LARGE WOODY DEBRIS FISH HABITAT STRUCTURE PERFORMANCE AND BALLASTING REQUIREMENTS

by

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ABSTRACT

Many stream restoration efforts include placement of constructed large woody debris (LWD) habitat structures. These structures are installed in stable channels to rehabilitate summer habitat and critical overwintering refuges in streams, thus attenuating stresses on the aquatic ecosystem until logged riparian areas naturally supply mature windfalls (Slaney & Martin, 1997). This study addresses one of the main problems faced by restoration practitioners: The lack of physically based design guidelines for LWD habitat structures.

This study presents the theoretical basis behind design methodologies for three types of LWD structures: (1) Single-LWD, (2) Single-LWD with intact root wad, and (3) Multiple-LWD structures. A field verification program was undertaken to test the applicability the theoretical basis and to refine the design guidelines. Over 80 LWD structures were assessed after construction and again after the fall 1997 to spring 1998 floods.

Results indicate that the design approach for single-LWD and single-LWD with root wad structures, based on a factor of safety against sliding failure, successfully predicted the stability of the structures during the past fall to spring floods. The stability of the multiple-LWD structures proved to be more complex to predict since a greater number of design and construction-related factors influence stability. Nonetheless, a design approach based on a safety factor against buoyant failure is recommended.

Recommendations with respect to the design and construction of LWD structures are also presented as part of this study.

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1 INTRODUCTION

Throughout this century there has been a substantial decrease in salmonid populations within the Pacific Northwest. In Canada, over 1,000 stocks of anadromous salmon and trout have been classified to be at risk, of special concern or extirpated (T.A. Slaney et al. 1996). In the states of California, Idaho, Oregon and Washington, 314 stocks of salmon and steelhead are reported at risk of extinction, whereas fewer than 99 native wild stocks have been identified as "healthy" (Huntington et al., 1996; Nehlsen et al. 1991). It is difficult to attribute these declines to any one cause, but the major factors are considered to be (Nehlsen et al. 1991):

- Over-harvesting of weaker stocks.
- Degradation of spawning and rearing habitat.
- Obstructions to fish migration (dams etc).
- Problems associated with hatcheries (primarily in the US).

While the above are not necessarily in any particular order, it is widely acknowledged that a major factor is habitat degradation. Recognition of this problem is not new. As early as 1885, Van Cleef (1885) concluded that "*the destruction of the trees bordering on these streams and the changed conditions of the banks produced thereby, has resulted in the destruction of the natural harbours or hiding places of the trout, that this is the main cause of depletion, and that until these harbours are restored, it will be useless to hope for any practical benefit from restocking them*". While his conclusions were drawn from investigations of Catskill streams in New York State, they are applicable to most fish bearing streams in British Columbia. Many of our past and present resource management approaches have had synergistic effects that have led to habitat degradation and significant declines in fish populations. These human impacts are so perverse, and have occurred for so long, that few appreciate their magnitude and implications (Harmon et al, 1986). Degraded streams and rivers now appear "normal" to untrained eyes.

1.1 LWD in Streams

Under natural conditions, large woody debris (LWD) finds its way into streams via slow recruitment processes. These include undercutting of streamside trees by gradually migrating streams, and windfall of riparian trees. Currently, many streams in British Columbia are deficient in LWD compared to pristine conditions. This is the result of two, often compounded, activities: (1) clearcut logging of the riparian zone and (2) removal of in-stream LWD.

1.1.1 Harvesting of riparian zone

Until the introduction of the Coastal Fisheries-Forestry Guidelines in 1988, forest harvesting to the stream bank was common practice in British Colombia. This has resulted in a dramatic reduction in the volume of LWD available for recruitment into many streams and rivers.

Murphy and Koski (Koski, 1992) have modelled changes in the amount of LWD present in small streams following clear-cut logging with no buffer strips along banks. As illustrated by Figure 1-1, this model shows that minimum LWD abundance occurs at about 110 years after the original disturbance, and full recovery to pre-logging levels may take as much as

250 years. This implies that the minimum LWD levels in most of our streams is still to come. This interruption in the natural recruitment cycle of mature windfall, and available in-stream LWD, contributes to a long-term decline in the complexity and diversity of stream aquatic ecosystem including fish habitat (Slaney & Martin, 1997).

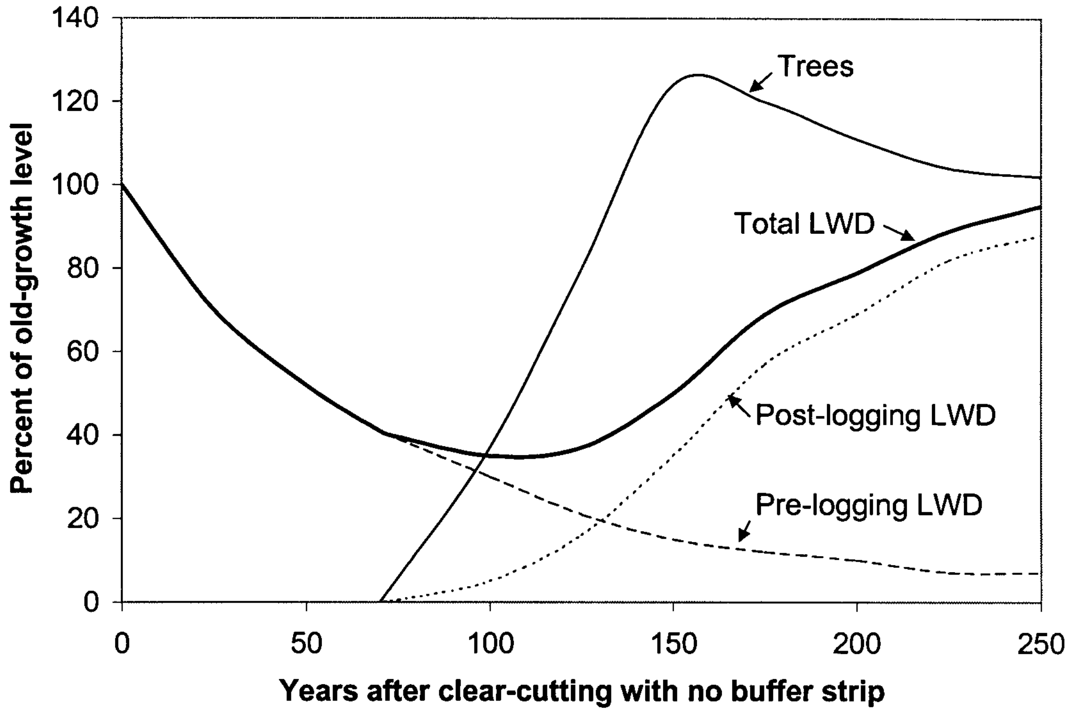


Figure 1-1: Model of changes in LWD amount in small (2nd- 3rd order) streams after clear-cutting (from Koski, 1992)

1.1.2 De-snagging

Historically, removal of in-stream LWD (also referred to as de-snagging, river clearing, river improvement or channelisation) has been actively pursued and encouraged within many rivers and streams across British Columbia and throughout the world. Reasons for "de-snagging" are numerous and include:

- Provision and maintenance of navigability of larger river systems.
- Maintenance or improvement of water conveyance channels in an effort to alleviate overbank flooding and lessen the risk of damage to property and bridges.
- Improvement of recreational amenity (boating, swimming, fishing and water skiing).
- Promotion of channel stability (based on grounds that LWD could divert flow or cause turbulence near banks, resulting in bank erosion).
- Allow splash damming for timber transportation.
- Supply of irrigation water.

Ironically, LWD and logjams were once considered as barriers to fish migration. Until the 1960's, the Canadian Fisheries Act actively promoted the removal of LWD from British Columbia streams.

1.1.3 Importance of LWD

Since the late 1970's we have become increasingly aware of the importance of LWD within our stream environments. Several studies have documented the significance of LWD in the function of stream ecosystems (Koski, 1992; Thomson, 1991; Hicks et al., 1991; Shrivel, 1990; Harmon et al, 1986; Swanson and Lienkaemper, 1978; Bustard and Narver, 1975). Thomson (1991) has summarised an extensive bibliography of LWD with regards to stream channels and fish habitat. Some of the key ecological functions of LWD in stream systems include:

- Creation of low-velocity zones that act as refuge for fish and invertebrates.

- Provision of shelter from predators for both adult and juvenile fish while serving as resting locations during migration.
- Creation of pools that increase habitat diversity, facilitate migration through maintenance of a stepped profile and trap spawning gravel.
- Serving as stable host surface for the growth of algae and invertebrates and provision of locations for nitrogen fixation.
- Contribution to the dissolved and particulate load of the system, while retaining fine particulate organic matter for processing by invertebrates.
- Retention of salmon carcasses after spawning.
- Provision of habitat for fauna (e.g. animal crossing, resting and hunting locations).
- Submerged and overhanging trees may reduce water temperatures.

There is extensive evidence that river systems with LWD are significantly different from those devoid of debris. Distinctions may be observed in the type and amount of pools, the regularity of the pool spacing, the kinds of sediment storage sites present and width variations. LWD provides sediment storage "compartments" which provide an important buffer system regulating channel form, profile and sediment discharges (Keller and Tally, 1979). The presence of LWD within small to intermediate channels will tend to increase the variability of the channel dimensions as well as the relative distribution of pools and riffles. LWD has been found to be the structural element most often associated with pool formation within small to large river systems (Abbe and Montgomery, 1996; Robinson and Beschta, 1990; Bilby and Ward, 1989). With increasing debris load, the average pool to pool spacing will decrease from 5 to 7 bankfull widths to 3 to 5 bankfull widths (Hogan et al., 1996).

1.2 LWD Structures and Fish

Fish have an affinity for various types of irregular features found in streams. Complexity occurs naturally by fallen LWD, bedrock outcrops, rubble and other debris. Salmonid species rearing in streams depend on these features during different life stages and seasons for food, reproduction and shelter from predators and environmental stresses (Bustard and Narver, 1975; Fontaine, 1988; Shrivel, 1990). Studies have found that salmonid juveniles rearing in streams are often closely associated with LWD particularly during winter (Bustard and Narver, 1975; Cederholm & al., 1997a; Cederholm & al., 1997b, Koski, 1992; Slaney et al, 1997).

It is apparent that stream rehabilitation efforts that reintroduce LWD within impacted streams are beneficial. Since the onset of habitat rehabilitation efforts several studies have been undertaken to determine their benefits and provide support for their implementation. House and Boehne (1985), and House (1996) have reported results from a coastal Oregon stream where stream rehabilitation works (including LWD, boulder clusters and gabion mats) have increased the streambed diversity, trapped gravel, created shallow gravel bars and increased the number, size and quality of pools. A substantial increase in the number of coho (2.5 times) and steelhead adults spawning reflected these habitat enhancements. Treated areas also supported significantly more juvenile coho salmon and cutthroat trout and salmonid biomass than untreated control areas.

Slaney et al. (1994) have also found that colonisation densities of constructed LWD structures by juvenile Chinook salmon was similar to naturally occurring debris cover within a large regulated river in central British Columbia. Debris catchers, that resemble natural logjams, were most highly colonised by juvenile salmon and adult rainbow trout. Cederholm et al, (1997b) reports increases in winter juvenile coho salmon abundance in rehabilitated reaches of a coastal Washington stream. The increases were in the order of 6 and 20 times pre-treatment levels respectively for a "logger's choice" (lower density of non-anchored red alder LWD) and an engineered treatment (anchored coniferous LWD and boulders), whereas the reference site exhibited no change in density. The number of coho salmon smolts migrating from both the engineered and "logger's choice" sites increased significantly following enhancement.

With the numerous complexities involved in evaluating fish habitat and fish responses, one frequent criticism raised of habitat enhancement is that the fish are simply migrating from a less desirable area within the system to the treated areas with little increase in output from the entire system. A number of earlier studies have shown that habitat improvement works will provide a net increase in salmonid production where the lack of rearing habitat is a limiting factor within the system (numerous authors as cited by Slaney et al. 1994). This stresses the need for watershed and fish habitat assessments that enable to determine the limiting factors and guide in prescribing rehabilitation efforts.

Most studies concur that rehabilitated streams are more complex, more effective at dissipating energy, better able to trap detritus and generally provide much better fish habitat than the homogeneous channels that existed prior to rehabilitation efforts. Where rearing habitat is inadequate, constructed LWD structures can provide an interim source of cover and complexity until the riparian area recovers and LWD recruitment occurs naturally.

1.3 Problem identification

Given the recognised benefits of reintroducing LWD into streams, the problem facing river restoration practitioners today is a lack of physically-based design guidelines for constructed LWD structures. In the past, most structures were designed and constructed based solely on the judgement and experience of the designers and/or builders, many of whom had little training in hydrology, geomorphology and river engineering. The results were often under-designed structures that failed during floods or did not perform as originally intended.

Naturally occurring LWD and LWD jams result from random events that take place over decades and even centuries. A relatively small fraction of the natural LWD that enter the streams creates quality habitat for fish. By systematically placing LWD, watershed rehabilitation practitioners seek to maximise habitat production and enhance the habitat creation potential of LWD (Hilderbrand et al., 1997). However, to ensure that the rehabilitation work is cost effective, the LWD habitat must remain in place and function as intended for a predetermined duration.

The majority of the LWD being replaced within streams as part of rehabilitation efforts lacks the size, the length, the branches and the root wads typical of naturally recruited LWD. Logically, larger LWD pieces whose lengths exceed the channel width are more likely to remain in place (Hilderbrand et al., 1998) for decades and even centuries. However, since such large and intact LWD is seldom available, smaller LWD must be utilised and secured into place. The use of boulder anchors, cabled to the LWD, appear to be one of the most cost-effective method to secure the habitat structures while imposing the least disturbance to the riparian area and banks (Fontaine and Merritt, 1988).

Considerable effort and expense are being directed at LWD placement for the rehabilitation of stream habitat. Failures, due to inadequate ballasting, construction and/or positioning should be avoided. Sound ballasting guidelines, based on physical principles, tested and verified in the field would ensure that the LWD structures provide longer-term return on investment.

1.4 Study Objectives

The primary objective of this study is to develop and further refine ballasting guidelines for three types of LWD habitat structures. Preliminary design guidelines for the ballasting of two types of lateral logjams have been presented in a Technical Memorandum by Millar (1997). As well, additional design guidelines for the ballasting of a single LWD with an intact rootwad have been prepared and the theory is presented herein (Chapter 3). This work forms the basis for this study. Figures 1-2 and 1-3 illustrate the types of LWD habitat structures of interest.

As part of our field investigations, over 80 LWD structures have undergone detailed post-construction and post-flood assessments. This information will serve to verify the proposed design guidelines and provide the basis for refinement of the design methods. This information will also be used to investigate current design and construction methods in an effort to identify successful and not so successful approaches.

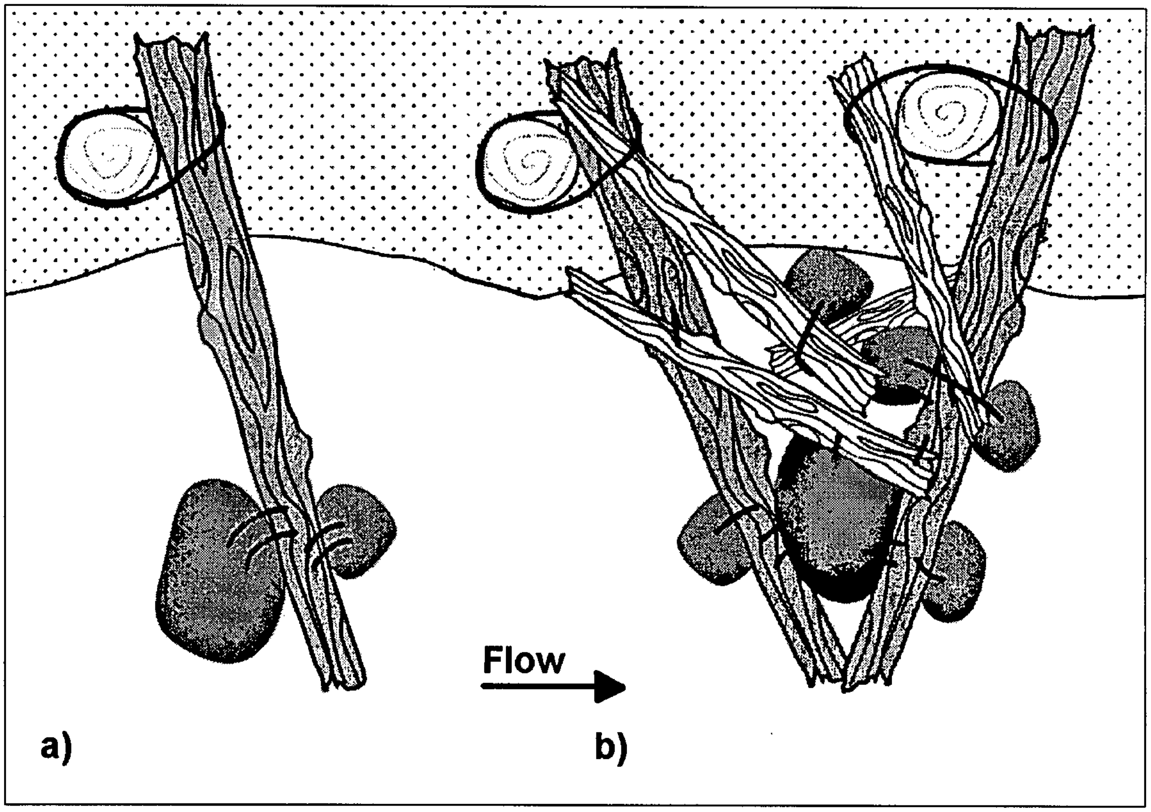


Figure 1-2 : Illustrations of a) Single-LWD, and b) Multiple-LWD Structures (Modified from Slaney et al., 1997)

1.5 Thesis Layout

This thesis consists of seven major chapters and five appendices. The chapters and their content are outlined below.

- Chapter 1 provides some background into the functions of LWD, the problems encountered by practitioners and the objectives of the study.
- Chapter 2 undertakes a review of the literature focusing on the stability of in-stream LWD habitat structures.
- Chapter 3 presents the basic theory on which the design guidelines are founded.

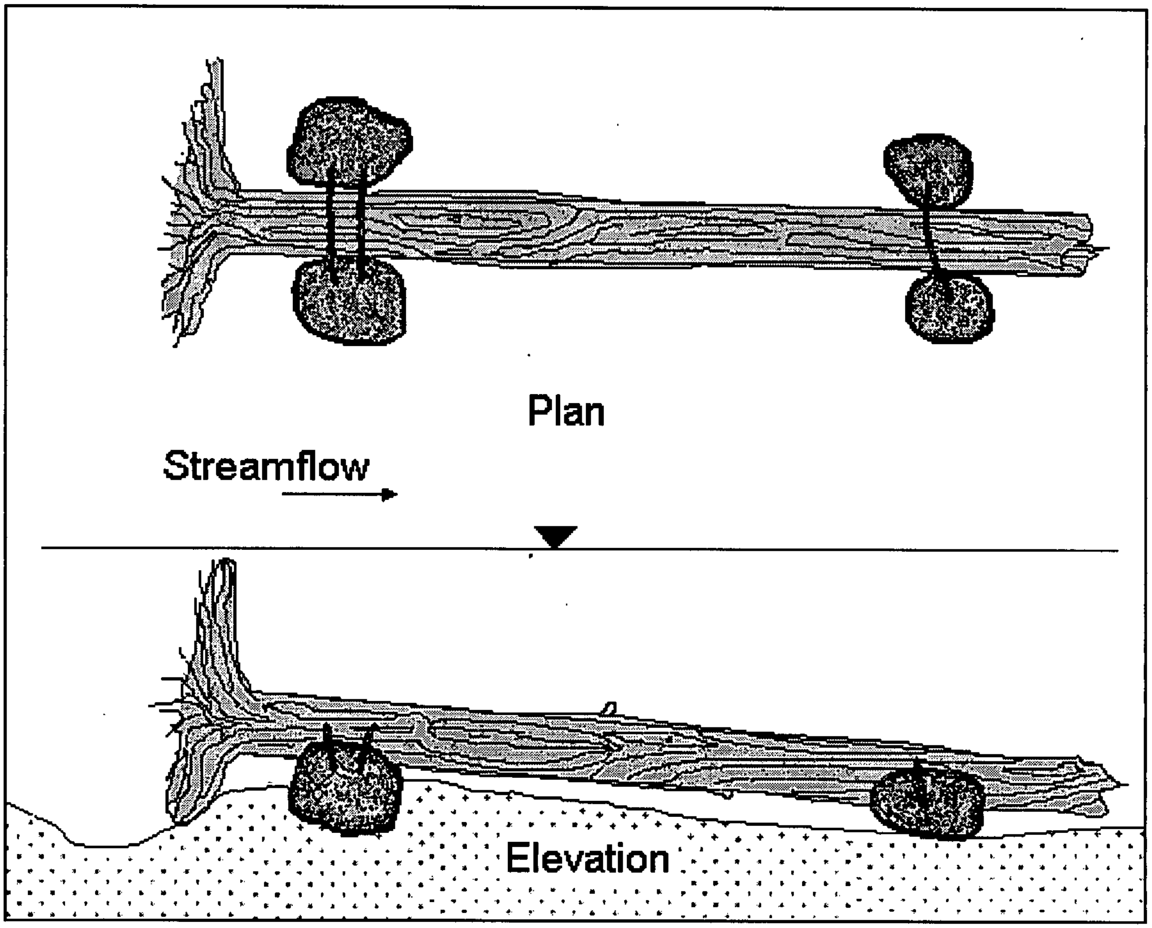


Figure 1-3: Illustration of a Single LWD with intact Root Wad

- Chapter 4 details the method used to undertake the field investigations and the performance evaluation of the LWD structures.
- Chapter 5 describes the seven sites selected for the study.
- Chapter 6 provides the results and discussion of the performance evaluation of the three types of LWD structures under investigation.
- Chapter 7 summarises the conclusions and recommendations that have emerged from this study.

2 LITERATURE REVIEW

The following chapter provides a review of the literature with respect to the physical performance of constructed habitat structures. The first part will look at existing design guidelines, whereas the second part will look at structure durability studies. The factors that influence structure durability and performance of constructed LWD habitat will be discussed.

2.1 Existing Design Guidelines

Numerous stream rehabilitation handbooks have been published over the last couple decades. Some examples include Slaney and Zaldokas (1997), Anonymous (1995), Hunt (1993), Newbury and Gaboury (1993), Hunter (1991), Reeves et al. (1991), Envirowest Environmental Consultants (1990), House et al. (1988), Wesche (1985) and Department of Fisheries and Oceans Canada et al. (1980). With few exceptions, these manuals consist of planning/design recommendations with conceptual drawings that lack rigorous criteria for development of engineering specifications. Anchoring of structures is often discussed with little or no guidelines on material selection and construction. Fontaine and Merritt (1988) first introduced an anchoring system, which make use of steel cables anchored to ballast boulders or bedrock with an epoxy resin. This system proved revolutionary in that it provides added flexibility in the design and construction of in-stream rehabilitation structures while making use of native materials and minimising the disturbance to the stream and riparian area. However, Fontaine and Merritt (1988) provided only conceptual drawings of ballasting techniques with no help on ballasting mass requirements.

The author is not aware of any reliable quantitative guidelines for specification of boulder ballast, prior to the formulation of the preliminary design guidelines for lateral logjams developed by Millar (1997) (partially reprinted in Slaney et al., 1997). Only one attempt at computing ballast requirements for a LWD structure was found, unfortunately some of the methods used were not based on sound hydrodynamic principles.

2.2 Structure Durability Studies

Following a surge of construction of in-stream structures for stream rehabilitation, a number of studies were conducted throughout the Pacific Northwest (PNW) in an attempt to describe the factors that affect the physical performance of constructed LWD structures. These studies are primarily statistical, and have focused most of their attention on the external factors influencing structures. A few reports have broken down their statistical analysis to structural types, or in some cases anchored and non-anchored, but none have closely examined the design of the structures. Nonetheless, these studies do provide some valuable information with respect to the planning and design of LWD structures. Table 2-1 provides an overview of the structure durability studies that will be discussed herein.

Table 2-1: Overview of Structure Durability Studies

Authors	Location	Number of Streams	Number of Structures	Channel		Basin Area (km^2)	Flow Return Period (yrs)
				Width (m)	Slope (%)		
Higgins and Forsgren (1986)[a]	OR	N/R	632	N/R	N/R	N/R	15-25
Doyle(1991)[a]	WA	N/R	2000	N/R	N/R	N/R	50-100
Frissell and Nawa (1992)	OR WA	8 7	161	5.5-31	1.0-6.0	1.2-632	<2-10
Metzger (1997)[a]	OR	26	674	4.0-20	1.0-4.0	3.9-150	<10->100
Roper et al (in prep.)[a, b]	OR & WA	94	3946	N/R 1st to 6th order streams			5-<100
Hartman and Miles (1995)	BC	99[c]	≈110	N/R	N/R	N/R	N/R

N/R – not reported, OR - Oregon, WA – Washington, BC – British Columbia
[a] Suspected overlap in sampled structures
[b] Heller et al. (1997) is a preliminary report based on same data set as Roper et al. (in prep.)
[c] Stream rehabilitation projects

2.2.1 Overall durability

Prior to discussing the findings of these studies, some of the terms used when describing the "durability" and "functionality" of LWD structures must be defined since these often lead to some confusion when interpreting and comparing results across studies. It should be noted that some of these definitions still necessitate some interpretation in their application.

Damage rate: Proportion of structures in the failed or impaired category. Structures not successfully meeting physical objectives (Frissell and Nawa, 1992).

Durability: Classified based on the movement of structures within three categories, in-place, shifted-on-site and left site (Roper et al., in prep.).

Failure: A structure that has been washed downstream, severely fragmented, or grossly dislocated so it retained little or no contact with the low-flow channel or was otherwise incapable of achieving its intended physical objective (Frissell and Nawa, 1992).

Impaired: A structure that remained in its original location but, because of alteration to it or the stream channel, no longer functioned in the intended mode or appeared to be at least temporarily ineffective. A structure buried under bedload was considered impaired (Frissell and Nawa, 1992).

Performance: Different degrees of performance can be defined as fully functional (that is meeting their original objectives), partially functional and not functional. Note that structure performance is highly related to their stability (Metzger, 1997)

Stability: Different degrees of stability can be defined as in-place, shifted, partially washed away or partially gone and washed away or gone (Metzger, 1997).

Successful: A structure not visibly damaged or debilitated (Frissell and Nawa, 1992).

Success rate: Proportion of structure in the fully functional and partially functional category (Metzger, 1997). Note that Frissell and Nawa's (1992) definition of success rate (by extension) is only based on physical integrity (not functionality) and would include only structures that are successful.

In one of the earliest studies of this type, Higgins and Forsgren (1986) reported that 90% of the structures evaluated within the Mt. Hood National Forest were functioning after being subjected to the 1986 floods. The flood return period for these streams were estimated to be between 15 and 25 years. It must be noted however, that only about 5% of the structures investigated were built of LWD; all other structures were constructed of boulders.

Following the November 1990 flood within the Mt. Baker-Snowqualmie National Forest, Doyle (1991) found an overall damage rate of 24% (12% lost, 6% moved and 6% buried) on a total of about 2,000 structures. A significant portion of the structures were located within alluvial fans (25%) and large tributaries (15%) and were not designed to withstand large flood events (Doyle, 1991).

Frissell and Nawa (1992) found an overall damage rate (failed or impaired) of 62%. The damage rate per project was highly variable from 0 to 100%. The actual failure rates reported per project ranged from 0 to 100% with a weighted average of 31%. 38% of all the structures assessed were successful.

In the Suislaw National Forest, Metzger (1997) reported more promising observations. Overall, 86% of the structures investigated remained essentially in place, 5% were partially washed out and only 9% left site. The structural stability for individual projects ranged from 33% up to 100% (Metzger, 1997). It is unclear here if the author's use of stability includes structures that have shifted and partially washed-away.

In the most extensive study conducted within 7 National Forests, Roper et al. (in prep.) found that the overall durability was high with only 16% of structures leaving site of original placement. Once again a significant variability in results was reported; 48 streams had less than 10% of the structures leave the site, whereas 14 other streams had more than 50% of the structures leave the site (Roper et al., in prep.).

It is thought that the higher damage rates reported by Frissell and Nawa (1992), may reflect the state of stream rehabilitation in its infancy, where earlier planning, design and anchoring methods were still being tested and the learning curve very steep. Furthermore, none of the studies discuss maintenance of the structures. Given the well established maintenance programs in place in National Forests, it is speculated that in the larger studies, the structures may have been subject to "upgrades" rendering them better suited to resist floods. Unless a good monitoring/tracking program is in place from construction, damage rates may be underestimated since some of the structures, or parts thereof, that have left the site may not always be accounted for (Frissell and Nawa, 1992). This is especially true for major projects where the number of structures, and components per structure, is large and tremendous effort is required to monitor progress with time.

The great variability in reported results for these studies attest to the dynamic nature of high-energy streams found in the Pacific Northwest. This variability indicates that in-stream structures may not be durable at sub-watershed scale (Roper et al., in prep.) and that great care is required in establishing watershed prescriptions.

2.2.2 Structural types

A summary of the structure types and their success rates reported in the durability studies is provided in Table 2-2. Note that some of the most extensive studies, such as those by Doyle (1991) and Roper et al. (in prep.), did not provide details with respect to structure type. Other studies such as the Coquihalla highway (Miles, 1995) and Oldman River Dam projects (Hartman and Miles, 1995) are not included since only a few structures were constructed of LWD. Figure 2-1 presents a schematic of typical in-stream structures used in fish habitat rehabilitation projects in small to medium streams.

Table 2-2 : Reported Structure Types and Success Rates

Structure type	Frissell and Nawa (1992)		Metzger (1997)		Higgins and Forsgren (1986)		Hartman and Miles (1995)	
	n	%[a,b]	n[a]	%[a,c]	n[a]	%[c]	n[a]	%[a,d]
Multiple log & jam	36	72	230	79	-	-	8	75
Single log deflector	30	45	190	85	30	93	6	83
U/S & D/S V	12	56	30	99	-	-	4	100
Diag. log weir	23	87	60	85	6	83	3	100
Transv. log weir	30	70	60	85	-	-	4	100
Cover log	-	-	50	70	-	-	10	90
Boulder cluster	15	90	20	100	-	-	21	90
Boulder weir	-	-	10	95	115	84	-	-
Individual boulder	9	67	-	-	-	-	12	83
Overall structures[e]	161	69	674	83	632	93	110	91

n – number of structures in study sample
[a] Numbers are approximate
[b] Success + impaired
[c] Fully functional + partially functional
[d] Limited success, successful and outstanding; note small samples
[e] Note that numbers do not add up along column since details are not available for all structural types

Of the type of structures evaluated within the durability studies, boulder clusters and boulder sills displayed the highest success rates (generally above 90%). Only the boulder structures from the Mt. Hood National Forest (Higgins and Forsgren, 1986) displayed lower success rates, where the failures were attributed to the use of undersized rock collected from within the channel. High success rates have also been observed within the Keogh River were boulder cluster studies have been conducted in the last two decades (Ward and Slaney; 1979, 1993). These statistics appear quite promising when compared to those reported by Miles (1995) where at least 50% of the boulders were missing, buried or not in low water for 87 and 78% of the structures, respectively in the Coquihalla and Coldwater rivers, some 8 to 14 years after construction.

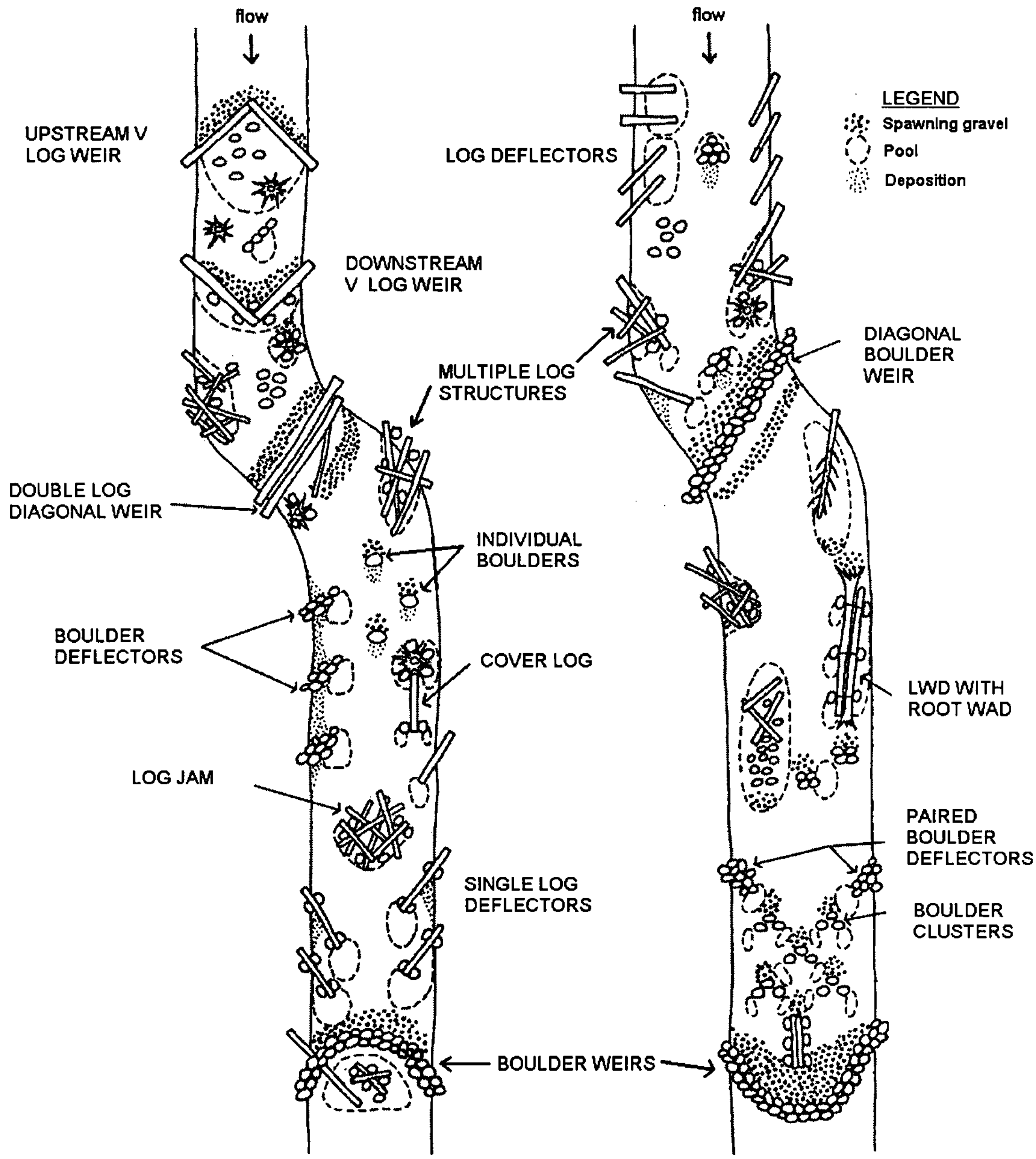


Figure 2-1: Typical stream rehabilitation structures (modified from Crispin, 1988)

The damage rates for weir type structures (diagonal and transverse weir/sill, upstream and downstream V weir) reported by Frissell and Nawa (1992) are higher than those reported by others. Typical success rates reported, excluding those by Frissell and Nawa (1992), indicate that more than 80% of the structures remain functional. Similar numbers are found for the Oldman River Dam Project in Alberta where 84% of the 61 structures were functional after 4 year of operation (cited by Miles, 1995). It is important to note however, that only 62% were functional after the first year of operation and that maintenance was necessary to obtain the 84% figure above.

Results for the Single-LWD deflector structures are also significantly different between Frissell and Nawa (1992) and the other studies (success rate greater than 80%). It is suspected that some of the difference may be attributed to the fact that a greater fraction of the deflector logs reported upon by Frissell and Nawa (1992) were not anchored compared to the other reports.

The durability of the multiple-LWD structures reported is comparable across studies. The relatively high success rate of these structures is attributed to the fact that shifting may take place without compromising the integrity and function of the structures (Metzger, 1997).

2.2.3 Use of anchors

The use of anchors, steel cable and boulders or streamside trees have proved quite effective at maintaining high success rates. The cabled natural LWD or jams reported by Frissell and Nawa (1992) were about 30% more successful than other multiple-LWD structures. Likewise, Roper et al. (in prep.) found that structures cabled in place were more durable (15% left site) than those not cabled in place (22% left site).

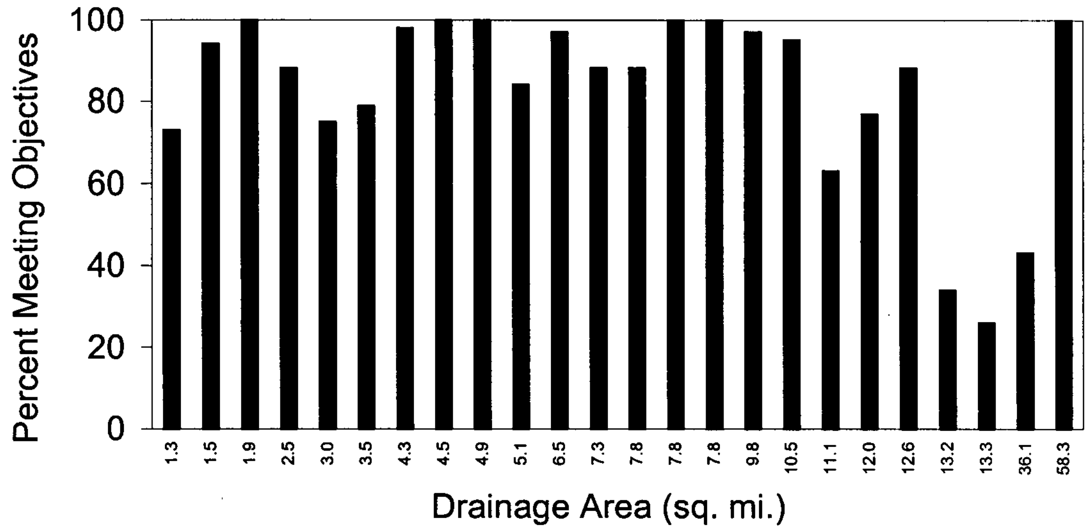
Metzger (1997) reported that anchoring was 90% effective at keeping structures in place whereas only 5% washed away. On the other hand, Metzger (1997) found that 33% of unanchored structures were washed away while only 60% remained functional. It should be noted that about 50% of the unanchored structures were located within the 2nd largest system investigated and therefore these results may be influenced by stream size (Metzger, 1997). Cabling of LWD to other LWD without anchors was less effective where 65% of the structures remained in place and 16% were washed away (Metzger, 1997).

Unfortunately, none of these studies have differentiated between anchor types used. When anchoring is mentioned, they typically group all methods together including, bedrock anchors, boulder ballast, dead-man anchors, re-enforcing steel, steel cables, etc. Nonetheless, the net increase (7 to 30 %) in structure durability provided by anchoring lends support to more research on this topic and justifies the extra expense of producing detailed designs.

2.2.4 Stream size

Frissell and Nawa (1992) observed that damage rates were greater in rivers with larger active channel widths. No clear relationship emerged between damage/failure rates and basin area. They maintain regional variations in climatic and hydrologic processes render channel width a better measure of stream flow and hydraulics stresses than basin area. Stream width also reflects bank stability that in turn affects structure performance (see Failure modes; Frissell and Nawa, 1992).

