STABILITY OF BALLASTED WOODY DEBRIS HABITAT STRUCTURES

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ABSTRACT: An important component of stream restoration in the Pacific Northwest is the reintroduction of large woody debris (LWD) and construction of LWD habitat structures in degraded systems. A significant problem faced by engineers involved in stream restoration is a lack of physically based guidelines for design and construction of ballasted LWD habitat structures. A simple theoretical approach is developed that forms the basis for determining ballast requirements for three types of LWD structures. Field monitoring and assessment were undertaken to test the approach and to compare predicted and observed stability for approximately 90 ballasted LWD structures. The results indicate that the stability of single-LWD and single-LWD with root wad structures can be successfully predicted by the theory. The stability of the multiple-LWD structures proved to be more complex to predict because numerous design and construction-related factors influence stability. A design approach based on factors of safety is recommended.

INTRODUCTION

Since the late 1970s there has been an increasing awareness of the important functions of large woody debris (LWD) within stream environments. There is extensive evidence that the dynamics and morphology of stream systems rich in 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 (Hogan et al. 1996). Furthermore, LWD has been indicated as the structural element most often associated with pool formation within small to large river systems (Robinson and Beschta 1990; Abbe and Montgomery 1996). Several studies have documented the significance of LWD in the function of stream ecosystems (Bustard and Narver 1975; Swanson and Lienkaemper 1978; Harmon et al. 1986; Hicks et al. 1991; Thomson 1991; Koski 1992). Aquatic fauna has an affinity for irregular stream features caused by fallen LWD, bedrock outcrops, rubble, and other debris. Fish species reared in streams depend on these features during different life stages and seasons for food, reproduction, and shelter from predators and environmental stresses (Shrivel 1990; Cederholm et al. 1997).

Under natural conditions, LWD finds its way into streams by means of progressive recruitment processes. These include undercutting of streamside trees by gradually migrating streams and windfall of riparian trees. Currently, many streams in British Columbia, Canada, are deficient in LWD compared to pristine conditions. This is the result of two, often compounded, activities: (1) Clear-cut logging of the riparian zone; and (2) removal of in-stream LWD. Habitat degradation resulting from such activities is believed to represent a major factor in the declines of salmonid stocks (Nehlsen et al. 1991). The Watershed Restoration Program of British Columbia is currently addressing these issues in an effort to improve fish spawning and rearing habitat. A major component of the in-stream restoration work includes the reintroduction of LWD into affected streams. Large wood, boulders, and other structural elements that emulate nature are being installed in disturbed streams to rehabilitate summer habitat and critical over-wintering refuges.

Natural LWD clusters or ''jams'' are often initiated by a large immobile log that acts as a stable key member (Abbe and Montgomery 1996). This key member acts as a trap for smaller, more mobile debris. The dimensions of LWD typically used in restoration is much smaller than that derived from a mature riparian forest. In the general absence of very large pieces to act as stable key members, boulders can be used as ballast to ensure the stability of constructed LWD structures. A significant problem facing stream restoration practitioners has been the lack of physically based design guidelines for the construction of ballasted LWD habitat structures. To date, most structures have been designed and constructed based largely on the judgment and experience of the designers and builders, many of whom had little training in hydrology and river engineering. The results were often underdesigned structures that incurred significant movement during flood events (Frissell and Nawa 1996; Hartman and Miles 1995). The current study was initiated in an effort to develop quantitative design guidelines for the construction and ballasting of LWD habitat structures.

Three types of ballasted LWD structures have been considered: (1) Single-LWD structures (SLS); (2) single-LWD with intact root wad structures (SLRWS); and (3) multiple-LWD structures (MLS) [Figs. 1(a–c)]. SLS, also known as single-log lateral jams or single-log deflectors, consist of a log projecting from the bank into the stream. At the bank end, the log is attached to a fixed point such as bedrock, a tree, or a stump, while the stream end is ballasted with one or more anchor boulders to prevent movement during floods [Fig. 1(a)]. The MLS, also known as triangular logjam, consists of two logs that are attached to trees or stumps on the bank and

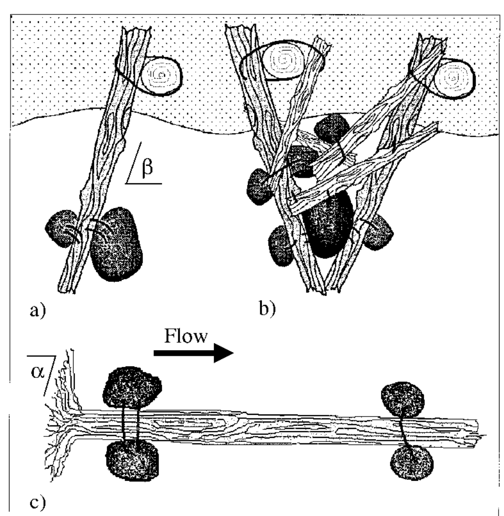

FIG. 1. Illustration of: (a) SLS; (b) MLS; (c) SLRWS

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that are both anchored by common boulders in the stream [Fig. 1(b)]. Once the basic triangular configuration has been provided, additional LWD and root wads may be added to increase the cover and habitat potential of such structures. Although the SLRWS may provide some direct benefits in terms of habitat cover, its primary use in river restoration is for bank and bar stabilization (through sheltering). It is generally composed of an LWD with its root wad intact and facing upstream [Fig. 1(c)]. Structure anchoring is provided via boulders.

