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Dam Removal Express Assessment Models (DREAM). Part 1: Model development and validation

Les modèles DREAM d'évaluation rapide d'effacement de barrage. Partie 1: Développement du modèle et validation

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ABSTRACT

Many dams have been removed in the recent decades in the U.S. for reasons including economics, safety, and ecological rehabilitation. More dams are under consideration for removal; some of them are medium to large-sized dams filled with millions of cubic meters of sediment. Reaching a decision to remove a dam and deciding as how the dam should be removed, however, are usually not easy, especially for medium to large-sized dams. One of the major reasons for the difficulty in decision-making is the lack of understanding of the consequences of the release of reservoir sediment downstream, or alternatively the large expense if the sediment is to be removed by dredging. This paper summarizes the Dam Removal Express Assessment Models (DREAM) developed at Stillwater Sciences, Berkeley, California for simulation of sediment transport following dam removal. There are two models in the package: DREAM-1 simulates sediment transport following the removal of a dam behind which the reservoir deposit is composed primarily of non-cohesive sand and silt, and DREAM-2 simulates sediment transport following the removal of a dam behind which the upper layer of the reservoir deposit is composed primarily of gravel. Both models are one-dimensional and simulate cross-sectionally and reach averaged sediment aggradation and degradation following dam removal. DREAM-1 is validated with a set of laboratory experiments; its reservoir erosion module is applied to the Lake Mills drawdown experiment. DREAM-2 is validated with the field data for a natural landslide. Sensitivity tests are conducted with a series of sample runs in the companion paper, Cui *et al.* (2006), to validate some of the assumptions in the model and to provide guidance in field data collection in actual dam removal projects.

RÉSUMÉ

De nombreux barrages ont été supprimés dans les récentes décennies aux U.S.A. pour des raisons comprenant l'économie, la sûreté, et la réhabilitation écologique. Davantage de barrages sont à l'étude pour leur déplacement; certains d'entre eux sont de taille moyenne ou grande et remplis de millions de mètres cubes de sédiment. Prendre une décision de suppression de barrage et décider comment il doit être enlevé, n'est cependant pas une chose aisée, surtout dans le cas des barrages moyens ou grands. Une des raisons majeures de la difficulté dans la prise de décision est le manque de connaissance des conséquences d'une libération des sédiments de la retenue vers l'aval, ou alors, le coût élevé de leur élimination par dragage. Cet article récapitule les modèles DREAM d'évaluation rapide de suppression de barrage, développés au Stillwater Sciences de Berkeley, Californie pour la simulation du transport de sédiment suite à un effacement de barrage. Il y a deux modèles dans le package: DREAM-1 simule le transport de sédiment consécutif à l'effacement d'un barrage derrière lequel le dépôt de retenue se compose principalement de sable non cohésif et de vase, et DREAM-2 simule le même phénomène, mais dans le cas où la couche supérieure du dépôt de retenue se compose principalement de gravier. Les deux modèles sont unidimensionnels et simulent en coupe l'aggradation et la dégradation moyennes de sédiment consécutives à l'effacement du barrage. DREAM-1 est validé avec un ensemble d'expériences de laboratoire; son module d'érosion de retenue est appliqué à l'expérience de vidange de Lake Mills. DREAM-2 est validé avec les données de terrain d'un éboulement naturel. Les essais de sensibilité sont conduits avec une série de tests de l'article associé, de Cui *et al.* (2006), pour valider certaines des hypothèses du modèle et pour fournir des conseils sur la collecte des données naturelles pour les projets réels de suppressions de barrage.

Keywords: Dam removal, dam decommissioning, sediment transport, numerical model.

1 Introduction

Citing the U.S. Army Corps of Engineers, the Bureau of Reclamation, the Tennessee Valley Authority and other U.S. sources, The Guidelines for Retirement of Dams and Hydroelectric Facilities (the Guidelines hereafter, ASCE, 1997) state that there were more than 75,000 dams in the U.S. in 1996. The majority of these dams were built before the late 1960s, and are now approaching

or exceeding their average designed life expectancy of about 50 years. In light of the aging of these facilities and in light of economic and ecological considerations, some dams have been decommissioned and removed, and many more will be removed in the future. As pointed out in the Guidelines, the key element in a dam removal project is usually sediment management, which normally constitutes more than a third of the total dam removal cost. The Guidelines listed three sediment management options associated with dam removal: river erosion, mechanized removal, and stabilization, each with different advantages and disadvantages. Overall, mechanized removal has the least impact on the downstream geomorphic/ecological system but has the highest cost. In contrast to mechanized removal, the river erosion option has the greatest downstream impact but the lowest cost. Within each individual option, there may be many implementation alternatives, and each of them may have different downstream impacts and project costs. In the river erosion option, for example, the dam can be partially or completely removed by either a one-shot removal (i.e., to remove the entire dam before reservoir sediment deposit is allowed to erode and transport downstream) or a staged removal. The choice of a removal method among available options and the variety of design alternatives within an option are largely determined by the predicted downstream impacts of the sediment release, as well as the confidence level of the predictions.

Because dam removal is a relatively recent issue, and because of the complexities involved in sediment transport following dam removal, a sediment transport model designed to simulate dam removal and the eventual fate of the reservoir sediment has not been available to the public. Instead, engineers and geomorphologists have been using sediment transport models that were developed for other purposes to address the problem. For example, HEC-6, in combination with several other reservoir erosion models, was used to model the proposed removal of the Elwha and Glines Canyon Dams on the Elwha River, WA (Bureau of Reclamation, 1996b). The problem with such a modeling exercise is that the sediment transport model used for simulation, HEC-6 in this particular case, was not developed for simulation following the removal of a dam, and thus is not capable of simulating the steep slope in the vicinity of the dam immediately following removal. A practice modelers have adopted to overcome such problems is to model the reaches upstream and downstream of the dam separately. That is, reservoir erosion upstream of the dam is simulated independently, and the results are used to define the upstream boundary condition for the simulation of the downstream reach (e.g., Bureau of Reclamation, 1996b). This practice, however, is valid only if part of the dam is still in place and the upstream and downstream reaches of the dam are still separated by the remaining portion of the dam. That is, the combined models cannot be used for the simulation of a one-shot removal, nor can they be used for simulation of the later stages in a staged removal. In such cases, the deposition downstream of the dam greatly affects the erosion and transport of sediment in and upstream of the reservoir, and thus the independent simulation of reservoir erosion upstream of the dam becomes invalid.

To simulate the potential removal of Soda Springs Dam on the North Umpqua River, OR, and Marmot Dam on the Sandy River, OR, Stillwater Sciences developed two customized numerical models that specifically address the sediment transport issues following the removal of the dams (Stillwater Sciences, 1999, 2000; Cui and Wilcox, 2006) based on the sediment pulse work of Cui and Parker (2005). In the Soda Springs Dam case, the reservoir deposit is composed primarily of sand and silt, and the river downstream of the dam is a high-gradient bedrock-dominated gravel-bedded channel (Stillwater Science, 1999). In the Marmot Dam case, the reservoir deposit is stratified, with the upper layer of the deposit composed of a mixture of gravel and sand, and the lower layer composed of primarily sand and silt. The Sandy River downstream of Marmot Dam is a high-gradient bedrock-dominated gravel-bedded river, with a gradual transition further downstream to a lower-gradient gravel-bedded river (Stillwater Sciences, 2000; Cui and Wilcox, 2006).

The Dam Removal Express Assessment Models (DREAM) presented in this paper are modified from the Soda Springs Dam and the Marmot Dam models: DREAM-1 is designed for the simulation of sediment transport following the removal of a dam behind which the reservoir deposit is composed primarily of non-cohesive sand and silt, and DREAM-2 is designed for the simulation of sediment transport following the removal of a dam behind which the upper layer of the reservoir deposit is composed primarily of gravel. Channel characteristics can include any combination of bedrock, gravel-bedded and sand-bedded rivers for a DREAM-1 simulation, and a combination of bedrock and gravel-bedded rivers for a DREAM-2 simulation.

The Marmot Dam removal model (Cui and Wilcox, 2006) differs from the generic model of Cui and Parker (2005) in that Cui and Parker (2005) assumes gravel-bedded without geological controls such as bedrock outcrops while Cui and Wilcox (2006) allows such geological controls. The implication is that the pre-disturbance bedload transport in Cui and Parker (2005) is at capacity while the pre-dam-removal condition in Cui and Wilcox (2006) can be under-capacity. The other major difference between Cui and Parker (2005) and Cui and Wilcox (2006) is that Cui and Parker (2005) considers only bedload transport, while the Marmot Dam removal model (Cui and Wilcox, 2006) considers the transport of both gravel and sand. In addition, Cui and Parker (2005) uses only one discharge station for input to the model and assumes that the discharge at any cross section is proportional to the local drainage area. The Marmot Dam removal model (Cui and Wilcox, 2006) allows any number of hydrologic stations, and the discharge at each cross section can be linked to one of those stations.

The major improvement of DREAM-1 over the Soda Springs Dam removal model (Stillwater Sciences, 1999) is that the current model assumes trapezoidal cross sections in the reach upstream of the dam, and allows channel widening due to the erosion of both banks during the period of downcutting of the reservoir deposit, while the Soda Springs Dam removal model (Stillwater Sciences, 1999) assumes set rectangular cross sections for the entire river reach. This improvement is also reflected in DREAM-2 presented in this paper as compared to the Marmot Dam removal model (Cui and Wilcox, 2006). In addition, the gravel and sand transport models are built as an integrated model in DREAM-2,

although gravel and sand transport capacities are still calculated separately with their respective equations, representing another major improvement over the Marmot Dam removal model (Cui and Wilcox, 2006), in which the gravel model is run independently and the resulting fine sediment erosion from the reservoir deposit is used as input to the sand model. The integration of gravel and sand transport into a single model allows the fine sediment generated from gravel abrasion to be accounted for. The integrated model also allows accounting for the fine sediment deposited in the interstices of gravel deposits.

