chapter 7

Designing log structures

7.1 Data requirements to perform force-balance stability analysis and design of a wood reintroduction strategy

To undertake the design of a wood reintroduction strategy, the following data is regarded as being the minimum to adequately complete a full reach design.

- Channel cross section surveys to morphological bankfull height and onto the floodplain. Representative sections spaced at no more than one channel widths separation, ideally located at the site of each structure location, with a minimum of 10 per site to try to encapsulate more than one complete riffle-pool sequence (if they exist).
- Channel long profile survey (at least three riffle pool sequences or 15–20 channel widths long). This is for determining the reach bed slope — i.e. as a regression from riffle to riffle (if riffles exist).
- Bed material samples: one per cross section.
- Some flow discharge magnitude/frequency data from which a design discharge can be selected (e.g. 10 year ARI discharge). If gauging data is not available, a regional catchment area/discharge relation can be used. As a minimum, the morphological bankfull discharge can be estimated using the Manning equation from the slope and cross section area data.
- Volume of wood being reintroduced can be estimated based on assumed log diameter at breast height (dbh) and lengths.
- Wood dry density (use 900 kg m^{-3} if a eucalypt, and species not known).

See Table 4 overleaf.

7.2 Selecting a design flood

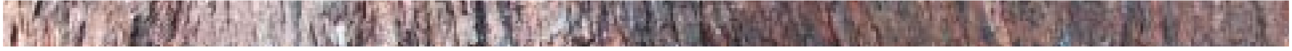
There is no hard and fast rule for the selection of a design discharge. Different engineers will have their own design discharges that they use in different circumstances, but by and large it is a question of the risk (of structure failure) that you are prepared to accept. The primary reason for selecting a design flood is that you want your structure to withstand, with high probability, a flood of this magnitude. As a result, you will design your structures to withstand the forces imposed by a flood of this magnitude within standard factors of safety (to account for sources of error in the calculations and an additional comfort margin). Typical (minimum) factors of safety that most engineers are prepared to live with are in the order of 1.5 to 2. Ideally, once a design discharge is selected, a sensitivity analysis will also be undertaken to assess the performance of the structures under conditions that are more extreme than the design flood.

Factors that must be taken into account when selecting a design flood are as follows:

- The index of flow variability (Iv) of your stream (see Finlayson & McMahon 1988, Rutherfurd et al. 2000). The higher the Iv, the greater the difference between the frequent small floods and rare large floods. In the situation where you have a high Iv it will be prohibitively expensive to design structures that will withstand the rare extreme events. To design something capable of withstanding the 1:100 year event would require structures to be massively over-designed for the circumstances likely to be encountered for the majority of the time. Conversely, in streams with a low Iv, there will be little difference

Table 4. Wood density characteristics for common Eucalypt species (Bootle 1983).

Common name	Species name	State	Green density ($kg\ m^{-3}$)	Dry density ($kg\ m^{-3}$)
Rough bark apple	*Angophera floribunda*	NSW, Qld	1180	850
Smooth bark apple	*A. costata*	NSW	1240	990
Alpine ash	*Eucalyptus delegatensis*	Tas, NSW, Vic	1050	620
Mountain ash	*E. regnans*	Tas Vic	1030	680
Silvertop ash	*E. sieberi*	NSW, Vic	1200	820
Blackbutt	*E. pilularis*	NSW, Qld	1100	900
WA blackbutt	*E. patens*	WA	1120	850
Red bloodwood	*E. gummifera*	NSW, Vic, Qld	1150	900
Mountain grey gum	*E. cypellocarpa*	NSW, Vic	1100	880
Forest red gum	*E. tereticornis*	Vic, NSW, Qld	1200	1050
River red gum	*E. camaldulensis*	Vic, NSW, Qld	1130	900
Sydney blue gum	*E. saligna*	NSW	1070	850
Spotted gum	*E. maculata*	NSW, Vic, Qld	1150	950
Karri	*E. diversicolour*	WA	1200	900
Jarrah	*E. marginata*	WA	1170	820
Grey ironbark	*E. paniculata*	NSW	1210	1120
White stringybark	*E. globoidia*	NSW, Vic, Qld	1100	880
River sheoak	*Casuarina cunninghamiana*	NSW, Qld	970	770
Southern mahogany	*E. botryoides*	NSW, Vic	1180	920
Silky oak	*Grevillia robusta*	NSW, Qld	1100	620



in design specifications for a structure that can withstand the 1:10 year event and the 1:50 or the 1:100 year event.

- Consequences of structure failure. The extent to which you may over-design a structure and hence increase costs and material requirements, will in part be a function of the consequence that would result from the structure's failure. If, for example, you are undertaking a rehabilitation program within a semi-urban environment in a stream immediately upstream of a sequence of bridges or culverts adjacent to a floodplain housing development, then the consequences of the failure of your rehabilitation strategy are potentially quite extreme. Under these circumstances you might select a higher magnitude design flood, and possibly higher factors of safety to ensure that risk of structure failure is minimised. This will of course result in higher construction costs, and possibly mean that a significant portion of the structure is functionally redundant for the majority of the time. In the majority of cases, however, where rehabilitation programs are undertaken in rural streams where there is very little risk of damaging critical infrastructure, smaller design discharges can be used and/or less conservative (lower) factors of safety. As outlined in Section 3.3, when using good quality eucalypt hardwoods, even if some structures partially fail it is unlikely the logs will move far beyond the structure from which they originate. In the Williams River study approximately 14 logs became dislodged from various structures and none moved beyond the study reach.
- Expected or desired longevity of the structure. The design flood selected should bear some relationship to the expected, or desired, life of the structures.

