Tillegra Dam

Planning and Environmental Assessment

WORKING PAPER

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Fluvial Geomorphology

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Hunter Water

Tillegra Dam Planning and Environmental Assessment

Working Paper

Fluvial Geomorphology

Report to

Connell Wagner

by

Dr Christopher Gippel and Dr Brett Anderson

September 2008

FLUVIAL SYSTEMS
GEOMORPHOLOGY • CATCHMENT HYDROLOGY
RIVER REHABILITATION • ENVIRONMENTAL FLOWS

in association with

WATER TECHNOLOGY
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Tillegra Dam Planning and Environmental Assessment Working Paper: Fluvial Geomorphology

For Connell Wagner

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Fluvial Systems Pty Ltd in association with Water Technology Pty Ltd

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1 Introduction

This Working Paper was prepared as the fluvial geomorphology component of the Tillegra Dam Planning and Environmental Assessment (EA). The report addresses three main topics: (i) key features of the existing environment, (ii) potential environmental impacts, and (iii) mitigation and management measures. Various methodologies were used in this investigation and these are detailed in the first section of this paper.

There is no standard method of assessing the geomorphology of a river with respect to the potential impacts of a dam. Geomorphologists utilise a wide range of tools to undertake such analyses, and the tools of choice depend on the nature of the river, the nature of the development, and the availability of data and knowledge about the system. In the case of Tillegra Dam, the river system was relatively well known from previous investigations, and during the course of this investigation detailed information on the hydrology and hydraulics of the river was obtained or modelled. This allowed application of a relatively sophisticated approach to geomorphological modelling.

The geomorphological work examined suspended and bed sediment transport, making quantified estimates of transport rates for both current and ‘with dam’ scenarios. Other geomorphic processes were also investigated, as were the geomorphological forms. The river was examined in terms of relatively homogeneous reaches, with each reach being represented by a site that was investigated in the field and also modelled numerically. Where possible, potential impacts due to operation of the proposed Tillegra Dam were quantified.

As is common with every modelling exercise, the results are not without error and uncertainty. In this study, every effort was made to minimise error through careful measurement of input data and careful model parameter selection. The quality of the predictions is considered adequate to confidently inform the management decision making process.

This is intended as a stand alone report, but as with most environment flow-related geomorphological investigations, the results have implications for the ecology of the river. The work presented herein is complemented by the investigations of the ecology of the river.

2 Methodology

2.1 Overview

The first stage of the investigation was to review the available literature. As the Project would involve change to the sediment transport and flow regime of the Williams River downstream of the proposed Tillegra Dam, it was necessary to undertake fieldwork and modelling to characterise geomorphic processes under current conditions. The objective of this work was to develop models that relate geomorphic processes to discharge thresholds. These models also allowed assessment of potential impacts of the operation of the proposed Tillegra Dam on geomorphic processes.

Most of the geomorphological modelling was undertaken for two scenarios, one simulating current hydrology, and one a base case 'with dam' scenario intended primarily as a means of characterising the important geomorphological processes in the river. The 'with dam' hydrological scenario was never intended as a preferred flow regime. The process of determining an appropriate environmental flow regime followed, in logical sequence, the investigative geomorphological work (described here) and the investigative ecological work (described in a separate report). During the course of these investigations an alternative approach to delivery of the bulk water transfers was conceived. Instead of delivering the flows over long duration periods of relatively low magnitude flows (as for the base case 'with dam' scenario), the proposed alternative was to deliver the water as pulsed events of higher magnitude, with these pulses performing ecological functions. The geomorphological implications of his mode of delivery were evaluated in terms of bed material transport rate.

In reading this report, it is important to understand that the analysis is concerned with exploring the geomorphological processes associated with various hydrological scenarios as a means of providing input to the wider process of developing an environmental flow regime, not as the final evaluation of a preferred flow regime.

The land within the proposed storage was also characterised, with a focus on the area that would become the shoreline, in order to assess the impact of the Tillegra Dam on erosion and exposure of the shoreline.

2.2 Literature review

The search of literature included reports, and published papers. There was no need to source old material relating to historical river channel changes and river works as this had already been thoroughly reviewed by Andrew Brooks and Wayne Erskine, and their co-workers, as part of their independent research efforts. This work is published in reports and peer reviewed journal articles, which are cited here.

2.3 Hydrology: mean daily discharge series'

Two modelled mean daily flow time series were available for the period spanning 1 January 1931 to 31 December 2007. Hunter Water Australia (HWA) undertook the hydrological modelling underpinning both time series. The first flow series, based on gauged historical flows, was developed to represent the 'current' hydrology of the river system. The second series was described as the '90/30 with flushing events and constant run-of-river transfers' case and for the purpose of the geomorphic analysis represents the 'with dam' hydrology. These two flow scenarios were used as the basis for predicting the likely geomorphic impacts of the proposed dam at Tillegra on downstream sites along the Williams River. Note that the 'with dam' scenario was never intended as a recommended operating regime, but as a means of generating information on the likely environmental responses to an initial flow regime, or base case. This information was then in turn used to assist the process of developing an environmental flow regime.

The current flow series' were based on historical gauged flows at Tillegra gauge, modelled flows at downstream of Chichester Dam assuming the current environmental flow rules, and gauged flows at Glen Martin (required infill of two percent of values). The HWA supplied Tillegra historical series differed from the Tillegra gauged flow series, with some of the peaks in the HWA supplied series being higher than in the gauged series. Only six percent of values differed by 1 ML/d or more, and three percent differed by 100 ML/d or more. Although overall these series were not very different, the difference was sufficient that flood frequency analysis produced a different result for the two series. In this report, the historical Tillegra flows were characterised on the basis of the HWA supplied series.

The 'with dam' flow scenario assumed the proposed Tillegra Dam was in place and operated according to the following transparency/translucency rules:

1. For inflows up to the 90th percentile exceedence discharge at Tillegra (ie 7.4 ML/d), release 100 percent of inflow;

2. For inflows exceeding 7.4 ML/d, and less than and including the 30th percentile exceedence discharge at Tillegra (ie 100 ML/d), release 60 percent of inflow; and

3. Release a flushing flow of 2,000 ML for one day if average 3 month flow at Glen Martin drops below a set threshold in summer. These thresholds were set to achieve 30 events over the 77 year time series.

These rules were intended to maintain a variable minimum flow in the river, and also to allow a fresh event to pass in summer if a protracted period of low flow occurred. This flushing flow was incorporated mainly to assist with maintenance of water quality in Seaham Weir pool and downstream of the weir, and also to ensure regular flushing of fines from the surface of riffle and pool habitats on the river bed between Tillegra and upstream of Seaham Weir pool. Flow transfers from Tillegra Dam to Seaham (for pumping to Balickera Canal) were made at rates ranging from 250 to 500 ML/d.

The process of full development of an environmental flow regime followed on from the work described in this report, and as well as utilising the information that was generated from analysis of these two regimes, it also considered a more comprehensive set of flow objectives (with these objectives emerging from, or being confirmed by, the geomorphological and ecological investigative work). During the course of the work undertaken for this geomorphological investigation an alternative to the 250 – 500 ML/d flow transfer rate was suggested. This involved pulsing flows over a number of events with peaks of the order of 1,500 ML/d and event durations of 10 - 12 days. The geomorphological implications of this pattern of flow transfer were evaluated in terms of modelled bedload transport rates.

Daily time series were provided by HWA for Tillegra, Chichester Dam outflows and Glen Martin. A fourth time series was computed for downstream of Chichester River junction by adding the Tillegra flows to the Chichester Dam outflow series. The field inspection sites were not necessarily located right at the gauges, although sites were located very close to the Tillegra and Glen Martin gauges and also just downstream of the Chichester River junction. The three other field sites were associated with the most proximal discharge time series (Table 1).

Table 1.
Location of field sites with respect to proposed dam and sites where discharge series' were available.

Location/Site	Chainage from Tillegra Bridge (river kilometres)	Applicable discharge series
Tillegra Gauge	0.0	
S7	0.2	Tillegra
S8	4.2	Tillegra
Chichester junction	5.0	
S9	8.5	Downstream of Chichester
S10	12.9	Downstream of Chichester
S11	39.0	Glen Martin
S12	58.1	Glen Martin
Glen Martin Gauge	58.1	

2.4 Hydraulic modelling

The geomorphology investigation utilised hydraulic modelling results supplied by Connell Wagner. The hydraulic models (HEC-RAS) developed for each field site downstream of the proposed Tillegra Dam (ie S9, S8, S9, S10, S11 and S12) predicted water level, water surface level, energy slope, mean shear stress, and mean velocity for a range of discharges.

2.5 Coarse sediment transport

2.5.1 *Concepts and theory*

Maintenance of downstream coarse sediment (bedload) transport is important in rivers because:

- a supply of sediment is required at any point to replace that mobilised in events, otherwise the channel may scour and erode, leading to structural changes that may impact the biota; and
- a supply of sediment is required to build in-stream geomorphic features such as benches and bars that may be habitat for biota.

Bedload transport in a gravel and cobble bed river such as the Williams is usually conceptualised as a threshold process; that is, the rate of transport is considered to be very low up to a certain critical streamflow, and beyond this streamflow transport increases at a faster-than-linear rate (Ferguson 2005). There are generally two approaches for defining the threshold; either mean shear stress (τ) is used or, following Bagnold (1977; 1980), unit stream power (ω).

The mean shear stress, averaged across the channel in uniform flow, depends on the depth-slope product given by duBoys Equation (1) (Elliott & Hammack 2000):

$$\tau = \rho g R S \tag{1}$$

By comparison, Bagnold (1980) calculated the mean value of stream power per unit bed area as (2):

$$\omega = \rho g Q S / w = \tau U \tag{2}$$

where: ρ [1000 kg/m^3] is the density of water; g [9.8 m/s^2] is gravitational acceleration; R [m] is hydraulic radius; S [m/m] is the energy slope usually approximated as the stream bed slope; Q [m^3/sec] is the total discharge through the cross-section; w [m] is the width of flow in the channel; and U [m/s] is the mean channel velocity.

Recent work by Ferguson (2005) convincingly argues in favour of adopting stream power rather than shear stress for the estimation of bed load transport rates in coarse-bedded rivers. Ferguson's work addresses three key deficiencies of Bagnold's (1980) critical stream power formulation; incorporating each of the following transport characteristics:

- it differentiates between the grain size transported by the flow (D_i) and the (usually higher) grain size which characterises bed roughness (D_b);
- it recognises a critical depth for motion of each grain size, rather than a constant depth value such as the bankfull depth; and
- it differentiates between the critical Shields stress applicable to grains of different sizes and accounts for the strong influence of hiding effect caused by the protrusion of larger grains and clasts.

In the work presented herein, threshold stream power for the *i*'th size class is estimated by application of (3), the simplified relationship proposed by Ferguson (2005):

$$\omega_{ci} = 0.104 \frac{D_b^{1.5}}{S^{0.17}} \left(\frac{D_i}{D_b} \right)^{0.67} \tag{3}$$

where grain size is measured in millimetres.

Assumptions include:

- adopt the Manning-Strickler resistance law with the coefficient (*a*) taking a value of 8.2 (in order to define the velocity profile and hence shear stress);
- a hiding function coefficient of 0.6;
- the submerged specific gravity of sediment is 1.65; and,
- the dimensionless Shields stress for entrainment of median-sized substrate is 0.045.

These assumptions were used to simplify Ferguson's (2005) general equation. The calculated critical stream power values were then converted into an estimate of actual bed load transport. Recent work in coarse bed rivers by Ashiq et al (2006) suggest the efficacy of (4):

$$q_{bed\_i} / P_{\mathrm{bm}i} = \alpha \left(q - q_{ci} \right) \tag{4}$$

where: q_{bed_i} [m^2/sec] is the unit bed load discharge for the i'th size fraction; $P_{\mathrm{bm}i}$ is the proportion of bed load of a given size fraction assumed to be equal to the proportion of that size fraction found in a bulk bed material sample; q [m^2/sec] is unit water discharge; and q_{ci} [m^2/sec] is the critical unit water discharge required to entrain the i'th size fraction. Ashiq et al (2006) developed (5) based on measured data to define the coefficient of proportionality (α) as a function of stream slope and the sediment relative size ratio (σ_g):

$$\alpha = \left[0.872 - 3.27S - 0.286\sigma_g \right] / 10^8 \tag{5}$$

where

$$\sigma_g = \sqrt{D_{84} / D_{16}} \tag{6}$$

This series of equations were applied at sites S7 to S12 by determining the critical stream power for the set of substrate sizes measured at each site during the field inspection (Section 2.5.3). Equation (2) was rearranged to determine the critical unit water discharge for each size fraction and, for a given daily discharge, (4) was evaluated to determine the unit bed load transport rate [tonne/day], based on the channel width for that discharge. This series of calculations was repeated for values of discharge ranging from zero flow up to the maximum flow in the hydrologic series ('current' and 'with dam'); effectively producing a rating curve of bedload transport versus discharge. Finally, this rating curve was used to predict bedload transport for each day in each of the discharge record at each cross-section identified as a riffle in the HEC-RAS models (excluding downstream boundaries).

A customised numerical algorithm was used to implement these calculations and to post-process the results (presented later in Section 2.13)

2.5.2 *Hydraulic data employed*

One-dimensional hydraulic models of six study sites (S7 to S12) downstream of the proposed Tillegra Dam site on the Williams River were developed by Connell-Wagner as part of this project. The models were constructed using HEC-RAS software (www.hec.usace.army.mil/software/hec-ras/) with reaches defined by typically five to seven cross-sections. Simulation output was provided at a series of nine discrete flow levels ranging from low to high flow. At each discharge, the relevant data for the analysis of bedload transport included:

- the energy slope (S), required to evaluate (3) and (5); and
- the top width of the flow, required to determine unit discharge in (4).

Intermediate values were determined by linear interpolation between the supplied values and to zero. In order to model coarse sediment transport and suspended sediment fluxes, extrapolation beyond the provided data was necessary due to the presence of larger discharge values in the daily time series' of stream flow. The extrapolation method chosen was to hold the hydraulic characteristics (in particular the energy slope and unit discharge) constant for all discharges greater than the

40 year ARI flow. Specifically, this had the effect of causing coarse sediment transport for large events (>40 year ARI) to scale as a simple proportion of the discharge magnitude.

2.5.3 *Field characterisation of bed material size*

The distribution of the particle size of bed material in a river sampling site varies spatially, according to the variable hydraulics at this scale (10 – 1,000 metres). In rivers with infrequent disturbance of the bed, or regular but only partial disturbance of the bed, the particle size can also vary with depth of the bed (generally being coarser, or armoured on the surface). To characterise the spatial distribution of particle size over an entire site would require an unrealistically large sampling effort. An alternative is to sample sites consistently at points that have certain identifiable hydraulic/geomorphic characteristics. Gravel bed rivers like the Williams tend to have pool-riffle morphology. Riffle crests are easily identified, are accessible by wading at low flows, and generally represent the coarser end of the size distribution. Pools can have similar particle size as riffles, as shear stresses in pools can be even higher than on riffles under certain (high) stages of the hydrograph (Keller 1971, Richards 1982, p. 186). However, the bed surfaces of pools are also the most likely areas where any sand-sized material in suspension will deposit on the recession limbs of floods. Even during baseflows, the shear stresses on riffle crests may be sufficient to mobilise fine material, leaving a particle size distribution that reflects the shear stress that prevails during flood events. Riffle morphology generally only changes its gross form during flood events that have the energy to mobilise a significant percentage of the coarse material. For these reasons, the riffle crest surface is a conventional sampling location for particle size determination where the threshold of motion is of interest.

Pebble count is the most popular approach to sampling wadable gravel/cobble bed streams (Gordon et al 2004 p. 105). The original method was described by Wolman (1954). This involves walking along a transect and stopping at a consistent interval and measuring the particle located closest to the toe of one's boot. It is recommended to sample at least 100 particles. Particles must be selected at random. The beds of dominantly coarse bedded streams may contain a minor fine fraction that is not well sampled by this technique, either because the material is located under the surface, or within hollows that one's toe does not often naturally fall on. This is not a major limitation of the method if the main interest is to determine the threshold of motion for gross movement of the riffle material (as is the case for this investigation).

On the Williams River, riffle crests were sampled using the Wolman pebble count technique at sites S7, S8, S9, S10, S11 and a site 6.7 km upstream of S12. The latter site was used to represent S12 due to access difficulties at S12. Having previously inspected S12, the riffles at the upstream site were judged to be geomorphologically similar to those at S12. At each site a minimum of 100 particles were measured along the B-axis, as this most closely simulates the results obtained by sieving (Gordon et al 2004 p. 117). The sampling covered the entire width of each riffle. The intention was to sample only the surface material. The particle size distributions were described statistically in terms of D_{16}, D_{50} and D_{84}, which are standard percentile classes (Gordon et al 2004 p. 121).

2.6 Suspended sediment transport

2.6.1 *Methodology and data availability*

Water quality data are usually only collected at infrequent intervals, for limited periods of time, and at a limited number of sampling sites. This is the case for the Williams River at Tillegra, with the available records commencing in September 1987. The data of interest were total suspended solids concentration (*TSS*) [mg/L], measured gravimetrically in a laboratory from water samples. Such data cannot be used directly to calculate sediment loads, because the data are not available as a continuous time series. It is widely accepted that in most systems, the majority of sediment is transported during the few largest events of the year, but sampling regimes are rarely designed to preferentially sample these events. Rather, water quality data records are usually dominated by baseflow samples. This is the case for the Williams River with most measurements taken at flows of less than 100 ML/day (Figure 1).

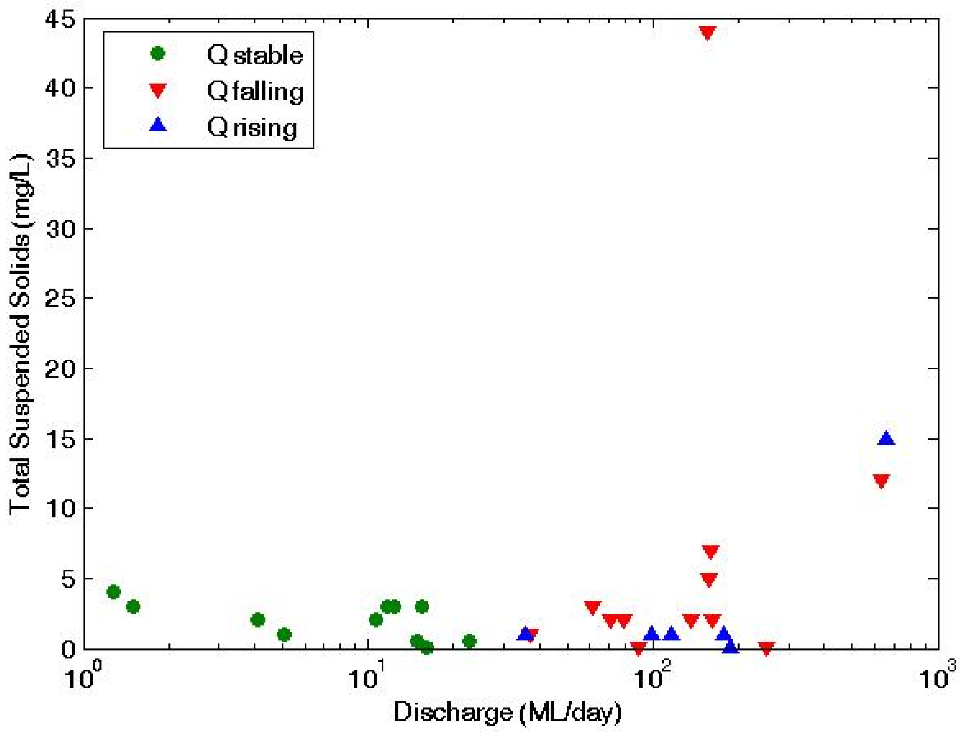

Figure 1. Plot of the available suspended solids data for the Williams River at the Tillegra Dam site versus discharge measured on the day.

Over the period 1995 to 1998 detailed water quality samples were collected at Tillegra during storm events in an attempt to characterise the true peak loads of: Total Phosphorus (*TP*); Total Suspended Solids (*TSS* via turbidity); Total Kjehldahl Nitrogen (*TKN*); and Nitrates (*NOx*). From a geomorphic perspective the turbidity data were of most interest.

2.6.2 *Turbidity storm event time series'*

The 1995 – 1998 event-based data were plotted, and a series of thirteen events were identified as having a peak discharge greater than 1,000 ML/day (ie a flow that represented a significant rise above baseflow). For each event the associated turbidity peak was identified (Table 2). In addition, the time between the turbidity peak and the discharge peak was estimated (termed the 'lead time'). The results indicated that turbidity tended to lead peak discharge by around 1 day (see column 4 in Table 2). Turbidity also varied markedly within a day, suggesting that multiple measurements would be required each day in order to properly characterise temporal patterns of turbidity. This meant that the peak turbidity values in the data may have underestimated the true peak values.

Table 2.
Selected peak event discharges and the associated turbidity peak and lead time recorded at the Tillegra Dam site between 1995 and 1998.

Date	Peak Discharge (ML/day)	Peak Turbidity (NTU)	Lead (Lag) (days)
18 June 1995	1,196	48	0.5
7 December 1995	1,342	335	1.5
11 December 1995	1,634	192	0.3
7 January 1996	4,877	73	1.0
12 January 1996	2,056	40	1.0
24 January 1996	1,160	65	0.0
9 March 1996	1,153	49	0.6
26 February 1997	1,227	222	1.0
7 March 1997	3,640	117	1.0
18 June 1997	4,109	66	1.8
5 July 1997	1,584	36	1.0
6 May 1998	3,159	41	0.0
20 May 1998	10,553	188	1.5

The relationship between peak discharge and turbidity was not simple, with high turbidity peaks evident in both small and large events (Figure 2). A further factor that complicated the relationship was that for closely spaced events (eg Figure 3), it was not uncommon for the turbidity peak of the second event to be lower than the first, even if the second discharge peak was higher (this is known as sediment exhaustion). In addition, while turbidity often followed the discharge trace closely through storms, there were also peaks between storms that were not caused by elevated discharge (perhaps by stock impacts or conducting in-channel works – see for example 7 February 1996 in Figure 4).

2.6.3 *Turbidity and suspended solids relationship*

While determination of suspended solids concentration (*TSS*) requires considerable laboratory time and expense, turbidity (*T*) is relatively easy to measure. Thus, water quality data records usually contain more values of turbidity than suspended solids concentration. Gippel (1995) found that in many cases, the correlation between *T* and *TSS* was sufficiently strong that turbidity could be used as a surrogate for suspended solids concentration. Using turbidity as a surrogate enables the suspended solids concentration record to be expanded.

Despite the paucity of overlapping turbidity and total suspended solids data, a relationship was found between these two parameters (Figure 5). It was evident that despite the high value of Pearson's correlation coefficient (R = 0.954) the relationship between these variables was heavily influenced by the three largest values. It is also noted that the highest value of around 50 NTU was considerably lower than the highest recorded values (200 – 300 NTU). Although the data were less than ideal, they were the only available data on which to base an estimate of suspended sediment load.

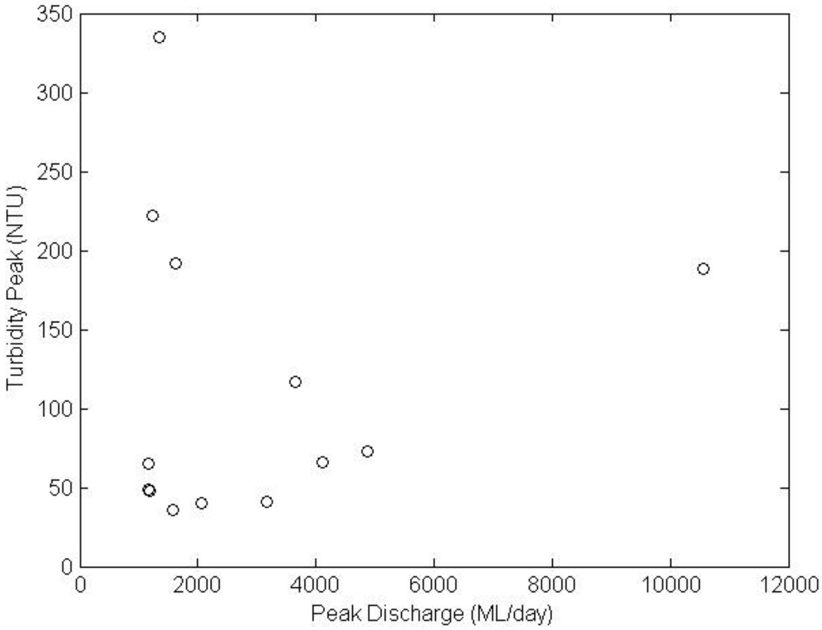

Figure 2. Relationship between peak discharge and the associated turbidity peak at Tillegra Dam for selected flow events between 1995 and 1998.

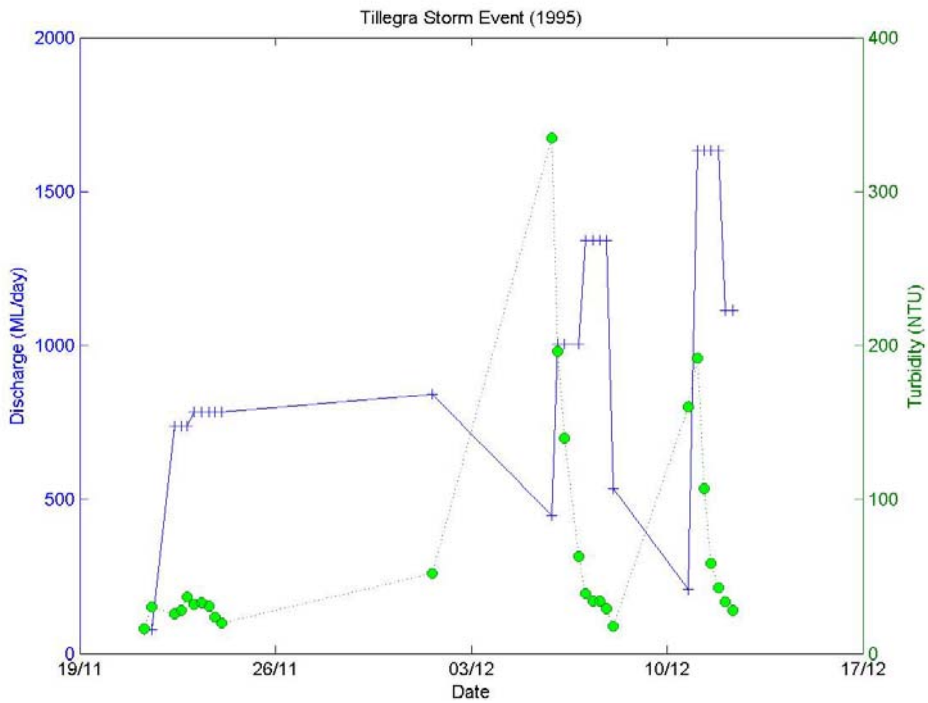

Figure 3. Detailed turbidity sampling during storm events at the Tillegra Dam site in late 1995. The blue line shows discharge (scale on left axis) while turbidity is shown by the green dots (scale on right axis).

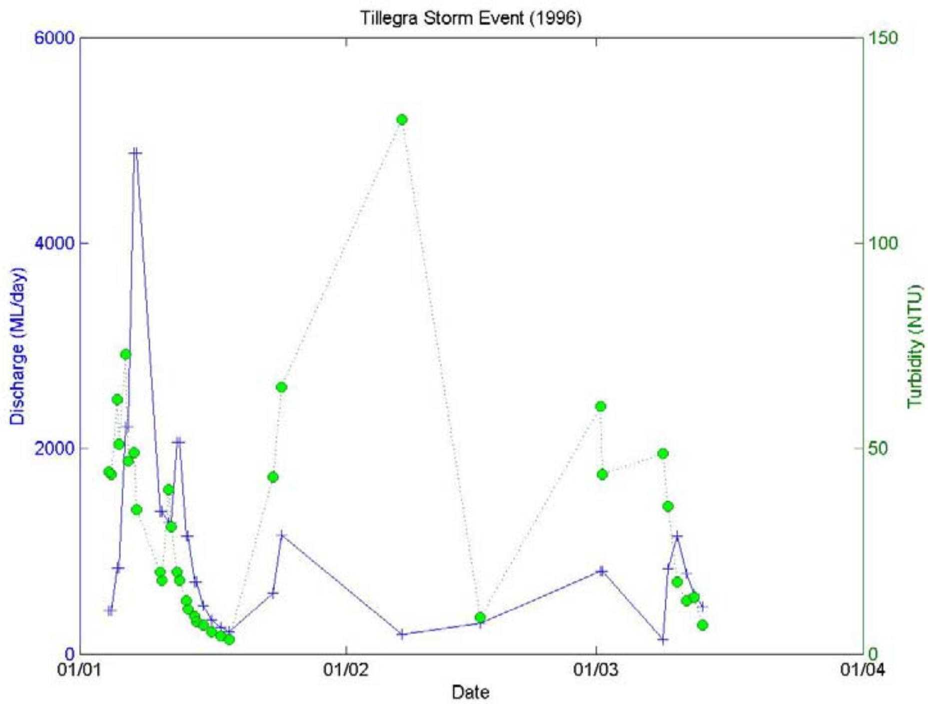

Figure 4. Detailed turbidity sampling during storm events at the Tillegra Dam site in late 1996. The blue line shows discharge (scale on left axis) while turbidity is shown by the green dots (scale on right axis).

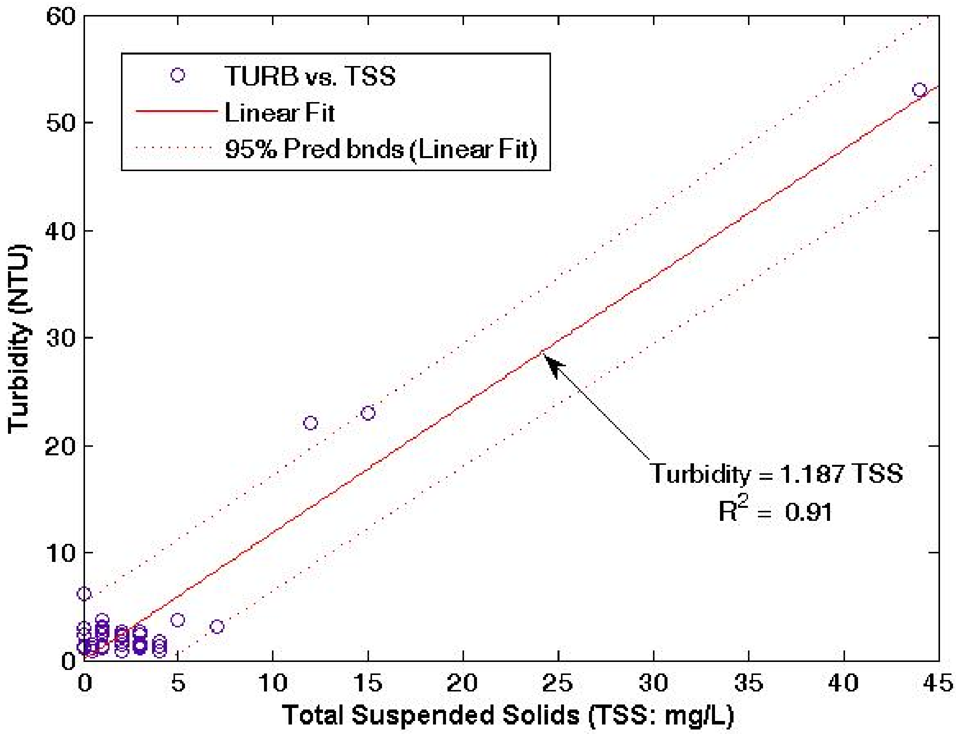

Figure 5. Relationship between 40 coincident measurements of total suspended solids and turbidity.

2.6.4 *Discharge and suspended solids relationship*

Having established a relationship between turbidity and suspended solids concentration, the next step in the method was to derive a 'sediment rating curve'. This is the relation between suspended solids concentration and discharge at the time of sampling (or mean daily discharge on the day of sampling). In this instance a relationship between turbidity and discharge was required. Analysis of the larger set of turbidity measurements (264 values) provided a second linear correlation (Figure 6). Again, the data showed considerable scatter around the regression line, in particular some very high turbidity values at reasonably low discharges. For the purpose of this work, a relationship was required to enable an estimate of mean daily suspended solids concentration time series and hence to estimate the net annual suspended load in the Williams River. Combining the two correlation functions gives (7), where *Q* is discharge in ML/day and *TSS* is the suspended solids concentration (mg/L):

$$TSS = 0.015\ Q + 4.97 \qquad (7)$$

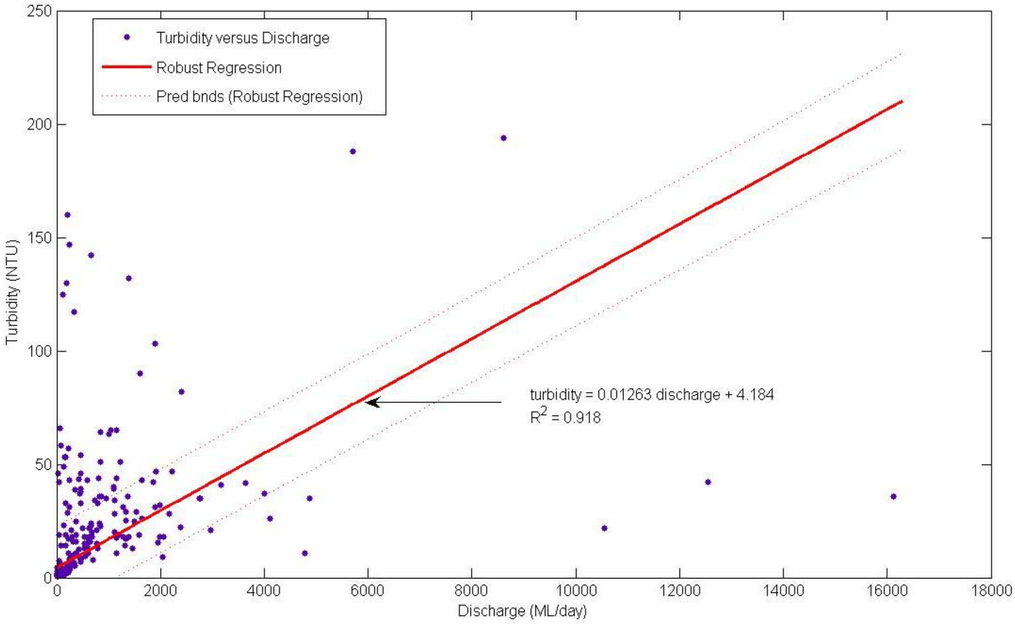

Figure 6. Relationship between 264 turbidity records and the discharge measured on the day.

2.6.5 Expected sediment trapping by Tillegra and Chichester Dams

Reservoirs and dams are efficient sinks of both suspended solids and bed load. In general 100 percent of the bed load that enters a dam is trapped, while a small proportion of the suspended load is conveyed downstream. The trapping efficiency of Tillegra Dam was estimated using the method adopted by Prosser et al (2001) for the Australian Land and Water Resources Audit. They employed (8), an enhanced version of the empirical Brune rule proposed by Heinemann (1981, in Prosser et al 2001) which expresses suspended sediment trap efficiency (*TE*, %) as a function of the storage volume of the reservoir (*C*) and the mean annual input discharge (*I*).

$$TE = \frac{119.6(C/I)}{0.012 + 1.02(C/I)} - 22.0 \quad (8)$$

The design capacity of Tillegra Dam is 450 GL (HWC 2006). An analysis of the daily modelled inflow sequence at Tillegra indicated that the mean annual discharge was 96 GL. Evaluating (8) with these numbers indicated that Tillegra Dam would trap approximately 95 percent of the suspended load.

An analysis of the daily modelled inflow sequence reported for Chichester Dam, the design capacity of which is 21.5 GL (http://www.hunterwater.com.au/281.aspx), indicated that the mean annual discharge was almost 120 GL. Evaluating (8) with these numbers indicated that Chichester Dam traps approximately 88 percent of the suspended load.

These trapping efficiencies were applied to the sediment transport modelling to reduce the *TSS* load in stream flows downstream of each dam. The approach used to predict *TSS* varied only with flow, hence sites with the same hydrology were predicted to convey identical *TSS* loads. Along the Williams River, three hydrological zones were recognised: a) downstream of Tillegra Dam site (S7, S8); b) downstream of the junction with the Chichester River (S9, S10); and c) in the vicinity of the Glen Martin gauge (S11, S12). The impact of trapping on the *TSS* load was estimated by applying a factor according to the trapping efficiency of either upstream dam (if present). A summary of the factors applied is shown in Table 3. The factor listed (*Tfactor*) was applied as per (9).

$$TSS = Tfactor \times (0.015\ Q + 4.97) \qquad (9)$$

Table 3.
***TSS* concentration reduction factors (*Tfactor*) to account for trapping by Chichester and Tillegra dams**

Sites	Current Flow Case	With Dam Flow Case
S7 and S8	1.0	0.05
S9 and S10	1.0 x Williams *TSS* 0.12 x Chichester *TSS*	0.05 x Williams *TSS* 0.12 x Chichester *TSS*
S11 and S12	0.74	0.40

Sites 7 and 8

Suspended load immediately downstream of the proposed dam site is unchanged in the current case (*Tfactor* = 1.0), but would be reduced by around 95 percent following construction of the dam (*Tfactor* = 0.05).