In addition to these major improvements of the current models over the previous models, this paper and the companion paper, Y. Cui, C. Braudrick, W.E. Dietrich, B. Cluer and G. Parker (unpublished data) focus on different issues of interest from (a) those in Cui and Parker (2005), which focuses on the relative importance of gravel abrasion on the evolution of gravel pulses in mountain rivers, and (b) those in Cui and Wilcox (2006), which presents a case study of a dam removal project. This and the companion paper, Cui *et al.* (2006), focus on: (a) the development of the two models and the underlying assumptions, with special attention to the reservoir erosion module; (b) validation of the models with field and laboratory data; and (c) sensitivity tests to major fixed and user-defined parameters, which provide guidance for future model applications and field data collection, and provide a reference for development of similar models in the future.

2   Hypotheses on the morphologic adjustments and sediment transport processes following dam removal and governing equations

Many small dams have been removed in the U.S. and around the world with very little documentation. Removal of medium- to large-sized dams is very rare, and no documentation of morphological adjustments and sediment transport processes associated with such cases was found. In order to develop the DREAM, we hypothesized the following morphologic adjustments and sediment transport processes following a dam removal.

Consider a dam, behind which the reservoir area is either fully or partially filled with non-cohesive sediment, which is in the process of being analyzed for removal. To begin the dam removal process, as much water as possible is drained out of the reservoir during the low flow season and a cofferdam is constructed at a certain distance upstream of the dam to divert the flow away from the dam. With the protection from the cofferdam, the sediment between the dam and the cofferdam is excavated to expose and eventually remove the dam. The dam can be a one-time complete removal (one-shot removal), a partial removal across the dam, or the opening of a notch at the dam. The cofferdam is then artificially or naturally breached at a design discharge after the dam and other facilities are physically removed. In case of opening a notch on the dam, it is assumed that no flow control structure is installed on the notch, i.e. free-surface flow is maintained at all times. The regulated (gated) notch such as in one of the options for the proposed Glines Canyon Dam removal (Bureau of Reclamation, 1996a) cannot be modeled with the current model without site-specific modification to the code.

Note that the sediment deposit in the reservoir following the removal of the dam and before the breaching of the cofferdam has steep slope facing downstream (often at the angle of repose), as demonstrated in the sketch in Fig. 1(a). The sketches in Figure 1 are vertically exaggerated, and as a result, the above slope appears much steeper than the angle of repose. This steep slope allows for quick erosion of the reservoir sediment and its subsequent deposition downstream as a fan-delta as soon as the cofferdam is breached. The rapid downcutting of the reservoir deposit very likely drains the flow from any existing secondary channels, thus preventing further downcutting and leaving them perched. As a result, it is very likely that only one channel is formed in the reservoir deposit following the removal of the dam. Due to the lack of field data, it is not clear how wide a channel develops in the reservoir reach. It is reasonable, however, to assume that the active channel will have a geometry similar to that found in the reach immediately downstream of the dam. Also because the channel will tend to cut down rapidly, it will very likely experience relatively minor lateral migration. Depending on the relative widths of the reservoir deposit and the active channel, part of the reservoir deposit may not be eroded and transported downstream even if the channel reaches its pre-dam gradient. This deposit remains
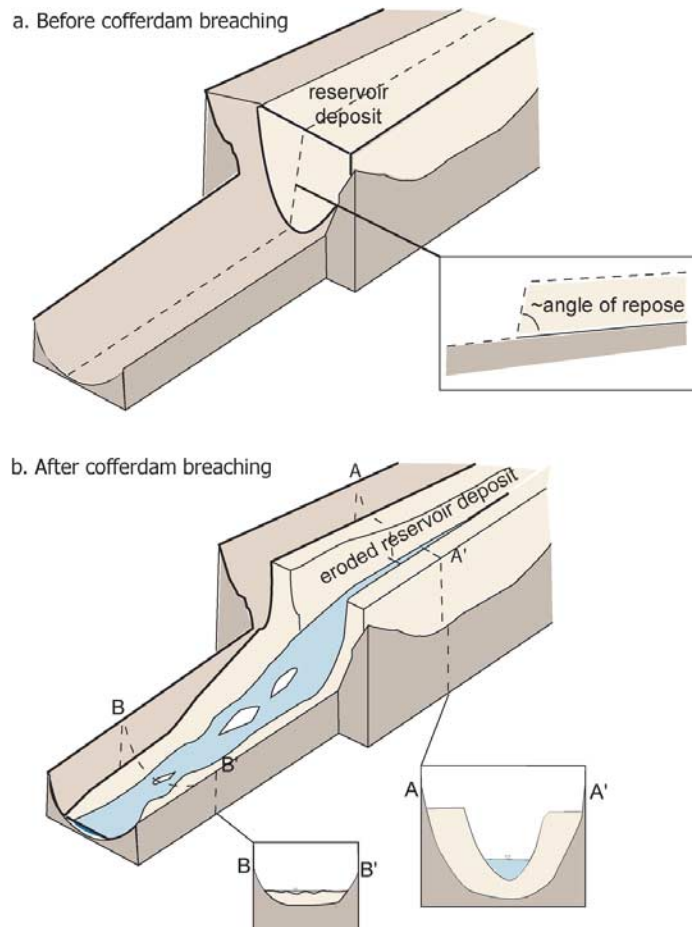

Figure 1   Sketch of the typical geomorphic characteristics of a dam removal project.

in the form of terraces. Once the channel reaches a relatively stable gradient and the degradation rate falls off, the channel may begin to migrate laterally and to erode these terraces. Figure 1(b) shows a sketch of a river after dam removal that is still adjusting its gradient. In case the upper layer of the reservoir deposit is composed of primarily coarse sediment (gravel, pebbles, and boulders), it is reasonable to assume that the erosion of the reservoir deposit is governed by gravel transport because of the relatively smaller transport capacity of coarse sediment compared to that of sand.

The above hypotheses are incorporated into the DREAM presented below. The possible lateral migration once the channel reaches a relatively stable gradient, however, is not implemented in the models.

A unique feature of sediment transport modeling following dam removal is the steep slope in the vicinity of the dam shortly after dam removal. Simulation with a steep slope requires that the model be capable of simulating sub-critical, super-critical, and transient flows.

Many existing numerical models of mobile-bed open-channel flow are equipped with the ability to simulate sub-critical, super-critical, and transient flows (e.g. Li *et al.*, 1988; Rahuel *et al.*, 1989; Holly and Rahuel, 1990a,b; Bhallamudi and Chaudhry, 1991; US Army Corps of Engineers, 1993; Cui *et al.*, 1996, 2003b; Cui and Parker, 1997, 2005). With appropriate modifications, those models should be capable of simulating sediment transport processes following a dam removal. The same procedure as used in Cui and Parker (2005) and Cui and Wilcox (2006) is applied in the flow simulation described below.

For the purpose of flow calculation, the channel is assumed to be rectangular with width equal to the local bankfull width, and flow parameters are calculated with a combination of a standard backwater calculation and the quasi-normal flow assumption,

$$
\begin{aligned}
\frac{\mathrm{d}h}{\mathrm{d}x} &= \frac{S_0 - S_f}{1 - F^2}, & F < 0.9 \\
S_0 &= S_f, & F \geq 0.9
\end{aligned}
\tag{1a}
$$

in which h denotes water depth; x denotes downstream distance; S_0 denotes channel bed slope; S_f denotes friction slope; and F denotes the Froude number;

$$
S_0 = -\frac{\partial(\eta_b + \eta_g + \eta_s)}{\partial x}
\tag{2}
$$

$$
F^2 = \frac{Q_w^2}{gB^2h^3}
\tag{3}
$$

in which η_b denotes the elevation of non-erodible material such as bedrock; η_g denotes the thickness of the gravel deposit; η_s denotes the thickness of any sand deposit on top of gravel deposit or the non-erodible material; Q_w denotes water discharge; g denotes acceleration of gravity; and B denotes bankfull channel width. It should be noted that bankfull channel width upstream of the dam site follow dam removal is assumed to be equal to the average bankfull width within a short distance downstream of the dam. The friction slope S_f will be discussed later in conjunction with the discussion of sediment transport equation. A sketch
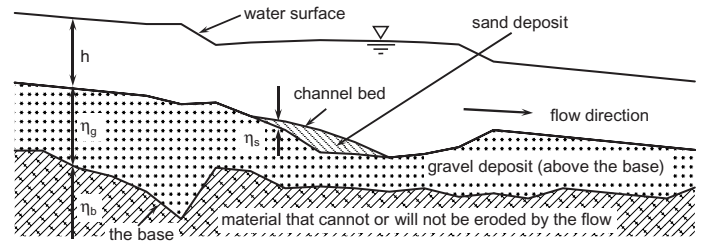

Figure 2   Sketch defining some of the terms used in the models.

illustrating these definitions is given as Fig. 2. It needs to be clarified that the thickness of gravel deposit, η_g, should be considered as constant in a DREAM-1 simulation based on the assumption that the aggradation and degradation of the gravel bed is relatively slow compared to the transport of sand. This assumption is not used in DREAM-2, which calculates gravel as well as sand transport.

Cui and Parker (1997) show that the quasi-normal assumption provides a good approximation of the full backwater equations for flows with high Froude number. Cui and Parker (2005) use this finding to simulate the evolution of sediment pulses in mountain rivers. In Cui and Parker (2005), the flow is calculated with the backwater equation whenever local Froude number is lower than 0.75, and with quasi-normal assumption otherwise. Their simplified treatment enables them to model the sub-critical flow upstream of the sediment pulse, the super-critical flow at the steep downstream face of the sediment pulse, and the transient flows linking the two states.

This simplified treatment recognizes the fact that sediment transport simulations are almost always performed at a much larger grid scale than the scale of the transient flow so that it is not necessary to capture the exact location of transient features such as hydraulic jumps. For example, the typical grid for a one-dimensional sediment transport simulation is on the order of two channel widths or larger. The scale of a hydraulic jump in the mean flow transition from super-critical flow to sub-critical flow, however, is usually less than the channel width. With this in mind, the exact location of the hydraulic jump is not important in a one-dimensional sediment transport simulation, as long as one can predict the two grid points between which the hydraulic jump is located, as demonstrated in Fig. 3.