As for the Suislaw National Forest, Metzger (1997) reports a strong relationship between stream size (in terms of watershed area) and stability/performance (Figure 2-2). This conclusion is not obvious however, when the data presented is plotted on a scatter plot (correlation coefficient –0.19). Only two systems are larger than 35 km^2 and about half of the structures in the second largest system (square marker on Figure 2-3) were not anchored thus affecting the conclusions that can be drawn from the data. A higher variability in performance is obvious as the drainage area goes beyond about 25 km^2 (10 sq. mi.).



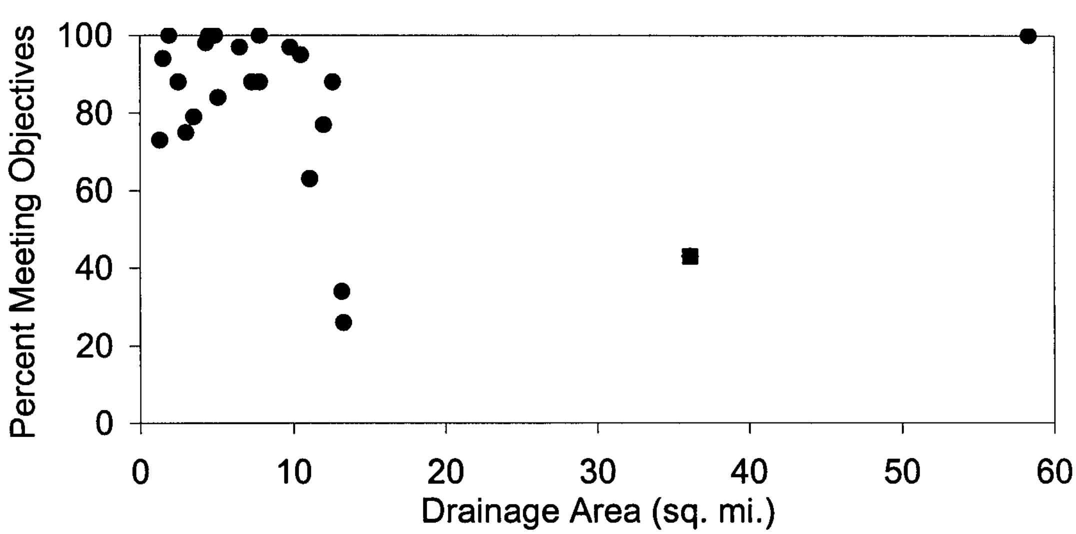


Figure 2-2: Structure performance vs. stream size – as Bar Chart (Figure 13 from Metzger, 1997) and Scatter plot (modified from Metzger, 1997)

Roper et al. (in prep.) reported that movement of structures was significantly less ($P < 0.05$) in low order streams than in high order streams. Structures in 6th order streams (63% left site) were 20 times more likely to leave site than those within 1st order streams (3% left site)(Roper et al., in prep.).

2.2.5 Flood frequency

Frissell and Nawa (1992) and Metzger (1997) did not find any relationship between flood magnitude and damage rates or stability or performance. This was attributed to the uncertainty in determining return periods for floods, fewer high magnitude floods and the few points reported in each category (Metzger, 1997; Roper et al., in prep.).

Based on four broad categories of flood magnitude (<15, 15 to 40, 40 to 65 and >65), Roper et al. (in prep.) reported a significant ($P < 0.05$) effect on durability. Interestingly, 15% of structures that experienced a flood magnitude of <65 year return left the site compared with 25% for those that experienced a flood magnitude of > 65 years return (Roper et al., in prep.).

2.2.6 Failure modes

Some of the most frequently reported failure modes included undermining of structures from scour of bed and/or banks (including undercutting of trees), burial from channel aggradation and/or shifting and failure of anchoring materials (cables, epoxy and bolts), (Metzger, 1997; Hartman and Miles, 1995; Frissell and Nawa, 1992). Frissell and Nawa, (1992) also noted that failure occurrences appeared to decreased when less disturbance was brought to the existing channel.

In an evaluation of fish habitat improvement projects, Hartman and Miles (1995) enumerated items having been responsible for projects classified by others as failures or of limited success. These items fell in three broad categories:

- Inadequate planning, design and construction - inappropriate physical location, failure to emulate natural habitat, insufficient rock sizes, budgetary constraints, poor construction practices, lack of experience, and insufficient anchoring.
- Morphological impacts - unstable stream banks and bed, high sediment loads and washing away of placed gravel.
- Hydrologic/hydraulic impacts - stream gradients too low or two high, stream flows which are too high, too variable or too low and unexpected large flood.

2.2.7 Other factors

Roper et al. (in prep.) have also found that durability of habitat structures is affected by upslope landslide frequency. In-stream structures were almost 3 times more likely to leave the site in a high landslide frequency basin than in a low landslide frequency basin. This is not surprising since high landslide frequencies are indicative of watershed disturbances and often closely related to debris flow occurrences.

Frissell and Nawa, (1992) also noted the high failure rates for terrace bound valley sites. This can be explained by more extreme hydraulic conditions under flood flows as little relief is available combined with an increased connectivity between the stream and hillslope where debris torrents can reach rehabilitated sites.

Structures not connected to the edges of the channel were 50% more likely to have left the site than those placed on channel edges or spanning the entire channel. (Roper et al., in prep.). Similarly, Higgins and Forsgren (1986) reported that structures spanning the entire channel failed 30 times more often than non-cross channel structures.

Stream power is often discussed in relation to site selection (Miles, 1995) or structural stability of in-stream structures (Doyle and Sheng, 1996). However, no evidence was found to support that slope, and by extension stream power, affects failure rates (Heller et al., 1997; Frissell and Nawa, 1992).

2.3 Summary

Previous studies reviewed in this chapter have proved to mainly statistical accounting exercises. While they may help in the preparation of prescriptions at the watershed level, they do not significantly increase our knowledge at the reach or site level, and offer few quantitative design guidelines.

There is a pressing need to elucidate and understand the physical processes that govern the design of various LWD habitat structures. The investigative approach taken by Millar (1997), and which will be expanded herein, is based on the accounting of hydraulic and gravitational forces for each single structure. The theoretical background in support of this approach is presented in the following chapter.

3 THEORETICAL STABILITY ANALYSIS

This chapter presents the basic theory behind the stability analysis for the three LWD structure types of interest. This analysis, initially presented by Millar (1997), forms the basis on which the design guidelines for these structures were formulated. The theoretical analysis for the case of a single-LWD structure will be introduced first and the development for the multiple-LWD and single-LWD with root wad will follow.

3.1 Single-LWD Structures

Single-LWD structures, also known as single-log lateral jams or single log deflectors, consist of a log projecting from the bank into the stream. The log is attached at one end to a tree or stump on the bank, while the stream end is ballasted with one or more anchor boulders to prevent movement during floods (Figure 1-2a).

LWD structures are subjected to a combination of hydrodynamic, frictional and gravitational forces that act on either the LWD or the anchor boulders. This stability analysis is based on the forces that are acting on, or being transferred to the anchor boulders (Figure 3-1). The principal forces acting on the LWD and anchor boulders are:

- Vertical buoyancy force acting on the LWD and transferred to the anchor boulder (F_{BL}).
- Horizontal fluid drag force acting on the LWD and transferred to the anchor boulder (F_{DL}).
- Horizontal fluid drag force acting directly on the anchor boulder (F_{DB}).

- Vertical lift force acting directly on the anchor boulder (F_{LB}).
- Immersed weight of the anchor boulder (W').
- Frictional force at the base of the anchor boulder that resists sliding (F_F).

These principal forces are discussed individually in the following sections.

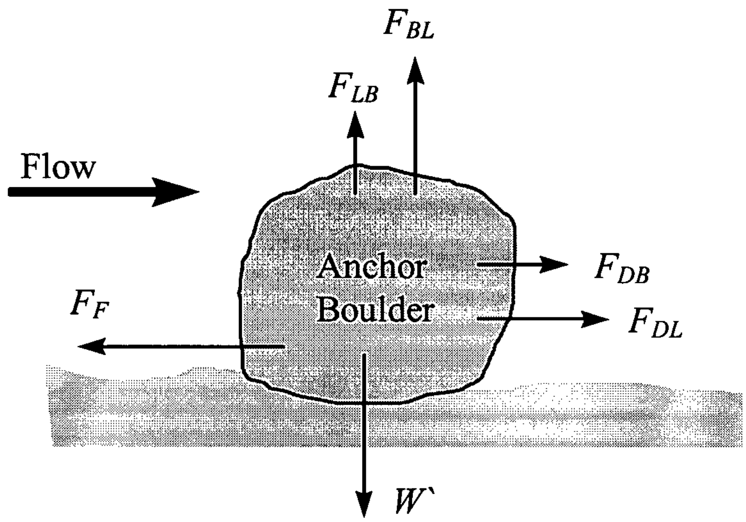


Figure 3-1: Principal Forces Acting on Anchor Boulder

Note that the following is a simplified analysis that assumes full submergence of the LWD and anchor boulders.

3.1.1 Vertical buoyancy force from the LWD (F_{BL})

The specific gravity of wood is typically less than water, therefore an immersed piece of LWD will be subject to a net upward buoyancy force. With the LWD being fixed a both ends (Figure 1-2a), the vertical buoyancy force transferred from the LWD to the anchor boulder is equal to half of the total buoyancy force acting on the LWD:

$$F_{BL} = 0.5\,BF\frac{L\pi D_L^2}{4}\rho\,g(1-S_L) \tag{3-1}$$

Where *BF* is a ballast factor (value of 1 or 2, discussed below); *L* is the length of LWD (m); D_L is the diameter of LWD (m), ρ is the density of water (1,000 kg/m^3); *g* is the gravitational acceleration (9.81 m/s^2); and S_L is the specific gravity of the LWD. Typical specific gravity values of air-dried timber are given in Table 3-1. These values would represent relatively dry LWD and are potentially adequate upon construction. For LWD that is fresh cut or has remained submerged for a period of time, these values would prove to be conservative. However, since a portion of the LWD would likely spend a significant amount of time out of the water, these values are recommended for design. In cases were the LWD cannot be identified, an average value of $S_L = 0.5$ is recommended.

Table 3-1: Specific densities of air-dried timber[a]

Species	S_L
Cedar	0.36
Spruce (Sitka, White and Englemann)	0.43
Hemlock, Pine (Jack and Lodgepole), Spruce (Black)	0.48
Pine (Ponderosa)	0.51
Fir (Douglas)	0.54

[a] Modified from Laminated Timber Institute of Canada (1980)

An additional variable, the ballast factor (*BF*), has been added to Millar's (1997) analysis in order to evaluate LWD structures or members that rely solely on boulders for anchoring (Figure 3-2). These cases occur when single-LWD structures are not anchored to a fixed point on the bank (no tree or stump nearby). Under these circumstances, additional boulder mass must be provided to counter the buoyant and drag forces transferred to the bank end of the LWD. A *BF* of 2 is provided in Equation (3-1) to signal full compensation of the buoyant forces transferred to anchor boulders. If the single-LWD structure is fixed to a bank anchor other than boulders, then a BF of 1 signals the compensation of the in-stream half of the buoyancy forces by anchor boulders.

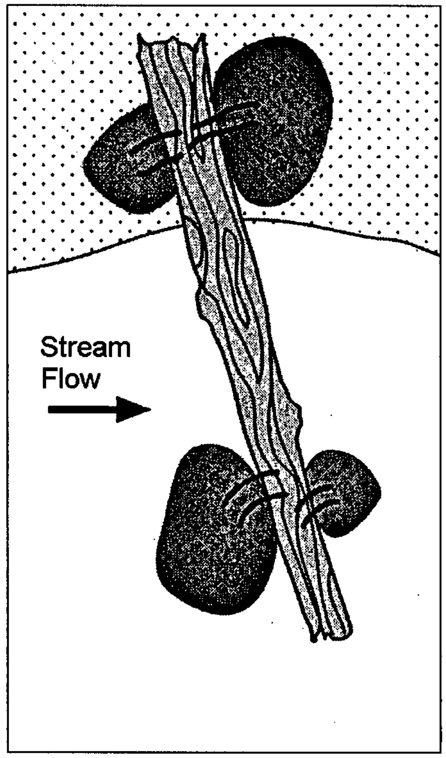


Figure 3-2: Single-LWD Structure Relying Solely on Boulders for Anchoring (BF=2)

3.1.2 Horizontal drag force transferred from the LWD (F_{DL})

Flowing water exerts drag forces onto immersed objects. High pressure develops on the upstream side whereas, low pressures are present on the downstream side. The difference in pressure applied over the projected area facing the flow results in a net horizontal drag force acting in a downstream direction. The LWD being fixed at both ends, the force transferred to the anchor boulder is equal to half the total drag force on the LWD:

$$F_{DL} = 0.5\, BF\, C_{DL}\, \rho \frac{V^2}{2} L D_L \sin\beta \qquad (3\text{-}2)$$

Where BF is a ballast factor (value of 1 or 2, discussed previously); C_{DL} is the drag coefficient; V is the velocity (m/s), and β is the angle between the log and stream bank (°). For the current analysis a value of $C_{DL} = 0.3$ is assumed. This value is based on work by Hoerner (1965) for smooth cylinders with a Reynolds number in the range of 5×10^5 to 5×10^6 (range expected for LWD and anchor boulders in natural streams). More recent work reported by Gippel et al. (1992) suggests that this value may be low and that the C_{DL} ranges from about 0.4 to 1.0 as the surface roughness of the cylinder increases to that expected of a natural log with bark. Nonetheless, initial investigations conducted by Millar (1997) would appear to favour a C_{DL} of 0.3.

As discussed in section 3.1.1, a ballast factor has been added to the analysis by Millar (1997). In the case of Equation (3-2), a BF of 2 indicated full compensation of drag forces by anchor boulders. Conversely, a BF of 1 maintains the assumption that half of the drag forces will be compensated by a fixed bank anchor in the form of a tree or stump on the bank. Note that the values of BF in both Equations (3-1) and (3-2) are the same.

3.1.3 Horizontal drag force on anchor boulder (F_{DB})

As with the LWD, the flowing water exerts a horizontal drag force directly on the anchor boulder. Assuming that the boulder is represented by a sphere of diameter D_B, F_{DB} can be estimated using:

$$F_{DB} = C_{DB}\, \rho \frac{V^2}{2} \frac{\pi D_B^2}{4} \qquad (3\text{-}3)$$

Where D_B is the anchor boulder diameter (m) and C_{DB} is the drag coefficient. A value of $C_{DB} = 0.2$ is assumed. This value is based on work by Hoerner (1965) for a smooth sphere with a Reynolds number in the range of $5x10^5$ to $5x10^6$ (range expected for LWD and anchor boulders in natural streams).

3.1.4 Vertical lift force on anchor boulder (F_{LB})

The lift forces acting on boulders are generally expressed in a form similar to Equation (3-3) or it is simply expressed as a fraction of the drag forces. Researchers such as Einstein and El Samni (1949) and Cheng et al. (1972) have found that the lift coefficients (C_{LB}), for large roughness objects, is equal to 0.18 (where V was measured at a depth of $0.35D_{35}$). Other sources refer to a ratio of the lift to drag forces (F_L/F_D), and typically cite a value of about 0.85 (Chepil, 1958). Assuming a value of 0.2 in Equation (3-3), this is equivalent to $C_{LB} = 0.17$. A value of $C_{LB} = 0.17$ will be assumed for the anchor boulder in this study. The lift force acting on the anchor boulders, F_{LB}, will be estimated using this result:

$$F_{LB} = C_{LB}\rho\frac{V^2}{2}\frac{\pi D_B^2}{4} \qquad (3\text{-}4)$$

3.1.5 Immersed weight of anchor boulder (W')

The immersed weight of the anchor boulder W' is equal to the dry weight of the boulder minus the upward buoyancy force. It is given by:

$$W' = \frac{\pi D_B^3}{6}\rho g(S_S - 1) \qquad (3\text{-}5)$$

Where S_S is the specific gravity of the anchor boulder (≈2.65).

3.1.6 Frictional force resisting sliding (F_F)

For the anchor boulders to remain stationary, the critical frictional force may be given by:

$$F_F = (W' - F_{BL} - F_{LB}) \tan \phi \tag{3-6}$$

Where ϕ is the friction angle of the boulder on the streambed, F_{BL} is computed using Equation (3-1) and F_{LB} using Equation (3-4). The value of ϕ can be estimated from the angle of repose. The U.S. Bureau of Reclamation (referenced in Henderson, 1966) present a value of $\phi = 40°$ for rock with a diameter greater than 0.1 m. This frictional force is considered to be conservative, and does not consider the effect of partial burial of the boulder on the resisting force.

3.1.7 Factor of Safety

A factor of safety (*FS*) is defined as the ratio of the resisting forces divided by the driving forces. Values of $FS > 1.0$ indicate that the structure is stable, and, conversely, a value of $FS < 1.0$ indicates that the structure would not be stable. In the case of our single-LWD structures, two *FS* can be defined for two different modes of failure; (1) sliding of the anchor boulder and structure may be experienced in a horizontal direction, and (2) lifting of the anchor boulder and structure in the vertical direction.

Isolating the forces acting on the anchor boulder in a horizontal direction (Figure 3-1), a factor of safety with respect to sliding FS_S may be defined.

$$FS_S = \frac{F_F}{F_{DL} + F_{DB}} \tag{3-7}$$

For values of $FS_S < 0$, the immersed weight of the anchor boulder(s) is not sufficient to counter the buoyancy force transferred from the log, and the LWD will float when submerged. This leads us to our second FS with respect to buoyancy.

In a similar fashion, the forces acting on the anchor boulder in a vertical direction may be isolated to obtain a factor of safety against buoyant failure FS_B:

$$FS_B = \frac{W'}{F_{BL} + F_{LB}} \tag{3-8}$$

For design purposes, Equations (3-8) and (3-9) must yield values of FS_S and FS_B greater than 1.0 that provide an acceptable level of safety to the designer. In general, for flow velocities in excess of about 2.0 m/s, the FS_S will govern the design. However, both FS_S and FS_B should be verified. These computations can easily be programmed in a computer spreadsheet or in a hand held calculator.

3.2 Multiple-LWD Structures

The basic form of a multiple-LWD structure, also known as triangular logjam, consists of two logs that are attached to trees or stumps on the bank, and that are both ballasted by common anchor boulders in the stream (Figure 1-2b). The basic multiple-LWD structure is inherently more stable than the single-LWD structure since lateral bracing, provided by its triangular shape, resists drag forces exerted by the flow. As illustrated in Figure 1-2b, once the basic triangular structure has been constructed, additional LWD and root wads may be added to increase the cover and habitat potential of such structures.

Compared to the single-LWD structures, the stability analysis for the multiple-LWD structure is relatively simple. Assuming that the lateral structural stability provided by its triangular construction effectively resists the horizontal drag forces, ballast is required only to prevent the structure from floating during high flows. Hence, the need to satisfy only a factor of safety with respect to buoyancy (FS_B):

$$FS_B = \frac{W'}{\sum F_{BL} + F_{LB}} \tag{3-9}$$

Equation (3-9) is essentially the same as Equation (3-8) except that the sum of the buoyancy forces (F_{BL}, Equation (3-1)) from each LWD piece forming the structure must be taken. Note that a BF = 2 must be used for LWD pieces (including root wads) that are completely within the stream since these pieces transfer their buoyancy forces to the in-stream end of the main structural members of the structure (Figure 3-3).

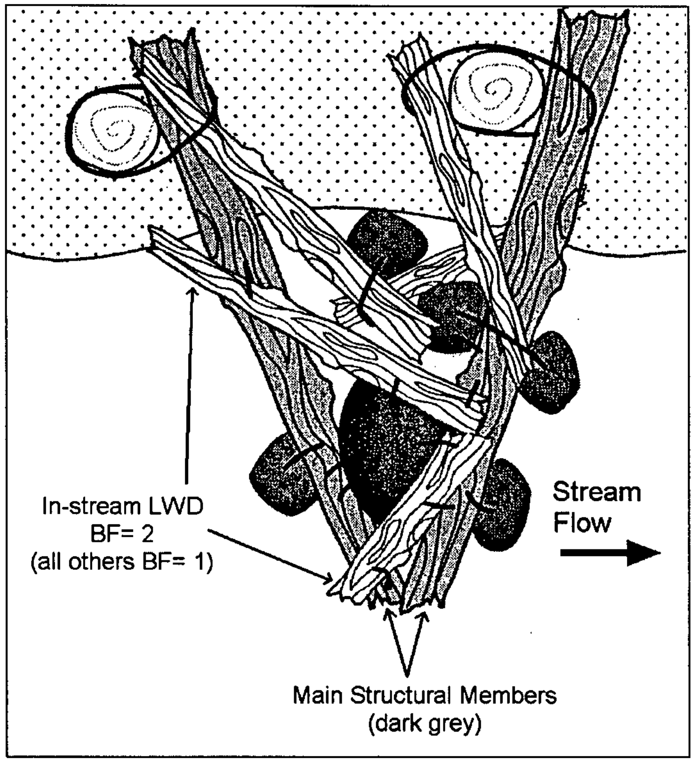


Figure 3-3: Use of *BF* with Multiple - LWD Structures

3.3 Application of Analysis to this Study

The theoretical analysis presented above was developed for design purposes with the assumption that the LWD is fully submerged. However, in order to test the validity of the design approach, the factors of safety, and respective stability, must be evaluated under partially submerged conditions. Two methods were utilised as part of this study:

1. Simplified method as presented in sections 3.1 and 3.2 substituting L with L_S (submerged length of LWD). This method assumes that while the structure may not be fully submerged, the buoyancy and drag forces on the submerged portion of the LWD will be equally distributed among bank and stream anchors. The weight of LWD above water is not considered in the analysis.

2. Full method of moments to describe, as accurately as possible, the distribution of buoyancy and drag forces among anchors. The weight of LWD above water is taken into account in computing the factor of safety.

The full method of moments involves simple substitutions as follows. Equation (3-1) is replaced by:

$$BF = 1 \qquad F_{BL} = \left[L_S \left(1 - \frac{L_S}{2L} \right) - 0.5L\ S_L \right] \rho g \pi \frac{D_L^{\ 2}}{4} \qquad (3\text{-}10)$$

$$BF = 2 \qquad F_{BL} = \left[L_S - L\ S_L \right] \rho g \pi \frac{D_L^{\ 2}}{4}$$

And Equation (3-2) is replaced by:

$$BF = 1 \qquad F_{DL} = C_{DL}\ \rho \frac{V^2}{2} L_S \left(1 - \frac{L_S}{2L} \right) D_L\ Sin(\beta) \qquad (3\text{-}11)$$

$$BF = 2 \qquad F_{DL} = C_{DL}\ \rho \frac{V^2}{2} L_S\ D_L\ Sin(\beta)$$

Where L_S is the submerged length of LWD, L is the full length of LWD and all other symbols have been defined previously.

To apply these two methods, it was necessary to determine the submerged length of LWD. For partially submerged LWD and $(BE - SE) > D_L$, the following formulae were used to estimate an equivalent submerged length of LWD:

$$SE - D_L < WS < SE \qquad L_S = 0.5\,L\frac{(WS - SE + D_L)}{(BE - SE)} \tag{3-12}$$

$$SE < WS < BE - D_L \qquad L_S = L\frac{(WS - SE + 0.5D_L)}{(BE - SE)}$$

$$BE - D_L < WS < BE \qquad L_S = L - 0.5\,L\frac{(BE - WS)}{(BE - SE)}$$

Where *SE* is the stream-end elevation of the LWD, *BE* is the bank-end elevation of the LWD and *WS* is the water surface elevation.

3.4 Single-LWD with Intact Root Wad Structure

The theoretical analysis for design of single-LWD structures with intact root wads (Figure 1-3) is similar that developed for single-LWD structures. Once again, the approach is centred on the forces transferred to the anchor boulders.

The forces that will be considered are:

- Vertical buoyancy forces acting on the LWD and associated root wad and transferred to the anchor boulders (F_{BL}).
- Horizontal fluid drag force acting on the root wad and transferred to the anchor boulders (F_{DRW}).
- Hydraulic frictional forces acting on the LWD (aligned with the flow) and transferred to the anchor boulders (F_{FL}).
- Horizontal fluid drag force acting directly on the anchor boulders (F_{DB}).
- Vertical lift forces acting directly on the anchor boulders (F_{LB}).

- Immersed weight of the anchor boulders (W').
- Frictional forces at the base of the anchor boulders that resist sliding (F_F).

As in the case of the single and multiple-LWD structures, this analysis assumes that the LWD is fully submerged within the flow.

3.4.1 Buoyancy forces acting on LWD (F_{BL})

Based on the assumptions that the root wad has the geometry of a cone, and that the root wad has a porosity of about 20% (based on visual field survey of root wads), buoyancy forces transferred to the anchor boulders may be estimated by:

$$F_{BL} = \left(\frac{\pi D_L^2 L}{4} + \frac{1}{3} \frac{\pi D_{RW}^2 L_{RW}}{4} 0.80 \right) \rho g (1 - S_L) \tag{3-13}$$

Where L is the length of the log (m); D_L is the average log diameter (m); D_{RW} is the average root wad diameter (m) and L_{RW} is the estimated length of the root wad (m). Refer to section 3.1.1 for selection of S_L values.

3.4.2 Fluid drag forces acting on root wad (F_{DRW})

Assuming that the surface area subject to drag is a disk of diameter D_{RW}. Hence, the drag force on the root wad transferred to the anchor boulders can be written as:

$$F_{DRW} = C_{DRW} \frac{\pi D_{RW}^2}{4} \frac{V^2}{2} \rho \sin(\beta) \tag{3-14}$$

Where V is the average flow velocity (m/sec), β is the angle of the root wad face with respect to the direction of flow (conservatively assumed to be 90° in most cases), and C_{DRW}

is the root wad drag coefficient. C_{DRW} is expected to be similar to that of a circular plate suspended in flow where $C_D = 1.2$ for values of $R_d = 10^4$ to 10^6 (Hoerner, 1965). This value is similar to that of a cube resting on a flat surface where $C_D = 1.05$ (Hoerner, 1965) and slightly more conservative than a value of $C_D = 1.0$ recommended by Petryk et al. (cited by Abbe and Montgomery, 1996) for flow through standing (living) vegetation.

3.4.3 Fluid frictional forces acting on the log (F_{FL})

Water flowing along the axis of the log will induce some skin friction onto the LWD. Neglect the fact that the log will be partially sheltered from the flow (as it lies in the wake of the root wad), the skin friction may be estimated by:

$$F_{FL} = C_{FL} \pi D_L L \frac{V^2}{2} \rho \tag{3-15}$$

Where $C_{FL} = 0.004$ for a smooth cylinder (Hoerner, 1965). The value of C_{FL} would be greater for a rough cylinder, however no representative values were found. Based on Equation (3-15), the skin friction exerted on the LWD may be neglected, as it is much smaller than the drag forces exerted on the root wad and anchor boulders. In cases were branches remain on the LWD, friction/drag forces may become significant (refer to Gippel et al., 1992).

3.4.4 F_{DB}, F_{LB}, W' and F_F

The remaining forces acting on the anchor boulders can be computed from Equations (3-3) through (3-6). In computing the drag and lift forces acting directly on the anchor boulders, it is assumed that four boulders are provided as ballast (Figure 1-3).

3.4.5 Factors of Safety

As with the single-LWD structures, two factors of safety must be considered: (1) against sliding of the LWD and anchor boulders, and (2) against the buoyant uplift of the LWD. A factor of safety less than 1.0 indicates unstable conditions i.e. sliding or floating of the LWD and anchor boulders will occur.

Isolating the forces acting on the anchor boulders in a horizontal direction a factor of safety with respect to sliding FS_S may be defined.

$$FS_S = \frac{F_F}{F_{DRW} + F_{DB} + F_{FL}} \qquad (3\text{-}16)$$

Where F_F is computed using Equation (3-6), F_{DRW} using Equation (3-14), F_{DB} using Equation (3-3) and F_{FL} using Equation (3-15).

Isolating the forces acting on the anchor boulders in a vertical direction, a factor of safety against buoyant uplift may be defined by Equation (3-8).

In general, for flow velocities in excess of about 2.0 m/s, the FS_S will govern the design of single-LWD with root wads. However, both FS_S and FS_B should be verified. These computations can easily be programmed in a computer spreadsheet or in a hand held calculator.

4 METHODOLOGY

A field program was developed and implemented to verify the adequacy of the stability analysis described in Chapter 3. Unlike previous monitoring studies that were statistical in nature, this program was formulated to determine the theoretical factors of safety of LWD structures and to compare the observed stability to the theoretical value. The following chapter provides details on the methods utilised to undertake this study. Descriptions of the field methods, hydrology, hydraulics and analytical methods are provided.

4.1 Field Assessments

In order verify the adequacy of the proposed design guidelines, field assessments of over 80 constructed LWD structures, located within seven different rivers and streams, were undertaken. All of these river systems have undergone rehabilitation, including placement of LWD in the summer of 1997. The first set of assessments was undertaken after the initial construction of the structures, between August 20 and October 31, 1997. The second set took place after the annual fall to spring peak flows, between March 02 and July 10, 1998. Table 4-1 summarises the distribution of structures among the 7 river systems and reaches. A brief description of each river system is provided in Chapter 5, Study Sites.

The structure types assessed were represented by multiple-LWD structures [including triangular-LWD structures, complex structures (without triangular bracing and/or did not fall under other categories) and V-type structures (spanning the channel)], single-LWD structures, and single-LWD with intact root wad. Eleven of the structures from the Keogh River were not investigated as part of the initial assessments in the summer of 1997. These structures were observed to have shifted during the high winter flows and were added to the sample set during the follow-up assessments. The information with respect to these additional structures [6 single-LWD, 4 complex (double-log longitudinal cover) and 1 triangular-LWD structure] was collected with the help of the field biologist responsible for the planning and design of the structures.

Table 4-1: Distribution of structures among sites

River System / Reach		No. of LWD Structures[a]				
		C	R	S	T	V
Keogh River	Wolf Creek	1			4	2
	103-104	5		7	5	1
	Tributary 13			2	2	
	West Main	1		6	2	
	West 80		1	3	2	
Lost Creek					2	
Lukwa Creek		1			1	
Sampson Creek		1			7	
San Juan River		2	7	1		
Shovelnose Creek		4	1	3	2	
West-Kettle River		1			7	
Sub-totals		16	9	23	34	3

[a] C: complex, R: LWD w/ Root wad, S: Single log, T: Triangular and V: V-type structures.

The field assessments included the collection of qualitative and quantitative information with respect to each LWD structure location. This information was entered on a LWD Structure Assessment Form (sample provided in Appendix B).

A two-pin method, similar to that described by Koonce (1990), was established as reference points for the surveying. Two 10-cm nails were driven in the base of stumps or trees (most often red alders) in the vicinity of the LWD structure. To facilitate triangulation measurements (explained below) within small streams, the pins were generally situated on the bank opposite the structure. On larger systems, the pins are on the same or closest bank to the structure. Our convention was to number the upstream pin as Pin #1 and the downstream one as Pin #2. For each structure, orange fluorescent flagging tape marked with the structure and pin number was used to clearly identify the pins. The location of the pins was also drawn on the reach detail sketch. Pin #1 served as a local benchmark at an assumed elevation of 100m. A typical field assessment layout is illustrated in Figure 4-1.

All of the structural components of the structures were located on a horizontal plane through triangulation with the two pins. The distance between each pin and the ends of each piece of LWD was measured using a 30m tape. If the structure included a rootwad, the measurements were to the high point of the root wad and the butt (cut) end. Measurements from both pins to the centre of each anchor boulder were also recorded. A rod and level were used to measure the elevation, relative to the local benchmark, of each LWD and anchor boulder. The high point of each boulder was recorded while the crown elevation of both ends of each LWD was read. Furthermore, the average diameter of each LWD, average diameter of root wads and the a, b and c-axis measurements of each anchor boulder was taken. When possible, the type of trees used in the structure was noted.

Between 3 and 5 cross-sections were surveyed at each structure with the centre-line section taken along a tape strung across the stream from the centre-line pin (often taken as one of Pin #1 or Pin #2). Elevations were recorded with an accuracy of 0.5 cm. Additional sections were measured upstream or downstream of the centre-line section. On wide river systems (greater than 100m bankfull width), full cross-sections were taken using stadia measurements. Longitudinal profiles along the centre-line of the bankfull channel, in the vicinity of the structures assessed, were recorded using stadia measurements both upstream and downstream of the LWD structures.

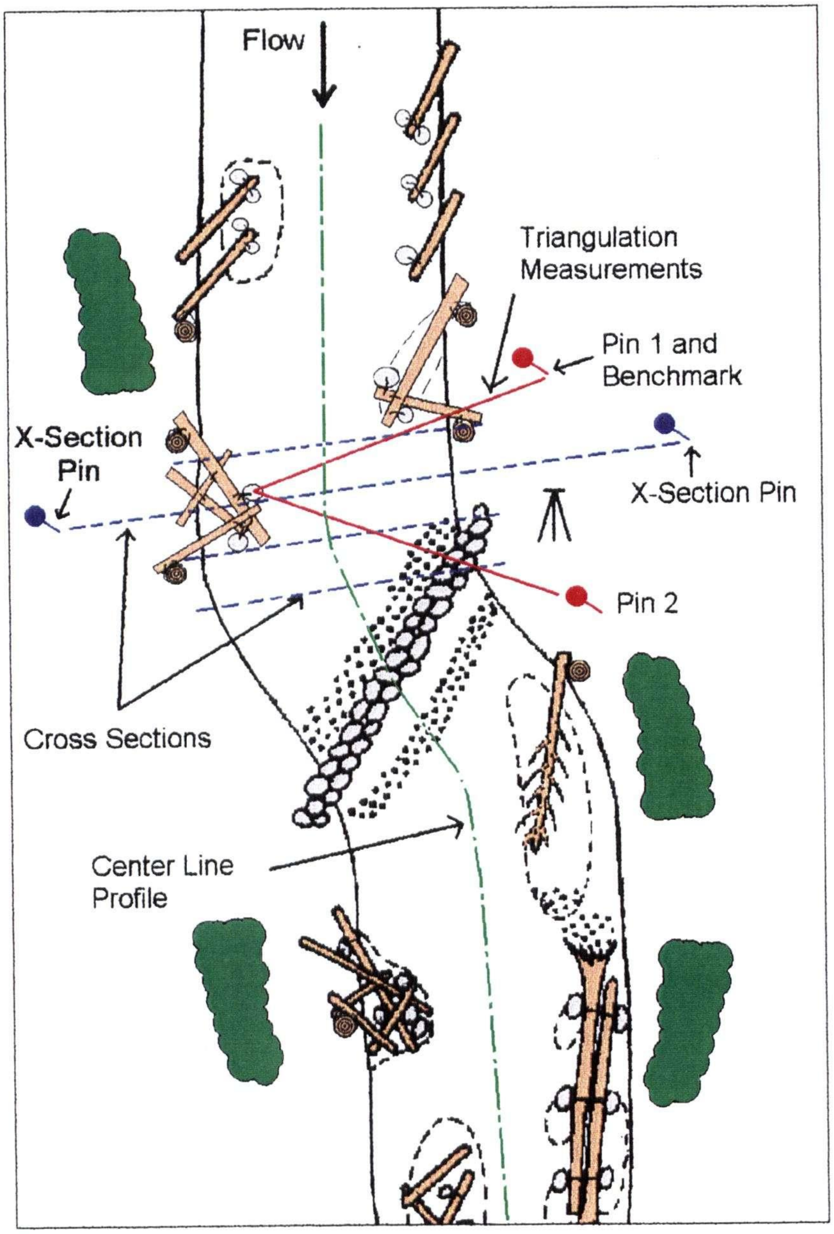


Figure 4-1: Typical field assessment layout

The bed and bank materials were assessed both qualitatively, through a visual estimation of the dominant and sub-dominant materials and the size of the largest mobile material, and quantitatively, through the measurement of the b-axis of 50 randomly selected pebbles.

A number of crest gauges were installed in the Keogh River and Sampson Creek in an effort to record peak water surface elevations. They were constructed of 3/4" clear vinyl tubing clamped onto trees adjacent to (sometimes overhanging) the stream. Small styrofoam beads were left floating on the water within the tubing. At high flows, the foam was floated up and stuck to the wall of the tubing indicating the crest level. All gauges were identified with red fluorescent flagging tape. To reset the gauges, a small stream of water was allowed in the tubing from the top. Foam beads then dropped to the existing water surface.

The type of riparian vegetation and relative overhead cover it is contributing to the LWD structures was recorded along with the in-stream cover provided by the LWD and boulders themselves.

Sketches of the reach and structure were prepared in order to complement the photographs of the structures. These sketches include major in-stream and bank features such as; steep and undercut banks, bars, riffles, pools, scour holes, LWD, boulders, cables and fasteners, trees, bank vegetation, pin locations, flow direction, centre-line cross-section, north orientation, roadways/railways and side channels.

Upon return to the sites after the fall/winter high flows, pin measurements were taken of the anchor boulders and in-stream end of the LWD. If movement was detected from the measurements, pin measurements of bank ends were also taken. All cross-sections were resurveyed along with the elevation of the LWD structure members that had apparently moved. High water marks, generally composed of small twigs and leaves caught on shrubs, were detected and surveyed along with the crest elevations of pools in the vicinity of the LWD structures. When necessary, additional profile information was collected in order to supplement the data from the initial assessments. Sketches along with notes of any apparent change to the structures and reaches were noted.

4.2 Hydrology

Field data were utilised to systematically compute a number of parameters necessary to predict and evaluate the stability of the LWD habitat structures. The stability predictions were made for three flow conditions, high flow between assessments (fall to spring), bankfull flow and design flow conditions. Bankfull flow conditions were assumed to occur with a 2.33-year return period. Design flow conditions were determined for a 50-year return period flow. This last criterion is often stated as providing a reasonable probability for obtaining at least 20 years of functional durability from fish habitat projects (Slaney and Martin, 1997).

For the gauged systems - Keogh River, Trapping Creek (Lost Creek receiver), San Juan River and the West-Kettle River - bankfull and design flows were obtained from a flood-frequency analysis undertaken on the historical maximum annual instantaneous discharges. A Gumbel distribution was assumed. The gauges respectively had 8, 31, 31 and 21 years of available records. For the West-Kettle River, the average values between the gauges located at Carmi Creek and Westbridge were used in the computations.

Since the gauges on both the Keogh River and Lost Creek are located a significant distance downstream from the rehabilitated reaches, adjustment of the peak flows was undertaken using the following relation (Harris, 1986):

$$Q_U = Q_G \left(\frac{A_U}{A_G} \right)^n \tag{4-1}$$

Where Q_U, A_U and Q_G, A_G are the peak flows and drainage areas at the ungauged and gauged locations respectively and n is an adjustment factor. A typical value of $n = 0.75$ was used for Lost Creek whereas an $n = 0.90$ was used for the Keogh River to reflect the dampening effects of the Keogh, O'Connor and Muir lakes present within the watershed.

Flow information for Shovelnose Creek originated from a regional analysis undertaken by Hay & Company (1995) based on historical stream flow information from the Squamish and Elaho River gauges. Mean annual maximum instantaneous flows for Lukwa creek was estimated based on information from nearby stations (Nimpkish River, Tsitika River, Catherine Creek and tributaries and Salmon River stations). Larger flood flows for Lukwa Creek were estimated by applying the Creager C values from the Tsitika River below Catherine Creek to the Lukwa watershed (Northwest Hydraulics Consultants (NHC) and Alby Systems, 1996). As for Sampson Creek, the average geometry of the channel within the reach of interest was used to estimate a bankfull flow value. The design discharge for Sampson Creek was calculated by multiplying the bankfull discharge by 2.3; where 2.3 is taken from $Q_{2.33}/Q_{50}$ for Shovelnose Creek, a watershed similar in nature to Sampson Creek.

