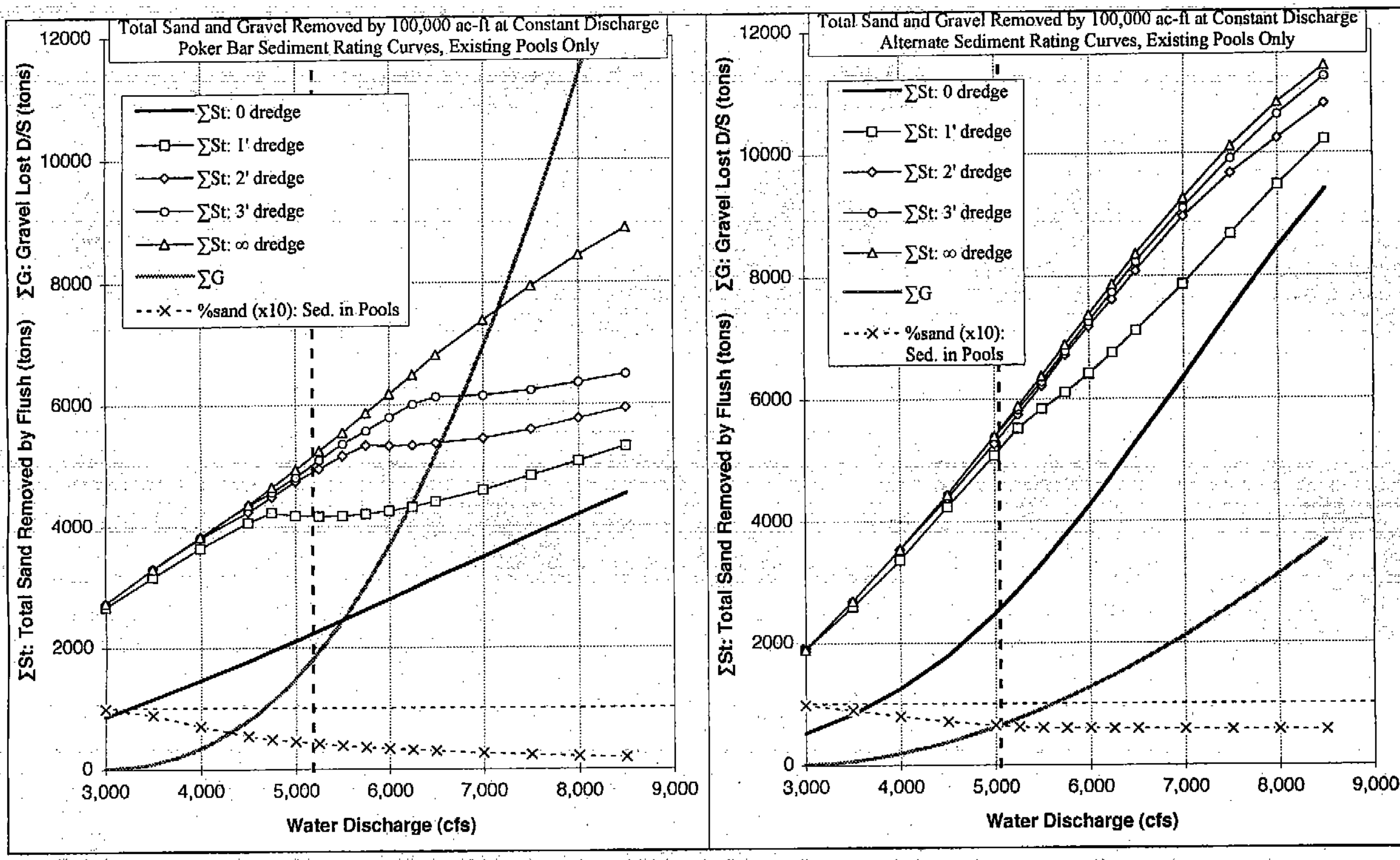


Figure 6.5.2 Sand and gravel removed by 100,000 acre-ft of water, as a function of discharge and dredge depth in existing pools. A trap efficiency of 0.8 is assumed for pools with available storage volume.