In streams that are ungauged and there is no flow rating curve available from which to select your design discharge (as per Figure 23), a regional flow rating curve (i.e. catchment area/discharge curve) will need to be derived from the available river gauge data in your region, from catchments having similar rainfall-runoff characteristics. Most state governments have a HydSys database of flow gauge data from which rating curves can be extracted. In most cases the standard Log-Pearson III (LP3) curve that has been fitted to the annual series curve, can be used to determine the annual

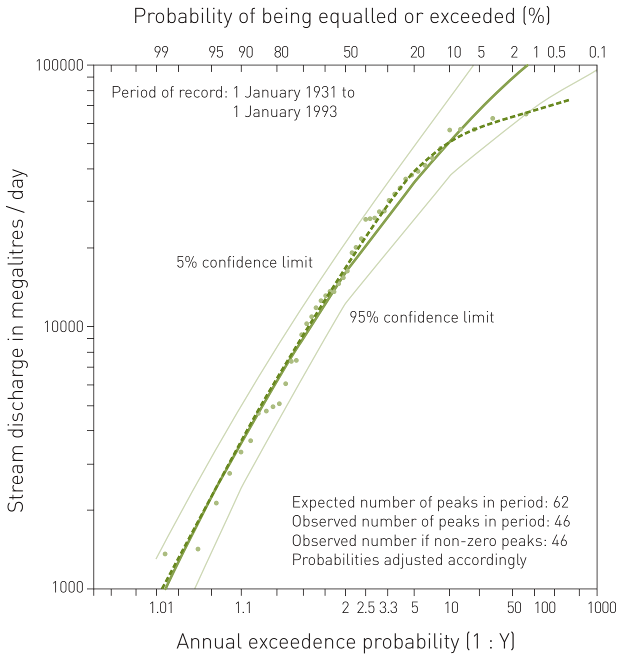


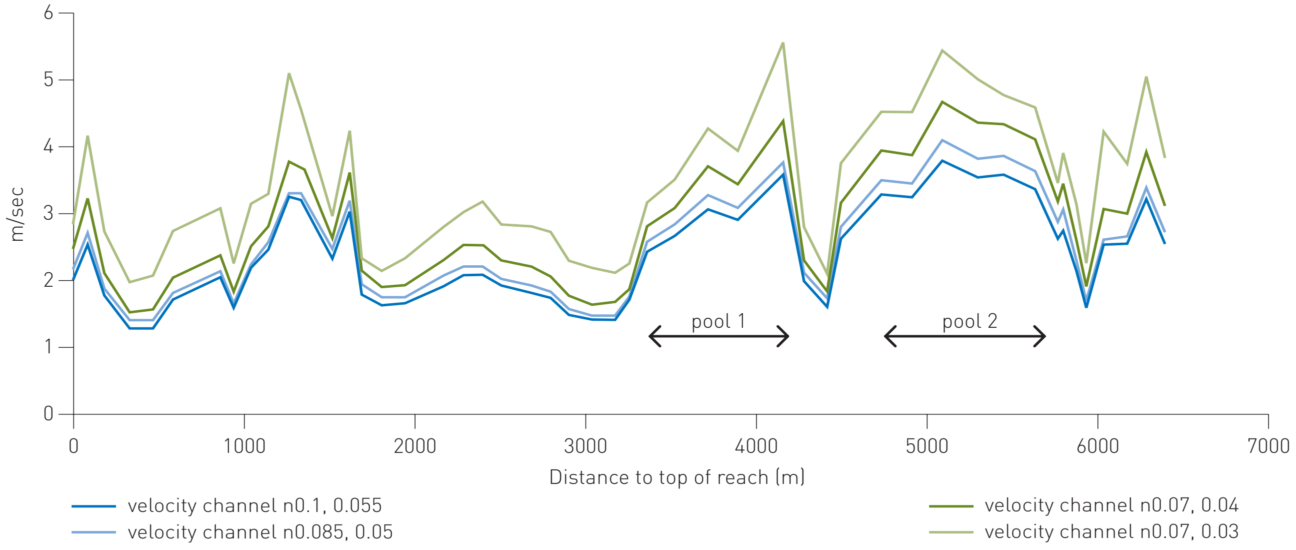
Figure 23. Annual exceedence probability curves for the Tillegra gauge (station 210011). Solid line is the Log Pearson (III) curve, dashed line fitted by eye.

exceedance probability. It should be pointed out, however, that in some cases the Log-Pearson III curve does not fit the data very well at the upper end of the curve, and it may be better to fit a curve by eye. In the example shown in Figure 23 there is a very poor fit at discharges greater than a 10 year ARI using the LP3 curve fit. If selecting a 20 year ARI design discharge based on the LP3 curve from the Tillegra gauge, a discharge would be derived that is greater than any flood recorded within the 70 years of gauging represented by this plot. This means that selecting the 20 year discharge based on LP3 would result in significant over design of the structures.

7.3 Hydraulic modelling

Having now collected your field survey cross sectional and long profile data and selected your design discharge, the next stage in the design process is to set up a 1D hydraulic model of your rehabilitation reach. The industry standard, and most accessible (i.e. free), hydraulic modelling software for undertaking this task is the HecRas model developed by the US Army Corp of Engineers (see http://www.hec.usace.army.mil/software/hec-ras/hecras-download.html). It is beyond the scope of this Guideline to outline the process required to undertake a HecRas modelling analysis, however, free tutorials are available for download on the web, and it is not particularly difficult to teach yourself how to do rudimentary 1D hydraulic modelling (see http://www.ce.utexas.edu/prof/maidment/grad/tate/research/RASExercise/webfiles/hecras.html). The primary output we are interested in from this exercise is the cross sectional depth averaged velocity at the intended locations of the structures. It must be remembered that HecRas is not a dynamic model and does not take into account bed scour (and hence increased cross sectional area) during high stage flows. On balance, this is likely to lead to an over prediction of peak flood stage. Figure 24 provides an example of a typical output from a HecRas model run, showing the within-reach variability in velocity down a 6 km reach of the Hunter River, as well as the sensitivity of the output to variations in hydraulic roughness. Similar model runs to this would ideally be undertaken using different discharge inputs. Note how velocities are higher in the pools at flood stage due to the greater hydraulic mean depth of the pool cross sections compared with the riffles. Depending on the scale of structures being designed, and if the rehabilitation reach is in an area where there are sensitivities to any increases in flood stage, it may be advisable to undertake model runs that include the projected cross sectional obstruction posed by the structures.

Figure 24. Example of HecRas model output for determining design velocities for structure emplacement locations.



7.4 Prediction of scour at LWD

As outlined in Section 6.2, scour depth prediction in streams with a mobile alluvial bed is notoriously difficult. In the absence of anything else, the two empirical approaches outlined in Section 6.2 (i.e. thalweg residuals and the Faraday and Charlton method) can provide a first order approximation of maximum scour depth. However, these approaches tend to work best in gravel bed rivers, where scoured pools are less likely to infill under moderate or low flow conditions or on the waning stage of the hydrograph. Sand bed rivers pose a particular problem because pools can form and disappear during the course of a single flood, particularly when sediment supply is high. Thus, predicting scour on the basis of observed reach geomorphology is likely to grossly under predict maximum scour depth. Conversely, the empirical approach of Faraday and Charlton tends to overestimate scour in sand bed channels.