Stream flow records were obtained from the Water Survey groups of Environment Canada and the BC Ministry of the Environment Lands and Parks, for the 1997-1998 fall to spring period. These records indicate that between the two survey periods the study streams were subject to floods ranging from 1.1 to 2.1-year return periods. The recorded peak flows and associated return frequency for the 4 larger systems are tabulated in Table 4-2. Collection of high water marks (accumulated leaf litter and small twigs) and crest gauge information indicated that very low intensity floods took place within Shovelnose Creek and Sampson Creek between assessments. Some observations (smothering of boulders in sand and fresh high water marks) within Shovelnose Creek lead us to believe that a more significant flow event took place in early fall (prior to first assessment) and may have caused some "settling-in" of structures. The average or slightly below average peak flows observed may reflect the El Niño weather fluctuations which were well developed during the study period.

Table 4-2: Recorded peak flows (fall 1997 - spring 1998)

River System	Flow (m^3/s)	Date	Return Period (yr.)
Keogh River	124	Dec. 14, 1997	2.1
Lukwa Creek	38[a]	N/A	> 2.33
Sampson Creek	9[a]	N/A	< 2.33
San-Juan River	560	Dec. 16, 1997	1.1
Shovelnose Creek	33[a]	N/A	< 2.33
West-Kettle River	124	May 3, 1998	2.0
Trapping Creek	13.8	May 4, 1999	1.4

[a] Average of computed high-water flows (no records obtained)

4.3 Hydraulics

Discharges for the inter-assessment high water conditions, and the depths of flow for the bankfull and design discharges were computed based on uniform flow hydraulics. To undertake these computations, estimates of flow or water level, geometry and roughness were necessary. For each structure, an equivalent trapezoidal geometry was fitted by eye to the surveyed bankfull cross-section. The average velocities and flow depths for the various flow conditions were computed using Manning's formula. The design depth and average velocities were computed by projecting the equivalent trapezoidal cross-section along the side slopes. A sample cross-section and equivalent trapezoidal section is illustrated in Figure 4-2.

The Manning's roughness (n) was approximated based on an initial estimate of the grain roughness (n') computed from:

$$n' = 0.038\, d_{90}^{1/6} \tag{4-2}$$

Where d_{90} is the 90th percentile diameter (m) of the stream's substrate sampled by the pebble count. This initial value of n was increased to reflect additional roughness from bed forms, sinuosity and LWD constriction. When available, recorded peak discharges were used in the final adjustment of roughness values. The range of Manning's n used was between 0.026 (San-Juan River) and 0.060 (Wolf Creek Reach with significant LWD constriction) with an average of about 0.045. The resulting velocities for the three discharge conditions of interest were compared with the method suggested by Millar (1997) based on the Chezy equation.

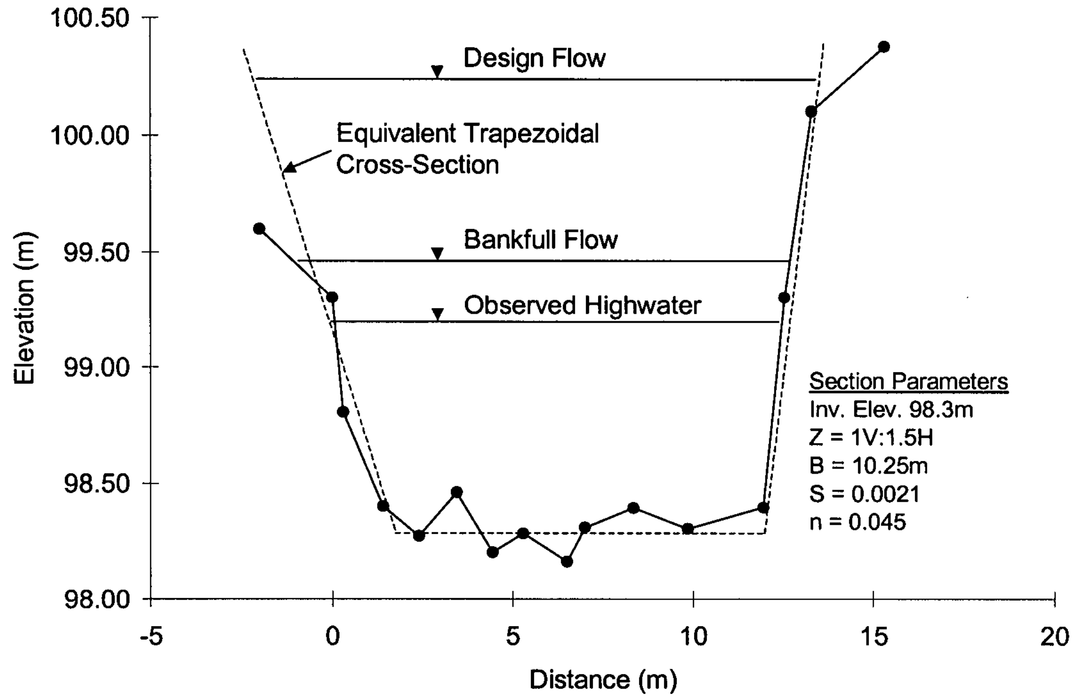


Figure 4-2: Sample cross-section (centre-line structure S-1, Sampson Creek)

Dimensionless shear stresses (τ^*) were computed for bankfull flow conditions from:

$$\tau^* = \frac{Y_B S}{(S_S - 1)d_{50}} \tag{4-3}$$

Where Y_B is the normal depth at bankfull flow, S is the channel slope, S_S is the specific gravity of the sediment and d_{50} is the average particle diameter.

The unit stream power (ω) was computed for bankfull flow conditions from:

$$\omega = V_B \, Y_B \, S \, g \, \rho \tag{4-4}$$

Where V_B is the average velocity at bankfull flow and ρ is the density of water.

4.4 Stability Analysis

Factors of safety were computed for the three flow conditions, past (fall-spring) high flow, bankfull flow and design flow conditions. The factors of safety against sliding failure FS_S and buoyant uplift failure FS_B of the LWD and boulder structures were computed using both a simplified method and a full method of moments as presented in Chapter 3.

Often, the type of LWD used in the construction of the structures could not be identified since no bark was present. In computing the factors of safety for these members, a S_L of 0.50 was assumed; this value represents the average S_L of most commonly encountered species (excluding cedar, which is easily identified) (Table 3-1).

Figure 4-3: Looking downstream at SJ-108 root wad and fine sand (Sept. '97)

Three single-LWD structures with root wads (SJ-106, SJ-107 and SJ-108) located within the San Juan River were very large spruce trees (diameter > 1 m at butt) that had been naturally recruited from cut banks located on outside bends. SJ-106 and SJ-108 were assessed in detail and found to have a considerable amount of fine sand within and around their root wads. Fine sand sloughed off the banks and embedded the bottom part of the root wads. Figure 4-3 illustrates SJ-108 and the fine sand present in and around the root wad. In an effort to reflect this in the computations of the factors of safety, it was estimated that 20% of the face of the root wads were embedded in fine sand and therefore not subjected to the flowing water. Hence, the area of the root wad used in computing the drag force for those structures was reduced accordingly. It was also estimated that 25% of the root wad volume (assumed to be a cone) was filled with fine sand of a density of 1,800 kg/m^3. Note that these methods were used to compute the expected factors of safety for existing conditions only.

Structures SJ-101, SJ-102, SJ-107, SJ-109a and SJ-109b were not subjected to full detailed assessments. SJ-101 and SJ-102 are root wads that sloughed off a cut bank and were anchored in a standing position (root wad facing down). Extensive amounts of fine sand and earth remains in the root wads and deep water prevented collection of good measurements. These were not considered further as they do not represented the structure types under investigation. SJ-107 was not assessed in detail since it was not accessible; it was situated in water deeper than 1.5 m. It underwent a photo assessment and computations were based on conditions at SJ-106. Due to time constraints, structures SJ-109a and SJ-109b were only subjected to simple assessments including photos, alignments and structural dimensions.

Ballast mass requirements for bankfull and design flow conditions were determined based on the design guidelines presented in Chapter 3 with $FS_S = 2.0$ and $FS_B = 1.25$ as recommended by Millar (1997). In contrast to the approach used to compute the factors of safety under observed high flow conditions, full submergence of the LWD was assumed to evaluate ballast mass requirements. This is a simplified approach that does not require water level estimates and will yield conservative results since it does not take into account potential LWD mass above water. Fine sand in and around root wads was ignored as it is expected to be washed away with time and will not provide added stability.

In order to compute the mass of anchor boulders, they were assumed to be spherical with a diameter equal to the geometric mean of the a, b and c-axis measurements. Furthermore, a specific gravity of 2.65 was used.

Subsequent to the follow-up assessments, the structures were described as:

Non functional: when they shifted considerably and were not in contact with the water at the time of the second assessment.

Partially functional: when they shifted considerably but remained in contact with the water at low flow. Although they may not be meeting their original objective, they were providing in-stream cover. The notation "partial" is also used since their stability may be precarious and future floods may render them non-functional.

Functional: when they were essentially intact, appeared to function as intended and still had the potential of achieving their intended objectives.

Note that it is difficult to comment on the fulfilment of objectives since they tend to vary between structures. The assessor does not always know the objectives and they may take some time to be achieved. For example some structures placed within Shovelnose Creek were meant to promote aggradation and speed-up channel narrowing, a process that may take years to achieve.

4.5 Analytical Methods

Predictions on the stability of the structures were based on the computed FS_S and FS_B from the observed high flow conditions. For single-LWD and single-LWD with root wad, a $FS_S < 1.0$ would indicate potentially unstable conditions whereas a $FS_S \geq 1.0$ would suggest stable conditions. In the case of multiple-LWD structures, the same approach was adopted based on FS_B (i.e. $FS_B < 1.0$: unstable and $FS_B \geq 1.0$: stable).

A criterion for movement of the various structures was established based on the measurement error involved in the triangulation measurements while allowing for some slight adjustment of the structural components (e.g. settling in of anchor boulders and tightening of cables). Within small to medium systems, it was possible to triangulate structural components to about 0.1 to 0.2 m depending on the density of riparian shrubs and complexity of the structure (e.g. pieces under water). Hence, a movement in excess of 0.5 m is thought to represent instability of the structure. For the San Juan River, the largest systems assessed, high vertical cut banks, dense riparian shrubs and the great distances to measure decreased the accuracy of the triangulation measurements. In some cases (SJ-103a, SJ-103b and SJ-104), temporary turning pins had to be laid out on a gravel bar to enable triangulation. In this system, the accuracy of the measurements was about 0.3 to 0.4

m. Therefore, within the San Juan River, a movement in excess of 0.7 m is thought to represent instability of the structure.

5 STUDY SITES

The following chapter provides a description of the seven study sites selected for the field assessments. The location of the sites along with their biogeoclimatic setting, hydrology, logging history and rehabilitative work will be described.

The location of each river system along with a 1:50,000 reference map number is provided on Figure 5-1. Table 5-1, provides a summary of the characteristics of the rehabilitated reaches for each site.

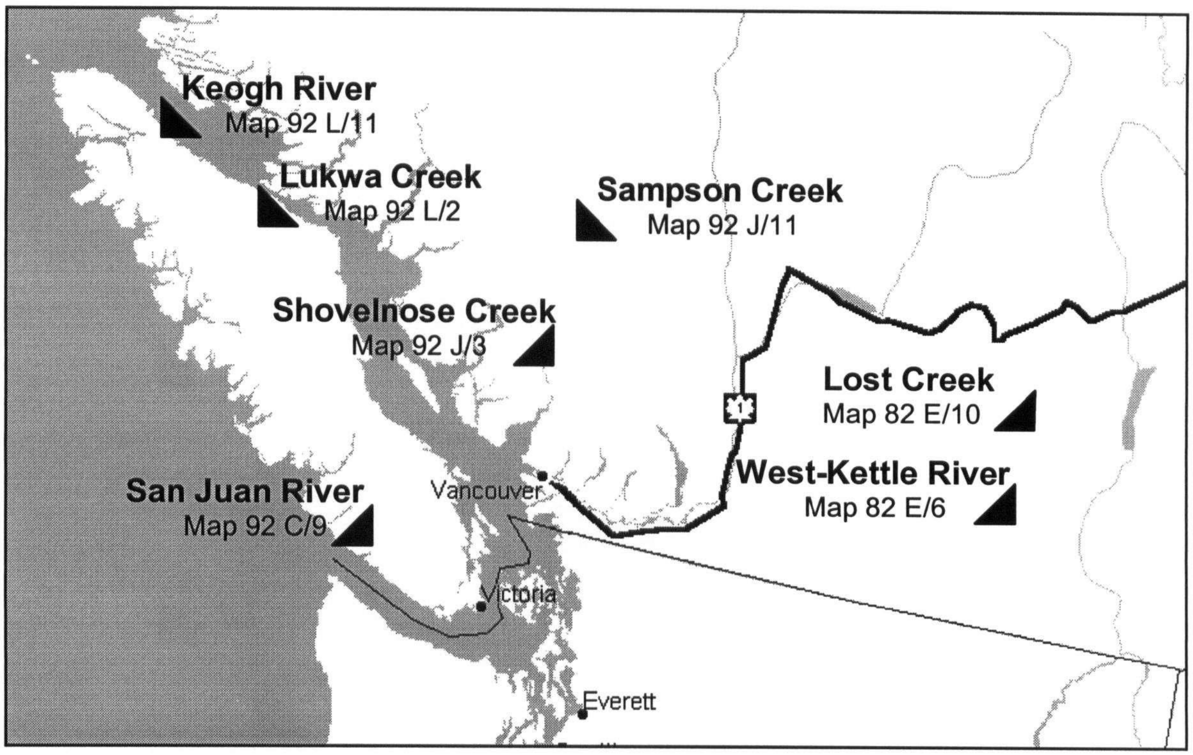


Figure 5-1: Site location map

Table 5-1: Summary of Site Characteristics

Reach		Trib. Area[a] (km^2)	Q_B (m^3/s)	W_B (m)	Slope (%)
Keogh River	Wolf Creek	19	22	11	1.5
	103-104	22	25	15-30	0.47
	Tributary 13	29	34	15-30	0.40
	West Main	39	46	18	0.90
	West 80	39	46	16	0.70
Lost Creek		45	7.2	8	2.3
Lukwa Creek		23	34	14	1.0
Sampson Creek		40	15[b]	12	0.21
San Juan River		730	795	120	0.036
Shovelnose Creek		21	45	38	0.45
West-Kettle River		1,520	116	44	0.28

[a] Tributary area to reach of interest
[b] Estimate based on uniform flow and reach-averaged bankfull geometry

5.1 Keogh River

The Keogh River is a 4th order coastal system draining a 135-km^2 area into the Queen Charlotte Strait on the northeast end of Vancouver Island. It is located between the towns of Port Hardy and Port McNeill (Figure 5-2). The physiography of the watershed is typical of the Coastal Western Hemlock biogeoclimatic zone, with shallow, easily disrupted soils. The upper part of the watershed is typically steep mountainous terrain while the lower portion is dominated by gentler terrain typical of coastal plains (Potyrala, 1997).

Stream flow in the Keogh is dominated by high rainfall during the fall/winter period, with low flows occurring in late summer. Mean annual precipitation is approximately 1700 mm/year.

Salmonberry bushes and other shrubs along with red alders and western hemlock dominate the riparian vegetation along the rehabilitated reaches. Some cedar patches are also present in the uppermost reaches. The dominant bed material within all of the reaches was cobble

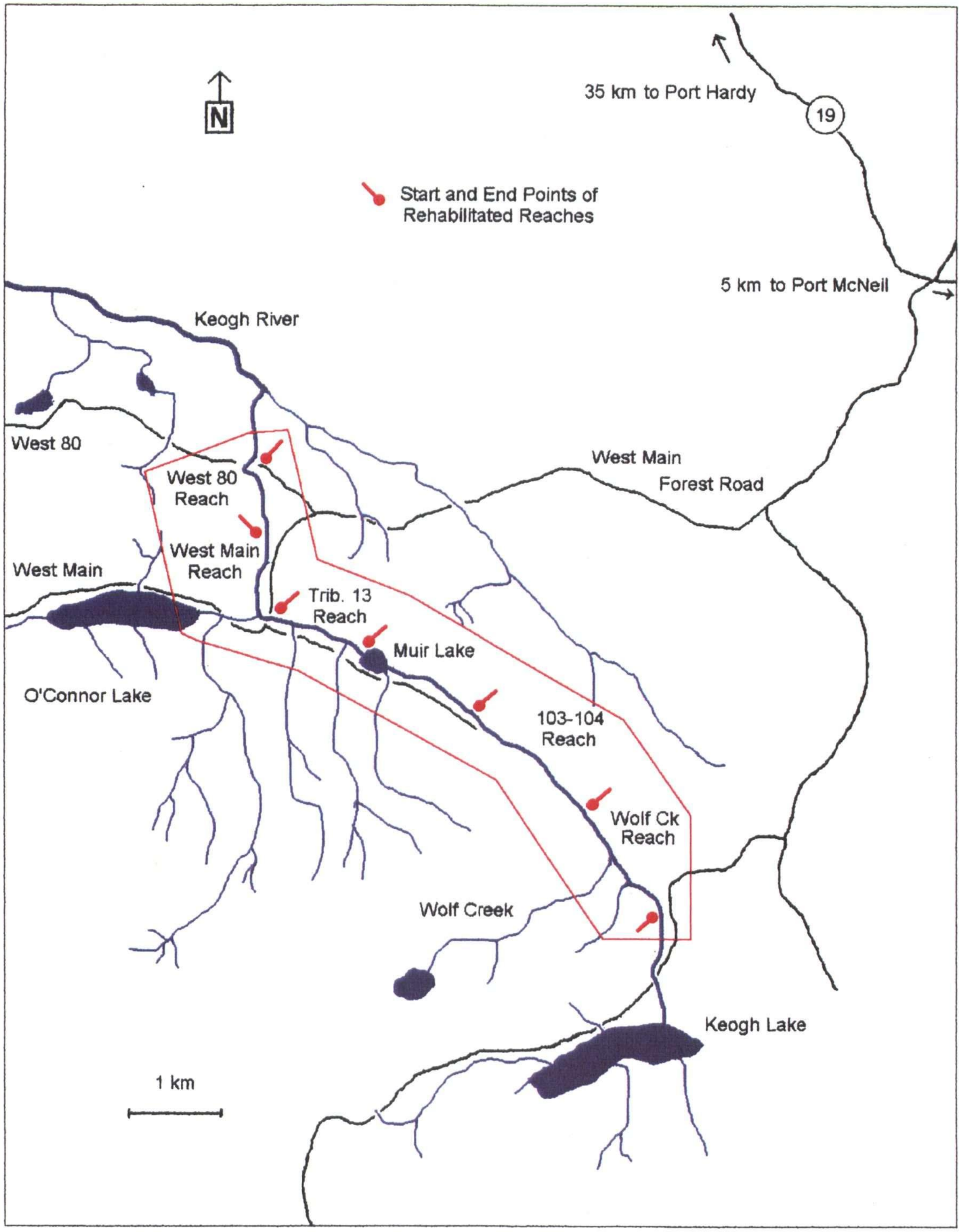


Figure 5-2: Keogh River

with gravel as the sub-dominant material. Approximately 53% of the basin has been logged since the 1940's, including 55 % of the flood plain and up to 70% of the sub-basins draining steeper terrain in the upper watershed. The riparian forest has also been harvested to a large extent. At least 23 km of main stem riparian area along with 26 km of tributaries have been harvested in the past. About 10 logging and road-related landslides have been identified in the basin.

The Keogh River is subject to frequent overbank flow events. All reaches assessed, except for the West Main reach, had isolated secondary overflow channels. Small pools in densely vegetated areas (located as far as 10 m from the low-flow channel) were often found to support young fry. Conversely, the West Main reach, located along the West Main forestry access road and embankment, was relatively well entrenched and did not provide significant side channel habitat.

The loss of riparian trees has had a detrimental effect on fish habitat in the Keogh River. Habitat assessments along the main stem Keogh River found that the presence of large trees within the river, and associated pool habitat, is at a critically low level (Potyrala, 1997). Salmonid populations utilising the system have been experiencing steady declines over the past 10 years. Because of the existence of a very good record of salmonid escapement and juvenile abundance data, the Keogh River has been selected as a main WRP site for stream channel habitat rehabilitation. The emphasis has been to undertake and evaluate stream habitat restoration techniques in order that they can be used as examples for similar projects elsewhere in the province, particularly on the coast.

Stream rehabilitation work has been previously undertaken on a small scale (3 km partially rehabilitated) in the Keogh, and included the installation of boulder clusters, log cover, and gabion weirs in the upper and mid-reaches in the late 1970's (Ward and Slaney, 1979). Experimental stream fertilisation was undertaken in the watershed in the early 1980's (Slaney and Ward, 1993). New experiments with slow release fertiliser pellets began in 1997 and will continue over the next few years (Ken Ashley, personal communication). The removal of obstructions to fish passage (usually beaver dams or debris) has been carried out periodically since the early 1960's.

Numerous habitat restoration works were performed along the Keogh River during the summer of 1996. A total of 119 LWD and boulders structures were installed in the upper part of the river including 92 wood and boulder habitat structures and 20 boulder clusters of various configurations. In 1997, a further 117 LWD and boulder habitat structures were built (Potyrala, 1998). The rehabilitation works span 5 reaches from the confluence of Wolf Creek down to the West-80 Bridge (refer to Table 5-1). The top three reaches of interest are 3rd order whereas the bottom two are 4th order systems.

5.2 Lukwa Creek

Lukwa Creek is a tributary of the Nimpkish River. It is located about six kilometres north east of the small community of Woss (Figure 5-3). Lukwa Creek is a 4th order stream with a total tributary area of approximately 54 km^2. Both east and west valley walls of the

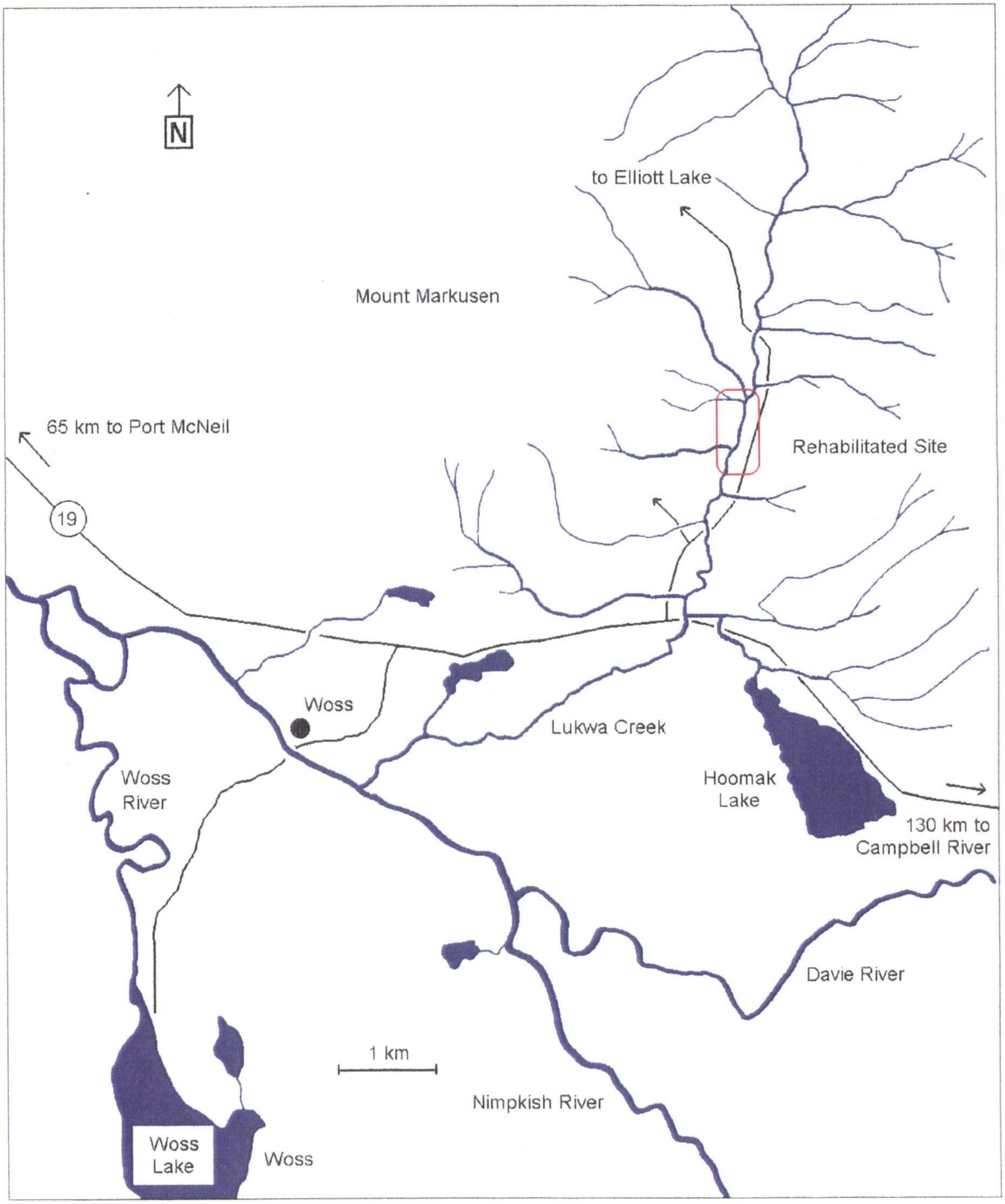


Figure 5-3: Lukwa Creek

Lukwa Creek are steep, rising to maximum elevations of approximately 1,200 m (NHC and Alby Systems,1996). Three biogeoclimatic zones occur within the watershed. The drier maritime Vancouver Island Coastal Western Hemlock zone is encountered at the lowest elevation, followed by the wetter sub-montane and montane maritime Coastal Western Hemlock zone, and finally the maritime forested Mountain Hemlock zone at the highest elevation.

Stream flows within the Lukwa are dominated by high rainfall during the fall/winter period, with low flows occurring in late summer. Mean annual precipitation is approximately 1400 mm/year.

Forest harvesting in the watershed began in the 1960's and by 1995, about 42% of the forested area had been logged. Logging activities began in the vicinity of the lowest reach of the creek, whereas in the last 20 years, harvesting has occurred up to the mid-slopes of the valley and in the upper watershed. The west side of the valley, including upper Lukwa Creek, has been identified as an area of active debris slides and flows (as cited by NHC and Alby Systems, 1996). In 1983, a large logging road-related failure entered the main creek from the west valley wall. Signs of high sediment loads such as extensive cobble-gravel bars, channel cut-offs and bank erosion are still visible and active throughout the rehabilitated reach.

The majority of the LWD structures built within Lukwa Creek have been implemented to enhance bank stability, and to arrest bank undercutting and channel migration. Since the two lowermost structures are similar to the habitat structures of interest they were included as part of our study.

5.3 Sampson Creek

Sampson Creek is a steep mountain creek with a watershed area of about 40 km^2 and is tributary to the Lillooet River. The 3^{rd} order stream runs from a swampy area near its mouth for a distance of about 11 km to its headwaters (elevation 2,390 m) (Figure 5-4). A total of four biogeoclimatic zones characterise the watershed. With increasing elevation are found, the southern Interior Douglas-Fir zone, the drier sub-maritime southern Coastal Western Hemlock zone, the wetter sub-maritime southern Coastal Western Hemlock zone and the forested Engleman Spruce - Sub-alpine Fir biogeoclimatic zone.

Stream flows in Sampson Creek are dominated by high rainfall during the fall period and rain-on-snow events in early summer. Low flows typically occur in winter and late summer. Mean annual precipitation is approximately 1000 mm/year.

The Sampson Creek watershed has undergone clear-cut logging, however, some buffer zones along the creek are evident. Since the early 70's, forest harvesting has claimed about 285 ha of forest or roughly 7% of the watershed area.

The lower portion of Sampson Creek flows parallel to the Lillooet River within the floodplain for approximately 5 km. Sections of the lower Sampson have been channelised, presumably as part of road construction, and is typically characterised as glide habitat (Anonymous, 1996). High fish values were determined in 1995 for coho and sockeye salmon and cutthroat trout while chinook salmon and rainbow trout were also found within the system (Anonymous, 1996).

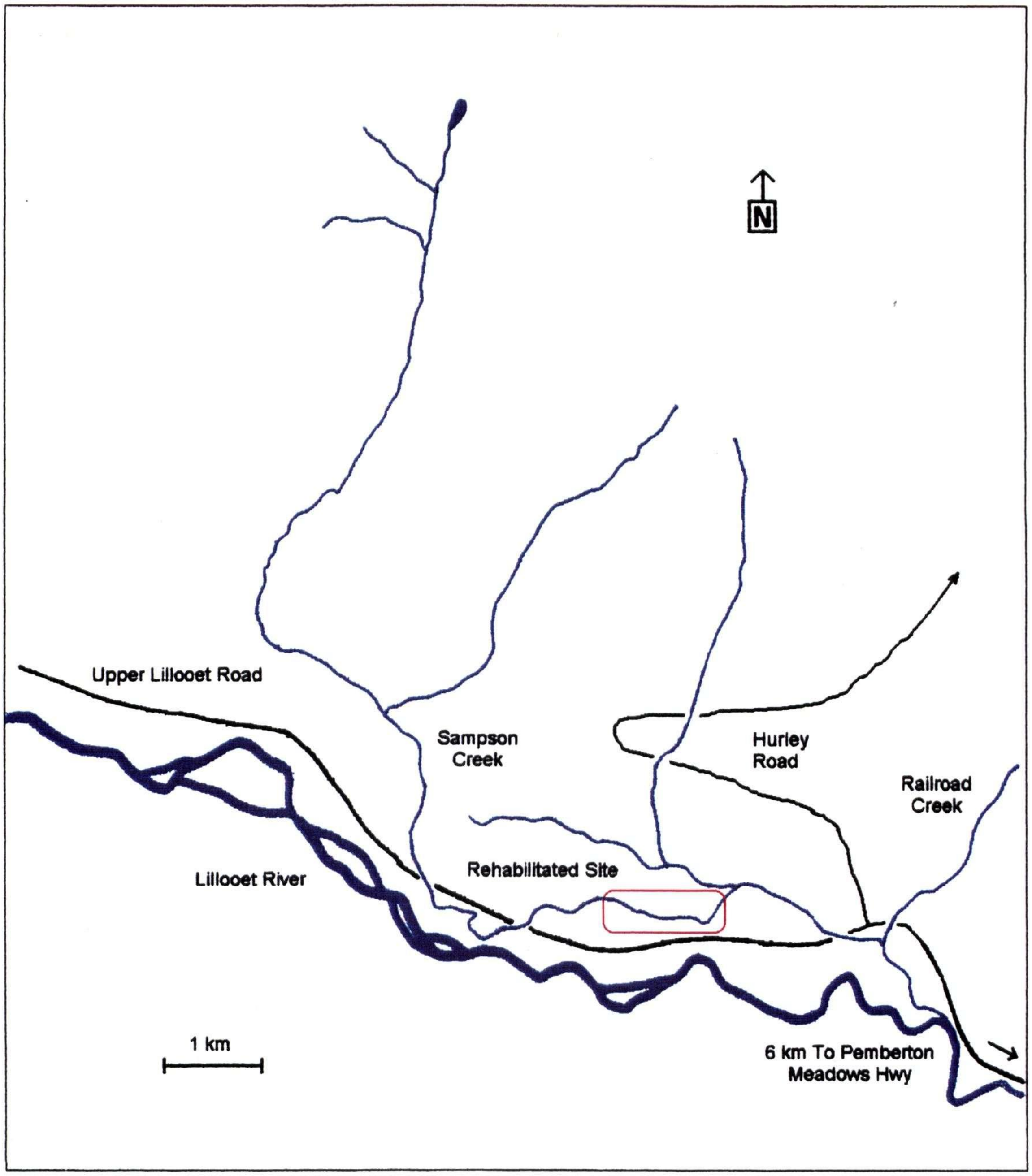


Figure 5-4: Sampson Creek

The riparian vegetation in the lowermost reach consists mainly of red alder, willow and cottonwood with some patches of western red cedar and western hemlock. Riparian area planting with coniferous species was undertaken in 1997 as part of rehabilitation efforts. Although no apparent channel instabilities have ensued from human impacts, the absence of quality riparian trees has resulted in the lack of in-stream LWD and channel complexity. A total of 8 habitat structures were constructed within the lower creek during the summer of 1997.

5.4 San Juan River

The San Juan River is a 5th order system draining about 730 km^2 of the Vancouver Island range located east of the community of Port Renfrew on Vancouver Island (Figure 5-5). The San Juan River and its tributaries lie within the Coastal Western Hemlock biogeoclimatic zone.

Stream flow in the San Juan is dominated by high rainfall during the fall/winter period, with low flows occurring in late spring and summer. Mean annual precipitation is approximately 2100 mm/year. The gauging station used to derive flow estimates is located 1.5 km downstream of the lowermost LWD structure.

The bottom of the San Juan valley was railway-logged prior to the 1950's. Since then, harvesting activities has moved into the tributaries and concentrates in the upper reaches. Roughly 25% of the watershed, which is mostly privately owned, has been harvested in the past 20 to 30 years. Second growth forests primarily composed of red alders with some mature coniferous species border the reach of interest.

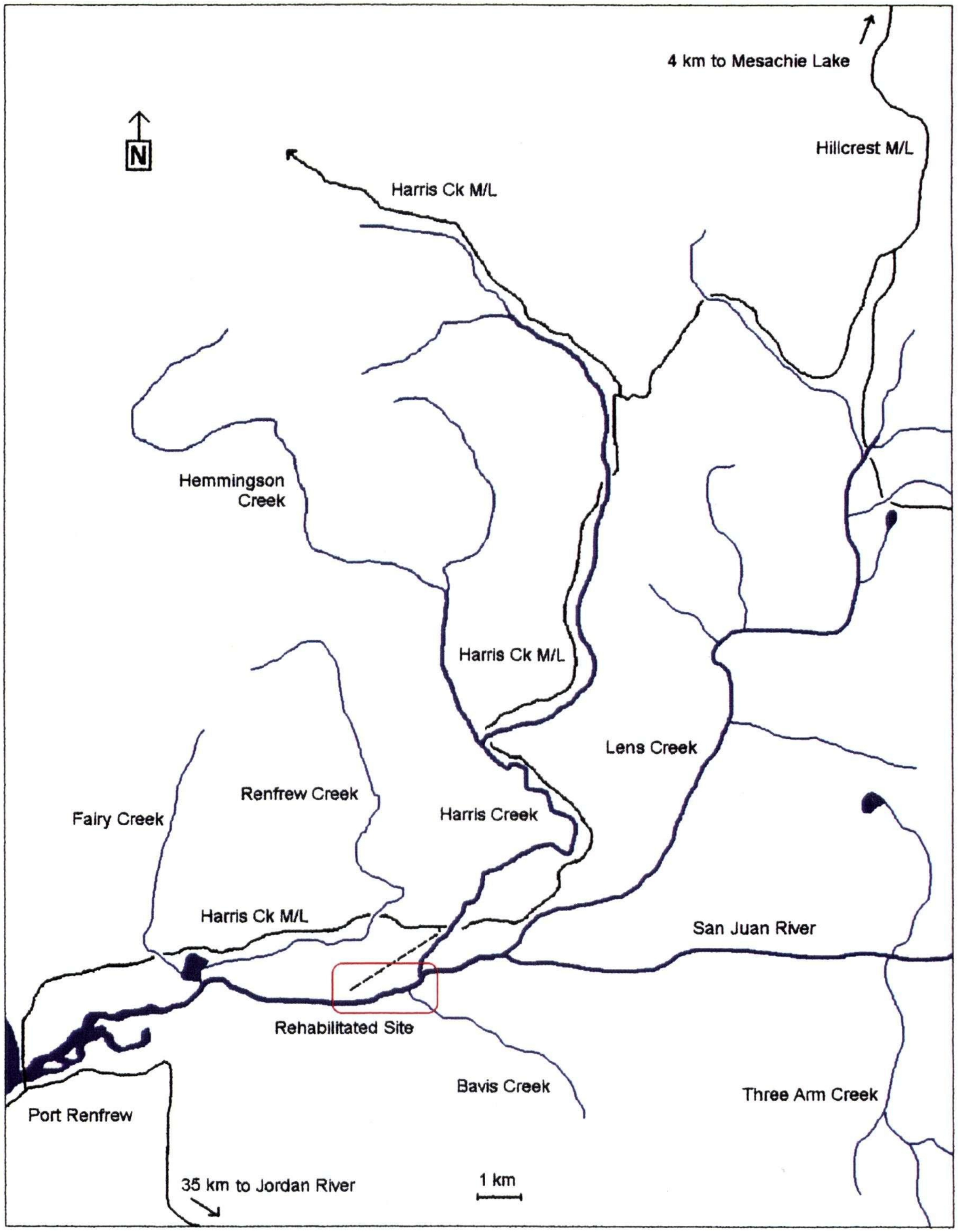


Figure 5-5: San Juan River

The San Juan River has received increased sediment supply as a result of logging in tributaries on the south and north valley sides and in the upper watershed. More than 250 known sediment sources related to forestry were identified within the San Juan basin throughout the 1970's. This number has decreased to less than 140 over the last 20 years (NHC, 1994).

Since 1980, the lower San Juan River has widened greatly and straightened, primarily as a result of sediment contributed from its tributaries, though riparian logging may have played a very minor role (NHC, 1994). Large bars in the channel take up most of the increased channel width. Secondary channels have disappeared through erosion, in-filling with sediment or blockage with logjams. The contribution of sediment to the San Juan River lags behind the landslide disturbance in the upper watershed. Sediment storage along lower Harris Creek and other major tributaries suggests that elevated supply to the San Juan River will continue for many years (NHC, 1994).

The San Juan River supports populations of coho, chinook, chum, remnant escapements of pink and sockeye salmon, steelhead (winter and summer run), and resident rainbow and cutthroat trout. The steelhead sport fishery has declined in the last decade and it is suspected that habitat degradation related to increased sediment production from roads and clear-cut blocks is the major cause (NHC, 1994).

The rehabilitation efforts undertaken within the San Juan River were experimental in nature and focused on potential stabilisation measures of in-stream LWD. Approximately 12 pieces of LWD, almost all recruited naturally through bank failure, were anchored in place using boulder and cable attachments flown in by helicopter. A total of 4 stumps and

9 LWD with intact root wads were anchored in order to provide bank and gravel bar stabilisation (through sheltering) along with some habitat benefits along the banks.

5.5 Shovelnose Creek

Shovelnose Creek is a 3rd order stream draining portions of Mt. Fee and Cypress Peak, to the Squamish River. Its 25 km^2 tributary area is located 45 km north of Squamish BC (Figure 5-6). Note that Turbid Creek is often mislabelled on maps (including the referenced 1:50,000 topographic map) as being Shovelnose Creek - Shovelnose is the southernmost creek.

Three biogeoclimatic zones are represented within the drainage basin. Both the drier sub-maritime southern Coastal Western Hemlock zone and the wetter sub-maritime southern Coastal Western Hemlock zone are encountered at the lower elevations, while the sub-maritime forested Mountain Hemlock Zone is found at higher elevations. The catchment consists of steep slopes with evidence of natural slope instabilities. Previous mapping of parts of the watershed indicated that more than 35% of the catchment is considered potentially unstable and about 20 landslides have been identified.