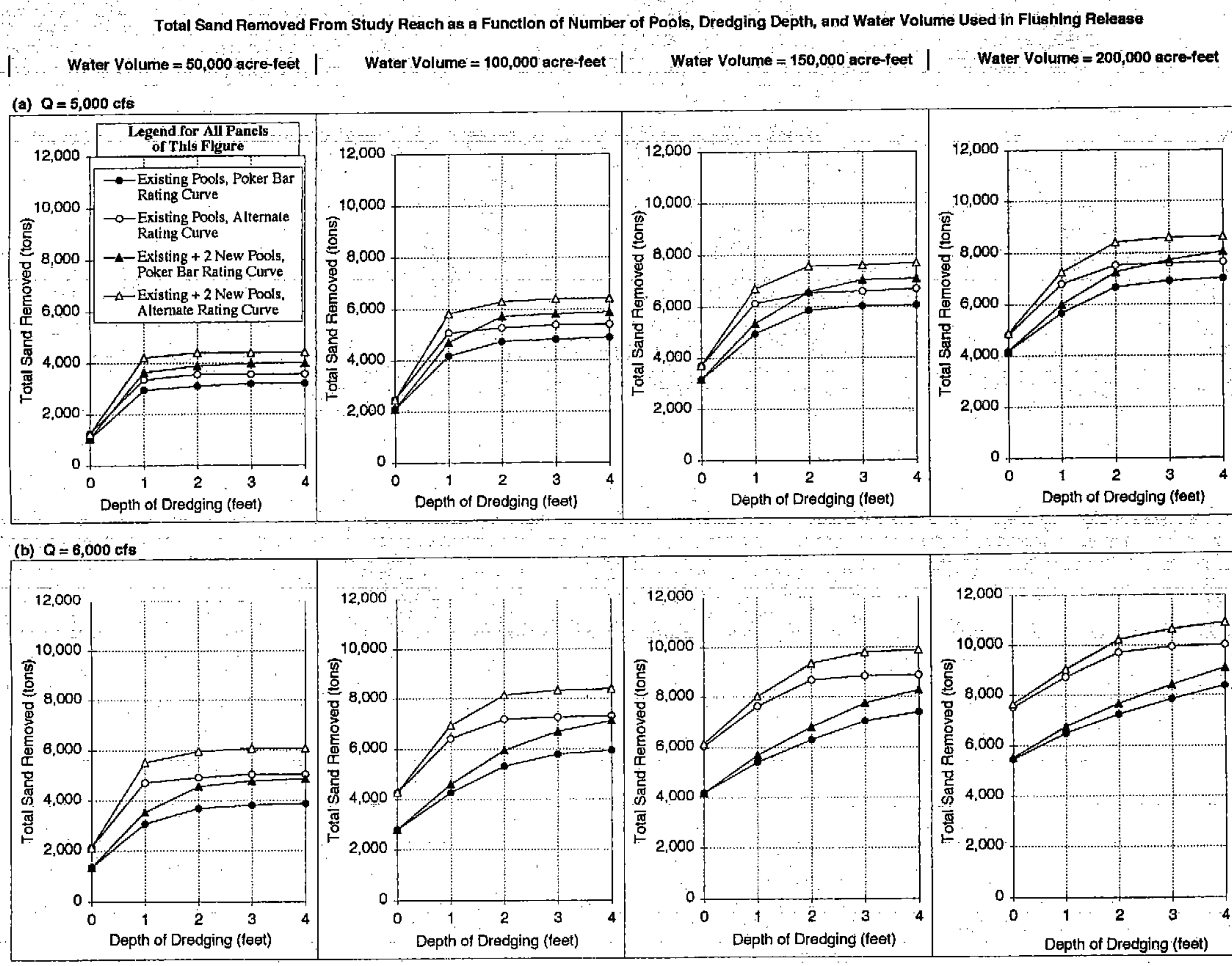


Figure 6.5.3 Total sand removed from study reach as a function of number of dredged pools and dredging depth for Q=5,000 cfs and Q=6,000 cfs.

Same key for all panels of figure. PB RC = Poker Bar sediment rating curve. Alt. RC = Alternate sediment rating curve.

