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transfer (Table 2). Therefore, in a statistical sense, the viscous sublayer does represent a steady-state condition of a specific thickness, which in turn defines the diffusional sublayer in the same statistical manner. The frequency of the burst is related to the thickness of the sublayer. As the velocity and shear increase, the viscous sublayer is eroded [see Figs. 7–8 Hinze (14)] to a point where the resistance due to viscous transfer is minimal. In a comparable manner, the friction factor varies from smooth to rough conditions in pipe flow, independent of viscosity in the latter region, as is the transfer coefficient at high shear.

The discussion of waves is relevant—whether they be gravity waves, wavelets or cepillary waves. The sea surface subject to winds is composed of the spectrum. All are significant in mass and momentum transfer and are related to the shear and pressure effects at the interface. The waves are not necessarily the source of surface renewal. In some experiments, mechanically generated waves had little effect on mass transfer. The point to be made is that the waves, shear and roughness are interrelated and possibly other characteristics of the interface such as surface and formation of spray and bubbles, all of which may be related to transfer. Therefore one could choose one or more of these factors in developing a mass transfer model. The writer simply chose the shear and roughness to reflect the overall process without specifying the precise physical mechanism. Hopefully the discussor's proposed experiments will clarify some of these issues.

The last point raised by the discussion was directed to the drag coefficient. C_d, which relates the shear to the wind velocity. The writer fully agrees that the shear velocity is a more fundamental measure of transfer and therefore placed emphasis on these experiments in which both shear and transfer coefficients were measured (Fig. 4 for the lab and Fig. 5 for the field). However, for engineering application, wind velocities as a designated elevation (commonly 10 m) is more useful and practical. To answer the discussor's point directly: for the experiments in which both shear and wind velocity were measured, the roughness and the drag coefficient may be calculated directly and thus the 10 m wind. In those cases in which only the wind velocity was reported, the author accepted the original source as the basis for extrapolation to the 10-m, which appeared reasonable in all cases. The calculated relationships shown in Fig. 6 are based on Eqs. 10 and 11, for conversion to the 10-m elevation.

In laboratory experiments, the wind velocity is invariably measured at convenient elevations above the water surface, usually within 1 m. In some of these cases, the original references extrapolated to the 10-m elevation and these were accepted by the writer since they appeared reasonable. In those cases in which such an extrapolation was not made, the writer used Eqs. 10 and 11, with $\lambda = 8$, $U_t = 10$ cm/s and $Z_e = 0.25$ cm, as presented in Figs. 1 and 2. Use of a different, (e.g. $\lambda = 20$ and $U_t = 9$, Fig. 1) resulted in small differences, indistinguishable with respect to Fig. 6.

In conclusion, the writer closes with a note of appreciation to the discussers for provocative and informative comments and questions. Both the theoretical and applied aspects of the paper were addressed, reflecting the appropriate and necessary elements of environmental engineering.

APPENDIX.—REFERENCE

John A. Mundell

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Errata.—The following corrections should be made to the original paper:

Page 741, Fig. 4 caption: Should read "Transfer Coefficient as Function of Shear Velocity: Lab Data"
Page 743, Fig. 5 caption: Should read "Transfer Coefficient as Function of Shear Velocity: Field Data"

DESIGN OF NATURAL ATTENUATION LANDFILLS[a]

Discussion by John A. Mundell,[2] A. M. ASCE

The author should be congratulated for addressing such an important topic and presenting such a timely paper regarding the design of landfills for attenuating leachates generated from sanitary landfills. Several of the steps in the semi-quantitative design method merit further discussion and clarification.

In determining the volume of soil available for attenuation reactions below the landfill, the author uses the base area of the landfill and the average depth of the unsaturated zone as the dimensions of the volume. Long-term steady-state seepage from the base of a landfill would actually occur through a cross section larger than that used by the author.

Seepage from the base of an impoundment generally takes place in three stages (40). During the first stage, a wettting front advances downward through the partially saturated underlying soil. The development of a groundwater mound occurs during the second stage as the wetting front contacts the phreatic surface of an aquifer and rises in response to seepage from the impoundment. In the third stage, saturated seepage begins to occur through the groundwater mound as it comes in contact with the base of the landfill.

During the development of the mound in the second stage, the groundwater has a tendency to spread laterally as the phreatic surface rises, until it reaches an equilibrium configuration during steady-state conditions. The flow lines have been shown to diverge from the vertical as they pass from the impoundment and approach the initial phreatic surface (32,37,39).