Stream flows in Shovelnose Creek are dominated by high rainfall during the fall period and summer rain-on-snow events. Low flows typically occur in winter. Mean annual precipitation is approximately 2000 mm/year.

Over the past twenty years, approximately 8% of the watershed, along the lower slopes of the basin north of Shovelnose Creek, has been logged (Steelhead Society, 1996).

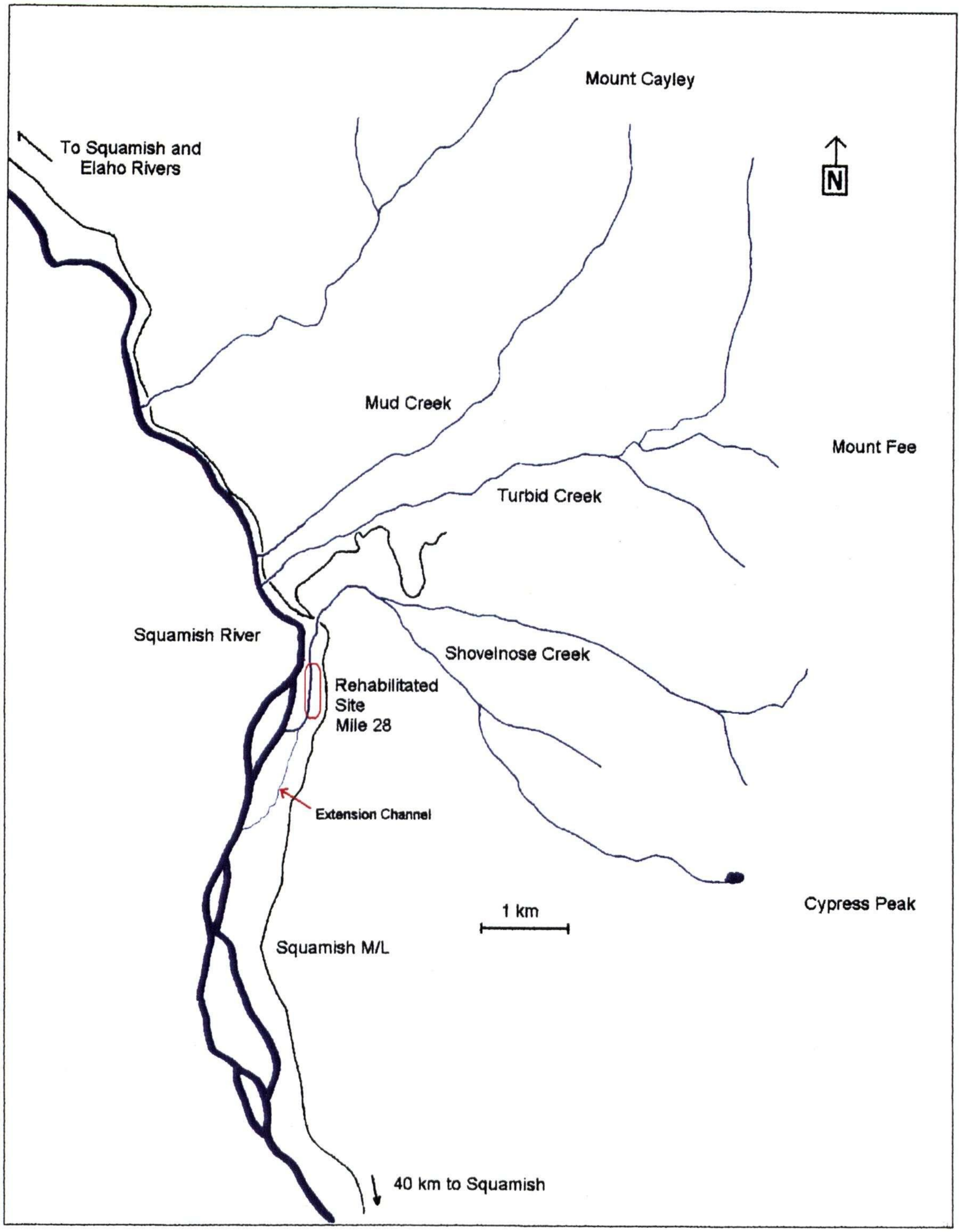


Figure 5-6: Shovelnose Creek

The anadromous zone of the creek spans 3.5 km of which 2 km used to lie in the floodplain of the Squamish River. Prior to the construction of a protective berm in 1994, infrequent flooding events within the Squamish River would send cold, silt-laden waters flowing through the lower reaches of Shovelnose Creek, altering the morphology and adversely impacting the fish habitat. This inundating effect has widened the creek and greatly reduced the number of pools.

In 1995 a number of habitat enhancement projects were undertaken including the construction of a 800m extension channel, a side channel and placement of artificial reef structures (Dave Duff, personal communication. In 1996, opposing wing deflectors were constructed in the lower reach of Shovelnose Creek upstream of the extension channel. In 1997, LWD structures were placed within the extension channel and in the main creek in the area rehabilitated with wing deflectors. Some of the structures placed in the creek were built for fish habitat whereas others were intended to aid in flow retardation and sediment deposition in an effort to narrow the creek.

5.6 West-Kettle River

The West-Kettle River is a 6^{th} order system draining a tributary area of about 1,870 km^2. The watershed extends north from its confluence with the Kettle River near the community of Westbridge. Four biogeoclimatic zones occur in the West-Kettle River watershed. Upper elevations belong to the Sub-alpine and Montane Spruce zones and lower elevations are in the Kettle Dry Mild Interior Douglas Fir and Interior Cedar-Hemlock zones. Mean annual precipitation is about 560 mm/year.

Stream flow hydrographs for the West-Kettle are snowmelt dominated with peak flows occurring in late May and early June. The stream flow gauging stations used in the frequency analysis are at Carmi, located near the confluence of Trapping Creek and the West Kettle River (about 22km upstream of the rehabilitated section), and Westbridge, near the confluence of the West Kettle and the Kettle River (about 18km downstream of the rehabilitated section).

The watershed supports a significant amount of land and resource-based activities including agriculture within the valley bottom and floodplain, cattle grazing and forestry. The majority of the valley lands downstream of Carmi are privately owned and the riparian vegetation has been adversely affected, with trees often being removed to the river's edge. Upstream of Carmi, lands bordering the river are primarily crown lands and the riparian areas are in good condition. Approximately 25% of the basin above Trapping Creek has been logged. The bulk of this harvesting has taken place in the last 30 years in order to control infestations of both the Mountain Pine Beetle and the Spruce Bark Beetle (Les Molnar, personal communication).

The West-Kettle River provides habitat for numerous fish species, including rainbow trout, eastern brook trout, mountain whitefish, northern squawfish, longnose sucker, sculpins, redside shiners and speckled dace (as cited by Timberland,1997).

The rehabilitated river section is located about midway between the communities of Westbridge and Beverdell (Figure 5-7). About 7 debris catcher type structures were constructed in the early 1990's (Griffith, 1991 as cited by Ward and Slaney, 1993). The original design was not effective at trapping debris and modifications were brought to the

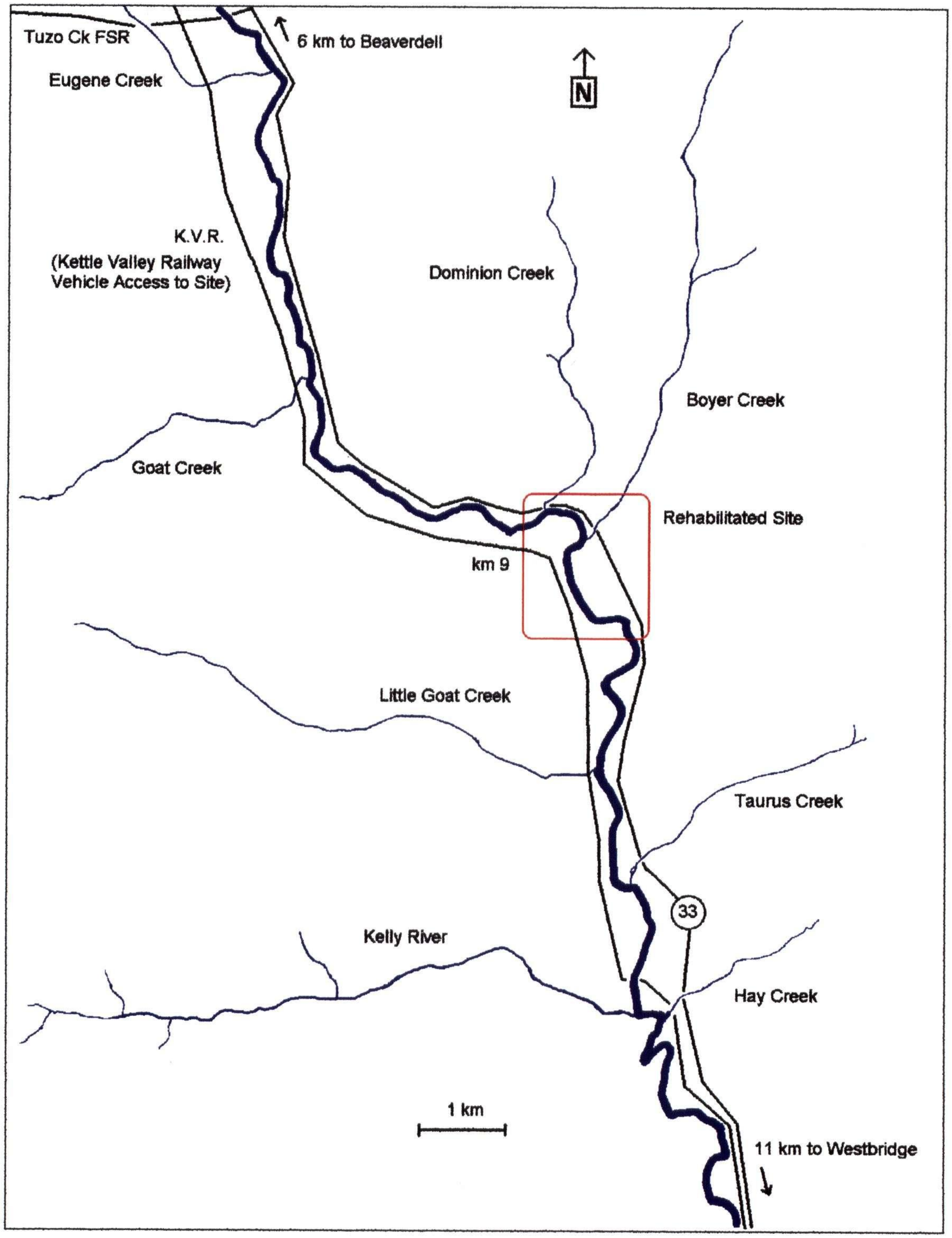


Figure 5-7: West-Kettle River

structures. By the summer of 1997, only one of these structures was still functioning as intended; it had accumulated significant woody debris and caused a significant scour pool downstream (W. Koning, personal communication). In September 1997, 8 new triangular-LWD habitat structures were installed in the West-Kettle River. Some of them make use of remnant LWD and railroad tie/chain anchors from the older debris-catcher type structures (Pat Slaney, personal communication).

5.7 Lost Creek

Lost Creek is a 4th order stream tributary to Trapping Creek, part of the larger West-Kettle River system. Lost Creek drains an area of about 45 km^2 or 31% of the Trapping Creek watershed. Trapping Creek is located about 17 km north of the community of Beaverdell (Figure 5-8). Two biogeoclimatic zones occur in the Trapping Creek watershed. Upper elevations belong to the Okanagan Dry Mild Montane Spruce biogeoclimatic zone and lower elevations are in the Kettle Dry Mild Interior Douglas-Fir biogeoclimatic zone.

As with the West-Kettle River, flow records for Trapping Creek indicate a snowmelt-dominated watershed where runoff peaks during late May and early June. Mean annual precipitation is about 560 mm/year.

The Trapping Creek watershed hosts a variety of resource-based activities including cattle grazing, recreation, and forestry. Despite the high use of the watershed for these activities, forestry remains the primary resource based activity of the drainage. Forest harvesting within the Trapping Creek watershed has been extensive. Approximately 40% of the watershed has been logged within the past 25 years, primarily to control the Mountain Pine

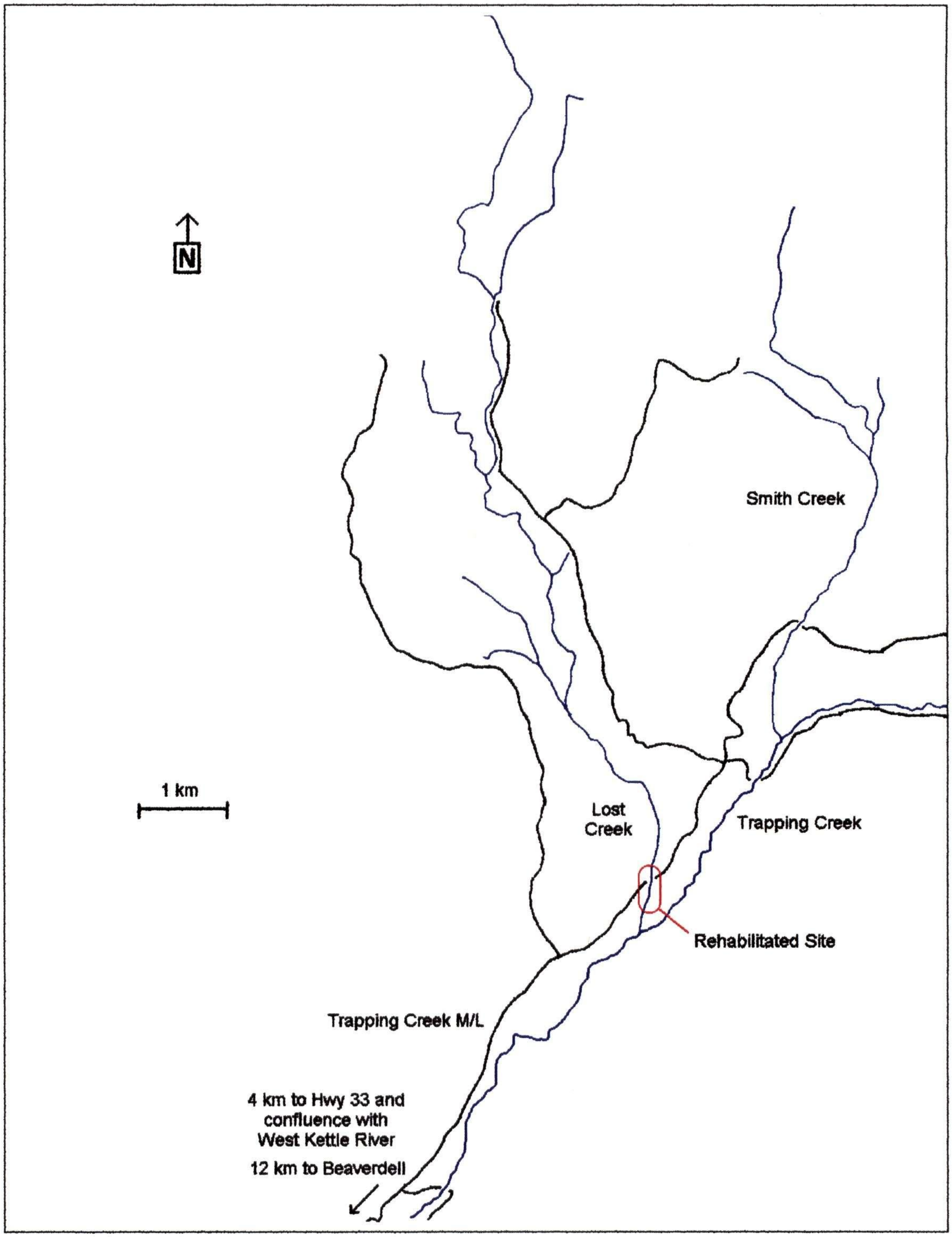


Figure 5-8: Lost Creek

Beetle. Harvesting has included floodplains and has extended to stream banks in many locations. Furthermore, a very high road density of 2.5 km of forest road for every square kilometre is present (Timberland, 1997). The riparian vegetation in the vicinity of the structures assessed consist of shrubs with some patches of Engleman Spruce.

Trapping Creek and its tributaries have a substantial amount of high value fish habitat, with numerous lakes located in headwater regions. Rainbow trout, the predominant salmonid species in the area, is found throughout the watershed and utilises some areas for spawning and rearing activities.

In 1997, a total of 9 km of stream within Trapping Creek and Lost Creek have undergone rehabilitation. Placement of LWD was undertaken to promote the formation of pools or to provide bank protection of vulnerable sites. A total of about 60 pieces of LWD were placed within the 3 lowermost reaches of Lost Creek.

6 RESULTS AND DISCUSSION

The following chapter provides the results, field observations and discussions of the performance evaluation conducted on the three types of structures investigates as part of this study. Issues relating to the design and construction of these structures will also be discussed in light of the preliminary design guidelines.

6.1 Single-LWD Structures - Results

A total of 23 single-LWD habitat structures were investigated as part of this study. As discussed earlier, six of these structures (103-103 Single 1 through 5 and WM Single) were not part of the initial post-construction assessment. They were added to the sample during the post-flood assessments since significant movement of these structures was detected after the winter flood in the Keogh River. One of the structures investigated as part of the post-construction assessments (W80-18) was destroyed and rendered inaccessible by the windfall of two large hemlocks that were at the edge of an undercut; one of which served as a bank anchor. The single log habitat structure is not suspected of being responsible for the undercut as it was observed to be in an advanced state during the initial post-construction assessment.

Factors of safety with respect to sliding and buoyancy were computed for the observed high water using the analysis presented in section 3.3. Note that almost all of the analyses are for partially submerged conditions. As can be see in Figure 6-1, there is a very good agreement between predicted and observed stability of the single log structures based on their FS_S. All single log structures with a $FS_S > 1.0$ were immobile, while only 2 of the

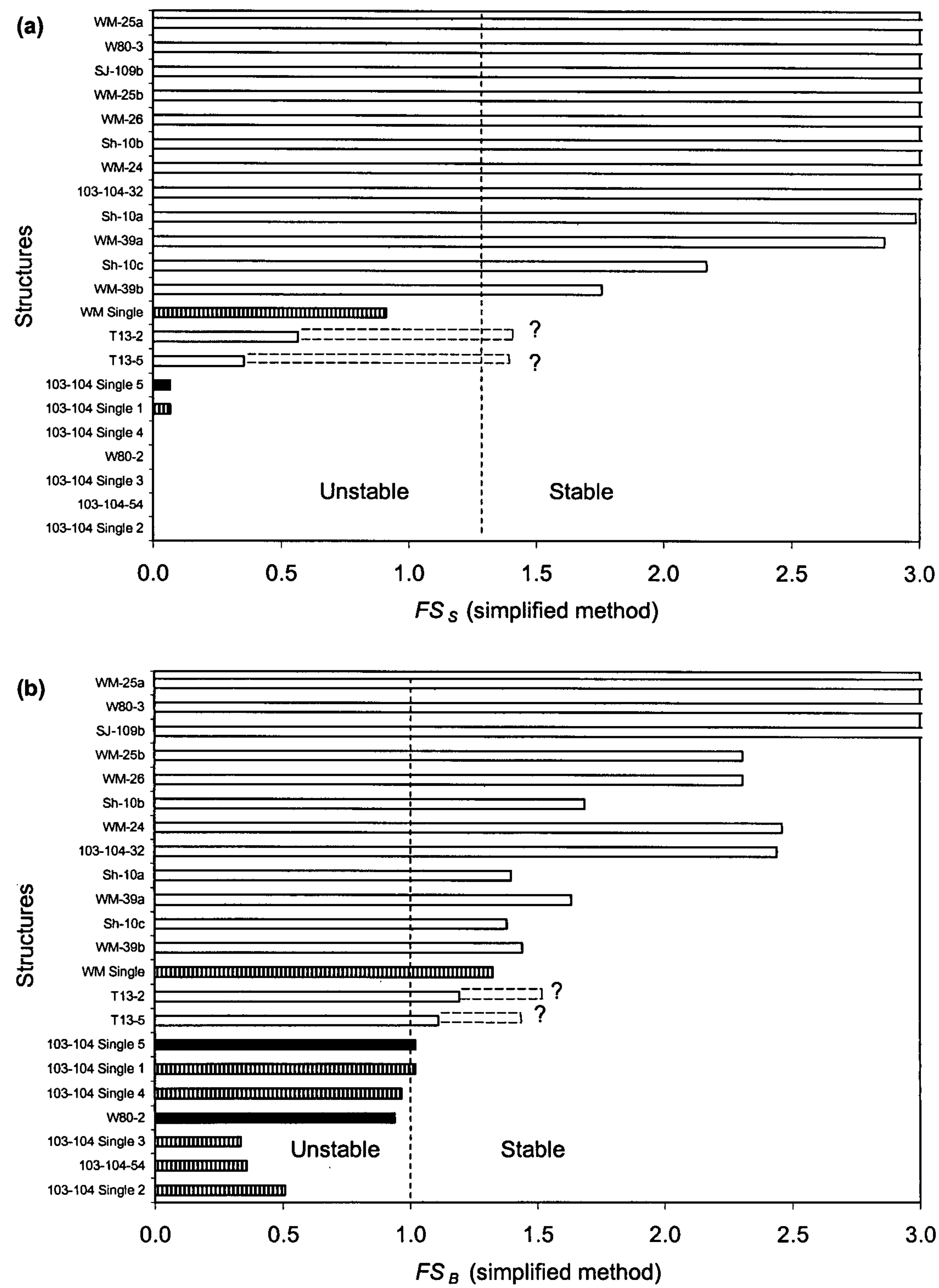


Figure 6-1: Stability of single-LWD structures (a) Factor of safety with respect to sliding, FS_S (b) Factor of safety with respect to buoyancy, FS_B. (Solid bars indicate structures that have shifted in place by more than 0.5 m but remain fully functional. Striped bars indicate structures that have shifted off site or to such a degree that they are considered partially functional or non-functional.)

structures with a $FS_S < 1.0$ did not move as expected. It was noted during the field assessments however, that both these structures (T13-2 and T13-5) had anchor boulders that were partially buried within the stream substrate. Hence, the size of the boulders may have been underestimated and the partial burial provided additional resistance to a buoyant or sliding failure. It was observed that most of the structures that moved were not fastened to a fixed bank anchor or were not provided extra ballast mass on the bank to counter transferred forces.

Once dislocated, structure WM Single was pushed downstream and came to rest in the middle of the stream at an angle of about 50°. This translates to a revised FS_S of about 1.1, which is close to the expected theoretical value of 1.0. Three single log structures, 103-104 Single 2, 103-104 Single 3 and 103-104-54, were grossly under-ballasted ($FS_B \leq 0.5$, $FS_S < 0.0$). They were pushed nearly parallel to the stream and were no longer in contact with the water at low flow; they were considered non-functional after the winter. Four other structures (103-104-1, 103-104-4, 103-104-5 and WM Single) with $FS_S < 1.0$, shifted considerably but remained partially functional. The other 16 structures were intact and functional.

Table 6-1: Single-LWD Structures - Computed Factors of Safety

Structure[a]	FS_B			FS_S[b]		
	High-water	Bankfull 2.33-yr	Design 50-yr	High-water	Bankfull 2.33-yr	Design 50-yr
103-104 Single 2	**0.51**	-	-	**0**	-	-
103-104-54	**0.36**	**0.35**	**0.26**	**0**	**0**	**0**
103-104 Single 3	**0.34**	-	-	**0**	-	-
W80-2	**0.94**	**0.90**	**0.74**	**0**	**0**	**0**
103-104 Single 4	**0.97**	-	-	**0**	-	-
103-104 Single 1	**1.02**	-	-	**0.07**	-	-
103-104 Single 5	**1.02**	-	-	**0.07**	-	-
T13-5	1.11	1.11	1.06	0.35	0.34	0.09
T13-2	1.19	1.20	0.99	0.56	0.59	-0.02
WM Single	**1.33**	-	-	**0.91**	-	-
WM-39b	1.44	1.42	1.22	1.76	1.65	0.39
Sh-10c	1.38	1.65	1.62	2.17	3.03	1.62
WM-39a	1.63	1.61	1.41	2.86	2.70	0.84
Sh-10a	1.40	1.18	0.74	2.98	1.15	-0.96
103-104-32	2.44	2.27	1.58	3.56	2.94	0.74
WM-24	2.46	2.38	1.77	3.62	3.35	0.98
Sh-10b	1.69	1.35	1.20	3.77	1.61	0.53
WM-26	2.31	2.28	2.16	4.73	4.51	2.05
WM-25b	2.31	2.23	1.24	5.70	5.28	0.59
SJ-109b[c]	4.90	-	-	7.86	-	-
W80-3	6.20	6.03	4.10	9.14	8.67	3.01
WM-25a	4.33	4.18	1.83	11.2	10.6	1.80
W80-18	-	3.05	1.77	-	4.93	0.90

[a] Structures that have shifted by more than 0.5 m are indicated in bold: structures added to sample because of observed movement are italicised.
[b] FS_S is set to zero when $FS_B < 1.0$ since frictional forces F_F can not be negative.
[c] Computed using average values from other San Juan structures and estimate of conditions.

The factors of safety were also computed for the bankfull and design flows (Table 6-1). Since the inter-assessment high water discharge and the computed bankfull flow are essentially the same for most of the structures, the factors of safety computed for both conditions are similar. About 55% (9/16) of the structures that underwent both post-construction and post-flood assessments had a bankfull flow FS_S at or above the recommended value of 2.0 (Millar, 1997). On the other hand, about 70% (11/16) had a bankfull flow FS_B at or above the recommended value of 1.25 (Millar, 1997). If the 50-

year return flood is used as design criterion, only two structures are sufficiently ballasted to meet or exceed a FS_S of 2.0. However, about 50% (8/16) of the structures could provide a FS_B in excess of 1.25 if subjected to design flood conditions.

The use of the full method of moments (section 3.3) to compute the factors of safety did not influence stability predictions. Generally, the FS_S computed from the full analysis of moments was slightly inferior to those obtained from the simplified method. This can be explained by the fact that under partially submerged conditions, the simplified method reduces the buoyancy and drag forces being transferred to the in-stream anchor. This effect is lessened, as more of the LWD is submerged under bankfull and design flow conditions.

A comparison has been made between the actual anchor boulder mass provided by the builder, and that required to provide a $FS_S = 2.0$ for bankfull and design flows (Table 6-2). For bankfull flood flow velocity, seven structures (out of the 16 originally assessed), require additional ballast mass to maintain a FS_S of 2.0. If the 50 year flood flow velocity is used, 12 structures (75%) require additional ballast mass. The average increase in ballast mass requirement going from bankfull discharge to design flood conditions is about 50%. This reflects an average increase in main-channel velocities of about 45%.

The Single-LWD structures were not observed to collect any significant amount of LWD or coarse particulate organic matter. The little that was captured by the structures was very loose and often dislodged by the wake of people wading about the structures.

Table 6-2: Single-LWD Structures - Ballast Mass Requirement

Structure	Ballast Factor	Ballast Mass (kg)				
		Provided	Required[a]		Difference	
			Bankfull	Design	Bankfull	Design
103-104-54	1	400	2,380	3,380	1,980	2,980
W80-2	1	1,620	3,420	5,340	1,800	3,720
T13-5	1	670	1,030	1,660	360	990
T13-2	1	580	970	1,560	390	980
WM-39b	1	1,500	1,800	2,770	300	1,270
Sh-10c	2	1,000	600	770	-	-
WM-39a	1	2,350	2,310	3,420	-	1,070
Sh-10a	2	1,360	1,220	1,490	-	130
103-104-32	1	1,280	1,390	2,240	110	960
WM-24	1	1,700	1,620	2,600	-	900
Sh-10b	2	920	550	710	-	-
WM-26	1	3,600	2,400	3,540	-	-
WM-25b	1	2,150	2,360	3,300	210	1,150
W80-3	1	4,630	1,860	3,230	-	-
WM-25a	1	2,490	2,170	3,070	-	580
W80-18		1,210	1,130	2,050	-	840
Average		1,720	1,490	2,240	560	1,080

[a] Assuming full submergence of LWD, FS_S = 2.0.

6.2 Single-LWD Structures - Discussion

The predictions made with respect to the stability of single log structures and the observations recorded as part of this study lend a strong support to a design approach based on a factor of safety against sliding (FS_S). The predictions based on the FS_S were accurate in all but 2 cases where their deviation could be readily explained. These results strengthen Millar's (1997) original findings from his preliminary evaluation of single-LWD structures. Hence, the use of this design method enables to determine anchor mass requirements for single log structures while allowing an acceptable safety margin through the use of a FS_S greater than 1.0.

It is also evident that a good fixed bank anchor in the form of a tree or stump is required to ensure stability. Most of the failures observed were the result of not securing the LWD at the bank end. It must be emphasised that Millar (1997) assumed that half of the drag and buoyancy forces exerted on the piece of LWD were transferred to its bank end. Therefore, resting the LWD on the upstream side of a tree or stump and cabling it in place is a simple method to ensure stability. Alternatively, if no tree or stump is present, doubling of the ballast mass requirement ($BF = 2$) and fastening half of the boulders at the bank end of the LWD may replace a fixed bank anchor.

In some cases along the entrenched West Main reach, some large logs unfastened at the bank end remained stable. These were supported by high banks and rested on the upstream side of riparian trees. They had sufficient mass resting out of the water at the bank end to counter in-stream buoyancy forces (i.e. WM-24, WM-25b and WM-39a). The FS_S of these structures however, decreases rapidly under design flow conditions (refer to Table 6-1) and their apparent stability may be precarious. Determining boulder mass requirements based on full submergence of the LWD and cabling structures to streamside trees would solve this problem.

No significant differences were observed in the factors of safety computed using the two different computational methods. As expected, the differences between the two methods were reduced with increasing flood flow as more of the structure is submerged. Note that under full submergence of the structural components both computational methods will yield identical results.

No significant scouring was observed around the single-LWD structures evaluated. The fact that the structures have been in place for such a short period, and the coarse nature of the bed material is believed to be responsible for the limited scour. The effects of single log structures in medium sized streams (10-30 m bankfull width) are quite localised and a detailed survey grid (0.50 m) may be required to quantify these effects. With time, and a few bankfull events, it is expected that some lateral scour pools will develop and be maintained in the reaches with finer bed material.

6.3 Single-LWD with Root Wad Structures - Results

Factors of safety with respect to sliding and buoyancy were calculated for the observed high water for nine structures using the analysis presented in section 3.4. As illustrated by Figure 6-2, the predictions made with respect to structural stability based on the FS_S were accurate. As predicted, three out of the nine structures evaluated proved unstable. Structure SJ-104 was completely washed away (Figures 6-3 and 6-4) while both structures W80-15 and SJ-106 shifted only slightly. Some small movement of structures SJ-103a and SJ-103b was observed, however the distance travelled by the "settling in" of the anchor boulders was less than 0.7 m. The FS_B of these 2 structures are well above 2.0. The factors of safety were also determined for the bankfull and design (50 year return) flows (Table 6-3).

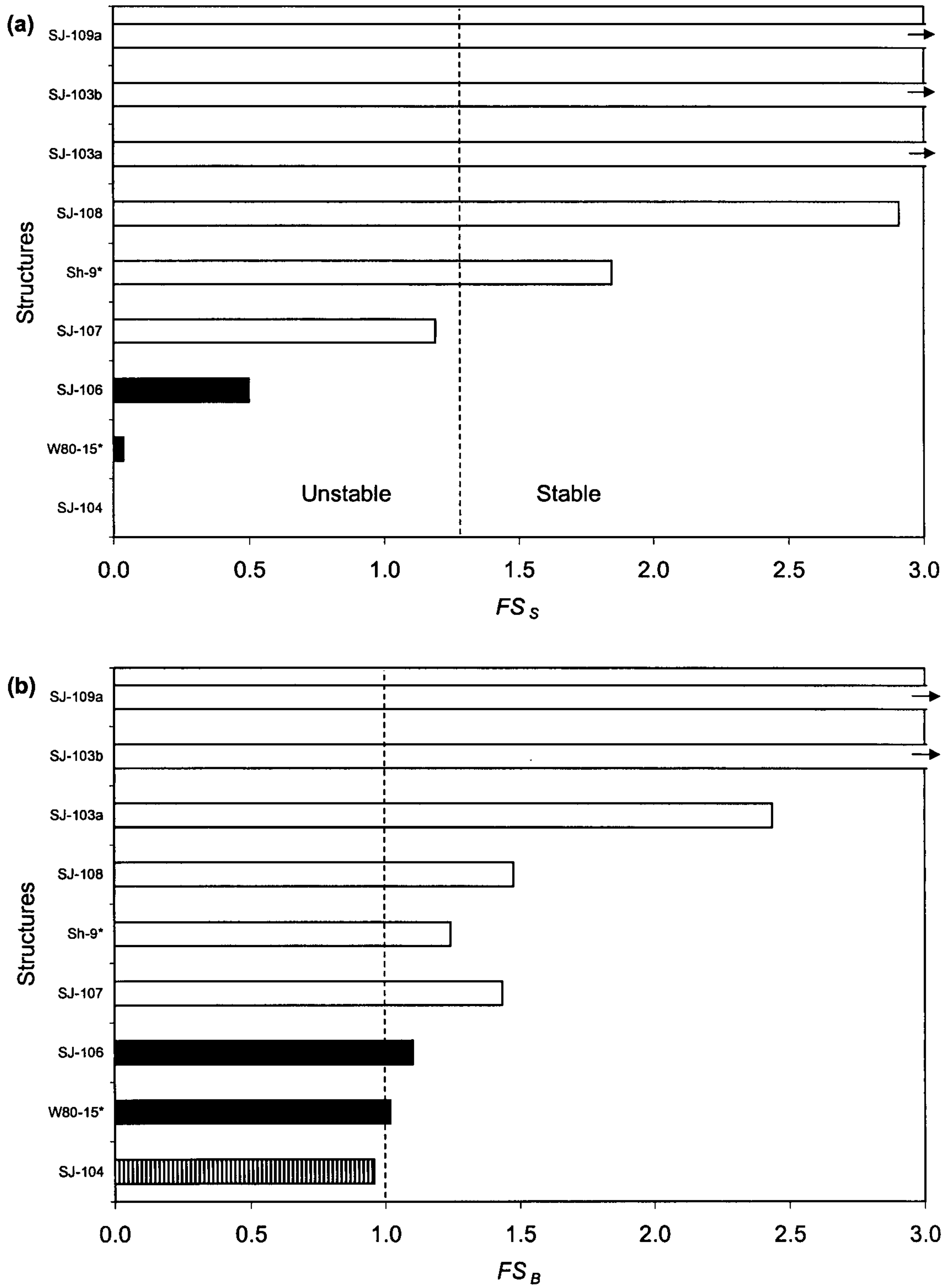


Figure 6-2: Stability of Single-LWD with Root Wad Structures (a) Factor of safety with respect to sliding, FS_S (b) Factor of safety with respect to buoyancy, FS_B. (Solid bars indicate structures that have shifted in place by more than 0.7 m [0.5 m with *] but remain fully functional. Striped bars indicate structures that have shifted off site or to such a degree that they are considered partially functional or non-functional.)

It appears that close to half of the structures had a FS_S greater than the recommended 2.0 (Millar, 1997) during the last high flow conditions. Under bankfull and design flow conditions, only three out of nine structures would maintain a FS_S in excess of 2.0. In terms of buoyancy, all but one of the single-LWD with root wad had $FS_B \geq 1.0$. Five of the structures maintained a FS_B in excess of the recommended 1.25 for the three discharge conditions.

Table 6-3: Single-LWD with Root Wad Structures - Computed Factors of Safety

Structure[a]	FS_B			FS_S[b]		
	High-water	Bankfull	Design	High-water	Bankfull	Design
SJ-104	**0.96**	**0.96**	**0.95**	**0**	**0**	**0**
W80-15*	**1.02**	**1.02**	**0.98**	**0.04**	**0.04**	**0**
SJ-106	**1.10**	**1.10**	**1.09**	**0.50**	**0.30**	**0.21**
SJ-107[c]	1.44	1.44	1.43	1.19	1.18	0.89
Sh-9*	1.24	1.24	1.22	1.85	1.47	0.76
SJ-108	1.48	1.47	1.47	2.91	1.58	1.08
SJ-103a	2.44	2.42	2.39	3.97	3.25	2.40
SJ-103b	3.83	3.80	3.73	6.07	4.97	3.68
SJ-109a[c]	4.33	-	-	6.83	-	-

[a] Structures that have shifted by more than 0.7 m (0.5 m for *) are indicated in bold.
[b] FS_S is set to zero when FS_B < 1.0 since frictional forces F_F can not be negative
[c] Computed using average values from other San Juan structures and estimate of conditions.

The design mass requirements for the single-LWD with root wad, summarised in Table 6-4, indicate that three out of the nine structures evaluated require at least a doubling of the as-built anchor mass to ensure a FS_S of 2.0 under design flood conditions. The three large spruce trees placed within the San Juan River require a considerable amount of additional mass (about 14 tonnes each) to provide a $FS_S > 2.0$ under design conditions. The average increase in ballast mass between bankfull and design flow conditions is about 20%. This reflects an average increase in main-channel velocity in the order of 20%.

Table 6-4: Single-LWD with Root Wad Structures - Ballast Mass Requirement

Structure	Ballast Mass (kg)				
	Provided	Required[a]		Difference	
		Bankfull	Design	Bankfull	Design
SJ-104	5,380	11,200	13,270	5,850	7,890
W80-15	2,880	7,760	13,720	4,880	10,840
SJ-106	13,690	23,310	27,000	9,620	13,310
SJ-107	33,960	42,040	48,260	8,080	14,300
Sh-9	2,560	2,650	3,350	90	790
SJ-108	31,340	39,110	45,490	7,770	14,150
SJ-103a	4,280	3,370	3,980	-	-
SJ-103b	5,300	2,960	3,560	-	-
SJ-109a[b]	7,940	2,540	3,010	-	-
Average	11,900	15,000	17,900	5,980	10,100

[a] Assuming full submergence of LWD and root wad, FS_S = 2.0 and FS_B = 1.25.
[b] Computed using average values from other San Juan structures and estimate of conditions.

In terms of functionality, all structures but SJ-104 were essentially in place and performing as intended. SJ-104 could not be found downstream and is assumed to be non-functional. Figures 6-3 and 6-4 show the pre and post flood photos of structures SJ-103a, SJ-103b and SJ-104. Note that structure SJ-104 was displaced following a first flood in early November of 1997 while the peak instantaneous discharge was recorded in mid December 1997.