Sites 9 and 10

Flow in this reach is a combination of flow in the Williams River and the Chichester River. The *TSS* load in Chichester flows was estimated to be around 12 percent in both the 'current' and 'with dam' cases due to the presence of Chichester Dam (factor = 0.12). The *TSS* load from the Williams River would be expected to decline to 5 percent of the 'current' value following construction of the dam. Note also that at these sites the *TSS* load was computed independently for each upstream tributary branch and then summed to give the total load.

Sites 11 and 12

An analysis of mean annual flows suggested that the Williams River and Chichester Rivers (above their junction) contributed around 30 percent each of the flow at Glen Martin. Given these contributions, *TSS* was reduced in the current flow case to account for entrapment of sediment by Chichester Dam:

(12% x 0.3) + (100% x 0.7) = a factor of 0.74.

Under the 'with dam' flow case, 95 percent of the suspended load from the 30 percent of flow contributed by the Williams River would also be lost, hence the reduction is:

(12% x 0.3) + (5% x 0.3) + (100% x 0.4) = a factor of 0.40.

The *TSS* load was calculated using this set of factors and the results are reported later in Section 0. The trapping of suspended load by the dams was predicted to have a substantial impact on the total *TSS* loads carried through the system.

2.7 Geomorphic process discharge thresholds

The aim of fluvial geomorphology is to describe and analyse landform features created by flowing water, and to develop an understanding of the ways in which surface processes operate and control the development of these landforms. The geomorphology of rivers is mainly concerned with sediment erosion, transport and deposition.

The objective of managing the geomorphic aspects of a river is to maintain or rehabilitate channel forms and processes in order to assist achievement of certain ecological management objectives. In the case of the Williams River, it is assumed that the ecological objective is to maintain or improve the current ecological health. In this context, for the Williams River, the relevant geomorphic objectives are to maintain:

- substrate type, diversity and degree of mobility (habitat disturbance);
- presence and form of pools and riffles;
- channel shape and dimensions, including the presence of backwaters and undercut banks;
- presence of woody debris and riparian vegetation; and
- connectivity, described as the degree to which there are opportunities for biota, organic material and sediments to move both along the river and laterally to/from in-channel features such as benches and bars, and to/from floodplains and wetlands.

Geomorphic objectives are closely linked to those for riparian and aquatic vegetation, because of the role of vegetation in stabilising sediments. The geomorphic objectives are all connected to the processes of sediment mobilisation, transport and deposition. Riparian zone condition is relevant to achievement of geomorphological objectives. Bank stability is partly related to the integrity, coverage and structure of riparian vegetation.

It is well established that in-channel and riparian vegetation has a mediating influence on channel morphology, principally via the impact of plants on sediment dynamics (Ikeda & Izumi 1990; Marston et al 1995; Rutherfurd et al 1999; Trimble 1997; Zimmerman et al 1967). In general, the behaviour of vegetation is to colonise and exploit the fertility of the riparian zone and surfaces within the stream channel, behaviour that favours encroachment and channel narrowing (Hupp & Osterkamp 1996; Tabacchi et al 1998). However, the hydrologic regime holds encroachment in check, with periods of inundation and the destructive power of floods acting to inhibit growth or to clear the channel by force (Nakamura et al 2000). This dynamic balance can be adversely affected by regulated flow regimes (eg Nilsson & Svedmark 2002).

The health of the riparian zone vegetation is partly reliant on appropriate flows, but it is also dependent on non-flow related factors. Riparian zone vegetation can be affected by stock access and invasion by pest plant species. The riparian zone is also the source of large woody debris, which interacts with the hydraulics and sediment dynamics in the channel to create habitat for various organisms. Stock access to the riparian zone can also directly affect bank stability and water quality, through pugging, trampling and other physical disturbance.

A number of discharge thresholds related to channel dynamics were calculated. These were the discharge required for:

- coarse bed material mobilisation,
- fine bed surface material flushing,
- grass and shrub disturbance, and
- macrophyte disruption,

2.7.1 Coarse bed material mobilisation

As explained previously, the stream power method of Ferguson (2005) (3) was used to determine the critical stream power corresponding to measured substrate size indices. This critical stream power was related to discharge in the river through the HEC-RAS hydraulic models developed for each site.

2.7.2 Fine bed surface material flushing

Sediment-entrainment theories can be used to predict the mobilisation of unconsolidated surface deposits on the bed (silt- and sand-sized). These sediments might accumulate during long periods of low flow. Over-accumulation of this material is generally considered undesirable due to the risk of infilling of interstitial pore spaces in the gravel and cobble bed substrate. It is normally assumed that fine particles will be flushed out when the threshold of motion for some percentage of the particles is reached. One method of predicting when particles will become entrained in the flow is based on the Hjulström curves, which relate particle size to mean velocity required for erosion, deposition and transportation (Gordon et al 2004 p.192). The critical velocity [m/s] for initiation of sediment movement (for particles >1 mm diameter) is $V_c = 0.155 \sqrt{D}$, where D is the average particle diameter in millimetres. The Hjulström curve also predicts the limits for erosion of fine sands down to clay size sediment, and these values can be read from the curve (Gordon et al 2004, p. 192). The velocity near the bed is predicted by $V_b = 0.7\ V$, where V is the mean channel velocity (Gordon et al 2004, p. 193). The bed surface material will become unstable when $V_b > V_c$. Estimates of the discharge required to initiate movement of fine surface accumulations were made based on the basis of assuming that the material covered the size range coarse silt- to sand-sized. This size material is entrained at mean channel velocities greater than 0.5 m/s. This is a conservative estimate, as much of this surface material is flocs of organic/mineral/biological material that is lower in density than mineral material, and would be more easily entrained. Velocity was computed by the HEC-RAS models developed for each site. Velocity was assumed to be cross-section mean.

2.7.3 Grass and shrub disturbance

Fluvial scour depends on the erosion resistance offered by the substrates forming the wetted perimeter, with vegetation increasing erosion resistance substantially. A field study by Prosser & Slade (1994) of grasslands in southeastern Australia examined gully erosion. They reported that widespread gully erosion could be explained solely by degradation of valley floor vegetation. Using a high-discharge flume, Blackham (2006) identified two key mechanisms by which herbaceous vegetation reduces scour. Firstly the swards (plant stems above the ground surface) act as roughness elements, reducing the velocity, and hence the erosive potential, of overland flows. Secondly, shear stress is partitioned between soil particles and the root system, with the dense root mats of grass species absorbing the bulk of the shear (Blackham 2006).

Blackham's (2006) flume data confirmed the work of other investigators in showing that a critical shear stress in the range 80 – 200 N/m^2 is required to strip grass swards from stream beds. Blackham (2006) went on to demonstrate that hydraulic conditions (shear stress and duration) in small to medium sized streams are rarely sufficient to scour well-grassed surfaces. The minimum shear stress required to impact the least hardy of grasses (ie poorly established bunch grass) is 80 N/m^2 (Reid 1989; Hudson 1971) This was adopted as an indicative threshold of when grass-lined banks may start to be adversely affected by flow. Grasses that form matts and sods have a higher shear stress. Couch and kikuyi was observed on in-channel benches in the Williams River. For this type of vegetative cover a critical shear stress for disturbance of 150 N/m^2 was assumed. This is a very high threshold and it is recognised that the distribution of grass in the channel is likely to be influenced by other factors (such as inundation duration). This threshold simply provides one part of the picture, indicating the likelihood that a given flow has sufficient energy to remove strip or partially strip grass coverings.

Shrubs and small trees are present on the margins of the river, and it is ecologically desirable to occasionally check their growth to prevent them colonizing the channel. There are no published data available on which to base an index for removing shrubs and small trees, so here we assumed that the shear stress to remove grass would disturb shrubs, as shrubs are less flexible and present a greater drag on the flow (due to larger projected area). Countering this is the possibility of greater rooting strength. It is emphasised that the removal of vegetation can only be predicted as a probability, and that for any given event only a proportion of vegetation will be disturbed. Vegetation may remain undisturbed due to variations in flow velocities within a stream, the stability provided by the substrate in which plants grow, or the path taken by debris entrained by the flow. For this reason, this criterion is expressed in terms of 'checking the growth of' or 'disturbing' shrubby vegetation. It is not intended to represent the removal of all shrubby vegetation. Shear stress was computed by the HEC-RAS models developed for each site. Shear stress was assumed to be cross-section mean.

2.7.4 *Macrophyte disruption*

Emergent macrophytes commonly act as a hydraulic/geomorphic agent in stream channels. Shih & Rahi (1982) attributed intra-annual resistance (channel roughness) variations of around one order of magnitude to seasonal stem density changes. Similarly, Mierau & Trimble's (1988, in Kadlec & Knight 1996) measurements in Boney Marsh showed a four-fold increase in the annual average Manning's n, which was primarily attributed to the increases in stem density associated with the maturation of the marsh over the ten year study period. Cases of extreme resistance occur where the channel is choked, with Guscio et al (1965) reporting reductions in the design channel capacity of up to 97 percent. When a channel becomes extensively colonised by macrophytes, geomorphic processes are slowed by two inter-related factors: (i) greater coverage and binding of the bed and bank sediments, and (ii) reduction of shear stress on the bed and banks due to higher channel roughness.

Chemical and mechanical control methods are often deployed to prevent infestation of macrophytes, however natural hydrodynamic controls can obviate the need for such interventions (Duan et al 2002). Groeneveld & French (1995) found that colonisation by macrophytes could be prevented if flow events of sufficient water velocity and depth were delivered. They showed that sufficient bending stress induced by hydrodynamic drag on the macrophyte stem caused stem rupture (lodging) – failure involving permanent deformation and loss of plant function. They quantified the depth-velocity envelope required to induce rupture, providing a means to estimate the flow required to provide hydrodynamic protection against encroachment by macrophytes. For the Williams River, the discharge required to rupture macrophyte stems was computed by application of Groeneveld & French's (1995) relationship. The diameter of the macrophyte stems tested was set, as recommended by Groenveld & French (1995), to 11.9 mm. Two thresholds were then evaluated to give a 95 percent and 99.9 percent chance of stem rupture respectively. The thresholds were reported as the discharge required for the product of flow depth and velocity to exceed either 0.152 ($Q_m^{95\%}$) or 1.52 ($Q_m^{99.9\%}$). Flow depth and velocity were computed by the HEC-RAS models developed for each site. Depth and velocity were assumed to be cross-section means.

2.8 Geomorphic form discharge thresholds

2.8.1 *Types of geomorphic form thresholds*

The Williams River has some variation in macro-scale morphology between the five defined reaches (ie the reaches are morphologically different in some respects). The river also has variable morphology at the meso-scale (ie within reaches). In this context, meso-scale features are channel and floodplain physical forms such as gravel bars, pools, riffles, rock bars, undercuts, benches, floodplains, wetlands, anabranches, islands, flood chutes, and the bed and banks. These features can sometimes be linked either directly or indirectly to important ecological processes, usually through the physical habitat that they provide, which will vary through time as hydrological conditions change. Thus, it is important to know at what discharge these various morphological forms become hydraulically active, and also to know how prevalent the various forms are throughout the river reaches under investigation.

The bankfull stage is an important hydraulic parameter. It's most obvious use is to demarcate in-channel flows from overbank flows. However, the more important aspect of bankfull stage is that, in alluvial channels, it is a good indicator of the dominant discharge, or discharge that is responsible for most sediment transport. This may not hold for incised channels, where the banktop is higher than would be expected, such that flows that occur every 1 to 3 years (the typical range of recurrence interval for bankfull flows) may be well contained within the channel.

For many channels, bankfull stage is a difficult feature to identify with great accuracy. Gordon et al (2004) listed a range of criteria that can be applied to assist in the determination of bankfull stage. It is a property best estimated by a qualified geomorphologist using a combination of field inspection, examination of transects, and analysis of the return interval of the discharge required to produce a given water surface elevation.

The Williams River has a narrow floodplain, and some remnant depressions. These represent former wetlands, now generally in a degraded condition from the perspective of vegetation health. Nevertheless, they may hold some potential for rehabilitation, provided they remain hydrologically connected to the river.

In-channel benches and gravel bars are present in the Williams River. From a purely physical perspective there is no requirement to inundate these features (mobilisation of gravels and cobbles making up the features requires flow significantly higher than the flow that just inundates the features). However, these morphological features also perform ecological functions, such as supporting particular vegetation associations and providing a source of carbon and propagules to the river when inundated.

Fish passage requires certain hydraulic conditions to be satisfied at rock bars and riffles, which are the limiting locations. These requirements can be defined in terms of species-specific minimum depth and maximum velocity criteria. These hydraulic criteria can be expressed as a threshold discharge range when fish passage is available.

2.8.2 *Measurement of geomorphic form discharge thresholds*

Channel morphology data are usually derived from on-ground surveys, and aerial photograph interpretation. In the case of the Williams River, a 2-metre grid cell LiDAR-derived DEM (Digital Elevation Model) was available for the study area upstream of Dungog. The channel morphology upstream of Dungog (S7, S8, S9 and S10) was characterised from this DEM. Downstream of Dungog (S11 and S12) the channel morphology was characterised using cross-sections surveyed by a previous flood study The defined morphological levels were related to a discharge threshold via the HEC-RAS models developed for each site.

2.9 Relating geomorphic form and process discharge thresholds to flow event frequency in the river

The geomorphological characteristics of the Williams River were investigated using fieldwork and cross-section analysis at six sites located between the proposed Tillegra Dam site and Mill Dam Falls. In order to investigate the relationships between discharge and the identified geomorphic processes and features, a time series of discharge was associated with each of these sites. Time series' of mean daily discharge from 1 January 1931 to 31 December 2007 were supplied by Hunter Water Australia. The two scenarios considered were current and a base case 'with dam'.

A river flow series can be considered as comprising a group of flow components, with each flow component defined in terms of its peak magnitude and/or duration and/or timing. These are flow events, which are concerned with times when the river flow is changing fairly rapidly. Such events cover the range of small, short-lived freshes up to high magnitude, but infrequent, over-bank events. Components of the river's ecology usually respond rapidly to flow events, or are critically dependent on them. Flow events such as these occupy a relatively small percentage of the total time that a river is flowing. Most of the time the river flow is relatively constant or changing slowly. Under these conditions, the magnitude of the flow sets the hydraulic conditions of depth, velocity and shear stress at any point in the river. The animals and plants in the river respond by either utilising, enduring, or, if possible, avoiding those conditions, depending on their tolerances and requirements. The critical aspect then becomes the duration of the conditions. Thus, for low and steady flows, the main aspect of interest is duration of flow at that level, while for events of a given threshold magnitude the main aspect of interest is frequency of occurrence.

The geomorphic process and form thresholds identified at the Williams River sites relate to events, as opposed to low steady baseflows. Thus, the main hydrological statistic of interest is flow event frequency. Techniques for estimating flow event frequency are well established (Gordon et al 2004 pp. 204 – 215).

For flood frequency analysis it is important to use a record that is representative of the total population of floods that are likely to occur on the river. Gordon et al (2004) reproduced a table indicating the record length required to estimate floods of various ARI with 95 percent confidence (Table 4). This suggests that the 77 year long gauged and modelled time series' available for the Williams River are adequate to estimate floods between the 50 year and 100 year ARI with 10 – 25 percent error. For more frequent events the accuracy is better, being close to 10% for the 10 year ARI event.

Table 4.
Length of record required to estimate floods of various average recurrence interval (ARI) with 95% confidence. Source: Reported in Gordon et al (2004, p. 205).

Average recurrence interval	Years of record required	
	10 % error	25 % error
10 years	90 years	18 years
50 years	110 years	39 years
100 years	115 years	48 years

Water year is the period over which a hydrological year is defined. Water year usually begins with the month of lowest mean monthly discharge (Gordon et al 2004, p. 69). Water year is usually defined to avoid splitting the flood season between consecutive years. The Williams River has distinctive summer flow dominance, so the calendar year is unsuitable as a water year. Historically, the month of lowest mean flow at Tillegra gauge was September (Figure 7), so the water year was defined as extending from September to August. This water year was used for flood frequency analysis and for sediment load estimation.

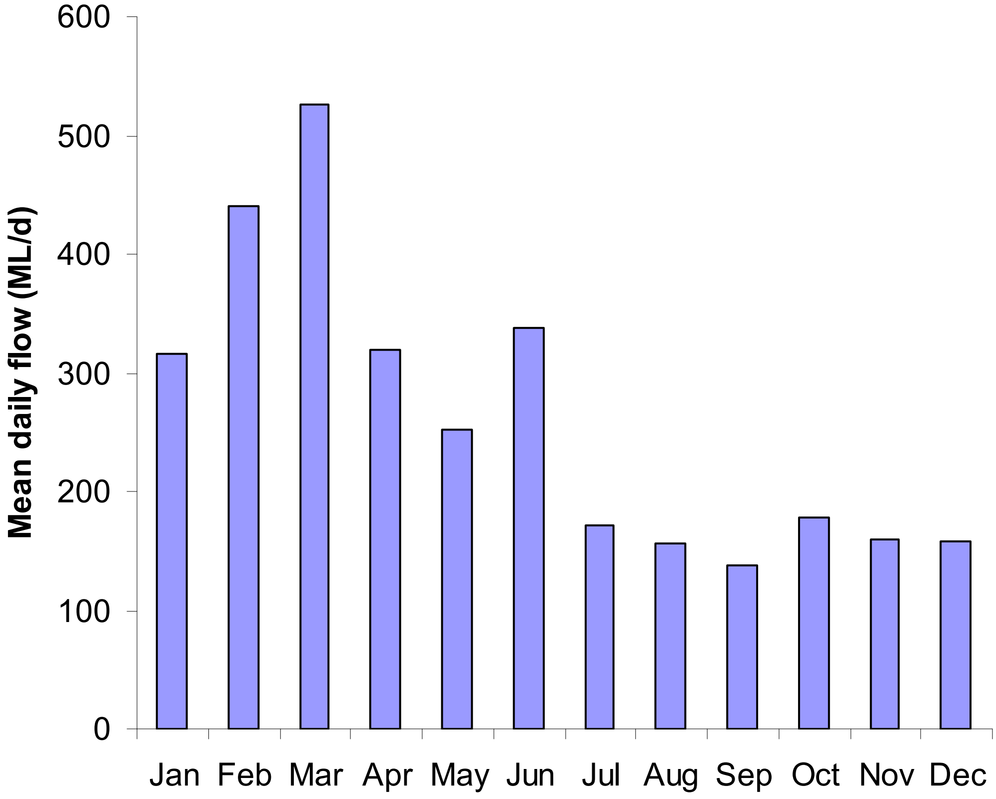

Figure 7. Distribution of average daily flow for each month for Tillegra (210011) historical series 1931 to 2007.

Flood frequency analysis is usually performed on a data series composed of the single highest peak flow in each year, termed the annual maximum series (Gordon et al 2004, p. 206). For ecological and geomorphological applications, where there is usually greater interest in the smaller more frequent events, it may be preferable to use the partial duration series. A special case of the partial duration series is the annual exceedance series, where the *N* highest peaks within the *N* years of data are chosen, regardless of when they occur (Gordon et al 2004, p. 206). Smaller floods will occur more frequently than indicated by the annual maximum series because several peaks in one year may be higher than the highest flood in other years. For average recurrence intervals of about 10 years or more the difference between the annual maximum series and the partial duration series is small. IEA (1998) recommends that the annual maximum series be used for ARI ≥10 years.

It is important to objectively fit flood frequency data to a distribution rather than using eye fitted curves to estimate average recurrence intervals. The flood peaks extracted from the flow record for the annual exceedance series were fitted to a polynomial of log(ARI) to log(discharge). The Log Pearson Type III (LPIII) distribution has been selected by government agencies in the United States and Australia as a standard for the annual maximum series (Gordon et al 2004, p. 208). However, a large variety of distributions have been investigated for application to flood data, and there is merit in selecting an alternative to the LPIII for the Williams River if one can be found to better fit the data. The annual maximum series' were analysed using EasyFitXL V4.1 (©MathWave Technologies) to determine the best fit distribution for each series. Selecting the best fit distribution was based on examination of the objective 'goodness-of-fit' rankings determined by EasyFitXL, and also by consideration of how well the distributions fitted the data for ARIs greater than 10 years. In general, the LPIII was not a particularly good fit to the data, with the Generalised Pareto distribution (a right-skewed three parameter distribution) being the best performing fit overall, and the Johnson's SB distribution (a four parameter bounded log-normal distribution function) being the best fit to the data from downstream of Chichester River.

A comparison of the partial series and the annual series revealed that the flood distributions were best described by the annual exceedance series curve for ARI <10 years, and the annual maximum series for ARI ≥10 years. The combined results predicted flood frequency across a range of ARIs.

Flood frequency analysis is ideally undertaken using the peak instantaneous daily discharge series rather than the mean daily discharge series. The implication of this is that the reported flows for a given average recurrence interval (ARI) occur only momentarily, not necessarily for an entire day. This is appropriate for characterising events that only need to reach the defined hydraulic threshold momentarily, rather than exceed the threshold for a period of 24 hours or more (bankfull is an example of such a threshold).

A near-complete daily hydrological record was available from the Williams River Tillegra gauge (210011) from 1931. Glen Martin (Mill Dam Falls) gauge on the Williams River (210010) record began in 1928, but peak daily values were available only since 1974. Dungog gauge (210903) record began in 1995. Mean daily and instantaneous peak daily flows were extracted from Pineena 9.0 for the Tillegra, Dungog and Glen Martin gauges. Significant relationships existed between mean daily and peak daily discharge at these gauges, although there was considerable scatter present (Figure 8). Annual maximum series flood frequency analysis was undertaken for each of the mean and peak data series'. Each series was fitted to a distribution and the predicted peak and mean daily discharge for a range of ARI were compared. This procedure removed the scatter and produced relationships between peak discharge and mean daily discharge (Figure 8). For Tillegra and Glen Martin gauges these relationships were considered reliable, but the Dungog record was considered to be too short to produce a reliable flood series. It was noticed that, for Tillegra and Glen Martin data, linear relationships based on the flood series were similar to those formed from the entire daily data sets, but with a coefficient lower by a factor of 0.936 (Figure 8). On the basis of this observation, a relationship to predict peak flows from mean flows for Dungog was estimated (Figure 8).

The relationships between mean and peak daily flows were used to predict peak discharge associated with any given ARI from flood frequency curves derived from mean daily discharge data. The Dungog relationship was applied to flow data derived for downstream of Chichester River junction. These relationships were applied to both the current and the base case 'with dam' scenarios, although it is likely that attenuation of flood peaks by the Tillegra Dam would change the relationships (i.e. bring them closer to 1:1). This effect could not be accounted for, so the results here probably overestimate the 'with dam' flood peaks. This means that for any given discharge, under the 'with dam' scenario, the floods are likely to be less frequent than indicated by the predicted ARIs.

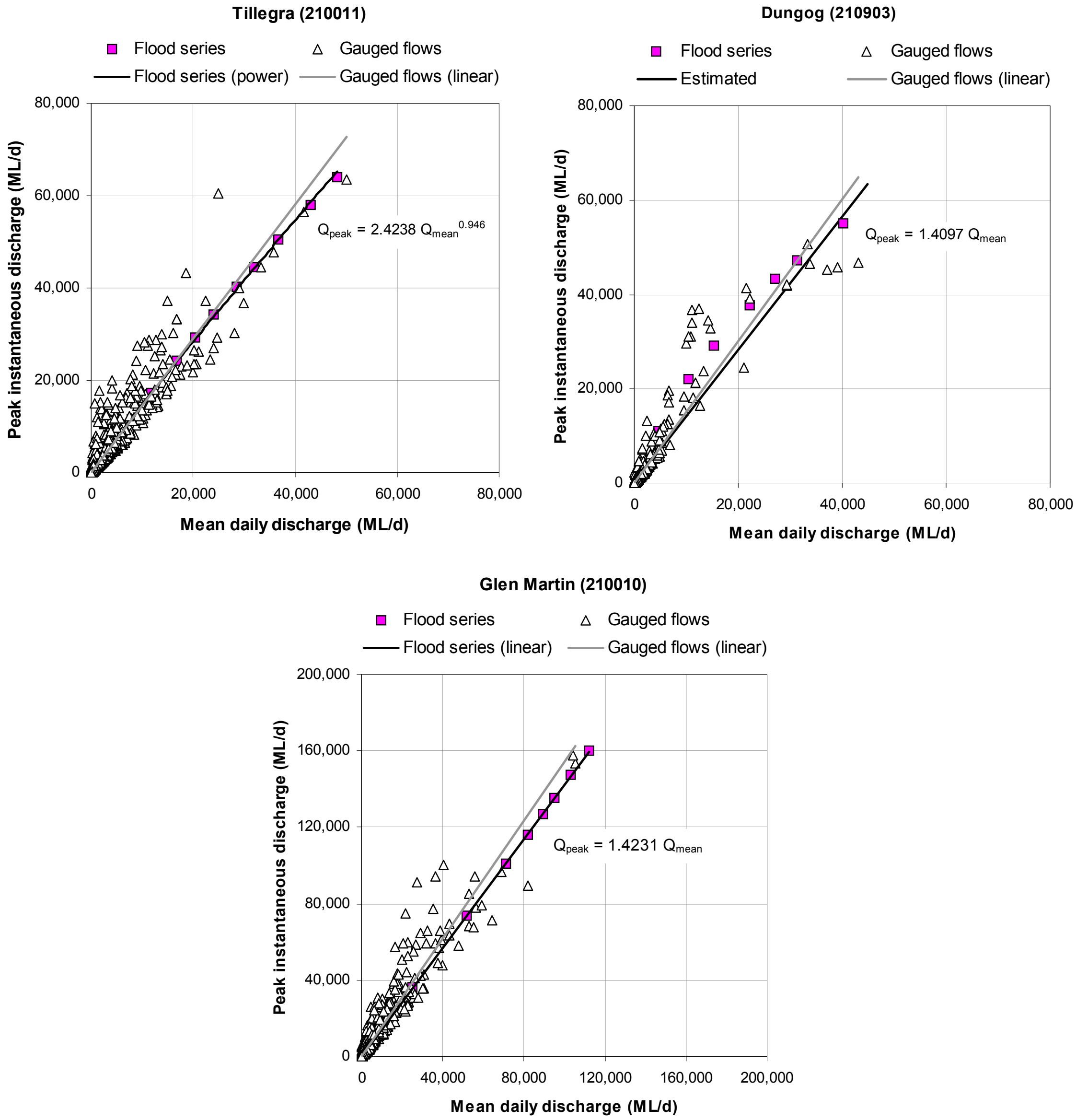

Figure 8. Relationships between gauged daily peak instantaneous discharge and gauged mean daily discharge for three gauges. Regressions formed on the basis of data derived from flood frequency analysis on peak and mean daily data series', except for Dungog, where the time series was too short and the flood series was considered to be unreliable.

The flood frequency curves allowed expression of each identified geomorphic process and form discharge threshold in terms of average recurrence interval. In other words, it was possible to say how frequently these events occurred. This was done for both the current conditions and for the 'with dam' scenario, which allowed a quantitative comparison between the two scenarios on in terms of geomorphological processes and forms. This information requires expert interpretation in terms of likely ecological implications. In this report, only broad likely consequences are noted.

2.10 Dam storage shoreline erosion

2.10.1 Review of processes

Shoreline erosion will occur in the proposed reservoir. Sediments from eroded soils could increase turbidity and potentially reduce water storage capability. Nutrients from eroded soils could increase risk of algal blooms. Recession of the shoreline would result in loss of land and accompanying vegetation.

The dam will be operated to be usually within 96 to 100 percent of capacity. This will concentrate erosion over the top part (< 1 metre) of the rim of the storage. The shoreline is likely to erode due to two main factors:

- Attack by wind generated waves, and
- Slumping of saturated banks during drawdown

If recreational boating is permitted on the reservoir then a third factor, boat wakes, also needs to be included.

In the case of Tillegra Dam it was assumed that the drawdown rate would be so slow that the bank soils would be able to drain, so slumping due to surcharge of saturated banks would probably not be a general problem. However, the rim of the reservoir would be exposed to the effects of wind waves. The existing vegetation under the full supply water level water would die, leaving the face of the shoreline exposed to wave attack. If left unprotected, with time, as the shoreline retreated from erosion, it would leave a wave-cut bench around the perimeter. The rate of erosion would decline to a very low rate when the wedge of easily eroded surface soil was removed.

The height of the waves generated by winds depends primarily on three factors:

- wind speed,
- wind duration, and
- fetch over which the wind blows

Waves approaching the shoreline would be progressively modified by decreasing water depth. The waves would slow down, their direction could change through refraction and they would steepen.

Prediction of the impact of wind-generated waves on shoreline erosion rate is a complex problem. Ekebom et al (2002) derived a method for measuring fetch length, fetch direction and wave exposure in coastal areas by applying GIS, averaged wind data, wave height forecasting curves and a linear wave model. This methodology cannot be readily applied at the site of Tillegra Dam because of the lack of detailed local wind data. The second part of the problem, which is predicting the geomorphological response of the shoreline to the prevailing wave climate, is also problematic, requiring a number of simplifying assumptions. Elçi & Work (2003) developed a method for predicting wave induced shoreline erosion on a reservoir by relating erosion rates to wind wave forces (via shear stress) and assuming a simplified representation of the shape of the beach profile. Elçi & Work (2003) and Elçi et al (2007) quantified shoreline erosion rates on Harwell Lake, South Carolina and Georgia, based on measured fetch, parameterized beach profile shape, and measured wind vectors. The methodology for the prediction of shoreline erosion was calibrated and validated using digital aerial photos of the reservoir taken in different years, and indicated approximately one metre per year of shoreline retreat for several locations. Once again, it would be difficult to apply this method to the proposed Tillegra Dam due to lack of local wind data and calibration data.

Lorang and Stanford (1983) observed that since regulation of Flathead Lake, Montana, wave erosion occurred on the entire lake shoreline, but erosion was greatest along the north shore because of the prevailing winds and more easily erodible soils. This lake was around 10 km wide and 35 km long, so it was much larger than the proposed reservoir associated with Tillegra Dam (approx. 1.5 km wide and 9 km long).

Lorang and Stanford (1983) identified three main modes of shoreline erosion: undercutting, endstripping, and over-wash (Figure 9). Each resulted in different rates of shoreline retreat and produced spatial variations in measured shoreline change. Each of these modes of shoreline erosion could be active on the shoreline of the reservoir impounded by the proposed Tillegra Dam.

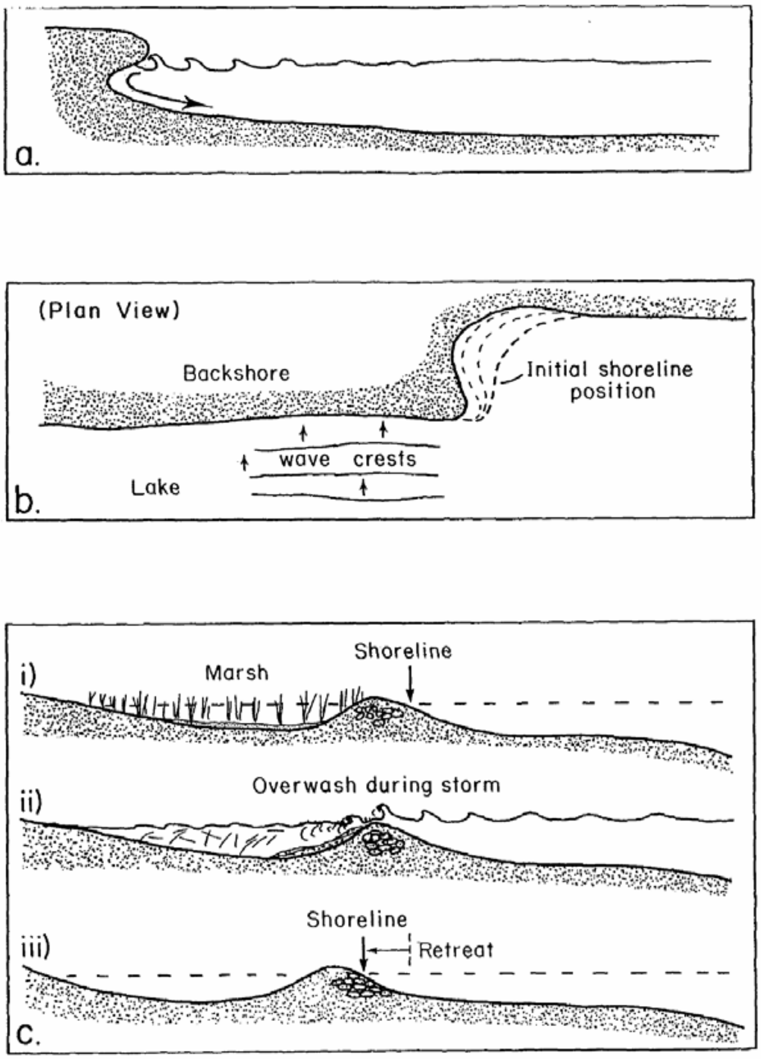

Figure 9. Three processes of geomorphic wave-shoreline interaction: a-undercutting; b-endstripping; c-over-wash. Source: Lorang **and Stanford (1983).**

Undercutting dominated as an erosion process along forested banks where elevations were well above the full pool lake level (Lorang and Stanford 1983). This wave erosion process resulted in average retreat rates of less than one metre per year, but retreat due to undercutting was highly variable. The process was one of relatively constant undercutting in response to wave energy (high and low wave energy), but sporadic slumping when the vegetation could no longer support the bank. On occasions, high winds associated with storm events blew trees down because erosion beneath the root wads had weakened anchorage (Lorang and Stanford 1983). On Lake Hawea in New Zealand, Kirk et al (2000) reported extensive cliff erosion during an extreme event when a period of very strong winds coincided with high lake levels.

The process of endstripping resulted in the most rapid shoreline retreat, being in the order of five metres per year (Lorang and Stanford 1983). Erosion was concentrated at the location of an offset or discontinuity in the shoreline. Waves wrapped around the shoreline discontinuity, where one portion of the shoreline was set back from the other, and rapidly gouged out the bank (Lorang and Stanford 1983).

Overwash operated in tandem with undercutting at elevations very near the full supply level (Lorang and Stanford 1983). Wave overwash during storms stripped the backshore vegetation while undercutting eroded from beneath. The combination of over-wash and undercutting resulted in a rapid rate of shoreline retreat of the order of one to two and a half metres per year (Lorang and Stanford 1983).

On a 63-year-old reservoir in the North Platte Valley of western Nebraska, Joeckel & Diffendal (2004) found that since construction, headland retreat was on a scale of magnitude of tens of metres. Serial observations of the shoreline made in the period 1999 – 2002 demonstrated that shoreline erosion was continuing.

On Lake Thunderbird, Oklahoma, Allen (2001) found that shoreline geometry and bathymetry played a major role in determining the degree of erosion at a particular shoreline site. Sites with straight shorelines or headlands that were exposed to long wind fetches from prevailing wind directions were particularly vulnerable to more frequent and higher waves. Conversely, sites that were within coves or that were behind peninsulas or islands that blocked the wind were more protected from waves. In these areas vegetation was often present and erosion was less severe or even minimal. Lake bathymetry also influenced wave action. The shallower was the nearshore, and the wider was an underwater bench, the higher was the drag or resistance to waves. Waves were subsequently smaller in such areas, in contrast to areas where the water deepened abruptly and there was less resistance or bottom roughness to influence the waves. Allen (2001) described a range of bioengineering techniques to suit different scales and forms of lake shore erosion.

2.10.2 *Adopted method for estimation of potential shoreline erosion*

The potential erosion of the proposed shoreline of the proposed Tillegra Dam reservoir was examined by assuming that for the majority of the time, the dam would be kept between 96 per cent full and FSL. It was also assumed that under variable levels, with wind generated waves, that the vegetation on the shoreline would, over time, most likely die, exposing the bank to erosion. Erosion would strip away the soil to the underlying weathered parent material. The eroded soil would deposit within the storage. The volume of material was estimated on the basis of assumed soil depth, assumed variation of storage water level between 96 percent of FSL and FSL, and measured slope. For the purposes of estimating shoreline erosion, the rim of the impoundment was divided into three zones, referred to here as the 'Northern and Eastern', the 'Western' and the 'Southern' (Figure 10).

