Design considerations for the use of vegetative controls for the treatment of highway discharges

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Abstract The design of highway drainage in the UK has traditionally provided for the rapid removal of surface runoff from the carriageway with positive discharges being directly routed either to a roadside soakaway system or to the nearest receiving water course. Such systems pay little attention to the potential pollution loads generated from rainfall-runoff events or their potential deleterious impacts upon receiving waters. Increasing regulatory agency concern over such discharges is leading to a review of such conventional drainage systems and an interest in the potential use of vegetative systems for the control and treatment of highway runoff quality. The paper identifies the range of pollutant removal efficiencies achieved by vegetative best management practice (BMP) controls and reviews available design procedures for grass-lined channels (swales) and constructed wetlands. Recommended design guidance for swales rarely specifically considers water quality criteria and usually results in over-design capacities; "over-the-edge" trough systems are preferred over conventional swale geometries. For wetland design, a multi-cell approach is recommended with sizing providing the critical control parameter in terms of performance efficiency and a quantitative comparison is given of hydraulic and kinetic sizing design approaches.

INTRODUCTION

It is widely recognized that highway runoff contains a range of toxic pollutants that can have detrimental impacts on receiving waters, both ground and surface. Whilst highways may represent only 5–8% of the urban catchment, highway drainage can contribute as much as 50% of Total Suspended Solids (TSS), 16–25% of Total Hydrocarbons (HCs) and between 35–75% of the total metal pollutant input budgets to a receiving water (Ellis & Revitt, 1991; Luker & Montague, 1994). Studies of multi-lane highways (Marsalek, 1998) carrying more than 100 000 vehicles per day have shown that 20–25% of runoff samples can be classified as being severely toxic (based on Microtox and SOS genotoxicity chromotesting) compared to only 1% of normal urban stormwater samples.

Conventional control and treatment of highway runoff is through kerbside inlet/gully pot (or catchpit) drainage to a piped or ditched system which may have oil interceptor and silt trap controls at the outlet prior to eventual discharge from the site. On motorway and trunk road systems extensive use is made of french and/or fin filter drains as well as soakaway systems with excess drainage passing on to a pipe or ditch conveyance system. It is also common practice to have stormwater storage basins (wet or dry) at outlets draining highways carrying more than 30 000 AADT flows. Very little use is currently made of vegetative systems (such as grass swales or wetlands) for management of highway runoff although 10% of major UK highways have central grass-lined (or extruded concrete) channels at 0.8% longitudinal gradients draining to a fin-drain type outfall connection. Informal vergeside drainage is also practised on some 20% of trunk/country roads. Such "over-the-edge" drainage is usually facilitated by "grip" channels cut through the grass verge and filter margins which totally negates the pollution attenuation benefits that would otherwise accrue from direct overland sheet flow and infiltration into the underlying active soil layer.

Studies have consistently shown that existing designs for the control of highway discharges are primarily if not exclusively intended to control runoff volumes and rates providing little if any pollution treatment function (Ellis *et al.*, 1997). However, regulatory pressures on highway authorities and agencies are increasingly requiring that drainage controls should include reference to water quality treatment and the clear inadequacy of conventional drainage systems has stimulated interest in the design, implementation and operation of alternative vegetative systems.

BEST MANAGEMENT PRACTICE FOR RUNOFF CONTROL AND TREATMENT

The variable volumes and flow rates generated by impermeable highway surfaces generally render infeasible any application of the sort of conventional unit processes used for sanitary or industrial effluents. Control and treatment alternatives therefore must resort to much more passive techniques combining retention/detention facilities with biofiltration contact treatment. Such best practice can be achieved through a variety of soft-engineered configurations such as grass channels, filter strips and shallow wetlands.

Roadside and median grass-lined depressions and channels are now commonly used as low-cost practices in North America, Australia, France and Germany to convey impermeable runoff from the carriageway surface although they may have land uptake costs which are difficult to meet in some restricted urban situations. However, swale techniques have been slow to take-off in the UK and many other European countries. In the UK for example, swales only comprise some 15–18% of all BMP source control devices constructed, principally in association with new industrial/commercial estates. On the other hand, the use of constructed wetlands for the treatment of highway runoff has become accepted as a best practice option with design procedures generally adopting wastewater and stormwater wetland formulae (Hall *et al.*, 1994; Cooper *et al.*, 1996; Nuttall, 1997; Lawrence & Breen, 1998).