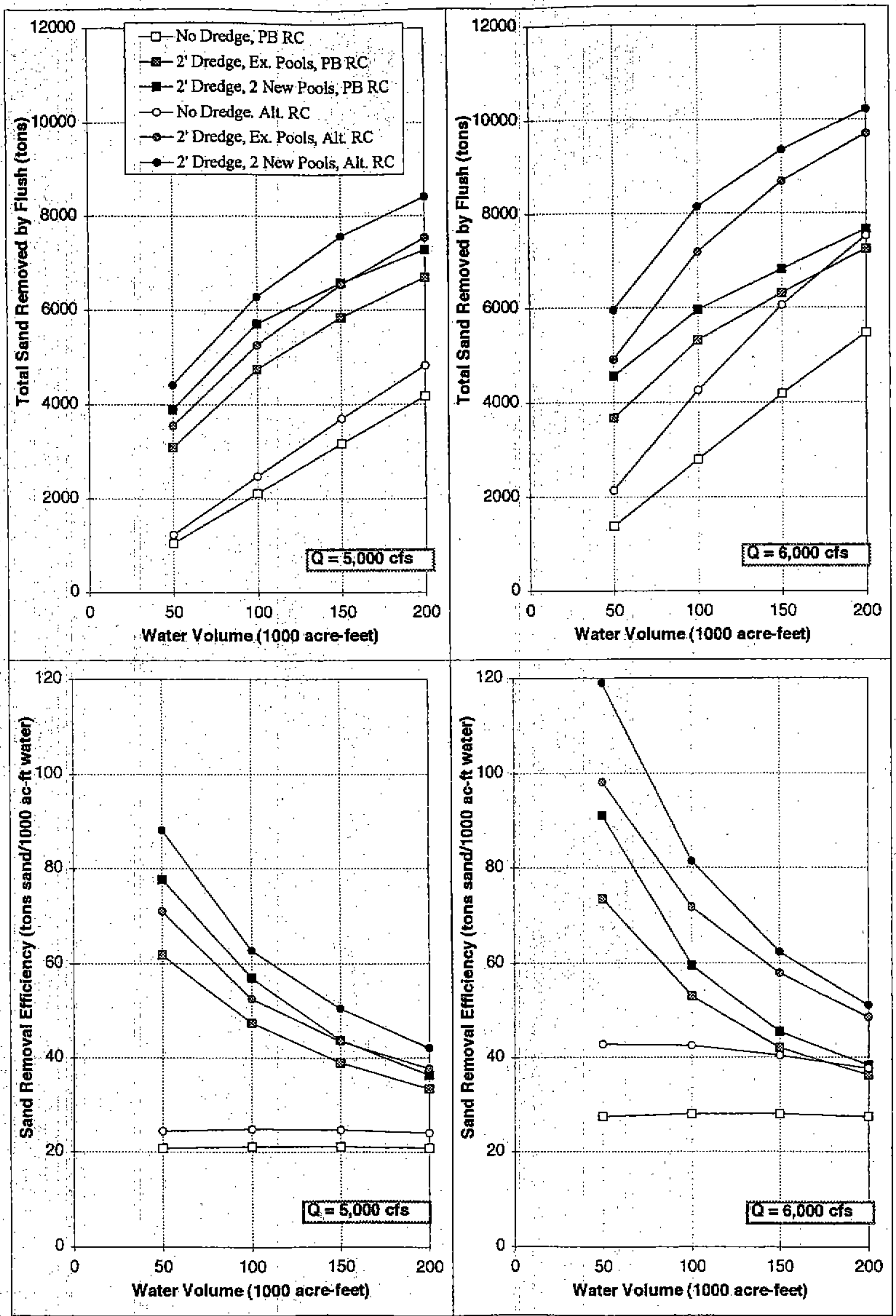


Figure 6.5.4 Total sand removed from study reach as a function of water volume for Q=5,000 cfs and Q=6,000 cfs.

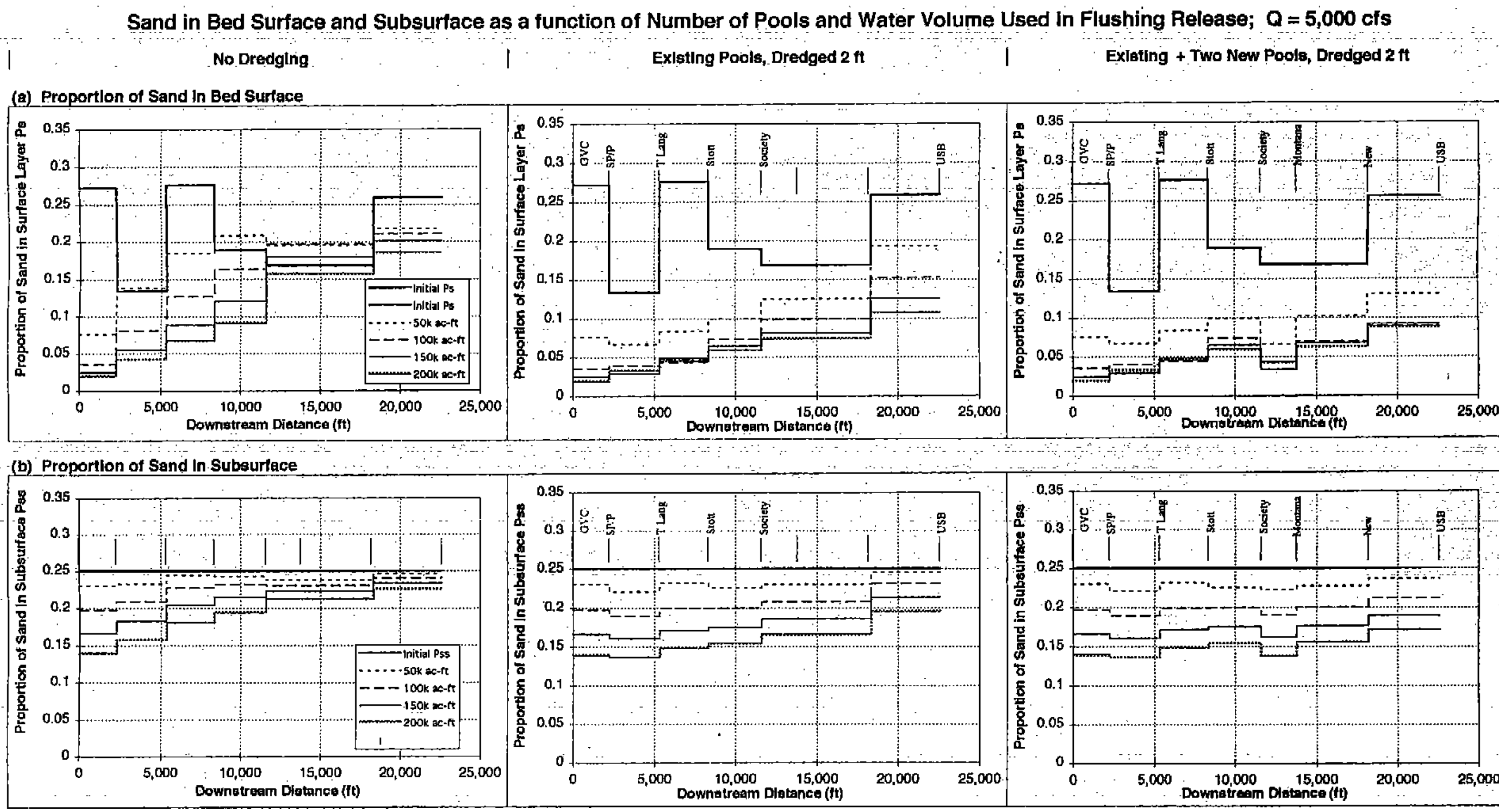


Figure 6.5.5 Sand in the bed surface and subsurface before and after release at Q=5,000 cfs.