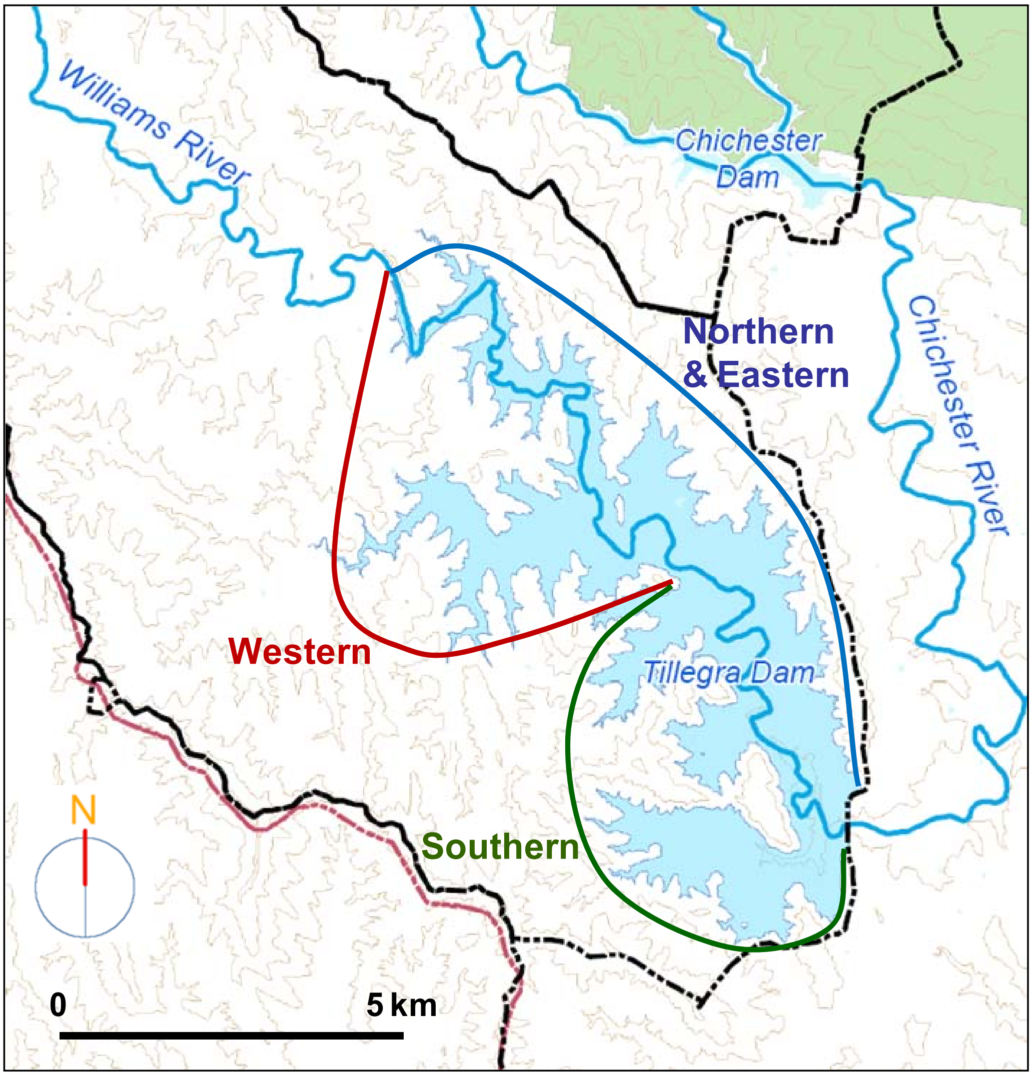

Figure 10. Rim of Proposed Tillegra Dam storage divided into three zones for the purpose of estimating shoreline erosion. Contour lines are spaced 10 metres.

The rim of the impoundment intercepts two of the Dungog Soil Landscapes described and mapped by Henderson (2000a, 2000b) (Figure 11, Table 5). The Tillegra Soil Landscape (ti) is found in rolling hills in Carboniferous sediments. The variant (**tia**) is found in undulating low hills. Otherwise **tia** is similar to **ti**. Five profiles described by Henderson (2000a) were 1.3 m deep (70 % of Landscape), >0.8 m (10 % of Landscape), 1.1 m deep (10 % of Landscape), 0.9 m deep (10 % of Landscape) 2.0 m deep (10 % of Landscape). The sum of these percentages exceeds 100, so there is an error here with the areas. However, it would appear that the Tillegra Soil Landscape ranges in depth from 0.8 to 2.0 metres, with most of it around 1.3 metres deep. The Williams Range Soil Landscape (**wi**) is found in steep hills and mountains on Carboniferous sediments. The variant (**wia**) occurs on dry exposed slopes. Otherwise **wia** is similar to **wi**. Five profiles described by Henderson (2000a) were 0.25 m deep (30 % of Landscape), 0.3 m deep (20 % of Landscape), 1.0 m deep (10 % of Landscape), 1.3 m deep (5 % of Landscape) and 0.9 m (5 % of Landscape).

These percentages do not add to 100, so there would have been some undescribed profiles. In the small Nerrigundah catchment, just downstream of Tillegra, Walker et al (2001) carried out a very detailed soil mapping exercise, and found that the majority of the soil had a depth of less than 0.6 m.

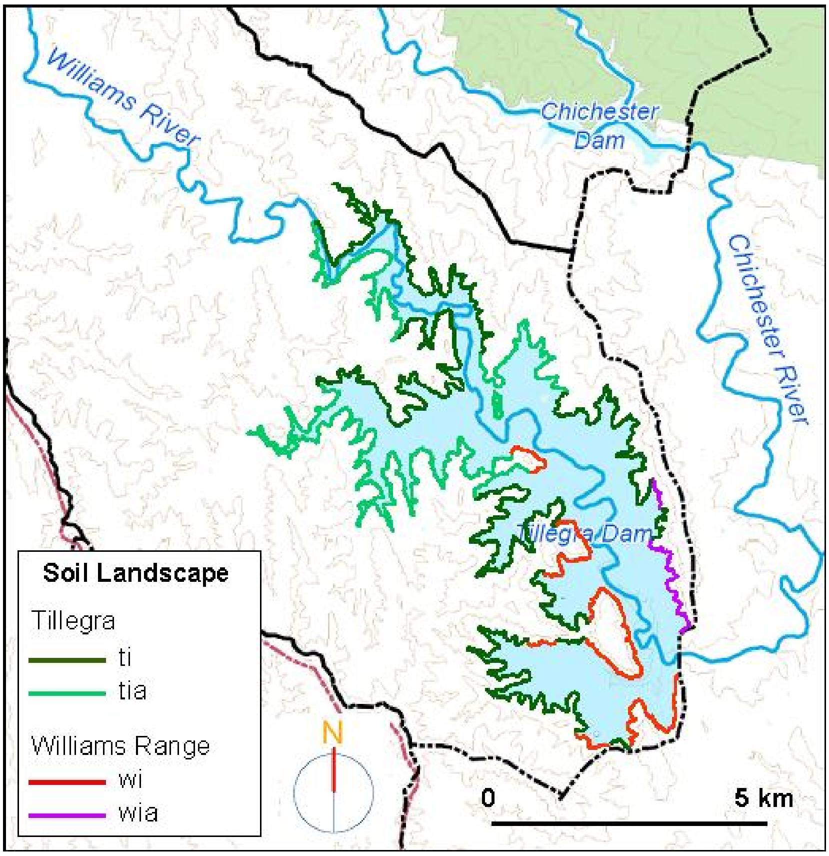

Figure 11. Soil Landscape types around the rim of proposed Tillegra Dam storage. Derived from Henderson (2000a).

On the basis of the soil depth descriptions, for the estimation of shoreline erosion, it was assumed that the mean soil depth for Tillegra Landscape soils was 1.3 m and for Williams Range Landscape soils a mean depth of 0.3 m was assumed. In reality, the soil depth would vary around the rim much more than this, but the purpose of this exercise was not to predict erosion at particular points on the rim, but to make an estimate of total shoreline erosion within the perimeter of the impoundment. It was assumed that the proposed re-vegetation of the side-slopes of the impoundment would reduce erosion rates to a minimal level, so these slopes were not regarded as being a potentially significant source of sediment to the impoundment.

Table 5.
Descriptions of the two Soil Landscapes found on the rim of the impoundment of the proposed Tillegra Dam. Source: Henderson (2000a).

Soil Landscape	Soils	Soil depth	Slope
Tillegra	Moderately deep to deep, well to imperfectly drained Brown Sodosols (Soloths) with moderately deep, moderately well-drained Brown Kurosols (Yellow Podsolic Soils) on sandstone. Shallow to moderately deep, well to moderately well-drained Palic Leptic Tenosols (Lithosols) and Melanic Leptic Tenosols (No Suitable Group) on siltstone. Deep, well-drained Red Kurosols (red Podsolic Soils) on shoulders of crests.	0.8 to 2.0 m (majority 1.3 m)	5 to 25 % (up to 33 %)
Williams Range	Shallow to moderately deep, well to rapidly drained Bleached-Leptic Tenosols (Bleached Loams) and well-drained Chernic-Leptic Tenosols (Structured Loams) on siltstone. Shallow to moderately deep, well to rapidly drained Orthic tenosols (Lithosol/minimal Brown Earths) on sandstone. Moderately deep, well drained Red Kurosols (Red Podzolic Soils) an shoulders of crests on siltstone. Moderately deep, well-drained Red Dermosols (Terra Rossa Soils) on limestone outcrops.	0.25 to 1.3 m (majority 0.25 – 0.3 m)	25 to >50 %.

In the small Nerrigundah catchment, just downstream of Tillegra, hillslope gradients measured by Walker et al (2001) were typically 11 % with a range from 3 to 22 %, and the main drainage line had an average slope of 9 % with a range from 1 to 17 %. Henderson (200a) described the slopes of the Tillegra Soil Landscape as ranging from 5 to 25 %, but up to 33 % (Table 5). The Williams Soil Landscape was steeper, with slopes of 25 to >50 % (Table 5). The slope of the rim of the impoundment of the proposed Tillegra Dam was determined by measuring the slope of the land around the rim at 267 equi-spaced locations. The measurements were taken over the three defined zones of the rim (Figure 10). The measured distributions of slope values (Figure 12) were consistent with those reported by Walker et al (2001) and Henderson (2000a). In general, the Southern zone had steeper slopes than the Northern and Eastern and Western zones. This is explained by the presence of significant areas of Williams Range Soil Landscape in this zone (Figure 11).

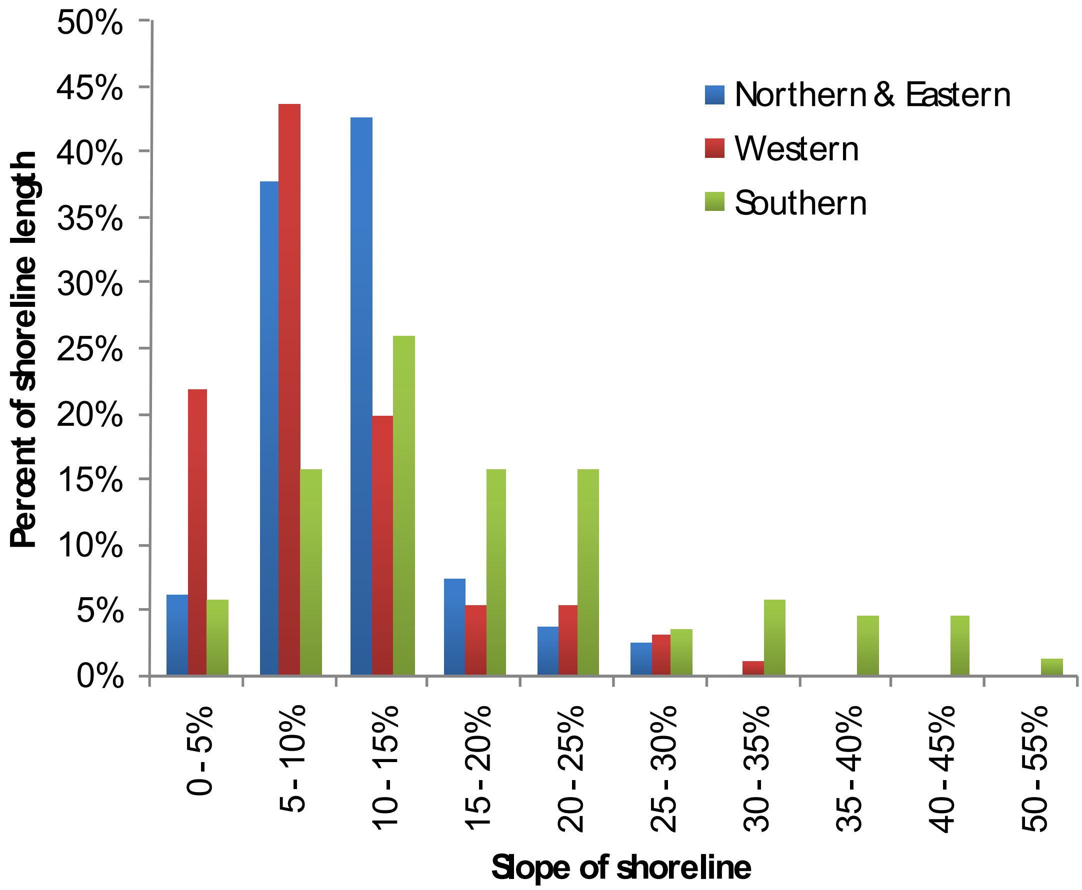

Figure 12. Slope of shoreline around rim of Proposed Tillegra Dam storage for three defined zones.

The closest weather station to Tillegra where wind is measured by the Bureau of Meteorology is Lostock Dam (Station ID 061288), about 20 km due west. This station opened in 1969 and is current. The data indicate that it is uncommon for wind to blow from the north, northeast and south (Figure 13). The longest duration high winds blow from the west, southwest and southeast. This means that the Northern and Eastern zone of the dam rim (as defined) is the most exposed to prevailing winds.

A simple model of maximum potential shoreline erosion was devised on the basis of assuming that all of the soil would be removed in the zone between 96 percent of FSL and FSL, plus an additional 0.4 m allowance for wave height (Figure 14). This model was used to estimate the volume of soil loss and the resultant exposed width of the shoreline when the water level was at 96 percent of FSL and at FSL.

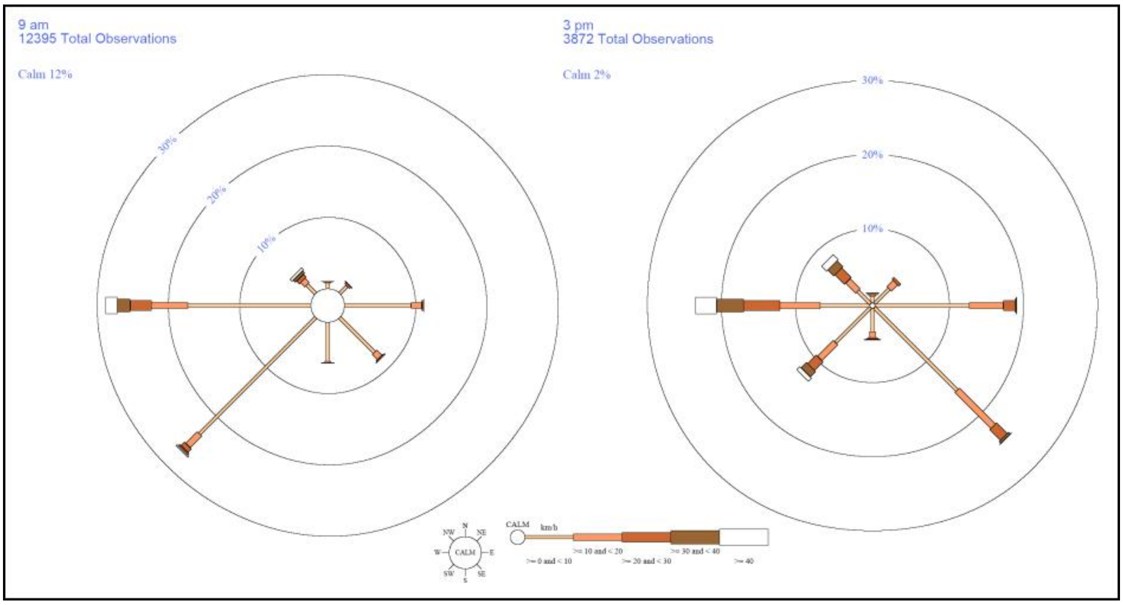

Figure 13. Wind roses for Lostock Dam weather station. Source: Bureau of Meteorology (http://www.bom.gov.au/climate/averages/tables/cw_061288.shtml).

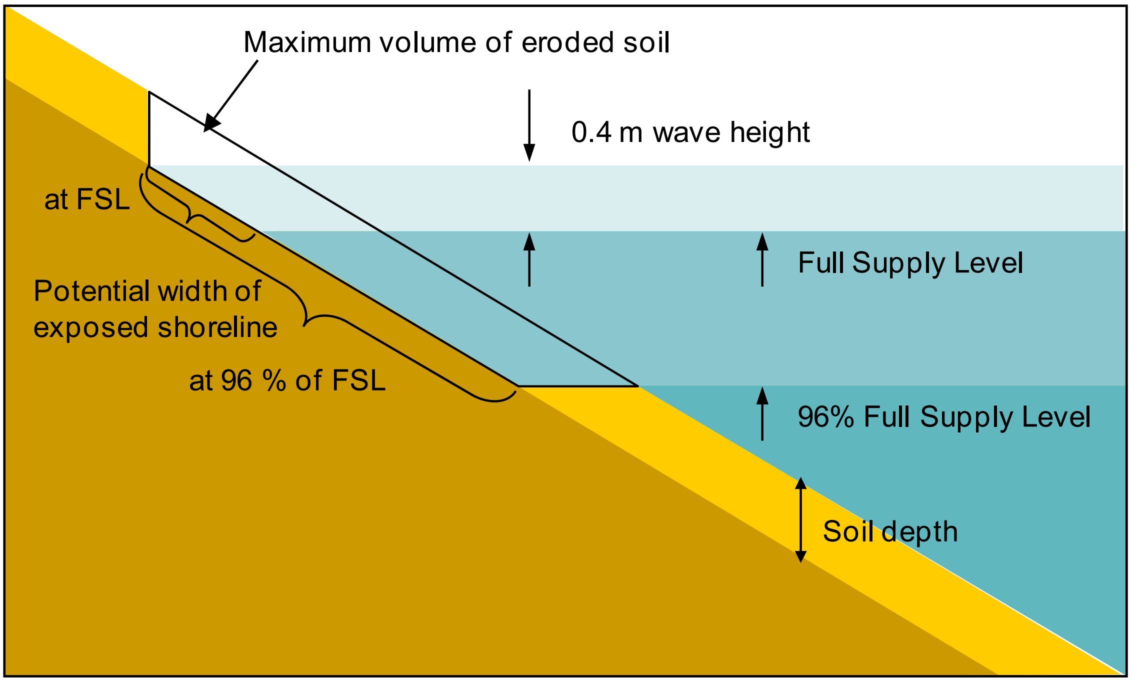

Figure 14. Conceptualisation of maximum shoreline erosion. Not drawn to scale.

Key features of the existing environment

2.11 Existing information

2.11.1 Catchment-scale overview

The Williams River flows through four identified geomorphic units (Galloway 1963). Erskine (2001) described the units as:

> ‘The Coastal Zone forms a short section of the lower Williams River where there are estuarine and freshwater swamps, tidal channels, mangroves, salt marsh and coastal floodplain. Extending inland from the Coastal Zone is a small corridor of lowlands (Central Lowlands), developed on less resistant Permian sedimentary rocks. This undulating terrain has been largely cleared. An extensive mountainous tract of largely cleared country called the Northeastern Mountains covers most of the Williams River catchment. These mountains contain Carboniferous sedimentary rocks and late Early Permian granitoid rocks, and flank the higher Barrington Tops. The Tops correspond to an extensive plateau or palaeosurface eroded into late Early Permian granodiorite, which is capped by early Eocene basalt.’

Thus, the Coastal Zone is identified here as the reach from the Hunter River confluence to Seaham Weir. The Central Lowlands extends from Seaham Weir to Mill Dam Falls (Glen Martin). The Northeastern Mountains extends from Mill Dam Falls to at least Salisbury, with only the upper headwaters in the Barrington Tops.

Elevations range from over 1,500 metres at Barrington Tops, to 10 metres at Seaham Weir. Fifty-six percent of the catchment is cleared pasture, with 42 percent under forest or woodland, and the remaining 2 percent either urbanised or under cropping or intensive animal production (Krause et al 2003).

The Williams River is a relatively steep, large-capacity, gravel bed channel with in-channel benches and various types of gravel and bedrock bars (Erskine 1986; Erskine 1998; Erskine & Livingstone 1999; Erskine 2001). Brooks et al (2004) classified the river near Munni as discontinuous floodplain river style, which is typical of many coastal gravel-bed rivers in eastern Australia. As such, it exhibits alternating reaches of close bedrock confinement and unconfined floodplains (Erskine 2001). Significant lateral migration is restricted to unconfined bends (Erskine 1998 p. 14). The resistance of the channel boundary is also enhanced by dense bankside vegetation, coarse bed material and bedrock bars in the bed (Erskine 2001).

The floodplains and hillslopes of the Williams River have been cleared since first European settlement in the early 1800s (Erskine 2001). Both Erskine (2001) and Brooks (2004) assumed that widespread forest clearance during the initial settlement phase increased runoff and flood peak discharges. Erskine (1998) was of the view that improved pastures, which have been in extensive use since the 1960s, would have decreased runoff since then.

2.11.2 Hillslope erosion

Erskine (1998 p. 29) cited reports that indicated that the upper Hunter Valley experienced widespread soil erosion post-European settlement, which had worsened considerably by the mid-1940s in association with severe drought, prickly pear infestation and rabbit plague. In 1948 it was estimated that the total soil loss from erosion in the Hunter Valley was in excess of 765,000 cubic metres per year (MHL 2003 p. vi). Erskine (1998 p. 29) cited work by Erskine & Bell (1982) that found evidence for a progressive decrease in the area affected by soil erosion between 1943 and 1973, combined with no increase in gully erosion. This change came about through increased ground cover, improved pastures, better land management and completion of soil conservation works.

Erskine (1998, p. 29) speculated that the pattern of high post-European soil loss and post-1940s recovery was similar to that of the Upper Hunter in general. Krause et al (2003) estimated current soil erosion rates in the Williams River catchment by applying the SOILOSS model (an Australian version of the Universal Soil Loss Equation) and caesium-137 tracing techniques. This work revealed that the net median surface erosion rates ranged between zero and 0.64 tonnes per hectare per year, with an average median value of 0.19 tonnes per hectare per year. Krause et al (2003) noted that these rates were among the lowest measured values in Australian studies. Packed earth has a bulk density of around 1.5, so with an active catchment area of 958 km^2 at Glen Martin (excludes 197 km^2 of inactive catchment upstream of Chichester Dam which traps most sediment produced), this average erosion rate equates to approximately 12,000 cubic metres per year (18,000 tonnes per year) at Glen Martin. It is unlikely that all of this eroded material would manifest as suspended sediment in the channel system, as a significant proportion of it would be stored in low energy areas of hillslopes, and especially on floodplains, prior to reaching the channels.

Over the period 1997-2002, the National Land and Water Resources Audit (the NLWRA) (URL: http://www.nlwra.gov.au/) coordinated and commissioned a range of assessments that encompassed Australia's land, water and biodiversity. All the information gathered by the Audit on sediment supply, transport and deposition in river systems can be found at National Land and Water Resources Audit (2001 2002). The methodology is reported in Prosser et al (2001a; 2001b). The streams theme has 39 layers preserved in its attribute table, with several relevant to sediment supply, transport and deposition.

The median of 0.19 tonnes per hectare per year soil loss for the Williams River catchment quoted by Krause et al (2003) is within the lowest 20 percent of modelled current (averaged over last 100 years) hillslope erosion values for all 14,412 Australian river and stream links on the NLWRA database, confirming the claim of a relatively low current rate of soil loss. For the Williams River catchment, the NLWRA modelled current hillslope erosion was 0.32 tonnes per hectare per year for the Williams River upstream of Chichester River junction, and 0.55 tonnes per hectare per year for the Williams from Chichester River junction to Dungog. These values, which integrate over the past 100 years (and include the period of high erosion from settlement to the mid-1940s), are expectedly somewhat higher than the current rates measured by Krause et al (2003), but are of the same order of magnitude. Downstream of Dungog to the confluence with the Hunter River the NLWRA modelled rate of soil loss increased markedly, but this does not necessarily mean that there is a proportionately large increase in suspended sediment supplied to the river (as proportionately more sediment may be trapped in hillslope and floodplain storages). The NLWRA also estimated pre-European or natural hillslope soil erosion rates, assuming natural vegetation cover. For the Williams River upstream of Dungog, the post-European erosion rate was 1.4 – 3.1 times higher than the natural rate, and downstream of Dungog it was 36 – 70 times higher.

Although the volume of material delivered to the channel system from soil erosion has declined since the 1940s, this is not the case with sediment sourced from the channel itself (ie bed and bank erosion), which appears to have increased since the

1940s (Erskine 1998).

2.11.3 History of channel works and geomorphic response

Erskine (1998; 2001) documented the history of river training works undertaken on the Williams River and Chichester River discontinuously over the period 1954 to 1991. According to Erskine (2001), the river training works were undertaken for one or more of the following reasons: (i) to stop bank erosion; (ii) to remove obstructions that partially block the channel and concentrate flows against the banks, causing erosion; (iii) to provide a stable channel pattern; (iv) to protect specific structures, such as bridges; and (v) to stop a potential change of river course by alluvial stripping.

Erskine (1998; 2001) and Brooks (2004) pointed out the negative geomorphic consequences of the river training works. Erskine (2001) listed these as: (i) extensive removal of natural gravel armour layers, and natural boulder and log steps, resulting in the loss of natural energy dissipation and the consequent initiation of bed erosion; (ii) loss of pools either by infilling with sediment or by bed erosion of the downstream riffle; (iii) excessive removal of LWD and trees from the channel; (iv) planting of large numbers of exotic trees in the riparian zone; and (v) extensive bulldozing of the channel to remove bars, particularly mid-channel bars, to artificially create a single thread channel and hence reduce morphological channel complexity.

Erskine (2001) cited evidence that there was extensive erosion present prior to the river training works being implemented, and that by the 1990s this had markedly decreased. While the rate of channel erosion may have declined since its peak, Brooks et al (2004) regarded the channel as currently having ‘oversized channel dimensions’. Based on a cross-section defined by alluvial banks at Munni (5.1 km upstream of the Tillegra gauge), Brooks et al (2004) estimated that ‘bankfull discharge’ equalled 800 m^3/s (70,000 ML/d), a flood with a recurrence exceeding 100 years. In contrast, Erskine (1998 p. 12) reported that the bankfull discharge of the river at the Tillegra gauge was equal to a return period of only 7 years on the annual maximum series (calculated here to be approximately 29,000 ML/d), although he still regarded this channel capacity to be ‘large’ (Erskine 1998, p. 14). The flood frequency curve for Tillegra is relatively flat on the high end of the recurrence intervals (Erskine 1998 p. 19), such that the flood peak magnitude of the 1 in 100 year flood is only just over double that of the 1 in 7 year flood; relatively small differences in cross-sectional area would explain the contrasting estimates of bankfull discharge by Brooks et al (2004) and Erskine (1998).

Brooks et al (2004) were of the view that the present channel size reflects riparian zone disturbance since European settlement, and that channel expansion accelerated with the onset of desnagging in the 1960s coincident with a series of large floods. Erskine (2001) agreed that desnagging and riparian vegetation removal destabilise channels, but he explained the Williams River pre-works channel expansion more in terms of a series of large floods between April 1946 and March 1963. The recovery he explained in terms of the lack of catastrophic floods during the works programme and the vegetated nature of the channel banks interacting with the river management works to assist channel recovery. Brooks et al (2004) pointed out that the 1946 flood (the flood of record), which preceded the beginning of desnagging in the 1960s, did not induce riffle crest lowering at Tillegra gauge, suggesting that the channel was stable at that time. There is no doubt that river stability is a function of both resistance (offered by large woody debris and riparian vegetation) and fluvial energy. Significant geomorphic change results when the energy or shear stress of the flow exceeds the resistance of the channel boundary. This is more likely in extremely large flood events.

Erskine (1998 p. 17) concluded from an analysis of flood variability that the Williams River is likely to be more stable than other Hunter Valley rivers. The relatively low flood variability and relatively low flood magnitudes, combined with the relatively dense riparian vegetation, presence of rock bars and bedrock valley walls, and coarse bed material particle size (which offer resistance to erosion and degradation), means that the Williams River has relatively good prospects for physical rehabilitation (stabilisation) compared to most other Hunter Valley rivers.

2.11.4 *Reach geomorphic characteristics*

The descriptions of the reaches were based on a review of the literature, two reconnaissance site visits, on 10 and 20 August 2007, a helicopter flight on 14 November 2007, and two detailed geomorphological field surveys, the first conducted on 3 December 2007 (at S7, S8 and S9), and a second on 10 May 2008 (at S10, S11 and a site near S12). The most recent large flood event in the river prior to the 2007 surveys occurred on 9 – 10 June 2007. The instantaneous peak of this event at Tillegra (53,000 ML/d) corresponded to an average recurrence interval of approximately 37 years and at Glen Martin (98,000 ML/d) it was approximately 5 years. Just prior to the May 2008 field visit, on 26 April 2008, a moderate sized flood event occurred. The instantaneous peak of this event at Tillegra (31,000 ML/d) corresponded to an average recurrence interval of approximately 8 years and at Glen Martin (69,000 ML/d) it was 2.5 years. At Tillegra these events had sharply rising and falling hydrographs, with a big difference between instantaneous peak flow and mean daily flow. The hydrological conditions prevailing at the time of the field visits are described below.

According to NSW Provisional River Data (http://waterinfo.nsw.gov.au) on 10 August 2007 the mean daily flow at Tillegra was 19 ML/d; at Dungog gauge it was 84.6 ML/d; and, at Glen Martin it was 96.6 ML/d. A minor flood event occurred on 20 August 2007, with the mean daily flow at Tillegra peaking at 9,200 ML/d at 10.00 pm; Dungog gauge peaked at 13,000 ML/d at midnight; and Glen Martin gauge peaked at 18,600 ML/d at 11.00 am on 21 August. On the morning of the helicopter flight on 14 November 2007, the flow in the river was approximately 78 ML/d at Tillegra, 290 ML/d at Dungog and 380 ML/d at Glen Martin. On 1 – 2 December 2008 (the days prior to field work) the Williams River experienced a minor storm event that peaked at approximately 2,000 ML/d at Tillegra. By 3 December this event had receded to 300 – 500 ML/d for most of the day. However, the weather was stormy, and a storm event rose late in the day. On the morning of 4 December 2007 this event peaked at approximately 3,200 ML/d at Tillegra gauge, approximately 2,200 ML/d at Dungog and 2,700 ML/d at Glen Martin. This flow was sufficient to prevent wading into the stream. On 10 May 2008 flow was 340 ML/d at Tillegra and 628 ML/d at Glen Martin.

2.11.4.1 **Reaches 1 and 2: Upper Williams River to storage and storage**

These two reaches extend from the headwaters of the Williams River through Munni (Figure 15) to the site of the proposed dam at Tillegra (Figure 16). Brooks et al (2006) estimated that the Williams River at Munni expanded in cross-sectional area by 50 percent after the 1940s. This was attributed to channel and riparian zone disturbance since European settlement, particularly desnagging, and that channel expansion accelerated with the onset of river training works (which included desnagging) in the 1960s coincident with a series of large floods (Brooks et al 2004; Brooks et al 2006).

Figure 15. Williams River at Munni bridge. E 56 375017; N 6426443; 204° bearing (upstream); 13:08 hr; 10/08/2007. Flow 15 ML/d. Engineered log jam built in 1999 on left foreground.

Figure 16. Williams River at Tillegra Bridge. E 56 376466; N 6423424; 168° bearing (upstream); 12:57 hr; 10/08/2007. Flow 15 ML/d. Photograph taken immediately upstream of bridge.

The entire upper Williams River has been the subject of river training schemes and a Rivercare Plan (Erskine 2001). The Munni river training scheme was conducted over the period 1966 – 1968 and the Salisbury scheme 1979 – 1984, both with ongoing periodic maintenance up until 1991. Bed erosion followed the undertaking of works in the Munni and Salisbury scheme areas. The lips of pools were lowered by both excavation and subsequent bed degradation, and downstream pools were infilled with the mobilized sediment. The relaxation time (i.e. time required for pools to reform) was 15 to 20 years (Erskine 2001). These changes in medium scale bedforms occurred because the armoured bed layer and boulder steps were removed from the channel (Erskine 2001). Erskine (1998 p. 14) cited previous a study by Erskine & Livingstone (1998) that found that benches (ephemeral in-channel sediment storages, above the bed but below bankfull level) at Tillegra were relatively stable between 1981 and 1996, after the completion of river training works.

Brooks et al (1999), Brooks et al (2004) and Brooks et al (2006) undertook a large woody debris re-introduction experiment in the Williams River near Munni (Figure 15), beginning in 1999. The test reach drained an upstream area of 185 km^2 and the upstream control reach drained an area of 180 km^2 (compared to Tillegra gauge 5.1 km downstream, draining an area of 194 km^2). The median clast size of the test reach was 76 mm while the median clast size of the control reach was 77 mm. The test reach had a bed slope of 0.0025, while the control reach had a bed slope of 0.017. The mean annual flood (arithmetic mean of the annual flood series, 1931 – 1993) at Tillegra gauge was calculated by Brooks et al (2004) to be 170 m^3/s (14,688 ML/d).

2.11.4.2 Reach 3: Storage to Glen Martin

The reach of river from Tillegra downstream to Mill Dam Falls receives inflows from the regulated Chichester River, plus other smaller unregulated tributaries. This reach covers a long distance, passing through Bandon Grove (Figure 17), Fosterton (Figure 18), Dungog (Figure 19), Thalaba Bridge crossing near Alison/Warragulla (Figure 20), Glen William Bridge (Figure 21), and ending at Mill Dam Falls, also known as Glen Martin (Figure 22).

Erskine (2001) noted that prior to river training works, bank erosion resulted in up to 120 m of bank retreat and removal of up to 6 ha of floodplain in the Bandon Grove and Bendolba scheme area.

River training schemes were undertaken on the Williams River at Bandon Grove and Bendolba (1962 – 1968), Newells Crossing (1979 – 1980), Fosterton (1954, 1960 – 1969), Dingadee (1974 – 1980), Coorei (1976 – 1980) and Brookfield (1980 – 1983) (Erskine 1998, Erskine 2001). This reach was also included in a Rivercare Plan.

The Healthy Rivers Commission of NSW (1996 p. 36) and Erskine (1998 p. 45) briefly reported anecdotal claims that the river training works increased the velocity of flood flows as well as negatively impacting the pool-riffle structure of the river (presumably, degrading riffles and infilling pools).

Figure 17. Williams River at Bandon Grove. E 56 379194; N 6425397; (upstream); 12:06 hr; 10/08/2007. Flow 70 ML/d.

Figure 18. Williams River at Fosterton (Newells Crossing). E 56 382194; N 6422876; 132° bearing (downstream); 11:34 hr; 10/08/2007. Flow 70 ML/d. Photograph looking downstream from bridge. Flow 70 ML/d.

The Williams River downstream to Glen Martin is reasonably high gradient, and the bed material remains dominantly coarse grained (gravel to cobble). There is a substantial rock bar Glen William Bridge (Figure 21) and Mill Dam Falls represents a major hydraulic and bed control, with a rapid forming in this location at high flows (Figure 22). Coarse gravel benches and bars are present at Mill Dam Falls.

Figure 19. Williams River at Dungog, upstream of Coorei Bridge. E 56 383575; N 6414936; 320° bearing (upstream); 10:59 hr; 10/08/2007. Flow 82 ML/d.

Figure 20. Williams River at Thalaba Bridge, Alison Road. E 56 383602; N 6406840; (downstream); 14:11 hr; 20/08/2007. Flow approx. 300 ML/d.

Various types of gravel bars are common in the Williams River, particularly upstream of Dungog (Erskine 1998, p. 29). In some confined locations with well vegetated banks, rivers can respond to floods by 'floodplain stripping' on cleared floodplains (which have lower resistance to erosion, and high shear stresses during overtopping events). Erskine (1998 p. 26) noted that floodplain stripping at Brookfield was one of the reasons that river training works were undertaken there.

Figure 21. Williams River at Glen William Bridge, Pine Brush Road. E 56 387191; N 6401215; left is 66° (downstream) showing vegetated island (left side of photograph) and right is 268° (upstream) showing rock bar (left foreground); 14:28 hr; 20/08/2007. Flow approx. 1,400 ML/d.

Figure 22. Williams River at Mill Dam Falls, just downstream of Glen Martin gauge. E 56 387219; N 6397012; 50° bearing (upstream); 15:44 hr; 20/08/2007. Flow approx. 3,200 ML/d.

2.11.4.3 Reach 4: Seaham Weir pool

Bars and benches of coarse-grained sediment (sand to cobble size) are present in the channel at Mill Dam Falls. This is a natural geomorphic break in the system, below which the bed gradient decreases sharply. Mill Dam Falls would have been the natural tidal barrier, and the current Seaham Weir pool extends to a point between Mill Dam Falls and Clarence Town (Figure 23). Thus, this was naturally, and is currently, a zone of deposition. The bed of Seaham Weir pool is probably coarse (gravel and cobble), overlain by fine material.

Figure 23. Williams River at Clarence Town. E 56 385889; N 6393357; 102° bearing (upstream); 19:18 hr; 10/08/2007. Flow 87 ML/d.