Maximum scour occurs where maximum turbulence is induced, and this will tend to be at sites where obstructions such as log structures protrude into the high velocity flow thread. Figure 25 shows the zone of maximum scour around a bar apex jam (sensu Abbe et al. 1996) in the Queets River, in Washington state, USA, which behaves as a mid-channel bluff body obstruction. From this example, it can be seen that there is major scour at the front of the structure associated with downwelling flow separation, and a more linear pool along each flank of the log jam associated with the vortex street shed off either side of the structure. Bank attached structures show a very similar pattern of scour, albeit only on the streamward side of the structure, as this example of the measured scour at the Williams River site demonstrates (Figure 26). Maximum depth of scour in this case was in the order of 2 m.

7.4.1 2D hydrodynamic modelling

In some circumstances, where your budget allows it, and where detailed insights into the predicted response to rehabilitation works are required, it may be justified to undertake 2D hydrodynamic modelling to predict scour and deposition likely to result from the proposed treatment. This is an expensive option and in most cases is probably not justified. It is also subject to the vagaries of any modelling exercise in that the output is only as good as the input data and assumptions. Model parameterisation and calibration may indeed present more of a problem than the rehabilitation exercise itself. Nevertheless, in some high profile projects, such as the one undertaken by Tim Abbe (Figure 27) to construct large log jams to halt erosion of an interstate highway, 2D modelling can provide convincing evidence of the

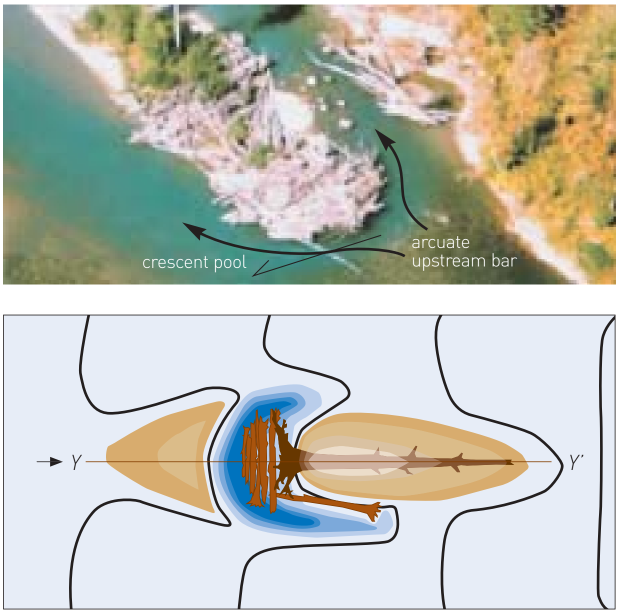


Figure 25. Scour associated with a bar apex jam — Queets River, USA. Courtesy T. Abbe.

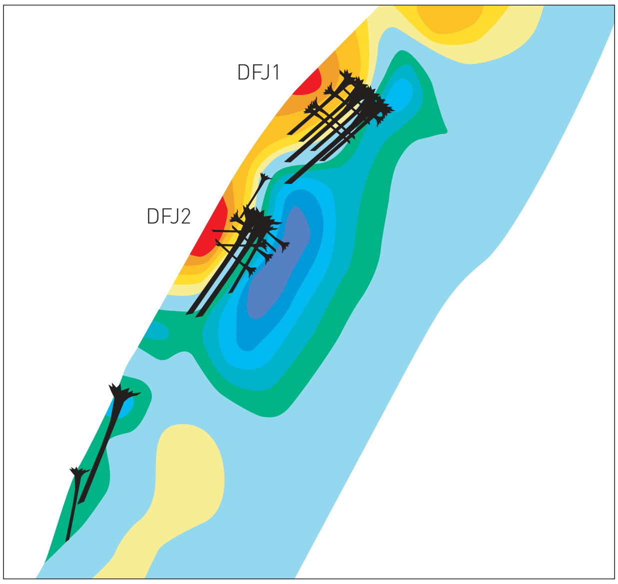
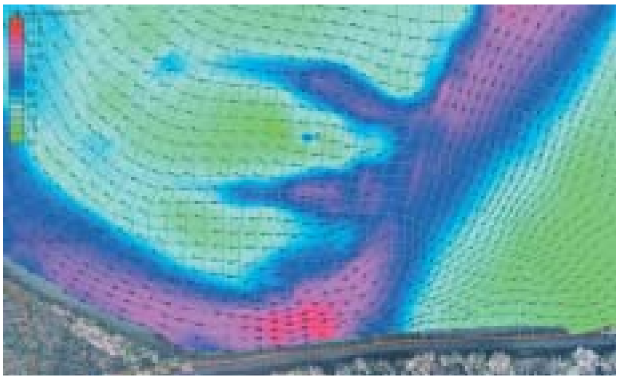


Figure 26. Scour (blues and greens) associated with bank attached jams — Williams River.

Figure 27 (below and right). Example output of 2D hydrodynamic modelling from the Hoh River ELJ project WA, USA. Courtesy T. Abbe. **27a** (below) shows the situation before construction, **27b** (right) illustrates the predicted changes after structure emplacement.