Extensive comparisons of the simplified treatment with more complicated methods by the first author indicate that the simplified treatment produces results almost identical to that from the other methods. Such an example is presented in Fig. 4 for the simulation of Run 2 of the St. Anthony Falls Laboratory (SAFL) downstream fining experiments reported by Paola *et al.* (1992), Seal *et al.* (1997) and Toro-Escobar *et al.* (1996), and previously simulated by Cui *et al.* (1996) and Cui and Parker (1997). Details of the SAFL downstream fining experiments and the numerical simulations by Cui *et al.* (1996) and Cui and Parker (1997) are not discussed here. Of interest here is the fact that Cui *et al.* (1996) applied a time-relaxation method to solve the full St. Venant equations in order to solve the transient flow, and Cui and Parker (1997) applied a shock-fitting method to locate the exact location of the hydraulic jump. Figure 4 indicates that the
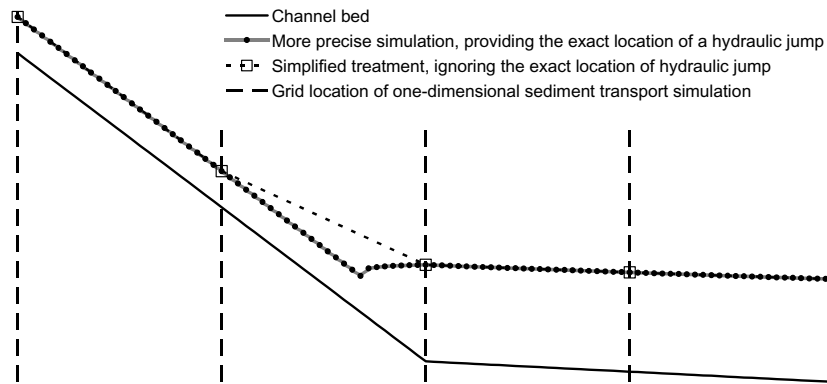
Figure 3   Sketch of simulated water surface profile, demonstrating the validity of the simplified treatment.
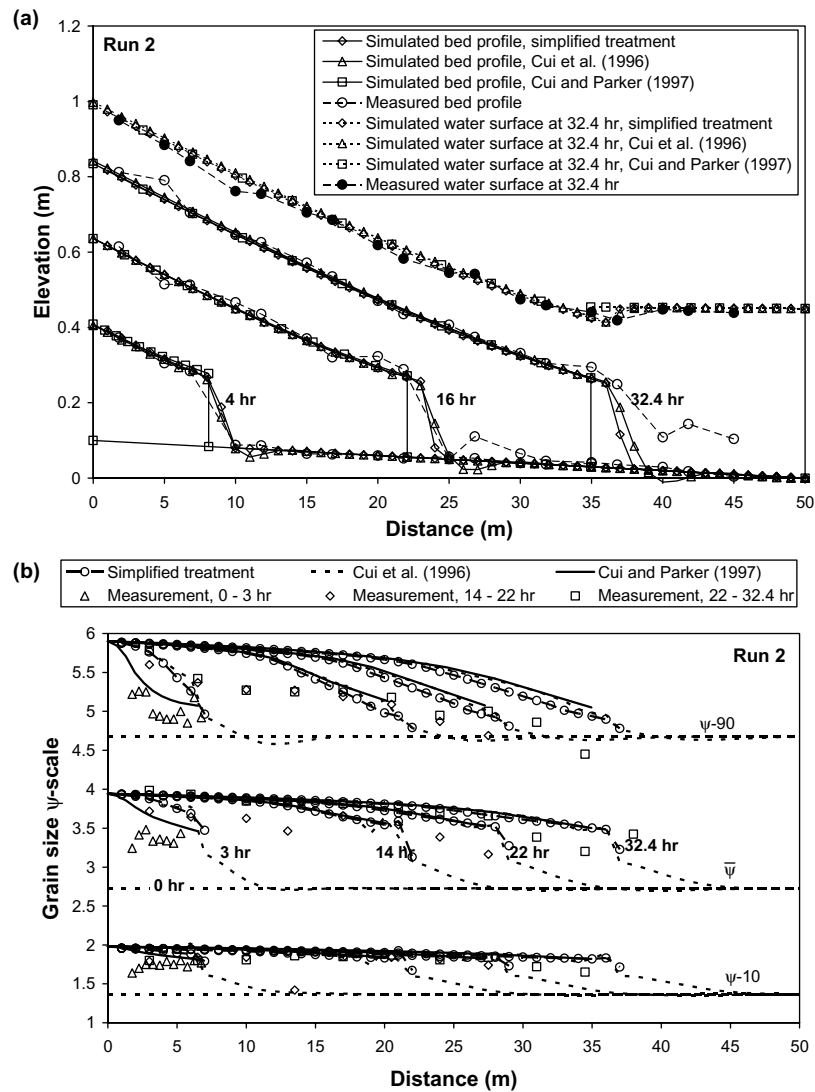

Figure 4   Simulation of SAFL downstream fining Run 2, demonstrating that the simplified treatment of flow simulation is adequate for sediment transport simulation. (a) water surface and bed elevations; (b) characteristic grain size using the ψ-scale defined in Eq (4).

simplified treatment produced results almost identical to those of the more complicated treatments of Cui *et al.* (1996) and Cui and Parker (1997). Comparisons for other SAFL downstream fining runs produced results similar to that shown in Fig. 4 and are not presented here.

It should be noted that in order to achieve relatively high accuracy in flow calculation, the coarse grid shown in Fig. 3

may have to be divided into a number of sub-grids for the back-water calculation. The number of sub-grids within a particular grid is dependent on the ratio of the grid length to water depth, Froude number, and local channel bed slope. Details about the sub-grids are not discussed here because the implementation of the technique is more of a trial-and-error exercise than a solid theory. It is useful, however, to point out that increases in the

ratio of grid length to water depth, Froude number, or local channel bed slope should normally result in an increase in the number of sub-grids within the grid in order to achieve a similar relative accuracy at all the grid points as illustrated in Fig. 3.

For the purpose of sediment mass conservation calculations, the channel downstream of the dam is assumed to have the same rectangular cross-sections as those used in flow calculation. The Exner equations of sediment continuity for the reach downstream of the dam used here have been modified from Cui and Parker (2005), which in turn have their origins in continuous forms in Parker (1991a,b). Similar but simpler forms of the Exner equations have been used in Parker (1990b), Cui and Parker (1998) and Cui and Wilcox (2006).

Since sediment transport of gravel is computed on a grain size-specific basis, it is first necessary to specify the discretization of the gravel grain size distribution. Here "gravel" means gravel and coarser sizes. Grain size D can be equivalently characterized in terms of the (base-2) logarithmic ψ-scale;

$$\psi = -\phi = \log_2(D) \qquad (4)$$

In the above relation ϕ denotes the ϕ-scale familiar to sedimentologists. Gravel grain size distributions are discretized into N bins $j = 1, \ldots, N$ bounded by $N + 1$ grain sizes $D_1, \ldots, D_{N+1}$ ($\psi_1, \ldots, \psi_{N+1}$) progressing from smaller to larger size with increasing j. Here D_1 always corresponds to 2 mm (i.e. a value of ψ_1 of 1), i.e. the border between sand and gravel. The jth grain size range is bounded by the sizes D_j and D_{j+1}, and has the characteristic size

$$\bar{D}_j = \sqrt{D_j D_{j+1}}, \quad \bar{\psi}_j = \log_2(\bar{D}_j) = \frac{1}{2}(\bar{\psi}_j + \bar{\psi}_{j+1}) \qquad (5a,b)$$

The fraction of the deposit that is gravel is denoted as f_g and the fraction that is sand is denoted as f_s. The two need not add up to unity due to the possible presence of silt in the deposit. The fractions of the gravel in the surface layer of the stream and the bedload in the jth grain size range are denoted respectively as F_j, and p_j, where both are normalized to sum to unity over all gravel sizes. The formulation presented below also uses surface fractions F_j' that have been adjusted according to Parker (1991a) to reflect exposed surface area available for abrasion;

$$F_j' = \frac{F_j \big/ \sqrt{\bar{D}_j}}{\sum F_j \big/ \sqrt{\bar{D}_j}} \qquad (6)$$

The Exner equation for the total gravel load (bedload) for the reach downstream of the dam is

$$(1 - \lambda_p) f_g B \frac{\partial \eta_g}{\partial t} + \frac{\partial Q_g}{\partial x}$$
$$+ \beta Q_g \left(2 + \frac{1}{3\ln(2)} \frac{p_1 + F_1'}{\psi_2 - \psi_1}\right) = q_{gl} \qquad (7)$$

The Exner equation for gravel (bedload) of an individual grain size range (the jth size range) for the reach downstream of the dam is

$$(1 - \lambda_p) f_g B \left(\frac{\partial (L_a F_j)}{\partial t} + f_{Ij} \frac{(\eta_g - L_a)}{\partial t}\right) + \frac{\partial (Q_g p_j)}{\partial x}$$

$$+ \beta Q_g \left(p_j + F_j'\right) + \frac{\beta Q_g}{3\ln(2)}$$
$$\left(\frac{p_j + F_j'}{\psi_{j+1} - \psi_j} - \frac{p_{j+1} + F_{j+1}'}{\psi_{j+2} - \psi_{j+1}}\right) = q_{glj} \qquad (8)$$

The Exner equation for sand for the reach downstream of the dam is

$$(1 - \lambda_p) B \left(\frac{\partial \eta_s}{\partial t} + f_s \frac{\partial \eta_g}{\partial t}\right) + \frac{\partial Q_s}{\partial x} - \frac{\beta Q_g}{3\ln(2)} \frac{p_1 + F_1'}{\psi_2 - \psi_1} = q_{sl} \qquad (9)$$

In the above relations λ_p denotes the porosity of the deposit; t denotes time; Q_g denotes volumetric transport rate of gravel (bedload); x denotes downstream distance; β denotes volumetric abrasion coefficient of gravel (bedload); q_{gl} denotes lateral gravel (bedload) supply rate per unit distance (i.e., volume of bedload supplied to the river per unit time per unit distance from tributaries and bank erosion); L_a denotes the active layer (surface layer) thickness, which is assumed to be a constant value of 0.5 m for simplicity and is discussed in Run 2 of the sample runs in the companion paper, Cui *et al.* (2006); q_{glj} denotes lateral gravel (bedload) supply rate per unit distance in the jth size range; Q_s denotes volumetric transport rate of sand; and q_{sl} denotes the lateral sand supply rate per unit distance. In addition, f_{Ij} denotes the fraction in the jth size range of the gravel that is exchanged between the channel bed and bedload as the channel aggrades or degrades. A relation for f_{Ij} is provided below.