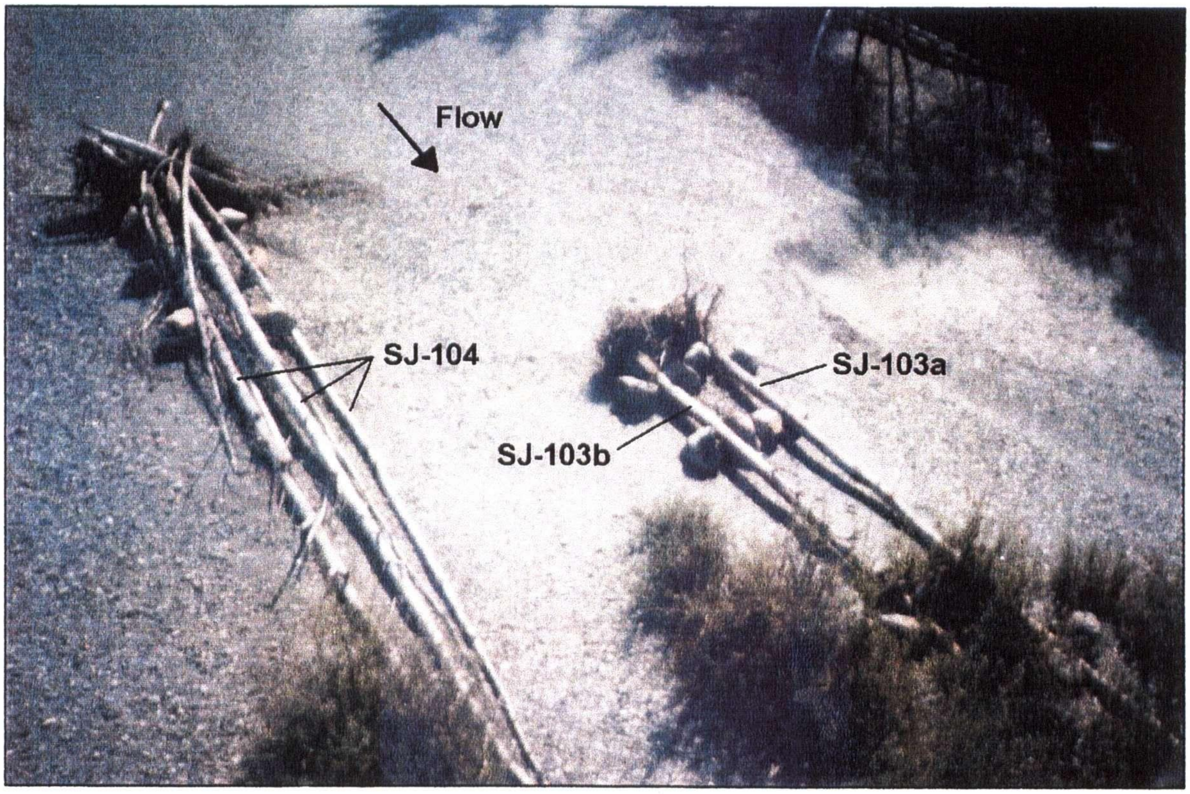


Figure 6-3: Aerial Photo of SJ-104, SJ-103b and SJ-103a (Sept. '97)

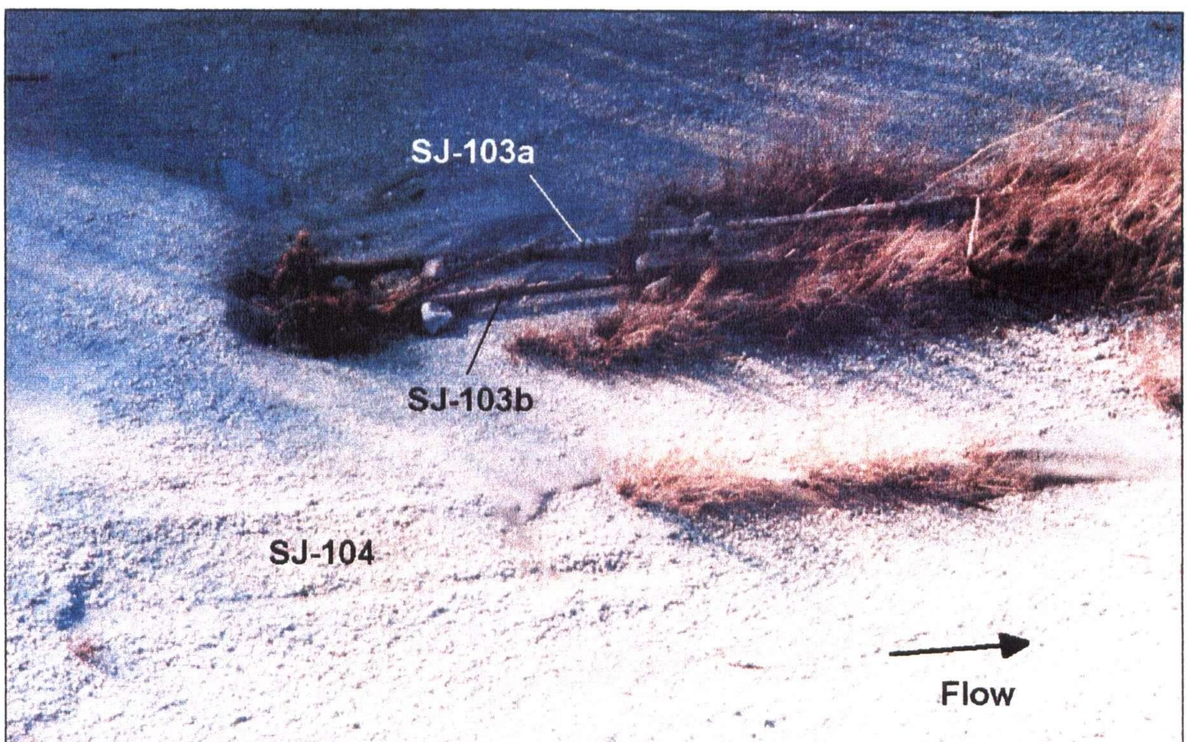


Figure 6-4: Aerial Photo of SJ-103a, SJ-103b and print of SJ-104 (Nov. '97)

6.4 Single-LWD with Root Wad Structures - Discussion

A number of issues had to be taken into account while evaluating the single-LWD with intact root wads. Estimates of earth volume within the root wads and embedment /sheltering of the wads by earth sloughing off the banks complicated the safety factor computations for structures SJ-106 and SJ-108 (refer to section 4.5). Furthermore, pin measurements within the San Juan River with its high banks and wide bed were made difficult and decreased their accuracy. Despite these difficulties, it is apparent that the approach used to determine factors of safety can predict the stability of single-LWD structures with an intact root wad. Hence, applying the proposed design method with an adequate FS_S should ensure the durability of such structures.

One characteristic of this type of structure is that the anchor boulders located near the root wad are sheltered from the flow and deposition of material scoured at the base of the root wad tend to smother the boulders. The embedment of the anchor boulders with time provides increased resistance against buoyant uplift and sliding failures. This smothering phenomenon was observed to take place for at least six of the nine structures. Figure 6-5 illustrates the observed smothering of the anchor boulders on structures SJ-103a and SJ-103b, where only the top third to half of the boulders are now exposed.

Although most of the single-LWD structures were not subjected to large floods, they (with the exception of SJ-104) appear to be providing an acceptable level of bank and bar protection.

Figure 6-5: Smothering of anchor boulders on SJ-103a and SJ-103b (April '98)

6.5 Multiple-LWD Structures Results

The interpretation of the results for the multiple-LWD structures is not as straightforward as in the two previous cases. Of the 51 multiple-LWD structures that were investigated, 23 experienced some lateral movement of their LWD by more than 0.5 m. Five of the unstable structures reported (103-104-DF 1 to 4 and Wolf Triangular) were surveyed only during the post-flood assessment as they were observed to have moved significantly since construction.

In contrast to the single-LWD type structures, drag forces acting on the multiple-LWD structures are difficult to quantify. It is assumed (Millar, 1997) that the lateral stresses are resisted through structural bracing. For this reason, the factor of safety against buoyancy

(FS_B) (section 3.2) is used as a simple design criterion that is indirectly related to the lateral stability of the structures. While this criterion ensures that structures do not float under high flows, excess vertical forces indicated by a value of $FS_B > 1.0$ provide additional lateral stability through frictional forces on the bed. Furthermore, it is desirable that the multiple-LWD structures remain in contact with the bed during high flows to promote scour and pool formation. This should occur if a structure is constructed with a $FS_B > 1.0$.

Factors of safety with respect to buoyancy were computed for the observed high flows using the analysis presented in section 3.2 and 3.3 (Figure 6-6). 27% (8/30) of the structures that had a $FS_B > 1.0$ under the recent flood conditions exhibited some movement, whereas 75% (15/21) of the structures with a $FS_B < 1.0$ proved unstable. Eight of the ten structures that were subject to the most shifting and/or designated as partially or non-functional had a $FS_B < 1.0$ during the observed high flows.

19 structures out of the 51 evaluated did not exhibit triangular bracing; they were either V-type or complex structures. The FS_B of the non-triangulated structures, neglecting the four double floating log structures (103-104 DF 1 to 4) which were grossly under-ballasted, are evenly distributed amongst our sample. 40% (6/15) of non-triangulated structures proved unstable compared to 41% (13/32) for the triangulated structures.

Table 6-5 provides a summary of computed (section 3.3) safety factors under different flow regimes. Under bankfull flow conditions, only about 35% of the structures would possess a FS_B greater than the recommended 1.25 (Millar, 1997). This figure drops to about 20% if the structures would be subjected to a 50-year flood event. In general, the FS_B computed using the full method of moments tend to be greater than those computed using the simplified method since the mass of LWD above water is taken into account. If the stability predictions were based on FS_B computed based on the method of moments instead of the simplified method, 6 additional structures (LC-104, 103-104-33, S-6, S-9, S-12 and Sh-6) would have been expected to be stable under the observed high flow. Despite these revisions, three of these six structures proved unstable.

Based on the foregoing, it is not surprising that 80% of the multiple-LWD structures require additional ballast mass (on average 1,100 kg/structure) to provide a minimum recommended FS_B of 1.25 (Millar, 1997). Table 6-6 summarises ballast mass requirements for the multiple-LWD structures. The difference in ballast mass requirement between bankfull and design conditions is insignificant (≈2%) since it is assumed that the LWD is fully submerged under both conditions. The only difference in mass is to compensate for increased lift forces (due to higher velocities) acting on the anchor boulders.

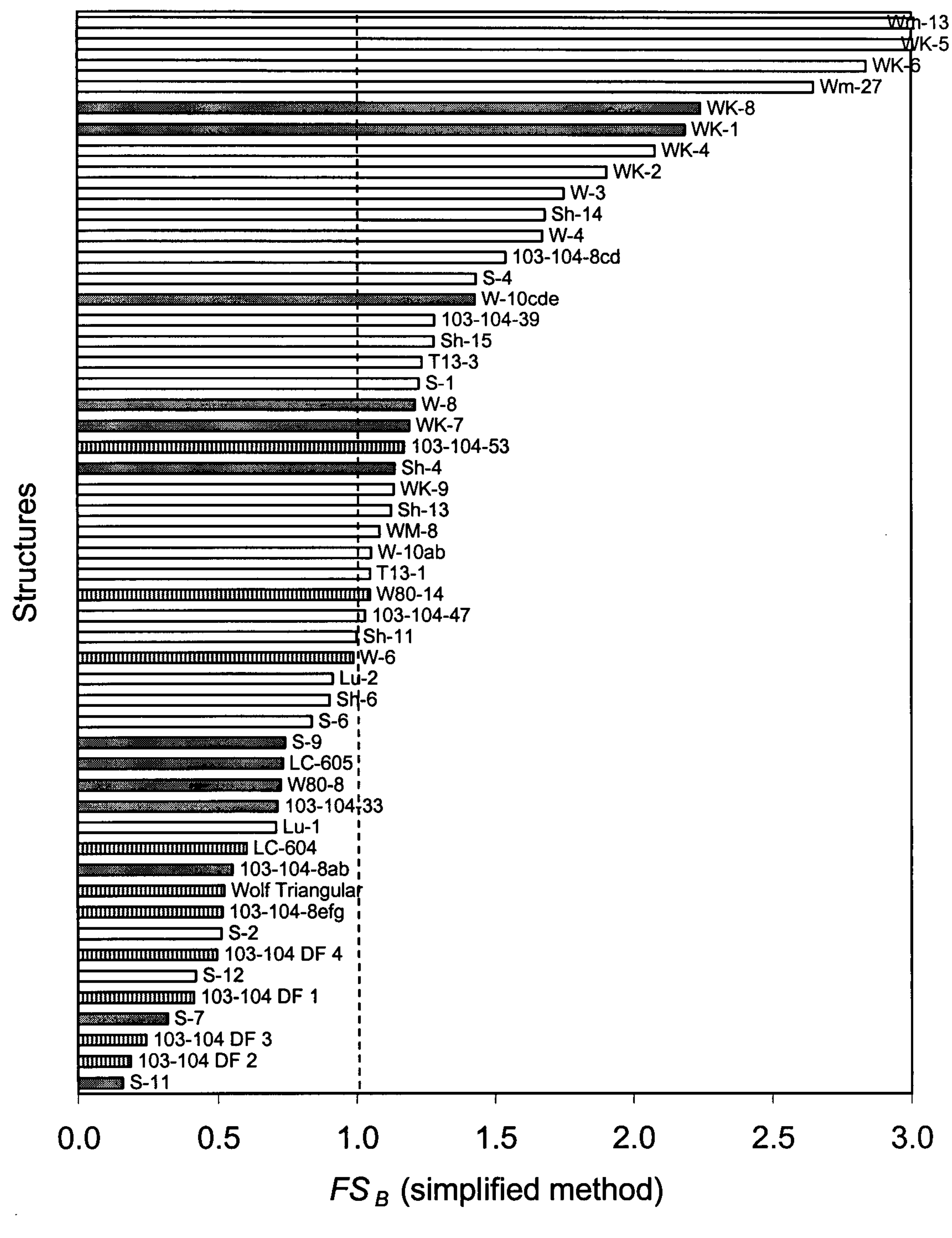


Figure 6-6: Stability of Multiple-LWD Structures (Solid bars indicate structures that have shifted in place by more than 0.5 m but remain fully functional. Striped bars indicate structures that have shifted off site or to such a degree that they are considered partially functional or non-functional.)

Table 6-5: Multiple-LWD Structures - Computed Factors of Safety

Structures[a]	FS_B High-water	FS_B Bankfull 2.33-yr	FS_B Design 50-yr
S-11	**0.16**	**0.06**	**0.04**
103-104 DF 2	**0.19**	-	-
103-104 DF 3	**0.24**	-	-
S-7	**0.32**	**0.25**	**0.24**
103-104 DF 1	**0.41**	-	-
S-12	0.42	0.29	0.22
103-104 DF 4	**0.50**	-	-
S-2	0.51	0.48	0.38
103-104-8efg	**0.52**	**0.52**	**0.50**
Wolf Triangular	**0.52**	-	-
103-104-8ab	**0.55**	**0.54**	**0.48**
LC-604	**0.60**	**0.57**	**0.40**
Lu-1	0.71	0.72	0.67
103-104-33	**0.72**	**0.70**	**0.58**
W80-8	**0.73**	**0.71**	**0.52**
LC-605	**0.73**	**0.71**	**0.48**
S-9	**0.74**	**0.67**	**0.45**
S-6	0.84	0.69	0.67
Sh-6	0.90	0.83	0.79
Lu-2	0.91	0.93	0.79
W-6	**0.99**	**1.04**	**0.95**
Sh-11	1.00	1.02	0.66
103-104-47	1.03	1.00	0.88
W80-14	**1.05**	**1.04**	**0.91**
T13-1	1.05	1.04	0.99
W-10ab	1.05	1.04	0.90
WM-8	1.08	1.13	0.85
Sh-13	1.13	1.01	0.85
WK-9	1.14	1.19	0.97
Sh-4	**1.14**	**1.10**	**1.05**
103-104-53	**1.17**	**1.14**	**1.12**
WK-7	**1.19**	**1.22**	**0.87**
W-8	**1.21**	**1.31**	**0.92**
S-1	1.23	0.77	0.58
T13-3	1.24	1.16	1.11
Sh-15	1.28	1.70	1.12
103-104-39	1.28	1.28	1.26
W-10cde	**1.43**	**1.37**	**0.83**
S-4	1.43	1.27	0.88
103-104-8cd	1.54	1.51	1.21
W-4	1.67	1.72	1.25
Sh-14	1.68	1.52	1.48
W-3	1.75	1.16	1.11
WK-2[b]	1.91	1.84	1.55
WK-4[b]	2.08	1.89	1.68
WK-1	**2.19**	**2.04**	**1.80**
WK-8	**2.24**	**2.39**	**1.56**
Wm-27	2.65	2.76	2.18
WK-6[b]	2.84	2.54	1.88
WK-5[b]	3.01	2.91	2.61
Wm-13	3.35	3.32	2.57

[a] Structures in bold have shifted by more than 0.5m: structures added to sample because of observed movement are italicised

[b] Structures anchored with remnants from old debris-catcher structures.

Table 6-6: Multiple-LWD Structures - Ballast Mass Requirement

Structures	Ballast Mass (kg)			
	Provided	Required[a]		Difference
		Bankfull	Design	Design
S-11	90	2,570	2,590	2,500
S-7	890	4,560	4,580	3,690
S-12	980	5,600	5,620	4,630
S-2	510	1,930	1,940	1,430
103-104-8efg	270	660	690	420
103-104-8ab	1,020	2,660	2,730	1,710
LC-604	660	2,250	2,280	1,630
Lu-1	5,230	6,220	6,430	1,200
103-104-33	2,270	5,360	5,520	3,250
W80-8	2,990	7,250	7,420	4,430
LC-605	240	550	560	320
S-9	1,260	3,530	3,550	2,290
S-6	1,740	3,260	3,270	1,530
Sh-6	1,320	2,080	2,120	800
Lu-2	850	1,290	1,370	520
W-6	3,100	4,010	4,110	1,010
Sh-11	5,180	10,640	10,730	5,550
103-104-47	4,450	6,320	6,480	2,030
W80-14	3,430	4,640	4,760	1,330
T13-1	3,490	4,310	4,450	960
W-10ab	1,320	1,860	1,920	600
WM-8	7,890	11,380	12,060	4,170
Sh-13	4,760	6,930	7,000	2,240
WK-9	8,670	10,900	11,200	2,530
Sh-4	3,770	4,470	4,540	780
103-104-53	5,380	5,890	6,000	620
WK-7	4,400	6,290	6,430	2,030
W-8	2,110	2,760	2,890	780
S-1	1,800	3,880	3,900	2,100
T13-3	1,540	1,650	1,740	200
Sh-15	5,050	5,640	5,730	680
103-104-39	6,120	5,970	6,070	-
W-10cde	730	1,380	1,430	700
S-4	1,100	2,470	2,490	1,390
103-104-8cd	1,710	1,700	1,760	50
W-4	2,830	2,680	2,830	-
Sh-14	6,530	5,010	5,080	-
W-3	3,630	3,500	3,730	100
WK-2[b]	6,220	4,920	5,000	-
WK-4[b]	10,210	7,350	7,500	-
WK-1	7,370	4,950	5,050	-
WK-8	7,470	7,840	8,080	610
Wm-27	4,820	2,710	2,840	-
WK-6[b]	8,040	5,250	5,360	-
WK-5[b]	5,760	2,560	2,640	-
Wm-13	6,620	2,840	2,990	-
Average	3,600	4,400	4,500	1,100

[a] Assuming full submergence of LWD and FS_B = 1.25.
[b] Structures anchored with remnants from old debris-catcher structures

6.6 Multiple-LWD Structures - Discussion

At first glance, the results or the multiple-LWD structures are not as definitive as those presented for the single-LWD structures. Despite the relatively high number of observed instabilities for structures with $FS_B > 1.0$, there is an apparent trend of increased stability and decreased damage as the FS_B increase. All of the multiple-LWD structures observed to be partially functional or non-functional after flooding had a $FS_B < 1.25$. Since these structures are built from multiple pieces of LWD, there are more possible permutations in the design and construction of each structure and hence increased chances of observing instabilities due to inadequate structural bracing. It must also be emphasised that, in order to evaluate design approaches, the movement criterion (lateral movement of 0.5 m for any piece of LWD or anchor boulder) is quite strict and the majority of structures observed to have moved remain functional.

Taking a closer look at some of the structures that exhibited instabilities reveals that additional boulder mass alone would not have prevented movement; in some cases, poor triangular bracing, loose cabling and lack of fixed bank anchors is suspected to have contributed to the instabilities. Conversely the structures with a $FS_B < 1.0$ which remained intact owe their success to a good triangular bracing (including bank anchors) and tight cabling. Table 6-6 provides a summary of field observations for the structures that did not perform as expected.

It must be stressed that the design method proposed by Millar (1997) relies on the triangular bracing of the primary structural members to provide lateral stability (in horizontal plane). As well, the substitution of fixed bank anchor points with additional boulders for multiple-LWD structures is not recommended since Millar's analysis relies on the inherent lateral stability of the structure, provided by these anchor points, to reduce the mass requirements.

Table 6-7: Multiple-LWD Structures - Summary of Outliers

Case	Structure	Comment
Unstable with $FS_B \geq 1.0$	WK-1	Significant LWD on face (Figure 6-7)
	WK-8	Significant LWD on face, undermining of anchor tree on downstream bank (Figure 6-8 & 6-9)
	W-10cde	Inadequate triangular bracing, loose cabling
	W-8	Principal LWD member overhanging at bank end, FS_B moments 1.06, loose cabling
	WK-7	In-stream boulders submerged (sizes estimated)
	103-104-53	Upstream-V structure spanning entire channel, boulder ballasted at bank ends
	Sh-4	Steel cabling quite loose and boulder ballasted at bank ends
	W80-14	Inadequate triangular bracing, no bank anchors
	W-6	Downstream-V structure spanning entire channel, not tied back at bank ends, 2nd largest anchor boulder split in two
	S-9[a]	no in-stream ballast, boulder ballasted at bank ends only
	103-104-33[a]	one epoxy cable/boulder connection came undone, lost 1 LWD piece while the rest of the structure is intact
	LC-604[a]	in-stream connection using reinforcing steel came undone, other structural components intact
Stable with $FS_B < 1.0$	Lu-2	Good triangular bracing and tight cabling
	Sh-6	Some anchor boulders partially covered in aggraded sands, FS_B moments 1.04
	S-6	FS_B moments 1.12
	Lu-1	Good lateral stability provided by tight cabling
	S-2	Good triangular bracing
	S-12	FS_B moments 4.83 (significant LWD above water surface)

[a] FS_B moments $\geq$ 1.0

Both structure WK-1 and WK-8, which had sufficient ballast mass to provide a $FS_B = 1.25$ under design conditions, experienced some minor shifting. It is suspected that the significant amount of debris accumulation on the face of these two structures along with some bed and bank scouring may be responsible for the slight shifting. Refer to Figure 6-

7, 6-8 and 6-9. While the bank erosion and undercutting of the downstream bank anchor tree on structure WK-8 was obvious, high water levels within the West-Kettle River during post flood assessments prevented the in-stream survey and evaluation of possible bed scour around the anchor boulders.

Considering the success of the methods developed for the single-LWD structures, the use of a FS_S for multiple-LWD structures would appear to be a solution for improving the predictive capability of the approach and refining the design guidelines. However, in attempting to develop such an approach, practical problems surfaced. Each multiple-LWD structure is configured differently where only a few pieces of LWD contribute the bulk of the drag forces; the other pieces of LWD are sheltered in the wake of those pieces. Determining which of the LWD members are contributing to drag and evaluating them individually for both sliding and buoyant failure lengthens and complicates the design procedure.

Figure 6-7: Debris accumulation on face of WK-1 structure. As built - Oct. 1997 (top, note human figure at right for scale), post freshet - June 1998 (bottom) and looking downstream from bank (inset).

Figure 6-8: WK-8 as built Oct. 1997 (top) and post freshet looking upstream June 1998 (bottom). Note (bottom photo) debris on upstream face, fallen bank anchor tree (centre left) and eroded bank downstream.

Figure 6-9: Eroded bank looking downstream from top of structure WK-8 (June 1998)

In applying such a method to multiple-LWD structures, it was often found that little or no change in ballast mass was required when both FS_S and FS_B were set to values of about 2.0. In some cases, a $FS_B = 2.0$ governed the design. To clarify this, the ballast mass requirement to satisfy both FS_S and FS_B, have been plotted together on the same chart (Figure 6-10). Solids lines represent the mass requirement per metre of LWD (90° to flow) in order to satisfy a FS_S of 2.0 when subjected to velocities of 2 m/s and 3 m/s. Dashed lines represent the ballast mass requirement per metre of LWD to satisfy FS_B values of 1.5 and 2.0. From this chart, it is evident that for LWD with diameters less than 0.6 m, the anchor mass requirement based on a $FS_B = 2.0$, is essentially the same as for that based on a $FS_S = 2.0$ and a head on flow velocity of 2 m/s. For LWD diameters in excess of 0.7 m, the mass requirement based on a $FS_B = 1.5$ is roughly equivalent to that based on a $FS_S =$ 2.0 and a head on flow velocity of 2 m/s. Since many of the structures investigated as part of this study are subject to velocities in the range of 2.0 m/s, and that they are generally not oriented 90° to the flow, a $FS_B \geq 1.5$ will often govern the design.

Based on the above information, it would be simpler to increase the minimum FS_B used in the design of the multiple-LWD structures in order to obtain ballast requirements similar to those obtained from a more detailed evaluation which considers drag forces. Furthermore, as discussed previously good cabling and triangular bracing appears to be a key factor in design to enable the structures to resist drag forces.

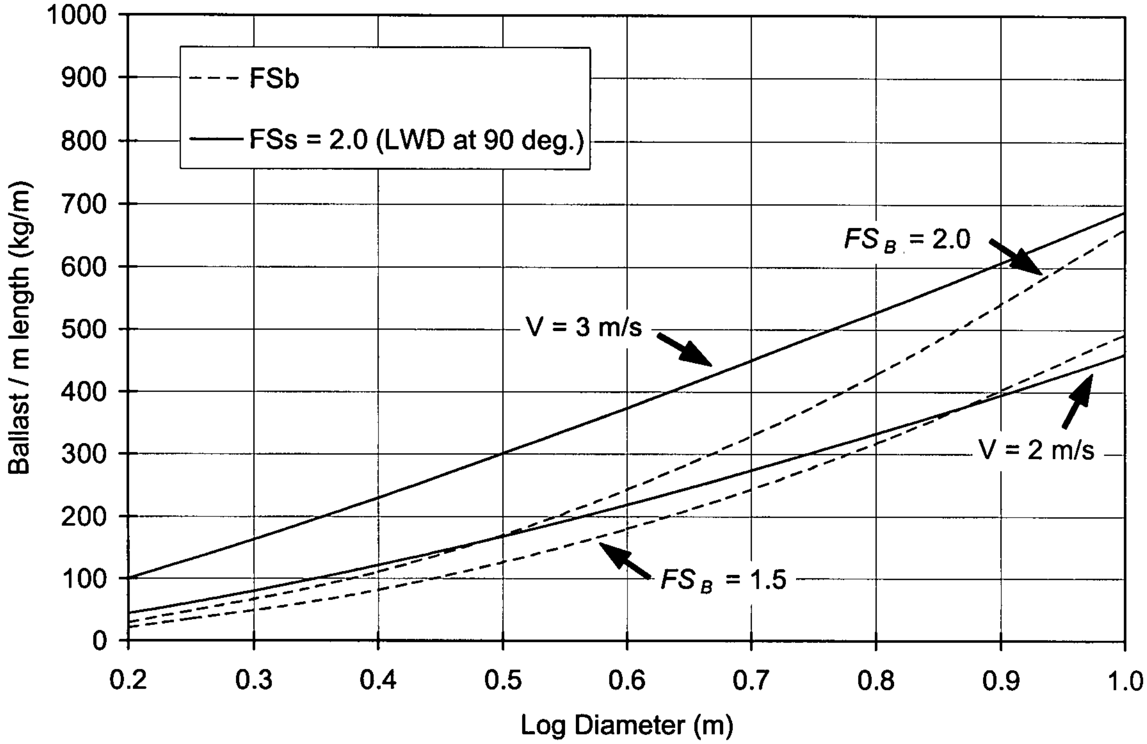


Figure 6-10: Ballast Mass Requirement, Single and Multiple-LWD Structures

Information collected with respect to the double floating LWD structures (2 pieces of LWD aligned parallel with the flow, sitting on top of their anchor boulders), indicate that, despite their lack of triangular bracing, they offer little drag resistance and should remain stable when ballasted using the design guidelines. Those investigated were grossly under-ballasted and simply floated away. The double floating LWD structures were found to be highly utilised by young coho salmon in the summer (Mark Potyrala, personal communication) and observed to serve as shelter for migrating steelhead during the post-flood assessments.

Two out of the three full-spanning V-type structures located within the Keogh River shifted during floods. These type of structures offer significant resistance to the flow and essentially act like a small dam impounding water across the entire channel width. The development of the design guidelines did not consider the hydrostatic forces that would act on such structures. A different design approach specific to these structures should be used to establish their ballast and anchoring requirements.

Keeping in mind that the field observations are based on slightly less than a year of operation, it appears that the debris capture efficiency of the multiple-LWD structure ranges from good to poor. Approximately 20% of the structures were found to have accumulated some small and LWD. Factors which seem to influence the capture efficiency of structures is the size of the system (affects ease of transport), quantity of structures/obstacles in place, presence of debris and structure characteristics. The provision of secondary LWD pieces, cabled above and/or below the primary LWD members, at an orientation parallel to the flow appears to increase the "snagging" capability of structures. Structure WK-9 is a good example (Figure 6-11) where such secondary members increase debris capture.

Figure 6-11: Structure WK-9 (looking downstream from bank, flow is left to right). Both LWD pieces oriented roughly parallel to flow (centre right) increase the structure debris capture efficiency.

6.7 Other Considerations

6.7.1 Channel Entrenchment

Channel entrenchment of a river may have a significant effect on ballast requirements for habitat structures. In practice, the floodplain provides flood relief and minimises flood level fluctuations for flows beyond bankfull. Since little flood relief is available in entrenched channel reaches, both flood stage and main channel velocities are expected to be greater than in non-entrenched conditions. The degree of entrenchment is expressed in terms of an entrenchment ratio (*ER*), which is the ratio of the width of the main flow channel (W_{MC}) divided by the width of the floodplain (W_{FP}) (or surface width of water)(Figure 6-12). Rivers with wide floodplains have low values of *ER* (<<1.0). The maximum value of *ER* is 1.0, which corresponds to an entrenched channel with no floodplain.

A simple theoretical analysis was undertaken on a cross-section from the Keogh River to illustrate the effects of entrenchment on the ballast requirements of single-LWD structures. The value of *ER* was increased progressively from its observed value (0.25) to the maximum value of 1.0. The results are tabulated in Table 6-3 and the cross-section used for the analysis (103-104-53) is illustrated in Figure 6-12.

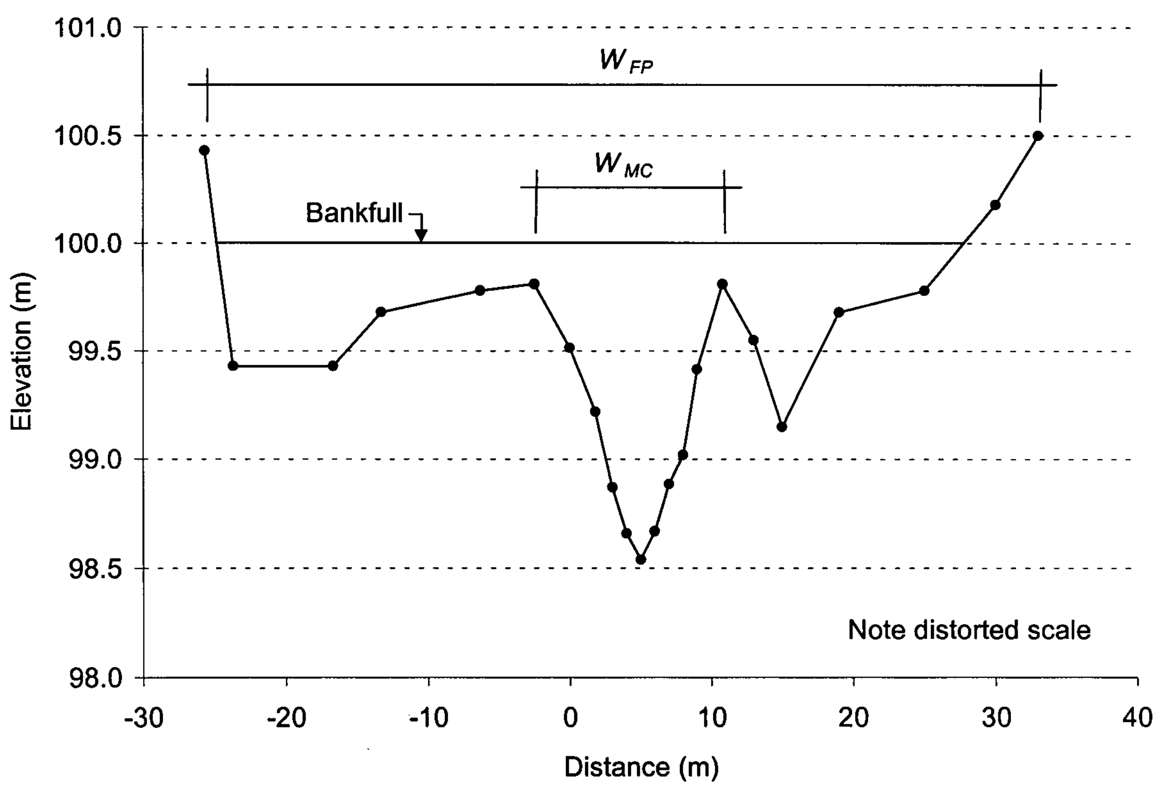


Figure 6-12: Channel Entrenchment Analysis - Cross-section 103-104-53

Table 6-8: Entrenchment Analysis (Keogh River 103-104-53)

Entrenchment Ratio[a]	2.3 year return			50 year return		
	V^b (m/s)	El. (m)	M (kg)[c]	V^b (m/s)	El. (m)	M (kg)[c]
0.25	2.04	100.00	1485	2.58	100.56	1965
0.50	2.07	100.10	1510	2.84	100.82	2250
0.75	2.12	100.13	1550	2.95	100.93	2385
1.00	2.13	100.14	1555	3.00	100.97	2445

[a] Entrenchment Ratio (E/R) = W_{MC}/W_{FP}
[b] Main channel velocity
[c] Mass required to ballast a single LWD 10 m long, 0.5 m diameter, S_L = 0.50, 45° angle from flow with a FS_S = 2.0 and assuming full submergence of LWD.

As expected, flows close to bankfull conditions are not significantly affected by entrenchment since the bulk of the water is contained within the banks and in-channel velocities remain essentially constant. Under existing non-entrenched conditions (entrenchment ratio (ER) = 0.25), the difference between the bankfull and design flow velocities in the main channel prompt an increase in ballast mass requirement of about 30%. For fully entrenched conditions (ER = 1.0), the ballast mass requirement increase, going from bankfull to design flow conditions, is about 60%. This number coincides with the mass requirements reported in Table 6-2. Under design flow conditions, the ballast mass requirement increases by about 25% going from a non-entrenched to an entrenched state. For single-LWD structures with intact root wads the results are similar to those obtained for the single-LWD structures. Under design flow conditions, going from an entrenchment ratio of 0.25 to 1.0 will prompt an increase in ballast mass of about 30%.

Based on the foregoing, one of the main difficulties encountered in applying the design guidelines is establishing design (50-year return) flood conditions once a design discharge has been estimated. Characterising overbank flow requires more detailed cross-sectional surveys (that extend beyond the banks), estimation of overbank roughness and the use of more involved hydraulic computations to account for compound conveyance channels. A simple approach that can be used to avoid such complications is to make use of an equivalent trapezoidal cross-section representing the bankfull flow area and extending the banks along the side slopes to compute design flow conditions (refer to section 4.3). This will simplify the design method by representing an entrenched state and providing conservative design conditions. If the reach under rehabilitation is not entrenched, then the designer has the option of reducing the mass requirement by allowing a reduced FS_S (to a maximum mass reduction of 25%). This should not affect mass requirements computed based on the FS_B criterion since it is assumed that the structures are fully submerged within the flow.

6.7.2 Stream Power as Screening Criterion

From a management perspective, it would be desirable to develop a simple criterion that could be used to screen potential sites to determine their suitability for restoration. A number of such criteria have been suggested based on basin area (Metzger, 1997), channel width and slope (Cederholm et al, 1997a) and stream power (Doyle and Sheng, 1996). The field data from this study was used to determine if a simple relation based on stream power was evident.

As depicted in Figure 6-13, no relationship was found between unit stream power and the observed stability. When all the structures are ranked based on their unit stream power, the instabilities were proportionately distributed among 4 arbitrary stream power classes (< 125, 125 to 250, 250 to 375 and > 375 N/m/s). In fact, only 20% of structures within the upper class exhibited instabilities compared to about 33% of structures for the other 3 stream power classes. This would tend to indicate that the design and construction of the habitat structures investigated has a greater influence on the physical performance than the stream power. It also supports the use of velocity as the design criterion of single-LWD structure rather than stream power.

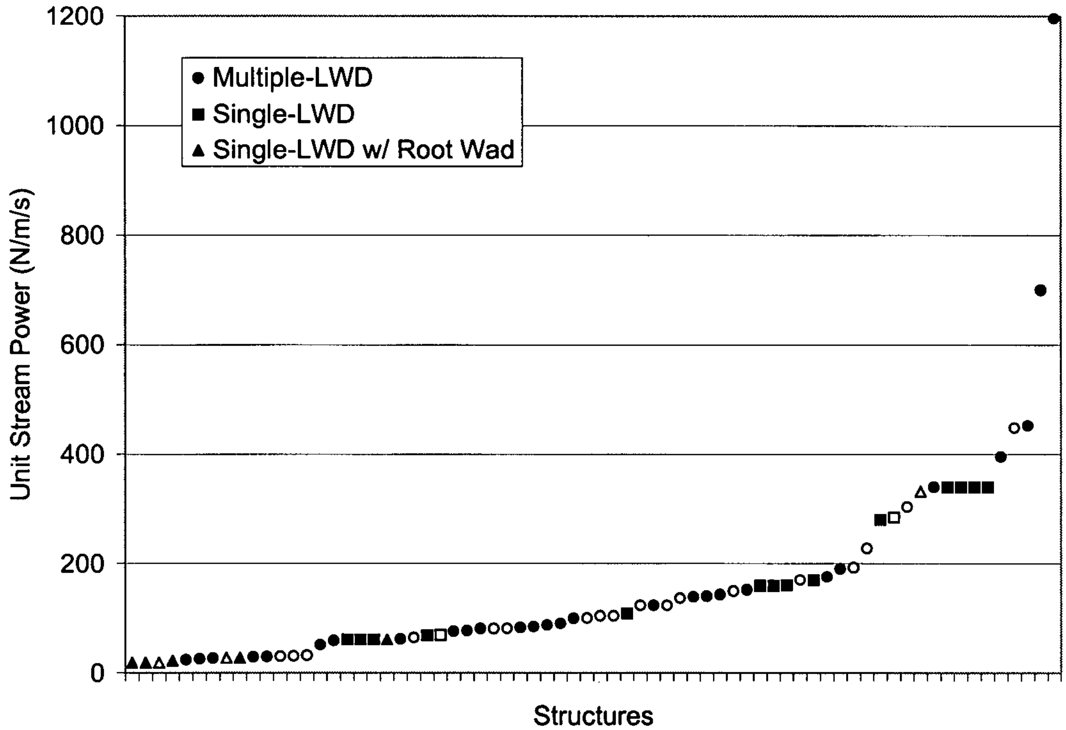


Figure 6-13: Observed structure stability vs stream power (solid symbols indicate stable structures and open symbols indicate unstable structures)

7 CONCLUSIONS AND RECOMMENDATIONS

The following chapter will summarise the results of the performance evaluations undertaken as part of this study for the three LWD habitat structures of interest. Recommendations will be made with respect to the suitability and application of the design guidelines. Furthermore, consideration will be given to construction, monitoring and maintenance of structures and research opportunities.