OBJECTIVES

The principal objectives of this study are as follows:

- Develop a simple theoretical approach to evaluate the stability of the three types of LWD structures.
- Undertake field-testing and verification.
- Develop design recommendations for ballasting of LWD habitat structures.

THEORY

LWD structures are subject to a combination of hydrodynamic, frictional, and gravitational forces that act on either the LWD or the anchor boulders. The stability analysis is based on the accounting of forces acting directly on, or transferred, to the anchor boulders (Fig. 2). The basic stability analysis will initially be developed for the SLS and will then be modified for SLRWS and MLS. Full submergence of the LWD and anchor boulders is assumed. Partial burial or shielding of anchor boulders is not considered.

SLS

The principal forces considered are as follows:

- F_{BL}—Net buoyancy force acting on the LWD and transferred to the anchor boulder
- F_{DL}—Horizontal drag force acting on the LWD and transferred to the anchor boulder
- F_{DB}—Horizontal drag force acting directly on the anchor boulder
- F_{LB}—Vertical lift force acting directly on the anchor boulder
- W'—Immersed weight of the anchor boulder
- F_F—Frictional force at the base of the anchor boulder that resists sliding

These individual forces are discussed below.

Net Buoyancy Force Transferred from LWD (F_{BL})

The LWD is fixed at both ends [Fig. 1(a)], and, therefore, assuming that the forces are uniformly distributed along LWD, the net vertical buoyancy force transferred to the anchor boulder will be equal to half of the total net buoyancy force acting on the LWD

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

where L = length of LWD (m); D_L = mean diameter of LWD (m); ρ = density of water (1,000 kg/m^3); g = gravitational acceleration (9.81 m/s^2); and S_L = specific gravity of the LWD. A representative value for S_L must be specified, which is dependent upon LWD species and moisture content. In the present study, a value of S_L = 0.5 has been used, which is typical of coniferous species such as Douglas fir (*Pseudotsuga menziesii*) with a moisture content of about 15% (*Wood* 1995). Moisture content of 15% is considered representative of the

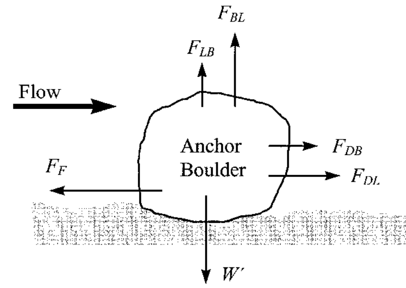

FIG. 2. Principal Forces Acting on Anchor Boulder [after Millar and D'Aoust (1998)]

LWD in this study at the time of placement and represents fairly dry conditions prior to submergence. Following submergence, S_L would be expected to increase appreciably, and long-term submergence could result in ''waterlogged'' values of S_L ultimately as high as 0.8 or 0.9. The assumed value of S_L = 0.5 represents a value at the lower end of the probable range.

Horizontal Drag Force Transferred from LWD (F_{DL})

The LWD is fixed at both ends, and, assuming that velocity is uniform along the LWD, half of the total drag force acting on the LWD will be transferred to the anchor boulders

$$F_{DL} = 0.5C_{DL}\rho\,\frac{V^2}{2}\,LD_L \sin\beta \tag{2}$$

where C_{DL} = drag coefficient; V = mean flow velocity (m/s); and β = angle, in horizontal plane, between the log and the stream flow (degrees) [Fig. 1(a)].