The derivation of Eqs (7)–(9) and an explanation of the terms in the equations are not presented in this paper. Interested readers should be able to derive those equations in reference to similar equations in Parker (1991a,b) and Cui and Parker (1998).

It should be noted that the full set of Eqs (7)–(9) apply to DREAM-2, in which both gravel and sand transport are modeled. In case of modeling with DREAM-1, it is assumed that gravel transport is insignificant compared to sand transport, and thus, Eqs (7) and (8) become irrelevant. Furthermore, Eq. (9) is simplified as

$$(1 - \lambda_p) B \frac{\partial \eta_s}{\partial t} + \frac{\partial Q_s}{\partial x} = q_{sl} \qquad (10)$$

Considerations are taken in the Exner equation of sediment continuity in the reach upstream of the dam to allow for bank erosion during the period of downcutting. A proper mechanism for bank erosion is especially important in case the reservoir deposit is wide and deep. As shown in Fig. 5, the sediment is assumed to have deposited across the valley and the assumed active channel in the model takes a trapezoidal shape with bank slopes at the angle of repose. In aggradational cases, the active channel is assumed to aggrade only on the channel bed and to preserve the two banks. As a result, the trapezoidal channel will increase its bottom width B_b as channel aggrades (Fig. 6a). In degradational cases, the active channel will be allowed to degrade only on the channel bed and to preserve the two banks before the bottom of the trapezoidal channel reaches a predefined minimum width B_m (Fig. 6b). Note that the bottom width of the trapezoidal channel B_b decreases with the degradation of the active channel. Once the bottom width of the trapezoidal channel B_b reaches the predefined
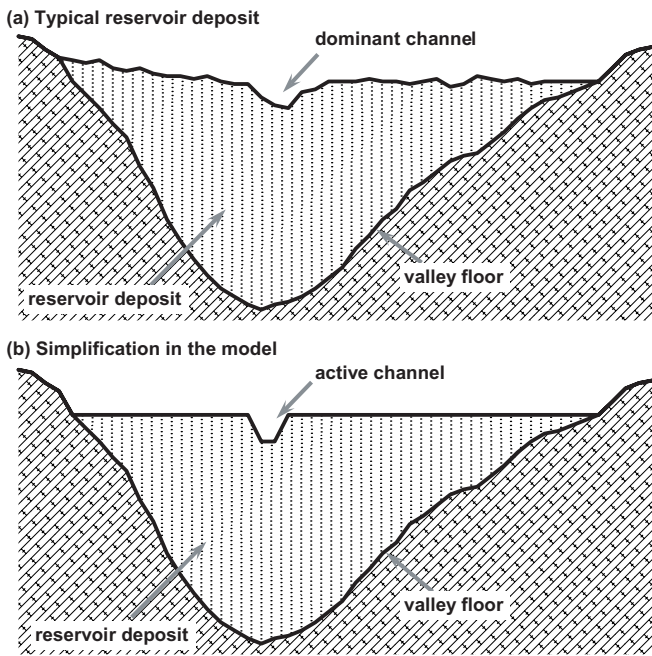
Figure 5 Sketch of a typical reservoir deposit and the simplification of the active channel following dam removal. (a) Typical reservoir deposit, showing poorly-defined channels with a dominant channel; (b) the simplification in the model, representing a single active channel.
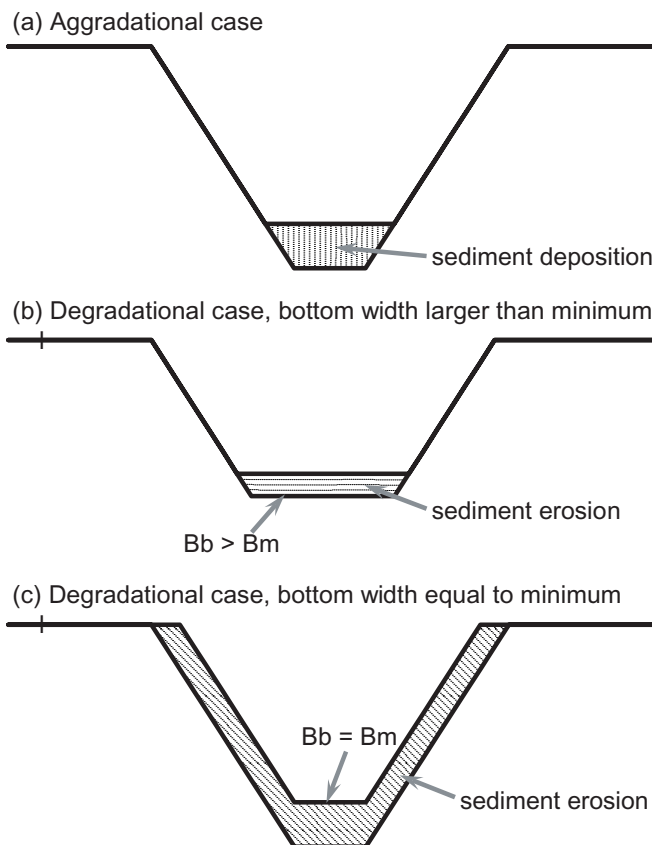

Figure 6 Sketch demonstrating the treatment of aggradation and degradation in the reservoir deposit.

minimum width B_m, the active channel will erode the channel bed and both banks, preserving the bottom width of the trapezoidal channel at the predefined minimum value (Fig. 6c). The above rules are enforced because they offer a very simple treatment for

bank erosion in a one-dimensional model in which the detailed evolution of channel cross-sections cannot be modeled.

The value for B_m is chosen by assuming similarities of active channels in the reaches upstream and downstream of the dam. This was done by first calculating the average bankfull depth and bankfull width in the vicinity and downstream of the dam, and assuming that the channel downstream of the dam can be approximated with the same trapezoidal channel as the active channel upstream of the dam. The bottom width of this channel downstream of the dam serves as the minimum value for the bottom width of the trapezoidal channel upstream of the dam, i.e.,

$$B_m = \bar{B} - \frac{2\bar{H}_b}{\tan(\theta)} \tag{11}$$

in which $\bar{B}$ denotes the average bankfull width in the vicinity and downstream of the dam; $\bar{H}_b$ denotes the average bankfull depth in the vicinity and downstream of the dam; and θ denotes the angle of the banks of the trapezoidal channel, which is assumed to be the angle of repose, or approximately 35°.

In aggradational cases or in degradational cases in which the bottom width of the trapezoidal channel B_b is larger than the predefined minimum width B_m, the Exner equation of sediment continuity takes the same form as in the reach downstream of the dam as given in Eqs (7)–(9) for DREAM-2, or Eq (10) for DREAM-1, except that bankfull width B is replaced with the bottom width of the trapezoidal channel B_b. In degradational cases where the bottom width of the trapezoidal channel B_b is the same as the minimum width B_m, the Exner equation of sediment continuity takes the same form as in the reach downstream of the dam as given in Eqs (7)–(9) for DREAM-2, or Eq. (10) for DREAM-1, except that the bankfull width B is replaced with the top width of the trapezoidal channel B_t, where

$$B_t = B_b + \frac{2H_d}{\tan(\theta)} = B_m + \frac{2H_d}{\tan(\theta)} \tag{12}$$

in which H_d denotes the depth of the trapezoidal channel, i.e., the elevation difference between the surface of the reservoir deposit and the bottom of the trapezoidal channel. Hence, degradation at minimum bed width strips a layer of constant thickness off the channel bed and banks. This allows the active channel to preserve its shape. As pointed out earlier, the geometry of the active channel in the reservoir deposit following dam removal is not known. The above assumption should be taken as a first approximation and can be modified later once field data are available.

In order to calculate sediment transport capacity, sediment is classified as (a) gravel (or coarser), which moves as bedload in DREAM-2; (b) sand, which moves as undifferentiated bed material load in DREAM-1, and bedload or suspended load in the sand range (62.5 μm to 2 mm) in DREAM-2; and (c) silt, which is treated as wash load in both models. The implementation of DREAM-2 requires the calculation of the transport capacity of both gravel and sand. Here we assume that gravel and sand transport by different processes (bedload versus suspended load) and in different time scales (years versus days), and their transport and deposition are only weakly correlated. This assumption allows us

to use their respective transport equations to evaluate gravel and sand transport capacities in the model. The recent development in unified gravel/sand transport equations (e.g., Wilcock 1997, 1998; Wilcock and Crowe, 2003) can be implemented into the model in the future.

The sediment transport equation employed for calculation of sand transport capacity in DREAM- 1 and DREAM-2 is the bed material equation of Brownlie (1982), which was empirically derived from a very large database of flume experiments and field measurements. The application of Brownlie's bed material equation (Brownlie, 1982) requires the coupling of the sediment transport equation with a friction formulation. Brownlie (1982) classified the flow into lower and upper flow regimes, and friction formulations were given for each regime. The lower regime corresponds to a state with ripple and dune bedforms and upper regime corresponds to the plane bed or antidune state. An upper limit for lower regime and a lower limit for upper regime are provided in Brownlie (1982), and the actual transition between the two regimes is dependent on whether the flow stage is rising or falling. Because the model applies the daily average discharge record as model input, the specifics about whether the flow is rising or falling are not included in the use of the Brownlie's formulation, and as a result, the average of the upper limit of the lower regime and the lower limit of the upper regime is used to define the transition between the two regimes in the model. In addition, in applying the bed material equation of Brownlie (1982) and its associated friction formulations, the median grain size is replaced with geometric mean grain size because (a) geometric mean grain size is usually very close to the median size; (b) geometric mean grain size is usually more representative of the characteristics of a grain size distribution; and (c) geometric mean grain size is relatively easy to calculate because it eliminates the interpolation process in a median grain size calculation. Versions of the Brownlie equations that are slightly modified in this way have been presented in Cui and Wilcox (2006) and are not described here.