7.1 Single-LWD Structures

The preliminary design guidelines developed by Millar (1997) were successfully used to accurately predict the stability of single-LWD structures during the past fall to spring floods. Only two out of 22 structures did not follow predictions where partial burial of their anchor boulders increased the sliding resistance and prevented any movement. Hence, it is believed that the design approach based on FS_S can effectively be used to determine ballast mass requirements of single-LWD structures.

Most failures observed were the result of not securing the LWD at the bank end. To avoid such failures, a ballast factor (*BF*) equal to 2 must be used to double the mass requirement when no fixed bank anchor is provided. Grossly under-ballasted structures ($FS_B \leq 0.5$) sustained the most damage and were considered non-functional after the annual flood. Structures with a $FS_B > 0.8$ and $FS_S \leq 1.0$ were observed to have shifted but remained partially functional.

35% (8/23) of the single-LWD structures investigated as part of this study had sufficient ballast to provide a $FS_B > 1.25$ in the event of a 50 year flood. However, only 9% (2/23) of the structures could provide a $FS_S > 2.0$ under the same design flood conditions. In order to rectify this situation and ballast the structures according to the guidelines, an average of 1,100 kg of additional mass per under-ballasted structure would be required.

7.2 Single-LWD with Root Wad Structures

Keeping in mind the small sample size (n = 9), the approach used to predict the stability of single-LWD structures with intact root wads proved accurate. As predicted, six of the nine single-LWD with root wads investigated proved stable. Five of these structures would be able to maintain a $FS_B > 1.25$ in the event of a 50 year flood event. However, only three would be able to maintain a $FS_S > 2.0$ under these design-flood conditions. On average an additional 10 tonnes per under-ballasted structure is required to meet this criteria

Significant aggradation was observed behind 6 of the 9 structures thus smothering some of the anchor boulders and increasing the structural stability. This phenomenon is not taken into consideration during design since it is formed only once the structures have been subjected to scouring flows. These observations demonstrate the bank and bar sheltering/stabilisation potential of such structures.

Because of the limited sample size utilised to evaluate the design method, it is recommended that further monitoring of this type of structure be undertaken.

7.3 Multiple-LWD Structures

Based on the strict stability criterion established, about 25% of the multiple-LWD structures did not follow the predictions set out with the help of the design guidelines. A number of factors are suspected of having contributed to the observed instabilities:

- Increased chances of exhibiting movement due to nature of structures (i.e. multiple pieces of LWD).
- Failure of V-type structures which act as small dams and should be designed accordingly.
- Inadequate triangular bracing including loose cabling and/or no provision of fixed bank anchors.
- Significant LWD accumulation on upstream face of structure combined with some localised bed and bank scouring.

Conversely, it was observed that good triangular bracing and tight cabling of under-ballasted structures helped to ensure their physical integrity.

Considering the above factors, and recognising that the structures that exhibited significant damage had $FS_B < 1.25$, it appears that the design method holds merit. Furthermore, the successful use of the design method on single-LWD structures, which rely on the same fundamental principles in developing the FS_S and FS_B, indirectly lends support to its use for multiple-LWD structures. In order to improve the physical performance of such structures, it would be advisable to increase the minimum FS_B beyond the value of 1.25 recommended as part of the preliminary design guidelines (Millar, 1997). Some recommendations on the selection of an appropriate FS_B will be made in section 7.4.2.

Of the 51 multiple-LWD structures investigated, only about 20% have sufficient ballast in order to provide a FS_B of at least 1.25 in the event of a 50-year flood. Under-ballasted structures would require on average 1,100 kg/each (one 0.95 m diameter boulder per structure) to offer a minimum $FS_B > 1.25$ under design flood conditions.

7.4 Factors of Safety

7.4.1 Factor of safety against sliding (FS_S)

Based on the results obtained for both the single-LWD and single-LWD with root wad structures, there is a clear transition in observed stability as the FS_S increases above 1.0. Few structures fell within the range of $1.0 < FS_S < 2.5$ and therefore, no indication is provided to further refine the FS_S use in design. Hence, it is recommended that a minimum FS_S of 2.0 be maintained for design purposes in order to account for:

- Uncertainties in establishing hydraulic parameters during the design process e.g. flow velocity.
- Impacts and loading of additional LWD on structures.
- Other unaccountable factors such as quality of construction and materials, geomorphic stability of stream reaches.

A reduction in the minimum FS_S for design is not recommended since it would only provide a marginal reduction in mass requirement (approximately 10% mass reduction going from a FS_S of 2.0 to 1.8).

In cases where the rehabilitation site is in a higher risk area (i.e. subject to lots of floating debris, significant sediment transport, ice cover or entrenchment) the design FS_S should be increased above 2.0. A quick method, which may be used to make initial adjustments to the FS_S under such conditions, is to set it equal to the average flow velocity (in m/s) while targeting a range of FS_S between 2.0 and 3.5.

Note that both FS_S and FS_B criteria must be met when establishing ballast mass requirements; this is especially important for velocities inferior to 2.5 m/s since either FS_S or FS_B may govern.

7.4.2 Factor of safety against buoyancy (FS_B)

When examining the results of our three types of structures, it appears that the preliminary recommendation of a minimum **FS_B of 1.25 may be too low** to provide an acceptable level of safety and/or return on investment. For the single-LWD structures, instabilities were observed with FS_B as high as 1.3 whereas for multiple-LWD structures, instabilities occurred with FS_B as high as 2.2. Despite the increased difficulty in predicting the stability of multiple-LWD structures, a reduction in observed instabilities is apparent for a FS_B in excess of about 1.25.

It is therefore recommended that the minimum design **FS_B be increased to 1.5**. This measure would entail an increase in ballast mass requirement of up to 20% from the previously recommended FS_B of 1.25 (40% for FS_B = 1.75 from 1.25). Higher risk rehabilitation sites should provide an even higher level of safety. As explained in section 7.4.1, a quick method, which may be used to make initial adjustments to the FS_B under such conditions, is to set it equal to the average flow velocity (in m/s) while targeting a range of FS_B between 1.75 and 3.0.

It must be stressed that both FS_S and FS_B criteria must be met when establishing ballast mass requirements; this is especially important for velocities inferior to 2.5 m/s when either FS_S or FS_B may govern.

7.5 Design Considerations

A few steps should be observed during the design of LWD structures. The use of the following conservative design assumptions will simplify design computations:

- Assume that the LWD is fully submerged within the flow. This will eliminate the uncertainties inherent in establishing flood levels and corresponding submerged lengths of LWD while ensuring that the force distribution between anchors is consistent with the simplified method (Millar, 1997).
- The use of an equivalent trapezoidal cross-section is recommended to determine average channel velocity under flood conditions. This method can be readily solved (and calibrated if sufficient information exists) using a hand-held calculator or computer worksheet and would represent entrenched channel conditions. The method presented by Millar (1997) may also be used to estimate channel velocities.

The guidelines for multiple-LWD habitat structures are adequate for the design of floating LWD type structures aligned parallel to the flow.

The design guidelines evaluated herein are not to be used for full spanning structures such as V-type weirs. These structures act like dams and are subject to significant hydrostatic stresses that are not accounted for in the design guidelines. Furthermore, these structures may be more susceptible to moving debris as they have a significant impact on flow characteristics.

Design methodologies and design curves based on the recommendations herein are presented in Appendix D and E.

7.6 Construction Considerations

Based on field observations, recommendations can be formulated with respect to the construction of LWD structures.

- The use of fixed bank anchors has proved to provide a simple means of enhancing stability while significantly reducing ballast mass requirements. If no trees or stumps are present, doubling of the ballast mass requirement (through the use of a ballast factor) and fastening half of the boulders at the bank end of a single-LWD may replace a fixed bank anchor. Such a substitution is not recommended for multiple-LWD structures. However, if riparian trees are scarce and other anchoring methods not feasible, one of the anchors for multiple-LWD structures may be replaced with boulders. However, the ballast requirement for the piece of LWD with the substitute anchor should be determined from single-LWD guidelines (considering drag forces) with a ballast factor of 2 in order to double the mass required.

- Tight cabling between the LWD and anchors boulders, between the LWD and bank anchors and between LWD pieces is essential in preventing excessive "settling-in" and/or deformation of structures where the physical integrity may be lost.

- Adequate triangular bracing or cross bracing of multiple-LWD structures is important in ensuring that drag forces are evenly transferred to the anchors. An angle of about 60 degrees is recommended between the primary structural LWD members at the in-stream apex of the structure.

- Avoid overhanging LWD beyond the support points on the bank. Such overhangs act as teeter-totters and increase the destabilising forces transferred to the anchor boulders.

7.7 Monitoring and Maintenance Considerations

For many stream rehabilitation projects, the monitoring of LWD habitat structures is generally limited to personal observations. These post-implementation evaluations, if they occur, may last for only one or two years after construction and are seldom documented. A well established monitoring program is key in gaining a better understanding of the factors affecting the biological and physical performance of rehabilitation measures while providing the opportunity of formulating quantitative conclusions. Furthermore, routine monitoring of LWD structures aids in prescribing maintenance requirements to ensure long-term success and a good return on investment for in-stream work.

Koning et al. (1997), have presented evaluation techniques and provided guidelines to set up monitoring programs for various fish habitat restoration works.

7.8 Further Work/Research Opportunities

Pursuant to the research work conducted as part of this study a number of opportunities for further work have surfaced. These include:

- Continued monitoring and evaluation of single-LWD with intact root wad type structures. These structures appear promising in stabilising banks and bars while providing much-needed habitat within stressed systems. The small sample of structures evaluated as part of this study support the preliminary design guidelines, however, more information would be needed to corroborate these findings.

- Monitoring the use of the design guidelines in order to ensure that they are utilised effectively by watershed restoration practitioners. Undertake follow-ups on projects that have made use of the guidelines, verifying their adequacy with the practitioners and implementing changes to simplify their use.

- Evaluate other anchoring techniques. In many cases, riparian vegetation is scarce and inadequate in providing a good fixed bank anchor while some watersheds are devoid of good calibre ballasting boulders. Other anchoring systems, such as timber piles, dead-man, metal plates with cables, etc., can provide effective replacement anchors. A review of these and other methods could provide information with respect to design and construction specifications, costs, local impacts/benefits and be a useful tool for practitioners.

- Detailed study of hydrologic and morphologic interactions with habitat structures through evaluation of bed and bank scour processes. Very few known field and flume studies have concentrated on understanding the effect of LWD structures on the local stream morphology. Better knowledge of these processes would help in the prescription of rehabilitative works through a better control of pool size and depth, flow orientation and downstream influences.

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APPENDIX A: LIST OF SYMBOLS

Symbol	Units	Description (default value)
BF	-	Ballast factor
C_{DRW}	-	Drag coefficient of root wad (1.2)
C_{DB}	-	Drag coefficient of boulder (0.2)
C_{DL}	-	Drag coefficient of LWD (0.3)
C_{FL}	-	Skin friction coefficient of log (0.004)
C_{LB}	-	Lift coefficient of boulder (0.17)
D_B	m	Anchor boulder diameter
D_{Best}	m	Estimated anchor boulder diameter
D_L	m	Average diameter of log
D_{RW}	m	Average root wad diameter
ER	-	Entrenchment ratio; W_{MC}/W_{FP}
F_{BL}	N	Buoyancy forces of LWD transferred to anchor boulders
F_{DRW}	N	Drag forces acting on root wad transferred to anchor boulders
F_{DB}	N	Drag forces acting on anchor boulders
F_{LB}	N	Lift forces acting on anchor boulders
F_{FL}	N	Frictional forces acting on log transferred to anchor boulders
F_F	N	Frictional forces resisting sliding
FS_B	-	Factor of safety against buoyant uplift
FS_S	-	Factor of safety against sliding
g	m/sec^2	Gravitational acceleration (9.806 m/sec^2)
L	m	Length of LWD excluding root wad
L_{RW}	m	Length of root wad
L_S	m	Submerged length of LWD
M_B	kg	Ballast mass required to provide FS_B against buoyancy
M_{BL}	kg	Ballast mass required to counter buoyancy of LWD
M_{BLWD}	kg	Ballast mass required to counter buoyancy of LWD (w/ FS_B)
M_{DRW}	kg	Ballast mass required to counter root wad drag forces
M_{DB+LB}	kg	Ballast mass required to counter boulder drag and lift forces
M_{LB}	kg	Ballast mass required to counter boulder lift forces
M_S	kg	Ballast mass required to provide FS_S against sliding
S_L	-	Specify gravity of LWD; typical range 0.36-0.54 (0.50)
S_S	-	Specific gravity of anchor boulders (2.65)
V	m/sec	Average stream flow velocity
W'	N	Immersed weight of anchor boulders
W_{FP}	m	Width of floodplain
W_{MC}	m	Width of main channel
β	°	Angle of single LWD or root wad face w/r to flow (90°)
ϕ	°	Friction angle of anchor boulders on stream bed (40°)
ρ	kg/m^3	Density of water (1,000 kg/m^3)

APPENDIX B: LWD STRUCTURE ASSESSMENT FORM

LWD STRUCTURE ASSESSMENT FORM

WATERSHED

Watercourse name		Date	
Reach		Time	
1:50,000 ref. map No.		Survey crew	

SITE (Refer to Scketch No. 1)

Structure No.		Streamflow	
Feature (R/G/P)			
Pool	Max. depth	Crest depth	Residual
Bed material	Dominant	Sub-dominant	Largest mobile
Bank material	Dominant	Sub-dominant	Largest mobile
Width	Wetted	Bankfull	
Cover	%	Type	Vegetation

CROSS-SECTION SURVEY

BS:	FS Readings				Elevations			
Distance[1]	m u/s	C/L	m d/s		m u/s	C/L	m d/s	
Left bank[2]								
Highwater								
Bankfull								

2 - looking u/s

1 - from benchmark

LWD Structure Assessment Form

Structure type		Pin 1 to CS Pin left		Pin 2 to CS Pin left	
Pin 1 to Pin 2		Pin 1 to CS Pin right		Pin 2 to CS Pin left	

LWD details (Refer to Sketch No.2)

No.	Diameter (m)		Length	Location (bank end)				Location (stream end)			
	Butt	Root wad	(m)	Pin 1 (m)	Pin 2 (m)	FS (m)	Elev. (m)	Pin 1 (m)	Pin 2 (m)	FS (m)	Elev. (m)

Anchor boulder details

No.	Diameter (m)				LWD No.& Loc.	Location			
	a	b	c	mean	(bank, middle, stream)	Pin 1 (m)	Pin 2 (m)	FS (m)	Elevation (m)

LWD STRUCTURE ASSESSMENT FORM

Centerline Profile Survey

Description	Upper	Center	Lower	Difference	Distance	Elevation

Sketch No.2: Structure Details

LWD STRUCTURE ASSESSMENT FORM

Pebble count	
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Sketch No.1: Reach/Location Details

Comments:

APPENDIX C: ASSESSMENT SUMMARY TABLES

Structure No.	System	Date	G/P/R	Resid Pool	Bd Dom	Bd S-Dom	Bd LM	Bk Dom	Bk S-Dom	Ww	Flow	Wb	Cover %	Cover type
WK-8	West Kettle	08/10/97			cobble			earth	cobble	20.2	12.3	40		LWD
WK-1a	West Kettle	09/10/97			cobble	gravel	15	gravel	sand	20.8	12.3	44		LWD
WK-1b	West Kettle	08/10/97			cobble	gravel	15	gravel	sand	20.8	12.3	44		LWD
WK-4	West Kettle	08/10/97			cobble	gravel		cobble	gravel	26.7	12.3	36		LWD
WK-6	West Kettle	09/10/97			cobble	gravel	10	cobble	gravel	33.6	12.3	42		LWD
WK-7	West Kettle	09/10/97			cobble	gravel		cobble	earth	25.7	12.3	35	5	trees
WK-5	West Kettle	09/10/97			cobble	gravel		cobble	earth	26.8	12.3	34		LWD
WK-2	West Kettle	08/10/97	G		cobble	gravel		cobble	gravel	35.9	12.3	58		LWD
WK-9	West Kettle	09/10/97			cobble	gravel		gravel	cobble	48.6	12.3	60		LWD
LC-606	Lost Creek	10/10/97			cobble	boulder		gravel					60	LWD
LC-604	Lost Creek	10/10/97	P	0.55	cobble	gravel	15	gravel	cobble	4.7		9	20	LWD,Boulder
LC-605	Lost Creek	10/10/97		0.5	cobble	gravel		gravel	cobble, earth	8.5		10	15	LWD
L-2	Lukwa	29/08/97	R		cobble	gravel	10	gravel	sand,earth	5.4		12	20	LWD,Boulders
L-1	Lukwa	29/08/97	P	0.65	gravel	cobble	10	sand	gravel	6		13	40	LWD,Boulders
W3	Keogh	23/08/97	P		cobble	gravel	15	boulder	cobble	6.1		17	15	LWD,Boulder
W4	Keogh	23/08/97	P/G		boulder	cobble	15	gravel	cobble	10.3		16	5	LWD,Boulders
W6	Keogh	23/08/97	R		boulder	cobble	20	gravel	cobble	6.6		30	15	LWD,boulders
W8	Keogh	23/08/97	P	0.67	cobble	gravel		earth		9		11	15	trees
W10	Keogh	23/08/97	P	0.71	cobble	gravel		gravel	earth	9.3		12	50	trees
103-104-8ab	Keogh	25/08/97	P	0.65	cobble	gravel	7	gravel	earth,cobble	7.5		35	30	LWD,boulders
103-104-8efg	Keogh	25/08/97	P	0.65	cobble	gravel	7	gravel	earth,cobble	7.5		35		LWD,boulders
103-104-8cd	Keogh	25/08/97	P	0.65	cobble	gravel	7	gravel	earth,cobble	7.5		35		LWD,boulders
103-104-32	Keogh	25/08/97	G		gravel	cobble		gravel	earth	7.4		16		LWD,boulders
103-104-33	Keogh	25/08/97	P	0.58	gravel	cobble		gravel	earth	7.4		16		LWD,boulders
103-104-39	Keogh	25/08/97	P		cobble	gravel		earth	cobble	8		15	20	LWD,boulders
103-104-47	Keogh	25/08/97	P	0.6	cobble	gravel		gravel	earth	8		23	70	tree,LWD,Boulders
103-104-53	Keogh	22/08/97	G/R		cobble	gravel	15	gravel		6.1		15	5	LWD,boulders
103-104-54	Keogh	22/08/97	P	0.35	cobble	gravel		earth	gravel			18		LWD,boulders
T13-1	Keogh	22/08/97	P	0.68	gravel	cobble		gravel		9.8		16	50	trees
T13-2	Keogh	22/08/97			cobble	gravel		gravel	cobble	6.3		55	75	trees
T13-3	Keogh	22/08/97	P	0.27	cobble	gravel		gravel	cobble	7.5		20+	50	trees,LWD,boulders
T13-5	Keogh	28/08/97	P	0.6	cobble	gravel		sand	earth	12.5		22		LWD,boulders
WM-8	Keogh	27/08/97	P		cobble	gravel	15	gravel	cobble	9.3		16	40	LWD,boulders
WM-13	Keogh	27/08/97	R		cobble	gravel		gravel	sand	10.3		19	75	trees
WM-24	Keogh	27/08/97	R		cobble	boulders	15	cobble	gravel	14.5		18	20	LWD,boulders
WM-25a	Keogh	27/08/97	R		cobble	boulders	15	cobble	gravel	14.5		18		LWD,boulders
WM-25b	Keogh	27/08/97	R		cobble	boulders	15	cobble	gravel	14.5		18		LWD,boulders
WM-27	Keogh	27/08/97	P		cobble	boulders	15	cobble	gravel	14.5		18	20	LWD,boulders
WM-26	Keogh	27/08/97	P		cobble	boulders	15	cobble	gravel	14.5		18		LWD,boulders
WM-39a	Keogh	28/08/97	G/R		cobble	gravel		sand	gravel	15.1		17		LWD,boulders
WM-39b	Keogh	28/08/97	G/R		cobble	gravel		sand	gravel	15.1		17		LWD,boulders
W80-2	Keogh	20/08/97	R/G		cobble	gravel	15	gravel	sand	11.4		15		shrubs,trees
W80-3	Keogh	20/08/97	R/G		cobble	gravel	15	gravel	sand	11.4		15		shrubs,trees
W80-8	Keogh	21/08/97	P		cobble	gravel		gravel	cobble	7.7		16	40	LWD,boulders

Structure No.	Vegetation	X-Sections	Type	# Boulds	# LWD	Pebble	Comments
WK-8	conifers	4,2,0,-2	T	10	5	No	
WK-1a	grasses	4,2,0,-2	T	9	4	No	
WK-1b	grasses	4,2,0,-2	S	1	1	No	
WK-4	grasses,alders	4,2,0,-2	T	11	3	Yes	
WK-6	alders,cedars	3,2,0,-2	T	6	3	No	
WK-7	alders,cedar	4,2,0,-2	T	6	3	No	
WK-5	shrubs,trees	4,2,0,-2	C	5	4	Yes	
WK-2	grasses	4,0,-4	T	11	3	No	
WK-9	shrubs,cottonwoods conifers	2,0,-2	T	10	4	No	
LC-606	shrubs,conifers	n/a	C	2	7	No	Took only picture assessment
LC-604	shrubs,alders	2,0,-2,-4	T	3	4	No	
LC-605	shrubs,alders	2,0,-2	T	1	3	No	
L-2	hemlock	2,0,-2	T	4	3	Yes	
L-1	hemlock	2,0,-2,-4	C	8	11	Yes	
W3	shrubs,alders, cedars	4,2,0,-2	T	4	3	Yes	2nd structure from top of Wolf trib
W4	shrubs,cedars	1,0,-1,-2	V	4	3	Yes	
W6	alders	3,2,0,-2	V	6	3	Yes	Old channel/puddle on LHS w/ fry
W8	alders,shrubs	2,0,-1,-2	T	3	2	Yes	
W10	conifer,alders	2,0,-2,-4	C	3	5	Yes	Structure is collecting CPOM
103-104-8a b	dead alders, low shrubs	2,0,-2	T	2	2	Yes	Overflow channel behind bar (RHS)
103-104-8efg	dead alders, low shrubs		C	1	3	Yes	Overflow channel behind bar (RHS)
103-104-8c d	dead alders, low shrubs		T	1	2	Yes	Overflow channel behind bar (RHS)
103-104-32	shrubs		S	3	1	Yes	
103-104-33	shrubs	4,3,2,0	T	3	4	Yes	
103-104-39	alders,salmonberry	2,1,0,-2	T	9	4	Yes	Lots of crayfish, Old overflow channel (RHS)
103-104-47	hemlock,alders	2,1,0,-2	T	6	6	Yes	Crest gauge on hemlock
103-104-53	shrubs,alders,grasses	2,0,-2	V	9	2	Yes	
103-104-54	shrubs,alders	2,0,-2	S	1	2	Yes	Old crossing abutments immediately d/s
T13-1	hemlock,alders	2,0,-2,-3	T	4	3	Yes	Densely veg. side channels to NE inv. 0.5m abv H2O
T13-2	alders	2,0,-2	S	2	1	Yes	68m vw-vw, old chanls/puddles w/ fry on both sides
T13-3	salmonberry,alders	3,2,0,-2	T	3	2	Yes	Refer to Trib 13 #2 for pebble count and scketch
T13-5	alders,shrubs	2,0,-2	S	2	1	Yes	
WM-8	alders,salmonberry	2,0,-2,-4	T	17	7	Yes	Crest gauge on alder u/s on left bank.
WM-13	alder	2,0,-2	T	7	3	Yes	Crest gauge on alder u/s of structure
WM-24	aldera,salmonberry,hemlock		S	2	1	Yes	Steep banks on either side
WM-25a	aldera,salmonberry,hemlock		S	2	1	Yes	Steep banks on either side
WM-25b	aldera,salmonberry,hemlock		S	4	1	Yes	Steep banks on either side
WM-27	aldera,salmonberry,hemlock	4,2,0,-2	C	2	1	Yes	Steep banks on either side
WM-26	aldera,salmonberry,hemlock	4,2,0,-2	S	3	1	Yes	Steep banks on either side
WM-39a	alders,hemlock	2,0,-2	S	2	1	Yes	
WM-39b	alders,hemlock	2,0,-2	S	2	1	Yes	
W80-2	alders,berry	2,0,2	S	2	1	Yes	Crest gauge u/s of structure (RHS)
W80-3	alders,berry	4,2,0,2	S	2	1	Yes	Refer to W80-2 for PC and scketch, LWD on LHS
W80-8	alder,shrubs	2,0,-1,-2	T	5	6	Yes	

Structure No.	System	Date	G/P/R	Resid Pool	Bd Dom	Bd S-Dom	Bd LM	Bk Dom	Bk S-Dom	Ww	Flow	Wb	Cover %	Cover type
W80-14	Keogh	21/08/97	P		cobble	gravel	15	clay		8		15		LWD,boulders
W80-15	Keogh	21/08/97	P	0.45	Boulder	cobble	18	clay		8.2		17	5	LWD,boulders
W80-18	Keogh	21/08/97	P	0.67	cobble	gravel	15	gravel	cobble	7.4		23	5	shrubs,LWD,boulders
Sh-4	Shovelnose	28/10/97			sand	cobble		cobble	boulder	26		32	5	LWD,boulders
Sh-6	Shovelnose	28/10/97			sand	gravel		boulder	cobble	14		34		LWD,boulders
Sh-10	Shovelnose	28/10/97			sand	gravel		boulder	cobble	31		38		LWD,boulders
Sh-9	Shovelnose	29/10/97			sand	gravel		sand	boulder	31		38		LWD,boulders
Sh-11	Shovelnose	29/10/97			sand	gravel		sand	boulder	31		38		LWD,boulders
Sh-13	Shovelnose	30/10/97			sand	cobble		cobble	boulder	33.7	6.3	40	5	LWD,boulders
Sh-15	Shovelnose	30/10/97			sand	gravel		boulder	cobble	33.4	6.3	41		LWD,boulders
Sh-14	Shovelnose	30/10/97	G		boulder	cobble		boulder	cobble	31.8	6.3	42		LWD,boulders
S-11	Sampson	24/10/97	G		gravel	sand		sand	silt			15	15	LWD,brush
S-9	Sampson	24/10/97			gravel	sand		silt	sand	9		10	20	LWD,brush
S-12	Sampson	24/10/97	P		sand	gravel		sand	silt	13.5		14.5	20	LWD
S-1	Sampson	23/10/97			gravel	sand		sand	gravel	11.5		12.5	40	LWD,boulders,brush
S-2	Sampson	23/10/97	P		gravel	sand		sand	gravel	10.5		13	35	LWD,boulders,brush
S-4	Sampson	23/10/97	P		gravel	sand		sand	clay	9.4		15.5	35	LWD,boulders,brush
S-6	Sampson	24/10/97	R/G		gravel	sand		earth	silt	13.5		14		LWD,boulders,brush
S-7	Sampson	24/10/97	P		gravel	sand		silt	sand	10.5		18		LWD,boulders,brush
SJ-108	San Juan	26/09/97			sand	gravel		gravel	earth	80		130	5	LWD,boulders
SJ-109	San Juan	26/09/97			gravel	sand		gravel	sand					LWD,boulders
SJ-103a	San Juan	25/09/97			gravel	cobble	10	gravel	sand	28		145		
SJ-103b	San Juan	25/09/97			gravel	cobble	10	gravel	sand	28		145		
SJ-104	San Juan	25/09/97			gravel	cobble	10	gravel	sand	28		145		
SJ-101	San Juan	24/09/97	P	1.6	gravel	sand		silt	sand	40		100	30	LWD,boulders
SJ-106	San Juan	25/09/97	G	1.1	gravel	cobble		silt	sand	58		95	10	LWD,boulders
SJ-102	San Juan	24/09/97	P		gravel	sand		silt	sand	41		100		LWD,boulders

Structure No.	Vegetation	X-Sections	Type	# Boulds	# LWD	Pebble	Comments
W80-14	shrubs	2,1,0,-1,-2	T	6	3	Yes	Structure almost like a compound single?
W80-15	conifers,shrubs	3,2,0,-2	RW	4	1	Yes	Structure 16 across is incomplete
W80-18		2,0,-1,-2	S	2	1	Yes	
Sh-4	alders	2,0,-2,-4	T	12	6	No	Intended for bar formation?
Sh-6	small alders	2,0,-2,-4	T	8	5	No	Intended for bar formation?
Sh-10	none	4,2,0,-2	S/C	13	4	No	
Sh-9	alders		RW	8	1	No	
Sh-11	alders	2,0,-2,-4	C	7	3	No	
Sh-13	alders 6m	2,0,-2,-4	C	12	4	No	
Sh-15	alders 6m	2,0,-2,-4	C	12	6	No	
Sh-14	alders 6m	2,0,-2,-4	C	14	7	No	
S-11	alders	2,0,-2,-4	T	0	2	No	1 instream boulder not anchored?
S-9	alders, cedars,shrubs	7,2,0,-2,-4	T	2	3	No	No instream anchor boulders (not natural???)
S-12	cedars, grasses	2,0,-2,-4	T	2	4	No	No instream anchoring (ran out of time)?
S-1	alders,cottonwood,cedar	2,0,-2,-4	T	4	3	No	Potential of ice cover in Sampson - check
S-2	cottonwood,pine, alder,cedar	2,0,-2,-4	T	1	4	No	Lots of alder and brush in creck
S-4	cedar,alders	2,0,-2,-4	T	3	5	No	
S-6	alders,cedar	2,0,-2,-4	T	2	2	No	Some loose cabling used to hold falling alder
S-7	alders,grasses	2,0,-2,-4	C	3	4	No	
SJ-108	spruce,alders	3,0,-3	RW	13	1	No	V large spruce next to cut bank
SJ-109	spruce,alders		RW	12	2	No	Side channel, only dims and orientation taken
SJ-103a	alders	0	RW	6	1	Yes	High and dry - island stabilisation
SJ-103b	alders	0	RW	6	1	Yes	High and dry - island stabilisation
SJ-104	alders	0	RW	6	5	Yes	High and dry - island stabilisation
SJ-101	alders	3,0,-3	RW	2	1	Yes	fish present, refer to No. 102 for scketch
SJ-106	alders	4,0,-4	RW	6	1	No	V large spruce next to cut bank
SJ-102	alders	3,0,-3	RW	2	3	Yes	refer to PC No. 101

Structure No.	FS_B						FS_S						Equiv. Trapezoidal Channel			
	Highwater		Bankfull		Design		Highwater		Bankfull		Design		Slope	n	Bottom	d50
Method	Millar	Full	Millar	Full	Millar	Full	Millar	Full	Millar	Full	Millar	Full			Width (m)	cm
103-104 Single 2	**0.51**	**0.51**	-	-	-	-	**-3.31**	**-3.31**	-	-	-	-				
103-104-54	**0.36**	**0.31**	**0.35**	**0.30**	**0.26**	**0.26**	**-2.90**	**-2.91**	**-2.88**	**-2.91**	**-1.61**	**-1.61**	**0.28%**	**0.038**	**7.0**	**4.50**
103-104 Single 3	**0.34**	**0.34**	-	-	-	-	**-2.41**	**-2.41**	-	-	-	-				
W80-2	**0.94**	**0.82**	**0.90**	**0.80**	**0.74**	**0.74**	**-0.24**	**-0.68**	**-0.37**	**-0.73**	**-0.47**	**-0.47**	**0.75%**	**0.045**	**11.0**	**5.65**
103-104 Single 4	**0.97**	**0.83**	-	-	-	-	**-0.13**	**-0.60**	-	-	-	-				
103-104 Single 1	**1.02**	**1.02**	-	-	-	-	**0.07**	**0.07**	-	-	-	-				
103-104 Single 5	**1.02**	**1.02**	-	-	-	-	**0.07**	**0.07**	-	-	-	-				
T13-5	1.11	1.11	1.11	1.11	1.06	1.06	0.35	0.35	0.34	0.34	0.09	0.09	0.49%	0.045	12.5	7.25
T13-2	1.19	1.07	1.20	1.07	0.99	0.99	0.56	0.20	0.59	0.21	-0.02	-0.02	0.40%	0.045	10.0	6.50
WM Single	**1.33**	**1.33**	-	-	-	-	**0.91**	**0.91**	-	-	-	-				
WM-39b	1.44	1.31	1.42	1.30	1.22	1.22	1.76	1.23	1.65	1.17	0.39	0.39	0.54%	0.050	16.0	6.75
Sh-10c	1.38	1.60	1.65	1.65	1.62	1.62	2.17	2.95	3.03	3.63	1.62	3.09	0.45%	0.045	28.0	
WM-39a	1.63	1.50	1.61	1.49	1.41	1.41	2.86	2.29	2.70	2.20	0.84	0.84	0.54%	0.050	16.0	6.75
Sh-10a	1.40	10.24	1.18	2.84	0.74	0.74	2.98	9.47	1.15	5.71	-0.96	-1.88	0.45%	0.045	28.0	
103-104-32	2.44	2.11	2.27	1.97	1.58	1.58	3.56	2.58	2.94	2.15	0.74	0.74	0.66%	0.045	4.5	5.25
WM-24	2.46	2.13	2.38	2.07	1.77	1.77	3.62	2.72	3.35	2.55	0.98	0.98	0.86%	0.065	9.5	7.50
Sh-10b	1.69	2.69	1.35	1.50	1.20	1.20	3.77	5.82	1.61	2.48	0.53	1.00	0.45%	0.045	28.0	
WM-26	2.31	2.28	2.28	2.28	2.16	2.16	4.73	4.65	4.51	4.51	2.05	2.05	0.86%	0.065	9.5	7.50
WM-25b	2.31	2.16	2.23	2.04	1.24	1.24	5.70	4.19	5.28	3.81	0.59	0.59	0.86%	0.065	9.5	7.50
SJ-109b	4.90	-	-	-	-	-	7.86	-	-	-	-	-		0.033	25.0	
W80-3	6.20	5.42	6.03	5.29	4.10	4.10	9.14	7.60	8.67	7.28	3.01	3.01	0.75%	0.050	10.0	5.65
WM-25a	4.33	8.35	4.18	7.1	1.83	1.64	11.23	9.72	10.6	9.1	1.80	1.40	0.86%	0.065	9.5	7.50
W80-18	-	-	3.05	2.74	1.77	1.72	-	-	4.93	3.64	0.90	0.84	0.76%	0.045	23.0	5.75

Structure No.	Inter-assesment High Flow			Bankfull Conditions					Design Flow Conditions		
	Depth	Computed	Velocity	Depth	Computed	Velocity	S.Power	Tau*	Depth	Computed	Velocity
Method	m	Flow cms	m/s	m	Flow cms	m/s	N/m/s		m	Flow cms	m/s
103-104 Single 2						**2.0**					
103-104-54	**1.46**	**30**	**1.7**	**1.49**	**31**	**1.7**	**70**	*0.056*	**2.65**	**107**	**2.5**
103-104 Single 3						**2.0**					
W80-2	**1.53**	**46**	**2.3**	**1.64**	**52**	**2.4**	**284**	*0.132*	**3.29**	**183**	**3.5**
103-104 Single 4						**2.0**					
103-104 Single 1						**2.0**					
103-104 Single 5						**2.0**					
T13-5	1.24	38	1.8	1.27	40	1.8	109	0.052	2.38	140	2.7
T13-2	1.04	41	1.7	1.03	40	1.7	69	0.038	1.74	140	2.5
WM Single						**2.1**					
WM-39b	1.61	50	1.8	1.65	52	1.8	160	*0.080*	3.53	183	2.8
Sh-10c	0.82	34	1.3	0.96	45	1.4	61		1.51	102	1.9
WM-39a	1.61	50	1.8	1.65	52	1.8	160	*0.080*	3.53	183	2.8
Sh-10a	0.82	34	1.3	0.96	45	1.4	61		1.51	102	1.9
103-104-32	1.14	27	2.1	1.22	31	2.2	170	*0.093*	2.05	107	3.1
WM-24	1.97	50	1.9	2.02	52	2.0	336	*0.140*	3.91	183	2.9
Sh-10b	0.82	34	1.3	0.96	45	1.4	61		1.51	102	1.9
WM-26	1.97	50	1.9	2.02	52	2.0	336	*0.140*	3.91	183	2.9
WM-25b	1.97	50	1.9	2.02	52	2.0	336	*0.140*	3.91	183	2.9
SJ-109b						1.7					
W80-3	1.67	50	2.2	1.71	52	2.2	280	*0.138*	3.30	183	3.3
WM-25a	1.97	50	1.9	2.02	52	2.0	336	*0.140*	3.91	183	2.9
W80-18	0.35		0.9	1.10		2.0	161	*0.088*	2.34	183	3.1

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Structure No.			Movement		
	Max. movement (m)		Criterion	Comments	Still
Method	LWD	Boulders	0.5		Functional
103-104 Single 2	**>5**		**moved**		**n**
103-104-54	**9.55**	**6.6**	**moved**	**log not tied back, washed along shore at 104 x-ing**	**n**
103-104 Single 3	**>5**		**moved**		**n**
W80-2	**0.6**	**0.7**	**moved**	**some slight movement, may be caused by backwater effects from W80 bridge**	**y**
103-104 Single 4	**>5**		**moved**		**y/partial**
103-104 Single 1	**>5**		**moved**		**y/partial**
103-104 Single 5	**0.7**		**moved**	**old cedar log across from 54**	**y**
T13-5	0.1	0.15		boulders are partially burried in substrate, excavated pool partially filled in	y
T13-2	0.1	0.1		boulders are partially burried in substrate, some scouring d/s right	y
WM Single	**>5**		**moved**		**y/partial**
WM-39b	0	0.1		resting against alders on d/s side	y
Sh-10c	0.05				y
WM-39a	0.05	0.2		resting against alders on d/s side	y
Sh-10a	0.15				y
103-104-32	0.15	0.2			y
WM-24	0	0.1		no significant change, located in cobble/boulder riffle	y
Sh-10b	0.2				y
WM-26	0.05	0.15		some scour d/s of l	y
WM-25b	0.35	0.7		no significant change, located in cobble/boulder riffle	y
SJ-109b	0	0			y
W80-3	0.15	0.1		potential backwater effects from W80 bridge	y
WM-25a	0.15	0.15		no significant change, located in cobble/boulder riffle	y
W80-18	n/a	n/a	moved	structure destroyed by windall of anchor tree on bank.	y/partial