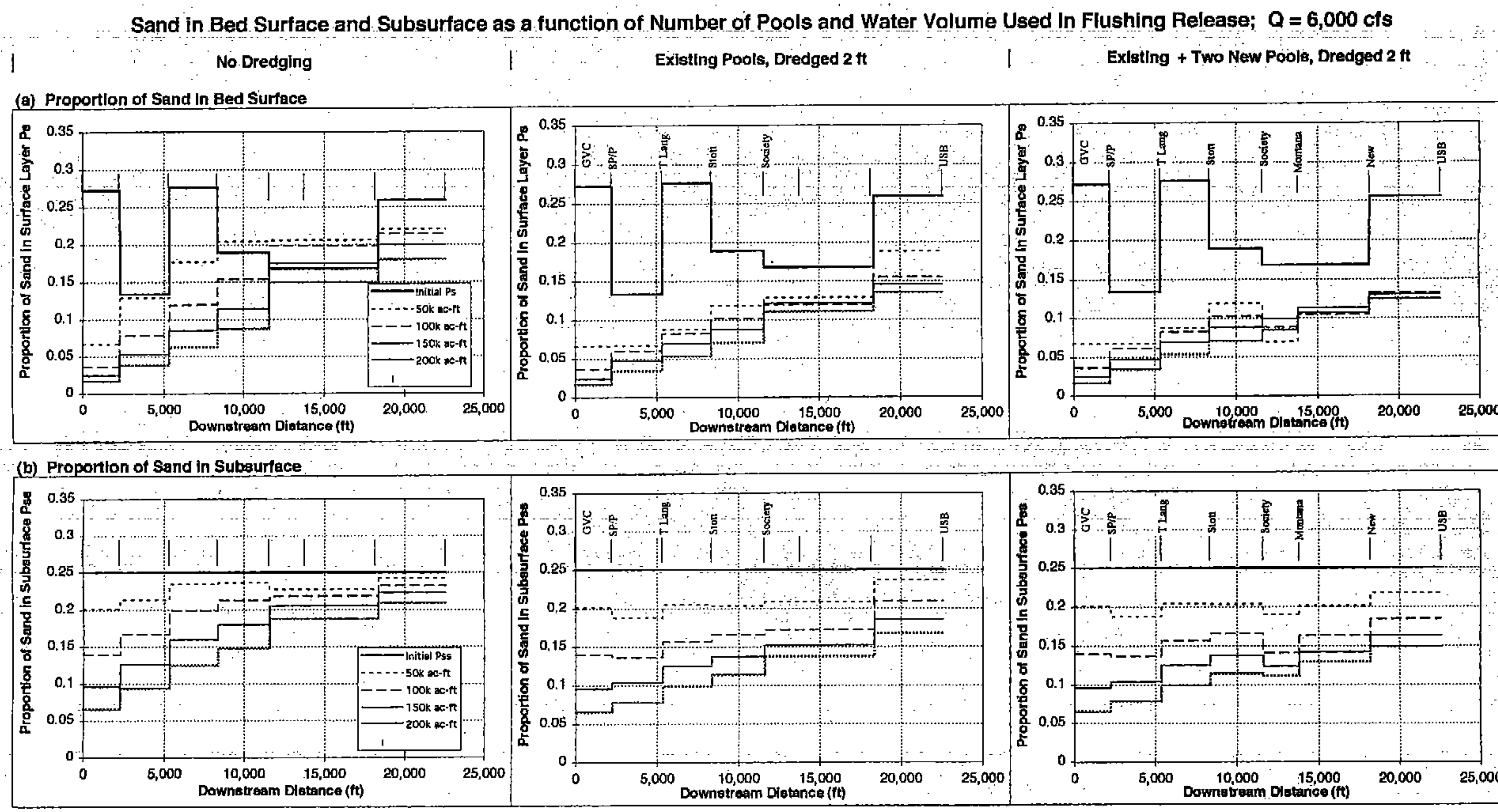


Figure 6.5.6 Sand in the bed surface and subsurface before and after release at Q=6,000 cfs.