The value of C_{DL} can probably vary in magnitude from about 0.3 to 1.2, and perhaps higher, depending upon flow conditions, orientation, and surface roughness. Assuming that LWD behavior is hydrodynamically similar to cylinders, the experiment has shown that the value of C_{DL} would be highly dependent upon Reynolds number, $\mathsf{R} = VD_L/\nu$, where ν = kinematic viscosity. For infinite, smooth cylinders a critical value of R occurs at about 5×10^5. For R less than critical, a laminar boundary develops along the leading edge of the cylinder, and a value of $C_{DL} \approx 1.2$ is appropriate (Hoerner 1965). For R greater than critical, a turbulent boundary layer develops, and there is an abrupt fourfold reduction in C_{DL} to approximately 0.3 (Hoerner 1965). The precise value of critical R and the values of C_{DL} are complicated by surface roughness, end-effects, blockage, and LWD orientation (Gippel et al. 1992; Shields and Gippel 1995). In the current analysis a value of C_{DL} = 0.3 has been assumed. This value corresponds to a turbulent leading-edge boundary layer and is considered appropriate for the range of R in the present study (5×10^5 to 1×10^6). Orientation and blockage effects (Shields and Gippel 1995) have not been considered in the estimation here of C_{DL}; however, they could be readily incorporated if desired. Uncertainty in the value of C_{DL} is accounted for in the factor of safety, which is discussed below.

Horizontal Drag Force on Anchor Boulder (F_{DB})

The magnitude of the horizontal drag force acting directly on the anchor boulders is estimated using

$$F_{DB} = C_{DB}\rho\,\frac{V^2}{2}\,\frac{\pi D_B^2}{4} \tag{3}$$

where D_B = mean diameter of anchor boulder (m); and C_{DB} = drag coefficient. As with C_{DL}, the value of C_{DB} varies as a function of Reynolds number, $\mathsf{R} = VD_B/\nu$. The critical value of R for spheres is about 2×10^5. For values of R less than critical, $C_{DB} \approx 0.4$, which drops to about 0.2 for R in the turbulent region. In the present study, values of R were estimated to be in the range 5×10^5 to 2×10^6, and therefore a turbulent value of $C_{DL} = 0.2$ has been assumed. Partial hiding and shielding of the boulders by LWD is not accounted for by (3).

Vertical Lift Forces on Anchor Boulder (F_{LB})

The magnitude of the lift forces acting directly on the anchor boulders is estimated using

$$F_{LB} = C_{LB}\rho \frac{V^2}{2} \frac{\pi D_B^2}{4} = 0.85 F_{DB} \tag{4}$$

where the lift coefficient $C_{LB} = 0.17$ (Cheng and Clyde 1972). This is equivalent to 0.85 times the value of C_{DB} (Chepil 1958). As with F_{DB}, partial hiding and shielding of anchor boulders is not accounted for by (4).

Immersed Weight of Anchor Boulder (W')

The immersed weight of the anchor boulder is given by

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

where S_S = specific gravity of the anchor boulder ($\approx$2.65).

Frictional Force Resisting Sliding (F_F)

The critical frictional force is estimated as follows:

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

where ϕ = friction angle of the boulder on the streambed; and forces F_{BL} and F_{LB} are substituted with (1) and (4), respectively. There are no data available that refer specifically to the friction angle of boulder-size material resting on a gravel substrate. A value of $\phi = 40°$ has been assumed based largely on values for the angle of repose reported by the U.S. Bureau of Reclamation [cited in Henderson (1966, p. 420)]. The angle of repose for uniform gravel with a diameter in excess of 100 mm approaches a maximum limiting value of approximately 40°. There is some uncertainty as to whether this value can be reasonably applied to a large boulder resting on gravel substrate. However, the slope of waste rock dumps as well as natural scree and talus slopes are observed to develop at about 40°, which suggests that the assumed value of $\phi = 40°$ is reasonable when applied to mixtures of boulders and gravel. Scour and partial burial of the anchor boulders is not accounted for by (6).

Factors 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 be unstable. Two factors of safety can be defined (1) with respect to sliding; and (2) with respect to buoyancy. The factor of safety with respect to sliding (FS_S) is defined as

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

whereas the factor of safety with respect to buoyant failure (FS_B) is defined by

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

When $FS_B < 1.0$, the LWD-boulder complex will float, and therefore F_F and FS_S are assigned values of zero.

In the design process, the value of D_B is adjusted by iteration until (7) and (8) yield values of FS_S and FS_B that provide an acceptable level of safety to the designer.

SLRWS

The theoretical approach for design of SLRWS [Fig. 1(c)] is similar to that developed for SLS. However, drag on the root wad must also be accounted for. Furthermore, there is no anchor in the form of a tree or stump on the bank and resistance to sliding is provided entirely by friction on the streambed (F_F).

Vertical Force from LWD (F_{BL})

Assuming the root wad has the geometry of a cone, the buoyancy force transferred to the anchor boulders is given by

$$F_{BL} = \left(\frac{\pi D_L^2 L}{4} + \frac{1}{3} \frac{\pi D_{RW}^2 L_{RW}}{4} (1 - p) \right) \rho g (1 - S_L) \tag{9}$$

where L = length of the log (m) (excluding the root wad); D_{RW} = mean root wad diameter (m); L_{RW} = length of root wad cone (along LWD axis) (m); and p = proportion of void space in the root wad. A typical value of $p = 0.2$ has been assumed based on a visual field survey.