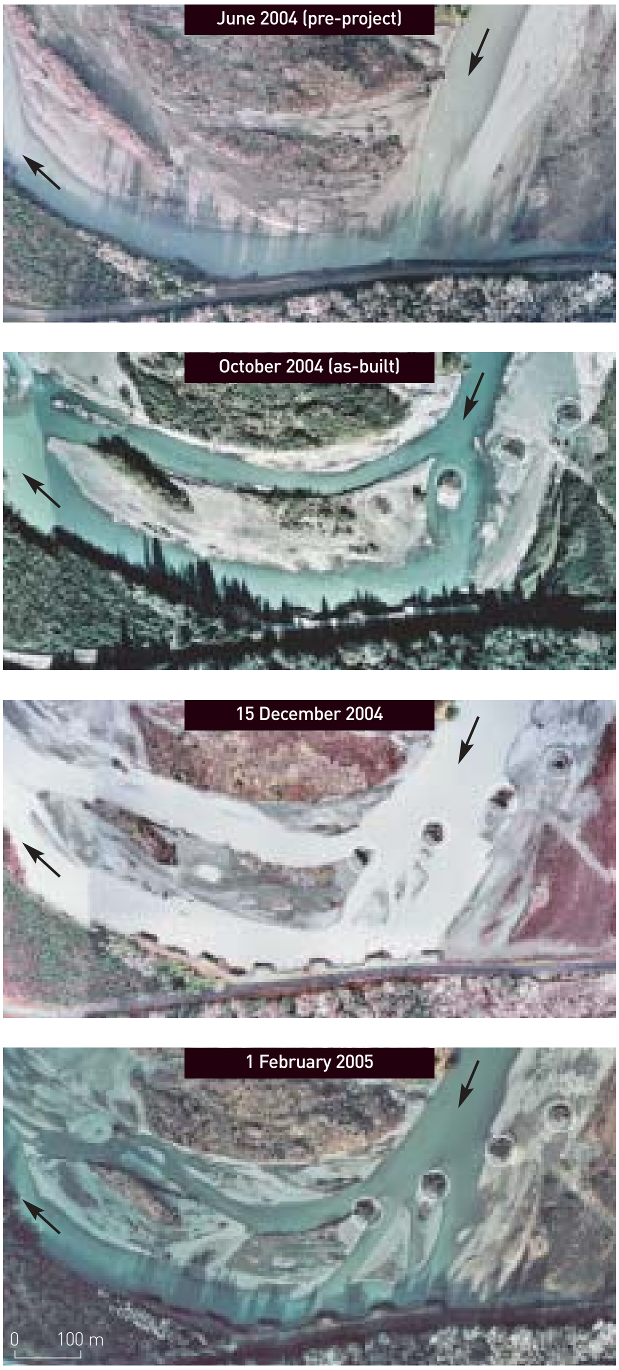
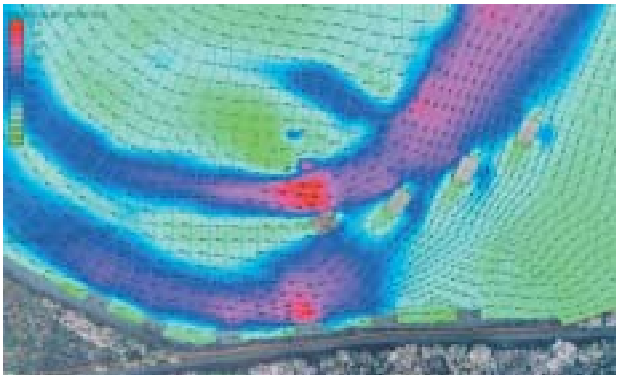


Figure 28 (above). Observed response of Hoh River project site to ELJ emplacement. Scale applies to all images. Courtesy T. Abbe.

Figure 27b.



viability of the proposed solution. In this case the model predictions appear to have predicted the actual stream response reasonably well (Figure 28).

7.4.2 Scour in sand-bed creeks — insights from the Granite Creeks sand-bed pool scour experiments

As mentioned previously, accurately predicting scour in sand-bed streams is fraught with difficulty. The problem is best illustrated with some results from experimental work carried by Nick Marsh, Dan Borg, Ian Rutherfurd, Mike Stewardson and others in the Granite Creeks (northern Victoria) and the lower Snowy River (east Gippsland, Victoria). The following are summaries and excerpts from the various reports of this work (see Marsh et al. 2001, Borg et al. 2004).

The Granite Creeks experiment

A study was conducted into the potential for creating persistent scour holes for their habitat value in a series of sand-bed streams in the Granite Creeks System of north-east Victoria. Large volumes of sand have been eroded from the granitic upper reaches of the system, infilling pools and burying woody debris in a classical sand slug stream (Davis & Finlayson 2000). As is typical of sand slugged streams, this system had been transformed from a physically diverse, heterogeneous environment, to a flat, homogeneous sand bed.

The experiment aimed to create pool habitat using a simple model of an elevated cross spanning log. The structures consisted of river red gum sleepers bolted together (cross section 200 mm x 200 mm) elevated just above the average height of the streambed, placed perpendicularly to flow and spanning the full width of the stream (Figure 29, A and B). The structures were keyed into the banks using steel-pickets. Flume studies suggested these structures should produce a relatively large amount of scour (Beschta 1983, Marsh et al. 2001) (Figure 30), and the maximum scour depth could be predicted for a given flow for this type of structure (Marsh 2001).

The results of the experiment showed that the model significantly over-predicted the resultant scour pool dimensions, which is not to say that this extent of scour did not occur during peak discharge. Repeat observations revealed that pools generally formed during a flood, and subsequently infilled during the waning stage of the hydrograph. In this mobile sand bed stream it was found that snapshot sampling of flow and scour depths failed to record the maximum scour depth (and pool dimension achieved). This highlighted two things: firstly that this strategy was ineffective at producing persistent pool habitat; and, secondly, that to record maximum scour depth required real time monitoring.

Figure 29. Artificial habitat structures in the Granite Creeks. (Left) One structure reach. (Right) Four structure reach.

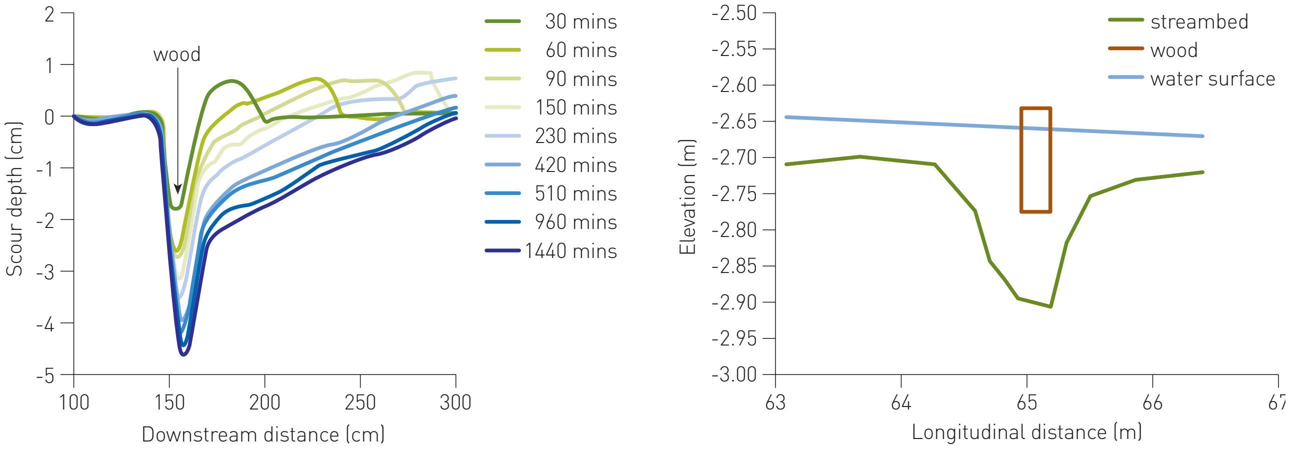


Figure 30. Anticipated and observed scour pool geometries. (Left) Laboratory study of temporal evolution of a scour pool (Marsh 2001). Note the exaggeration of the vertical axis. Position of the structure is marked with an arrow. (Right) Scour pool geometry for a log structure in the Granite Creeks system. The artificial scour pool is shallower, and does not extend as far downstream, as the model would predict.