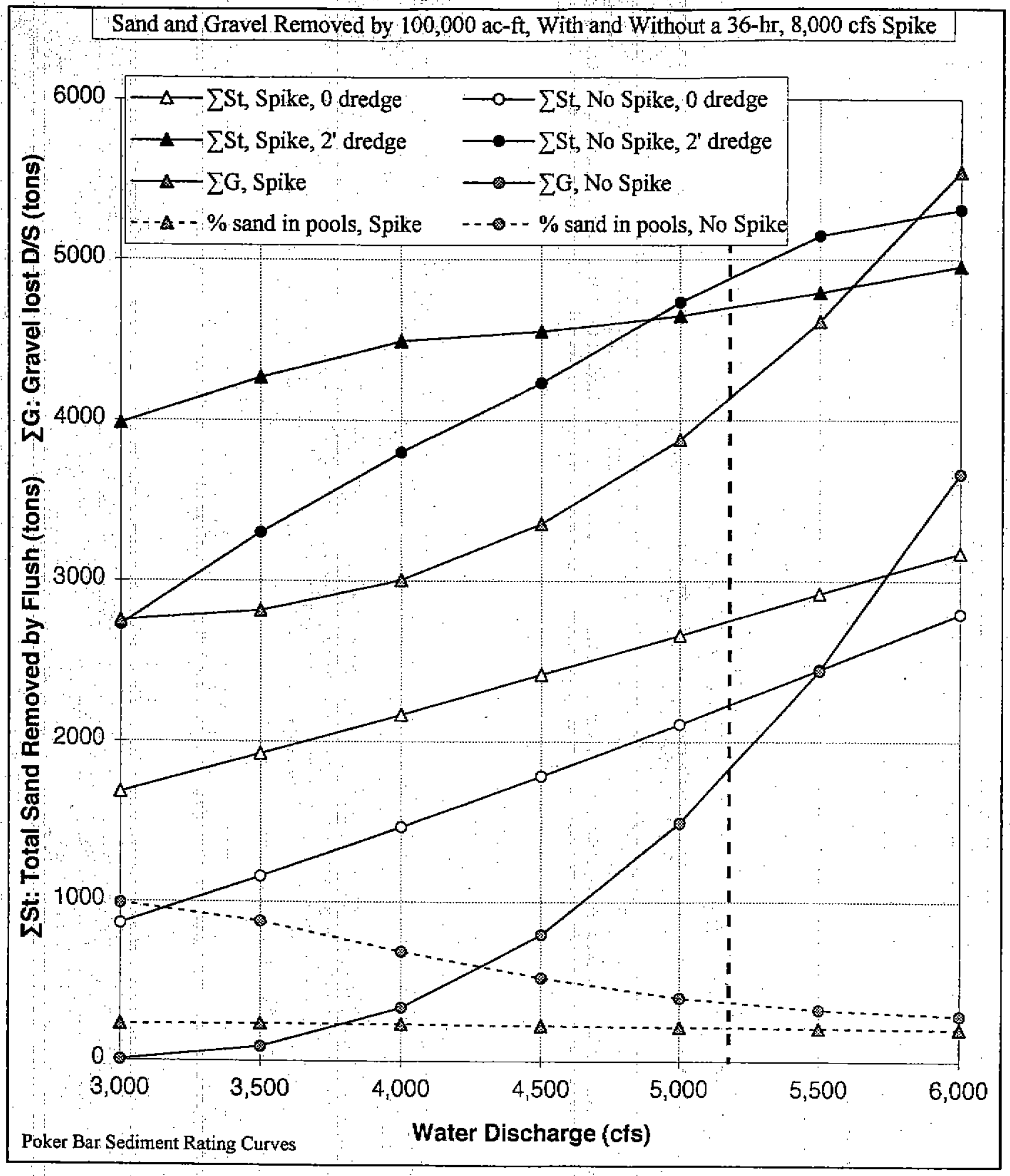


Figure 6.5.7 Sand and gravel removed by 100,000 acre-ft, with and without a 36-hr, 8,000 cfs spike.

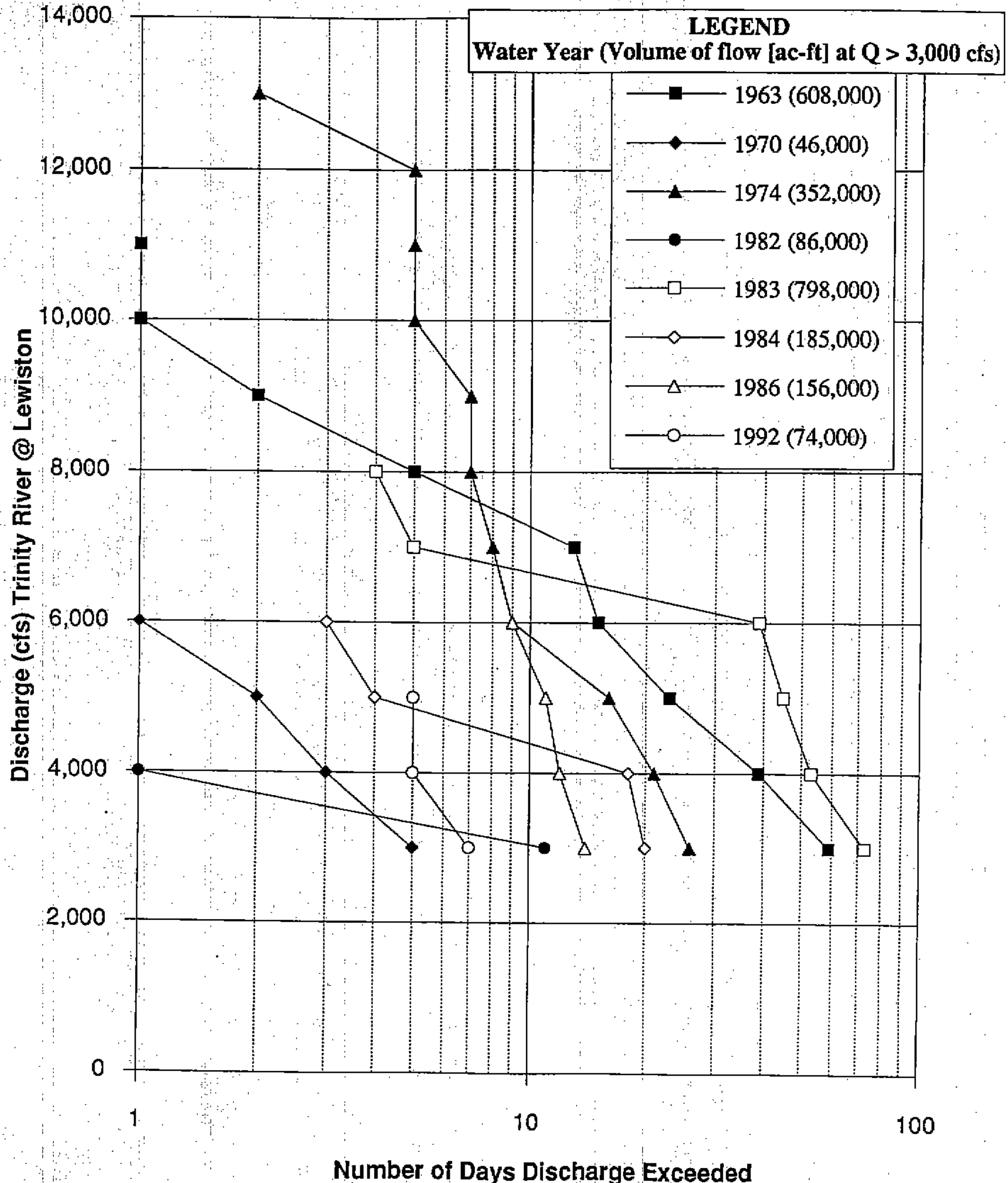


Figure 6.6.1 High discharges for the Trinity River at Lewiston for the period 1960-1994.