Brownlie's bed material load equation calculates the transport capacity of bed material, which is usually composed primarily of sand and can be transported either as bedload or suspended load. Finer particles such as silt and clay are considered as wash load, which can be supplied from upstream and tributaries, and is assumed not to be deposited onto the channel bed. The wash load deposited in the reservoir during the period of dam operation is assumed to be entrained into the water column and transported downstream without re-deposition once it is exposed to the flow. Given the importance of distinguishing the suspended load from the total load in order to assess potential biologically significant sediment concentrations in suspension, the portion of the bed material load that is in suspension is calculated using the following criterion (e.g., van Rijn, 1984):

$$\frac{v_s}{\kappa u_*} < 1 \tag{13}$$

where v_s is particle settling velocity calculated with the procedure given by Dietrich (1982); κ is the von Karman constant with a value of 0.4, and u_* denotes shear velocity.

The sediment transport equation employed for calculation of gravel transport capacity in DREAM-2 is the surface-based bedload equation of Parker (1990a,b). Parker's bedload equation (Parker, 1990a,b) calculates bedload transport rate and grain size distribution based on the local surface grain size distribution and shear stress. Details of the surface-based bedload equation of Parker (1990a,b) are not presented here; interested readers are referred to the original publications (e.g., Parker, 1990a,b).

Parker (1990a,b) suggested that a Keulegan type of resistance relation be used in conjunction to his surface-based bedload equation, by assuming the roughness height as twice the surface grain size D_{90}. Cui et al. (1996) slightly modified the resistance relation suggested by Parker (1990a,b) by replacing the roughness height k_s with

$$k_s = 2D_{sg}\sigma_{sg}^{1.28} \tag{14}$$

in which D_{sg} and σ_{sg} denote the surface geometric mean and geometric standard deviation, respectively. It should be noted that sand is excluded from the surface grain size distribution in calculating D_{sg} and σ_{sg} values. The slightly modified resistance relation (Cui et al., 1996) has been employed in the models of Cui and Parker (1997, 1998, 2005), Cui et al. (2003b), and Cui and Wilcox (2006).

It should be noted that there are two sets of resistance relations in DREAM-2; the modified resistance relation of Brownlie (1982) in calculating sand transport capacity and the Keulegan type relation in calculating gravel transport capacity. Realizing that the channel bed is primarily gravel-bedded in a DREAM-2 simulation, the Keulegan type of resistance relation is used for simulation of flow at all times in implementing DREAM-2.

Application of sediment continuity equation in DREAM-2 also needs a relation to link the grain size distributions in bedload and the channel bed (e.g., surface layer and substrate). The relations applied in DREAM-2 is the same as that in Cui and Parker (2005) and in the Marmot Dam removal model (Cui and Wilcox, 2006):

$$f_{1j} = \begin{cases} f_{bj}, & \partial\eta_g/\partial t < 0 \text{ (bed degradation)} \\ 0.3F_j + 07p_j, & \partial\eta_g/\partial t > 0 \text{ (bed degradation)} \end{cases} \tag{15a,b}$$

where f_{bj} denotes fraction of the subsurface deposit in the jth size range. Equation (15a) represents the assumption of Parker (1990a,b) that flow mines the subsurface material during degradational cases, and Eq. (15b) is a relation the form of which was proposed by Hoey and Ferguson (1994), and the coefficients of which were evaluated by Toro-Escobar et al. (1996) from a set of large-scale laboratory experiment.

DREAM-2 can describe the transport of gravel as throughput load over bedrock. In this case the gravel transport capacity over bedrock is calculated with the surface-based bedload equation of Parker (1990a,b) and with a surrogate surface grain size distribution borrowed from the nearest neighboring node. Both DREAM-1 and DREAM-2 can describe the transport of sand as suspended throughput load over a gravel bed or bedrock. In order to do this, the Brownlie (1982) relation is used to compute sand transport capacity over the existing bed. If the sand transport capacity is more than the local transport of sand, the sand

is moved downstream as throughput load. The sandy throughput load is similar to wash load, with the exception that it may be deposited in the interstices of an aggrading gravel deposit in a DREAM-2 simulation. This loss is described in Eq. (9) by means of the term f_s, which is set equal to 0.35 when gravel is aggrading the bed. That is, it is assumed that 35% of gravel deposit produced by aggradation consists of sand deposited in the interstices of the gravel. It is possible that the entire sand load is consumed in this way, so that the throughput load drops to zero downstream of a point. In the event that the model predicts such a condition, f_s is set equal to zero downstream of the point in question.

3 Boundary conditions

3.1 Discharge

The two models apply daily average discharge, which can usually be downloaded from USGS web pages or other data sources when data from a gauging station is available. In order to account for contributions from tributaries, the river can be divided into as many reaches as necessary, and individual discharge records can be applied to different reaches. The results of hydrologic modeling may be employed in the event that field measurements of discharge are insufficient.

3.2 Sediment supply

Long-term average sediment supply rates from the upstream end of the study reach and tributaries are required as model input. Sediment supply from bank erosion downstream of the dam is not specifically built into the current model, although it can be treated as the term q_{sl} in Eqs (7)–(9) by modifying the input module of the model. Significant bank erosion can also be accounted for in the current models by treating the location of bank erosion as a tributary. The sediment supply rate at any given time is distributed using the following assumptions as a first order approximation;

$$Q_{g0} = \alpha_0 Q_w^{2.5}, \quad Q_{s0} = \alpha_1 Q_w^{1.5}, \quad Q_{wash0} = \alpha_2 Q_w^{1.1} \quad \text{(16a,b,c)}$$

in which Q_{g0}, Q_{s0} and Q_{wash0} denote the transport supply rates for gravel, sand, and silt, respectively; and Q_w denotes water discharge that carries the sediment supply, e.g., discharge at the upstream end of the modeled reach, or from tributaries. That is, the gravel, sand, and wash load supplies are assumed to be proportional to discharge to 2.5, 1.5, and 1.1 powers, respectively, reflecting a relatively stronger non-linear relationship between sediment supply and discharge for coarser sediment. It needs to be stressed that the powers of 2.5, 1.5, and 1.1 are hypothetical, and modeler should find better relations for the case simulated, if field data are available. The coefficients α_0, α_1 and α_2 are calculated from the measured or inferred long-term average sediment supply and the discharge record as follows:

$$\alpha_0 = \frac{\bar{Q}_g}{\text{Average}\left(Q_w^{2.5}\right)}, \quad \alpha_1 = \frac{\bar{Q}_s}{\text{Average}\left(Q_w^{1.5}\right)},$$

$$\alpha_2 = \frac{\bar{Q}_{wash}}{\text{Average}\left(Q_w^{1.1}\right)} \quad \text{(17a,b)}$$

in which $\bar{Q}_g$, $\bar{Q}_s$ and $\bar{Q}_{wash}$ are long-term gravel, sand and wash load supply rates from the upstream end or tributaries.

3.3 Downstream end

Downstream end boundary conditions include bed elevation and water depth. In this model, the bed elevation at the downstream end is assumed to be constant throughout the run, i.e., the channel bed does not aggrade or degrade at the downstream end node. Water depth is calculated by assuming a normal flow condition at the downstream end, i.e., by combining Eq. (1b) with the appropriate friction formulations.

4 Staged removal and dredging operation

The models allow the implementation of staged dam removal and partial dredging as options. In a staged removal the dam is removed in sections, starting with the top. In some cases, a notch is placed at the bottom of the removed section to drain water, and possibly sediment, from the reservoir. During staged removal, the models assume that flow control structures will not be installed on the notch, and thus free surface flow will continue throughout the removal process. Staged removal is incorporated into the models by setting the base elevation (η_b) and channel width (B) at the dam site to the crest elevation of the remaining portion of the dam and the width of the notch, respectively, during each removal stage. In case of partial dredging, some of the sediment in the reservoir deposit is mechanically excavated before the dam is removed. Dredging is incorporated into the models by reducing the post-dredging elevation at each node to a specified value prior to dam removal. Dredging is assumed to be implemented to a cross section that is the same or wider than the assumed trapezoidal channel in case of natural erosion. It is assumed that dredging operation will always remove sediment only to the specified depth of dredging, without mixing the remaining sediment below.

5 Initial condition and zeroing process

The initial condition of the model simulation is a specified longitudinal profile of the river, including the base elevation (elevation to the top of the bedrock) and thickness of sand and/or gravel deposit. This initial longitudinal profile is adjusted by the zeroing process described below. The model also requires the sediment composition (i.e., fractions of gravel, sand and silt) in the reservoir deposit and downstream at different locations and depths as input.