Tables 1 and 2 indicate the range of pollutant removal efficiencies that have been noted for grass swales and constructed wetlands receiving highway runoff. It is clear that whilst in general, good removal rates can be achieved by such systems, there is still considerable variability in performance. In the case of grass channels and filter strips very little removal is achieved for soluble metal species, nutrients or bacteria and it may be that they can only provide an efficient performance for solids, oils and heavy organics such as leaf litter etc. Solids removal performance increases as flow TSS concentrations increase but with inflow concentrations below 30–40 mg l^{-1}, very little

Table 1 Swale pollutant concentrations, loadings and removal efficiencies.

Pollutant parameter	EMC and range ($mg\ l^{-1}$)	Load ($kg\ ha^{-1} year^{-1}$)	% Removal efficiency
TSS	25.0 (7.0–47.0)	–	86 (55–91)
Total zinc	0.032 (0.011–0.143)	7.05 (1.85–9.2)	83 (63–93)
Total lead	0.079 (0.014–0.144)	0.78 (0.25–2.61)	54 (17–76)
COD	39.0 (2.0–76.0)	–	62 (53–74)

Table 2 Percentage pollutant removal rates in constructed wetlands.

Wetland type	TSS	Faecal coliforms	Total Nitrogen	Total Phosphorus	Lead	Zinc
Subsurface flow	85 (67–97)	88 (80–97)	44 (25–98)	50 (20–97)	83 (5–94)	42 (10–82)
Free surface water flows	73 (13–99)	92 (86–99)	53 (10–99)	43 (7–98)	69 (41–83)	62 (36–75)

reductions can be expected. The variability of constructed wetlands has variously been attributed to short-circuiting, short detention and contact times, pollutant remobilization, seasonal vegetation effects etc. (Strecker *et al.*, 1992). Wetlands intended for highway discharges should have front-end oil interceptors and silt traps with facilities for spillage containment on critical road sections. A pre-treatment forebay (10–12% of the total wetland volume) adjacent to the inlet will provide a further solids settlement facility from which flow can pass on (over a level spreader for surface flows or through slotted inlet pipes in a rip-rap/gabion wall for subsurface flows) into the vegetated wetland.

There is no shortage of design manuals and guidance criteria for either swales or wetlands but almost all are exclusively empirical in character and often actual performance outcomes are site-specific. Most design procedures adopt a hydraulic approach based on traditional wastewater treatment formulae and contain little specific consideration of water quality criteria.

Swale design

The available procedures for trapezoidal swale design (Ellis, 1990; Horner *et al.*, 1994) are essentially based on the Manning open channel flow equation using a design flow rate to determine optimum width, area and length (assuming that maximum flow velocity in the swale channel is kept below 0.3–0.4 $m\ s^{-1}$ with hydraulic residence times being between 5–9 min). An alternative approach has been outlined by Leonard & Sheriff (1992) which is based on a design procedure developed for soakaways (Pratt, 1996). This utilizes soil infiltration rates (varying between 0.01–0.5 $mm\ s^{-1}$) and a depth based on a 24 h storage time for a given storm intensity-frequency (the M10-15 minute event is recommended). The swale length (per hectare of contributory catchment) is then determined from the required storage volume. The design method can also be applied to shallow swales which would contain a maximum of 0.5 m^3 of stormwater per metre length when filled to capacity (by the M10-15 min event). However, none of the methods incorporate specific criteria intended for water quality improvement and all tend to overestimate the required swale length in terms of biofiltration potential. The former two methods do not consider initial rainfall-runoff losses (which can be at least 20% of total rainfall depth) and the last makes no allowance for overland flow time or peak flow attenuation. All three methods possess considerable inherent factors of safety against surface flooding and the resultant designs can often safely accommodate even M50 events. Wang *et al.* (1981) maintain for example, that pollutant concentrations tend to decline exponentially with swale length. Therefore, a substantial fraction of total potential pollutant removal can most probably be accomplished in very much shorter channel lengths than the design value. The work of Walsh *et al.* (1997) confirms that the swale or median length has only a small effect on pollutant removal and that vegetation density, flow rates and type (open channel *vs* sheet flow) as well as size of contributory area may be much more significant controlling factors.