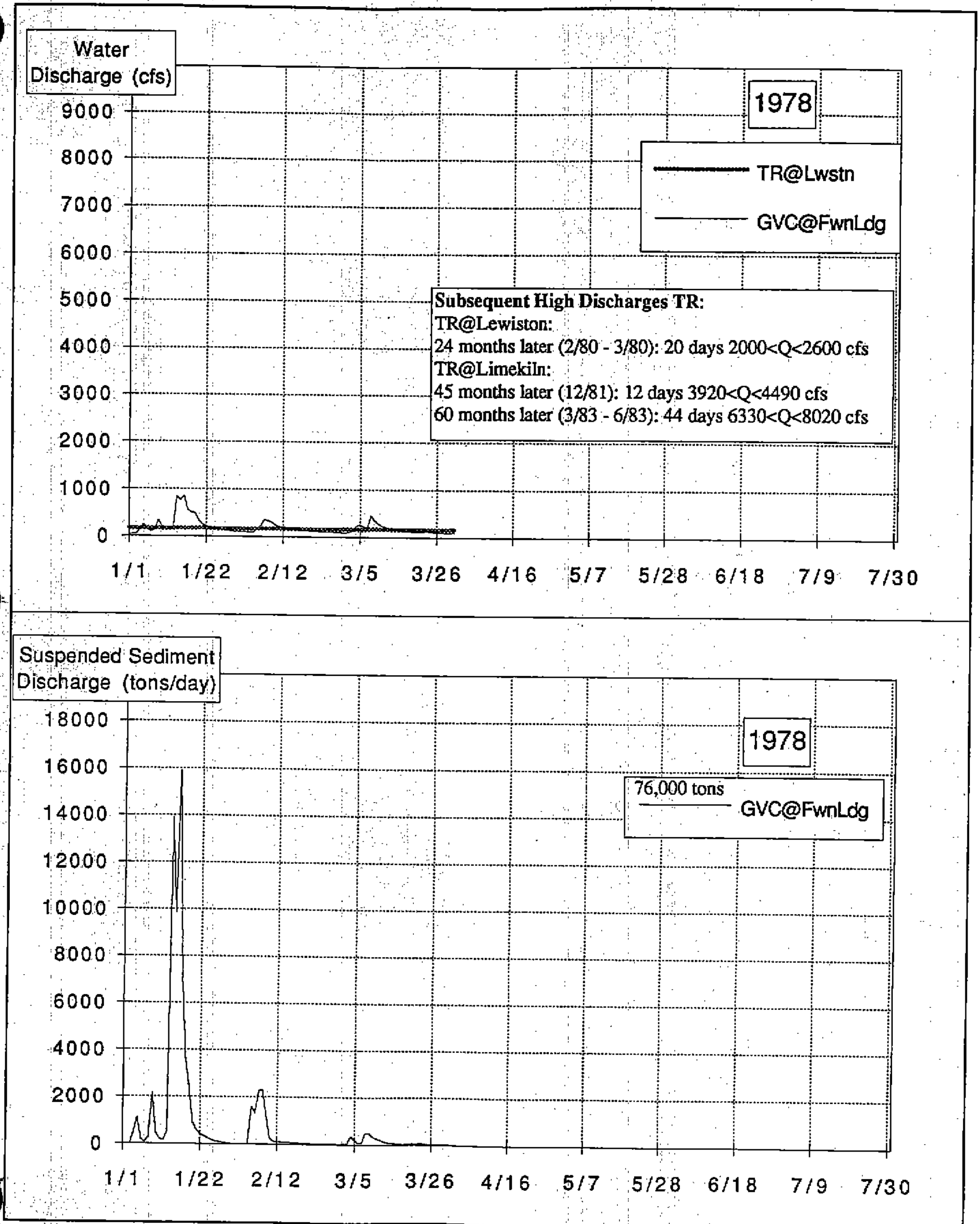


Figure 6.6.2 Water and sediment discharge for the Trinity River and Grass Valley Creek: January-March, 1978