Horizontal Drag Force Transferred from LWD (F_{DRW})

The projected area of the root wad is assumed to be a disk of diameter (D_{RW}). The drag force transferred from the root wad to the anchor boulders is therefore given by

$$F_{DRW} = C_{DRW} \frac{\pi D_{RW}^2}{4} \frac{V^2}{2} \rho \sin(\alpha) \tag{10}$$

where α = angle of the root wad face with respect to the direction of flow (generally assumed to be 90°); and C_{DRW} = root wad drag coefficient. A value of $C_{DRW} = 1.2$ has been assumed, which is equal to that of a circular plate suspended in flow with a Reynolds number of 10^4–10^6 (Hoerner 1965).

Other Forces

The skin friction force acting on the length of the LWD trunk is not considered important for two reasons: (1) The trunk lies largely in the wake of the root wad; and (2) a sensitivity analysis showed that the magnitude of the skin friction is likely to be one or two orders of magnitude smaller than the other forces. It is also assumed that the trunk is free of branches.

The remaining forces acting on the anchor boulders (F_{DB}, F_{LB}, W', and F_F) can be computed from (3)–(6) presented previously.

Factors of Safety

As with the SLS, two factors of safety must be considered: (1) FS_S; and (2) FS_B. The factor of safety with respect to sliding is defined as

$$FS_S = \frac{F_F}{F_{DRW} + \sum F_{DB}} \tag{11}$$

where F_F, F_{DRW}, and F_{DB} are computed using (6), (10), and (3), respectively. Eq. (8) may be used to compute the factor of safety with respect to buoyant uplift where F_{BL} is computed from (9).

MLS

The preceding theory developed for SLS will now be modified to apply to MLS [Fig. 1(b)]. In contrast to the SLS, drag forces acting on the MLS are difficult to quantify because significant sheltering occurs between LWD members. However, the structural stability in a horizontal plane is provided by lateral bracing, and not through frictional resistance acting on the ballast boulders. Therefore lateral drag forces do not need to be considered explicitly, and the factor of safety against buoyancy (FS_B) can be used as a simple design criterion that ensures MLS do not float when submerged. It is generally desirable that the structures remain in contact with the bed during high flows to promote scour and pool formation.

For MLS the factor of safety with respect to buoyancy (FS_B) is defined as

$$FS_B = \frac{\sum W'}{\sum F_{BL} + \sum F_{LB}} \tag{12}$$

Eq. (12) is similar to (8) except that the buoyancy force (F_{BL}) must be determined from (1) for each LWD and summed to yield a total for the complex.

Magnitude of Forces

It is worthwhile at this stage to review three hypothetical examples in order to appreciate the absolute and relative magnitude of the individual force components described above. An example for each structural type is presented in Table 1. For an SLS with values of L = 6 m, D_L = 0.5 m, β = 70°, and V = 2.5 m/s, solving (1)–(8) indicates that a ballast mass of M_B = 1,200 kg would be necessary to provide a factor of safety with respect to sliding of FS_S = 2.0. The corresponding factor of safety with respect to buoyancy FS_B = 2.2. The required ballast could be provided by one 0.96-m-diameter boulder or a combination of two or more boulders with the same total mass. The magnitude of forces are also presented for an SLRWS with values of L = 6 m, L_{RW} = 0.75 m, D_L = 0.5 m, D_{RW} = 1.4 m, α = 90°, and V = 2.5 m/s and for an MLS composed of three pieces of LWD: (1) L = 8 m, D_L = 0.6 m; (2) L = 8 m, D_L = 0.6 m; and (3) L = 6 m, D_L = 0.5 m, and V = 2.5 m/s.

These hypothetical examples demonstrate that the principal forces involved in the analysis are the gravitational body forces (F_{BL}, W'), frictional forces on the streambed (F_F), and the fluid drag forces on the LWD (F_{DL}, F_{DRW}). Fluid forces acting directly onto the anchor boulders (F_{DB}, F_{LB}) are relatively minor. It is evident from these computations that the ballast mass requirement increases significantly for SLRWS, which rely exclusively on boulders and cabling for lateral stability.