[a]August, 1983, by Amalendu Bagchi (Paper 18138).

[2]Environ. Research Assoc., Dept. of Civ. Engrg., Univ. of Notre Dame, Notre Dame, Ind. 46556.

TABLE 4.—Typical Specific Gravity Values and Their Variability

Soil type (1)	Specific Gravity Characteristics Mean (2)	Standard deviation (3)	Coefficient of variation (4)
Silt (44)	2.563	0.029	1.1
Silty clay (35)	2.66	0.05	1.87
Clay (34)			
High plasticity	2.63	0.115	4.4
Medium plasticity	2.66	0.060	2.3
Low plasticity	2.69	0.054	2.0

The error involved in assuming vertical flow lines varies depending on the ratio of the base width, B, of the landfill to the depth of the unsaturated zone, T. For T/B ratios typical of many sanitary landfills (say 0.05–0.2), the soil volume calculated by using the author's assumption may be on the order of 10%–20% less than is actually the case. The writer suggests using a 10% larger volume in this calculation to account for the additional soil involved in the attenuation process.

It appears that the reduction factor, R, used to account for soil fabric effect, should be defined as:

$$R = \frac{V_s}{V_t} = \frac{V_t - V_v}{V_t} = 1 - \frac{V_v}{V_t} = 1 - n \quad (7)$$

in which V_s = volume of soil; V_v = volume of voids; V_t = total volume; and n = porosity of the soil. From phase diagram relationships, it can be demonstrated that:

$$n = \frac{G_s\gamma_w - \gamma_d}{G_s\gamma_w} \quad (8)$$

in which G_s = specific gravity of soil; γ_d = dry unit weight of the soil; and γ_w = unit weight of water. It follows that R may be found by determining the specific gravity (ASTM D-857) and dry unit weight (using ASTM D-2216) from undisturbed soil samples (e.g., Shelby tube samples) taken from the unsaturated zone below the landfill. The volume of soils, V_s, should then be multiplied by the specific gravity of the soil to determine the total weight of soil available. Typical specific gravity characteristics for fine-grained soils are given in Table 4.

Alternatively, the total weight of the soil available can be calculated by taking the dry unit weight of the soil (as determined from laboratory tests on undisturbed soil samples) times the total volume involved in the attenuation process below the landfill to calculate the total weight of the soil available.

In the author's sample problem, values for peak concentrations of selected parameters in leachate are given in Table 1. Table 5 gives the range of concentration of leachate composition in sanitary landfills in the United States (33) for several pertinent parameters. Because of the broad range of concentrations given, the writer believes that an appropriately conservative assumption would be to design for 75% of the maximum concentration level of each parameter, unless more information can be provided regarding site-specific leachate characteristics. In addition, since design procedures relating to most engineering works generally incorporate an appropriate factor of safety to account for uncertainties in the design, the writer also suggests that it may be desirable in some cases to apply a factor of safety to the design concentrations based on the location of the landfill with respect to aquifers, wells, etc. The factor of safety could range from 1–10 depending on the amount of risk and uncertainties involved and the consequences of a system failure (i.e., too high a level of concentration of any pollutants).

TABLE 5.—Range of Selected Leachate Parameters in Sanitary Landfills in the United States (33)

Leachate parameter (1)	Range of analyses, in milligrams per liter (2)	Suggested design concentration,[a] in milligrams per liter (3)
Ammonia Nitrogen	0–1,106	830
Magnesium	17–15,600	11,700
Potassium	28–3,770	2,830
Sodium	0–7,700	5,780
Zinc	0–370	280
Copper	0–9.9	8
Lead	0.10–2	2

[a]75% of maximum concentration level given in range of analyses column.

It should be mentioned that a time leachate concentration function similar to that presented in Fig. 2 has been suggested previously (41) whereby the maximum concentration is reduced by 50% each year up to the fourth year when the concentration is assumed to remain constant at 12.5% of the initial maximum concentration. Leaching is assumed to take place for 20 yr.