Seaham Weir (Figure 24) was constructed in 1968 and later sealed in 1978. The initial construction altered the reach of the river between Mill Dam Falls and the weir, from that of a free flowing tidal estuary to a brackish pool. Later sealing prevented salt water penetrating the pool. The Healthy Rivers Commission of NSW (1996 p. 31) reported that there was anecdotal evidence that construction of the weir caused die-back of phragmites beds lining the lower parts of the banks, and that allowing stock to drink directly from the now freshwater pool resulted in decline of bank vegetation, erosion and degradation of water quality. The Healthy Rivers Commission of NSW (1996 p. 31) referred to a report by Patterson Britton and Partners (1996) that ongoing variations in water levels within the weir pool associated with the operation of the weir gates, cattle access, and the impact of waves generated by some power boats along the river were inhibiting the re-establishment of riparian and aquatic vegetation.

The Healthy Rivers Commission of NSW (1996 p. 33) considered that there was '…substantial and compelling evidence that power boat activity is having a significant detrimental effect on the health of the river in the reach between Clarence Town and 'Kurreki', approximately 3 kilometres downstream.' In other sections of the reach the Commission regarded other factors, such as cattle access and wind waves, to be the more dominant causes of observed erosion.

Figure 24. Williams River at Seaham Weir. E 56 381655; N 6385598; 303° bearing (left to right bank); 16:32 hr; 20/08/2007. Flow approx. 3,300 ML/d.

2.11.4.4 Reach 5: Seaham Weir to Hunter River confluence

On the basis of field inspection, Healthy Rivers Commission of NSW (1996 pp. 36-37) noted that the banks of the Williams River in the estuarine reach (ie Seaham Weir to Hunter River confluence) were:

> '…relatively stable with extensive phragmites beds in the intertidal zone and sparse riparian vegetation. There is substantial scope to improve the condition of the riparian vegetation in this area.
>
> Extensive bank protection works have already been undertaken along the estuary. The Commission believes that any proposals for further significant structural works should be assessed in terms of the assets they are proposed to protect, as well as in terms of what else could be done for the catchment with the same money.'

Thus, in the opinion of the Healthy Rivers Commission NSW (1996), although the riparian vegetation was generally poor, the banks were generally structurally sound, with problem areas where assets were under threat now stabilised by works.

Manly Hydraulics Laboratory (MHL) (2003) examined bank stability of the study reach, within the context of the entire Hunter estuary, where bank erosion has long been a recognised problem due to hydrological changes, riparian vegetation degradation, deposition of flood borne sediment and other factors. MHL (2003) surveyed the banks of the entire estuary using a qualitative, visual approach.

It is important to note that the MHL (2003) methodology included native riparian vegetation as a factor in their assessment of bank stability, based on the assumption that the two were causally linked. MHL (2003 p. 93) noted that '…the presence and absence of native vegetation formed the basis of the categorisation of bank stability…This method of assessment formed the basis of the bank stability assessment conducted by MHL…'. However, this assumption is not always true, and in the case of the Williams River there was one section with healthy native vegetation, but also having apparent bank erosion (near Eskdale Swamp). Also, native vegetation was included as a factor in the stability assessment, but non-native vegetation excluded. This was not based on the superior ability of native vegetation to impart stability, but on a factor unrelated to stability – its presumed superior ecological importance.

Other factors were also considered in the MHL bank stability assessment, but it is clear that the assessment methodology was a mixture of a priori cause and effect factors, not an objective assessment of physical stability or actual erosion rate. MHL (2003) mapped some of the factors that can contribute to accelerated bank erosion – vegetation cover, presence of structural works, and cattle access.

The conclusion of the MHL (2003) survey for the Williams River between Seaham and Raymond Terrace was that bank stability ranged from stable to unstable, with areas of bank undercutting present. MHL (2003) agreed with a previous Sinclair Knight Merz report prepared in 1990 that the main cause of bank erosion was wind and boat wave action and lack of riparian vegetation, resulting from cattle access. These conclusions were based on circumstantial evidence only. MHL (2003) did not identify an acceptable natural or background erosion rate (as it is normal for river banks to erode and accrete), making it difficult to decide if the observed erosion warranted intervention.

GHD (2006) undertook an initial one-day assessment of bank condition of the Seaham Weir to Fitzgerald Bridge (Hunter River confluence) and followed this up with five bank surveys undertaken at 14 selected locations on the river over the period December 2004 to September 2005. The surveys at each location were single transects of one side of the river bank, measured using a surveyor's level. The monitoring locations were 'chosen to allow comparison between areas of different erosion processes, vegetation types and adjacent land uses (GHD 2006 p. iii)…[and]…specifically chosen to monitor key erosion processes identified through the project brief, community consultation and the initial condition assessment' (GHD 2006 p. 16). Despite the monitoring locations being specifically chosen to monitor erosion, these surveys failed to record significant erosion at all but one location. In response, the sampling strategy was relaxed (the scientific validity of doing this is questionable), and the sampling zone was extended to include the river banks in areas '*adjacent to*' the transect locations, as in five cases erosion was apparently evident in these areas. Evidence for this erosion was 'clumps of bank material being deposited into the river channel' GHD (2006. p. iii). It would be a chance occurrence if this material were actually observed to be falling into the river on one or more of the five survey days, so it can be assumed that these data are largely based on the opinion of the observer.

GHD (2006) concluded that a number of processes were contributing to erosion of the banks: recreational boat wake; land and river management practices including the removal of remnant riparian vegetation and adjoining land use including cattle access to the river; wind waves whose impacts may have been increased through land and river management practices by the removal of mature trees from the riverbank; tidal processes that can focus the effects of wave action onto specific areas of the banks as well as providing a process for the removal of bank material; different soil types along the river that responds to different influences; flood flows mobilising bank material that has been left in a vulnerable position due to other processes; river encroachment/migration which is a natural process of any river system; and the construction of the Seaham Weir, which would have reduced the volume of sediment transported into the tidal reach of the Williams River.

Of the erosion processes listed by GHD (2006) only two relate to flows in the river from upstream. The first is flood flows, but these were not well investigated, as no major flood events occurred during the monitoring period (GHD 2006 p. 23) and the second is assumed sediment trapping by Seaham Weir.

It can be concluded that while river flows are an important component of the erosion processes downstream of Seaham Weir, it is a secondary role, in the sense that flows occasionally (during high flow events) provide the energy to remove material eroded or 'prepared' for removal by other processes. Any change to the mean suspended load of the river would be inconsequential to bank and bed erosion as the concentrations are too low have any significant impact on the capacity of the flowing water to scour. Large floods pass the Seaham Weir through open gates, so the major periods of sediment transport are relatively unaffected by the Weir.

2.11.5 *Sediment transport*

For a site near Munni (Reach 2), Brooks et al (2004) calculated the reach-averaged critical bed material entrainment threshold using the measured median particle size and the Ackers–White sediment transport equation and found it to be equivalent to the mean annual flood (ie 14,688 ML/d by his estimation). Erskine (1998 p. 14) applied Neill's (1968) threshold of motion criterion to the mean bed material size at Tillegra gauge (Figure 16) and found that events '…smaller than the mean annual flood are competent to transport bed material.' Brooks et al (2004) pointed out that the relative geomorphic effectiveness of flood events that exceed the entrainment threshold depends on the length of time that the hydrograph is above the threshold. Follow-up monitoring of the experiment of Brooks et al (2004) by Brooks et al (2006) revealed that the channel geomorphology adjusted measurably over the period of 5 years with significant increases in channel bed material storage, and increased pool and bar formation. Although the absolute channel change was measurable, it represented only an estimated 2 percent reduction in cross-sectional area of the channel.

Brooks et al (2006) suggested that the transport capacity of the channel at Munni was well in excess of that which could be sustained by the long-term sediment yield from the catchment (ie it was supply limited). This means that the channel was actively transporting sediment at a rate that exceeded the supply, in which case ongoing morphological adjustment (expansion of the channel) would be expected.

The Healthy Rivers Commission of NSW (1996 p. 32) noted the existence of differing views on the efficiency of Seaham Weir pool (Reach 4) as a sediment trap. At Seaham, the sediment load of the river would be principally fine-grained, and during flood events most of this material would be suspended in the water column and thus would be transported over the Weir. Healthy Rivers Commission of NSW (1996 p. 32) indicated that:

> 'Recent survey work undertaken by Hunter Water Corporation indicates only minor accumulation of fine material in the vicinity of the weir. The Hunter Water Corporation considers that the weir has little, if any, effect on the hydraulic performance of floods and the associated scouring of sediments. This view is based on the assumption that the weir acts like a barrage and is effectively drowned out during floods. The downstream water levels, which are also influenced by tides, provide the primary hydraulic control.'

In relation to Reaches 4 and 5, GHD (2006 p. 46) stated that the assumed trapping of sediment in Seaham Weir:

> '…has the potential to increase the sediment transport capacity of flows

downstream of the weir resulting in increased erosion rates within the estuarine reach. This downstream reach will continue to be subjected to increased erosion rates (compared to the rates prior to the construction of the Seaham weir) until the sediment carrying capacity of the overflows of the weir reflect a saturated sediment load.'

The assumption of significant sediment trapping is questionable. Even if Seaham Weir did act as a significant sediment trap, GHD (2006) appear to have misunderstood the basic process of sediment scour below structures. In such cases, where scour is observed, it is because of the lack of inflowing bedload to replace that which has been transported out of the downstream reach, rather than the stream having more capacity to erode because the water is lower in suspended sediment concentration. Most Australian rivers (including the Williams River) are sediment supply limited, and it would be unusual for sediment load to be at a 'saturated' (ie capacity) level.

The National Land and Water Resources Audit (NLWRA) modelled bedload transport for the Williams River, but did not explicitly provide the estimate in the Data Library. The Library did provide information on estimated bank erosion rate and gully erosion rate (the two sources of bedload considered), and adopting the Audit's assumption that 50 percent of erosion in both of these environments constituted bedload and 50 percent constituted suspended load, plus other assumptions (Prosser et al 2001a), the Audit's bedload can be calculated. For the Williams River at the Chichester River junction, the Audit's estimate for average bedload transport was 5,000 tonnes per year.

The National Land and Water Resources Audit (NLWRA) estimate of the average annual suspended sediment transported by the Williams River upstream of the Chichester River confluence was 4,600 tonnes per year.

2.12 Sampled bed material particle size distributions

At each sampled site the bed material on riffles was loosely arranged (not embedded), and minimal fines were released upon extracting stones from the bed. The stones were free of algae and diatoms (i.e. no appreciable biofilm). The bed was clearly well flushed of fines and mobile under the current regime. Particles less than 5 mm were rare on riffle crests. They could be found by searching in the lee of larger stones and by probing under the surface layer, but they were not included in any of the random surface samples. These fine and very fine gravels were uncommon in the riffle crests at sites S7, S8 and S9, but were more common at S10, S11 and S12. By mass and surface area, this very fine-grained fraction contributed only a tiny proportion of the bed material. Site S11 had significant sand deposits present on the lower bank of the inside bend, but this was not sampled as part of the bed material.

The sampled bed material in the Williams River covered the size classes Fine Gravel to Large Cobble, but at all sites except S9 the median fell into the Very Coarse Gravel size class (32 – 64 mm) (Figure 25, Table 6). Site S9 had notably coarser bed material. This site had been recently modified by construction of grade control structures upstream and downstream of the riffle. It likely that in the recent past the bed at this site was degrading (ie providing the prompt for installation of the structures), with the current coarse particle size being an artefact of the earlier period when shear stresses were higher.

A lobe of bed material recently deposited over the top of a point bar was sampled at S12 B (Figure 25). This material was much finer than the majority of material elsewhere on the bed of the river (Figure 25). It is likely that bed material undergoes sorting during transport in response to the spatially and temporally varying hydraulic conditions. Thus it was possible to find deposits of bed material both finer and coarser than that sampled at the riffle crests. It was considered impractical to sample all of these different hydraulic environments, so the bed material mobilisation and transport estimates were based on the material found on riffle crests.

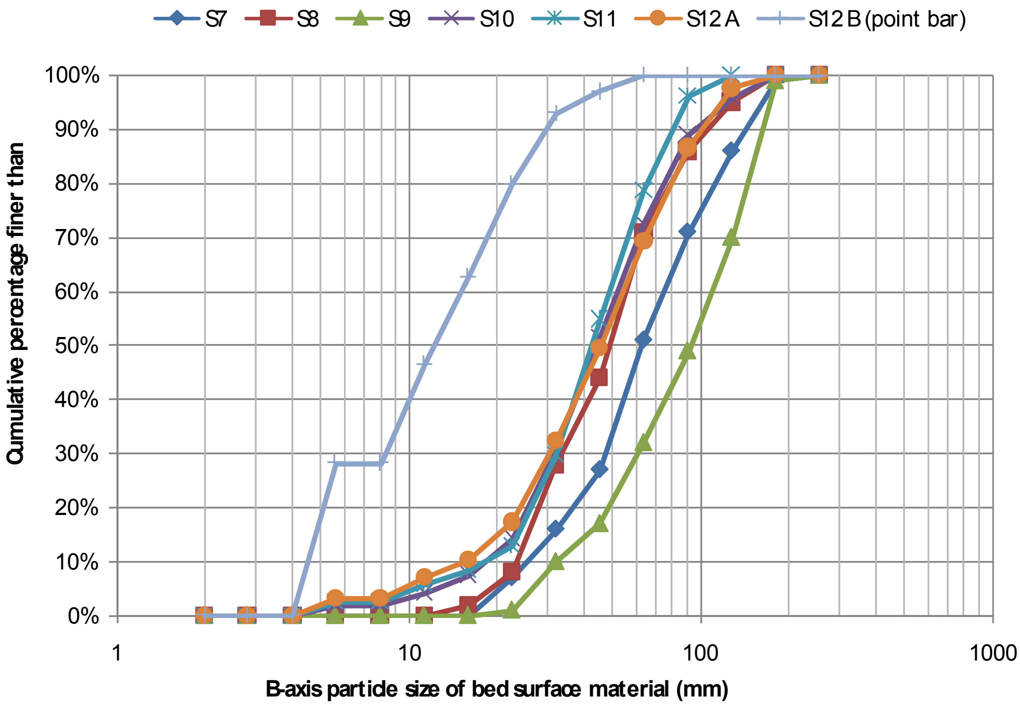

Figure 25. Measured bed material particle size distributions, based on B-axis size of 100+ particles sampled from riffle crests. Sample S12 B was sampled from a point bar and represented a lobe of finer-grained, recently deposited, material, while S12 A was from the riffle crest. Sampling site denoted S12 was located at Glen William, 6.7 km upstream of the site designated S12 for the purposes of hydraulic modelling (Glen Martin), but the measured particle size at Glen William was judged to be representative of the reach down to Glen Martin.

Table 6.
Range and median particle size of bed material sampled on riffle crests in the Williams River. Size classes are according to the Wentworth Scale.

Site	Range of material	Median material
S7	Coarse gravel – Large cobble	Very coarse gravel
S8	Medium gravel – Large cobble	Very coarse gravel
S9	Coarse gravel – Large cobble	Small cobble
S10	Fine gravel – Large cobble	Very coarse gravel
S11	Fine gravel – Small cobble	Very coarse gravel
S12	Fine gravel – Large cobble	Very coarse gravel

2.13 Modelled coarse sediment load

2.13.1 *A note on the limitations of the method*

A prediction of daily coarse sediment flux (bedload transport) was made for sites S7 to S12. The modelled bedload transport rates varied with the hydraulic characteristics of each site, the range of sediment sizes measured in the riffles at each site, and the daily discharge series'. The results should be considered as a sample of the range of possible bedload transport rates in the river. While a downstream pattern in bedload transport rate was apparent in the data, a detailed characterisation of this pattern would require consideration of more than six sites.

Daily bedload transport rates were predicted at each cross-section identified as a riffle in the hydraulic model results (provided by Connell Wagner). Riffle cross-sections at the downstream boundary were excluded from the analysis as their hydraulic characteristics are unduly influenced by modelling assumptions (e.g. normal depth).

The results presented in this section report the estimated annual average bedload transport rates at each site. While there was some intra-site variability (different rates at different cross-sections), the variability in the annual transport rate was far greater.

For convenience, bedload transport rate predictions for both the 'current' and the base case 'with dam' flow series' are presented in this section. A plot of annual bedload transport for each scenario is provided to allow comparisons to be drawn between the two cases. To assist the comparison, the mean transport rate of the entire period of record (1931 – 2007) is also listed. While the estimates of bedload transport rate were based on the best available methodology, the results contain uncertainty. Even though there is uncertainly in the absolute values of sediment transport, the relative differences between the current and base case 'with dam' scenario are considered to be sufficiently reliable on which to base management decisions.

2.13.2 *Predicted coarse sediment load at six sites*

Bedload transport rates for sites S7 to S12 are presented in Figure 26 to Figure 31. To facilitate comparison between the 'current' and base case 'with dam' scenarios the scales on the left and right plots are identical. Note that the units of bedload transport at each site is given in thousands of tonnes per annum (kt/annum), with the exception of Site 10 which is in tonnes per annum. Transport rates at Site 10 are different from each of the other sites due to the very low water surface slopes simulated through the reach (the energy slope ranged from 1 in 10,000 to 1 in 1,000, compared to the other sites which tended to be an order of magnitude higher in slope). Consequently, the capacity for flow to transport material was predicted to be substantially lower at Site 10.

Inter-annual variability in sediment transport was high, with the maximum annual transport tonnage typically between 5 and 10 times higher than the mean annual transport rate. For example, at Site 7 the maximum annual transport rates under the 'current' and 'with dam' scenarios were about 15 and 10 kt/annum respectively, while the mean annual transport rates were 3 and 1 kt/annum respectively.

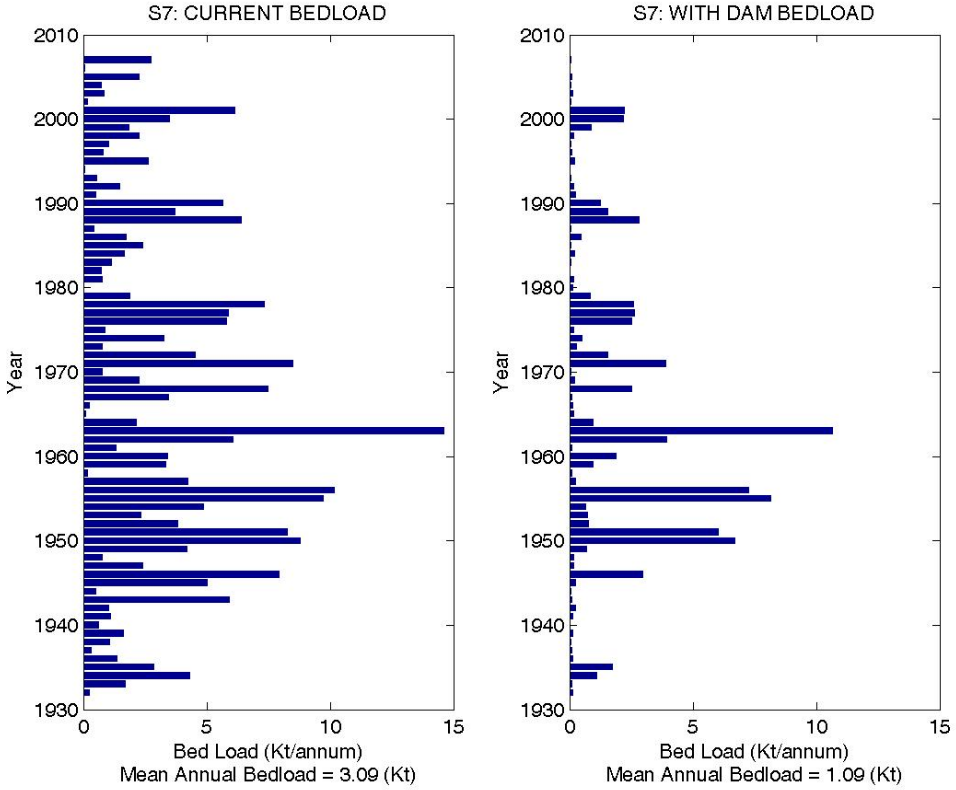

Figure 26. Predicted annual bedload transport series' and mean annual transport rate over the period of record for 'current' and 'with dam' flow scenarios at Site S7.

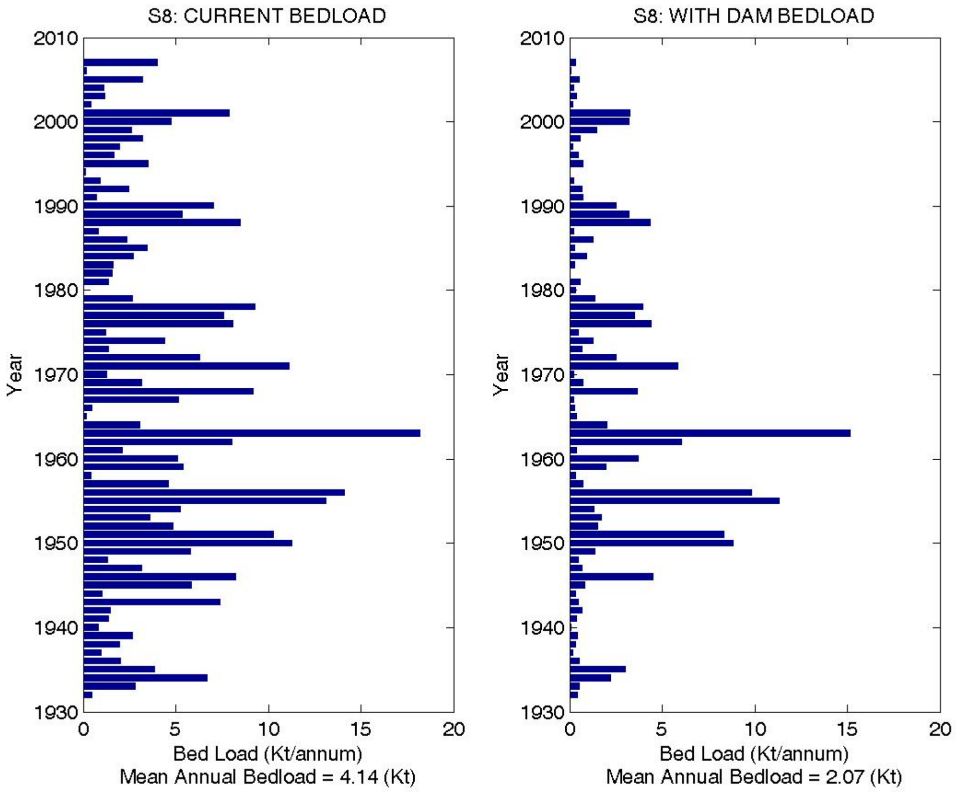

Figure 27. Predicted annual bedload transport series' and mean annual transport rate over the period of record for 'current' and 'with dam' flow scenarios at Site S8.

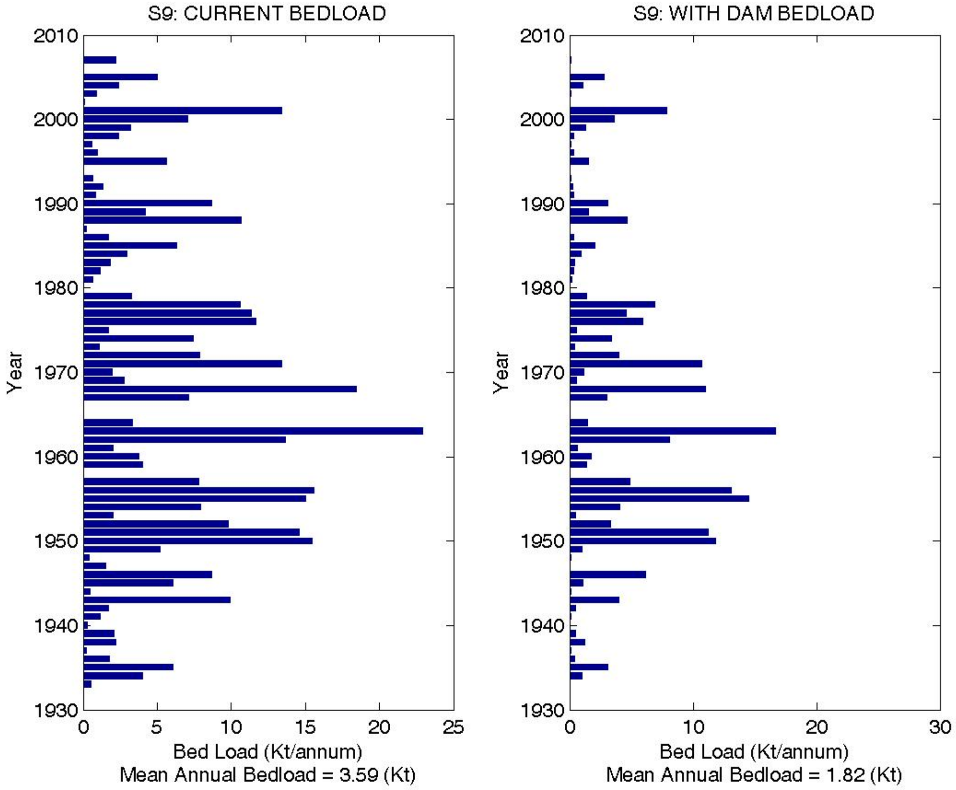

Figure 28. Predicted annual bedload transport series' and mean annual transport rate over the period of record for 'current' and 'with dam' flow scenarios at Site S9.

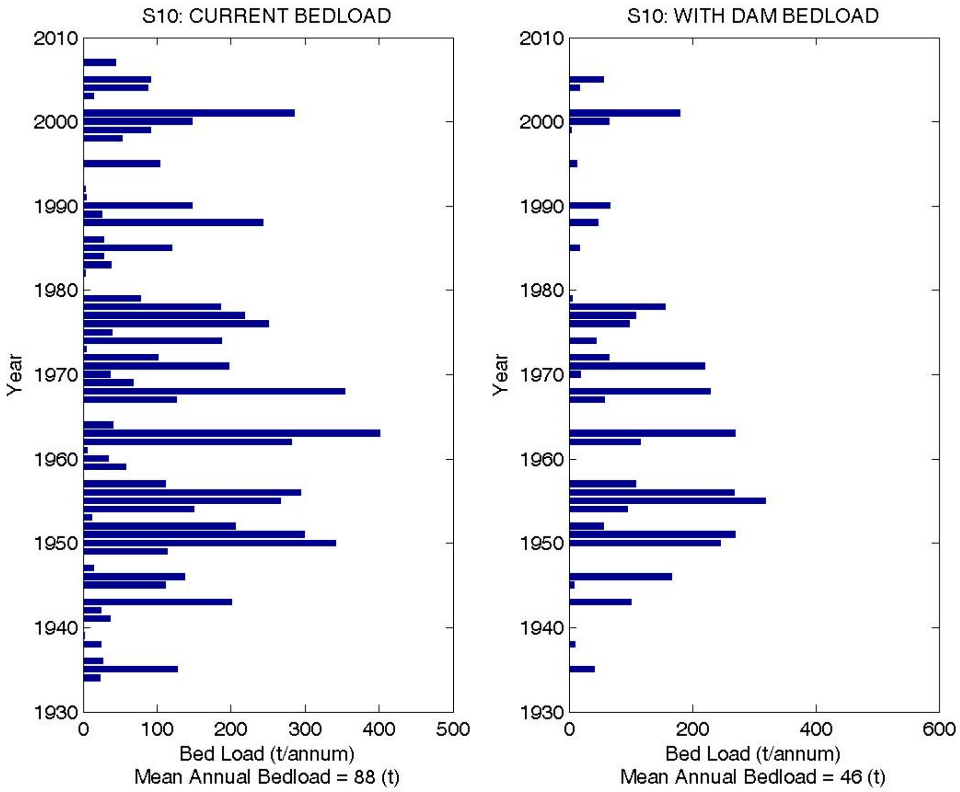

Figure 29. Predicted annual bedload transport series' and mean annual transport rate over the period of record for 'current' and 'with dam' flow scenarios at Site S10.

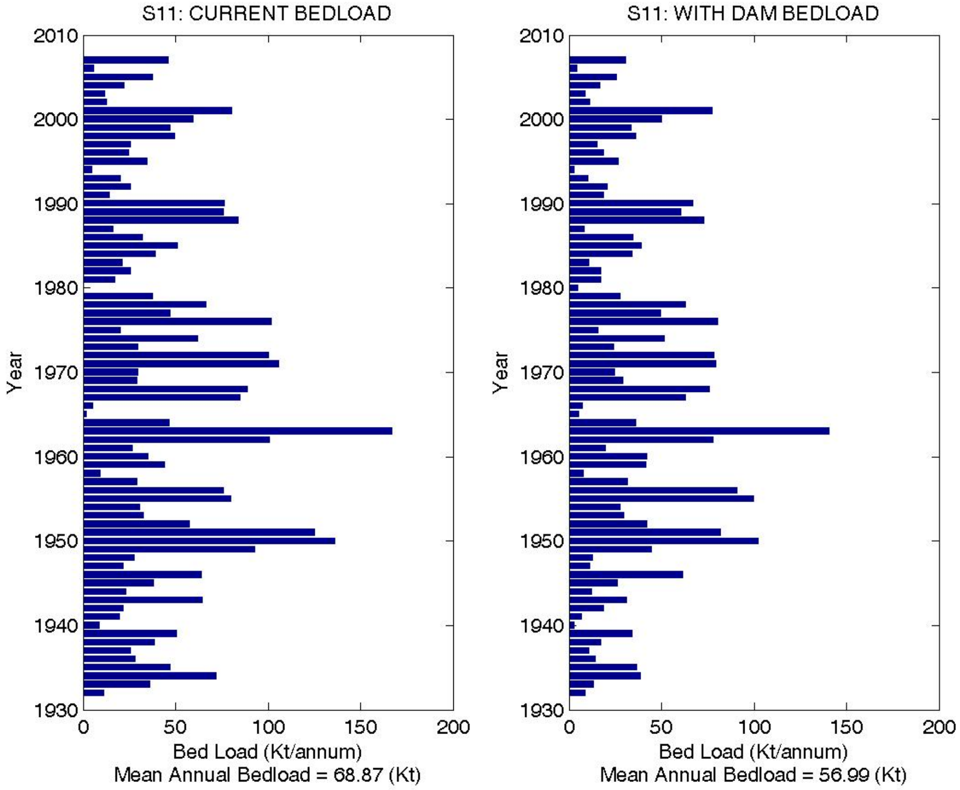

Figure 30. Predicted annual bedload transport series' and mean annual transport rate over the period of record for 'current' and 'with dam' flow scenarios at Site S11.

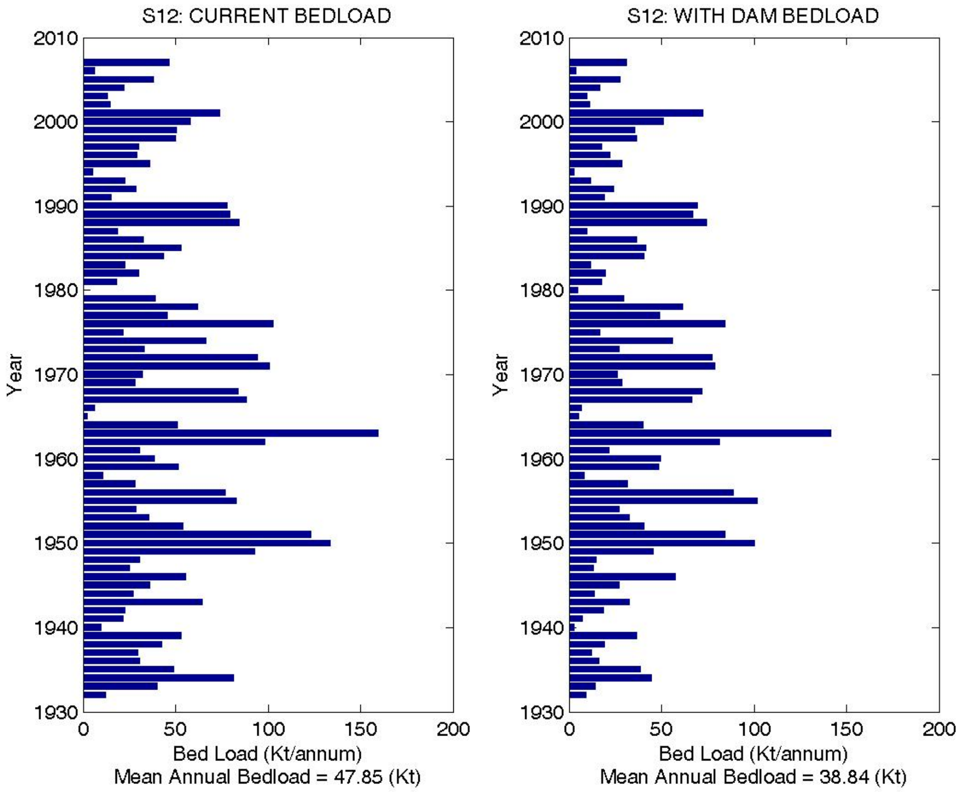

Figure 31. Predicted annual bedload transport series' and mean annual transport rate over the period of record for 'current' and 'with dam' flow scenarios at Site S12.

Bedload transport rates were lower under the base case 'with dam' flow scenario than the 'current' scenario in every case. The greatest proportional reduction in transport was predicted at Site 7, with the mean annual transport rate declining by a factor of 3. Sites 8, 9 and 10 also showed large reductions in bedload transport, reducing by a factor of 2 in each case (although the results at Site 10 must be interpreted with caution due to the nature of the hydraulics at this site). Near the downstream end of the system the predicted impact of the base case 'with dam' scenario was less, with Sites 11 and 12 maintaining about 80 percent of the 'current' bedload transport under 'with dam' flows.

2.14 Modelled suspended sediment load

2.14.1 *A note on the limitations of the method*

It is important to note that the methods applied to compute suspended sediment loads must be considered first order approximations. The principal uncertainty in the approach is caused by the limited event-based water quality (*TSS*) data with which to establish a *TSS* versus discharge relationship. Ideally, relationships for each of the three hydrological zones (as defined) would be available. The dam trapping efficiency relationship is considered adequate for the purpose of this analysis, but could be improved if required. However, in the absence of more detailed water quality data, the adopted method is considered to be the most suitable representation of the key processes that influence suspended sediment load.

2.14.2 *Predicted suspended load at three locations*

A suspended solids concentration series was modelled at each of three 'hydrological zones':

- the Williams River from Tillegra Dam site to the Chichester River junction (S7 and S8);
- the Williams River just downstream of the confluence with the Chichester River (S9 and S10); and
- the Williams River near the Glen Martin gauge (S11 and S12).

The concentration of suspended solids was factored to account for the trapping efficiency of the Chichester Dam in both the 'current' and base case 'with dam' scenarios, and the Tillegra Dam under the base case 'with dam' scenario only. Following this factoring, the product of the suspended solids concentration series and the mean daily discharge series gave the sediment load time series, enabling calculation of the annual load. For convenience, suspended solids load predictions for both the 'current' and the 'with dam' flow series' are presented in this section. Annual loads are presented for each of the three 'hydrological zones' in Figure 32, Figure 33 and Figure 34.

The result for the period of discharge records suggests that the Williams River at Tillegra under the current flow regime conveys, on average, almost 10,000 tonnes of suspended load per year, with marked inter-annual variation (Figure 32). Under the base case 'with dam' scenario, this load was predicted to drop to only 140 tonnes of suspended load as a consequence of trapping of sediment within the dam.

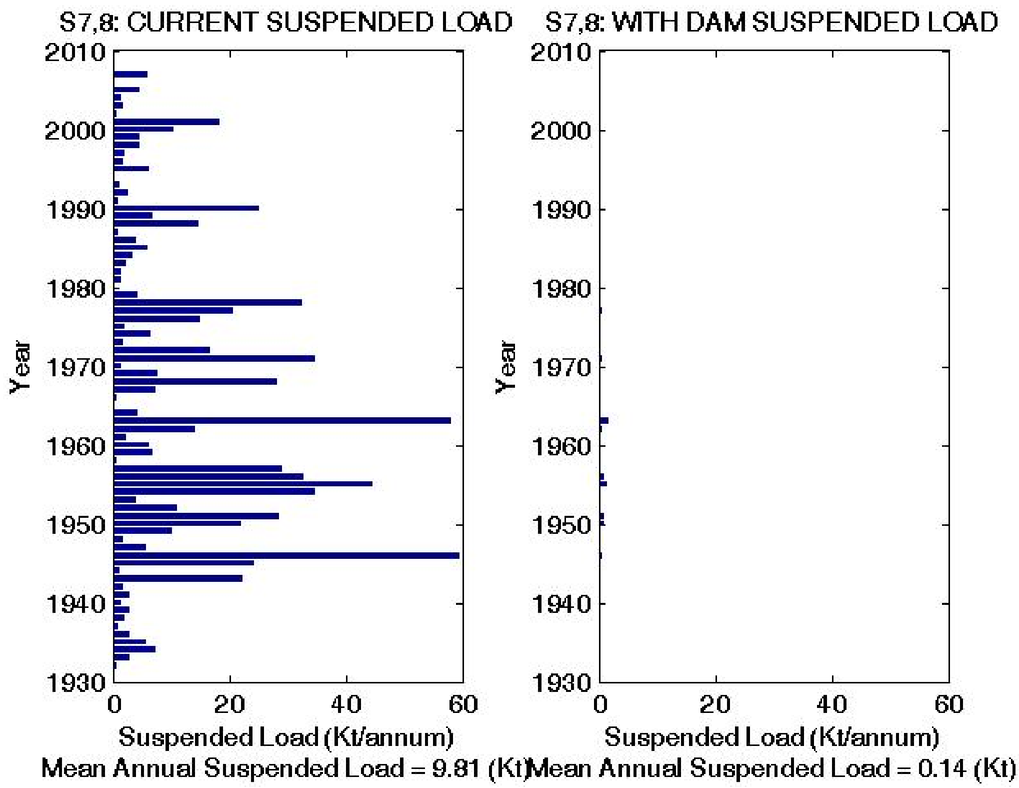

Figure 32. Estimated suspended sediment load (thousands of tonnes per annum) for the Williams River at the Tillegra Dam site.