TABLE 1. Magnitude of Forces for Three Hypothetical Structures

Parameter (1)	SLS (2)	SLRWS (3)	MLS (4)
F_{BL} (N)	2,900	7,300	14,000
F_{DL} (N)	1,300	—	—
F_{DRW} (N)	—	5,800	—
F_{DB} (N)	450	1,700	—
F_{LB} (N)	390	1,400	790
W' (N)	7,500	26,400	22,200
F_F (N)	3,500	14,900	—
M_B (kg)	1,200	4,300	3,600
FS_S	2.0	2.0	—
FS_B	2.2	3.0	1.5

METHODOLOGY

A program of field-testing and verification was undertaken in order to test the proposed theory. Ninety ballasted LWD structures underwent detailed preflood and postflood surveys. The structures were surveyed over the summer and fall of 1997 to determine location and to predict their stability under high flow conditions. A resurvey of all structures was completed during the spring and summer of 1998 following winter rainstorms and spring snowmelt floods. The structures in the

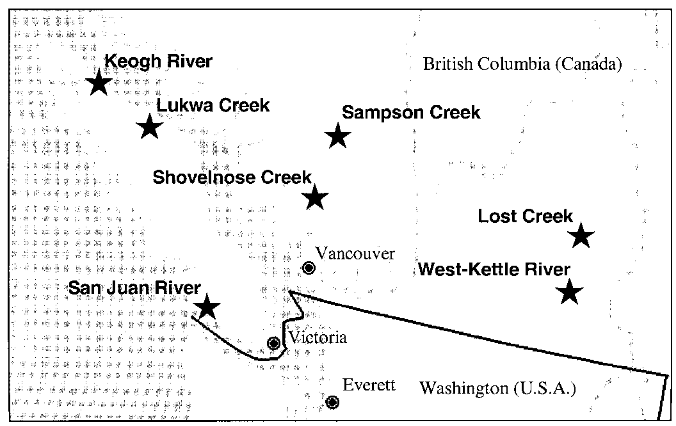

FIG. 3. Site Location Map

TABLE 2. Summary of Sites [Modified from D'Aoust (1998)]

River system/reach (1)	ID (2)	Area[a] (km²) (3)	Q_{Bf} (m³/s) (4)	W_{Bf} (m) (5)	Slope (%) (6)	SLS (7)	SLRWS (8)	MLS (9)
Keogh River[b]								
Wolf Creek	A	19	22	11	1.50	—	—	7
103–104	B	22	25	15–30	0.47	7	—	11
Tributary 13	C	29	34	15–30	0.40	2	—	2
West Main	D	39	46	18	0.90	7	—	3
West 80	E	39	46	16	0.70	7	1	2
Lost Creek	F	45	7.2	8	2.30	—	—	2
Lukwa Creek	G	23	34	14	1.00	—	—	2
Sampson Creek	H	40	15[c]	12	0.21	—	—	8
San Juan River	I	730	795	120	0.036	1	7	2
Shovelnose Creek	J	21	45	38	0.45	3	1	6
West-Kettle River	K	1,520	116	44	0.28	—	—	8

[a]Tributary area to reach of interest.
[b]Distributed over five rehabilitated reaches.
[c]Estimate based on uniform flow computations and reach-averaged bank-full geometry.

test sample were located in seven river systems throughout southwestern British Columbia (Fig. 3). A brief summary of the study sites is provided in Table 2. All of the LWD structures investigated were designed and built independently by watershed practitioners using their judgment and experience to determine ballast requirements. This resulted in a range of underballasted and overballasted structures that could be used to test the stability analysis. In the West Kettle River, the designer/builder estimated boulder requirements using the approach presented with an $FS_B = 2$.

Detailed information with respect to size, vertical and horizontal location of structural components, and flood stage was collected for each structure. Mean flow velocities and discharges were computed using Manning's equation together with an estimate of the roughness coefficient. This information provided the data necessary to compute the factors of safety under observed flood conditions and to detect any lateral movement of the structures.

Because the majority of the LWD structures were not quite fully submerged under the observed flood conditions, it was necessary to make a small adjustment in applying the theoretical approach developed previously. The submerged length of LWD (L_S) was used in the analysis instead of the full length of LWD (L). This substitution assumes that, although the structure may not be fully submerged, the buoyancy and drag forces on the submerged portion of the LWD will be equally distributed between bank and stream anchors. The weight of LWD above water was not considered in the analysis. A more detailed force analysis using moments indicated that this adjustment adequately estimated the factors of safety for the LWD structures in all but a few instances where the mass contribution from the LWD above water significantly increased the structural stability (Table 3).

Predictions on the stability of the structures were based on the computed FS_S and FS_B under observed high flow conditions. For single-LWD and single-LWD with root wad, an $FS_S < 1.0$ would indicate potentially unstable conditions, whereas an $FS_S \geq 1.0$ would suggest stable conditions. In the case of MLS, the same approach was adopted based on the computed FS_B.