Since the plume geometry the author developed is based on the groundwater flow being horizontal in the area underlying the landfill (which is seldom the case), any deviation from that assumption may cause the results to be either conservative or unconservative. If downward gradients are evident in the area (such as those present at sites located in upland recharge areas away from major water courses), further dilution within the aquifer will cause concentrations to be lower than those calculated by the author. Upward gradients (such as those present at sites near or adjacent to discharge areas) will cause less dilution and greater concentrations than those calculated. A study of the actual groundwater flow conditions in the vicinity of the landfill site should be made to determine any deviation from this assumption, and the concentrations adjusted accordingly.

The design method proposed assumes that the clay stratum which will attenuate several of the leachate parameters is relatively intact and free of the presence of any significant secondary structure which may include pervious seams, fissures or fractures. The presence of a high percentage of these defects in a clay stratum brought about by the method of deposition, or volume changes during dessication or geochemical processes, can greatly alter the stratum's ability to attenuate, and may instead allow the passage of leachate relatively unattenuated (36,38,42,43). The presence of such defects must be assessed during the hydrogeologic study of the landfill area by performing in-situ permeability testing.

Finally, the author's procedure may also be used to evaluate the feasibility of placing a compacted clay liner exhibiting a higher cation exchange capacity than the surrounding soil below the base of the landfill in order to make a site suitable fo rsanitary landfilling, or to provide greater protection at a site which may be near a critical groundwater supply.

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Closure by Amalendu Bagchi,[3] M. ASCE

The writer would like to thank Mondell for his interest in the paper and for his valuable comments. While the discusser is correct in stating that the volume of soil involved in the attenuation process will be more than what has been used in the paper, the increase is not suggested. The additional volume would play the role of a safety factor if it is not taken into account in the design.

The discusser has attempted to define the reduction factor "R." It should be noted however, that "R" has been introduced in Eq. 1 for the following reason, which is different from what the discusser has assumed. Flow of pore fluid through soil is mostly via large pores (45), therefore, the total Cation Exchange (CE) capacity of the attenuating soil mass must be reduced to account for the soil which does not come in contact with the leachate. Since pulverized soil mass is used in the laboratory test for CE capacity determination (46), the theoretical total CE capacity of the attenuating soil mass must be reduced by a factor to obtain the actually available total CE capacity. More research is needed to determine the value of the reduction factor "R."

Because of the broad range of concentration of different parameters in the leachate, the discusser suggested that 75% of the maximum concentration of a parameter should be used in the design procedure. Eventhough the maximum concentration will occur for a short period of time during the active phase of the site, the use of a reduced concentration will weaken the design logic by incorporating an element of definite failure during the occurrence of the peak concentration. In addition, since the design procedure does not involve analysis of forces, the use of a numerical safety factor is not advisable. One may note that because of the variability of the leachate quality, it is difficult to know the peak concentration of a parameter for any specific site. One approach could be to adopt a most probable municipal leachate characteristics typical for a state or a region. The probable range of leachate characteristics may be established by analyzing leachate obtained from containment type municipal landfills in that state or region, over a period of time.

The discusser has suggested that dilution rate will be influenced by the upward or downward gradient of the groundwater. While the writer thinks that the concept is possibly correct, he does not have sufficient data to comment on the dilution rate for nonhorizontal groundwater flow. Additional research is needed for proper conclusion about dilution rate for nonhorizontal groundwater flow.

Finally, the writer agrees with the discusser about the need for detailed hydrogeologic investigation for any natural attenuation type landfill.

Appendix.—References

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[3]Environ. Engr., Bureau of Solid Waste Management, Wisconsin Dept. of Natural Resources, Box 7921, Madison, Wisc. 53707.

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New Activated Sludge Theory: Steady State[a]

Discussion by Richard I. Dick,[2] M. ASCE

Mikesell addresses limitations of current models for the performance of activated sludge systems by considering the resistance to substrate and oxygen diffusion in floc particles and accounting for production of extracellular polymer. The author suggests that the approach can be used to predict activated sludge particle size and sedimentation velocity. This discussion is restricted to aspects of the paper concerning particle size and sedimentation velocity. Major points of the discussion are that particle size in aeration tanks cannot be deduced from measurement of activated sludge sedimentaiton velocity under quiescent conditioning and, conversely, that activated sludge zone settling velocity differs from the Stokian settling velocity of floc particles that exist in the aeration tank. Thus, sedimentation velocity cannot be predicted from performance of the substrate removal phase of the actiated sludge process.