A zeroing process should be applied in long-term, large-scale sediment transport simulations. The purpose of the zeroing process is to generate a starting point for the intended simulation and to evaluate certain input parameters. This process recognizes the imperfection of the numerical model as well as the database used to run the model. In the zeroing process, the model is run repeatedly under an appropriately chosen reference condition. If the model is fed with raw data without modification
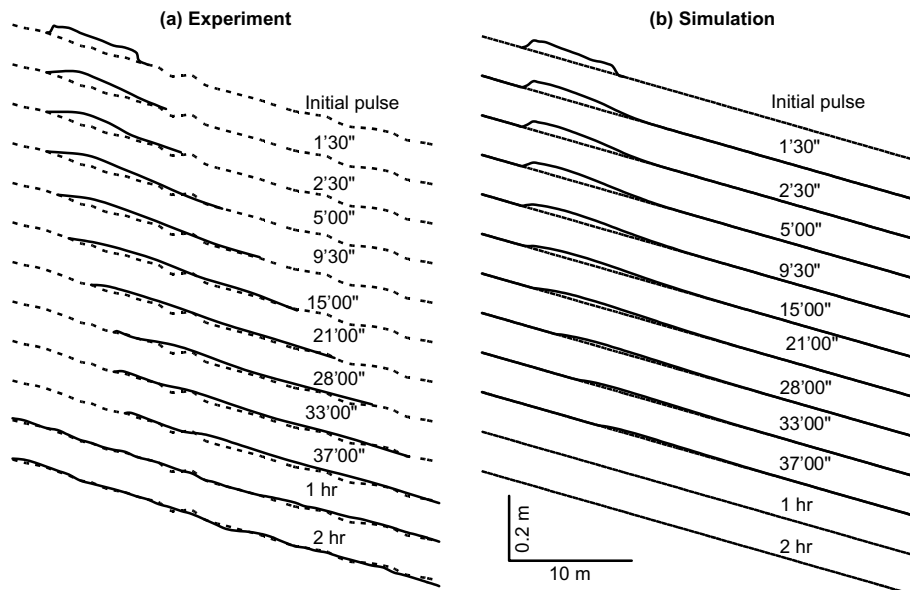
Figure 7   SAFL sediment pulse experiment Run 4b. (a) Experimental data (Lisle *et al.*, 2001, Cui *et al.*, 2003a); (b) simulation with the simplified DREAM-1 and a fudge factor of 2.2.

(i.e., a zero process is not applied), it typically will not produce "quasi-equilibrium" results at a pre-dam release reference condition due to the simplifications and imperfections in the model, the simplified data collection, and the dynamic nature of fluvial processes. Here "quasi-equilibrium" is used in a loose sense, and implies that the model predicts the slow, regular morphodynamic change in a river that can be expected in the absence of the dam and other major disturbances. A numerical model applied without zeroing to raw data often indicates zones of large and spurious aggradation and degradation that are neither realistic nor observed.

In the process of zeroing the model is repeatedly run at the reference state, modifying certain input parameters as required until the model produces "quasi-equilibrium" results, whereby the river experiences aggradation and degradation at different reaches over different periods of time and hydrological events, but overall, long-term aggradation or degradation is limited within the expected or observed range. The "quasi-equilibrium" longitudinal profile downstream of the dam established during the zeroing process provides the initial condition for the intended sediment transport simulation such as one following a dam removal. The aggradational and degradational patterns of the zeroing process provide a base level condition to which the intended sediment transport simulation can be compared. Examples of the zeroing processes can be found in Cui and Wilcox (2006) and in the companion paper, Cui *et al.* (2006). A zeroing process that requires more than minor adjustments to the longitudinal profile of the stream should be interpreted as evidence that the model is inapplicable to the case at hand or the input data are in error.

In the present work the zeroing process is applied to the reach downstream of the dam, and the reference state is based on the longitudinal profile of the river in that reach just prior to dam removal. Sediment supply is introduced into this reach as if there is no dam, and the longitudinal profile and other parameters are

modestly adjusted until the reach shows an acceptably low level of aggradation/degradation throughout. Ideally the reference state should be based on the longitudinal profile of the river before the dam was installed, but such information is often not available.

6   Model validation

Because there are no data available for model validation from actual dam removal projects, we used the St. Anthony Falls Laboratory (SAFL) sediment pulse experiment Run 4b (Lisle *et al.*, 2001; Cui *et al.*, 2003a) for validation of sand transport in DREAM-1 and DREAM-2, and the documentation of a natural landslide (Hansler, 1999; Lisle *et al.*, 2001; Sutherland *et al.*, 2002) for validation of bedload transport in DREAM-2. In addition, we simulated the Lake Mills drawdown experiment on the Elwha River (Childers *et al.*, 2000) to validate the reservoir erosion module. The validations are discussed below.

6.1   SAFL sediment pulse experiments Run 4b

The SAFL sediment pulse experimental runs (Cui *et al.*, 2003a) were conducted in a 0.5 m wide and approximately 45 m long flume. Before the experiments, an initial channel slope was established by continuously feeding water and sediment at constant rates of 9 l/s and 45 g/min, respectively. The sediment used to set up the initial slope was a mixture of pea gravel and sand with a median grain size of about 2 mm. The resulting channel bed was slightly armored with an equilibrium slope of about 0.0108. Upon reaching the equilibrium slope, a pulse of sediment was introduced in a section of the flume as shown in Fig. 7(a). The water discharge and sediment feed rate were then reestablished to their pre-pulse levels. To describe the evolution of the pulse, bed elevations were measured for the duration of the experiment. Of all the SAFL sediment pulse experimental runs, Runs 1, 2, 3,

4a, and 4b, only Runs 4a and 4b introduced a fine sediment pulse which can be viewed as the simulation of sand transport over a gravel-bedded river. Between Runs 4a and 4b, Run 4a was a trial run without intensive measurements. The fine sediment (sand) pulse introduced in Runs 4a and 4b had a geometric mean grain size of approximately 0.55 mm and geometric standard deviation of about 2.31. The experimental results for Run 4b are shown in Fig. 7(a).

DREAM-1 was developed to simulate dam removal at field scale, and the current model structure do not allow for simulation of flume experiments. For example, the output of the model is given in terms of daily, weekly and monthly results and cannot provide the fine time scales appropriate for a flume experiment. We therefore developed a simplified flume version of DREAM-1 to simulate the SAFL sediment pulse experiment Run 4b. In simplifying DREAM-1, a "fudge factor" was added into the model to allow the user to adjust the predicted sediment transport rates. For example, a "fudge factor" of 1 means that there is no adjustment to the sediment transport rate predicted with Brownlie's bed material load equation and a factor of 2 means that the sediment transport capacity used in the model is twice that predicted by Brownlie's equation.

The numerical experiments indicated that the simulation under-predicted the sand transport rate, evidenced by a slower pulse evolution in the numerical simulation. Increasing the calculated sediment transport rate by a "fudge factor" of 2.2, however, reproduced the experimental results satisfactorily via a visual inspection, as shown in Fig. 7(b). Comparison of Figs 7(a, b) indicates that the adjusted model provided a very accurate reproduction of the experiment results, including such features as the dispersion and downstream translation of the sediment pulse, the locations of the leading and trailing edges of the sediment pulse, and the time at which the sediment pulse became so diffuse that it was difficult to distinguish from the ambient sediment. Note in Fig. 7(b) that the initial bed profile has been smoothed before applying the model.

Wooster (2002) applied the simplified DREAM-1 to simulate his dam removal experiments and found that a fudge factor of 3.4 produced an excellent match between the simulation and experimental data.

6.2 Lake Mills drawdown experiment

DREAM-1 was also applied to simulate the Lake Mills drawdown experiment (Childers *et al.*, 2000) in order to validate its reservoir erosion module. Lake Mills, shown in Fig. 8, is the reservoir behind the Glines Canyon dam on the Elwha River under study for removal (e.g., Bureau of Reclamation, 1996a,b). The Lake Mills drawdown experiment (Childers *et al.*, 2000) was conducted between April 8 and 26, 1994, when the lake level was gradually lowered from 179.2 to 173.7 m over a 1-week period, with a lowering rate ranging between 0.3 and 0.9 m/day. The lake was then held at 173.7 m for a week and then gradually filled back to the predrawdown level of 179.2 m (Childers *et al.*, 2000; T. Randle, personal communication). The time variation
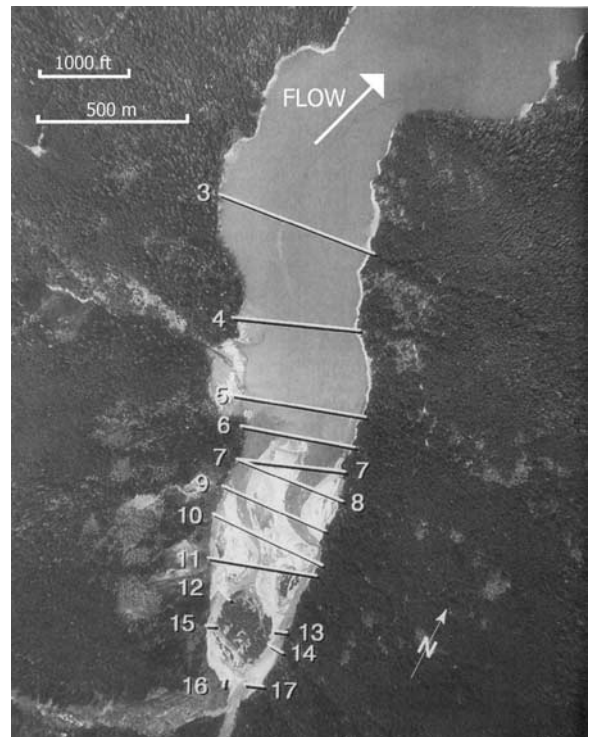

Figure 8 Lake Mills, Elwha river showing the monitoring cross sections. Modified from Childers *et al.* (2000).
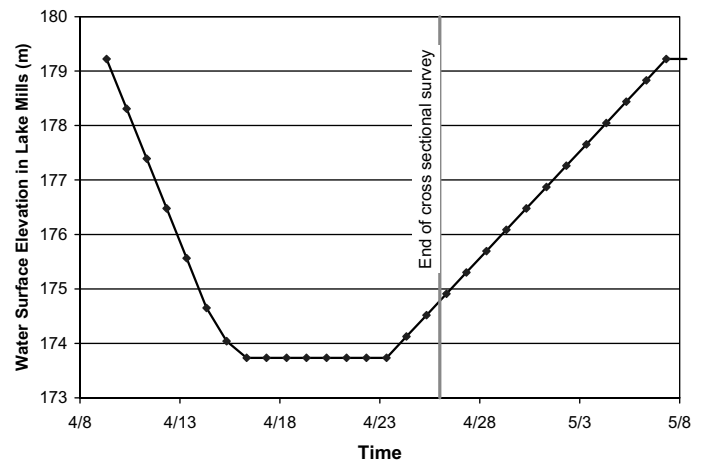

Figure 9 Lake Mills water surface elevation during the drawdown experiment; data based on Childers *et al.* (2000) and T. Randle (personal communication).

of lake level in Lake Mills during the drawdown experiment is given in Fig. 9.