Triangulation measurements similar to that described by Koonce (1990) were employed to monitor movement of the LWD and anchor boulders on a horizontal plane. It was possible to triangulate structural components to within 0.2 m depending on the density of riparian vegetation and complexity of the structure (e.g., pieces under water). Within small and medium systems, a displacement in excess of 0.5 m was considered to represent an instability of the structure. This criterion was established based on the measurement error involved in the triangulation measurements while allowing for some minor adjustment of the structural components (e.g., settling in of anchor boulders and tightening of cables). For the San Juan River, the largest system assessed, high vertical cut banks, dense riparian vegetation, and the great distances to measure decreased the accuracy of the triangulation measurements. In this system, the accuracy of the measurements was within 0.4 m. Therefore, for the SLRWS located within the San Juan River, a movement in excess of 0.7 m was thought to represent an instability of the structure. The structures were also qualitatively categorized according to their postflood functionality. Three categories were utilized for classification:

- Nonfunctional—These structures shifted considerably and were not in contact with the low water level at the time of the postflood assessment.
- Partially functional—These structures 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'' was also used because their stability may be precarious and future floods may render them nonfunctional.
- Functional—When they were essentially intact, these structures appeared to function as intended and still had the potential of achieving their original objectives.

Stream flow records for the fall 1997 to spring 1998 period indicate that the study streams were subject to floods ranging from a 1.1- to 2.1-year return period. The recorded peak flows and associated return frequencies for the study streams are tabulated in Table 4.

TABLE 3. MLS—Summary of Nonconforming Structures

Case (1)	Structure (2)	Observation (3)
Unstable with $FS_B \geq 1.0$	K1	Significant LWD on face of structure
	K7	Significant LWD on face, bed, and bank scour
	A6	Inadequate triangular bracing; loose cabling
	A4	LWD overhanging at bank end and loose cabling
	K6	In-stream boulders submerged (sizes estimated)
	B8	Upstream-V structure spanning channel, boulder ballasted at bank ends
	J1	Steel cabling quite loose, and boulder ballasted at bank ends
	E1	Inadequate triangular bracing; no bank anchors
	A3	Downstream-V structure spanning channel; not fixed at bank ends; one anchor boulder split in two
Stable with $FS_B < 1.0$	G2	Good triangular bracing and tight cabling
	J2	Some anchor boulders partially covered in sands; possible underestimation of boulder mass
	H4	Significant LWD above water surface
	G1	Good lateral stability provided by tight cabling
	H2	Good triangular bracing through geometry and cabling
	H8	Significant LWD above water surface

RESULTS

SLS

There is a good agreement between predicted and observed stability of the SLS based on their computed FS_S [Fig. 4(a)].

TABLE 4. Recorded Peak Flows (Fall 1997–Spring 1998) [Modified from D'Aoust (1998)]

River system (1)	Flow (m^3/s) (2)	Date (3)	Return period (year)[a] (4)
Keogh River	124[c]	December 14, 1997	2.1
Lukwa Creek[b]	38	N/A	>2.3
Sampson Creek[b]	9	N/A	<2.3
San Juan River	560	December 16, 1997	1.1
Shovelnose Creek[c]	33	N/A	<2.3
West-Kettle River	124	May 3, 1998	2.0
Trapping Creek	14	May 4, 1999	1.4

Note: N/A = not available (ungauged systems).
[a]Based on flood-frequency analysis fitted with a Gumbel distribution.
[b]Measured at river mouth (135 km^2).
[c]Ungauged systems, average of computed flow assuming uniform flow at observed high-water marks.

All 14 single-log structures with calculated values of FS_S > 1.1 (structures E6–D3) were observed to be stable. Eleven of the 13 structures with calculated values of FS_S < 1.1 were unstable including structures B2, B3, B4, B7, E1, E2, and E5 with values of FS_S = 0. There were two anomalies, structures C1 and C2, with computed FS_S < 1.0, that should have been unstable but were observed to remain in place after high flows. However, this can be readily explained as the anchor boulders of both structures were partially buried. Hence, the size of the boulders may have been underestimated, and the partial burial would have provided additional resistance to movement.

Values of FS_S were recalculated based on the postflooding orientation of structures D1 and E4. The recalculated values of FS_S were 1.1 and 1.2, respectively, which indicated that these structures had shifted to a stable configuration close to the theoretical value of FS_S = 1.0.

SLRWS

Predicted factors of safety for the SLRWS also agreed well with observed stability [Fig. 4(b)], as predicted based on the computed values of FS_S. Three out of the nine structures evaluated (structures I4, E8, and I6) were observed to be unstable. Structure I4, which had a calculated FS_S = 0, was washed away, whereas structures E8 and I6 shifted only slightly. The other structures remained intact in their preflood location.

MLS

A total of 51 MLS were investigated. Postflood surveys indicated that 23 structures shifted laterally by more than 0.5 m. Comparisons between the observed stability and computed factor of safety are less straightforward than for the SLS and SLRWS. Computed FS_B are not directly related to the lateral stability, which is presumed to be provided by the triangular bracing. Nonetheless, a general increase in stability with increasing FS_B is apparent (Fig. 5). For the 21 MLS with computed FS_B < 1.0, 15 (71%) shifted by more than 0.5 m, of which 8 were considered to be nonfunctional or only partially functional. For the 30 MLS with computed FS_B > 1.0, 8 (27%) shifted by more than 0.5 m, and only 2 were considered to be partially functional or nonfunctional. The instabilities observed for structures B5 and F1 can be attributed to the failure of one of their in-stream fasteners.