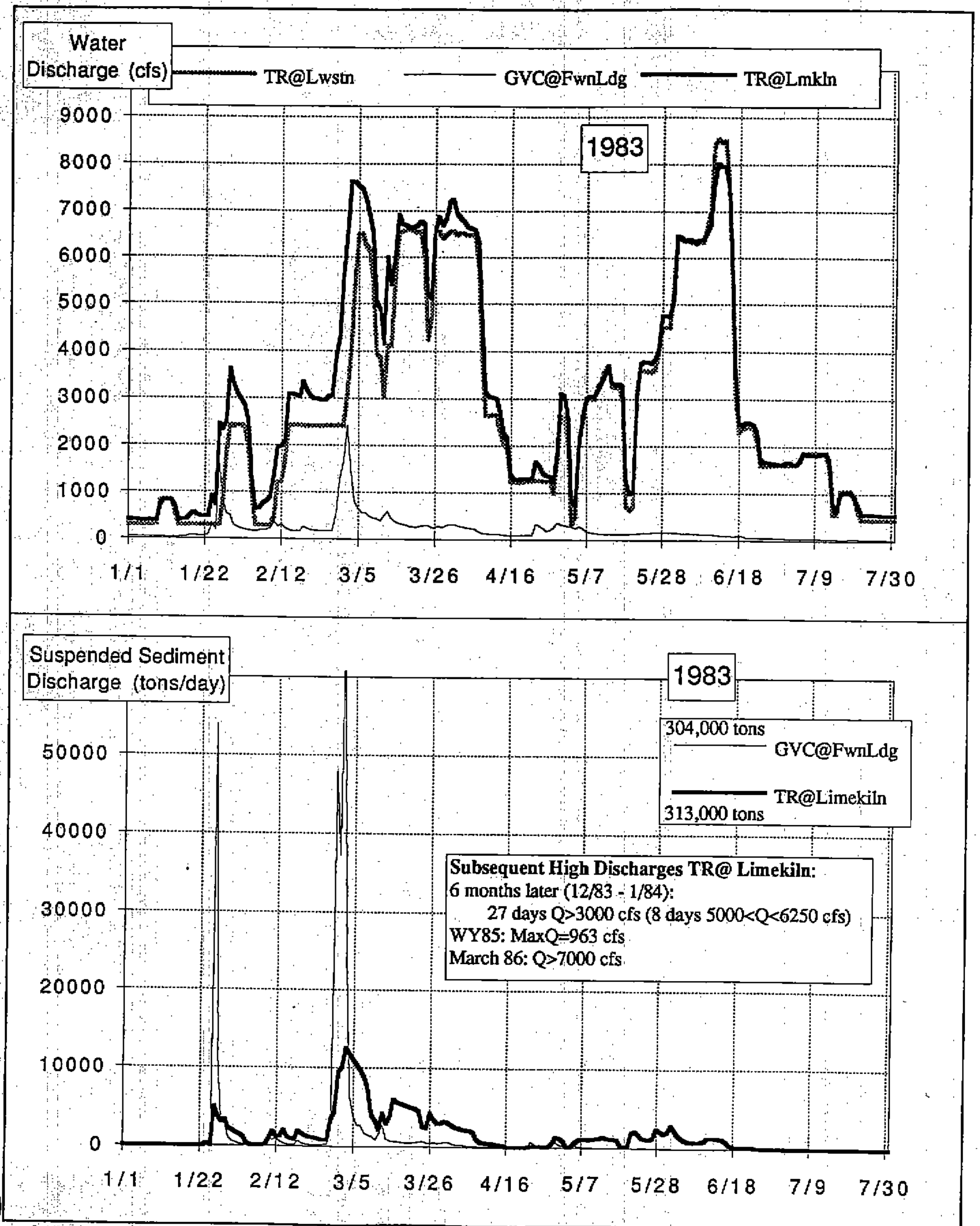
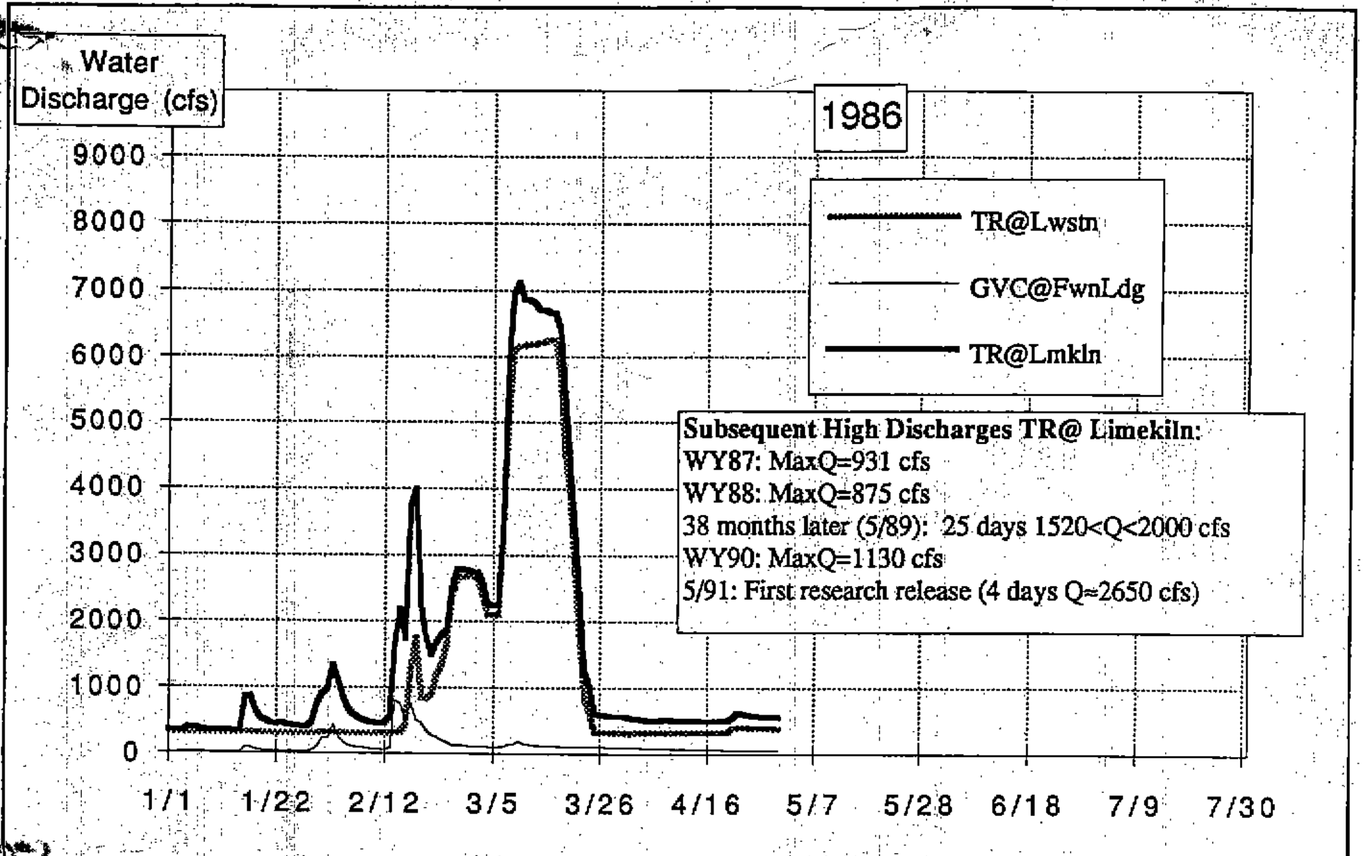