Real-time monitoring

To overcome the inadequacies of observing maximum scour depths based on snapshot surveys, Borg et al. (2004) developed a technique for continuously monitoring scour around log structures. The technique used buried pressure transducers to measure changes in pressure of above-lying sand, which was then related to a depth of sand. Continuous scour pool depth data from the Granite Creeks, and also from investigations of natural scour pools in the lower Snowy River, (eastern Victoria), provided further evidence that the flow-scour relationship is not as simple as initially anticipated.

Data from a natural scour pool in the lower Snowy River indicates both scour and infilling can occur following flow events, in addition to infilling as flow levels recede (Figure 31). As the stage rises 0.2 m, the pool progressively infills by 0.9 m (late January). Following this infilling, the pool then progressively scours following a 0.54 m increase in stage (early February). The pool then gradually infills throughout February and March as the stage recedes.

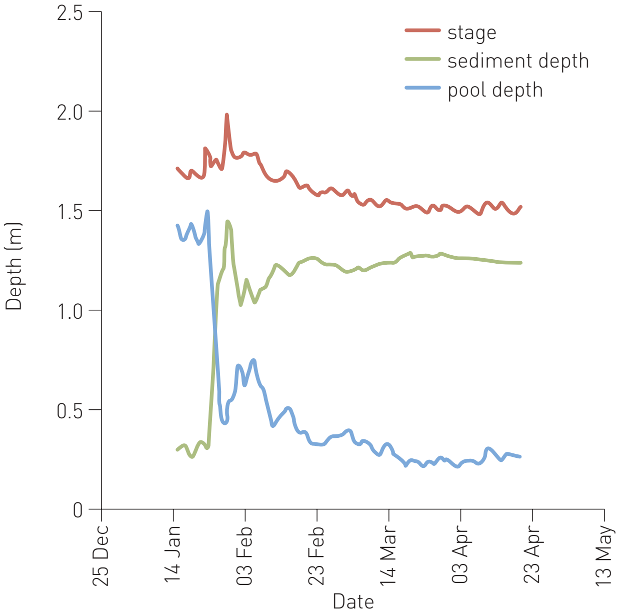


Figure 31. Scour pool data for a pool in the lower Snowy River. Sediment depth refers to the depth of sand above the sensor and pool depth is the difference between the stage and the sediment depth.

Study outcomes

The flow-scour relationship has been investigated in a number of flume studies (Beschta 1983, Cherry & Beschta 1989, Marsh et al. 2001), and these studies suggest that maximum scour can be predicted around instream wood structures (Marsh 2001, Wallerstein 2003). Results from the Granite Creeks revealed that the scour-flow relationship was not as simple as these flume modelling exercises predict. Long, deep meso-habitat scale features did not form, and the pools that did form were much more variable than expected (in space, and time). This variation was driven by the influence of geomorphic features and debris build up on local hydraulics, as well as a complex streamflow-scour relationship. These results are in marked contrast to the results obtained from gravel-bed streams in which large structures with appreciable blockage ratios were introduced, resulting in the formation of persistent meso-habitat features (Brooks et al. 2006, and Section 3.3). The question still remains as to whether large structures in sand bed systems can induce more persistent scour pools.

Scour pools in the Granite Creeks experiment were shown to infill and re-scour with different magnitude flows. Similar patterns of scour and deposition with time have been reported in other sand-bed stream restoration trials (Shields et al. 1995). Real-time monitoring in the Granite Creeks and lower Snowy River, provided further evidence that flow-scour model predictions were far too simplistic for predicting resultant scour pools, but were probably realistic for predicting maximum scour. It was demonstrated that pools sometimes scoured as flows increased, while other times they filled, and vice versa. These dynamics were further complicated by orientation of debris and reach geomorphic conditions.

It was concluded that such dynamics are not well represented by engineering theory. Much of the theory predicting scour around instream structure is concerned with peak flows, maximum scour depths, and structural failure. This type of information was found to be of little use to ecologists and in habitat applications. Rather, it was suggested that the full range of pool scour and fill needed to be known for key points in an organism's life cycle. It was thought that the probability of wood structures providing habitat at key times is increased with the installation of multiple structures.

7.5 Anchoring strategies

7.5.1 ELJs in a gravel-bed river

The standard approach for anchoring ELJs in gravel bed rivers is the excavation of the whole structure into the bed, and the partial burial of the structure once completed. The stability for the ELJ is predicated on the whole structure acting as a discrete cohesive entity, in which the frictional resistance afforded by the total mass of the structure exceeds the drag force imposed by the flow acting on the structure cross section exposed to the flow. A full explanation of the force balance analysis is outlined in Section 7.7.

For situations in which channel degradation has created conditions which are more inhospitable for wood stability than had naturally existed, such as incised channels or where large trees are no longer available, artificial means of stabilisation may be necessary. A quantitative assessment of site conditions and a force balance analysis can provide the means to evaluate the stability of a proposed wood placement and help determine where artificial ballast is appropriate (Abbe et al. 1997, D'Aoust & Millar 1999, 2000; Castro & Sampson 2000, Shields et al. 2000).

To cable or not to cable?

In gravel bed rivers, experience has shown that stable structures can be built without any cable at all. For piece of mind, however, some cable can be used to help secure the top layer of logs in place. When cable is used it should only be used to secure logs tightly to one another, or directly to rock ballast so that all the components act as one unified structure (D'Aoust & Millar 2000). It is only necessary to secure the top layer to the layer below as it would be virtually impossible to remove the upper two layers from the structure.

Cable anchoring (e.g. dead-man or duck-billed anchors) have been used in wood placements (Fischenich & Morrow 1999) in the USA, however they pose significant risks that should be considered. A flexible medium such as a cable will not prevent wood from moving up and down, or side to side, with fluctuating stage or turbulence. Movement of the wood will move the cable, and an oscillating or vibrating cable will tend to cut away the material within which it is set. The cable can become exposed to create an entanglement hazard, or simply fails and liberates the log that it was intended to secure. Stable wood structures can be designed without the use of any cable (Abbe et al. 1997, 2003; Brooks et al. 2001, Brooks et al. 2004).