Note that the Lake Mills drawdown experiment differed from our assumed dam removal scenario in that the lake drawdown experiment slowly lowered the lake level as described above and in Childers *et al.* (2000), while the base level for our assumed dam removal scenario would be lowered instantly. The above difference may result in differences in channel erosion patterns and other channel morphology. Despite the differences, the drawdown experiment offered an opportunity to see how the model results and field measurements compare at a scale much larger than a flume.

The current model (DREAM-1) was modified slightly to allow for the gradual decrease in lake level during the drawdown experiment. The input parameters for the simulation are summarized below.

6.2.1 Discharge

The numerical simulation assumes that only a single channel exists, and thus the total discharge measured at the Elwha River above Lake Mills near Port Angeles, WA (USGS station no. 12044900), located approximately 370 m upstream of the study area shown in Fig. 8 is used for the simulation. The daily average discharge record at the station during the drawdown period is shown in Fig. 10.

6.2.2 Grain size distribution

The experimental drawdown resulted in sediment erosion of only the Lake Mills delta. With this in mind, the average grain size distribution of the delta, as shown in Fig. 11, is used to represent the grain size distribution of all the sediment in the simulation.
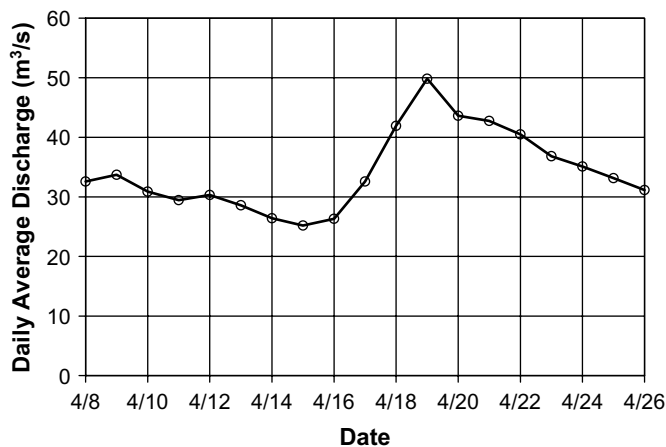
Figure 10 Daily average discharge at Elwha River above Lake Mills near Port Angeles, WA (USGS station no. 12044900) during the lake drawdown experiment.
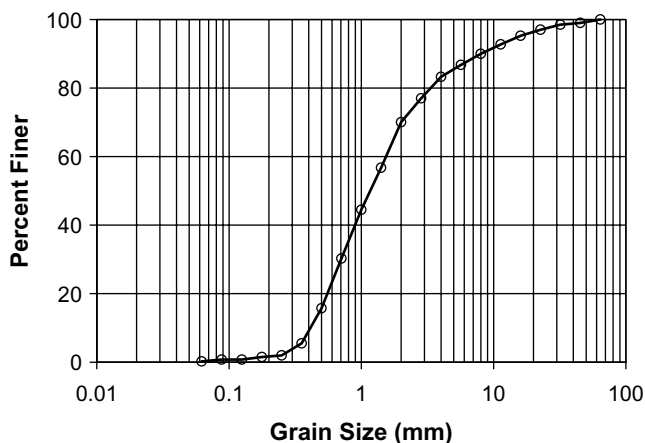
Figure 11 Average grain size distribution of the Lake Mills delta deposit (data source: Childers *et al.*, 2000).

The geometric mean grain size and geometric standard deviation of the delta sediment are approximately 1.44 mm and 3.07, respectively.

6.2.3 Sediment supply

The drawdown experiment was performed during a relatively low flow period, and thus it is likely that upstream sediment supply was minimal. In addition, the amount of reservoir erosion was very large, making any additional upstream sediment supply insignificant during the 3-week period. With that in mind, the upstream sediment supply is set to zero in the modeling exercise.

6.2.4 Active channel geometry

In DREAM-1, the active channel in the reach upstream of the dam is assumed to be trapezoidal. There are four parameters that define the initial active channel before dam removal: bed elevation, channel width at the bottom of the trapezoidal channel, bankfull depth, and bank slope. In addition, the width of sediment deposit in the reservoir limits how wide the active channel will develop once the active channel incises into the deposit. The widths of the sediment deposit in the reservoir were read from the aerial photograph in Fig. 8. The accuracy of the width of the reservoir deposit is not relevant to this modeling exercise because the river did not erode all the way to the valley walls. The bank slope of the active channel following the reservoir drawdown uses the default value in DREAM-1, i.e., the bank is assumed to be at the angle of repose, or 35°. Based on the field data of Childers *et al.* (2000), the initial channel in the reservoir reach is assumed to be 1 m deep and 97 m wide at the bottom, or 100 m wide at bankfull flow.

The observed channel evolution at cross section 8 in the deposit of Lake Mills is shown in Fig. 12. Note that two channels have formed, rather than the one channel assumed in the model. This issue is discussed below. Due to the difficulties in interpreting average bed elevations and channel widths from the field data as evidenced in Fig. 12, and because the most important function of the reservoir erosion module is to predict the amount of sediment erosion, only the simulated and observed amount of erosion are compared, as shown in Fig. 13.

Figure 13 shows that the simulation predicted the time variation of total sediment erosion relatively accurately. Further examination of the field data, however, indicates that reservoir erosion was primarily restricted to the reach between cross sections 7 and 11 of Fig. 8, with very limited erosion upstream of cross section 11. Compared to the field data, the simulation underpredicted the amount of erosion between cross sections 7 and 11 by approximately a factor of 2. The underprediction of sediment erosion between cross sections 7 and 11 is very likely caused by the extensive lateral erosion developed in the field. As a result, the combined channel width in the field is much larger than that predicted in the simulation. For example, the bankfull channel width at the end of the drawdown experiment increased to only about 107 m from the original 100 m in the simulation for cross section 8, as shown in
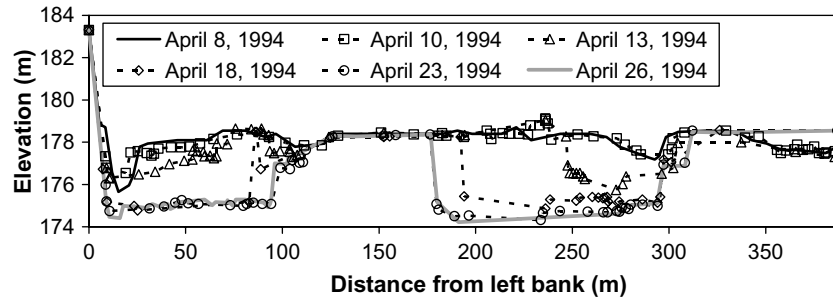
Figure 12   The evolution of cross section 8 surveyed during the Lake Mills drawdown experiment.
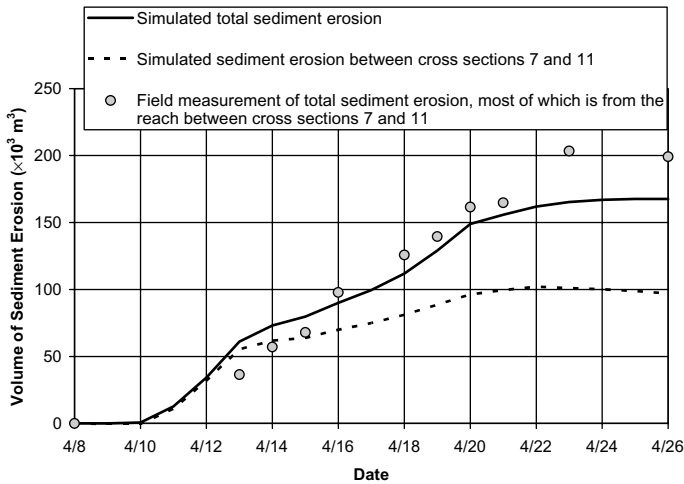

Figure 13   Measured and simulated reservoir erosion during the Lake Mills drawdown experiment.

Fig. 14. Field data, however, indicate that channel width of cross section 8 increased to about 210 m at the end of the drawdown experiment, as shown in Fig. 12. It is very likely that the large amount of lateral erosion in the drawdown experiment prevented erosion upstream of cross section 11.

As discussed earlier, our assumed dam removal scenarios differ from Lake Mills drawdown experiment in that Lake Mills level was lowered gradually with a maximum lowering rate of 0.9 m/day, while our assumed dam removal scenario lowers the base level instantly, which is very likely to lead to development of a narrower channel without extensive lateral erosion as shown in Fig. 14. Having said that, however, much more research and field observations are needed in order to better understand the reservoir erosion processes, which would in turn be used to improve the reservoir erosion module of the current models.
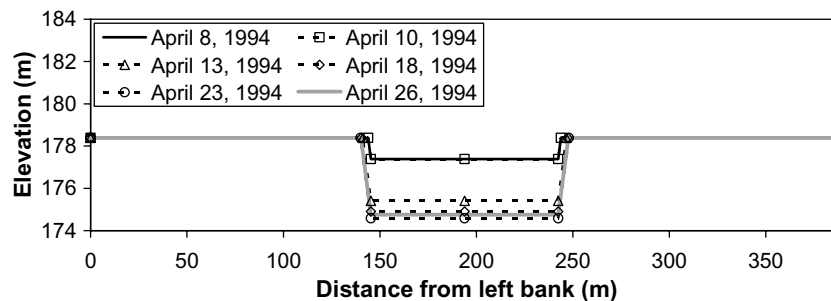