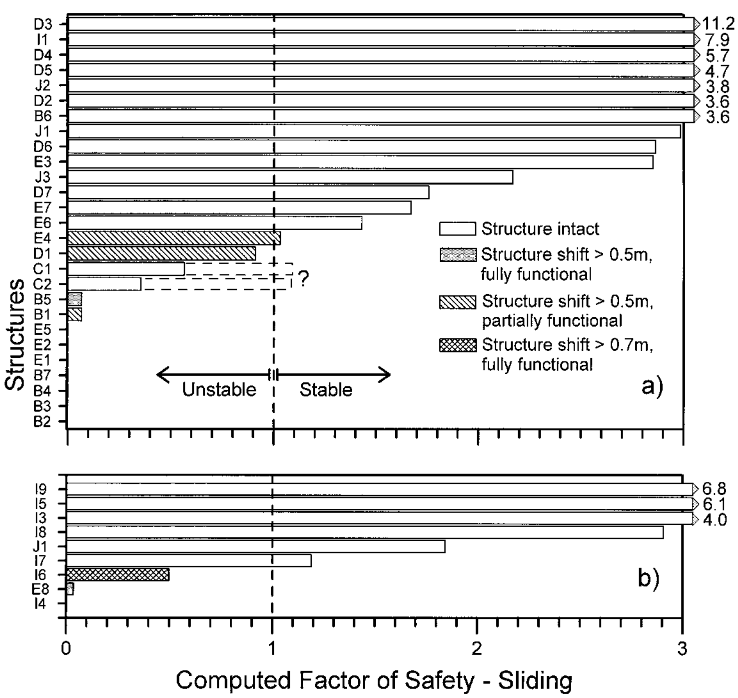

FIG. 4. Stability of: (a) SLS; (b) SLRWS (B2, B3, B7, and I4 Shifted/Nonfunctional, B4 and E5 Shifted/Partially Functional, and E1 and E2 Shifted/Functional)

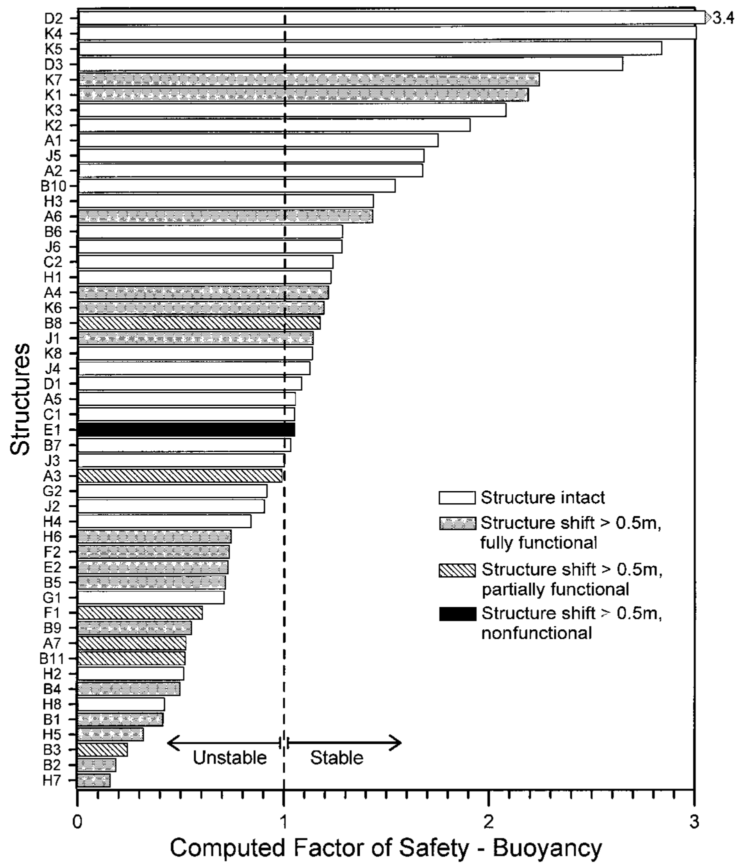

FIG. 5. Stability of MLS

DISCUSSION

SLS

The successful predictions of the stability of single-log structures lend strong support to a design approach based on a factor of safety against sliding (FS_S). The predictions based on the computed FS_S were accurate in all but two cases, where their deviation is attributed to the partial burial of their anchor boulders. Hence, the use of this design method enables one to determine anchor mass requirements for single log structures while allowing an acceptable safety margin through the use of an FS_S > 1.0.