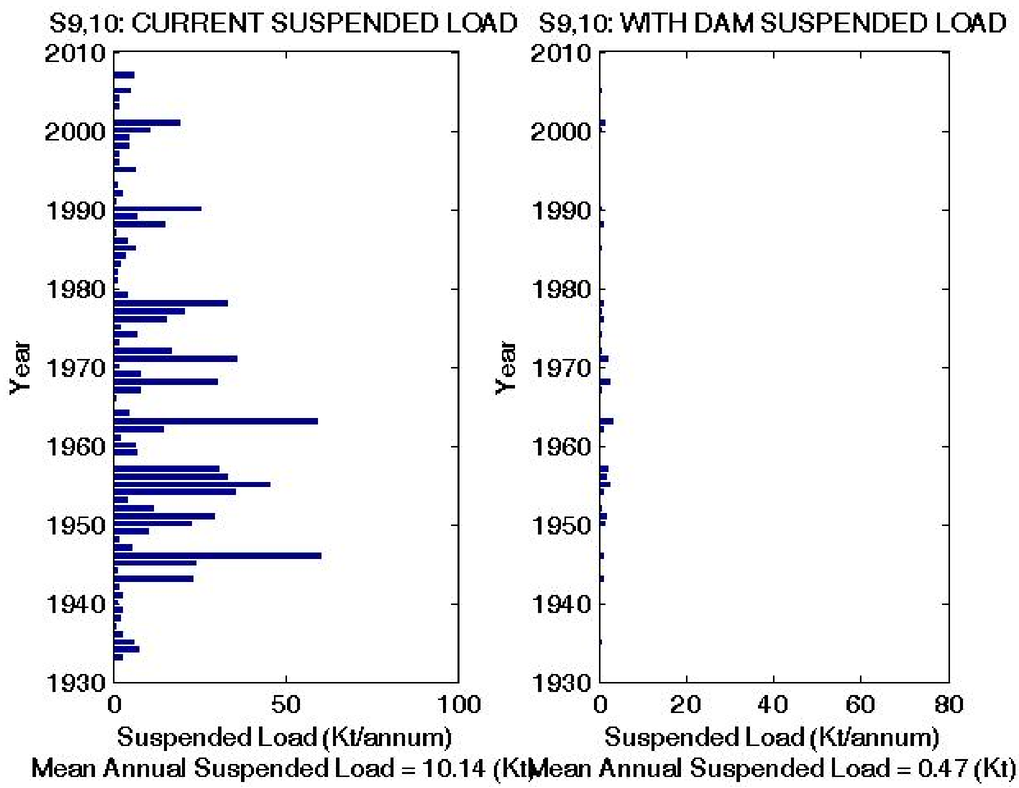

Figure 33. Estimated suspended sediment load (thousands of tonnes per annum) for the Williams River at Sites 9 and 10.

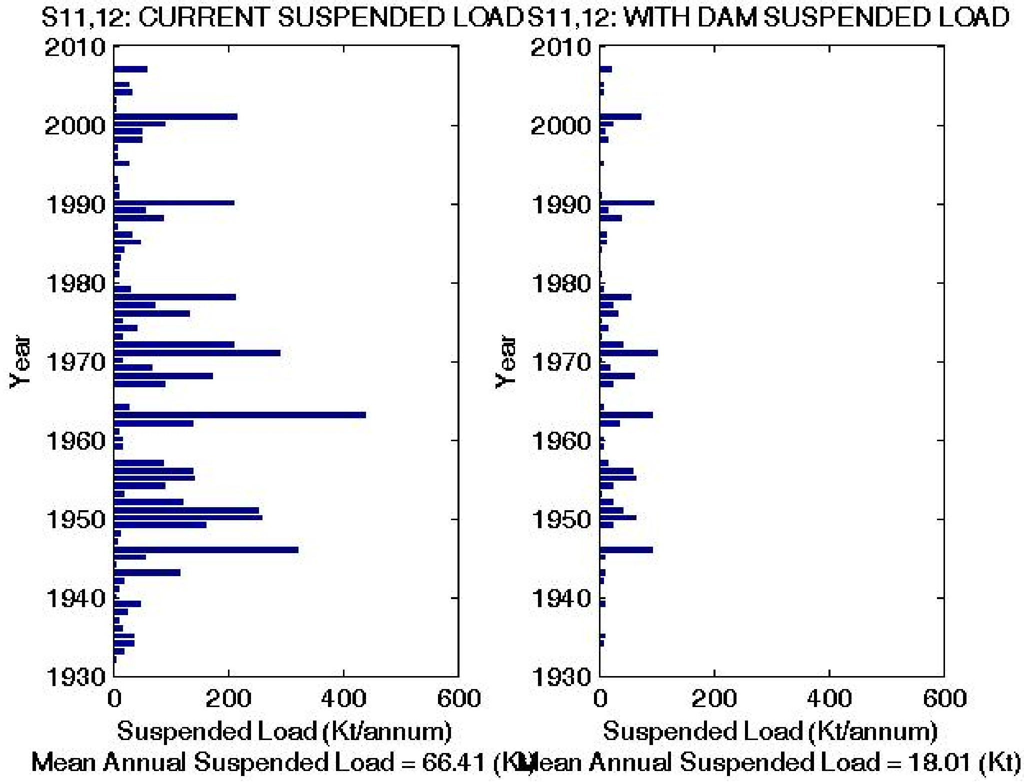

Figure 34. Estimated suspended sediment load (thousands of tonnes per annum) for the Williams River at Sites 11 and 12

The current suspended load at downstream of the confluence with the Chichester River is similar to that at Tillegra, being around 10,000 tonnes per annum (Figure 33). The similarity in the loads is due to the assumption that Chichester Dam traps 88 percent of the suspended load. Under the 'with dam' case the reduction in suspended sediment is again remarkable, dropping to only 470 tonnes per annum.

By the time the river reaches Glen Martin it picks up significant additional suspended load (Figure 34). Under the 'current' scenario the predicted load is more than 6 times higher than at the Tillegra Dam site (at 66.4 thousand tonnes per annum). Under the 'with dam' scenario this would drop to less than 30 percent of the 'current' load, due to the effect of both sediment trapping and discharge reductions. It is also noteworthy that in the 'current' scenario, the mean annual suspended load was greater than the bedload at S11 and S12, yet following the construction of Tillegra Dam, bedload was predicted to become the dominant proportion of the total load. This change in the balance of the loads may have ecological consequences.

2.15 Discharge event frequency (current scenario)

Flood frequency analysis was undertaken to estimate the discharge associated with a range of average recurrence intervals (ARIs) under the current scenario (Table 7, Figure 35). For each ARI the discharge was reported as both the mean daily and the peak instantaneous daily value. These relationships enabled calculation of the average recurrence interval corresponding to the defined geomorphic process and form thresholds.

Table 7.
Discharges (ML/d) corresponding to various flow duration indices. Discharge for current scenario. Values represent 2nd order polynomial fitted to annual exceedance series for ARI <10 years, and for ARI ≥10 years, Generalised Pareto (Tillegra and Glen Martin) or Johnson's SB (DS Chichester junction) distributions fitted to annual maximum series. Peak daily values are factored mean daily values.

	Tillegra (S7 and S8)		Downstream of Chichester River (S9 and S10)		Glen Martin (S11 and S12)	
ARI (years)	**Mean daily**	**Peak daily**	**Mean daily**	**Peak daily**	**Mean daily**	**Peak daily**
0.2	1,450	2,350	2,350	3,300	4,800	6,800
0.3	2,250	3,650	3,700	5,200	7,550	10,750
0.4	3,050	4,800	5,000	7,000	10,200	14,500
0.5	3,850	5,950	6,250	8,800	12,750	18,150
0.6	4,600	7,050	7,450	10,500	15,150	21,600
0.7	5,300	8,050	8,600	12,100	17,450	24,850
0.8	5,950	9,050	9,700	13,650	19,650	28,000
0.9	6,650	10,000	10,750	15,150	21,750	31,000
1	7,250	10,900	11,800	16,600	23,800	33,850
2	12,550	18,300	20,300	28,600	40,150	57,100
3	16,650	23,850	26,750	37,700	52,050	74,050
4	19,900	28,300	31,950	45,000	61,300	87,200
5	22,700	32,000	36,250	51,100	68,750	97,850
10	30,200	41,950	52,100	73,450	85,000	120,950
15	35,500	48,850	59,700	84,150	96,100	136,750
20	39,400	53,950	64,200	90,500	103,500	147,300
25	42,400	57,800	67,200	94,750	108,900	155,000
30	44,900	61,050	69,400	97,850	113,200	161,100
35	47,000	63,700	71,100	100,250	116,700	166,100
40	48,900	66,150	72,400	102,050	119,700	170,350
45	50,500	68,200	73,500	103,600	122,200	173,900
50	52,000	70,100	74,400	104,900	124,400	177,050
75	57,700	77,350	77,300	108,950	132,600	188,700
100	61,800	82,550	79,000	111,350	138,000	196,400

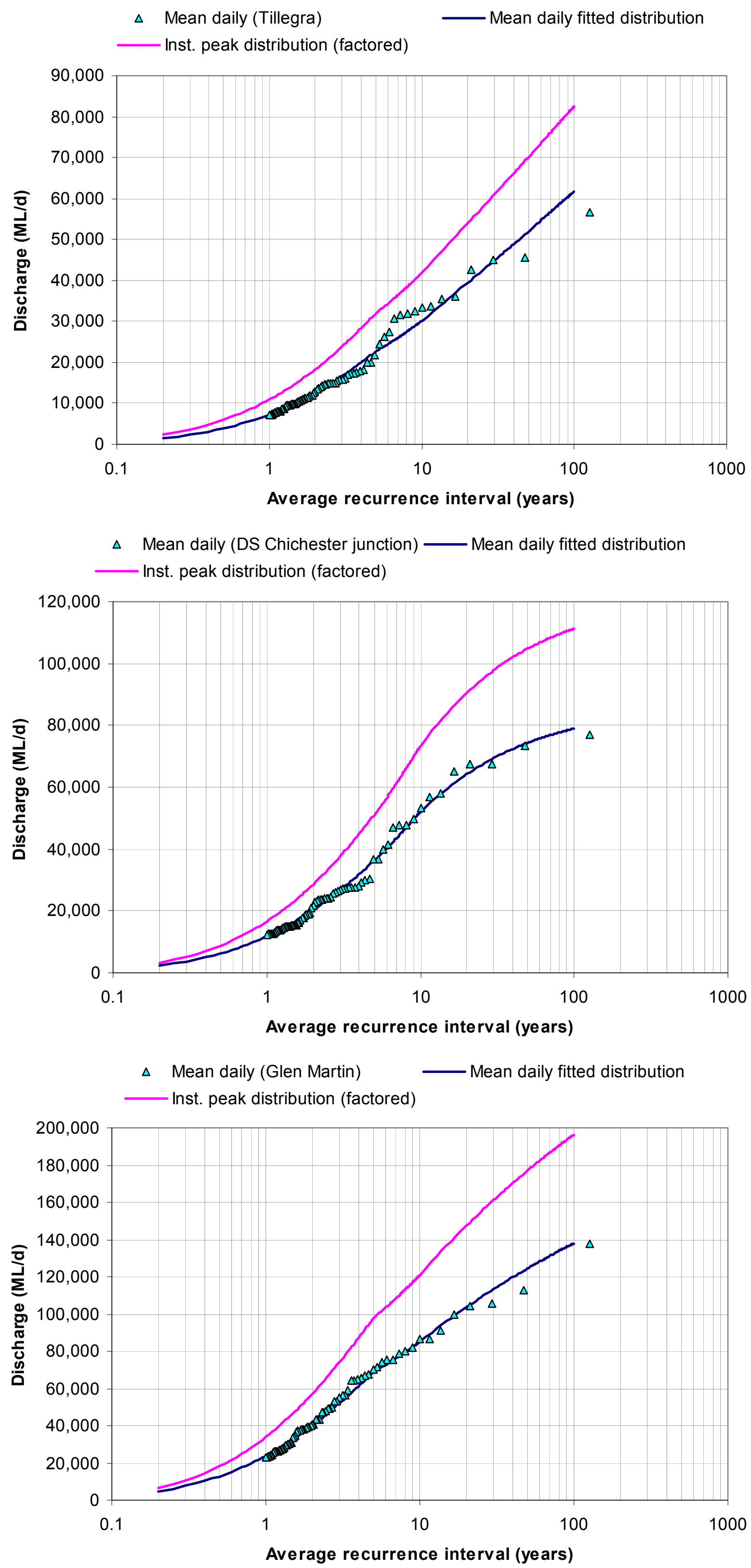

Figure 35. Flood frequency distributions for the Williams River, current scenario. Flood data plotted using the Cunnane plotting position formula (α = 0.4). Curves are polynomial fitted to annual exceedance series for ARI <10 years, and for ARI ≥10 years, Generalised Pareto (Tillegra and Glen Martin) or Johnson's SB (DS Chichester junction) distributions fitted to annual maximum series.

2.16 Geomorphic settings of the sampled sites

2.16.1 Site S7

Site S7 was located just downstream of the proposed Tillegra Dam site (Figure 36). The left bank of the upstream end of the site abutted the valley wall while the right bank was set in a narrow band of alluvial material at 90 – 92 m AHD elevation. At the lower end of the site the left bank emerged onto a narrow alluvial surface 89 – 90 m AHD elevation, while the right bank abutted a low and gently sloping valley surface.

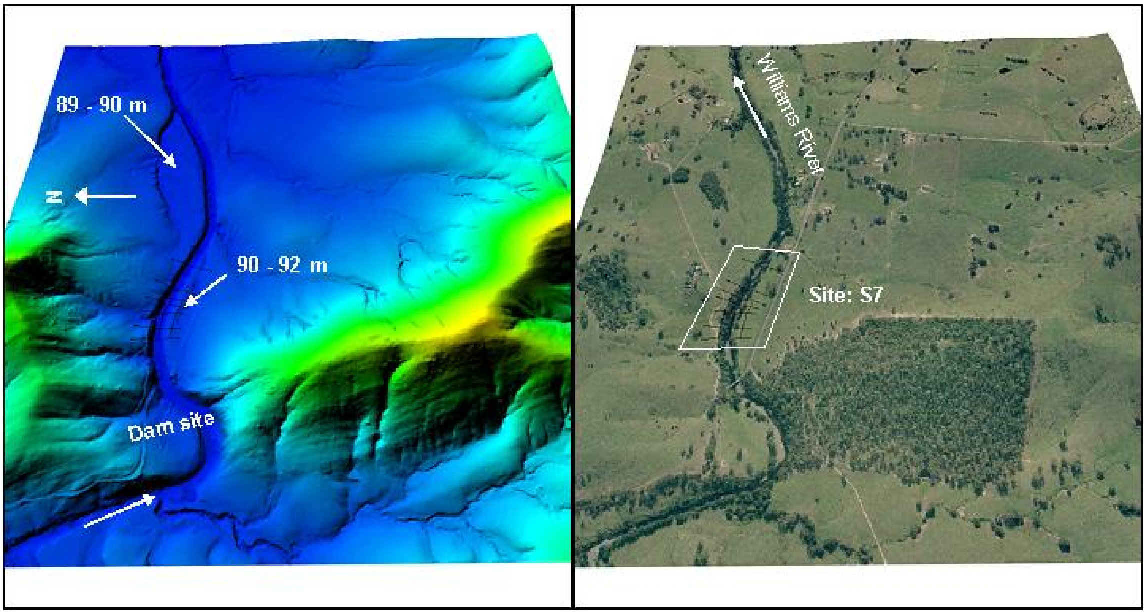

Figure 36. Topography and land use in the vicinity of S7. View is downstream. Images are LiDAR generated DEM and aerial photograph draped over the DEM surface.

2.16.2 Site S8

Site S8 was located just upstream of the Chichester River junction (Figure 37). The site was located just downstream of a tightly confined meander bend. At the lower end of the site the river emerged onto a broad alluvial surface at 80 – 82 m AHD on the left side, while it remained confined by the valley wall on the right bank.

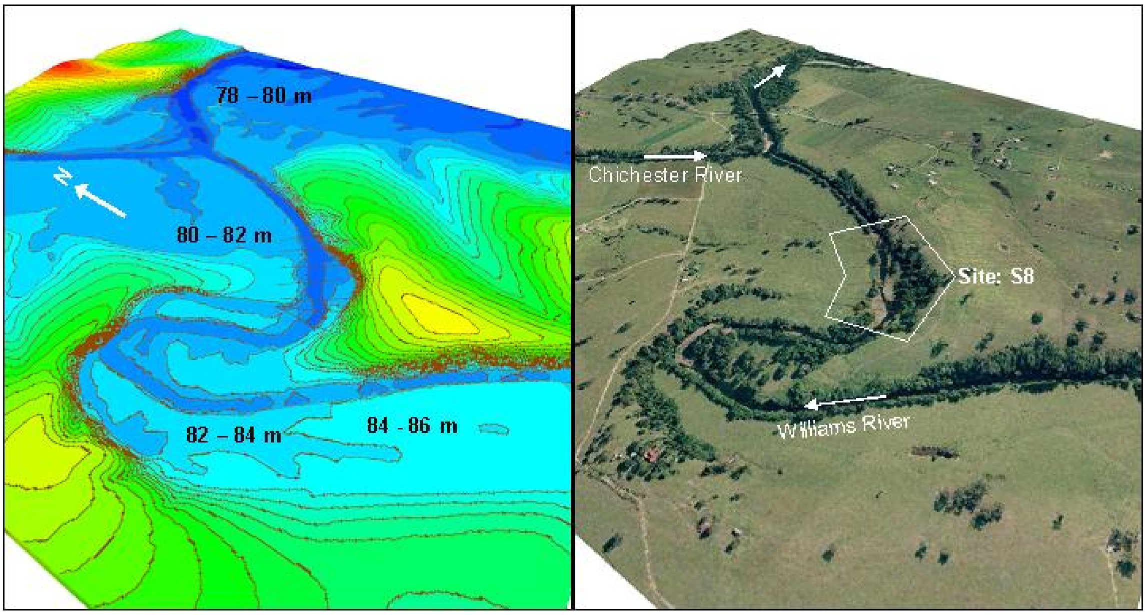

Figure 37. Topography and land use in the vicinity of S8. View is downstream. Images are LiDAR generated DEM and aerial photograph draped over the DEM surface.

2.16.3 Site S9

Site S9 was located downstream of the Chichester River junction on a section of floodplain characterised by breakout channels on the right and left floodplain surfaces (Figure 38). The upper part of the site was on a left bank point bar, while the lower part of the site was on a right bank point bar, with the left bank abutting a steep valley wall. Two grade control structures had been recently constructed on this bend, and these formed the upper and lower bounds of the site.

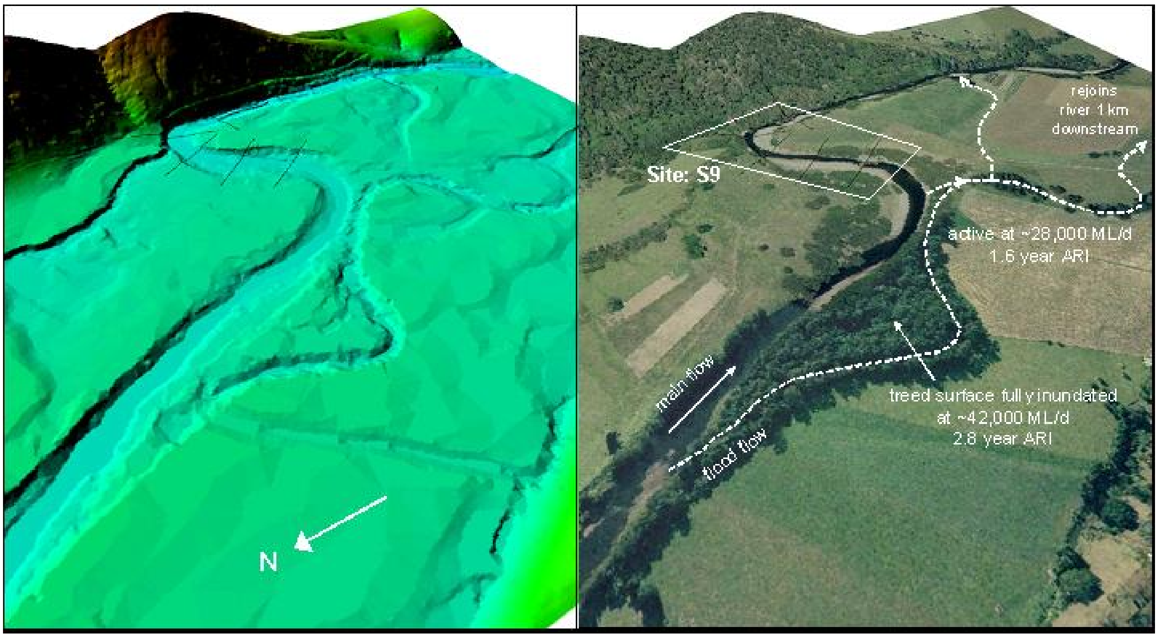

Figure 38. Topography and land use in the vicinity of S9. View is downstream. Images are LiDAR generated DEM and aerial photograph draped over the DEM surface.

2.16.4 *Site S10*

Site S10 was located on a river bend between the Chichester River junction and Dungog (Figure 39). The floodplain was one to two metres lower on the left bank than the right bank The upper part of the site was unconfined, while the lower part of the site abutted a steep valley wall on the left bank. A small tributary entered from the left midway through the site.

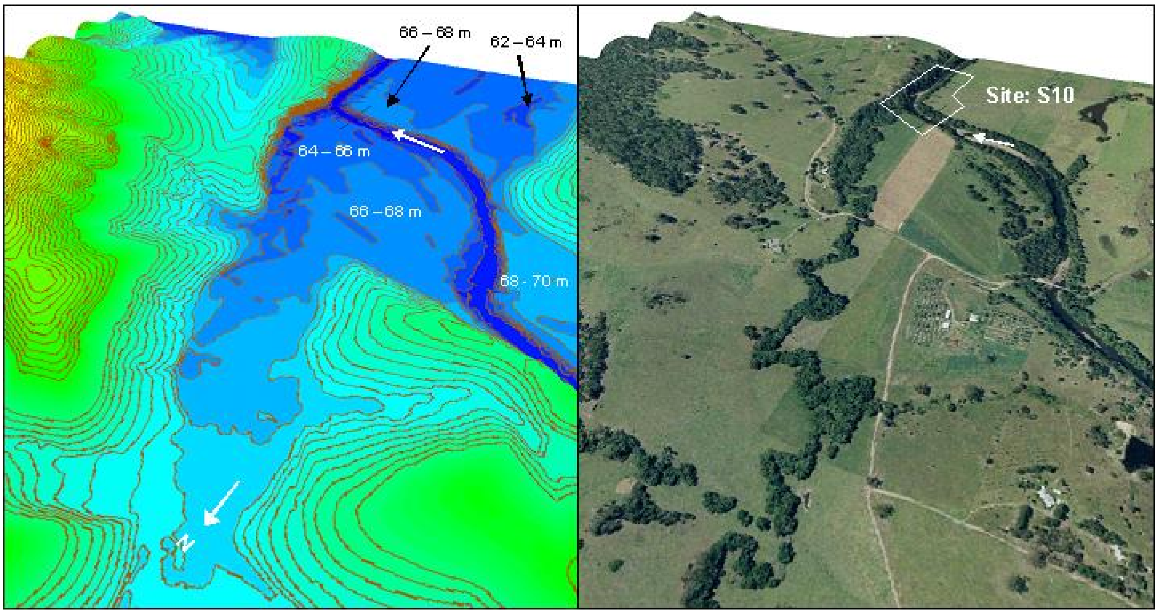

Figure 39. Topography and land use in the vicinity of S10. View is downstream. Images are LiDAR generated DEM and aerial photograph draped over the DEM surface.

2.16.5 *Site S11*

Site S11 was located in the vicinity of the Thalaba Bridge, on Alison Road. The site was on a tight meander bend, with both left and right banks in unconfined floodplain. There was a small break out channel cutting across the meander bend. This channel is actively incising into the floodplain and there is a chance that this will become the main course of the river in the not too distant future. This site was not within the area covered by LiDAR.

2.16.6 *Site S12*

Site S12 was located from the rock bar at Mill Dam Falls, upstream through the long pool formed by the falls, to the next upstream gravel/cobble riffle (also with rock outcrops). The downstream end of the site is just upstream of the upper end of the Seaham Weir Pool. This site was not within the area covered by LiDAR.

2.17 Geomorphic process discharge thresholds

2.17.1 Event duration criterion

In order for the geomorphic processes considered here to be effected, flow needs to be above threshold for a certain time. There is little in the way of empirical or theoretical information available on which to base a minimum duration for these processes to be effective, but the choice here is between instantaneous or days (ie the available data do not allow interpretations to be made at the sub-daily time scale). For the selected processes it was decided that a minimum duration of 1 day was adequate, so the appropriate flood frequency relationships (Table 7, Figure 35) are those based on mean daily data.

2.17.2 Site S7

Site S7 was located 200 m downstream of Tillegra Dam site (Reach 3). The site contained pool and riffle sections. Bed material had relatively high mobility, with the majority becoming mobile at most modelled transects in the 1 in 5 year flood (Table 8). Macrophytes were predicted to be regularly disturbed, but grasses and shrubs were not. Silt was predicted to be readily flushed from riffles and pools.

Table 8.
Discharges (ML/d) corresponding to exceedance of geomorphic process thresholds for Site S7. Maximum modelled discharge was 82,000 ML/d and minimum modelled discharge was 47 ML/d.

Section	Pool/ riffle	Bed material mobilisation			Macrophyte disruption		Grass/shrub disturbance		Surface silt flushing
		D_{16}	D_{50}	D_{84}	Q_m^{95}	$Q_m^{99.9}$	80 N/m^2	150 N/m^2	0.5 m/s
7.7	Riffle	6,083	9,828	16,580	267	3,146	19,762	64,954	<47
7.6	Pool	12,145	17,264	25,123	177	2,826	28,609	71,066	1,229
7.5	Riffle	372	1,104	2,410	101	3,844	>82,000	>82,000	<47
7.4	Pool	26,150	40,301	>82,000	247	3,580	>82,000	>82,000	1,100
7.3	Riffle	14,098	24,414	40,085	209	3,721	49,328	>82,000	<47
7.2	Pool	9,052	12,575	18,256	194	2,413	21,676	>82,000	802
7.1	Riffle	1,477	2,382	4,050	262	3,172	5,133	15,595	<47

2.17.3 Site S8

Site S8 was located just upstream of Chichester River junction (Reach 3). The site contained three riffle sections, one short pool and one long pool. Bed material had high mobility in the riffles, but not the pools (Table 9). Macrophytes were predicted to be regularly disturbed, but grasses and shrubs were not. Silt was predicted to be readily flushed from riffles and pools.

Table 9.
Discharges (ML/d) corresponding to exceedance of geomorphic process thresholds for Site S8. Maximum modelled discharge was 82,000 ML/d and minimum modelled discharge was 47 ML/d.

Section	Pool/ riffle	Bed material mobilisation			Macrophyte disruption		Grass/shrub disturbance		Surface silt flushing
		D_{16}	D_{50}	D_{84}	Q_m^{95}	$Q_m^{99.9}$	80 N/m^2	150 N/m^2	0.5 m/s
8.7	Riffle	291	538	881	193	4,194	5,702	29,921	<47
8.6	Pool	>82,000	>82,000	>82,000	263	8,633	>82,000	>82,000	1,012
8.5	Riffle	375	749	1,346	300	5,039	>82,000	>82,000	<47
8.4	Pool	>82,000	>82,000	>82,000	193	11,051	>82,000	>82,000	765
8.3	Pool	>82,000	>82,000	>82,000	228	8,126	>82,000	>82,000	425
8.2	Pool	10,564	22,038	38,851	115	6,002	>82,000	>82,000	<47
8.1	Riffle	781	1,639	2,916	79	3,853	6,230	20,468	<47

2.17.4 *Site S9*

Site S9 was located 3.5 km downstream of Chichester River junction (Reach 3). The site contained alternating pool and riffle sections. Bed material had low mobility in the riffles and the pools (Table 10). In the past this site was recognised for its instability and was recently modified with engineering works. The previous period of instability probably resulted in loss of the finer fraction of the bed material. The relatively coarse particle size, combined with the presence of grade control structures, was conducive to relative stability of the channel. Macrophytes were predicted to be regularly disturbed, but grasses and shrubs were not. Silt was predicted to be readily flushed from riffles and pools.

Table 10.
Discharges (ML/d) corresponding to exceedance of geomorphic process thresholds for Site S9. Maximum modelled discharge was 102,000 ML/d and minimum modelled discharge was 115 ML/d.

Section	Pool/ riffle	Bed material mobilisation			Macrophyte disruption		Grass/shrub disturbance		Surface silt flushing
		D_{16}	D_{50}	D_{84}	Q_m^{95}	$Q_m^{99.9}$	80 N/m^2	150 N/m^2	0.5 m/s
9.5	Riffle	5,220	>102,000	>102,000	359	9,585	>102,000	>102,000	<115
9.4	Pool	>102,000	>102,000	>102,000	286	7,907	>102,000	>102,000	680
9.3	Riffle	1,804	>102,000	>102,000	342	6,007	>102,000	>102,000	<115
9.2	Pool	>102,000	>102,000	>102,000	230	3,159	>102,000	>102,000	749
9.1	Riffle	3,216	5,898	8,842	150	4,038	5,997	21,569	<115

2.17.5 *Site S10*

Site S10 was located 1 kilometre downstream of Fosterton Bridge, or approximately 8 km downstream of Chichester River junction (Reach 3). The site contained alternating pool and riffle sections. Bed material had reasonably high mobility (mobile in at least the 1 in 5 year flood) in the lower half of the site (Table 11). Macrophytes were predicted to be regularly disturbed, but grasses and shrubs were not. Silt was predicted to be readily flushed from riffles, but less so from pools.

Table 11.
Discharges (ML/d) corresponding to exceedance of geomorphic process thresholds for Site S10. Maximum modelled discharge was 102,000 ML/d and minimum modelled discharge was 115 ML/d.

Section	Pool/ riffle	Bed material mobilisation			Macrophyte disruption		Grass/shrub disturbance		Surface silt flushing
		D_{16}	D_{50}	D_{84}	Q_m^{95}	$Q_m^{99.9}$	80 N/m^2	150 N/m^2	0.5 m/s
10.5	Riffle	11,328	17,866	27,535	266	3,766	>102,000	>102,000	742
10.4	Pool	>102,000	>102,000	>102,000	393	6,411	>102,000	>102,000	3,091
10.3	Riffle	21,924	30,815	45,420	398	4,946	>102,000	>102,000	1,762
10.2	Pool	17,186	23,286	28,912	333	3,574	47,181	>102,000	2,640
10.1	Riffle	531	1,019	1,890	100	3,231	5,487	21,046	<115

2.17.6 *Site S11*

Site S11 was located just downstream of Thalaba Bridge, upstream of Dungog (Reach 3). Bed material had reasonably high mobility across the entire particle size range for floods of 1 in 2 year and 1 in 5 year average recurrence interval (Table 12). Macrophytes were predicted to be regularly disturbed, but grasses and shrubs were not. Silt was predicted to be readily flushed from riffles and pools.

Table 12.
Discharges (ML/d) corresponding to exceedance of geomorphic process thresholds for Site S11. Maximum modelled discharge was 102,000 ML/d and minimum modelled discharge was 115 ML/d.

Section	Pool/ riffle	Bed material mobilisation			Macrophyte disruption		Grass/shrub disturbance		Surface silt flushing
		D_{16}	D_{50}	D_{84}	Q_m^{95}	$Q_m^{99.9}$	80 N/m^2	150 N/m^2	0.5 m/s
5.47	Riffle	115	168	296	<115	1,274	989	>102,000	<115
4.48	Pool	39,119	53,227	71,119	249	3,481	>102,000	>102,000	1,799
3.49	Pool	12,963	22,027	47,671	221	3,155	>102,000	>102,000	281
2.50	Riffle	7,553	15,181	25,062	222	3,239	97,174	>102,000	230
1.51	Riffle	26,761	46,069	72,869	364	4,390	>102,000	>102,000	549

2.17.7 *Site S12*

Site S12 was located at Glen Martin. Most of the site was in the backwater of the rock bar at Glen Martin, where bed material has low mobility, and the probability of grass and shrub disturbance was generally low (Table 13). Bed material was predicted to be mobile at low discharges in the shallow sections. Silt was predicted to be regularly flushed from riffles but less so from the pools, especially towards the downstream end of the site, which is probably a zone of fine sediment deposition. Macrophytes were predicted to be often disturbed over the majority of the site, but less so towards the downstream end.

Table 13.
Discharges (ML/d) corresponding to exceedance of geomorphic process thresholds for Site S12. Maximum modelled discharge was 102,000 ML/d and minimum modelled discharge was 115 ML/d.

Section	Pool/ riffle	Bed material mobilisation			Macrophyte disruption		Grass/shrub disturbance		Surface silt flushing
		D_{16}	D_{50}	D_{84}	Q_m^{95}	$Q_m^{99.9}$	80 N/m^2	150 N/m^2	0.5 m/s
15.63	Riffle	115	239	473	<115	2,043	1,545	>102,000	<115
14.64	Pool	>102,000	>102,000	>102,000	426	4,821	>102,000	>102,000	4,554
13.65	Riffle	33,330	48,004	67,646	320	3,609	>102,000	>102,000	1,814
12.66	Riffle	168	369	702	141	2,342	1,903	16,406	<115
11.67	Pool	80,549	>102,000	>102,000	413	4,493	>102,000	>102,000	3,614
10.68	Riffle	28,904	55,584	93,791	308	4,180	>102,000	>102,000	1,115
9.69	Pool	>102,000	>102,000	>102,000	483	5,172	>102,000	>102,000	4,770
8.692	Riffle	115	295	1,141	<115	4,480	3,118	>102,000	<115
7.70	Pool	>102,000	>102,000	>102,000	583	6,320	>102,000	>102,000	9,758
6.71	Pool	99,726	>102,000	>102,000	437	5,955	>102,000	>102,000	2,842
5.72	Pool	>102,000	>102,000	>102,000	705	7,509	>102,000	>102,000	8,551
4.73	Pool	84,788	>102,000	>102,000	612	7,019	>102,000	>102,000	6,747
3.74	Pool	>102,000	>102,000	>102,000	1,163	14,038	>102,000	>102,000	27,327
2.75	Pool	>102,000	>102,000	>102,000	1,093	11,760	>102,000	>102,000	23,844
1.00	Pool	>102,000	>102,000	>102,000	528	43,055	>102,000	>102,000	753

2.17.8 *Discussion*

Modelling of geomorphic process thresholds revealed a consistent pattern in the river. The bed material was at least partly mobile at most riffle sites under conditions of small freshes that occurred multiple times per year (Table 14). In general, the data indicated bed material was stable in pools even under high flow conditions. In practice, the bed material is likely to be mobile in the pool environments under high flow conditions. This is explained by the ‘velocity or bed shear stress reversal effect’ whereby under the conditions of high, channel-forming discharges, the pool experiences hydraulic conditions favouring scour and erosion (Keller 1971, Richards 1982, p. 186). The hydraulic data from the Williams River were indicative of a river with active bed material transport, which confirmed the earlier assumptions made independently by Wayne Erskine and Andrew Brooks.

Site S9 appeared to be relatively stable under current conditions. This was largely because of the noticeably coarser bed material found at this site. The coarse material may have been an artefact of previous unstable conditions, prior to the recent installation of grade control structures.

The bed of the river was observed to have few macrophytes present (Figure 40). The analysis indicated that hydraulic conditions were usually sufficient to exceed the thresholds associated with rupturing macrophyte stems, so it was not surprising that this plant form was uncommon (Table 15).

The banks of the Williams River appeared to be relatively stable, a characteristic that seemed to be imparted by the reasonably complete vegetative cover. However, the river was observed actively migrating in places; this was evidenced by bare banks cut into the alluvium, and fallen trees (Figure 41). It is natural for a lowland river to erode and migrate within certain bounds. Despite the reasonably good vegetative cover on most of the banks, the current rate of bank instability of the Williams River is likely to be higher than natural. This is due to the incised nature of the channel, which creates higher than natural shear stresses within the channel during high flow conditions. The tendency towards instability would have been moderated to some degree by the river works that have been undertaken over the years (revetments, bed control structures etc.

The modelling suggested that matted grasses and shrubs were reasonably resistant to hydraulic disturbance under most conditions (Table 16), and this was evidenced by bent but not uprooted shrubs in the channel (Figure 42) and intact grass mats on the surfaces of benches (Figure 43), despite large events in June 2007 and April 2008. Although grass mats and shrubs seem to be resistant to hydraulic disturbance, they could be adversely affected by other factors, such as protracted inundation, stock or human intervention.