Water quality improvements can be aided by the introduction of a level spreader at the inlet and the use of check dams on long swale lengths or with longitudinal gradients above 3%. The top surface of the vegetation also needs to stand at a height exceeding the design water depth by at least 5 cm and for maximum pollutant removal, the design depth of flow should be at least 400–500 mm less than the winter vegetation height. The height, density and type of vegetation cover can be much more important in terms of pollutant removal than the length of the swale channel. The traditional trapezoidal cross-section advocated in most design methods may also encourage flow sinuosity and channel incision which can yield an additional source of sediment.

Simple, shallow and broad V-shaped grass troughs (5–8 m wide with side slopes of up to 9–12%) may be much more appropriate a shape than the conventional swale geometry. In this form they can be used as "over-the-edge" filter strips with pollutant removal occurring across the entire side slopes of the trough rather than relying on the more restricted surface area offered by the base of the swale channel. Such forms would also be more convenient for routine maintenance. The work of Walsh *et al.* (1997) and Murfee *et al.* (1999) has demonstrated the better removal efficiency and enhanced performance of such trough-like shapes and associated vegetated side slopes over conventional swales and kerb-gutter systems. Schueler (1987) has also noted mediocre to negligible quality performance during rapid runoff or when the grass sward is mown short. Pollutant uptake by plants and denitrification processes both require a hydrology which allows shallow percolation with relatively long residence times. In the particular case of phosphorus, grass filters may themselves act as sediment stores which can exacerbate the wash-off and receiving water problem.

Irrespective of these reservations, the data in Table 3 implies that pollutant loadings in conventional swale channels are generally well below most national criteria for biosolid disposal to land. The table gives "trigger" or threshold loading limits as defined by various national US and European agencies expressed in either annual or total cumulative loadings. The UK ICRCL values are those quoted for parks and open spaces whilst the Dutch values are those defining clearly contaminated land. Even adopting the maximum loading rates shown in Table 1 with the most restrictive criteria of Table 3 would suggest operational site lives of well beyond 50 years especially if given regular and proper maintenance. However, the relatively low loading limits specified for cadmium might provide a more critical restriction.

Table 3 Loading criteria for biosolid disposal to land.

Pollutant parameter	UK ICRCL ($mg\ kg^{-1}$)	Dutch Ministry of Public Housing ($mg\ kg^{-1}$)	New Jersey EPA ($mg\ kg^{-1}$)	US 503 Regulations ($kg\ ha^{-1} year^{-1}$)
Total zinc	300	720	350	140
Total lead	2000	530	250–1000	15
Total cadmium	15	12	3	1.9

Wetland design

There is considerable guidance available on the hydraulics, configuration and planting of constructed wetlands (Hall *et al.*, 1994; Merritt, 1994; Cooper *et al.*; 1996; Nuttall, 1997). The engineering design emphasis is essentially placed on the hydraulic controls of volumes, depths and levels with some consideration of front-end sedimentation facilities. However, the principal problem of wetland design for the treatment of highway (and urban) runoff is that of optimum sizing particularly given the episodic nature and possible superimposition of inflow events. Sizing is crucial in controlling both the hydraulic loading and retention times needed to give maximum contact and biofiltration/uptake opportunities. The sizing of constructed wetlands for runoff treatment can be done in three ways:

- as a percentage of the contributory catchment area or connected impervious area e.g. figures of between 1 to 5% have been commonly suggested;
- hydraulically by determining the amount of storage required (low basin to inflow volume storage ratios will achieve very little attenuation or polishing);
- kinetic sizing criteria using the decay (or assimilative) rates for specific pollutants as a guide to total area requirements. Lawrence & Breen (1998) suggest the use of an alternative biofilm adsorption rate expressed as uptake rate in $g\ m^{-2}\ day^{-1}$ contact time.