The author calculated particle sizes from experimental data of Bisogni and Lawrence (19) by assuming that sedimentation was unhindered so that Stokes Law for sedimentation of discrete particles at low values of Reynolds Number applied. Not only is the sedimentation of ordinary activated sludge at a suspended solids concentration of 2,000 mg/L hindered, it is so severely hindered that all particles settle at the same collective velocity regardless of their individual discrete sedimentation velocity, and, thus, a distinct liquid-solids interface is observable. It was the subsidence rate of this interface that Bisogni and Lawrence reported. The Richardson-Zaki (20) equation

$$v_s = v_0 \epsilon^n \qquad (18)$$

illustrates the distinction between suspension sedimentation velocity, v_s, and the Stokian velocity of particles comprising the suspension, v_0. Epsilon in Eq. 1 is the suspension porosity and n is a coefficient that Richardson and Zaki found to have a value of 4.65 under viscous flow conditions. Because the volumetric concentration of activated sludge solids is high even at low gravimetric concentrations, ϵ in Eq. 1 is significantly less than one, and the suspension subsides at a velocity less than the settling velocity of individual particles.

[a]February, 1984, by Ritchie D. Mikesell (Paper 18547).

[2]The Joseph P. Ripley Prof. of Engrg., Cornell Univ., Ithaca, N.Y., 14853.

Extrapolation of porosity data for activated sludge reported by Javaheri and Dick (21) to a suspended solids concentration of 2,000 mg/L (as used by Bisogni and Lawrence) gives a value of approximately 0.85, and a porosity of this magnitude is consistent with observations based on long-term settled activated sludge volume. Use of a porosity of 0.85 in Eq. 1 indicates that Bisogni and Lawrence's activated sludge might have settled about one-half as fast as the individual particles that comprised it.

In calculating particle diameters from observed zone sedimentation velocities, Mikesell assumed $\Delta\rho/\rho = 0.1$, in which $\Delta\rho$ is the difference between the mass density of the floc particle and the mass density of water, ρ. Thus, floc mass density was taken as about 1.1 g/cm^3. This is approximately the mass density of the individual microorganisms that make up activated sludge floc (22). But water, not cells, is the major constituent of the loosely aggregated particles that settle in activated sludge sedimentation basins. Thus, $\Delta\rho/\rho$ is much lower than the 0.1 value assumed by the author. Based on calculations by Javaheri and Dick (21), $\Delta\rho/\rho$ for subsiding activated sludge particles at a suspended solids concentration of 5,000 mg/L would be 0.0015, but lower values would be expected at lower suspended solids concentrations. Another activated sludge examined by the same authors had a $\Delta\rho/\rho$ value of about 0.0003 at suspended solids concentration of 2,500 mg/L. Tambo and Watanabe (23) found $\Delta\rho/\rho$ values for settling activated sludge floc as low as 0.00013.

If the author had taken hindered settling into account and used a $\Delta\rho/\rho$ value of about 0.001, then floc diameters calculated from Bisogni and Lawrence's data would have ranged from about 600 μm to 2,500 μm (0.6 mm to 2.5 mm). These values compare well to the 0.2 mm to 2 mm range of settling activated sludge floc diameters reported by Tambo and Watanabe (23), to the 2.6 mm diameter for a settling activated sludge at 2,500 mg/L calculated by Javaheri and Dick (21), and to crude visual observations of settling activated sludge slurries.

Particle diameters calculated from sedimentation rates, however, indicate the size of particles that exist in a quiescent sedimentation test—not in a turbulent aeration basin. Because of disruption due to shearing forces, activated sludge floc particles are much smaller in aeration tanks than in settling tanks. Englande and Eckenfelder (24) reported laboratory aeration tank floc diameters ranging from 130 μm (in a sample from an aeration tank with low turbulence) to 31 μm (in a sample from an aeration tank with high turbulence). These values are roughly 20 fold lower than the diameters of settling activated sludge particles listed in the preceding paragraph. Floc density in areation tanks would be higher than in sedimentation vessels because of the compact nature of small aggregates (25), but, unless heavy abiotic particles were present, the value of about 1.1 g/cm^3 used by the author could only be approached as floc size diminished to that of an individual cell.

If the author succeeds in calculating a particle diameter based on the diffusive effectiveness factor, then it will be the size of floc particles as they exist in aeration tanks. Knowledge of that diameter will not be useful in calculating sedimentation properties of the sludge following reduction of shear rate and formation of large aggregate particles in the sedimentation basin. Floc diameter in aeration basins is controlled by