Most of the observed instabilities were due to the LWD being unsecured at the bank end. Because half of the drag and buoyancy forces are transferred to the bank end, cabling to a tree or stump is necessary to ensure stability. Alternatively, in the absence of a suitable tree or stump, the ballast requirement may be doubled, and half of the anchor boulders placed at the bank end.

SLRWS

Despite the relatively small test sample (n = 9), it appears that the stability of the SLRWS can be adequately predicted by the approach. Therefore, the proposed approach can be used as the basis for design and can provide an adequate FS_S to ensure the stability of these structures.

MLS

As mentioned previously, compared to the SLS and SLRWS, the interpretation of the results for the MLS is less straightforward. It should be realized that the factor of safety against buoyancy (FS_B) is used as a simple design criterion that is only indirectly related to the lateral stability of the MLS. 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 movement as the calculated FS_B increase. The MLS that sustained the largest reduction in habitat functionality (partially functional or nonfunctional after flooding) all had computed values of $FS_B < 1.25$. Only two of the 12 structures with computed values of $FS_B > 1.5$ experienced significant lateral movement (K1 and K7). 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 movement (Table 4).

In some cases additional boulder mass alone would not have prevented movement. Poor triangular bracing, loose cabling, and lack of fixed bank anchors are suspected to have contributed to the instabilities (Table 4). Conversely, structures that exhibited good triangular bracing (including fixed bank anchors) and tight cabling were not observed to shift laterally despite a value of $FS_B < 1.0$ (Table 4).

DESIGN RECOMMENDATIONS

Based on the results obtained for the SLS and SLRWS, there is a clear transition in observed stability as the FS_S increases above 1.0 (Fig. 4). Despite the increased difficulty in predicting the stability of MLS, a reduction in observed instabilities was apparent for an FS_B in excess of about 1.25 (Fig. 5). In British Columbia this work has led to minimum factors of safety for design of 1.5–2.0. These minimum values should provide the additional margin of safety necessary to account for uncertainties in values of the coefficients, design velocity, and loading from additional LWD. In cases where the rehabilitation site is in a higher risk area, such as upstream of a bridge crossing, or where a structure is likely to accumulate large amounts of additional debris, the factor of safety used in design should be increased accordingly.

Values for a number of coefficients have been assumed in this study, including S_L, C_{DL}, and tan ϕ. The values used in this paper reflect the materials and conditions encountered in the present study. Adjustment of coefficient values may be required if the method is applied to regions outside the Pacific Northwest.

CONCLUSIONS

The theoretical approaches presented herein were used successfully to predict the stability of SLS and SLRWS during the fall 1997 to spring 1998 floods. Hence, it is believed that a design approach based on computed FS_S can be effectively used to determine ballast mass requirements for these structures to ensure stability.

Based on the strict stability criterion established, observed stability of about 25% of the MLS did not agree with predictions based on the FS_B. A number of factors are suspected of having contributed to the observed instabilities including (1) inadequate triangular bracing; (2) loose cabling; (3) inadequate anchoring to the bank; (4) significant LWD accumulation on upstream face; and (5) localized bed and bank scouring. Conversely, stability of MLS was enhanced through good triangular bracing and tight cabling.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

C_{DB} = drag coefficient of boulder;
C_{DL} = drag coefficient of LWD;
C_{DRW} = drag coefficient of root wad;
C_{LB} = lift coefficient of boulder;
D_B = anchor boulder diameter (m);
D_L = average diameter of log (m);
D_{RW} = average root wad diameter (m);
F_{BL} = net buoyancy force acting on LWD transferred to anchor boulders (N);
F_{DB} = drag forces acting on anchor boulders (N);
F_{DRW} = drag forces acting on root wad transferred to anchor boulders (N);
F_F = frictional forces resisting sliding (N);
F_{LB} = lift forces acting on anchor boulders (N);
FS_B = factor of safety against buoyant uplift;
FS_S = factor of safety against sliding;
g = gravitational acceleration (9.8 m/s^2);
L = length of LWD excluding root wad (m);
L_{RW} = length of root wad (m);
L_S = submerged length of LWD (m);
M_B = ballast mass (kg);
p = proportion of voids in root wad;
Q_{Bf} = bank-full discharge (m^3/s);
R = Reynolds number;
S_L = specific gravity of LWD;
S_S = specific gravity of anchor boulders;
V = mean flow velocity (m/s);
W' = immersed weight of anchor boulders (N);
W_{Bf} = bank-full channel width (m);
α = angle of root wad face with respect to flow;
β = angle of LWD with respect to flow;
ν = kinematic viscosity (1.5×10^{-6} m^2/s);
ρ = density of water (1,000 kg/m^3); and
ϕ = friction angle of anchor boulders on streambed.