Table 14.
Range of discharges (ML/d) corresponding to exceedance of bed material mobilisation thresholds at riffle cross-sections, showing associated range of average recurrence intervals (ARIs) calculated for mean daily discharge.

Site	Threshold discharge range (ML/d)			Average recurrence interval range (years)		
	Bed (fine)	Bed (medium)	Bed (coarse)	Bed (fine)	Bed (medium)	Bed (coarse)
S7	372 – 14,100	1,100 – 24,400	2,400 – 40,100	<0.2 – 2.3	<0.2 – 5.9	0.3 – 21
S8	290 – 780	540 – 1,640	880 – 2,920	<0.2	<0.2 – 0.2	<0.2 – 0.4
S9	1,800 – 5,200	5,900 – >102,000	8,800 – >102,000	<0.2 – 0.4	0.5 – >100	0.7 – >100
S10	530 – 21,900	1,000 – 30,800	1,900 – 45,400	<0.2 – 2.2	<0.2 – 3.7	<0.2 – 7.5
S11	7,600 – 26,800	15,200 – 46,100	25,100 – 72,900	0.3 – 1.2	0.6 – 2.4	1.1 – 6.0
S12	<115 – 33,300	239 – 55,600	473 – 93,800	<0.2 – 1.5	<0.2 – 3.3	<0.2 – 13

Table 15.
Range of discharges (ML/d) corresponding to exceedance of macrophyte disturbance thresholds at riffle cross-sections, showing associated range of average recurrence intervals (ARIs) calculated for mean daily discharge.

Site	Threshold discharge range (ML/d)		Average recurrence interval range (years)	
	Macrophytes (low impact)	Macrophytes (high impact)	Macrophytes (low impact)	Macrophytes (high impact)
S7	101 – 267	3,100 – 3,800	<0.2	0.4 – 0.5
S8	80 – 300	3,900 – 5,000	<0.2	0.5 – 0.7
S9	150 – 360	4,000 – 9,600	<0.2	0.3 – 0.8
S10	100 – 400	3,200 – 4,900	<0.2	0.3 – 0.4
S11	220 – 360	3,200 – 4,400	<0.2	<0.2
S12	<115 – 320	2,000 – 4,500	<0.2	<0.2

Table 16.
Range of discharges (ML/d) corresponding to exceedance of grass matt and shrub disturbance thresholds at riffle cross-sections, showing associated range of average recurrence intervals (ARIs) calculated for mean daily discharge.

Site	Threshold discharge range (ML/d)		Average recurrence interval range (years)	
	Grass/shrub (low impact)	Grass/shrub (high impact)	Grass/shrub (low impact)	Grass/shrub (high impact)
S7	5,100 – >82,000	15,600 – >82,00	0.8 – >100	2.7 – >100
S8	5,700 – >82,000	20,500 – 82,000	0.8 – >100	4.1 – >100
S9	6,000 – >102,000	21,600 – >102,000	0.5 – >100	2.1 – >100
S10	5,500 – >102,000	21,000 – >102,000	0.4 – >100	2.1 – >100
S11	97,200 – >102,000	>102,000	15 – >18	>18
S12	1,500 – >102,000	>102,000	<0.2 – >18	0.7 – >18

The modelling suggested that fine surface sediment was frequently flushed from the surface of the bed of pools and riffles (Table 17). This was confirmed in the field, with virtually no fine sediment being evident on the wetted surface of the bed (Figure 44). Some sand deposits were found in some sheltered locations in the channel, mostly downstream of Dungog (on point bars, in the lee of shrubs, or within densely vegetated banks) (Figure 45).

Table 17.
Range of discharges (ML/d) corresponding to exceedance of silt and sand flushing thresholds at riffle and pool cross-sections, showing associated range of average recurrence intervals (ARIs) calculated for mean daily discharge.

Site	Threshold discharge range (ML/d)	Average recurrence interval range (years)
	Silt and sand	Silt and sand
S7	<47 – 1,200	<0.2
S8	<47 – 1,000	<0.2
S9	<115 – 750	<0.2
S10	<115 – 3,090	<0.2 – 0.3
S11	<115 – 1,800	<0.2
S12	<115 – 27,300	<0.2 – 1.2

Figure 40. Typical view of Williams River channel (at S8), being relatively free of macrophytes. E 56 378688; N 6424613; View is downstream; 14:12 hr; 3/12/2007. Flow approx. 340 ML/d.

Figure 41. Actively eroding banks on the Williams River. Left is S8 (3/12/2007) and right is site 6.7 km upstream of S12 (10/05/2008).

Figure 42. Casuarina saplings adjacent to a riffle in the bed of the Williams River 2 weeks after experiencing total immersion in a flood (peak 69,000 ML/d). Site located 6.7 km upstream of S12, on the property at the end of Glen William Church Road. E 56 388357; N 6401099; View is right to left bank; 13:32 hr; 10/05/2008. Flow approx. 500 ML/d.

Figure 43. Well grassed, stable, low gravel bench on the Williams River at S7, just downstream of the proposed Tillegra Dam site. E 56 376717; N 6423438; View is downstream; 12:21 hr; 3/12/2007. Flow approx. 350 ML/d.

Figure 44. Typical clean bed material, free of fine surface sediment and heavy biofilm build-up. Taken at Bandon Grove, just downstream of Chichester River junction, between S8 and S9, on 10/08/2007. Flow approx. 70 ML/d.

Figure 45. Sand and fine gravel deposits on the lower bank of the inside bend (left), and within dense riparian vegetation (right) at S11. E 56 383601; N 6406784; 15:55 hr; 10/05/2008. Flow approx. 460 ML/d.

2.18 Geomorphic form discharge thresholds

2.18.1 *Event duration criterion*

In order for the geomorphic forms considered here to be inundated the flow only has to be momentarily above the threshold level. Thus the appropriate flood frequency relationships (Table 7, Figure 35) are those based on instantaneous peak daily data.

2.18.2 *Site S7*

Site S7 was located 200 m downstream of Tillegra Dam site (Reach 3). The site was deeply incised with steep banks, and no prominent inset benches were present. Low grassed gravel benches (stabilised gravel bars) were present. At a flow of 350 ML/d the central riffle was almost fully inundated (24 m width) to an average depth of 0.19 m (range 0.10 – 0.22 m) (Figure 46). The low grassed bench would require flows of 400 – 1,400 ML/d for inundation. The upper valley surface appeared to be a terrace, with the hydraulic model predicting no inundation for events up to the highest modelled discharge (>100 year ARI event) (Figure 47).

Figure 46. Fully covered riffle in middle of Site S7, just downstream of the proposed Tillegra Dam site. Low grassed gravel bench in foreground. E 56 376717; N 6423438; View is right bank to left bank; 12:21 hr; 3/12/2007. Flow approx. 350 ML/d.

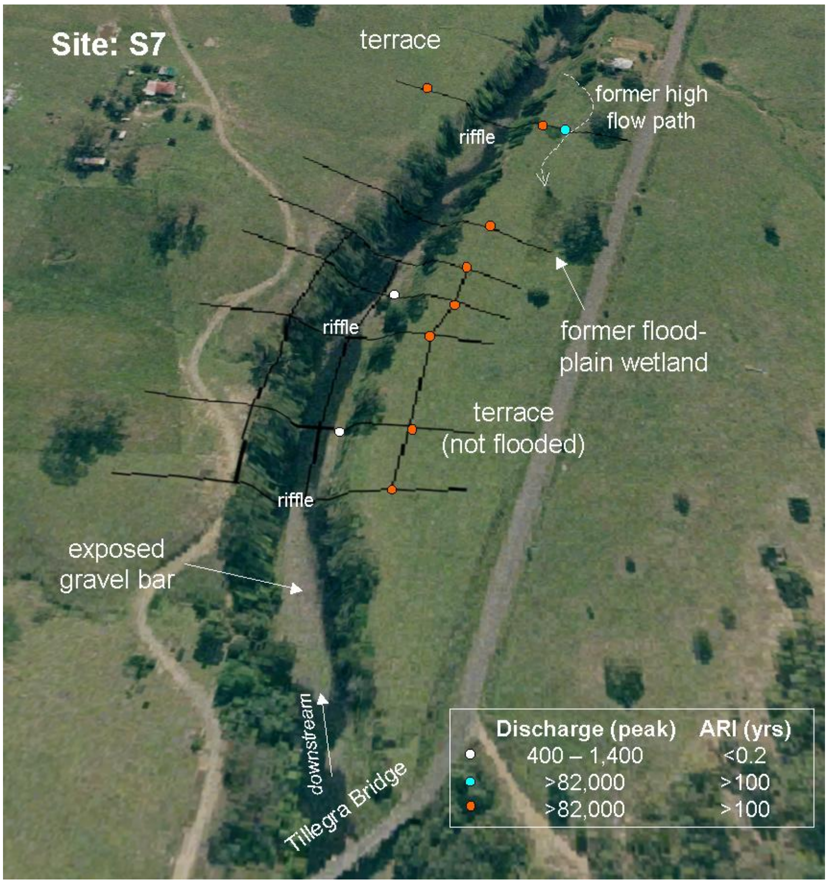

Figure 47. Site S7, showing major morphological surfaces and the discharge required to inundate them (instantaneous flow). Image is LiDAR generated DEM with draped aerial photograph.

2.18.3 *Site S8*

Site S8 was located just upstream of Chichester River junction (Reach 3). At a flow of 340 ML/d the central riffle was almost fully inundated (16.6 m width) to an average depth of 0.20 m (range 0.11 – 0.42 m) (Figure 48). This was a relatively narrow riffle, and it was observed to be narrower at 78 ML/d (Figure 49). The low gravel bars were inundated at flows in the range 1,000 – 3,000 ML/d while the higher gravel bar on the upstream left side of the site required 10,000 – 11,000 ML/d for inundation (Figure 50). There were other horizontal surfaces present at this site, inundated across a range of discharges. The upper valley surface was infrequently inundated. The ARI was around 30 to 60 years, meaning that this surface was effectively a terrace.

Figure 48. Fully covered riffle at Site S8. E 56 378701; N 6424587; View is right bank to left bank; 14:08 hr; 3/12/2007. Flow approx. 340 ML/d. Low gravel bar in foreground, and high gravel bar in background.

Figure 49. Site S8 from the air, with riffle depicted in Figure 48 circled. View is downstream. Taken at 08:42 hr; 14/11/2007. Flow approx. 78 ML/d.

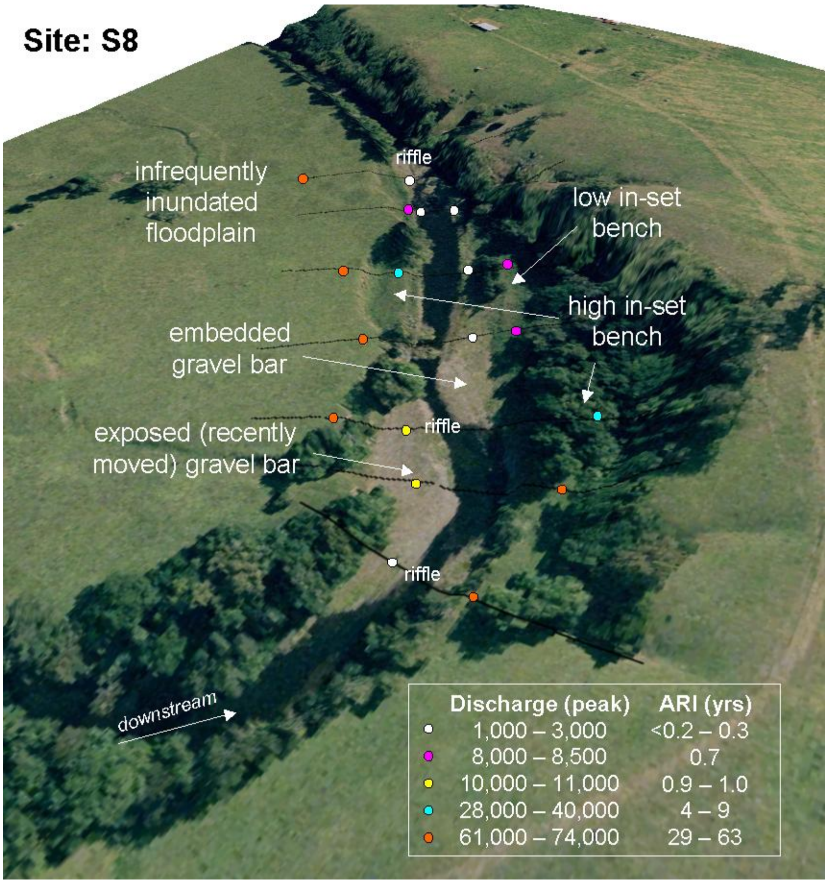

Figure 50. Site S8, showing major morphological surfaces and the discharge required to inundate them (instantaneous flow). Image is LiDAR generated DEM with draped aerial photograph.

2.18.4 Site S9

Site S9 was located 3.5 km downstream of Chichester River junction (Reach 3). At a flow of 750 ML/d the riffle was fully inundated (28.5 m width) by fast flowing water to a mean depth of 0.34 m (range 0.05 – 0.46 m) (Figure 51). At 190 ML/d about two-thirds of the riffle width was inundated (Figure 52). The exposed gravel bars required flows of 6,000 – 8,000 ML/d for inundation (Figure 53). The low floodplain surface was inundated on average one every 2.5 years, while the high floodplain (right bank) was inundated at floods with an average recurrence interval greater than 6 years. At discharges of this magnitude an unknown proportion of the flow bypasses the site via high flow channels (Figure 38). These flows were not included in the hydraulic model, so it was not possible to estimate flood frequency for high flow events.

Figure 51. Fully covered riffle at Site S9. E 56 380820; N 6423586; View is downstream; 16:40 hr; 3/12/2007. Flow approx. 750 ML/d.

Figure 52. Partly covered riffle (left of picture) at Site S9. Taken from helicopter at 08:38 hr; 14/11/2007. Flow approx. 190 ML/d. Flow is left to right.

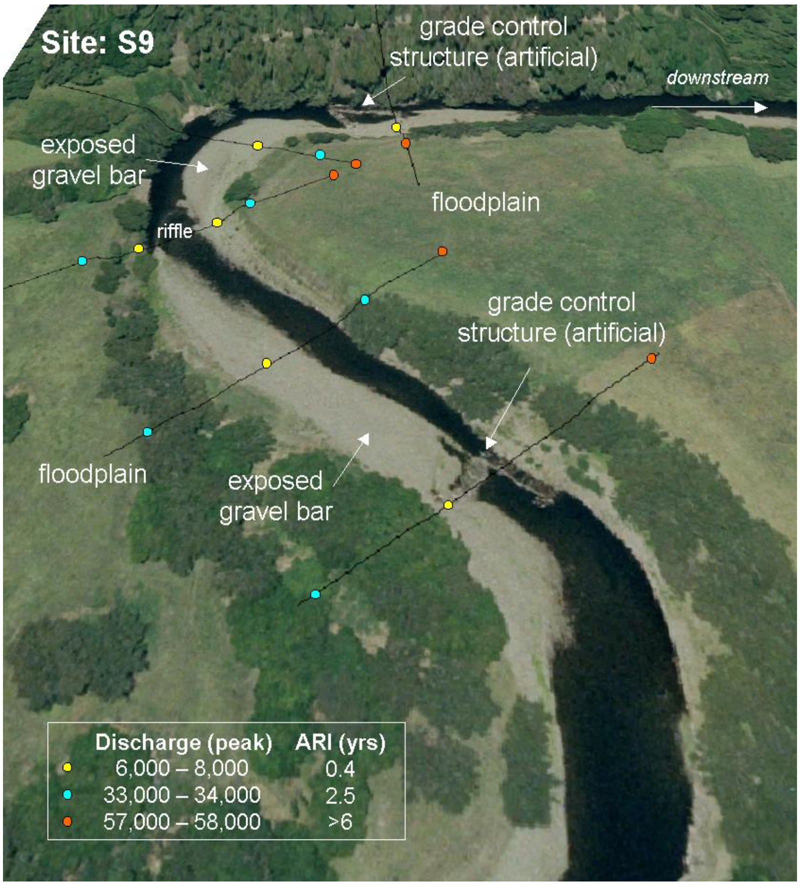

Figure 53. Site S9, showing major morphological surfaces and the discharge required to inundate them (instantaneous flow). Image is LiDAR generated DEM with draped aerial photograph.

2.18.5 Site S10

Site S10 was located 1 kilometre downstream of Fosterton Bridge, or approximately 8 km downstream of Chichester River junction (Reach 3) (Figure 54). Low flows of 70 ML/d exposed a large area of gravels, while flows of 2,000 ML/d fully inundated the bed of the channel with fast flowing water (Figure 55). There was a low bench present that required 27,000 ML/d for inundation (Figure 54). The left floodplain was lower than the right floodplain. The left floodplain was inundated by the 3 to 7 year ARI event. The right floodplain had two levels, one inundated by the 11 year ARI event and one by the 14 year ARI event (Figure 54).

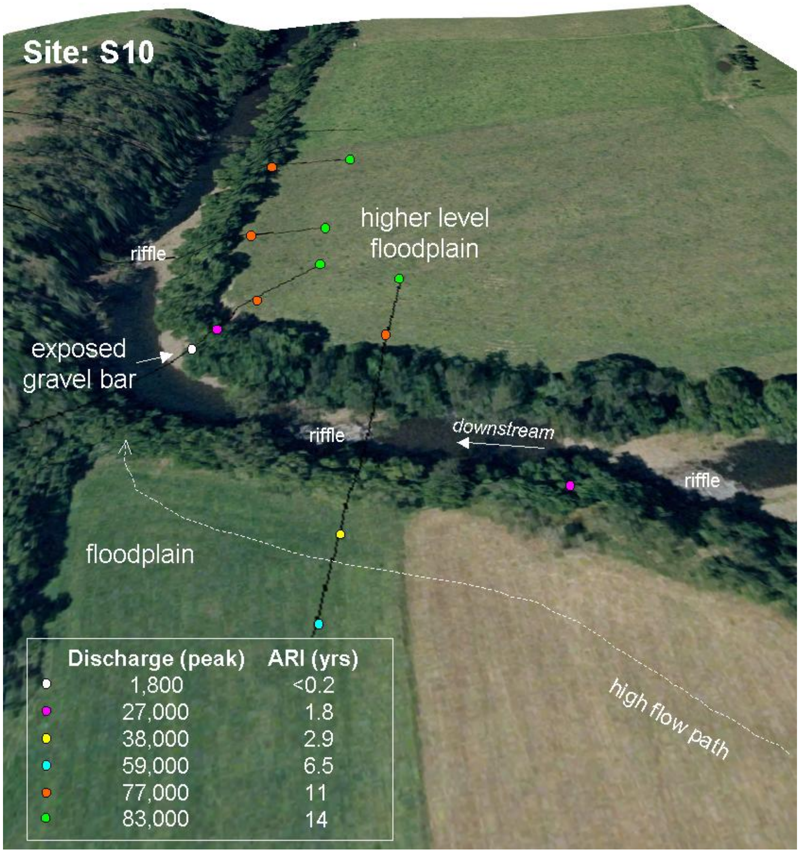

Figure 54. Site S10, showing major morphological surfaces and the discharge required to inundate them (instantaneous flow). Image is LiDAR generated DEM with draped aerial photograph.

Figure 55. View downstream from Fosterton Bridge, just upstream of S10, E 56 382194; N 6422876. Left view taken at 11:34 hr; 10/08/2007; flow approx. 70 ML/d. Right view taken at 07:29 on 4/12/2007; flow approx. 2,000 ML/d.

2.18.6 *Site S11*

Site S11 was located just downstream of Thalaba Bridge, upstream of Dungog (Reach 3). The cross-sections available for Site S11 were not surveyed in sufficient detail to depict any inset benches or stable bars that may have been present. Field inspection revealed that low elevated gravel bar surfaces were present at this site, but they were narrow. The riffle downstream of the bridge was partly inundated at 300 ML/d, and although more of it was inundated at 460 ML/d there was still an area of exposed gravels present (Figure 56).

This site experienced a moderate flood event two weeks before the site inspection on 10 May 2008. This event broke the left bank upstream of the site, with low-lying land on the left floodplain inundated to a depth of 1 – 2 m in places. The water reached the very top of the left bank (natural levee). On the right bank, the river broke out just downstream of the bridge, and flowed across part of the inside of the meander loop. The hydraulic model predicted that the top of bank flow ranged from 43,800 to 77,300 ML/d for the five cross-sections (corresponding to ARI 1.4 – 3.2 years), with the lower two cross-sections below the bridge being 64,000 ML/d and 69,000 ML/d. The peak of the April 2008 event was gauged to be 69,000 ML/d at Glen Martin, suggesting that the hydraulic model was a good representation of the river at the top of bank level.

Figure 56. View from Thalaba Bridge, S11, E 56 383602; N 6406840. Left view taken on 20/08/2007; flow approx. 300 ML/d. Right view taken on 10/08/2008; flow approx. 460 ML/d. Upstream view is to pool, downstream view is to riffle.

2.18.7 *Site S12*

Site S12 was located at the very end of Reach 3, from Mill Dam Falls to the riffle upstream of the Falls. The cross-sections available for Site S11 were not surveyed in sufficient detail to depict any inset benches or stable bars that may have been present. Field inspection revealed that low elevated gravel bar surfaces were present at this site, but they were narrow. Although 15 cross-sections were available for this site, 6 of them were incomplete. For the other 9 cross-sections, the top of bank corresponded to 30,300 to 60,100 ML/d (corresponding to ARI 0.9 – 2.1 years).

A site located 6.7 km upstream of S12, with similar morphology to the upper part of S12, was inspected on 10 May 2008. At the time, the riffle was fully inundated by a flow of approximately 500 ML/d (Figure 57). The Mill Dam Falls were inspected on 20 August 2007 when flow was approximately 3,200 ML/d. At that discharge the rock bar at the Falls was completely inundated and formed a swiftly flowing rapid (Figure 22).

Figure 57. Riffle site 6.7 km upstream of S12, E 56 383602; N 6406840. View is upstream. Taken at 13:58 on 10/05/2008; flow approx. 500 ML/d.

2.18.8 *Discussion*

The Williams River was observed to be incised upstream of Glen William. Incision of the river was previously noted by Erskine (1998), Erskine (2001), Brooks et al (2004) and Brooks et al (2006). The hydraulic/geomorphic modelling undertaken here suggested that the degree of incision was spatially variable. Between Tillegra and the Chichester River junction the river appeared to be deeply incised, such that the channel contained the 1 in 100 year event. At Sites S9 and S10 the river was evidently incised, but the floodplain was predicted to flood on average every 6 to 14 years. At S11 and S12 the river was apparently not incised, such that the floodplain was inundated on average every 1 to 3 years. The river had a series of low inset benches and stable gravel bars present at various levels in the cross-sections. These surfaces required events of 1,000 – 8,000 ML/d for inundation. Such events were frequent in the Williams River in the current discharge series, occurring on average more than once per year. Some sites had other higher benches present that were less frequently inundated. The riffles were mostly inundated by flows of around 350 – 500 ML/d. Such flows were very frequent in the current series, occurring multiple times per year on average.

3 Potential environmental impacts

3.1 Base case ‘with dam’ scenario

3.1.1 *Discharge event frequency*

Flood frequency analysis was undertaken to estimate the discharge associated with a range of average recurrence intervals (ARIs) under the base case ‘with dam’ scenario (Table 18). For each ARI the discharge was reported as both the mean daily and the peak instantaneous daily value. These relationships enabled calculation of the average recurrence interval corresponding to the defined geomorphic process and form thresholds. Comparison of the flood frequency curves with the curves for the current scenario indicated that the dam would have a significant impact on flood frequency (Figure 58).

3.1.2 *Impact on frequency of geomorphic processes*

The base case ‘with dam’ scenario would have the effect of reducing, for each ARI, the event magnitude (Figure 58). Thus, for a given discharge threshold, the ARI was predicted to decrease. The data indicated that bed material mobility would still be achieved under the base case ‘with dam’ scenario, but the frequency of occurrence would generally decrease at each site (Table 19). Macrophyte disturbance under the base case ‘with dam’ scenario continued to be a common occurrence (Table 20). However, there would possibly be more opportunities for macrophyte colonisation at Tillegra. Grass and shrubs were rarely disrupted under the current flow regime. Under the base case ‘with dam’ scenario this would continue to be the case, although such events would be even rarer (Table 21). Flushing of silt and sand from the bed surface would continue to be a common event under the base case ‘with dam’ scenario (Table 22).

The implication of the combined effects of reduced bed material mobilisation, increased chance of macrophyte colonisation, and reduced disruption to in-stream vegetation is that under the base case ‘with dam’ scenario, over time the channel would become more stable, with more in-stream vegetation. The flows would still maintain the basic geomorphic processes, but the useable (by biota) channel area may contract somewhat. This effect was predicted to lessen with distance from the proposed dam.

While the opportunity for bed material mobilisation would be reduced under the base case ‘with dam’ scenario, bed material transport would still occur, and under the situation of bed material being trapped by the dam, the bed would tend to scour, with more scour predicted closer to the dam. The implications of this are discussed in more detail in a later section of this report (Sections 3.1.4.1 and 3.1.5).

Table 18.
Discharges (ML/d) corresponding to various flow duration indices. Discharge for base case 'with dam' scenario. Values represent 2nd order polynomial fitted to annual exceedance series for ARI <10 years, and for ARI ≥10 years, Generalised Pareto (Tillegra and Glen Martin) or Johnson's SB (DS Chichester junction) distributions fitted to annual maximum series. Peak daily values are factored mean daily values.

	Tillegra (S7 and S8)		Downstream of Chichester River (S9 and S10)		Glen Martin (S11 and S12)	
ARI (years)	Mean daily	Peak daily	Mean daily	Peak daily	Mean daily	Peak daily
0.2	300	400	1,250	1,750	6,350	9,050
0.3	500	650	1,950	2,750	8,700	12,350
0.4	700	900	2,600	3,650	10,750	15,250
0.5	900	1,100	3,250	4,550	12,550	17,850
0.6	1,050	1,350	3,850	5,400	14,200	20,200
0.7	1,250	1,550	4,450	6,250	15,750	22,400
0.8	1,450	1,800	5,000	7,000	17,150	24,400
0.9	1,600	2,000	5,500	7,800	18,450	26,300
1	1,800	2,200	6,050	8,500	19,700	28,050
2	3,350	4,050	10,300	14,500	29,350	41,750
3	4,650	5,600	13,500	19,050	36,050	51,350
4	5,850	6,950	16,100	22,650	41,300	58,800
5	6,900	8,150	18,250	25,700	45,600	64,900
10	10,050	11,800	25,200	35,500	60,300	85,800
15	12,950	15,050	29,300	41,300	68,300	97,200
20	15,200	17,550	32,000	45,100	73,500	104,600
25	17,000	19,550	33,900	47,800	77,300	110,000
30	18,700	21,450	35,400	49,900	80,300	114,250
35	20,200	23,150	36,500	51,450	82,800	117,850
40	21,550	24,650	37,500	52,850	84,900	120,800
45	22,800	26,000	38,300	54,000	86,600	123,250
50	23,950	27,300	39,000	55,000	88,200	125,500
75	28,650	32,450	41,400	58,350	93,800	133,500
100	32,400	36,500	42,900	60,500	97,500	138,750

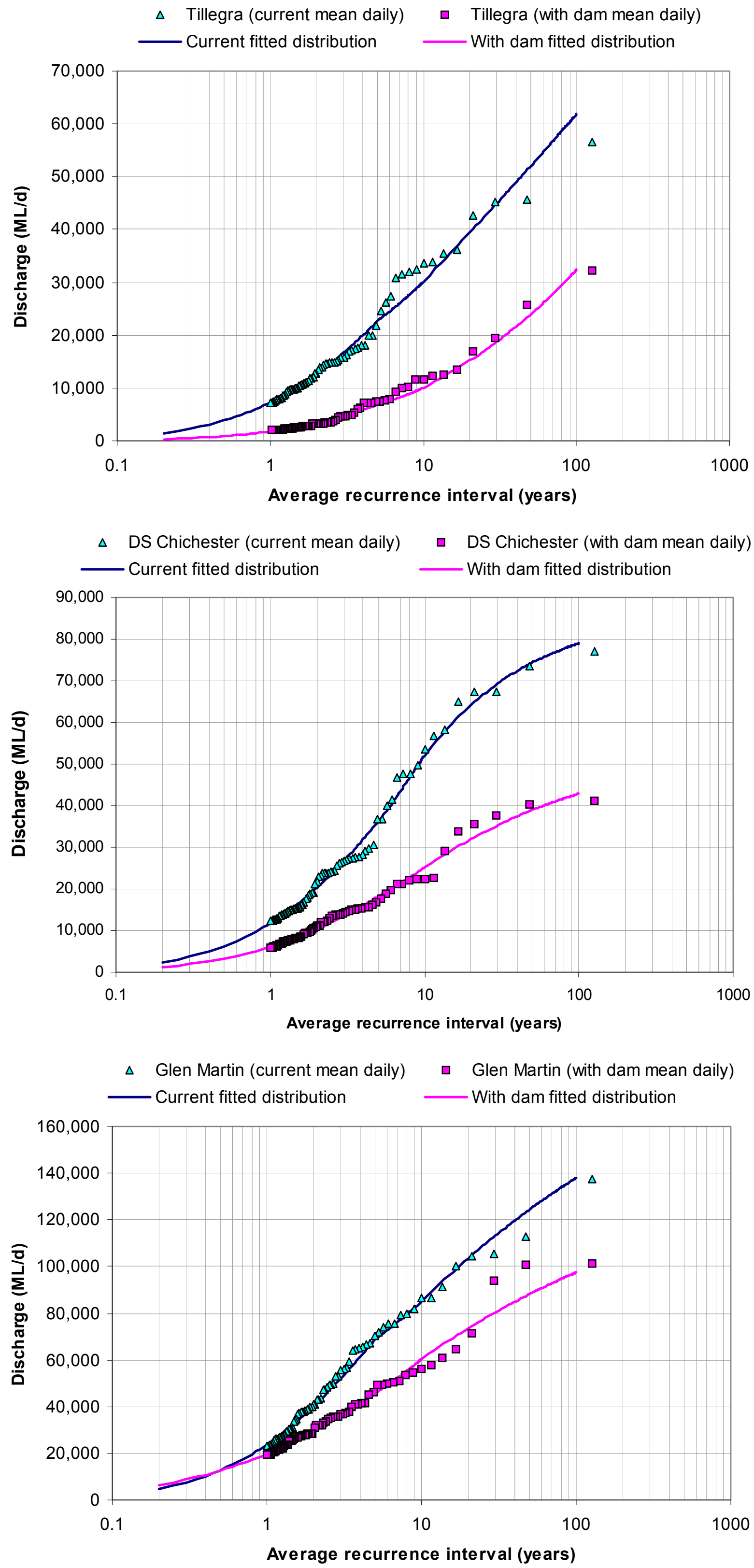

Figure 58. Flood frequency distributions for the Williams River, current compared to 'with dam' scenario, mean daily data. Flood data plotted using the Cunnane plotting position formula (α = 0.4). Curves are polynomial fitted to annual exceedance series for ARI <10 years, and for ARI ≥10 years, Generalised Pareto (Tillegra and Glen Martin) or Johnson's SB (DS Chichester junction) distributions fitted to annual maximum series.

Table 19.
Range of ARIs (for mean daily discharge) associated with exceedance of bed material mobilisation thresholds at riffle cross-sections for current and for 'with dam' scenarios.

Site	Current scenario Average recurrence interval range (years)			Base case 'with dam' scenario Average recurrence interval range (years)		
	Bed (fine)	Bed (medium)	Bed (coarse)	Bed (fine)	Bed (medium)	Bed (coarse)
S7	<0.2 – 2.3	<0.2 – 5.9	0.3 – 21	<0.2 – 17	<0.6 – 52	1.4 – >100
S8	<0.2	<0.2 – 0.2	<0.2 – 0.4	<0.2 – 0.4	<0.3 – 0.9	<0.5 – 1.7
S9	<0.2 – 0.4	0.5 – >100	0.7 – >100	<0.3 – 0.8	1.0 – >100	1.6 – >100
S10	<0.2 – 2.2	<0.2 – 3.7	<0.2 – 7.5	<0.2 – 7.6	<0.2 – 17	<0.3 – 100
S11	0.3 – 1.2	0.6 – 2.4	1.1 – 6.0	0.2 – 1.7	0.7 – 5.1	1.5 – 19
S12	<0.2 – 1.5	<0.2 – 3.3	<0.2 – 13	<0.2 – 2.5	<0.2 – 8.3	<0.2 – 74

Table 20.
Range of ARIs (for mean daily discharge) associated with exceedance of macrophyte disturbance thresholds at riffle cross-sections for current and for 'with dam' scenarios.

Site	Current scenario Average recurrence interval range (years)		Base case 'with dam' scenario Average recurrence interval range (years)	
	Macrophytes (low impact)	Macrophytes (high impact)	Macrophytes (low impact)	Macrophytes (high impact)
S7	<0.2	0.4 – 0.5	<0.2	1.4 – 2.3
S8	<0.2	0.5 – 0.7	<0.2	2.3 – 3.3
S9	<0.2	0.3 – 0.8	<0.2	0.6 – 1.8
S10	<0.2	0.3 – 0.4	<0.2	0.5 – 0.8
S11	<0.2	<0.2	<0.2	<0.2
S12	<0.2	<0.2	<0.2	<0.2

3.1.3 *Impact on frequency of inundation of geomorphic forms*

The morphological forms identified at each site were associated with a level and a discharge. This was expressed as an ARI (based on peak flow series) for the current scenario and for the base case 'with dam' scenario. The difference between these recurrence intervals was the predicted impact of the base case 'with dam' scenario on inundation of these surfaces. The upper morphological surface is referred to here as 'bankfull' – this applies to a morphologically defined surface, not a process defined surface, so no implications are intended regarding the frequency of inundation.

Table 21.
Range of ARIs (for mean daily discharge) associated with exceedance of grass matt and shrub disturbance thresholds at riffle cross-sections for current and for 'with dam' scenarios.

Site	Current scenario Average recurrence interval range (years)		Bas case 'with dam' scenario Average recurrence interval range (years)	
	Grass/shrub (low impact)	Grass/shrub (high impact)	Grass/shrub (low impact)	Grass/shrub (high impact)
S7	0.8 – >100	2.7 – >100	3.3 – >100	21 – >100
S8	0.8 – >100	4.1 – >100	3.8 – >100	35 – >100
S9	0.5 – >100	2.1 – >100	1.0 – >100	7.3 – >100
S10	0.4 – >100	2.1 – >100	0.9 – >100	7.0 – >100
S11	15 – >18	>18	>18	>18
S12	<0.2 – >18	0.7 – >18	<0.2 – >18	0.7 – >18

Table 22.
Range of ARIs (for mean daily discharge) associated with exceedance of silt and sand flushing thresholds at riffle and pool cross-sections for current and for 'with dam' scenarios.

Site	Current scenario Average recurrence interval range (years)	'With dam' scenario Average recurrence interval range (years)
	Silt and sand	Silt and sand
S7	<0.2	<0.2 – 0.7
S8	<0.2	<0.2 – 0.6
S9	<0.2	<0.2
S10	<0.2 – 0.3	<0.2 – 0.5
S11	<0.2	<0.2
S12	<0.2 – 1.2	<0.2 – 1.8

At S7, the 'with dam' scenario was predicted to have little impact on the frequency of inundation of geomorphic forms, largely because there were few forms identified (Table 23). The low unvegetated and vegetated bars would experience reduced frequency of inundation, but would still be inundated frequently. The morphological bankfull level would be unchanged as it is terrestrial under the current flow regime.

Table 23.
Site S7: ARIs (for peak flows) associated with exceedance of geomorphic form thresholds at riffle and pool cross-sections for current and for base case 'with dam' scenarios. Cross-sections numbered lowest is downstream.

Surface	Cross-section	Elevation (m AHD)	Discharge (ML/d)	ARI (years)	
				Current	With dam
Low unvegetated bar	XS4	85.3	425	<0.2	<0.2
Low unvegetated bar	XS6	86.2	1,440	<0.2	0.5
Sill to wetland	XS1	90.8	>82,000	>100	>100
Bankfull	XS1	92.2	>82,000	>100	>100
Bankfull	XS2	92.2	>82,000	>100	>100
Bankfull	XS3	92.3	>82,000	>100	>100
Bankfull	XS4	92.7	>82,000	>100	>100
Bankfull	XS5	93.0	>82,000	>100	>100
Bankfull	XS6	93.1	>82,000	>100	>100
Bankfull	XS7	93.1	>82,000	>100	>100

At S8, there was a high variety of surfaces present (Table 24). The low unvegetated bars were currently inundated multiple times per year. Under the 'with dam' scenario this frequency would reduce, but it would still be at least once per year for most of the surfaces. Under the 'with dam' scenario, the higher unvegetated bar and the low vegetated bench would likely change their character, as they would be inundated much less frequently, shifting from being flooded at least once per year on average to once every 3 to 5 years on average. The other benches were infrequently inundated under the current regime, and the frequency would reduce under the 'with dam' scenario. The bankfull level at this site could be described as a terrace, as it was infrequently inundated. Under the 'with dam' scenario, the 100 year ARI event would not reach this level, so the terrace would become fully terrestrialised.