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Structure No.	Governing FS						Boulder Mass (kg)				
	Highwater		Bankfull		Design		Provided	Required		Additional Required	
Method	Millar	Moments	Millar	Moments	Millar	Moments		Q 2.33	Q 50	Q 2.33	Q 50
103-104 Single 2	**FSs**	**FSb**									
103-104-54	**FSs**	**FSb**	**FSs**	**FSb**	**FSs**	**FSb**	402	2375	3375	1973	2973
103-104 Single 3	**FSs**	**FSb**									
W80-2	**FSs**	**FSs**	**FSs**	**FSs**	**FSs**	**FSs**	1624	0	0		
103-104 Single 4	**FSs**	**FSs**									
103-104 Single 1	**FSs**	**FSs**									
103-104 Single 5	**FSs**	**FSs**									
T13-5	FSs	FSs	FSs	FSs	FSs	FSs	673	1025	1663	352	990
T13-2	FSs	FSs	FSs	FSs	FSs	FSs	582	974	1563	392	982
WM Single	**FSs**	**FSs**									
WM-39b	FSb	FSs	FSb	FSs	FSs	FSs	1503	1801	2770	298	1268
Sh-10c	FSb	FSb	FSb	FSb	FSb	FSb	998	601	774		
WM-39a	FSb	FSb	FSb	FSb	FSs	FSs	2349	2305	3420		1070
Sh-10a	FSb	FSs	FSs	FSb	FSs	FSb	1359	1216	1490		132
103-104-32	FSb	FSb	FSb	FSb	FSs	FSs	1278	1385	2237	108	959
WM-24	FSb	FSb	FSb	FSb	FSs	FSs	1707	1622	2599		892
Sh-10b	FSb	FSb	FSb	FSb	FSs	FSs	923	550	709		
WM-26	FSb	FSb	FSb	FSb	FSs	FSs	3598	2402	3539		
WM-25b	FSb	FSb	FSb	FSb	FSs	FSs	2154	2360	3301	206	1146
SJ-109b				FSb							
W80-3	FSb	FSb	FSb	FSb	FSs	FSs	4629	1857	3225		
WM-25a	FSb	FSb	FSb	FSb	FSs	FSs	2490	2167	3070		580
W80-18			FSb	FSb	FSs	FSs	1207	1127	2046		839
						Average	1717	1485	2236	555	1076

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							Equiv. Trapezoidal Channel			
	FS_B			FS_S			Slope	n	Bottom	d50
	Highwater	Bankfull	Design	Highwater	Bankfull	Design			Width	m
SJ-104	0.96	0.96	0.95	-0.18	-0.11	-0.08	0.036%	0.026	145.0	0.033
W80-15*	1.02	1.02	0.98	0.04	0.04	-0.02	0.800%	0.050	7.0	0.073
SJ-106	1.10	1.10	1.09	0.50	0.30	0.21	0.036%	0.026	85.0	0.033
SJ-107	1.44	1.44	1.43	1.19	1.18	0.89	0.036%	0.026	85.0	0.033
Sh-9*	1.24	1.24	1.22	1.85	1.47	0.76	0.450%	0.045	28.0	
SJ-108	1.48	1.47	1.47	2.91	1.58	1.08	0.036%	0.026	122.5	0.033
SJ-103a	2.44	2.42	2.39	3.97	3.25	2.40	0.036%	0.026	145.0	0.033
SJ-103b	3.83	3.80	3.73	6.07	4.97	3.68	0.036%	0.026	145.0	0.033
SJ-109a	4.33			6.83						
average										

Single-LWD with Root Wad

	Inter-assessment High Flow			Bankfull Flow Conditions					Design Flow Conditions		
	Depth	Computed	Velocity	Depth	Computed	Velocity	S.Power	tau*	Depth	Computed	Velocity
	m	Flow cms	m/s	m	Flow cms	m/s	N/m/s		m	Flow cms	m/s
SJ-104	2.80	612	1.5	3.26	791	1.6	18	0.022	4.10	1165	1.9
W80-15*	1.83	56	2.4	1.77	52	2.4	332	0.118	3.20	183	3.5
SJ-106	3.08	468	1.6	4.12	791	1.9	28	0.028	4.12	791	2.2
SJ-107	4.10	782	1.9	4.12	791	1.9	28	0.028	4.12	791	2.2
Sh-9*	0.82	34	1.3	0.96	45	1.4	61	0.262	1.51	102	1.9
SJ-108	2.76	492	1.4	3.67	791	1.7	22	0.025	4.63	1165	2.0
SJ-103a	2.80	612	1.5	3.26	791	1.6	18	0.022	4.10	1165	1.9
SJ-103b	2.80	612	1.5	3.26	791	1.6	18	0.022	4.10	1165	1.9
SJ-109a						1.7					2.0
Average			1.6			1.8					2.2

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Single-LWD with Root Wad

	Max. movement (m)		Comments	Still
	LWD	Boulders		Functional
SJ-104	>5	>5	gone	n
W80-15*	0.7	0.7	slight adjustment toward bank, some scouring on bank opposite LWD and RW	y
SJ-106	0.7	1.1	difficult to measure, however C/L of log when down 0.3 m, appears to have rotated a bit.	y
SJ-107			no apparent movement, photo assessment only	y
Sh-9*				
SJ-108			intact, some sand/gravel accumulation downstream of wad, less earth in wad since 1st assessment.	y
SJ-103a	0.7	0.6	shift difficult to measure, smothering of boulders behind wad, some settling in of boulders	y
SJ-103b		0.5	smothering of boulders behind wad, some settling in of boulders	y
SJ-109a			no apparent movement, significant scour & aggradation	y
average				

Single-LWD with Root Wad

	Design mass requirement (kg)						
	Actual	Sliding (FSs = 2.0)		Buoy. (FSb = 1.25)		Difference	
		2 y	50 y	2 y	50 y	2 y	50 y
SJ-104	5,383	11,233	**13,269**	7,134	7,190	5,850	7,886
W80-15*	2,876	7,380	**13,261**	3,627	3,872	4,504	10,385
SJ-106	13,694	23,308	**26,996**	15,777	15,900	9,614	13,302
SJ-107	33,963	42,042	**48,257**	29,798	29,987	8,079	14,293
Sh-9*	2,560	2,650	**3,354**	2,367	2,418	90	793
SJ-108	31,343	39,107	**45,490**	26,794	26,948	7,764	14,147
SJ-103a	4,281	3,368	3,976	2,211	2,237		
SJ-103b	5,295	2,962	3,563	1,723	1,745		
SJ-109a	7,937	2,543	3,009	1,607	1,630		
average	11,926	14,955	17,908	10,115	10,214	5,984	10,134

Structures	FS$_B$ Highwater Millar	Highwater Moments	Bankfull Millar	Bankfull Moments	Design Millar	Design Moments	Equivalent Trapezoidal Channel Slope	n	Bottom Width	Substrate d50 cm	d90 cm
S-11	0.16	-0.10	0.06	0.08	0.04	0.04	0.21%	0.045	8.0		
103-104 DF 2	0.19	0.19									
103-104 DF 3	0.24	0.24									
S-7	0.32	0.37	0.25	**0.25**	0.24	0.24	0.21%	0.045	4.0		
103-104 DF 1	0.41	0.41									
S-12	0.42	4.83	0.29	0.44	0.22	0.22	0.21%	0.065	9.0		
103-104 DF 4	0.50	0.50									
S-2	0.51	0.75	0.48	0.59	0.38	**0.35**	0.21%	0.045	10.0		
103-104-8efg	0.52	0.52	0.52	0.52	0.50	0.50	0.43%	0.045	3.0	6.50	14.00
Wolf Triangular	0.52	0.52						0.040	0.0		
103-104-8ab	0.55	0.51	0.54	**0.50**	0.48	0.48	0.43%	0.045	3.0	6.50	14.00
LC-604	0.60	1.42	0.57	1.14	0.40	0.43	2.25%	0.080	3.0		
Lu-1	0.71	**0.69**	0.72	**0.70**	0.67	0.67	0.60%	0.045	12.0	3.00	8.00
103-104-33	0.72	1.07	0.70	1.00	0.58	0.65	0.66%	0.045	4.5	5.25	11.50
W80-8	0.73	0.68	0.71	**0.65**	0.52	**0.52**	0.36%	0.045	4.0	3.80	10.75
LC-605	0.73	0.63	0.71	**0.62**	0.48	0.58	2.25%	0.120	6.0		
S-9	0.74	2.13	0.67	1.30	0.45	0.45	0.21%	0.045	7.5		
S-6	0.84	1.12	0.69	0.71	0.67	0.67	0.21%	0.045	11.0		
Sh-6	0.90	1.04	0.83	0.86	0.79	0.79	0.45%	0.045	18.0		
Lu-2	0.91	0.84	0.93	**0.85**	0.79	0.79	0.60%	0.045	8.0	3.00	8.00
W-6	0.99	**0.97**	1.04	**0.98**	0.95	0.95	0.81%	0.060	4.5	8.25	19.00
Sh-11	1.00	1.56	1.02	1.64	0.66	0.70	0.45%	0.045	28.0		
103-104-47	1.03	1.20	1.00	1.13	0.88	0.88	0.47%	0.040	6.5	7.60	14.00
W80-14	1.05	1.01	1.04	**1.00**	0.91	0.91	0.35%	0.045	5.0	3.25	6.75
T13-1	1.05	1.09	1.04	1.07	0.99	0.99	0.54%	0.045	6.5	2.75	6.50
W-10ab	1.05	1.07	1.04	1.05	0.90	**0.88**	0.30%	0.035	6.0	7.75	13.50
WM-8	1.08	1.43	1.13	1.57	0.85	0.86	2.06%	0.050	4.0	7.25	16.50
Sh-13	1.13	1.62	1.01	1.21	0.85	0.85	0.45%	0.045	31.0		
WK-9	1.14	1.24	1.19	1.32	0.97	0.97	0.41%	0.040	6.0		
Sh-4	1.14	1.23	1.10	1.15	1.05	1.06	0.45%	0.045	11.0		
103-104-53	1.17	1.20	1.14	1.14	1.12	1.12	0.28%	0.040	3.0	7.90	14.50
WK-7	1.19	1.17	1.22	**1.21**	0.87	**0.86**	0.24%	0.035	18.0		
W-8	1.21	1.06	1.31	**1.12**	0.92	0.92	1.58%	0.055	7.0	5.75	11.50
S-1	1.23	-12.23	0.77	1.14	0.58	0.58	0.21%	0.045	10.3		
T13-3	1.24	1.18	1.16	1.16	1.11	1.11	0.40%	0.035	5.0	6.50	12.00
Sh-15	1.28	1.48	1.70	3.52	1.12	1.13	0.45%	0.045	7.0		
103-104-39	1.28	1.28	1.28	1.28	1.26	1.26	0.38%	0.050	5.0	6.50	12.50
W-10cde	1.43	2.37	1.37	2.03	0.83	**0.75**	0.30%	0.035	6.0	7.75	13.50
S-4	1.43	6.25	1.27	2.49	0.88	**0.84**	0.21%	0.045	6.0		
103-104-8cd	1.54	1.42	1.51	**1.39**	1.21	1.21	0.43%	0.045	3.0	6.50	14.00
W-4	1.67	1.88	1.72	1.97	1.25	1.25	1.68%	0.050	9.5	7.00	18.00
Sh-14	1.68	1.91	1.52	1.53	1.48	1.48	0.45%	0.045	11.0		
W-3	1.75	1.54	1.16	1.16	1.11	1.11	2.20%	0.050	5.0	7.50	20.50
* WK-2	1.91	2.04	1.84	1.93	1.55	1.55	0.24%	0.035	50.0		
* WK-4	2.08	2.32	1.89	1.99	1.68	1.68	0.24%	0.035	27.5	7.75	15.50
WK-1	2.19	**2.00**	2.04	**1.90**	1.80	1.80	0.24%	0.035	37.5		
WK-8	2.24	5.20	2.39	7.15	1.56	1.81	0.41%	0.035	18.0		
Wm-27	2.65	2.70	2.76	2.92	2.18	**2.11**	0.86%	0.050	9.5		
* WK-6	2.84	**2.49**	2.54	**2.22**	1.88	**1.85**	0.24%	0.035	30.0		
* WK-5	3.01	**2.84**	2.91	**2.78**	2.61	2.61	0.24%	0.035	23.5		
Wm-13	3.35	3.42	3.32	3.38	2.57	2.57	0.96%	0.050	8.0	10.20	24.00
Average								0.046			
						Range n		0.035	0.120		

* these structures make use of remnants from old debris catcher structures.

Multiple-LWD Structures

Structures	Inter-assessment High Flow Depth m	Inter-assessment High Flow Computed Flow cms	Inter-assessment High Flow Velocity m/s	Bankfull Conditions Depth m	Bankfull Conditions Computed Flow cms	Bankfull Conditions Velocity m/s	Bankfull Conditions Tau*	Bankfull Conditions S.Power N/m/s	Design Flow Conditions Depth m	Design Flow Conditions Computed Flow cms	Design Flow Conditions Velocity m/s
S-11	0.98	8	0.9	1.36	15	1.1		31	2.18	35	1.4
103-104 DF 2											
103-104 DF 3											
S-7	1.00	9	1.0	1.24	15	1.2		30	1.79	35	1.5
103-104 DF 1											
S-12	1.12	8	0.7	1.58	15	0.8		27	2.53	35	1.1
103-104 DF 4											
S-2	1.12	11	1.0	1.32	15	1.1	0.067	29	2.25	35	1.4
103-104-8efg	1.47	29	1.9	1.51	31	1.9	0.061	124	2.52	107	2.7
Wolf Triangular											
103-104-8ab	1.47	29	1.9	1.51	31	1.9	0.061	124	2.52	107	2.7
LC-604	0.85	6	1.5	0.90	7	1.5	0.153	303	1.39	18	2.0
Lu-1	1.43	40	2.0	1.30	34	1.9	1.582	143	3.03	151	3.0
103-104-33	1.14	27	2.1	1.22	31	2.2	0.093	170	2.05	107	3.1
W80-8	2.01	49	2.0	2.06	52	2.1	0.119	150	3.50	183	2.9
LC-605	0.98	228	1.0	0.98	228	1.1	0.133	228	1.67	228	1.4
S-9	1.19	11	1.0	1.41	15	1.1		32	2.25	35	1.4
S-6	0.87	9	0.9	1.14	15	1.0		24	1.83	35	1.3
Sh-6	1.02	33	1.5	1.21	45	1.7	0.439	88	1.86	102	2.2
Lu-2	1.33	36	2.0	1.30	34	2.0	1.573	152	2.61	151	3.1
W-6	1.42	31	1.9	1.33	27	1.8	0.079	193	2.29	96	2.6
Sh-11	0.82	33	1.3	0.97	45	1.4	0.265	62	1.53	102	2.0
103-104-47	1.39	27	1.9	1.50	31	2.0	0.056	139	2.76	107	2.9
W80-14	1.96	49	2.0	2.00	52	2.0	0.131	137	3.44	183	2.8
T13-1	1.56	39	2.1	1.59	40	2.1	0.189	176	2.83	140	3.0
W-10ab	1.45	25	1.8	1.51	27	1.8	0.035	81	2.79	96	2.6
WM-8	1.74	59	3.7	1.65	53	3.6	0.284	1196	2.87	183	5.1
Sh-13	0.80	33	1.3	0.95	45	1.4	0.347	59	1.53	102	1.9
WK-9	1.80	137	3.0	1.68	116	2.8	0.054	190	2.43	293	3.7
Sh-4	1.06	32	1.6	1.25	45	1.8	0.453	101	1.81	102	2.4
103-104-53	1.53	26	1.7	1.65	31	1.8	0.035	82	2.75	107	2.5
WK-7	2.08	121	2.2	2.03	116	2.2	0.038	105	3.22	293	3.0
W-8	1.34	32	2.5	1.23	27	2.3	0.205	447	2.38	96	3.4
S-1	0.92	9	0.9	1.20	15	1.0		26	1.94	35	1.4
T13-3	1.23	24	2.0	1.55	40	2.3	0.058	140	2.68	140	3.3
Sh-15	0.85	33	1.8	0.96	45	2.0	0.350	83	1.34	102	2.5
103-104-39	1.58	29	1.6	1.64	31	1.6	0.058	100	2.82	107	2.3
W-10cde	1.45	25	1.8	1.51	27	1.8	0.035	81	2.79	96	2.6
S-4	1.14	12	1.1	1.27	15	1.1		30	1.90	35	1.5
103-104-8cd	1.47	29	1.9	1.51	31	1.9	0.061	124	2.52	107	2.7
W-4	1.04	28	2.4	1.01	27	2.4	0.146	395	2.04	96	3.6
Sh-14	0.88	36	1.7	0.97	45	1.8	0.354	77	1.37	102	2.3
W-3	1.14	29	3.0	1.11	27	2.9	0.197	700	2.03	96	4.3
WK-2	1.24	103	1.6	1.33	116	1.7	0.025	52	2.29	293	2.3
WK-4	1.63	96	1.9	1.82	116	2.0	0.034	85	3.04	293	2.7
WK-1	1.37	96	1.7	1.52	116	1.8	0.029	65	2.56	293	2.5
WK-8	2.08	121	2.2	1.37	116	2.6	0.038	105	2.04	293	3.5
Wm-27	1.97	68	2.6	1.71	52	2.4	0.119	339	3.30	183	3.4
WK-6	1.49	93	1.8	1.68	116	1.9	0.031	76	2.77	293	2.7
WK-5	1.75	102	2.0	1.87	116	2.1	0.035	91	3.04	293	2.8
Wm-13	1.87	51	2.5	1.89	52	2.5	0.108	451	3.70	183	3.7
Average			1.8			1.9					2.6

Multiple-LWD Structures

Structures	Max. movement (m) LWD	Boulders	Criterion 0.5	Comments	Still Functional
S-11	0.53		moved	not anchored to boulder (crew ran out of time)	y
103-104 DF 2	5	5	moved		y
103-104 DF 3	5	5	moved		y/partial
S-7	0.72		moved	no significant change, post elevations approximate	y
103-104 DF 1	5	5	moved		y
S-12	0.1			**not anchored to boulder (crew ran out of time), method of moments at H/W =4+**	y
103-104 DF 4	5	5	moved		y
S-2	0.4			some logs are loosely cabled, good triangulation	y
103-104-8efg	1.2	N/A	moved	E shifted toward bank tied back, G & boulder are gone, E & F unballasted but remained	y/partial
Wolf Triangular	5	5	moved	not cabled together instream	y/partial
103-104-8ab	0.7	0.4	moved	Slight movement A tied back to alder	y
LC-604	4.2	2.65	moved	rebar attachement let go, some scour of left bank near end of sill and d/s	y/partial
Lu-1	0.4	0.3		some scouring visible, significant transport as u/s bar as aggraded	y
103-104-33	0.7	0.9	moved	lost LWD E cable pulled out (probably save structure), some scour apparent across from struc	y
W80-8	1.6	1.2	moved	big scour hole d/s, some scour across structure, good cabling bank and stream ends	y
LC-605	0.6		moved	adjustments made for overhanging LWD	y
S-9	0.6		moved	loose cabled	y
S-6	0.1			**some minor aggradation behind boulder minor infilling, moments > 1.0**	y
Sh-6	0.4			D in sand at bank end, structures sheltered from main flow, some aggradation at tail of pool, c:	y
Lu-2	0.15	0.1		some significant aggradation on bar across and scouring under LWD, see graph	y
W-6	1.2	1.05	moved	shifted d/s and up in elevation, boulder #4 has split, members not tied back on bank.	y/partial
Sh-11	0.35				y
103-104-47	0.1	0.25		good cabling and triangulation	y
W80-14	3.35	3.45	moved	no triangulation, acts more like a compound single	n
T13-1	0.15	0.1		excavated pool partially filled in	y
W-10ab	0.45	0.2		slight adjustment, no triangulation provided	y
WM-8	0.3	0.2		good triangulation and large alders as bank anchors	y
Sh-13	0.5				y
WK-9	0.15			some debris accumulated near bank	y
Sh-4	1.15		moved	very dense shrubs impinging measurements, cables are quite loose	y
103-104-53	3.1	2.7	moved	V closed in since not cabled on banks	y/partial
WK-7	0.6		moved	could not access stream end, no debris caught	y
W-8	2.45	1	moved	some movement apparent, A is overhanging & is tied to alder	y
S-1	0.25			some minor scour, appear to be scour near left bank	y
T13-3	0.3	0.4		slight adjustment, A tied back, low bar formation across from structure influenced by upstream	y
Sh-15	0.5				y
103-104-39	0.25	0.2		tied back to alder u/s side	y
W-10cde	2.6	0.9	moved	adjustment due to drag on RW and lack of triangulation	y
S-4	0.3			scour and fill compensated, scour hole shifted to the left	y
103-104-8cd	0.2	0.4		Slight movement C tied back to alder	y
W-4	0.3	1.1		some scour apparent	y
Sh-14	0.25			0.95 D u/s likely due to measurements inacuracies (jagged end & thick shrubs) - disregard	y
W-3	0.1	0.05		lots of simulidae on rocks in riffles	y
WK-2	0.15			no access to instream end for follow-up assessment, no apparent movement.	y
WK-4	0.1			bank end picked up slack by 0.8 m, instream intact, no debris caught	y
WK-1	1.8		moved	lots of debris on u/s face	y
WK-8	1		moved	could not access stream end, plenty of debris caught, eroded d/s bank extensively	y
Wm-27	0.2	0.2		some additional scour in pool between #27 and #26	y
WK-6	0.3			no debris caught	y
WK-5	0			not accessible, not assessedin follow-up assess., no appearance of movement, functioning	y
Wm-13	0.4	0.2			y
Average					

Multiple-LWD Structures

Structures	Mass Provided	Design Mass required (kg) 2y No BF	2y BF	50y No BF	50y BF	Mlb BF	Mlb/M50Y	2y/50y	Difference 50y BF
S-11	89	1,290	2,574	1,301	2,591	41	2%	0.7%	2,502
103-104 DF 2									
103-104 DF 3									
S-7	887	2,854	4,555	2,874	4,582	69	2%	0.6%	3,695
103-104 DF 1									
S-12	981	2,802	5,598	2,813	5,615	40	1%	0.3%	4,634
103-104 DF 4									
S-2	506	1,403	1,929	1,415	1,943	33	2%	0.7%	1,437
103-104-8efg	267	661	661	691	691	60	9%	4.3%	424
Wolf Triangular									
103-104-8ab	1,020	2,658	2,658	2,734	2,734	154	6%	2.8%	1,713
LC-604	655	1,309	2,254	1,330	2,284	74	3%	1.3%	1,629
Lu-1	5,229	4,308	6,223	4,469	6,429	334	5%	3.2%	1,199
103-104-33	2,272	2,698	5,356	2,800	5,519	316	6%	2.9%	3,246
W80-8	2,990	6,299	7,251	6,453	7,421	341	5%	2.3%	4,431
LC-605	235	1,264	552	1,275	558	14	3%	1.1%	323
S-9	1,263	1,770	3,532	1,783	3,553	52	1%	0.6%	2,290
S-6	1,744	1,631	3,256	1,643	3,274	44	1%	0.6%	1,531
Sh-6	1,321	1,046	2,079	1,069	2,115	85	4%	1.7%	795
Lu-2	846	1,294	1,294	1,368	1,368	124	9%	5.4%	522
W-6	3,103	4,013	4,013	4,111	4,111	189	5%	2.4%	1,008
Sh-11	5,180	5,334	10,639	5,391	10,729	201	2%	0.8%	5,549
103-104-47	4,449	3,242	6,320	3,345	6,480	310	5%	2.5%	2,032
W80-14	3,430	3,069	4,640	3,162	4,762	241	5%	2.6%	1,332
T13-1	3,490	2,421	4,311	2,512	4,446	259	6%	3.0%	956
W-10ab	1,319	1,188	1,858	1,232	1,918	114	6%	3.1%	599
WM-8	7,893	5,779	11,380	6,207	12,059	1,363	11%	5.6%	4,166
Sh-13	4,763	3,475	6,930	3,517	6,996	145	2%	0.9%	2,233
WK-9	8,668	7,108	10,904	7,329	11,200	713	6%	2.6%	2,531
Sh-4	3,765	2,249	4,472	2,291	4,540	166	4%	1.5%	775
103-104-53	5,376	2,961	5,891	3,031	6,003	228	4%	1.9%	627
WK-7	4,398	5,550	6,289	5,682	6,433	320	5%	2.2%	2,036
W-8	2,108	2,760	2,760	2,891	2,891	247	9%	4.6%	783
S-1	1,803	1,942	3,876	1,955	3,898	51	1%	0.6%	2,095
T13-3	1,536	1,654	1,654	1,738	1,738	163	9%	4.8%	202
Sh-15	5,052	2,838	5,642	2,892	5,728	217	4%	1.5%	676
103-104-39	6,123	2,999	5,973	3,061	6,071	195	3%	1.6%	
W-10cde	729	1,243	1,381	1,288	1,429	94	7%	3.4%	701
S-4	1,097	2,246	2,468	2,262	2,485	42	2%	0.7%	1,389
103-104-8cd	1,710	1,704	1,704	1,760	1,760	114	6%	3.2%	50
W-4	2,833	1,785	2,675	1,899	2,825	267	9%	5.3%	
Sh-14	6,532	2,520	5,013	2,565	5,084	175	3%	1.4%	
W-3	3,628	3,500	3,500	3,732	3,732	440	12%	6.2%	104
WK-2	6,215	3,214	4,916	3,278	5,002	172	3%	1.7%	
WK-4	10,205	4,781	7,352	4,890	7,498	308	4%	1.9%	
WK-1	7,367	4,354	4,949	4,444	5,047	201	4%	1.9%	
WK-8	7,468	5,373	7,841	5,559	8,081	530	7%	3.0%	613
Wm-27	4,821	2,643	2,709	2,773	2,841	247	9%	4.6%	
WK-6	8,041	5,247	5,247	5,358	5,358	232	4%	2.1%	
WK-5	5,764	1,908	2,564	1,969	2,638	160	6%	2.8%	
Wm-13	6,615	2,548	2,844	2,685	2,991	285	10%	4.9%	
Average		2,933	4,402	3,018	4,510	avg	5.0%	2.5%	1,131
						std	2.8%	1.6%	

Multiple-LWD Structures

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APPENDIX D: DESIGN METHODOLOGIES AND DESIGN CHARTS SINGLE-LWD AND MULTIPLE-LWD STRUCTURES

The following appendix presents design charts for single-LWD and multiple-LWD structures based on the design methodology presented by Millar (1997). Furthermore, design examples are provided as a guide to users.

The design charts provided below include revisions addressing recommendations presented in this report, and are intended to supersede those presented by Millar (1997). The revisions include:

- The use of a Ballast Factor (*BF*) to account for LWD relying solely on boulders for ballasting.
- Revised factor of safety against buoyancy, $FS_B \geq 1.5$ (increased from $FS_B \geq 1.25$; Millar, 1997).
- Revised $C_{DB} = 0.2$ (increased from $C_{DB} = 0.1$; Millar, 1997).
- Use of $S_L = 0.5$ (reduced from $S_L = 0.6$; Millar, 1997).

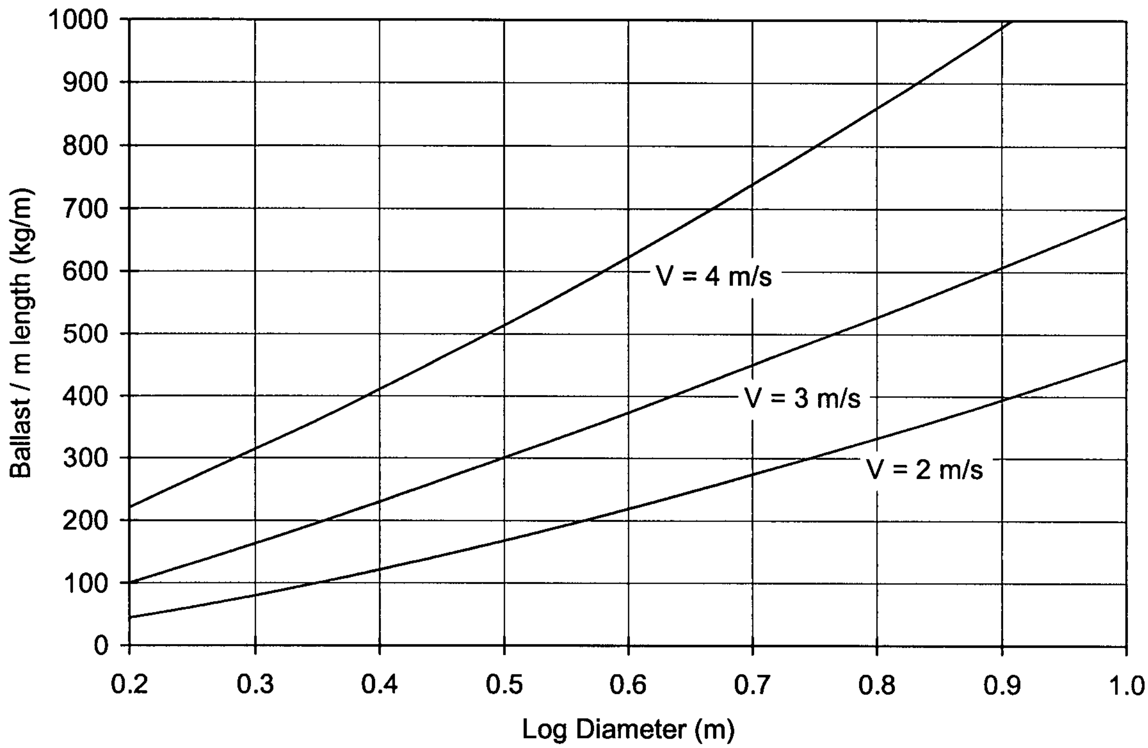


Figure 1: Design Chart for Single-LWD Structures based on Factor of Safety against Sliding (FS_S = 2.0, BF = 1, LWD 90 degrees to flow)

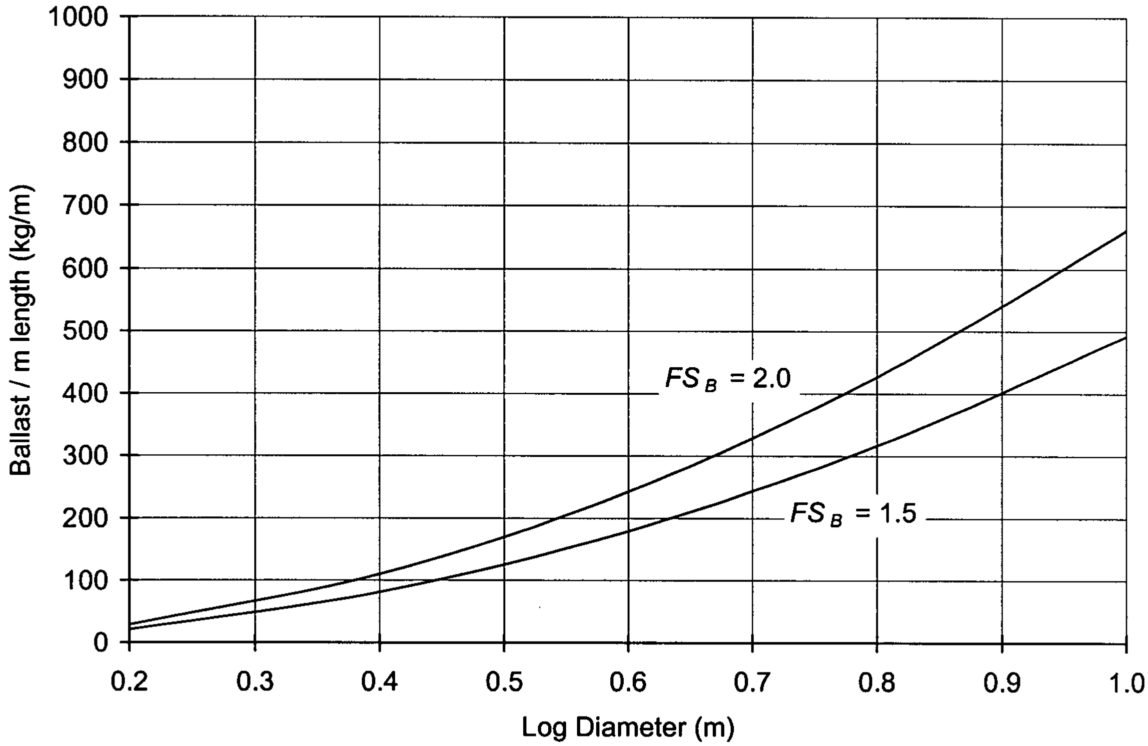


Figure 2: Design Chart for Single and Multiple-LWD Structures based on Factor of Safety against Buoyancy (*BF* = 1)

Sample Computations

Example 1:

A single-LWD is to be anchored in-stream with the use of boulders and cabled to a tree on the bank. Making use of the design charts determine the ballast mass required to withstand an average design velocity of 2.5 m/sec while providing a factor of safety against sliding (FS_S) of 2.0. The LWD characteristics are: D_L = 0.4 m and L = 9 m.

From Figure 1, the ballast mass per metre of LWD is about 190 kg. For design purposes it is assumed that the LWD is fully submerged.

Therefore the total ballast mass requirement: **9m x 190 kg/m = 1,700 kg** (or two boulders of 0.85m diameter)

If the LWD was ballasted using boulders both in-stream and on the bank (i.e. no streamside trees or stump), a $BF = 2$ must be used. This effectively doubles the mass requirement:

Therefore the total ballast mass requirement (BF = 2): **9m x 380 kg/m = 3,400 kg** (or four boulders of 0.85m diameter)

A quick check on the factor of safety against buoyancy reveals that about 120 kg/m are required to maintain a $FS_B = 2.0$. Therefore, the design based on a sliding failure governs (190kg/m).

Example 2:

Determine the ballast requirement for a multiple-LWD structure to provide a factor of safety against buoyancy (FS_B) of 2.0. The structure is as illustrated in Figure (3-3) and is composed of the following:

LWD	LWD Diam. (m)	LWD Length (m)	Ballast Factor	Mass/m (kg/m)	Total Mass (kg)
U/S main structural member	0.60	8.0	1	250	2,000
D/S main structural member	0.55	9.0	1	210	1,890
small LWD near bank (U/S)	0.30	5.0	1	75	375
small LWD near bank (D/S)	0.25	4.0	1	60	240
small LWD near bank (covered)	0.20	2.0	1	30	60
small in-stream LWD (U/S)	0.25	4.0	2	2x60	480
small in-stream LWD (D/S)	0.30	2.0	2	2x75	300
Total ballast mass required					**5,350**

Therefore the total ballast mass requirement for the multiple-LWD structure illustrated in Figure 3-3 is about 5,400 kg.

APPENDIX E: DESIGN METHODOLOGIES AND DESIGN CHARTS SINGLE-LWD STRUCTURES WITH INTACT ROOT WADS

1. Introduction

The following presents design methodologies along with graphical design methods to determine ballast requirement to stabilise a single LWD with root wad within a stream or river channel with the use of anchor boulders cabled to the LWD. As depicted by Figure 1, the LWD is oriented parallel to the flow with the root wad at the upstream end of the LWD. Also illustrated are the typical scour and fill patterns that develop with time around these structures.

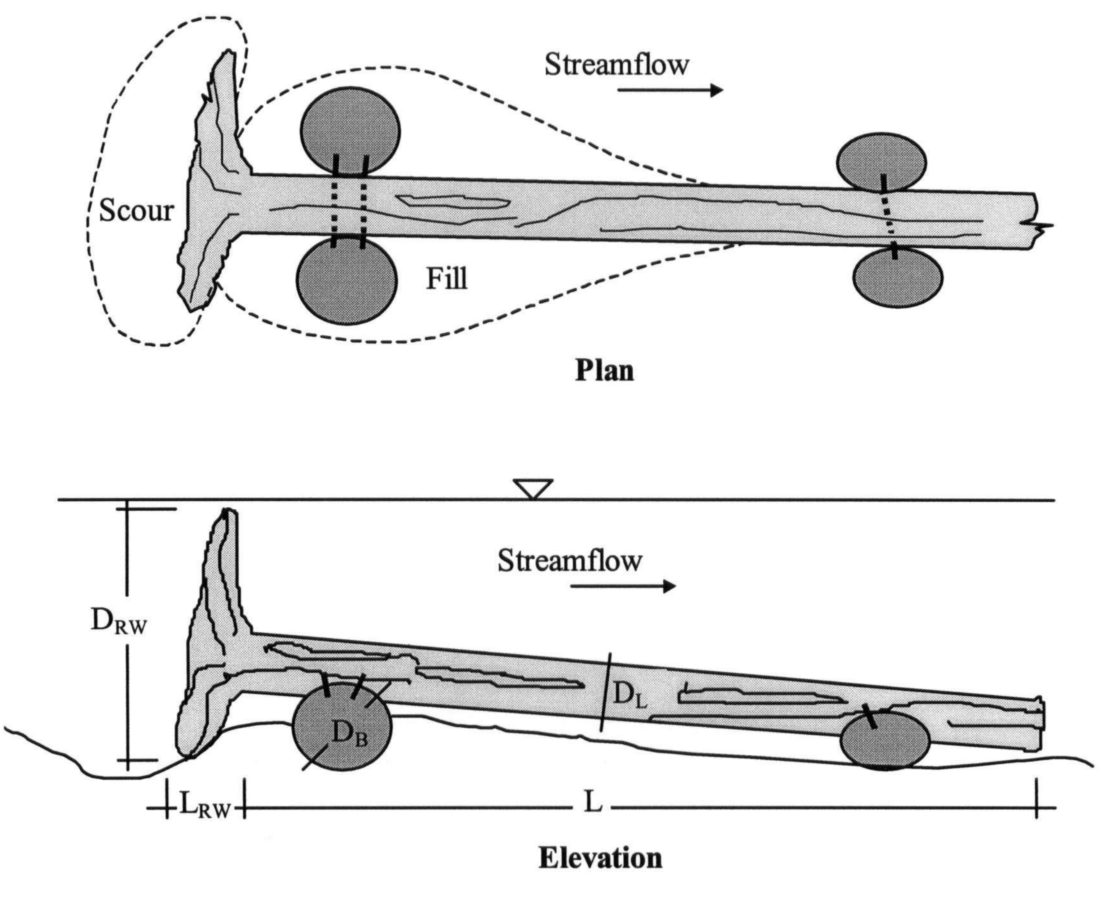


Figure 1: Conceptual drawing of LWD (with root wad) and ballast

2. Assumptions

It is important to highlight some of the main assumptions made in the process of establishing these design methodologies. This will allow the designer to know the applicability of the guidelines while being aware of their limitations.

In developing some of the equations and design curves, it is assumed that both the LWD and root wad are fully submerged under design flow conditions. This assumption is realistic since water generally submerges the structures under flood conditions. In cases where they are not fully submerged, this assumption simplifies the analysis (since it eliminates the need to know flood elevations) while providing conservative ballast requirements (in order to counter the full buoyancy of the structure).

Another assumption is that the anchor boulders are subject to average stream flow velocities. This is a conservative assumption since the boulders may be partially sheltered, by resting in the wake of the root wad, and therefore, subject to reduced velocities, drags and lift forces. We will also ignore the potential partial burial of the anchor boulders and partial embedment of the root wad in a scour hole induced by the presence of the structure. These conditions usually do not exist upon installation of the LWD and this phenomenon will only provide added stability to the LWD as the structure "settles" into place (Refer to Attachment 3 for an estimate of the added stability for embedded root wads.).

Furthermore, any additional debris/ice drag and/or impact loads are not accounted for in the analysis. If such conditions are anticipated, the designer should increase the factor of safety proportionately.

3. Design Methodology

The following section will present the development of the design equations used to determine ballast mass requirements for LWD with root wad structures. Furthermore, these equations will be utilised to develop graphical methods for design.

3.1 Design Methodology for V > 2.0 m/sec

When average stream flows are in excess of 2.0 m/sec it can be safely assumed that stability will be governed by sliding and therefore, Equation (8) will be used as the basis for determining the ballast requirements. Substituting Equation (3-6) into (3-16) and rewriting to isolate *W'* (immersed weight of ballast), yields:

$$W' = \frac{FS_S\,(F_{DRW} + F_{DB} + F_{FL})}{\tan\phi} + F_{BL} + F_{LB} \tag{1}$$

Neglecting F_{FL}, which is typically an order of magnitude smaller than $F_{DRW} + F_{DB}$, and multiplying by *s/[g(1-s)]* to convert the weight to a mass, yields:

$$M_S = \frac{FS_S F_{DRW}}{\tan\phi}\frac{s}{g(s-1)} + F_{BL}\frac{s}{g(s-1)} + \frac{FS_S F_{DB}}{\tan\phi}\frac{s}{g(s-1)} + F_{LB}\frac{s}{g(s-1)} \tag{2}$$

------Term 1 ------ ----Term 2--- -----Term 3------ ---Term 4---

Where M_S is the ballast mass required to provide a factor of safety of FS_S against sliding of the LWD.