Figure 6.6.3 Water and sediment discharge for the Trinity River and Grass Valley Creek: January-July, 1983



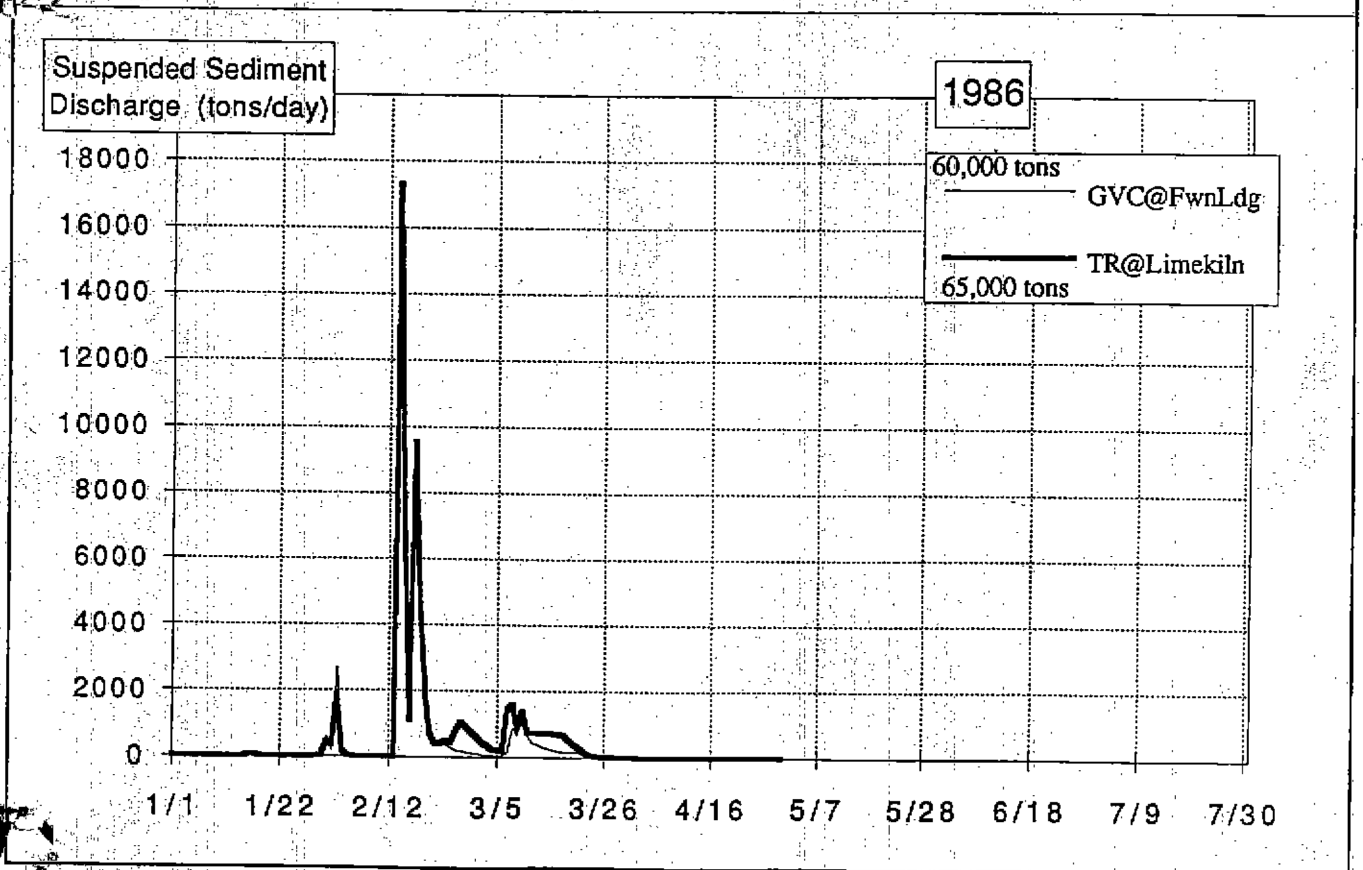


Figure 6.6.4 Water and sediment discharge for the Trinity River and Grass Valley Creek: January-April, 1986