At S9, three main surfaces were identified (Table 25). Under the 'with dam' scenario, the low unvegetated gravel bar would continue to be inundated more than once per year. The mid-level bench would shift from being inundated once every 2.5 years to once every 9 years. At this site the bankfull level was an active floodplain under the current scenario, although it was flooded only once every 6 years. Under the 'with dam' scenario the floodplain would be inundated on average once every 60 to 70 years, effectively becoming an inactive terrace.

At S10, the low unvegetated bench would continue to be inundated multiple times per year under the 'with dam' scenario (Table 26). The intermediate level surfaces would be flooded every 5 to 11 years rather than every 2 to 3 years. The left floodplain surface was lower than that on the right bank. The left floodplain would flood once every 81 years, which represents a large change from the current once every 6.5 year frequency. The higher right bank floodplain surfaces would undergo terrestrialisation, shifting from being flooded every 11 to 14 years to not being inundated by the 100 year ARI event.

Table 24.
Site S8: ARIs (for peak flows) associated with exceedance of geomorphic form thresholds at riffle and pool cross-sections for current and for base case ‘with dam’ scenarios. Cross-sections numbered lowest is downstream.

Surface	Cross-section	Elevation (m AHD)	Discharge (ML/d)	ARI (years)	
				Current	With dam
Low unvegetated bar	XS1	77.1	2,235	<0.2	0.8
Low unvegetated bar	XS2	77.2	1,062	<0.2	0.4
Low unvegetated bar	XS3	77.5	1,629	<0.2	0.5
Low unvegetated bar	XS4	77.9	3,141	0.3	1.1
Low unvegetated bar	XS7	78.3	2,409	<0.2	0.8
Recently mobilised coarse bar (left)	XS5	78.7	8,487	0.7	3.7
Recently mobilised coarse bar (left)	XS6	78.9	8,002	0.7	3.4
Low vegetated bench	XS2	78.4	9,990	0.9	4.7
Low vegetated bench	XS3	78.7	10,683	1.0	5.3
Low vegetated bench	XS4	78.9	10,778	1.0	5.4
Left grassed bench	XS3	80.1	28,275	3.9	33
Right treed bench	XS5	80.9	39,962	9	>100
Bankfull	XS1	81.1	82,131	>100	>100
Bankfull	XS2	81.2	53,398	19	>100
Bankfull	XS3	82.1	66,530	40	>100
Bankfull	XS4	81.9	60,784	29	>100
Bankfull	XS5	82.5	74,349	63	>100
Bankfull	XS6	82.2	64,754	37	>100
Bankfull	XS7	82.0	67,136	42	>100

At S11, only a bankfull surface could be identified from the cross-sections, although in the field some narrower lower benches were visible. Under the current scenario the floodplain was inundated reasonably frequently, at around once every 2 to 3 years (Table 27). This is within the range of expected bankfull flood frequency for un-incised rivers. Under the ‘with dam’ scenario, the flood frequency would halve, so that the floodplain would inundate on average once every 2 to 8 years.

At S12, only a bankfull surface could be identified from the cross-sections, although in the field some narrower lower benches were visible. Under the current scenario the floodplain was inundated reasonably frequently, at around once every 1 to 2 years (Table 28). This is within the range of expected bankfull flood frequency for un-incised rivers. Under the ‘with dam’ scenario, the flood frequency would halve, so that the floodplain would inundate on average once every 1 to 4 years.

Table 25.
Site S9: ARIs (for peak flows) associated with exceedance of geomorphic form thresholds at riffle and pool cross-sections for current and for base case 'with dam' scenarios. Cross-sections numbered lowest is downstream.

Surface	Cross-section	Elevation (m AHD)	Discharge (ML/d)	ARI (years)	
				Current	With dam
Unvegetated gravel bar	XS1	68.8	7,515	0.4	0.9
Unvegetated gravel bar	XS2	69.5	7,817	0.4	0.9
Unvegetated gravel bar	XS3	69.7	7,817	0.4	0.9
Unvegetated gravel bar	XS4	70.1	7,586	0.4	0.9
Unvegetated gravel bar	XS5	70.4	6,223	0.4	0.7
Mid-level bench (left and right)	XS3	71.6	33,613	2.5	8.8
Mid-level bench (left and right)	XS4	71.9	34,000	2.5	9.0
Mid-level bench (left and right)	XS5	72.0	33,571	2.5	8.8
Bankfull	XS2	72.7	58,110	6.3	72
Bankfull	XS3	72.8	56,839	6.0	62
Bankfull	XS4	73.0	56,759	6.0	61
Bankfull	XS5	73.1	57,143	6.1	64

Table 26.
Site S10: ARIs (for peak flows) associated with exceedance of geomorphic form thresholds at riffle and pool cross-sections for current and for base case 'with dam' scenarios. Cross-sections numbered lowest is downstream.

Surface	Cross-section	Elevation (m AHD)	Discharge (ML/d)	ARI (years)	
				Current	With dam
Unvegetated gravel bar	XS4	60.4	1,806	<0.2	<0.2
Small right bench	XS4	63.2	26,669	1.8	5.4
Low point left upper floodplain	XS5	63.8	37,513	2.9	11
Left upper floodplain	XS5	65.0	59,051	6.5	81
Bankfull	XS2	65.0	76,509	11	>100
Bankfull	XS3	65.6	76,857	11	>100
Bankfull	XS4	66.0	76,975	11	>100
Bankfull	XS5	66.0	77,704	11	>100
Upper floodplain	XS2	65.3	83,288	14	>100
Upper floodplain	XS3	65.9	83,091	14	>100
Upper floodplain	XS4	66.3	83,128	14	>100
Upper floodplain	XS5	66.3	82,642	14	>100

Table 27.
Site S11: ARIs (for peak flows) associated with exceedance of geomorphic form thresholds at riffle and pool cross-sections for current and for base case 'with dam' scenarios. Cross-sections numbered lowest is downstream.

Surface	Cross-section	Elevation (m AHD)	Discharge (ML/d)	ARI (years)	
				Current	With dam
Bankfull	XS5.47	33.14	43,750	1.4	2.1
Bankfull	XS4.48	33.15	77,322	3.2	7.6
Bankfull	XS3.49	30.93	55,898	1.9	3.5
Bankfull	XS2.50	29.96	63,854	2.3	4.8
Bankfull	XS1.51	29.39	68,990	2.6	5.8

Table 28.
Site S12: ARIs (for peak flows) associated with exceedance of geomorphic form thresholds at riffle and pool cross-sections for current and for base case 'with dam' scenarios. Cross-sections numbered lowest is downstream.

Surface	Cross-section	Elevation (m AHD)	Discharge (ML/d)	ARI (years)	
				Current	With dam
Bankfull	XS63	13.35	57,977	2.0	3.8
Bankfull	XS64	12.62	51,553	1.7	3.0
Bankfull	XS65	11.52	45,467	1.5	2.3
Bankfull	XS66	9.80	60,063	2.1	4.1
Bankfull	XS67	8.90	58,772	2.0	3.9
Bankfull	XS68	6.20	30,317	0.9	1.1
Bankfull	XS69	6.46	41,370	1.3	2.0
Bankfull	XS71	6.10	47,570	1.5	2.5
Bankfull	XS74	6.00	55,888	1.9	3.5

3.1.4 Impact on sediment transport

3.1.4.1 Bed material sediment transport

There is little doubt that the Tillegra Dam would lead to a degree of bed scour in the Williams River downstream of the proposed dam site. The reason for this is the trapping of the upstream sediment supply, but maintenance of flows that have the capacity to mobilise the bed material. This process also occurred on the Chichester River downstream of Chichester Dam when the dam was closed. The result was scour of the finer fraction of the bed material, leaving a mostly boulder sized bed in the area downstream of the dam. The same process would occur on the Williams River, with the bed scouring to bedrock and leaving the immobile boulders in place. This would change the physical (hydraulic) character of the bed, which would have implications for the biota.

Where the river is currently controlled by bedrock bars there will be no change to the bed level or the bed character. Bedrock bars are common downstream of Tillegra, so most of the scour is expected to be localised, rather than a uniform bed lowering. This river has a long history of instability and bed lowering due to ill-conceived works programs, so it has already incised down to resistant points of bed control.

The downstream extent of potential scour cannot be accurately predicted. However, it is likely that it would extend for some distance downstream of the Chichester River confluence, as this river is starved of sediment (from Chichester Dam). Prior to the construction of Chichester Dam, this river would have been the major supplier of coarse bed load to the Williams River. Certainly, the Williams River has the capacity to transport the current bed material at the surveyed sites all the way down to Glen Martin. The impact of scour would be offset in the downstream direction to some extent, as unregulated tributaries would inject some coarse sediment to the river. The potential of these tributaries to provide coarse bed material to the Williams River was not investigated as part of this project.

Sediment scour due to sediment starvation would be partly mediated by reduced frequency of flows with the capacity to transport coarse sediment. The base case 'with dam' scenario was predicted to significantly reduce discharge peaks, and this would reduce the potential bedload transport rate at Tillegra by a factor of three. The modelling suggested that with the dam in place, in the reach down to the Chichester River junction, the river had the capacity to transport an average annual load of 1,000 – 2,000 tonnes, although this varied from virtually nothing up to 18,000 tonnes per year depending on hydrological conditions.

A comparable case on the Trinity River, Texas, resulted in significant channel scour for 60 km downstream of the dam, limited only by the river reaching its delta zone, where stream powers were very low (Phillips et al 2005). Although the overall response was bed scour, there was no consistent channel response, as various qualitatively different combinations of increases, decreases or no change in width, depth, slope and roughness occurred (Phillips et al 2005). A similar degree of spatial variation in scour severity would be expected in the Williams River.

If the bed of the Williams River scours in the vicinity of the junction of the confluence of the Chichester River, this will lower the base level of the Chichester River. This would be expected to lead to scour of the bed of the Chichester River, through a process of upstream nickpoint migration. However, this is expected to be limited in scale and extent, because the Chichester River has long been subjected to sediment starvation due to the presence of Chichester Dam. The Chichester River would have already passed through a post-dam phase of scour, followed by adjustment to a new stable state. Thus, scour of the bed of the Chichester River could occur, migrating upstream through time, but only to the point where a bedrock bar is encountered, or where very coarse immobile bed material is encountered.

On the basis of the previous independent work of Erskine and Brooks, plus the observations made in this study, significant lowering of the bed of the Williams River is not regarded as a high risk, for the following reasons:

- The more recent phase of channel improvement works would have stabilised the most unstable sections of river.

- According to Brooks et al (2006), the river is sediment supply limited, so its trajectory would have been towards bed lowering for some time. Erskine (2001) documented evidence for a past history of bed scour associated with early channel improvement works (eventually, the rate of bed lowering has to slow or stop, as the river becomes controlled by resistant underlying bedrock),

- Field observations made during this study, and by Erskine (2001), suggested that in many reaches the river bed level is currently controlled by stable bedrock bars.

- The predicted tendency towards more vegetation in the channel under the 'with dam' scenario would tend to offset any tendency for banks to become more unstable through bed lowering

Any bed scour associated with operation of the proposed dam would likely involve localised scour in areas of deep deposits of bed material (i.e. gravel bars could degrade), general removal of the finer component of the bed material (leading to bed armouring – a coarse upper layer) and greater exposure of bedrock outcrops.

3.1.4.2 Suspended sediment transport

The Tillegra Dam would have a dramatic impact on suspended sediment load due to its high trap efficiency. Immediately downstream of the Dam, the load would reduce from an average 10,000 tonnes per year to only 140 tonnes per year. Although the majority of the Williams River channel is constructed from coarse-grained material, the upper banks and some in-channel benches were observed to be constructed from fine alluvial material (silt and fine sand). With the proposed dam in place, these components will suffer a reduced rate of construction. This could have ecological consequences, as the fine-grained channel forms are likely to favour different vegetation communities compared to the relatively sterile gravel and cobble bars and benches.

With the dam in place, the upper sections of Reach 3 in particular would tend to have clearer water than currently during high flows, which would mean lower nutrient concentrations, and greater light penetration. This could have implications for the ecology. Lower overall suspended sediment loads to the Seaham Weir pool would mean lower risk of algal blooms.

3.1.5 Impact on bank stability

The banks of the Williams River were observed to be relatively stable, but instances of bare eroded banks were not difficult to find (e.g. Figure 41). Significant lowering of the bed could potentially lead to an initially increased rate of bank instability, as channel cross-section adjustment to the new bed level could involve bank profile adjustment. However, as explained above (Section 3.1.4.1), while bed scour would be expected to occur downstream of the proposed dam, significant general bed lowering would not be expected.

Apart from bed-lowering, the only other dam-related cause of increased bank erosion would be the flow regime itself. Most instances of significant bank collapse in the Williams River are probably associated with large flood events. The frequency of such events would decline under a 'with dam' scenario (Figure 58). Bank scour can also occur under conditions of long duration flows that are above the level of the bank toe. At times of low flow, the Williams River flows over a gravel, cobble and bedrock bed. Under the base case 'with dam' scenario bulk water transfers would occur over the range 250 – 500 ML/d which is within the range of flows that are generally confined to the coarse bed. Thus, these flows would be unlikely to result in accelerated bank erosion.

3.2 Pulsed bulk water transfers scenarios and maximum outlet works capacity release

3.2.1 *Flow pulse scenarios*

In the base case 'with dam' scenario, bulk water transfers were made over the range 250 – 500 ML/d. As an alternative, these transfers could be made as a series of pulsed freshes (simulated minor flow events) with a peak up to 1,500 – 1,700 ML/d, receding over a period of 10 to 15 days (total volume of each event being around 4,300 ML). On average there would be around 7 of these events per year.

Some smaller events of 270 ML/d peak and receding over 2 days could also be released as part of an environmental flow regime. These events are smaller in magnitude than the 500 ML/d maximum water transfer rate evaluated as part of the base case 'with dam' scenario, so for the purpose of evaluating impacts, in this report a 500 ML/d peak flow was analysed, on the assumption that a 270 ML/d peak flow would have a significantly lower impact.

The maximum capacity of the outlet works of the proposed dam would be 4,000 ML/d. It is not currently proposed to release water from the dam at this rate, but to cover the possibility of such an event ever being released for some reason, the geomorphic impacts of such a release were considered.

The above two flow pulse scenarios (i.e. 500 ML/d, 1,500 ML/d), and the 4,000 ML/d release, were evaluated here from the perspective of bed material transport. If flow pulses were incorporated into a flow regime, it is not expected that the impacts on the other aspects of the geomorphology of the river would be significantly different from those evaluated for the base case 'with dam' scenario in detail elsewhere in this report. The 4,000 ML/d release, if it was ever made, would be a rare occasion. However, it is noted that this is not a rare event naturally, occurring on average more than three times per year as a peak event, and twice per year as a mean daily event (Table 7). Thus, release of a 4,000 ML/d flow would be extraordinary only if it was released for a long duration. Under the current regime, most events exceeding 4,000 ML/d were of 1 or 2 days duration (median 1.5 days), but 8 events in the period 1931 to 2007 were of 5 or more days duration (Figure 59).

3.2.2 *Methodology for bedload transport modelling*

Bedload transport rates were computed using the same sediment transport analysis used to examine the current and post-dam hydrologic series. Synthetic event hydrographs were defined based in part on hydrological analysis undertaken by Connell Wagner. The falling limb of each hydrograph was designed based on recession curves established by Connell Wagner:

1. The 500 ML/day recession being based on the recession curve established by Connell Wagner for a 270 ML/day flows:

   $Qrecession = 500\ [e^{-t/0.3} + e^{-t/1.2}\ ] / 2$

2. The 1,500 ML/d and 4,000 ML/day recessions were based on the recession curve established by Connell Wagner for events with peaks between 1,000 ML/d and 1,800 ML/day:

   $Qrecession = 1{,}500\ [e^{-t} + e^{-t/4}\ ] / 2$

   $Qrecession = 4{,}000\ [e^{-t} + e^{-t/4}\ ] / 2$

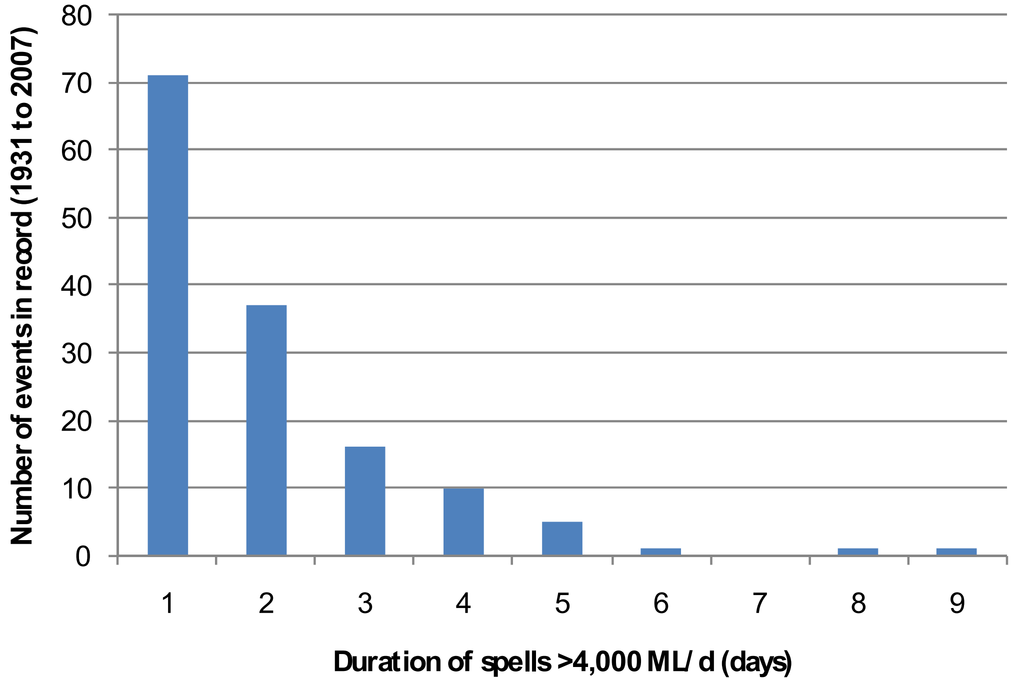

Figure 59. Frequency histogram of durations of spells exceeding 4,000 ML/d at Tillegra for the current flow series.

The rising limb of each hydrograph was assumed to be linear with the same rate of rise assumed for each hydrograph (Figure 60). A common baseflow discharge of 50 ML/day was assumed. Note that the total flow volume for the 1,500 ML/d event (4,022 ML/d minus the baseflow) was slightly less than the nominal 4,300 ML/d total volume for this pulse, but this difference is explained by the baseflow. It is possible that the pulses would be delivered at peak rates of 1,600 or 1,700 ML/d rather than the 1,500 ML/d peak analysed, but this difference is not great enough to significantly affect the conclusions drawn from the analysis.

Three flow series were constructed, each 210 days in length. Each flow series contained one or more events of a given magnitude. The number of events was specified so that:

- the total event volume was approximately the same for each flow series; and
- the gross volume of water delivered (i.e. including baseflow) was the same for each flow series.

The three flow series comprised (i) 1 x 4,000 ML/day event, (ii) 4 x 1,500 ML/day events, and (ii) 52 x 500 ML/day events (Figure 61). The chosen length of these series not that important, as the main objective of this analysis was to compare the sediment transport potentials of the three different sized pulses. Essentially, this analysis is a comparison of high frequency/low magnitude versus low frequency/high magnitude.

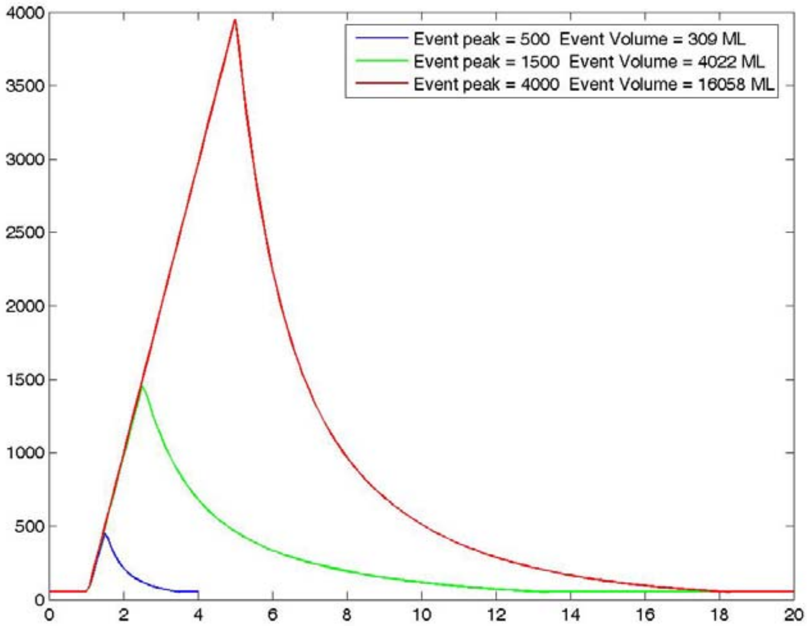

Figure 60. Event hydrographs used for evaluation of sediment transport. Event volumes are area under the hydrograph minus the baseflow of 50 ML/d.

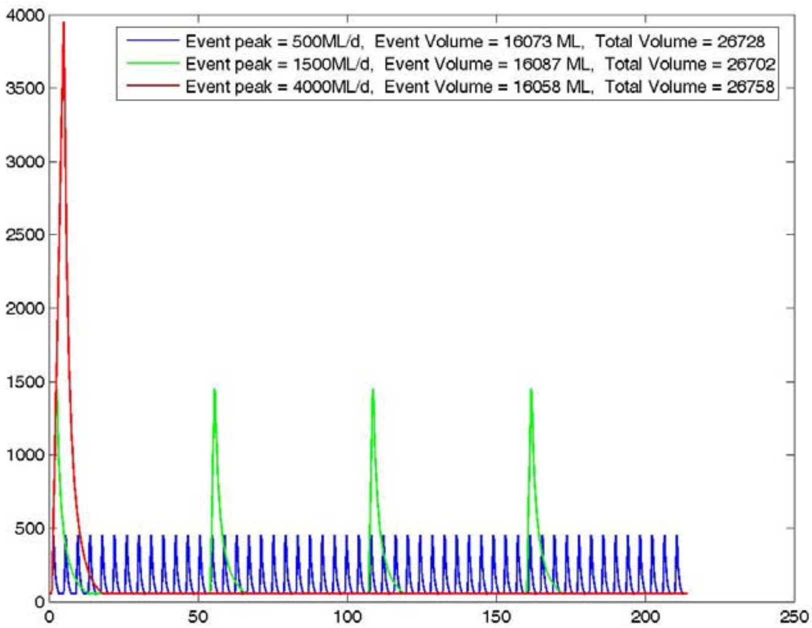

Figure 61. The 210 day long hydrographs used to analyse sediment transport for three different flow pulse magnitudes.

Sediment transport rates were computed for each flow series at a number of cross-sections at sites W7, W8, W9 and W10. Only riffle cross-sections were used in the analysis.

No analyses were conducted at sites W11 and W12, which are located a long distance from the proposed dam. In reality, the hydrographs would change substantially by the time they reach these downstream sites (channel routing effects and in-channel storage would attenuate the hydrograph peaks substantially). Furthermore, the analysis completed at the upstream sites indicated that sediment transport rates declined relatively rapidly downstream.

Bedload transport at site W10 was zero for all events (due to the hydraulic peculiarities of this site). Consequently the results presented herein are for sites W7, W8 and W9 only.

3.2.3 Results of bedload transport modelling

The total bedload volumes transported through each flow series were computed (Figure 62). The total load for each site was the mean of the results at each cross-section at the site. For reference, the mean annual bedload transport rates computed under current flow conditions were: 3,090 tonne for Site 7, 4,140 tonne for Site 8 and 3,590 tonne for Site 8.

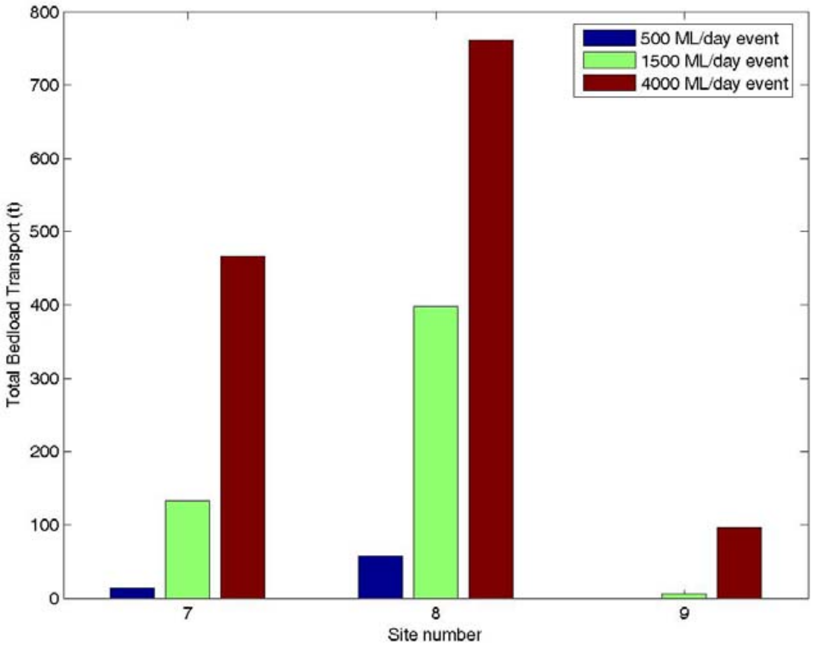

Figure 62. Total bedload transport in tonnes at each site for the two flow pulse scenarios (210 day duration) considered and the 4,000 ML/d release.

Bedload transport rates increase substantially as event size increases. The single 4,000 ML/day event transported more bedload than either of the other flow series. Bedload transport at sites W7 and W8 was predicted to be significantly higher than at site W9.

At Site W7 the 4 x 1,500 ML/day events transported the equivalent of about 5 percent of the current mean annual bedload transport. Given that on average 7 of these events would be implemented, these events would deliver the equivalent of around 8 percent of the current mean annual load. The 4,000 ML/day event transported 12.5 percent of the current mean annual total. Although a 4,000 ML/d event occurred twice per year in the current regime, on average these events were shorter than the modelled pulse. The 500 ML/d event transported only a small volume of bedload at Site W7.

At Site W8 the 4 x 1,500 ML/day events transported the equivalent of around 10 percent of the current mean annual bedload transport. If 7 events were implemented in a year, these would account for the equivalent of 17 percent of the current mean annual load. The 4,000 ML/day event transported 20% of the current mean annual total. The 500 ML/d event transported only a small volume of bedload at Site W8.

Detailed sedimentographs illustrate the sensitivity of total bedload transport to event size (Figure 63). Note in Figure 63 the sedimentograph near the bottom axis that shows the four very small increases in the bedload transport rate during the 500 ML/day events.

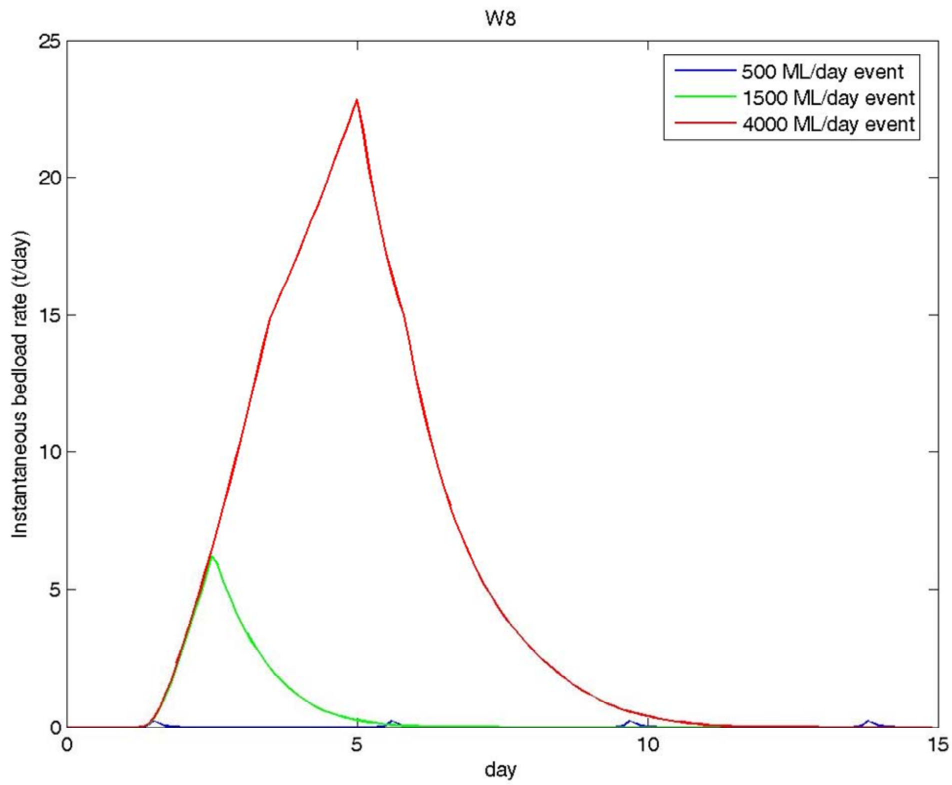

Figure 63. Sedimentographs for different event sizes at Site W8, cross-section 5.

Bedload transport rates were similar during the 500 ML/day event at Sites W7 and W8, although quite low in magnitude, peaking at around 0.2 tonnes/day (Figure 64). An event of 500 ML/day was not sufficient to entrain bedload at any cross-section at Site W9. This was due to the increase in size of the channel downstream of the confluence with the Chichester River.

Bedload transport magnitudes were considerably higher for a 1,500 ML/day event than a 500 ML/day event, achieving 6 tonnes/day at Site W8 (Figure 65). There was a clear distinction between the bedload transport rates at Sites W7 and W8 for the 1500 ML/day event (Figure 65). This was due to the larger substrate sizes at Site W7. Transport rates at site W9 were small by comparison with Sites W7 and W8.

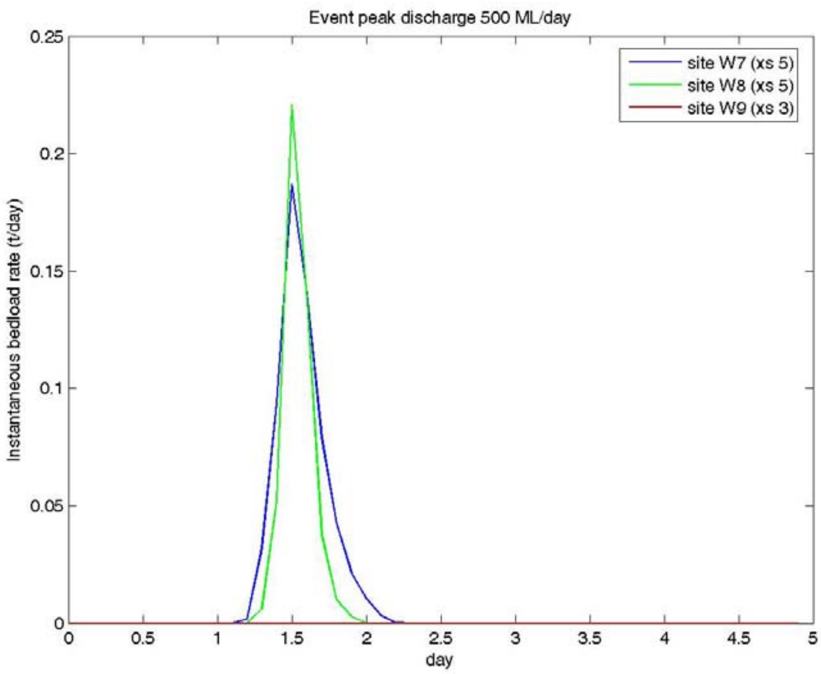

Figure 64. Sedimentographs for 500 ML/d peak event at Sites W7, W8, and W9.

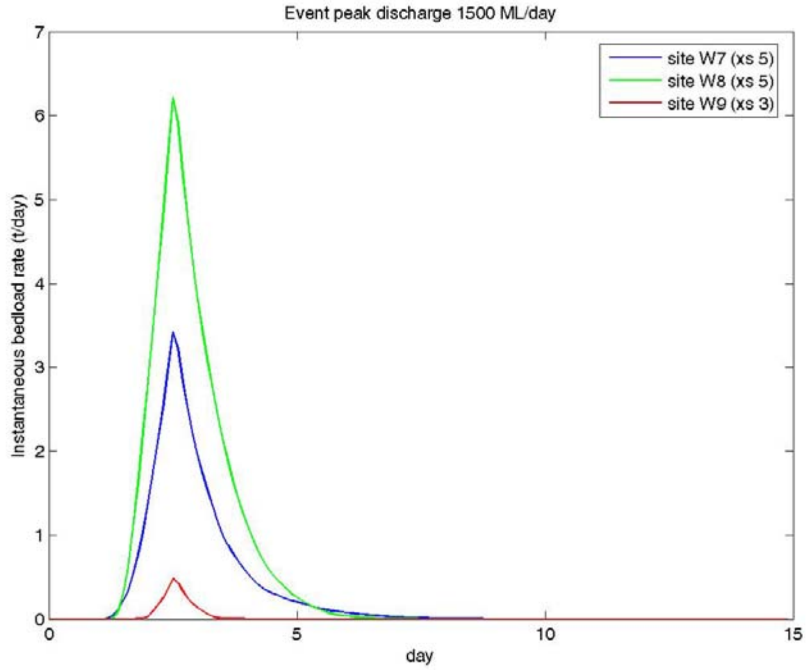

Figure 65. Sedimentographs for 1,500 ML/d peak event at Sites W7, W8, and W9.

Bedload transport rates for a 4,000 ML/day event exceeded 15 tonnes/day at Sites W7 and W8 (Figure 66). There was less difference between the transport rates at Sites W7 and W8 for the 4,000 ML/day event. This suggests that the higher discharge values are sufficient to entrain a greater range of the substrate sizes at site W7. Bedload transport at Site W9 was again considerably lower than at the upstream sites.

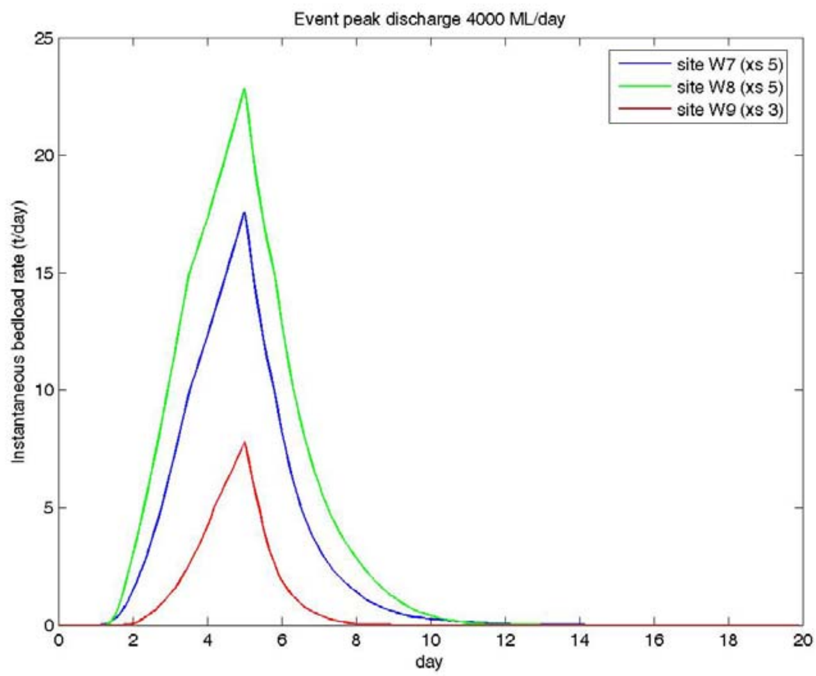

Figure 66. Sedimentographs for 4,000 ML/d peak event at Sites W7, W8, and W9.

The modelling suggested that bedload transport rates were highly sensitive to event magnitude. Bedload transport would be reduced by reducing the peak discharge magnitude. Limiting the duration of time that the flow is close to the peak would also reduce the total bedload transport. The reach downstream of the proposed Tillegra Dam site and upstream of the confluence with the Chichester River (represented by Sites W7 and W8) would be the most vulnerable to bedload transport and scour under the modelled flow regime. But even here, the 1,500 ML/d pulse events would not result in wholesale bedload transport, but selective transport of a proportion of the finer fraction. The bed would, over time, likely become armoured (i.e. developing a coarse, protective surface layer), which would slow the bed material transport rate.

3.2.4 *Impacts of flow pulses on bank erosion*

At Tillegra, under the current flow regime, events of 1,500 ML/d peak magnitude are very common, occurring on average around 7 to 8 times per year. Geomorphologically, these events mobilise the finer fraction of the bed material and flush fine silt and organic material from the bed surface. The low gravel bars are inundated at these flow levels, and the toes of the banks would be wet at most sections. However, as the duration of the events would be short, such events would not be expected to cause bank erosion to any greater extent than would similar events that regularly occur in the current flow regime.