Experience with duck-bill anchors has not found them to be particularly successful. An example where this approach was trialled was an experimental project conducted by the United States National Sediment Laboratory on Little Topashaw Creek (Shields et al. 2003). In this study, large wood was anchored by cable stretched over wood piles and attached to soil anchors. After several years and numerous floods, 21% of the structures anchored using soil anchors were destroyed because the soil anchors failed, compared with a 33% failure rate for those structures not anchored. The reason for failure of the soil anchors is not clear, although it is likely to be because the soil anchors are rated for a static load rather than dynamic loads. When attached to a flexible cable in fast flowing water, the cable would tend to vibrate, allowing the anchor to work its way to the surface in unconsolidated sediment (see http://ars.usda.gov/Research/docs.htm?docid=5533).

7.6 Stabilisation using piles

Piles are commonly used for securing structures in riverine and marine projects. Piles are very effective at supporting a vertically applied load, however, they can also be used for securing wood where the applied load is largely lateral. Design specifications for pile depths are somewhat arbitrary because in practice the size of piles and pile driving depth is often determined by the available machinery, piles and depth of sediment to bedrock. To determine the ideal depth that piles should be driven, the first step is to determine the maximum depth of scour predicted at the design discharge. Once this is established, the problem becomes one of determining the depth required to prevent rotational displacement of the pile under a given lateral load.

A freeware program called LLP99 created by Arnold Verruijt from the Delft University of Technology (http://geo.verruijt.net/), can then be used to determine the minimum pile depth required for a given lateral load. The lateral load can be determined for the whole structure as outlined in Section 7.7, and the load distributed between the number of piles used to secure the structure. A worked example of the LLP99 calculations is shown in Appendix B.

7.7 Structure stability analysis

7.7.1 Overview

The great efforts river engineers and others took to remove wood from rivers is testament to the natural stability of logs in rivers. Anecdotal and documentary evidence from people who have been involved with the removal of ‘snags’ attests to the fact that once logs become wholly or partially buried within the bed substrate they are extremely difficult to remove (e.g. Ruffner 1886, Russell 1909, McCall 1984). Nevertheless, logs can and do move within rivers, particularly in the period immediately following their recruitment to the channel. Methods are now well established for assessing the stability of logs and log jams (or multiple log structures) in alluvial river channels (D’Aoust & Millar 1999, 2000; Abbe 2000, Shields et al. 2000, Brauderick & Grant, 2000), and the designs included within this report incorporate aspects of each of these approaches in both the design conceptualisation and the stability analysis.

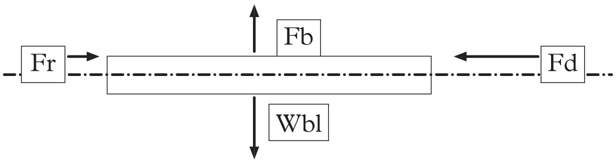


Figure 32. Conceptual model of forces applied to a cohesive structure within a river channel.

The key consideration when designing a stable log structure is to consider the circumstances under which the structure will fail within an alluvial river. It will fail when:

- buoyant force of the logs exceeds ballast weight,
- the net imposing forces on a structure exceed the net resisting forces,
- scour undercuts the structure and it disaggregates.

The forces acting on a log or structure within a channel can be represented by the simple conceptual model (Figure 32) where flow is from right to left.

Where Fb = buoyant force; Wbl = weight of ballast material; Fd = total drag force; Fr = friction between the total structure and the river bed. (Note: ballast material includes the weight of overburden associated with burial — assuming the material remains in place during a flood.) In this, no additional consideration is given to the anchoring effect of the key log root wads and, as such, it is a very conservative model, as the root wads add considerable frictional resistance. The anchoring effect of root wads is considered in the single log stability model.

According to model above, if Fb > Wbl the log is buoyant and therefore there is nothing to resist the imposing drag force. Even if a log has a large root wad, if the log is buoyant, then any additional frictional resistance offered by the root wad in the bed is negated. Thus, buoyancy is a major concern for log stability.

It is often assumed that Australian timbers are denser than water and, therefore, log buoyancy is not an issue in Australia rivers. This is a fallacy. Many Australian hardwoods, including species like river red gum (*Eucalyptus camaldulensis*) have dry densities that are less than the density of water (1000 kg m^{-3}) and can float when fully desiccated. From an engineering perspective, log structures must be designed for the worst case scenario. The worst case scenario from a log stability perspective is the situation, which commonly occurs in Australia, where a river dries out completely for an extended period of time, therefore, allowing the logs to fully desiccate. Under these circumstances wood density is represented by the dry density alone. If drought breaking floods then overtop the logs/structures before they have time to re-hydrate, also a common occurrence, the timber at this point may very well be buoyant. This is why log jams do (or did) form in many Australian rivers. Unless it can be assumed with complete certainty that a log structure is never going to dry out, the dry density should be used in any stability calculations.

It must also be remembered that many Australian timbers are subject to termite and borer attack, and timbers sourced for use in rehabilitation projects might be partially hollow or have significant portions of their total mass that is partially decayed and, as a result, of lower density than the reported density data for the species. Timber will also decay through time because of microbial attack and become less dense. For both these reasons, it is best to adopt a fairly conservative approach to structure design, and use conservative (i.e. low) density values in the stability analysis.

Of the three failure mechanisms identified above, the first and second mechanism are analysed as part of the overall force-balance analysis. The prediction of failure due to scour is the most difficult, due to the poorly developed theoretical basis of scour prediction around complex obstructions. The prediction of maximum scour depth is determined from one of the methods outlined in Section 6.2.3. It should be remembered, however, that because most of the structures are attached to the bank it is highly unlikely that they will scour uniformly around or under the entire structure. Indeed, it is most likely that deposition will occur within the distal (bank side) and downstream portions of the structure, while scour will occur upstream and around the streamward edge of the structure. Experience to date shows that it is highly unlikely that scour will occur under more than a third of the total width of the structure during a large flood that overtops a structure by several metres (pers obs, and Tim Abbe, pers comm.).

7.7.2 Computational procedure for force-balance analysis

Two types of analysis can be performed when assessing the stability of wood reintroduced into streams: multiple log structures or log jams, and single logs. Single log stability analysis is the most appropriate method for assessing the stability of single logs used as toe revetments (see Appendix B).