The first method is essentially for new, off-line, small-scale constructions. With a 2–3% wetland to drainage area ratio, for a 10 hectare development site and with retention volumes equal to 4–6 times the mean storm runoff volume, a 2 m deep wetland system would require storage of 1000 m^3 and a 2000 m^2 surface area. Such sizing criteria would pose considerable land take difficulties and in any case does not account for any performance considerations. Hydraulic sizing has been generally restricted to extended detention facilities where water quality improvements are required but where predicted performance cannot be quantified although recently Somes *et al.* (1996) have suggested using a probabilistic, time series-based residence time distribution approach to allow for unsteady, intermittent hydraulic loading conditions.

Kinetic sizing design approaches based on first-order reaction rates, have been quite widely applied to determine the size and contact time required to achieve target pollutant reductions in wetlands intended for municipal wastewater treatment (Reed *et al.*, 1993). Time series plots of pollutant decay through wetland systems have consistently shown organic carbon to be assimilated at very low rates and thus the use of BOD or TOC reaction rates for conservative sizing have been considered to be justified. BOD reaction rates for municipal sewage tend to be in the order of 1.1–2.0

(with loading rates of 1–30 g BOD $m^{-2} day^{-1}$). The reaction rates for highway stormwater effluent can be expected to be less given their much lower nutrient and organic mass loading rates. Reaction rates (k) derived from experimental 2 day retention data for a 10 000 m^3, 0.6 m deep, 1.3 ha highway wetland system in Pershore, Worcestershire, were 0.8, 1.05, 3.05 and 4.04 for BOD, Total P, Total N and Total Coliforms respectively.

An empirical design equation for kinetic sizing has been suggested by Reed *et al.* (1993):

$$As = (L \cdot W) = Q[\ln(C_0 / C_t)] / k \cdot D \cdot \rho$$

where As is surface area (m^2), L is length (m), W is width (m), D is depth (m) and ρ is porosity (in decimal form) with Q being discharge rate ($m^3 s^{-1}$) and C_0, C_t being initial and time-specific concentrations (mg l^{-1}) respectively. This equation was calculated for the Pershore highway wetland example quoted above in respect of a total runoff volume (7560 m^3) for a 2 year, 1 h storm event (with total rainfall-runoff depth of 25 mm) which generated a BOD EMC value of 32 mg l^{-1} and an observed removal performance of 50%. These values together with k and ρ values of 1.1 and 0.35 respectively, were used to determine treatment areas derived by the three alternative sizing methods. The results (for a specified 50% BOD target removal) are shown in Table 4 and it is interesting to note that both the kinetic and 1% contributory area methods approximate the actual 1.3 ha surface area of the highway wetland.

Whilst a kinetic sizing approach therefore seems reasonable in terms of water quality performance, it would be unwise to place too much reliance in this simple modelling procedure as noted by Kadlec (1994). First-order kinetics are probably not applicable beyond the initial rate period and hydraulic loading rates probably become much more dominant in (the typically) small highway wetland systems. Additionally, as residence time increases, the value of k will change and the use of a single reaction rate based on organic carbon may well substantially oversize a wetland system primarily intended to treat solids, hydrocarbons and metals. Considerable variations were also noted in the recorded k values between the inlet and outlet zones in the Pershore highway wetland, varying between –0.02 to 8.59 for Total P although BOD k values did remain more stable around the 0.8 value. Such variation reflects the variable sedimentation, filtration and uptake rates which occur within the wetland and which affect the overall assimilation rates. The advantage of kinetic sizing is that the design procedure does allow a fairly rapid and robust means of estimating removal

Table 4 Wetland treatment areas derived from sizing criteria.

Sizing method	Size without pre-treatment forebay (ha)	Size with pre-treatment forebay (ha)	Source
Percentage connected 1%	0.9	0.9	Ellis (1990)
Impervious area 5%	4.5	4.5	Marble (1992)
Hydraulic sizing	0.4	0.4	Schueler (1992)
	2.4	2.4	Ellis & Revitt (1991)
Kinetic sizing	1.1	0.4	Reed *et al.* (1993)
	2.3*	–	Lawrence & Breen (1998)

* Based on adsorption rate (calculated value of 2.5).

performance at different hydraulic loading rates and utilizes process-based water quality criteria in addition to hydraulic considerations.

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