Equation (2) represents the mass requirements for each component of the LWD and boulder structure. The Term 1 is the ballast requirement to counter the drag forces exerted on the root wad, Term 2 is the ballast requirement to counter buoyancy and Term 3 and 4 represent the ballast requirements to counter the drag and lift forces exerted on the anchor boulders.

The Terms 1 and 2 of Equation (2) can be solved explicitly as they are functions of the LWD characteristics and stream flow velocity. Substituting Equation (3-14) into Term 1 with $\beta = 90°$, $s = 2.65$, $\phi = 40°$ and $C_{DRW} = 1.2$, yields:

$$\text{Term 1: } M_{DRW} = 92 FS_S D_{RW}{}^2 V^2 \tag{3}$$

Whereas, substituting Equation (3-13) into Term 2 of Equation (2) yields:

$$\text{Term 2: } M_{BL} = 1260(1 - S_L)(D_L{}^2 L + 0.27 D_{RW}{}^2 L_{RW}) \tag{4}$$

Terms 3 and 4 however are functions of the anchor boulder size (which is being sought). In order to determine the mass required to counter boulder lift and drag, it is possible to estimate the boulder size(s) required by assuming that Terms 1 and 2 ($M_{DRW} + M_{BL}$) make up most of the boulder mass required. This assumption is valid since $M_{DRW} + M_{BL}$ generally make up more than 85% of the required ballast mass. The boulder diameter may then be estimated from:

$$D_{Best} = \left[\frac{(M_{DRW} + M_{BL})6}{n\pi \rho s} \right]^{1/3} \tag{5}$$

Where D_{Best} is the estimated boulder diameter (m) and n is the number of boulders of equal size used for ballasting the LWD. Then, taking Terms 3 and 4 of Equation (2) and substituting in Equations (3-3), (3-4) and (3-15) (as an estimate of D_B), and assuming that 4 boulders of equal size will make up the ballast (note that this assumption does not have a significant impact on the total boulder mass required) we obtain:

$$\boxed{\text{Terms 3 and 4: } M_{DB+LB} = (0.20FS_S + 0.14)(M_{DRW} + M_{BL})^{2/3} V^2} \tag{6}$$

One may wish to do a second iteration by determining the boulder size based on the total mass requirement (all three terms) and determining a new value for M_{DB+LB}. This refinement is not significant (generally less than 2% of total mass requirement) considering the number of assumptions made in developing the methodology.

Therefore, Equations (3), (4) and (6) will yield an adequate estimate of the ballast requirement to satisfy Equation (8).

$$\boxed{M_S = M_{DRW} + M_{BL} + M_{DB+LB}} \tag{7}$$

3.2 Design Methodology for V < 2.0 m/sec

In the cases where the average stream flow velocity is less than 2 m/sec, Equation (3-8) may govern the ballast requirement and therefore a verification of the structural stability against uplift must be undertaken. This is done by determining the mass requirement to provide a desired factor of safety against uplift and comparing it to the mass requirement against sliding (as determined in Section 4.1).

Rewriting Equation (3-8) in terms of mass and isolating the boulder mass required, yields:

$$M_B = FS_B F_{BL} \frac{s}{g(s-1)} + FS_B F_{LB} \frac{s}{g(s-1)} \qquad (10)$$

-----Term 1------ ------Term 2------

Where Term 1 is the mass of anchor boulders to counter buoyancy and Term 2 is the ballast mass required to counter the lift forces on the anchor boulders. Term 1 is equal to Term 2 of Equation (2) multiplied by FS_B. It can be written as:

$$\boxed{\text{Term 1: } M_{BLWD} = 1260 FS_B (1 - S_L)(D_L{}^2 L + 0.27 D_{RW}{}^2 L_{RW}) \qquad (11)}$$

Term 2 of Equation (10) is a function of the anchor boulder size (which is being sought). To determine the mass required to counter boulder lift forces, it is possible to estimate the boulder size(s) required by assuming that the first term (M_{BLWD}) makes up most of the boulder mass required. An estimate of the boulder diameter can be obtained from Equation (5) (while replacing M_{DRW}+M_{BL} with M_{BLWD}). Equations (3-3), (3-4) and (5) (as an estimate of D_B), may be substituted in Equation (10) obtain:

$$\boxed{\text{Term 2: } M_{LB} = 0.14 FS_B M_{BLWD}{}^{2/3} V^2 \qquad (12)}$$

Therefore, Equations (11) and (12) will yield an estimate of the ballast requirement to satisfy Equation (3-8), M_B.

$$M_B = M_{BLWD} + M_{LB} \tag{13}$$

Hence, the mass requirements from Equations (7) and (13) may be compared to determine which mode of failure governs, sliding or uplift.

3.3 Location of ballast

An analysis of the distribution of the buoyancy forces from the LWD to the anchor boulders, using the method of moments, indicates that the proportion of forces transferred to the upstream and downstream anchor boulders varies with log length. Assuming that the drag forces on the root wad are distributed to the upstream and downstream anchor boulders in the same ratio as the buoyancy forces, the distribution of mass among the anchor boulders may be determined.

Figure 3 illustrates the portion of the total ballast mass that is required at the upstream anchor. To generate this graph it was also assumed that:

1. The anchor boulders are located 1.5 m away from the root wad (at the upstream end) and 1.5 m from the downstream tip of the log and
2. The relationships determined from field survey data may be used to compute the root wad volume. These relationships are represented by:

$$D_{RW} = 1.90D_L + 0.60m \qquad L_{RW} = 0.80D_L + 0.35m \qquad (8), (9)$$

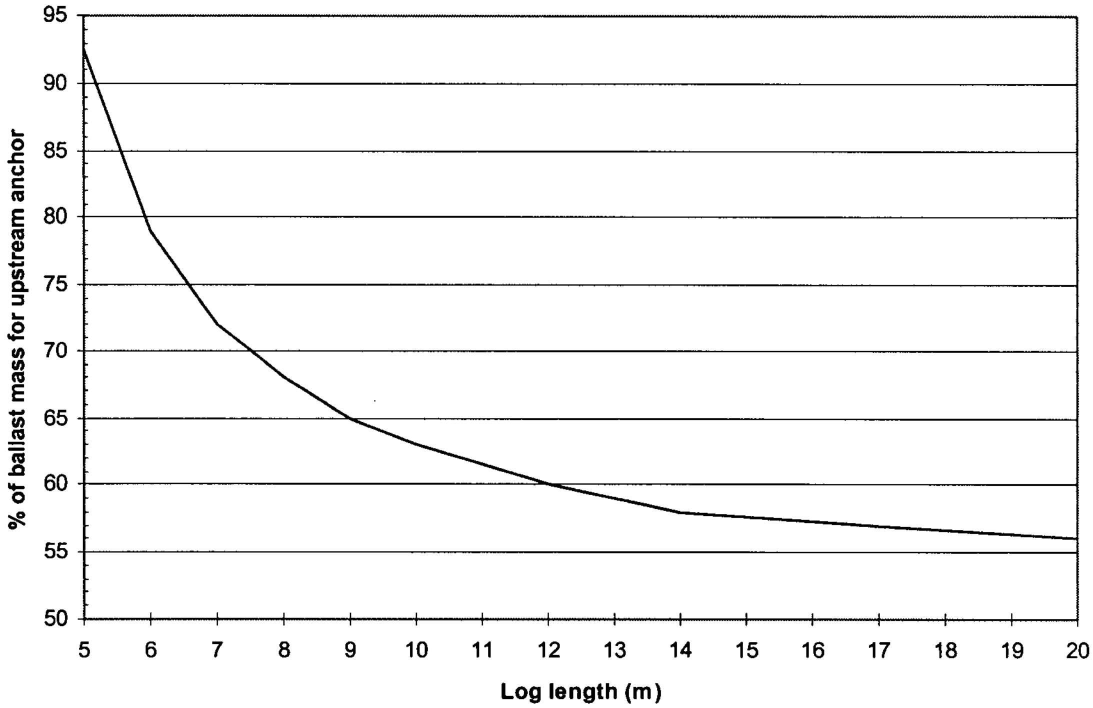


Figure 2: Distribution of ballast mass among anchor boulders

In the cases where only a stump is present (i.e. $L < 2$ m), the design methodologies presented herein may be applied as long as the LWD is aligned with the flow (root wad is normal to flow). It may be desirable however, to have the root wad facing downstream and providing ballast on each side of the stump along with some attached to the larger roots to provide some stability on the downstream side of the LWD. Figure 4 provides a concept that may be used to anchor a root wad and stump.

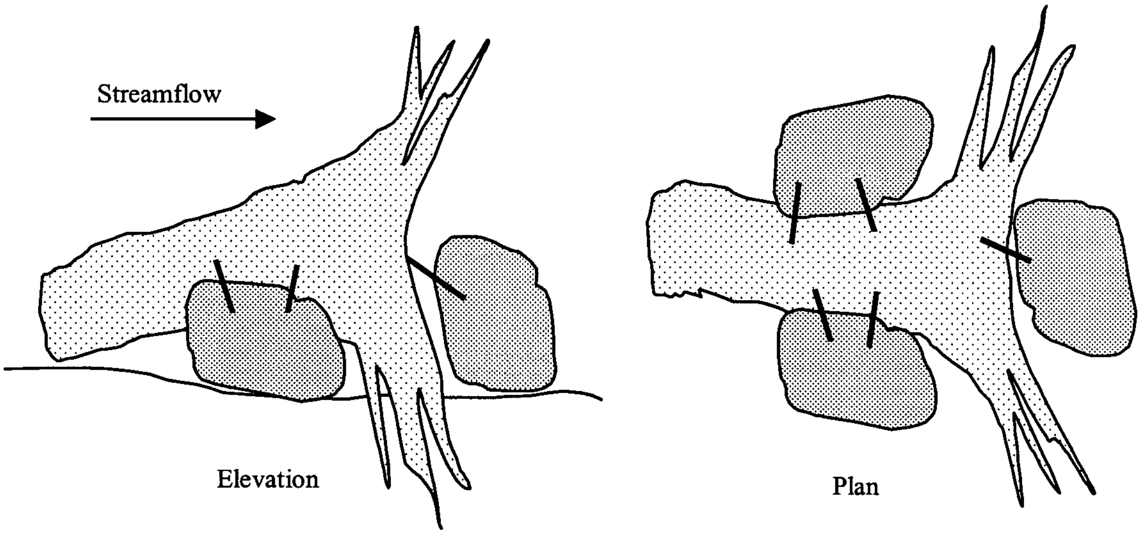


Figure 3: Conceptual drawing of ballasting of root wad and stump

3.4 Graphical method for V >2 m/sec

The methodology presented above can also be represented graphically. Based on the premise that a minimum factor of safety against sliding of 2.0 is acceptable for design purposes, design curves may be generated.

Equation (3) may be represented graphically by the design curves in Figure 5.

Equations (8) and (9) were substituted in Equation (4) in order to reduce the data requirements for the graphical solution. These relationships ((8) & (9)) are considered adequate for this purpose since the root wad volume is typically less than 20% of the total LWD volume. Note that the design curves in Figure 6 were derived for cedar and that the value of M_{BL} obtained from these curves must be corrected for the difference in density by multiplying M_{BL} by $(1-S_L)/(1-0.36)$. This does not apply if Equation (4) is used to determine M_{BL}.

Finally, Terms 3 and 4, Equation (3), may be represented graphically by the design curves in Figure 7.

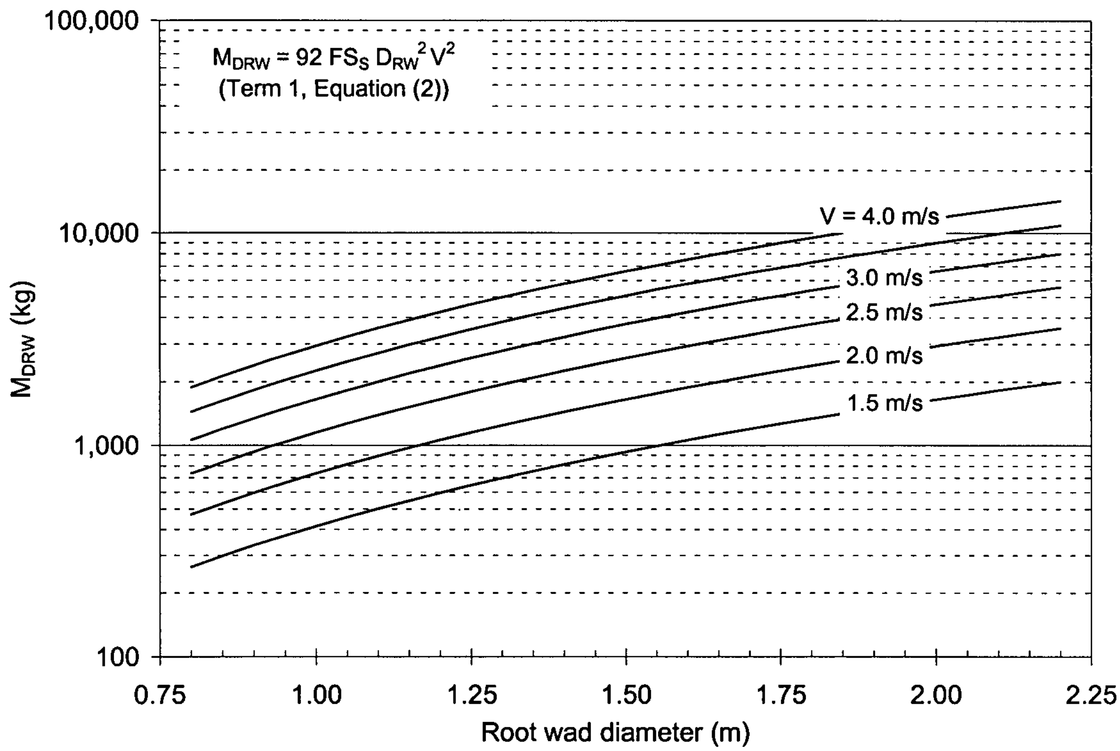


Figure 4: Mass required to counter root wad drag forces (FS_S = 2.0)

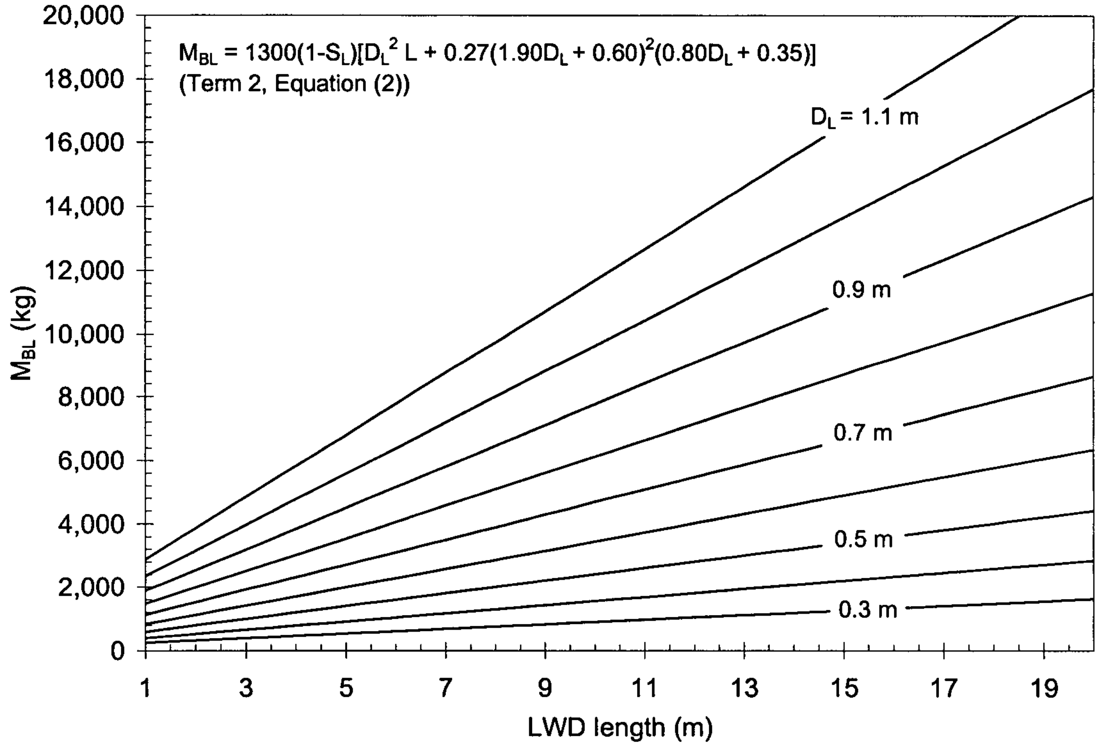


Figure 5: Mass required to counter LWD buoyancy (SL = 0.36)

Note that for species other than Cedar, the value of M_{BL} obtained from the design curves in Figure 6 must be corrected for the difference in density by multiplying M_{BL} by $(1-S_L)/(1-0.36)$. This does not apply if Equation (4) is used to determine M_{BL}.

Table 1: Correction to be applied to M_{BL} obtained from Figure 6

Species	$(1-S_L)/(1-0.36)$
Cedar	1.0
Spruce (Sitka, White and Englemann)	0.89
Hemlock, Pine (Jack and Lodgepole), Spruce (Black)	0.81
Pine (Ponderosa)	0.77
Fir (Douglas)	0.72

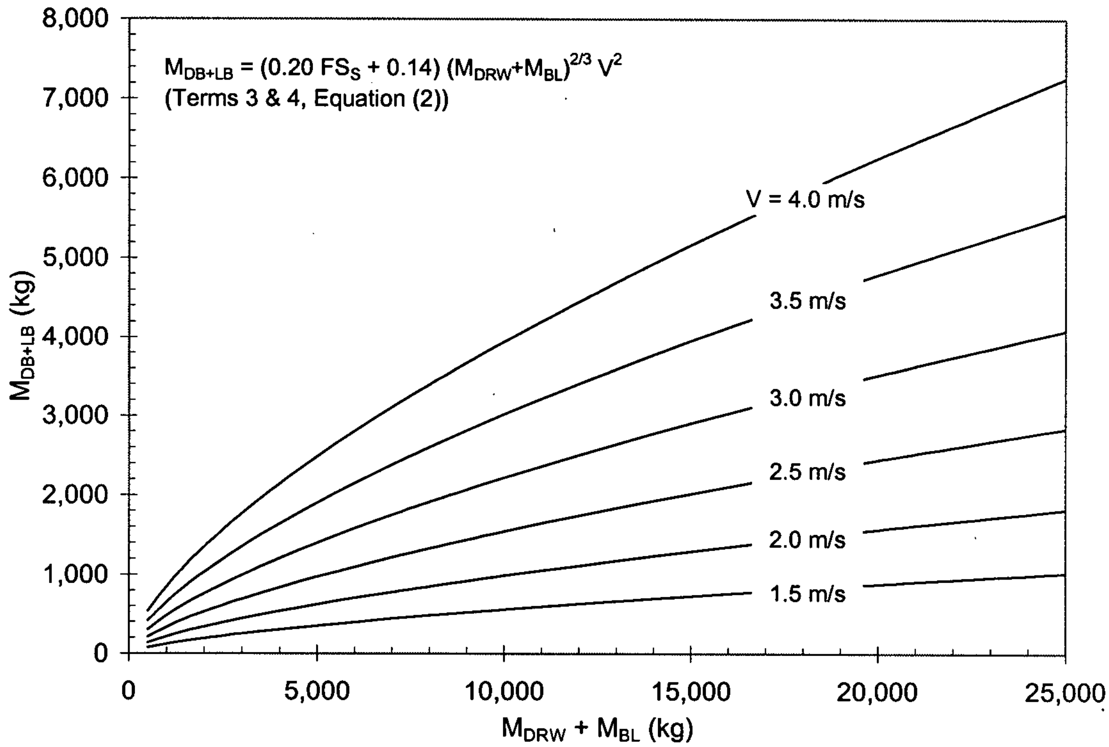


Figure 6 : Mass required to counter boulder drag and lift forces ($FS_S = 2.0$)

3.5 Graphical Method for $V < 2$ m/sec

The above design methodology also lends itself to a graphical approach. Based on the premise that a factor of safety against buoyant uplift of 1.5 is acceptable for design purposes, design curves may be generated.

As described previously, Term 1 can be simplified by using the relationships depicted by Equations (8) & (9). It may be presented in a single graph (see Figure 8).

Finally, Term 2 may be represented graphically by the design curves in Figure 9.

Sample computations to determine total ballast requirements and governing failure mechanisms for LWD with root wad are presented in Attachment 2.

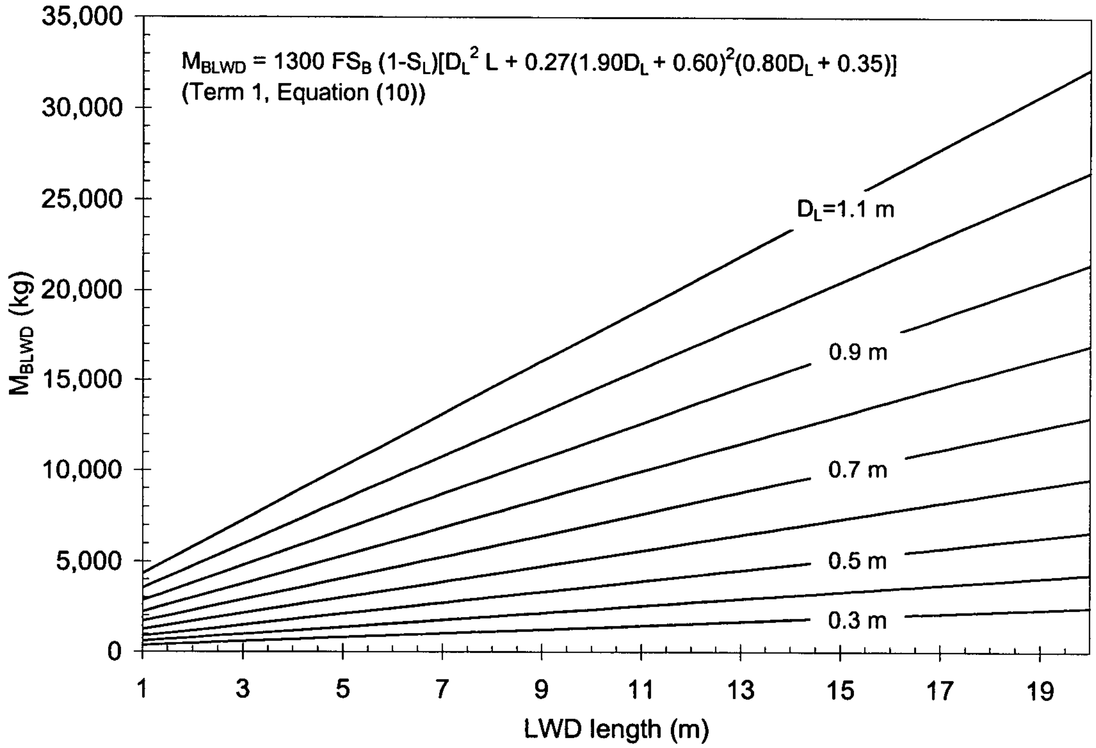


Figure 7: Mass required to counter LWD buoyancy (FS_B =1.5 & S_L= 0.36)

Note that for species other than Cedar, the value of M_{BLWD} obtained from Figure 8 must be corrected for the difference in density by multiplying M_{BLWD} by $(1\text{-}S_L)/(1\text{-}0.36)$. This does <u>not</u> apply if Equation (11) is used to determine M_{BLWD}.

Table 2: Correction to be applied to M_{BLWD} obtained from Figure 8

Species	$(1\text{-}S_L)/(1\text{-}0.36)$
Cedar	1.0
Spruce (Sitka, White and Englemann)	0.89
Hemlock, Pine (Jack and Lodgepole), Spruce (Black)	0.81
Pine (Ponderosa)	0.77
Fir (Douglas)	0.72

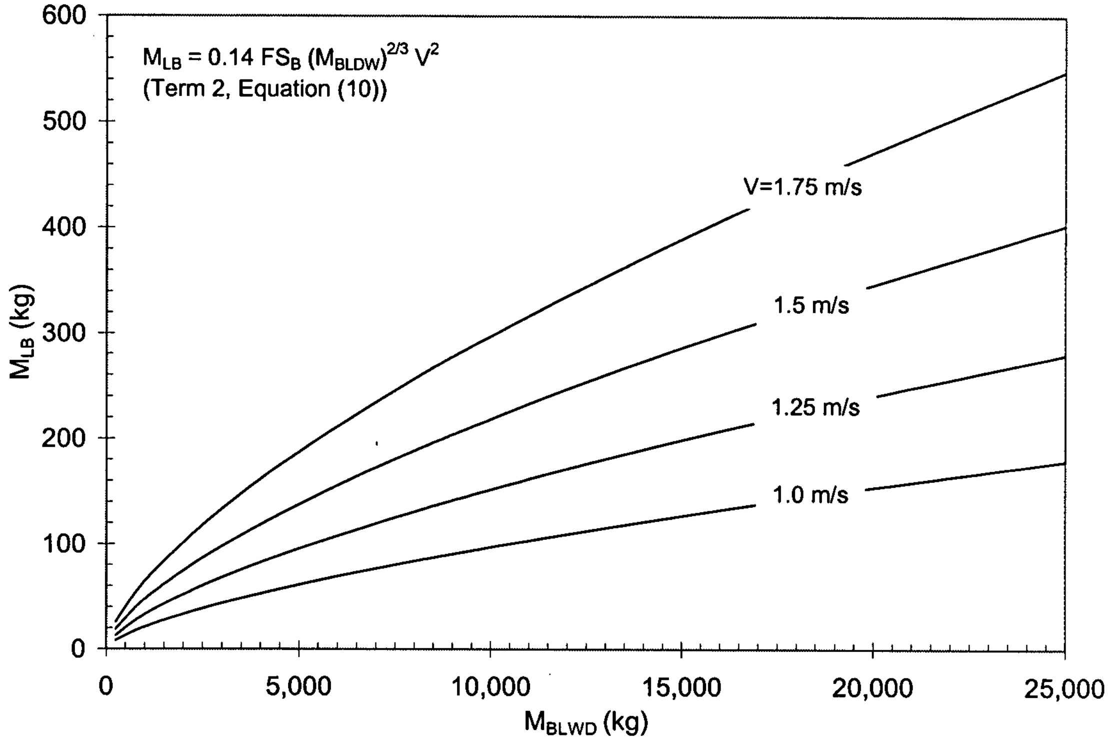


Figure 8: Mass required to counter boulder lift forces (*FS_B* =1.5)

Attachment 1: List of Symbols

Symbol	Units	Description
C_{DRW}	-	drag coefficient of root wad (1.2)
C_{DB}	-	drag coefficient of boulder (0.2)
C_{FL}	-	skin friction coefficient of log (0.004)
D_B	m	anchor boulder diameter
D_{Best}	m	estimated anchor boulder diameter
D_L	m	average diameter of log
D_{RW}	m	average root wad diameter
F_{BL}	N	buoyancy forces of LWD transferred to anchor boulders
F_{DRW}	N	drag forces acting on root wad transferred to anchor boulders
F_{DB}	N	drag forces acting on anchor boulders
F_{LB}	N	lift forces acting on anchor boulders
F_{FL}	N	frictional forces acting on log transferred to anchor boulders
F_F	N	frictional forces resisting sliding
FS_B	-	factor of safety against buoyant uplift
FS_S	-	factor of safety against sliding
g	m/sec^2	gravitational acceleration (9.806 m/sec^2)
L	m	length of LWD excluding root wad
L_{RW}	m	length of root wad
M_B	kg	ballast mass required to provide FS_B against buoyancy
M_{BL}	kg	ballast mass required to counter buoyancy of LWD
M_{BLWD}	kg	ballast mass required to counter buoyancy of LWD (w/ FS_B)
M_{DRW}	kg	ballast mass required to counter root wad drag forces
M_{DB+LB}	kg	ballast mass required to counter boulder drag and lift forces
M_{LB}	kg	ballast mass required to counter boulder lift forces
M_S	kg	ballast mass required to provide FS_S against sliding
S_L	-	specify gravity of LWD (air dried, range 0.36-0.54)
s or S_S	-	specific gravity of anchor boulders (2.65)
V	m/sec	average streamflow velocity
W'	N	immersed weight of anchor boulders
β	°	angle of root wad face w/r to flow (typically 90°)
ϕ	°	friction angle of anchor boulders on stream bed (40°)
ρ	kg/m^3	density of water (1,000 kg/m^3)

Attachment 2: Sample Computations

Example 1.0

A LWD with root wad intact is to be anchored with the use of boulders. Making use of the graphical method described herein to determine the ballast mass required to withstand an average stream flow velocity of 2.5 m/sec while providing a factor of safety against sliding (FS_S) of 2.0

LWD characteristics: Spruce LWD, $D_L = 0.4$ m, $L = 9$ m, $D_{RW} = 1.3$ m

1. The first step is to determine the ballast mass required to counter the root wad drag. From Figure 5, $M_{DRW} = 1{,}900$ kg.

2. The second step is to determine the ballast mass required to counter the LWD buoyancy. From Figure 6, $M_{BL} = 1{,}500$ kg x 0.89 (correction for difference in density between cedar and spruce) = 1,300 kg.

3. The third step is to determine the ballast mass required to counter the drag and lift forces acting on the anchor boulders. From Figure 7, with $M_{DRW} + M_{BL} = 3{,}200$ kg, $M_{DB+LB} = 700$ kg.

Hence, the total mass required to anchor the LWD parallel to the flow is:

$$\boldsymbol{M_S = M_{DRW} + M_{BL} + M_{DB+LB} = 3{,}900\ \textbf{kg}}$$

From Figure 3, 65% of M_S (2,500 kg) must be provided at the upstream end of the LWD and 35% (1,400 kg) must be provided at the downstream end.

Therefore with the help of the boulder selection chart, Figure 10, 2 boulders of 1.0 m average diameter (or equivalent mass) are needed at the upstream end of the LWD (1.5 m away from the root wad) while 2 boulders of 0.80 m average diameter (or equivalent mass) will be required at the downstream end of the LWD (1.5 m from the tip).

Example 2.0

Taking a look at the same root wad as in Example 1.0 but subject to an average stream flow velocity of 1.5 m/sec while providing a factor of safety against sliding $FS_S = 2.0$ and against uplift $FS_B = 1.5$.

Determining the mass requirement to counter sliding:

1. The first step is to determine the ballast mass required to counter the root wad drag. From Figure 5, $M_{DRW} = 700$ kg.
2. The second step is to determine the ballast mass required to counter the LWD buoyancy. From Figure 6, $M_{BL} = 1{,}500$ kg x 0.89 (correction factor for difference in density between cedar and spruce) = 1,300 kg.
3. The third step is to determine the ballast mass required to counter the drag and lift forces acting on the anchor boulders. From Figure 7, with $M_{DRW} + M_{BL} = 2{,}000$ kg, we obtain $M_{DB+LB} = 200$ kg.

Hence, the total mass required to prevent sliding of the LWD is:

$$\mathbf{M_S = M_{DRW} + M_{BL} + M_{DB+LB} = 2{,}200\ kg}$$

Determining the mass requirement to counter buoyancy:

1. The first step is to determine the ballast mass required to counter the buoyancy of the LWD. From Figure 8, $M_{BLWD} = 2{,}500$ kg x 0.89 (correction factor for difference in density between cedar and spruce) = 2,225 kg.

2. The second step is to determine the ballast mass required to counter the lift forces acting on the anchor boulders. From Figure 9, with $M_{BLWD} = 2{,}225$ kg, $M_{LB} = 90$ kg.

Hence, the total mass required to prevent the LWD from buoyant uplift is:

$\boldsymbol{M_B = M_{BLWD} + M_{LB} = 2{,}300}$ **kg**

Therefore, the boulder mass requirement to prevent a buoyant uplift failure governs the design (since $M_S < M_B$).

Once again with the use of Figures 3 and 10, we need 2 x 0.85 m diameter boulders (or equivalent mass) at the upstream end and 2 x 0.65 m diameter boulders (or equivalent mass) at downstream end of the LWD.

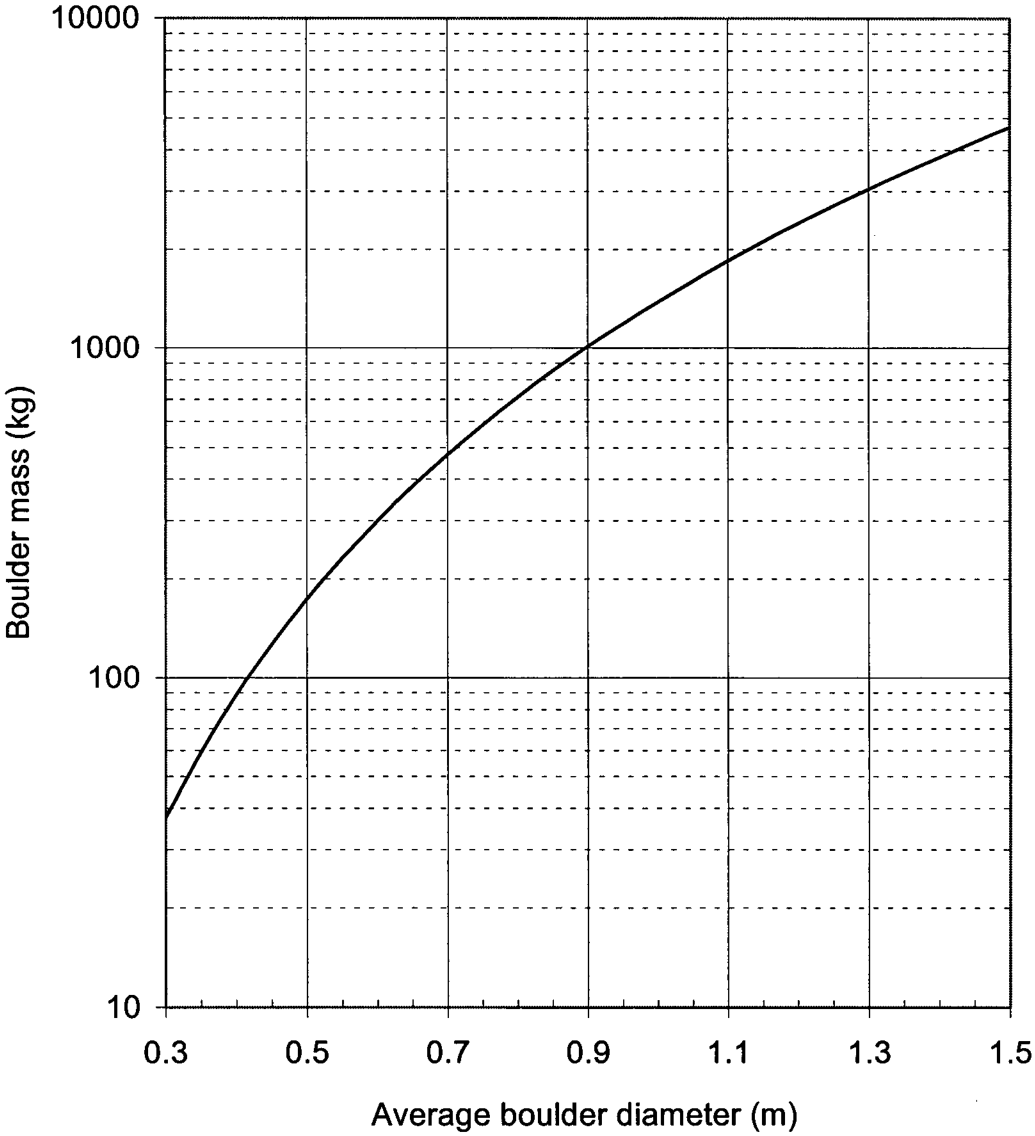
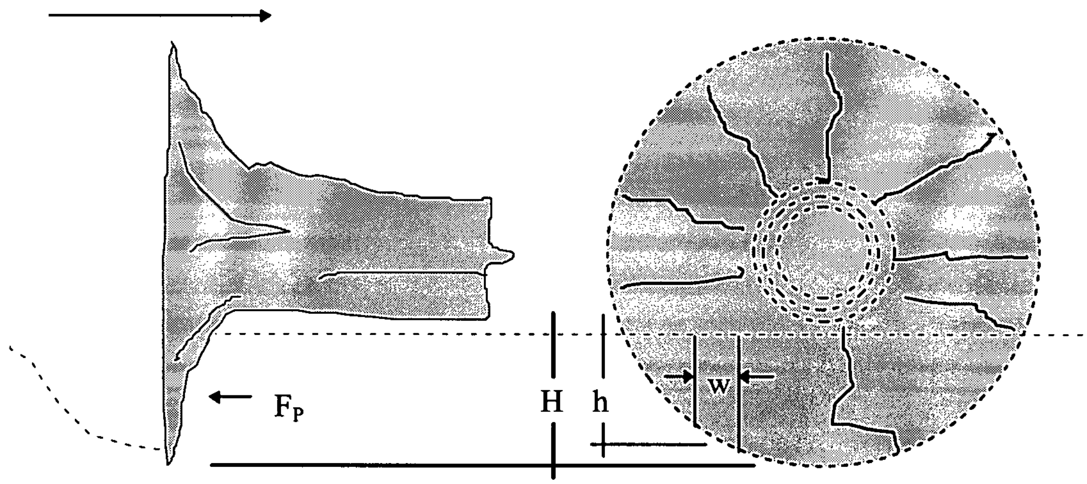


Figure 9: Boulder selection chart ($M_B = 1390D_B^3$)

Attachment 3: Added Stability through Partial Root Wad Embedment



Elevation -Side **Elevation - End**

If we look at the condition when the root wad is partially embedded in the watercourse's substrate and assume that the resisting force (F_P) exerted by the substrate downstream of the root wad may be estimated from theoretical passive earth pressures (P_P) defined as (Canadian Foundation Engineering Manual - 2nd ed., Canadian Geotechnical Society, 1985):

$$P_P = K_P \frac{\gamma' h^2}{2} \qquad K_P = \frac{1+\sin\phi}{1-\sin\phi}$$

Where, P_P is the force per unit width of root wad, K_P is the coefficient of lateral passive earth pressures, ϕ is the effective angle of internal friction, γ' is the submerged weight of the substrate (assumed as 10,000 N/m^3) and h is the height of the area acting against the root wad. If we assume that ϕ is equal to the angle of repose of approximately 35° (for rounded material with d_{50} = 25mm), K_P = 3.7.

If we take the end area of the root wad that is embedded in the substrate and subdivide it into a number of wedges of height h and width w, we may determine the magnitude of the added resisting force by:

$$F_P = \sum P_P w$$

For a 1.5 m diameter root wad embedded in the substrate by H = 0.5 m, F_P = 3,950 N. If we look at the effect of this added resistance to the LWD factor of safety. For a Douglas fir LWD with D_{RW}= 1.5 m, L = 10 m, D_L = 0.5 m and V = 2.5 m/sec, this added resistance will increase FS_S from approx. 2.0 to 2.45.

For a similar root wad embedded only by H = 0.3 m, F_P = 1,150 N and the FS_S increases from approx. 2.0 to 2.13.