Multiple log stability assessment

In this assessment the log jam is treated as a single coherent entity and a force-balance analysis is performed according to the following assumptions:

- The imposing force is due to the drag force associated with the cross sectional area of the structure obstructing the flow. The worst case scenario is assumed in which the full height of the structure is exposed to the flow, including the buried portion, where the structure is completely scoured. The full structure width is generally not used, as it is assumed that the portion buried within the bank will remain so.
- In the simplest analysis, the resisting force is a result of the net downward force associated with the weight of the ballasting gravel that is used to backfill the structure less the timber buoyancy times the bed material friction angle. It is assumed that the structure is roughly rectangular in shape, one log wide and one log long of whatever width is deemed appropriate for the structure in question.
  The buoyant force is a function of the total timber volume contained within the structure. The volume of each log can be determined from the length and mean diameter of the logs (note: it is not necessary to use a taper model for calculating the wood volume for Australian hardwoods, as most tree boles of Australian eucalypts — the timbers used in these structures — have relatively little taper in the main part of the trunk).
- Structure height is a function of the number of layers of logs used, and is essentially the sum of the log diameters with some extra at the top for the protruding root wads. A fairly conservative ratio of root wad diameter has been assumed — this being 1.5 x mean log diameter.
- The volume of the structure — and hence the volume of gravel ballast in the force balance analysis, is equal to the width x length of the whole structure x the structure height, but not including the extra height associated with the protruding root wads as it is assumed that the structure will only be back filled to the level of the upper horizontal logs.
- To account for that fact that most structures are going to be located on concave banks, and that in most cases only the mean one dimensional velocity will be available, velocity is increased by a factor of 1.5 times to account for the higher velocities encountered in the outer portion of a channel bend (after Shields et al. 2000).

Computational procedure (after Shields et al. 2000, D'Aoust & Millar 1999, 2000)

In light of the general description of structure design provided, the following is a description of the computational procedure for undertaking the force balance analysis of a multiple log structure. The drag force on the structure is equal to:

EQUATION 1

$$F_D = 0.5V^2 A\rho C_D$$

where: F_D = drag force in N; V = approach velocity of the design discharge $m\ sec^{-1}$; A = the cross sectional area (m^2) of the structure projected into the flow; ρ = fluid density (1000 $kg\ m^{-3}$); C_D = drag coefficient, which is assumed to be 1.2 — which is at the upper end of a range of values quoted in Gippel et al. (1996), Shields et al. (2000), D'Aoust & Millar (1999).

The buoyant force associated with the total volume of wood in the structure is represented by:

EQUATION 2

$$F_B = (\sum{}^n K)\rho g(1-S_L)$$

where F_B is the buoyant force N; K is the total volume of n logs; ρ = fluid density (1000 $kg\ m^{-3}$); g is gravitational acceleration (9.81 m $\sec^{-2}$); S_L is the dry density of the logs (g cm^{-3}). This relationship doesn't account for the volume of wood contained within the root wad portion of the tree, as this is assumed to be neutrally buoyant due to the sediment contained within the roots — a conservative assumption as the root wad is most likely to have a net negative density.

The volume of each log (K) can be calculated simply according to:

EQUATION 3

$$K = \pi l \left(\frac{d}{2}\right)^2$$

where l is the log length (m) and d is the diameter of the log measured in the log centre. This relationship assumes that log taper is negligible, which is a reasonable assumption for the majority of Australian eucalypt tree boles.

The overall immersed weight of the ballast material within the structure is a function of the total structure volume less the volume of wood. It can be calculated by:

EQUATION 4

$$W_{BL} = \rho g\,(S_S - 1)\,(\psi - (\sum{}^n K))$$

where W_{BL} is the ballast weight (N); ψ is the structure volume (m^3); and S_S is the specific gravity of the gravel ($g\ cm^{-3}$) — which in this case is conservatively assumed to equal 2.0 ($g\ cm^{-3}$) to account for the void space between the clasts.

The effect of friction between the total structure and the river bed is a significant component of the force resisting structure movement. The critical frictional force to initiate sliding of the whole structure can be estimated by:

EQUATION 5

$$F_{FS} = (W_{BL} - F_B)\tan\phi$$

where ϕ is the friction angle of coarse gravel and is estimated to be 40° (after D'Aoust & Millar 2000).

To determine whether a structure will be stable two conditions must be met.

First, the structure must be shown to have a net negative buoyancy, and this can be assessed using a factor of safety analysis. A factor of safety (FS) is defined as the ratio of resisting forces to imposing forces. Hence values of FS > 1 indicate the structure will be stable, while values < 1 indicate the structure may fail. The FS with respect to buoyancy can be represented by:

EQUATION 6

$$FS_B = \frac{W_{BL}}{F_B}$$

If $FS_B > 1$ then the factor of safety with respect to sliding can be represented by:

EQUATION 7

$$FS_S = \frac{F_{FS}}{F_D}$$

Permeable, non-embedded structures

The forces acting on a non-embedded structure are the same as those acting on the impermeable ELJ, except in this case the analysis aims to determine the mass of ballast required to be attached to the structure (i.e. as blocks of rock or concrete), for a given factor of safety, under given design discharge conditions. For the purposes of the analysis a worst case scenario is assumed whereby the upstream side of the structure becomes clogged with transported debris, making the structure impermeable. As with the previous analysis, the drag force Fd imposed on the structure is simply a function of the structure cross sectional area (equation 1). Fb is calculated in the same way (equation 2) and W_{bl} can be calculated using an iterative procedure to determine the F_{FS} for a desired FS_S. The analysis assumes that the ballast is firmly attached to the structure and that they are a coherent unit. Structure failure (i.e. sliding failure), requires the movement of the combined mass of all ballast blocks plus the structure.

Single log force/balance analysis

The forces acting on a single log with a root wad, where the log is perpendicular to flow with root wad facing upstream, can be calculated as follows (after D'Aoust & Millar 2000).

Assuming that the root wad portion of the log can be represented by the volume of a cone, the buoyant force acting on this log can be represented by:

EQUATION 8

$$F_{BL} = \left(\frac{\pi D_L{}^2 D}{4} + 0.33\pi\,\frac{D_{RW}{}^2 L_{RW}}{4}\,(1-\rho)\right)\rho g\,(1-S_L)$$

where F_{BL} is the buoyant force on the single log (N); D_L is the diameter of the log measured at the centre (m); L is the log length (m); D_{RW} is the average diameter of the root wad (m); L_{RW} is the length of the root wad (m) and S_L is the specific gravity of the log ($g\ cm^{-3}$); ρ is the porosity of the root wad.

Assuming the surface area of the root wad subject to drag is represented by a disk of diameter D_{RW}, the drag force acting on this disk can be written as:

EQUATION 9

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

where V is the average flow velocity (m $\sec^{-2}$), β is the angle of the rootwad with respect to the direction of flow in either the horizontal or vertical plane (assumed by default to be 90°); C_{DRW} is the drag coefficient for the root wad.

Single log partially buried

The buoyant force and the drag force on a single log can both be counteracted if the log and root wad is buried.