Although the overall risk of increased rate of bank erosion in response to operation of the proposed Tillegra Dam is not regarded as high, the process of bank erosion and channel migration will undoubtedly continue into the future. These processes are typical of lowland rivers, whether regulated or not. In the case of the Williams River, it will not be possible to attribute future bank erosion to operation of a dam at Tillegra, as this process occurs under the current flow regime, and will continue into the future, with or without a dam at Tillegra. This erosion may or may not be associated with local bed level adjustments – erosion could be related to natural channel migration, local scour from flow diversions (from large trees for example), high shear stresses during large floods, or low bank strength due to degraded riparian vegetation (due to loss of natural vegetation covering or damage by stock). None of these factors are inherently related to the operation of a dam. Bank erosion is currently, and will into the future, be the result of multiple factors. This means that monitoring bank morphology through a program of repeated survey, while it may indicate stability or otherwise of the banks at the selected monitoring sites, it will not reveal the cause of any change in bank morphology. Also, it will not be possible to monitor any change to the rate of channel stability post-dam due to the lack of baseline data. Many years (in the order of 20^{+} years) of careful data collection are required before the rate of these processes can be established (as the rate of erosion is a long-term mean about a pattern of high short-term variability or flood response).

3.3 Risk to in-stream structures

The river contains a number of structures managed by the Hunter-Central Rivers Catchment Management Authority. These mainly comprise bank revetments and grade control structures, and also a fishway at Bandon Grove. These structures would have been designed to withstand flows of a certain magnitude. The proposed dam at Tillegra would not increase the risk of failure due to high flows, as the dam would reduce the frequency of high flow events. However, these structures could be threatened by undermining if the bed progressively scoured deeper over time.

The overall risk of undermining due to bed changes is assessed to be relatively low, as these are structures designed to be more stable than the surrounding natural river materials. However, this risk depends to some degree on the characteristics and situation of each structure. The grade control structures are at relatively low risk, because they create hydraulic conditions that promote bed stability, such as at Site 9. Some structures may be at risk of failure from local bed scour, but this risk applies regardless of whether there is a dam present or not.

It is normal for in-stream structures to occasionally fail and require maintenance or reconstruction. Thus, should a dam be built at Tillegra, it would not necessarily be the case that all future maintenance that may be required on these in-stream structures would be attributable to a situation brought about by the dam. It would be necessary to independently determine the cause of each failure.

3.4 Stability of the channel banks in Seaham Weir pool

The Seaham Weir pool has been noted as a site of bank erosion (Healthy Rivers Commission of NSW 1996). Power boating has been implicated as the main cause in the upper weir pool and in the lower weir pool cattle access and wind waves are involved (Healthy Rivers Commission of NSW 1996). The flow rate of the river in a weir pool such as this does not play a major role in bank erosion per se. The main issue is the relatively constant water levels, regardless of flow rate. Power boating and wind are such problems here because of the relatively constant water level, which focuses the erosive energy of the waves on a very narrow band of the bank, causing undercutting. Large flow events may play a role in removing weakened bank material, but it is the constant water level that creates the conditions conducive to bank erosion.

The flow regime changes that may come about from operation of the proposed Tillegra Dam would be relatively inconsequential for the risk of erosion in the Seaham Weir pool, because, while the pattern of flow may change in some respects, these flow changes would have only a very small or no effect on the regime of water levels in the weir pool. Changes to the pattern of water levels in Seaham Weir are largely controlled by operation of the weir itself. The weir gates are opened during large events, and this practice would not change with Tillegra Dam operational. The frequency of large events would fall with the dam operational, so the rate of erosion due to large events removing the already weakened bank material might reduce.

3.5 Loss of dam capacity through river sediment inputs

Reservoirs and dams are efficient sinks of both suspended solids and bed load. When a dam fills with sediment to the extent that the reduced capacity compromises the functionality of the storage, it is either decommissioned or sediment has to be extracted. Either way, this situation presents a potential environmental hazard, as there is potential for release of undesirable quantities of sediment to the river below.

In general 100 percent of the bed load that enters a dam is trapped, while a small proportion of the suspended load is conveyed downstream. Tillegra Dam would trap approximately 95 percent of the suspended load (9,300 tonnes per year), plus all of the bed load (3,100 tonnes per year), totalling 12,400 tonnes per year, on average. Assuming an underwater bulk density of 1.5 for suspended material (partially organic) and 2.6 for bedload (assume hard rock), this rate of sediment trapping equates to 6,200 cubic metres per year of suspended material and 1,200 cubic metres per year of coarse material per year. This represents 0.002 percent of the proposed dam's capacity. In 100 years this would account for only 0.2 percent of the dam's capacity, so sedimentation of the dam would not be a significant problem over the normal life expectancy of the dam.

3.6 Potential shoreline erosion

The potential maximum long-term volume of shoreline erosion was calculated assuming that all of the soil between the 96 percent of FSL and FSL would be removed (plus an allowance for 0.4 m wave height).

The volume of eroded material per linear metre of shoreline varied with the slope and assumed soil depth (Figure 67). In general, the Western shoreline zone would have lower slopes than the other two defined zones, so the predicted volume lost per linear metre was higher there (also, it had none of the shallower Williams Range soils). The maximum volume of material calculated for assumed soil depths and slopes was 1,537 thousand cubic metres (equivalent to ML), which represented only 0.3 percent of the storage volume (Table 29). This is an estimate of the maximum potential volume (ie assumes all soil would be eroded around the entire rim). In reality, the lower and middle Northern and Eastern zone, and some southeast facing sections, would be more susceptible to erosion, as they would face the prevailing winds (Figure 68). Sheltered parts of the storage rim may not erode to the same extent. These erosion volumes assume no management action would be taken to prevent erosion. Regardless of the assumptions made, shoreline erosion would not threaten to significantly reduce the capacity of the dam.

The width of exposed shoreline was predicted to vary markedly with the slope but was less sensitive to soil depth (Figure 67). In general, the Western shoreline zone would have lower slopes than the other two defined zones, so the exposed shoreline width was predicted to be generally higher there. At 96 percent of FSL, the predicted exposed widths varied up to 90 metres (Figure 67). At FSL the predicted exposed widths were narrower (up to 60 m) (Figure 67). On average, at 96 percent of FSL, the average width of the exposed shoreline on the three zones ranged from 18 to 35 metres (Table 30), and at FSL, the average was predicted to be 12 to 25 metres. This represents an estimate of the maximum potential exposed width. In reality the sections facing the prevailing winds would be more susceptible to erosion (Figure 68). Sheltered parts of the storage rim may not erode to the same extent. These predicted widths assume no management action would be taken to prevent erosion.

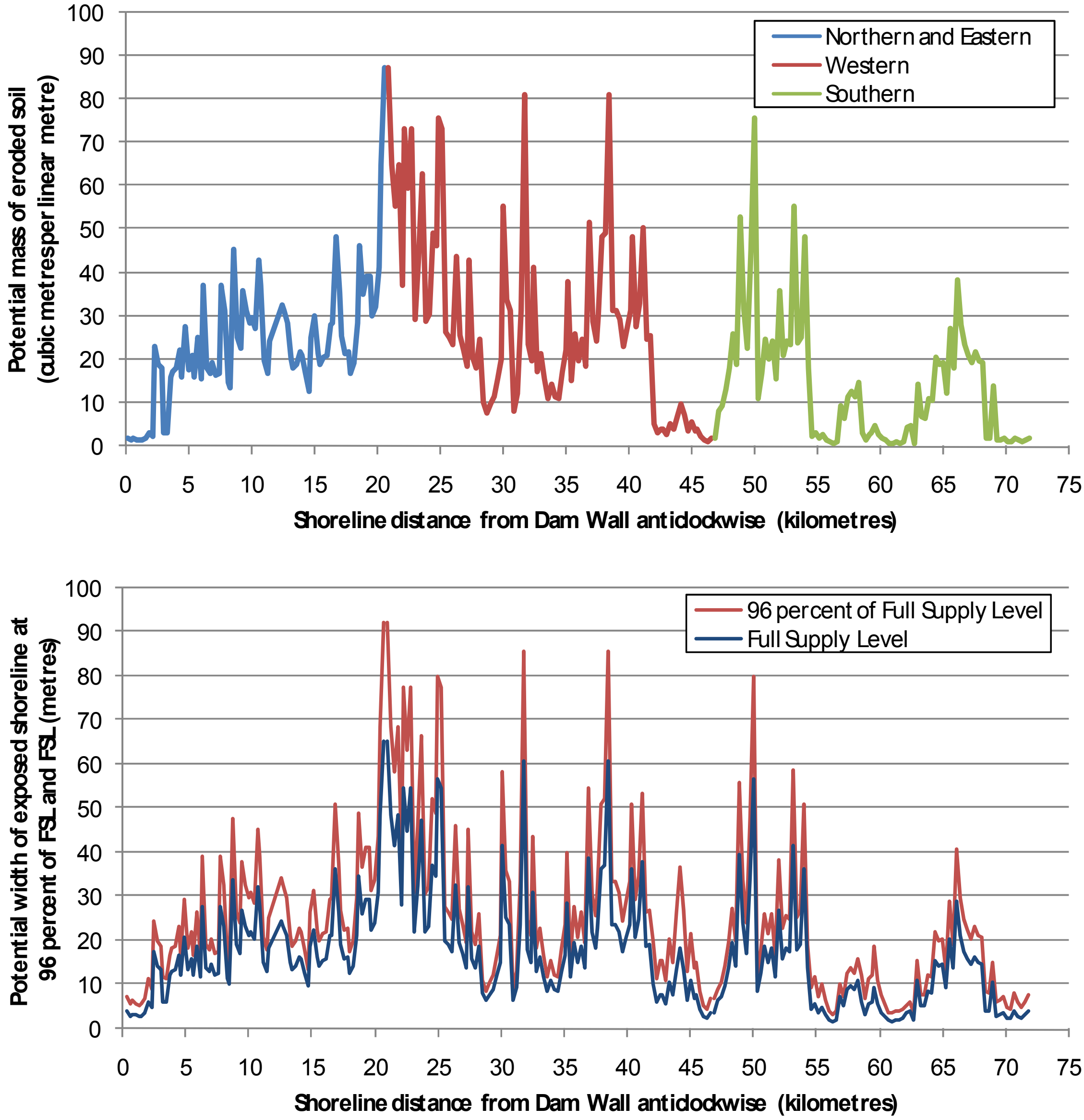

Figure 67. Predicted potential volume of soil eroded from the shoreline of the rim of the impoundment of the proposed Tillegra Dam (top), and the predicted exposed shoreline widths at 96 percent of FSL and at FSL (bottom).

Table 29.
Modelled maximum potential volume of material from shoreline erosion at Tillegra Dam.

Defined Shoreline Zone	Potential load (x 1,000 cubic metres = ML)
Northern and Eastern	471
Western	723
Southern	343
Total	1,537
Percentage of storage capacity	**0.3%**

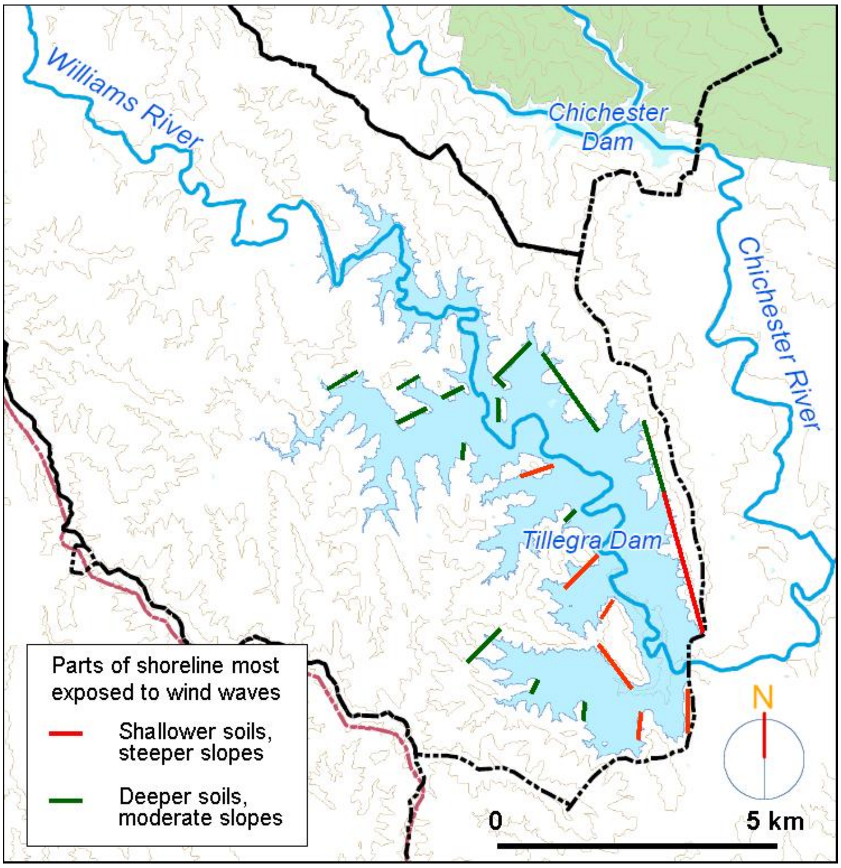

Figure 68. Parts of the shoreline of the impoundment of Tillegra Dam most exposed to the prevailing winds over a long fetch.

Table 30.
Modelled maximum potential exposed shoreline from erosion at the proposed Tillegra Dam.

Defined Shoreline Zone	Potential width of exposed soil on shoreline (m)	
	96 % of FSL	FSL
Northern and Eastern	26 (14)	18 (10)
Western	35 (21)	25 (15)
Southern	18 (14)	12 (10)

3.7 Dam filling phase and drought operation mode

The dam filling phase will be one of no spills from Tillegra Dam. This will be a period of minimal bedload transport in the reach down to the Chichester River junction. In the time taken to fill the dam, there could be an accommodation adjustment to this section of channel (encroachment of vegetation). Upon filling of the dam, and subsequent spilling, the channel would be expected to re-adjust through bed material mobilisation processes (although the new woody vegetation would act to resist this re-adjustment). Sediment starvation would lead to over-adjustment of the channel, as all but the coarsest bed material would eventually be scoured. This process would be most marked closer to the dam.

4 Management and mitigation measures

4.1 Potential issues

The following potential geomorphologic issues have been identified for the Williams River system downstream of the proposed dam at Tillegra:

- Altered frequency, duration and timing of channel maintenance flow events in the Williams River downstream of Tillegra, potentially leading to changes in the physical channel structure that could impact ecological processes. The channel would initially become more stable and have denser in-stream vegetation cover.
- Reduced sediment transport in the Williams River downstream of Tillegra due to trapping by the proposed dam, potentially leading to changes in the physical channel structure that could impact ecological processes. The bed would scour, leaving coarse sized bed material, and the channel bed would deepen. This effect would be partly mitigated by the dam itself, which would reduce the frequency of flows with the capacity to transport coarse bed material. Bedload transport capacity would be reduced downstream of the dam by a factor of three, but scour will occur due to the dam removing the upstream supply that would otherwise replace the transported material.
- Reduction of the base level of the Williams River in the vicinity of the confluence with the Chichester River, due to bed scour, could lead to the migration of a head cut up the Chichester River. This would probably not be of a catastrophic scale because the Chichester river has long been subject to scour due to the existence of Chichester Dam, and because any migrating head cut would only reach as far upstream as the first major bedrock bar, or deposit of coarse, immobile bed material.
- The altered bed material transport regime would present a risk to increasing bank instability, but the risk is considered to be relatively low. Many factors contribute to bank instability, and the existence of bank erosion at the present time demonstrates that at least some of these factors are currently active.
- Risks to stability of in-stream structures, such as revetments and grade control structures. The main risk comes about the potential for bed scour, not from altered hydrology. However, the risk is considered to be relatively low in most cases as general bed lowering is not expected along the length of the river, and these structures were designed to create geomorphologically stable conditions.
- Increased water clarity and lower nutrient concentration in the water immediately downstream of the dam. The difference compared to current would be most apparent during minor to moderate flood events.
- Altered hydrology leading to altered channel and overbank hydraulics, meaning that some physical features such as bars and benches, floodplain surfaces and wetlands, would experience reduced frequency of inundation. The implication of this is reduced opportunities for flushing of carbon and propogules to the river. The vegetation composition and structure on these surfaces could change, with the trend towards terrestrialisation.
- The risk of erosion of the channel banks within the Seaham Weir pool would probably not be increased significantly by operation of a dam at Tillegra.
- The above issues require consideration for the dam filling phase, normal operation mode and drought operation mode, as the pattern of outflows from the dam would be different in each case.

The following potential geomorphologic issues have been identified for the proposed inundation area upstream of the dam wall at Tillegra:

- Erosion of the reservoir shoreline, largely due to the effect of wind waves, leaving an exposed bank, and delivering a volume of eroded soil to the storage. The volume of eroded material would be relatively small and would not significantly threaten the capacity of the Dam (predicted maximum 0.3 percent loss of dam capacity).
- Deposition of river-sourced inflowing bed material within the storage, potentially decreasing its capacity over time. However, the rate in infilling would be very slow and the volumes relatively small, so this process would not significantly threaten the capacity of the Dam (predicted 0.2 percent loss of dam capacity over 100 years).

4.2 Mitigation measures

4.2.1 Flow management

The strategy of releasing flow transfers in the form of a series of pulses of peak magnitude in the order of 1,500 ML/d would inundate the riffles and the lower exposed gravel bars in most places. These pulses would also assist in maintaining clean gravel surfaces free of fine sediment deposits and heavy biofilm build-up. These events are predicted to transport relatively small quantities of bedload. Thus, they do not represent a catastrophic threat to the stability of the bed or banks of the river, nor do they present a major risk to in-stream structures.

The requirements for environmental flows cannot be decided on the basis of geomorphological processes alone. Processes that rely on flows greater in magnitude than 1,500 ML/d will likely suffer reduced frequency of occurrence under a ‘with dam’ scenario. Whether this reduced frequency of geomorphological processes is adequate to maintain the ecological processes that are directly or indirectly dependent on these events is a matter that only expert ecologists can determine.

4.2.2 River management

From a geomorphological perspective, the base case ‘with dam’ flow regime would be better suited to a river of smaller dimensions. Over the long time scale the river will adjust to suit the new regime. The readjustment could involve initial bed scouring, but this is likely to be localised and discontinuous. Mobilisation of bed material could also lead to deposition in places, such as building of in-channel benches at new levels. The predicted bed scour will not necessarily lead to increased rates of bank instability, because the bed level of river is currently fixed in many places by bedrock bars. This situation has a long history, with the river having incised in response to past management practices. The bed material comprises a wide range of sediment sizes. The scour process will selectively sort this material, so that while the fine component would likely be removed from the bed close to the dam wall, the coarse material will remain, and form an armour layer.

The channel may become more heavily vegetated with shrubs and trees. In the past there has been a policy of removing vegetation growing on bars in order to increase conveyance (presumably to reduce flood risk). The dam would have a significant flood mitigation effect, in which case the argument to remove vegetation on the grounds of reducing flood risk would be weakened. Increased riparian and in-stream vegetation is likely to improve habitat conditions for macroinvertebrates and fish. It would also act to slow the bed scouring process. Thus, the recommendation is to allow channel adjustments to take place.

4.2.3 Sediment management

There is little that can be done to prevent the scour process downstream of dams, short of ongoing augmentation of the sediment supply (Bunte 2004). In the United States, gravel augmentation for the purpose of salmonid spawning habitat improvement has been undertaken episodically by various government agencies since the 1960s and 1970s (Bunte 2004). These efforts stepped up after 1992, when there was a change to legislation that requested that all reasonable efforts be made to obtain a sustainable salmon population that would be doubled by 2002. Despite the numerous project undertaken in the USA in the past and underway at the time, Bunte (2004) found little in the way of published technical data.

Merz & Ochikubo Chan (2005) found that cleaned gravels artificially sourced from adjacent floodplain materials were quickly incorporated into the stream ecosystem. Benthic macroinvertebrate assemblages on salmonid spawning enhancement materials, as indicated by species richness, diversity and evenness, were similar to those of adjacent un-enhanced spawning areas within 4 weeks of augmentation and supported higher benthic density and dry biomass for up to 22 weeks after placement.

The feasibility of adding an annual load of one million tonnes of sand to the Colorado River was evaluated by Randle et al (2007). They found local sources of sand, and devised delivery methods that were technically feasible, met environmental requirements, and did not impact cultural resources. However, the supply was expected to last for only one or two decades. The potential of sediment augmentation is currently under investigation in some large rivers in the United States, such as the Colorado (see above), the Platte, Trinity and Tuolumne rivers.

Bed material augmentation downstream of dams is an expensive and logistically difficult procedure, and would only be warranted if it could be demonstrated that there would be no significant negative impacts and the gravel-dependent ecological, economic and social assets of the river were of sufficient value. Many factors related to gravel transport processes are still poorly understood. The outcome of gravel augmentation projects therefore involve a degree of uncertainty. Bunte (2004) suggested that one way forward was to use adaptive management. Under this strategy, the gravel augmentation project would be treated as a scientific experiment with uncertain outcomes, but managers would be prepared to make the necessary adjustments to the programme as more was learned about the process through observation.

4.2.4 Shoreline management

Treatment techniques for managing shoreline erosion range from rock rip-rap and gabion walls to bio-engineering (use of live and dead vegetation for reinforcement and protection of soil). Bio-engineering techniques may provide increased benefits to aquatic habitat, water quality, and aesthetics (USACE 1992). It would be a major undertaking to protect the entire shoreline of the impoundment of Tillegra Dam. However, it may be justified to protect certain areas, depending on their perceived value or intended use.

4.3 Implications of climate change for management

CSIRO (2007) climate change predictions for the Hunter region suggest that rainfall may increase or decrease (equal likelihood) but that evaporation is likely to markedly increase, frequency of high intensity rainfall may increase, and frequency of drought may increase. Also, the chance of extreme winds is likely to increase. The consequences of these changes for the fluvial geomorphology of the Williams River under a 'with dam' scenario are:

- Reduced baseflows in the river, inundating less of the channel, with the implication being:
    - the hydraulics (depth and velocity distributions) of certain geomorphic forms, such as pools, riffles, and undercuts would be different, and possibly less favourable to some biota; in other words, the quality of the hydraulic habitat could be reduced
    - the overall area of hydraulic habitat would be reduced
- Increased frequency of drought period water transfers from Tillegra to Seaham, probably leading to reduced frequency of dam spills, and thus:
    - reduced sediment mobilisation
    - reduced sediment transport (and hence, reduced rate of bed scour)
    - reduced frequency of inundation of defined in-channel surfaces

    [The above impacts could be partly offset or overwhelmed if the frequency of extreme rainfall events (and hence flood flows) increases.]
- Increased frequency of major wind driven shoreline erosion events (increasing the rate of shoreline erosion)

In general, the future climate will likely slow the rate of geomorphic processes, except in the case of shoreline erosion. Thus, mitigation of shoreline erosion, if considered necessary, would need to consider the possibility of more frequent than current extreme wind events.

4.4 Monitoring geomorphological change

There would be some value in including geomorphological variables in an ongoing monitoring program. Bank erosion is a notoriously difficult phenomenon to measure, and even more difficult to explain, as there are numerous contributing factors. Interpretation also requires a long term data set of baseline conditions covering a large number of sites, otherwise it cannot be known whether the measured rate of change is slower or faster than previously, and whether the observed changes are systemic or local. Simply measuring change does not indicate the cause of change.

Bed levels would be more readily surveyed and interpreted, provided sufficient measurements were undertaken (as there is considerable 'noise' in observed bed levels at any particular point on the river). Any long term observed change to bed levels could be attributed directly to the bed material transport process, and this process is predicted to be altered by the proposed dam. Thus, bed stability could be monitored using a hypothesis testing approach, while an investigation of bank stability would likely be confounded.

5 References

Allen, H. 2001. Shoreline erosion control plan, Lake Thunderbird, Cleveland County, Oklahoma. AllEnVironment Consulting, Vicksburg. Oklahoma Water Resources Board, Water Quality Programs Division, Oklahoma City. URL: www.owrb.ok.gov/studies/reports/reports_pdf/thunderbird_erosion.pdf (accessed 10 May 2008).

Ashiq, M., Doering, J.C. and Hosoda, T. 2006. Bed-load transport model based on fractional size distribution. (English). Canadian Journal of Civil Engineering, 33(1): 69.

Bagnold, R.A. 1977. Bedload transport by natural rivers. Water Resources Research, 13: 303-312.

Bagnold, R.A. 1980. An empirical correlation of bedload transport rates in flumes and natural rivers. Proceeding of the Royal Society London, A 372: 453-473.

Blackham, D. 2006. The relationship between flow and stream channel vegetation. Unpublished PhD thesis. The School of Anthropology, Geography and Environmental Studies (SAGES), The University of Melbourne, Parkville.

Brooks, A.P. Babakaiff, S., Gippel, C.J. and Abbe, T.B. 1999. Williams River LWD Reintroduction Project Draft Interim Report: Proposed Study and Structure Designs for Munni Study Reach. Report to Hunter Valley Catchment Management Trust and NSW Department of Land and Water Conservation.

Brooks, A.P., Gehrke, P.C., Jansen, J.D. and Abbe, T.B. 2004. Experimental reintroduction of woody debris on the Williams River, NSW: geomorphic and ecological responses. River Research and Applications 20(5): 513–536.

Brooks, A.P., Howell, T., Abbe, T.B. and Arthington, A.H. 2006. Confronting hysteresis: wood based river rehabilitation in highly altered riverine landscapes of south-eastern Australia. Geomorphology 79: 395–422.

Bunte, K. 2004. State of the Science Review. Gravel Mitigation and Augmentation Below Hydroelectric Dams: A Geomorphological Perspective. Engineering Research Center Colorado State University, Fort Collin. Stream Systems Technology Center USDA Forest Service Rocky Mountain Research Station, Fort Collins, CO.

CSIRO 2007. Climate Change in the Hunter-Central Rivers Catchment. New South Wales Government.

Duan, Q.Y., Sorooshian, S. and Gupta, V. 1992. Effective and Efficient Global Optimization for Conceptual Rainfall-Runoff Models. Water Resources Research 28(4): 1015-1031.

Ekebom, J., Laihonen, P. and Suominen, T. 2002. Measuring fetch and estimating wave exposure in coastal areas. Littoral 2002, The Changing Coast. EUROCOAST / EUCC, Porto, Portugal, pp. 155-160.

Elçi, S. and Work, P.A. 2003. Prediction of reservoir shoreline erosion. Proceedings of the 2003 Georgia Water Resources Conference, 23-24 April 2003, University of Georgia. K.J. Hatcher (ed.), Institute of Ecology, The University of Georgia, Athens, Georgia. URL:

http://cms.ce.gatech.edu/gwri/uploads/proceedings/2003/Elci%20and%20Work.PDF (accessed 10 May 2008).

Elçi, S., Work, P.A. and Hayter, E.J. 2007. Influence of stratification and shoreline erosion on reservoir sedimentation patterns. Journal of Hydraulic Engineering 133(3): 255-266.

Elliott, J.G. and Hammack, L.A. 2000. Entrainment of riparian gravel and cobbles in an alluvial reach of a regulated canyon river. Regulated Rivers: Research & Management, 16: 37-50.

Erskine, W.D. 1985. Downstream geomorphic impacts of large dams: the case of Glenbawn Dam, NSW. Applied Geography 5(3): 195-210.

Erskine, W.D. 1986. River metamorphosis and environmental change in the Hunter Valley, New South Wales. PhD thesis, University of New South Wales, Kensington.

Erskine, W.D. 1998. An independent assessment of the fluvial geomorphology of the Williams River and its relevance to future river management activities. NSW Department of Land and Water Conservation, Newcastle.

Erskine, W.D. 2001. Geomorphic evaluation of past river rehabilitation works on the Williams River, New South Wales. Ecological Management & Restoration 2(2): 116-128.

Erskine, W.D. and Bell, F.C. 1982. Rainfall, floods and river channel changes in the upper Hunter. Australian Geographical Studies 20: 183-196.

Erskine, W.D. and Livingstone, E.A. 1999. In-channel benches: the role of floods in their formation and destruction on bedrock confined rivers. In: Varieties of Fluvial Forms (eds A. Miller and A. Gupta). John Wiley and Sons, New York, pp. 445–475.

Ferguson, R.I. 2005. Estimating critical stream power for bedload transport calculations in gravel-bed rivers. Geomorphology, 70: 33-41.

Galloway, R.W. 1963. Geomorphology of the Hunter Valley. CSIRO (Australia) Land Research Series 8: 90–102.

GHD 2006. Williams River Bank Erosion Study, Final Report. Port Stephens Council, Raymond Terrace, August.

Gippel, C.J. 1995. Potential of turbidity monitoring for measuring the transport of suspended solids in streams. Hydrological Processes 8: 83-97.

Gippel, C.J. 2002. Geomorphic issues associated with environmental flow assessment in alluvial non-tidal rivers. Australian Journal of Water Resources 5(1): 3-19.

Gordon, N.D., McMahon, T.A., Finlayson, B.L., Gippel, C.J. and Nathan, R.J. 2004. Stream Hydrology: An Introduction for Ecologists. Second Edition, John Wiley & Sons, Chichester.

Groeneveld, D.P. and French, R.H. 1995. Hydrodynamic control of an emergent aquatic plant (scirpus acutus) in open channels. Water Resources Bulletin 31: 505-514.

Guscio, F.J., Bartley, T.R. and Beck, A.N. 1965. Water resources problems generated by obnoxious plants. Proceedings of the American Society of Civil Engineers, Journal of the Waterways and Harbours Division 10: 47-60.

Healthy Rivers Commission of New South Wales 1996. Independent Inquiry into the Williams River, Final Report, December.

Henderson, L.E. 2000a. Soil landscapes of the Dungog 1:100,000 Sheet Report. Department of Land and Water Conservation, Sydney.

Henderson, L.E. 2000b. Soil landscapes of the Dungog 1:100,000 Sheet Map. Department of Land and Water Conservation, Sydney.

Hudson, N. 1971. Soil Conservation, Cornell University Press, Ithaca.

Hupp, C.R. and Osterkamp, W.R. 1996. Riparian vegetation and fluvial geomorphic processes. Geomorphology, 14: 277-295.

HWC 2006. Tillegra Dam - Fact Sheet (December 2006), Hunter Water, Newcastle, NSW.

IEA 1988. Australian Rainfall and Runoff; A Guide to Flood Estimation. Institution of Engineers Australia, Barton.

Ikeda, S. and Izumi, N. 1990. Width and depth of self-formed straight gravel rivers with bank vegetation. Water Resources Research 26(10): 2353-2364.

Joeckel, R.M. and Diffendal Jr, R.F. 2004. Geomorphic and environmental change around a large, aging reservoir: Lake C. W. McConaughy, Western Nebraska, USA. Environmental and Engineering Geoscience 10(1): 69-90.

Kadlec, R.H. and Knight, R.L. 1996. Treatment Wetlands. CRC Press, Florida, U.S.A.

Keller, E.A. 1971. Areal sorting of bed load material: the hypothesis of velocity reversal. Bulletin of the Geological Society of America 82: 753-756.

Kirk, R.M., Komar, J.C. and Stephenson, W.J. 2000. Shoreline erosion on Lake Hawea, New Zealand, caused by high lake levels and storm-wave runup. Journal of Coastal Research 16(2): 346-356.

Krause, A.K., Loughran, R.J. and Kalma, J.D. 2003. The use of caesium-137 to assess surface soil erosion status in a water-supply catchment in the Hunter Valley, New South Wales, Australia. Australian Geographical Studies 41(1): 73–84.

Kriek, P.N. 1971. Sedimentation research in the Hunter Valley. Monograph No. 36. The Hunter Valley Research Foundation, Newcastle, New South Wales, 98 pp.

Lorang, M.S. and Stanford, J.A. 1983. Variability of shoreline erosion and accretion within a beach compartment of Flathead Lake, Montana. Limnol. Oceanogr., 38(8): 1783-l 795.

Marston, R.A., Girel, J., Pautou, G., Piegay, H., Bravard, J-P. and Arneson, C. 1995. Channel metamorphosis, floodplain disturbance, and vegetation development - Ain River, France. Geomorphology 13(1-4): 121-131.

Merz, J.E. and Ochikubo Chan, L.K. 2005. Effects of gravel augmentation on macroinvertebrate assemblages in a regulated Californian river. River Research and Applications 21: 61-74.

MHL 2003. Hunter Estuary Process Study. Report MHL 1095. DPWS Report No. 01010. Manly Hydraulics Laboratory, Department of Commerce NSW, Manly Vale, November.

Mierau, R. and Trimble, P. 1988. Hydrologic Characteristics of the Kissimmee River floodplain Boney Marsh experimental area, South Florida Water Management District Technical Memorandum, West Palm Beach, FL.

Nakamura, F., Swanson, F.J. and Wondzell, S.M. 2000. Disturbance regimes of stream and riparian systems - a disturbance-cascade perspective. Hydrological Processes 14(16-17): 2849-2860.

National Land and Water Resources Audit 2001. Sediment and nutrient supply to river links. Natural Resources Data Library, Commonwealth of Australia, Turner. URL: http://adl.brs.gov.au/anrdl/php/full.php?fileidentifier=http://adl.brs.gov.au/findit/metadata_files/a_scdrlr9cl__01011a00.xml (accessed 24th August 2007).

National Land and Water Resources Audit 2002. Australian Natural Resources Data Library. Commonwealth of Australia, March. URL: http://audit.ea.gov.au/anra/atlas_home.cfm (accessed 24th August 2007).

Neill, C.R. 1968. A re-examination of the beginning of movement for coarse granular bed materials. Hydraulics Research Station Report No. INT 68, Wallingford, UK. 37p.

Nilsson, C. and Svedmark, M. 2002. Changing water regimes: riparian plant communities. Environmental Management 30(4): 468-480.

Patterson Britton and Partners Pty Ltd. 1996. The effects of wave action and other influences on the water quality and ecology of Seaham Weir pool. Hunter Catchment Management Trust, Maitland.

Phillips JD, Slattery MC & Musselman ZA 2005. Channel adjustments of the lower Trinity River, Texas, downstream of Livingston Dam. Earth Surf. Process. Landforms 30, 1419–1439.

Prosser, I.P. and Slade, C.J. 1994. Gully formation and the role of valley-floor vegetation, southeastern Australia. Geology 22(12): 1127-1130.

Prosser, I.P., Rustomji, P., Young, B., Moran, C. and Hughes, A. 2001a. Constructing river basin sediment budgets for the National Land and Water Resources Audit, Technical Report 15/01, CSIRO Land and Water, Canberra, July. URL: http://audit.ea.gov.au/anra/land/docs/national/river_sediment_budget.pdf (accessed 24 August 2007).

Prosser, I.P., Hughes, A., Rustomji, P., Young, W.J. and Moran, C.J. 2001b. Predictions of the sediment regime of Australian rivers. In I.D. Rutherfurd, F. Sheldon, G. Brierley and C. Kenyon (eds), Third Australian Stream Management Conference, Brisbane, 27–29 August, Cooperative Research Centre for Catchment Hydrology, pp. 529–533.

Randle, T.J., Lyons, J.K., Christensen, R.J. and Stephen, R.D. 2007. Colorado River Ecosystem Sediment Augmentation Appraisal Engineering Report. Bureau of

Reclamation, Department of the Interior. URL: http://www.gcmrc.gov/library/reports/physical/Fine_Sed/Randle2007.pdf (10 May 2008).

Reid, L.M. 1989. Erosion of Grassed Hillslopes, University of Washington, Washington.

Richards, K. 1982. Rivers: Form and Process in Alluvial Channels. Methuen & Co. London.

Rutherfurd, I.D., Abernathy, B. and Prosser, I. 1999. Stream erosion. In: P. Price and S. Lovett (Editors), Riparian Land Management Technical Guidelines. LWRRDC, Canberra, Australia.

Shih, S.F. and Rahi, G.S. 1982. Seasonal variations of Manning's roughness coefficient in a subtropical marsh. Transactions of the ASAE 25(1): 116-120.

Tabacchi E., Correl, D.L., Hauer, R., Pinay, G., Planty-Tabacchi, A.M. and Wissmar, R.C. 1998. Development, maintenance and role of riparian vegetation in the river landscape. Freshwater Biology 40(3): 497-516.

Trimble, S.W. 1997. Stream channel erosion and change resulting from riparian forests. Geology 25: 467-9.

USACE 1992. Bioengineering technique of reservoir shoreline erosion control in Germany. Suppl 5, REMR Technical Note GT-SE-1.5. US Army Corps of Engineers. Environmental Laboratory. URL: el.erdc.usace.army.mil/elpubs/pdf/wgsw3-1.pdf (accessed 10 May 2008).

Walker, J.P., Willgoose, G.R. and Kalma, J.D. 2001. The Nerrigundah data set: soil moisture patterns, soil characteristics, and hydrological flux measurements. Water Resources Research 27(1): 2653-2658.

Zimmerman, R.C., Goodlett, J.C. and Comer, G.H. 1967. The influence of vegetation on channel form of small streams. In The Symposium on River Morphology, Publication 75, International Association of Hydrological Sciences, Bern, pp. 255-275.