Figure 14   The evolution of cross section 8 simulated with the reservoir erosion module of DREAM-1.
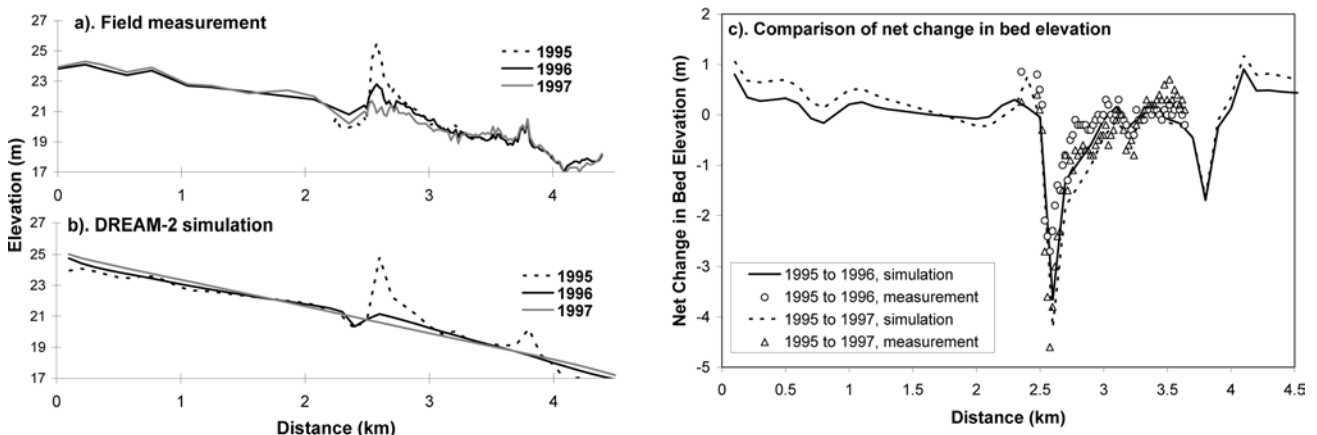

Figure 15   Landslide in the Navarro River, California. (a) field data documented by Hansler (1999), Lisle *et al.* (2001) and Sutherland *et al.* (2002); (b) simulated longitudinal profile with DREAM-2; (c) comparison of net change in bed elevation, i.e., deviation of bed elevation from the initial (immediately post-slide) values, between field measurements and DREAM-2 simulation.

6.3 Navarro River 1995 landslide

The natural landslide used for DREAM-2 validation was that of the Navarro River, California, which occurred in March 1995 and was documented by Hansler (1999), Lisle *et al.* (2001), and Sutherland *et al.* (2002). The landslide delivered approximately 60,000–80,000 m³ sediment to the channel from the adjacent hillslope. The evolution process of the landslide was documented in summer of 1995 and two subsequent summers (Hansler, 1999; Lisle *et al.*, 2001; Sutherland *et al.*, 2002). Simulation of the landslide with an early version of the Cui and Parker (2005) model by Hansler (1999) (also reported in Lisle *et al.*, 2001) produced good agreement. Here the input data are fed into the DREAM-2 without any adjustment to the model. The comparison of measured and simulated net changes in bed elevation is shown in Fig. 15, indicating good agreement. In particular, the simulation reproduced (1) the slight aggradation upstream of the landslide between 1996 and 1997; (2) the degradation of the landslide over the duration of observation; and (3) the almost indistinguishable change in bed elevation farther downstream.

Two features not reproduced by the model are: (1) the aggradation of fine sediment immediately upstream of the landslide between 1995 and 1996 and its subsequent erosion between 1996 and 1997; and (2) a hard point at roughly 3.8 km that was not eroded in the field. The model did not reproduce the two features because (1) deposition of fine sediment at upstream of the landslide is not modeled; and (2) the model was one-dimensional and may not be able to reproduce some of the local features observed in the field. Even with the two features not predicted by the model, the overall agreement between model prediction and field data is good, as evidenced in the comparison of bed aggradation in Fig. 15(c).

7   Discussions on the limitations of the models

The major advantage of the DREAM presented in this and the companion paper, Cui *et al.* (2006), is the simplified effort in collecting field data for model input, which normally constitutes the majority of a dam removal study. One of the simplifications is the treatment of channel cross sections, which are assumed to be rectangles with widths equal to bankfull channel widths. Because most of the sediment is transported during flow events at and near bankfull, this simplification adequately represents the cumulative sediment transport, erosion, and deposition processes. The simplification, however, does not adequately represent water depth along the river for most of the flow conditions as it will underpredict water depth for low flow events and over-predict water depth for over bank flow events. Due to this shortcoming, the model results should not be used independently for evaluation of flooding issues following dam removal. Instead model results can be combined into detailed cross sections to serve as input to a flow model such as HEC-RAS for more precise water surface predictions.

The simplification of cross sections upstream of the dam to a trapezoidal channel may also affect the accuracy of short-term predictions. In particular, the models assume that, regardless of flow, the upstream cross sections have similar dimensions with that immediately downstream of the dam. In short-term, the channel eroded in the reservoir would be narrower in the absence of a large flow event following the dam removal. On the long term, however, the assumption that the channel in the reservoir has the similar size with downstream cross sections should be reasonable because the river will usually experience a series of high flow events within no more than a year following dam removal.

It is also important to stress that neither one of the models is designed for dam removal simulation where silt and clay constitute a major portion of the reservoir deposit. Under such circumstances silt and clay will act as cohesive agents to slow down the erosion of reservoir sediment. Despite this limitation, the models may be used for dam removal simulations where silt and clay are the dominant deposit to obtain a worst-case-scenario estimate of certain parameters (e.g., Stillwater Sciences, 2004). Extreme cautions, however, must be taken in similar simulations because the model results will not be accurate and can only be presented as the worst-case-scenario for the given parameter.

8   Conclusions

This paper presents a summary of the DREAM: DREAM-1 for simulation of sediment transport following dam removal behind which the reservoir deposit is composed primarily of non-cohesive sand and silt, and DREAM-2 for simulation of sediment transport following dam removal behind which the upper layer of reservoir deposit is composed primarily of gravel. The models apply the simplified procedure of Cui and Parker (2005) that combines the backwater equation and quasi-normal flow assumptions for flow simulation. In applying the simplified procedure for flow calculation, the channel geometry of the entire calculation domain is assumed to be rectangular with bankfull channel width. For sediment continuity calculations the channel downstream of the dam is assumed to have the same rectangular cross-sectional shape as in the flow calculation, and the channel upstream of the dam is assumed to be trapezoidal, allowing for bank erosion during the period of downcutting. In DREAM-2, gravel and sand transport are integrated into a single model, although the transport capacities of gravel and sand are calculated with their respective equations. The integrated gravel and sand model allows sand generated from abrasion and lost to interstices of gravel deposit to be accounted for.

Comparison of a simplified version of the DREAM-1 model with SAFL sediment pulse experiment Run 4b (Lisle *et al.*, 2001; Cui *et al.*, 2003a) indicates that Brownlie's bed material equation (Brownlie, 1982) underpredicted sediment transport capacity. Increasing the sediment transport capacity predicted by Brownlie's bed material equation (Brownlie, 1982) by a factor of 2.2, however, produces good results. The reservoir erosion module of DREAM-1 is also applied to simulate the Lake Mills drawdown experiment (Childers *et al.*, 2000). Comparison of the simulation with experimental data indicates that the model closely reproduced the total amount of erosion in the reservoir. The simulated pattern of reservoir erosion, however, differs from that in the field. The difference in erosion patterns between the

simulation and the field experiment is, however, very likely caused by the extensive lateral erosion in the field induced by the slowly lowered lake level. DREAM-2 is validated with data for a natural landslide on the Navarro River, California, documented by Hansler (1999), Lisle *et al.* (2001) and Sutherland *et al.* (2002), with good agreement between simulation and field data.

The companion paper, Cui *et al.* (2006) provides a series of sample runs as sensitivity tests pertaining to some of the important user-defined and fixed parameters.

Acknowledgments

Notation

B = Bankfull channel width

$\bar{B}$ = Average bankfull width for the reach close to and downstream of the dam

B_b = Bottom width of the trapezoidal channel

B_m = Minimum bottom width of the trapezoidal channel

B_t = Top width of the trapezoidal channel

D = Particle grain size

$\bar{D}_j$ = Geometric mean grain size of the jth size group

D_{sg} = Geometric mean grain size of surface gravel

f_{bj} = Volumetric fraction of the jth size group in subsurface gravel

f_g = Fraction of gravel in sediment deposit

f_{Ij} = Volumetric fraction of the jth size group in the gravel that is exchanged between bedload and channel in a gravel-bedded river

f_{sa} = Fraction of sand in sediment deposit

F = Froude number

F_j = Volumetric fraction of the jth size group in surface gravel of a gravel-bedded river

F'_j = Fraction of the area for the jth gravel size group exposed to the flow in surface layer

g = Acceleration of gravity

h = Water depth

H_d = Depth of the trapezoidal channel

$\bar{H}_d$ = Average bankfull depth for the reach close to and downstream of the dam

k_s = Roughness height

L_a = Active layer (surface layer) thickness

p_j = Volumetric fraction of the jth gravel size group in bedload of a gravel-bedded river

q_{gl}, q_{sl} = Lateral gravel and sand supply to the channel, in volume per unit channel length per unit time

Q_g = Volumetric transport rate of gravel

$\bar{Q}_g$ = Long-term average volumetric rate of gravel supply

Q_{g0} = Volumetric rate of gravel supply

Q_s = Volumetric transport rate of sand

$\bar{Q}_s$ = Long-term average volumetric rate of sand supply

Q_{s0} = Volumetric rate of sand supply

Q_w = Water discharge

$\bar{Q}_{wash}$ = Long-term average volumetric rate of wash load supply

Q_{wash0} = Volumetric rate of wash load supply

S_0 = Channel bed slope

S_f = Friction slope

t = Time

u_* = Shear velocity

v_s = Sediment particle settling velocity

x = Downstream distance

$\alpha_0, \alpha_1, \alpha_2$ = Coefficients for proportioning sediment supply

β = Volumetric abrasion coefficient

ϕ = Grain size ϕ-scale

η_b = Non-erodible base (bedrock) elevation

η_g = Thickness of gravel deposit

η_s = Thickness of sand deposit

κ = von Karman costant

λ_p = Porosity of the sediment deposit

θ = Bank angle of the trapezoidal channel, which is assumed to be the angle of repose

σ_{sg} = Geometric standard deviation of surface gravel

ψ = Grain size ψ-scale, which is the negative of ψ-scale.

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