In the case where the root wad is partially buried the passive earth pressures exerted on an idealised root wad can be defined as:

EQUATION 10

$$P_P = K_P \frac{\gamma h^2}{2}$$

where P_P is the force per unit width of root wad; K_P is the coefficient of lateral passive earth pressures i.e.

EQUATION 11

$$KP = \frac{1 + \sin\phi}{1 - \sin\phi}$$

; γ is submerged weight of the substrate (Nm^{-3}), and h is the depth of the buried portion of the root wad (m), ϕ = substrate friction angle.

If it is assumed that the end area of the root wad is embedded in the substrate and subdivided into a number of wedges of height h (which will be a series of arcs across the buried root disk < the maximum burial depth at the centre of the disk) and width w, the magnitude of the added resisting force is equal to:

EQUATION 12

$$F_P = \sum P_P w_{(i+1)}$$

In the situation where a log is rotated into the bed at an angle θ so that it is buried within the substrate — we can assume that the sediment burying the log is ballasting it to an extent equivalent to the weight of the overlying sediment (in fact it will be greater than this due to the friction between the particles). The ballast effect of log burial can be represented as:

EQUATION 13

$$W_{BL} = \frac{DL^2 Sin\,\theta\rho g(S_S - 1)}{2}$$

In a similar fashion to the situation with the log jam, the stability characteristics of a single buried log can be represented as a Factor of Safety:

EQUATION 14

$$F_S = \frac{W_{BL} - F_{BL}}{F_{DRW} - F_P}$$

Clearly a single log with a specific gravity <1 can only be regarded as being stable within a river channel if it is partially buried, or the log is of such dimensions that it is not fully inundated. For the purposes of this analysis we will assume that the log is fully submerged and that the log is at least partially buried, otherwise, the analysis is similar to the multi log jam analysis, with the exception that the effect of the root wad burial is included here. In reality this should also be included in the log jam assessment as the basal logs have root wads that are buried within the river bed.

EQUATION 15

$$FS_{SB} = \frac{W_{BL}}{F_B}$$

If $FS_B > 1$ meets then the factor of safety with respect to sliding can be represented by:

EQUATION 16

$$FS_{SS} = \frac{F_{FS} + F_P}{F_D}$$

7.7.3 Example of spreadsheet computation procedure

Explanatory notes

Refer to Section 7.7 for detailed description of the variables and computational processes. The following is an explanation of the spreadsheet shown in Table 5 (overleaf) that detail the specifications for each structure. Each spreadsheet is arranged in two parts:

1. The upper section details the log specifications for each structure — layer by layer. **These specifications, coupled with the reach plans and the generic structure models, provide all the necessary details to construct each structure.**
2. The lower section provides the details of the structure stability analysis.

   The upper section is relatively self explanatory, however, the following assumptions have been made:
   - root wad diameter (D_{RW}) has been assumed to equal 2.5 x average log diameter,
   - root wad length (L_{RW}) has been assumed to be 2 x average log diameter,
   - cumulative height does not include the height of the protruding root wad — it is the cumulative height of the log diameters — as such total structure height will be slightly higher than that reported here,
   - cumulative width is the sum of the root wad diameters of the key logs x 1.2 — to allow for some space between logs
   - the structure dimensions (i.e. cross sectional area of the structure projected into the flow) are ~0.75 x the cumulative width x total structure height. This assumes that around 25% of the structure is buried into the bank and is not exposed to the flow. Note that the proportion of the structure projected into the flow varies slightly from structure to structure, depending on local site conditions.

Computational procedures for each of the parameters presented in the lower part of the spreadsheet are detailed in Section 7.7.2.

Rack log numbers are calculated on the assumption that the logs have a mean diameter of 0.25 m and that to allow for a stack of logs to be arranged in a stable configuration, there will need to be more logs at the bottom, tapering up to the top of the stack. Rack log numbers are calculated according to the relationship:

no. of rack logs = (structure height (m)/0.6)^2 x 2

Table 5. Example of spreadsheet used for undertaking a force balance analysis.

Structure 9.2 — Budget cost									
Layer	Type	Log length (m)	Av. diameter (m)	No. of logs/layer	Log volume (m^3)	D_{RW}	L_{RW}	Cumulative height	Cumulative width
1	Key footer	9	0.35	1	0.87	0.875	0.7	0.35	
2	Key logs	10	0.55	4	9.50	1.375	1.1	0.90	6.6
3	Cross spaners	9	0.35	2	1.73	0.875	0.7	1.25	
4a	Longitudinal logs (row 1)	10	0.45	4	6.36	1.125	0.9	1.70	
4b	Longitudinal logs (row 2)	10	0.35	3	2.89	0.875	0.7	1.70	
5	Cross spaners	9	0.35	4	3.46	0.875	0.7	2.05	
6a	Longitudinal logs (row 1)	10	0.45	4	6.36	1.125	0.9	2.50	
6b	Longitudinal logs (row 2)	10	0.35	3	2.89	0.875	0.7	2.85	
7	Diagonal logs	10	0.35	2	1.92	0.875	0.7	2.85	
	Total logs			27	35.98	Total wood volume			
Structure dimensions		Width (m)	Height (m)	Length (m)	XS area (m^2)	Volume (m^3)			
		7	2.85	10	19.95	199.5			

Shields Method for determining stability of whole structure. (Design of Structure 9.2 is for an impermeable deflector structure protruding ~2 m above bed, i.e. structure excavated into bed)									
Backfill volume required (m^3)		148.7							
Immersed weight of backfill (Nm^{-3})		W_{BL}	−1458257	(= a specific gravity for gravel of 2.0)					
S_{wood}	Fb	Fb_{net}	Velocity*	Structure area (m^3)	C_D	Fd	Friction angle (degrees)	Friction angle (RADS)	F_{FS}
0.9	35301	-1422955	6.51	19.95	1.2	507290	40	0.7	-1194001
Factor of safety (buoyancy)		41.31							
Factor of safety (sliding)		2.35							
Rack log requirements		Structure height (m)	Mean rack log diameter (cm)	Number layers required	Stack taper allowance				
		2.85	0.25	11.40	45 (total rack logs)				
Rack log requirements (continued)		Log length (m)	Diameter (cm)	Diameter (m)	Log volume (m^3)	D_{RW}	L_{RW}		
		9	20	0.20	0.28	0.400	0.4		
					12.76	Total rack log volume			

* Velocity based on modelled mean discharge for a 20 year ARI event x 1.5 to account for higher velocity in channel thalweg (after Shields et al. 2000).