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# Sediment Transport in Storm Drainage Systems



## Abstract
Much research has been conducted on the transport, settling, and scour of gross solids in sanitary sewers and in combined sewers, especially in Europe during recent years (Ashley, *et al.* 1999; 2000; 2002; Butler and Karunaratne 1995; Butler, *et al.* 1995; Cigana, *et al*. 1998a, 1998b, 1998c, 1999, 2000, 2001; plus review by Pisano, *et al*. 1998, amongst others). However, relatively little recent research has been conducted concerning the fate of larger particulates that enter separate stormwater drainage systems. Historical design approaches are intended to minimize particulate deposition in sewerage, and usually present a minimum pipe slope or a minimum velocity objective. If followed, these guidelines usually result in minimal maintenance problems associated with particulate accumulations. However, it is still important to minimize erosion sources in the watershed. In addition, catchbasin sumps have also been used to trap the larger particulates that enter storm drainage inlets. Therefore, particulate transport in separate storm drainage has not been considered to be a significant problem for public works managers. However, much more is needed to be known about particulate transport in stormwater drainage systems when conducting stormwater quality investigations, especially when examining the effects of source area controls on outfall quality. Prior studies have conducted mass balances in urban drainage systems and have found significant accumulations of solids in the drainage systems. These accumulations are mostly of the largest particulates that enter the drainage system, effectively preventing these from being discharged to the receiving waters. Many source area stormwater quality controls, such as street cleaning and the use of catchbasins, also preferentially remove the largest particulates. Modeling of the benefits of these controls therefore typically leads to inaccurate conclusions concerning reduced discharges of these particulates, and associated pollutants. The pollutants being removed would not likely be effectively transported through the drainage system, but would instead accumulate. In addition, source area and inlet samples frequently indicate much larger amounts of large particulates than would be discharged to the receiving water. This also misleads management strategies pertaining to stormwater quality. This paper presents observations from an urban area mass balance investigation along with some traditional particulate transport approaches in an attempt to explain some of these observations.

## Background
### *Effects of Catchbasin and Street Cleaning*
A study was conducted in Bellevue, Washington (Pitt 1985), in two mixed residential and commercial study areas, as part of the Nationwide Urban Runoff Program (EPA 1983). One task of this research included the monitoring of catchbasins, simple inlets, man-holes, and sewerage sediment accumulations at more than 200 locations for a period of three years. The sediment in the catchbasins and the sewerage was found to be the largest particles that were washed from the streets. The sewerage and catchbasin sediments had a much smaller median particle size than the street dirt and were therefore more potentially polluting than the particulates that can be removed by street cleaning. Cleaning catchbasins twice a year was found to allow the catchbasins to capture particulates most effectively. This cleaning schedule was found to reduce the total residue and lead urban runoff yields at the outfalls by between 10 and 25 percent, and COD, total Kjeldahl nitrogen, total phosphorus, and zinc by between 5 and 10 percent (Pitt and Shawley 1982).

This research examined two study areas, Lake Hills and Surrey Downs, both similar medium density residential areas. Each study area was examined with four separate experimental conditions: no controls, street cleaning alone, catchbasin cleaning alone, and both street cleaning and catchbasin cleaning together. This research was therefore conducted in a replicated complete block design, allowing runoff quality comparisons between periods having these different public works practices. The eight experimental categories were as follows:

1. Lake Hills, Active CB, No SC (catchbasins were accumulating material, but no street cleaning operations were being conducted during this project period).

2. Lake Hills, Active CB, SC (catchbasins were accumulating material, and street cleaning operations were being conducted during this project period).

3. Lake Hills, Full CB, No SC (catchbasins were full and not accumulating material, and no street cleaning operations were being conducted during this project period).

4. Lake Hills, Full CB, SC (catchbasins were full and not accumulating material, street cleaning operations were being conducted during this project period).

5. Surrey Downs, Active CB, No SC (catchbasins were accumulating material, but no street cleaning operations were being conducted during this project period).

6. Surrey Downs, Active CB, SC (catchbasins were accumulating material, and street cleaning operations were being conducted during this project period).

7. Surrey Downs, Full CB, No SC (catchbasins were full and not accumulating material, and no street cleaning operations were being conducted during this project period).

8. Surrey Downs, Full CB, SC (catchbasins were full and not accumulating material, street cleaning operations were being conducted during this project period).

Catchbasins were were cleaned and surveyed at the beginning of the project. The accumulation of material was then monitored through periodic measurements. The project periods were therefore categorized as "active" or "full." The active periods were when accumulation was taking place in the catchbasins, while the full periods were when the catchbasins were at an equilibrium, with no additional accumulation of material.

The following are Student *t* test results to measure the significance of the difference between selected data groups for outfall total solids concentrations. There would have to be a 50 to 75% difference between the sample means of the two categories to identify a significant difference, with 10 to 15 storms representing each of the two categories for each test site, using a power of 80%, and assuming a typical COV of about 0.75. P values smaller than 0.05 are usually considered as being significantly different (at the 95% confidence level), while larger P values indicate that not enough data are available to distinguish the data groups at the measured differences.

Student's *t*-test results:

2 vs. 6: both street and catchbasin cleaning in both areas, LH vs. SD
P value: 0.71 (not enough data to detect a difference)

3 vs. 7: nothing in both areas, LH vs. SD
P value: 0.031 (significantly different)

2 vs. 3 LH both street and catchbasin cleaning vs. nothing
P value: 0.037 (significantly different)

6 vs. 7 SD both street and catchbasin cleaning vs. nothing
P value: 0.99 (not enough data to detect a difference)

When both street and catchbasin cleaning was being conducted in both areas, the outfall total solids concentrations appeared to be the same (as expected). However, when no controls were in use in either area, the outfall total solids concentrations were significantly different (Lake Hills had lower total solids concentrations compared to Surrey Downs), which was not expected. When both street and catchbasin cleaning was conducted in Lake Hills, the outfall total solids concentrations were significantly larger than when no cleaning was being conducted, which also was not expected. In Surrey Downs, no differences were detected when cleaning was conducted compared to no cleaning.

These results are counter-intuitive. The hypothesis was that the two watersheds would behave in a similar manner when similar activities were being conducted in each, and that the cleaning would reduce the outfall total solids discharges. Over the years, a number of reasons have been given for the observed odd behavior. Older street cleaning equipment was not very efficient in removing the particles that are washed off, and in fact, have been found to actually remove the larger particles that actually armour the finer materials, potentially increasing the solids discharges. However, the catchbasins are removing particles that have washed off the watershed area and have been transported to the drainage system, but this material likely would not have been transported all the way to the outfall. Ashley, *et al.* (1999, 2000, 2002) has extensively researched the transport of solids in combined sewerage. Unfortunately, similar information is currently lacking for separate storm drainage. The initial objective for the use of catchbasin sumps was to reduce the accumulation of coarse debris in the sewerage. These Bellevue tests seem to indicate the substantial benefit of the removal of this material that may otherwise cause potential flow obstruction problems in the drainage system. However, it is quite likely that this large material would rarely flow completely to the outfalls, at least under the relatively mild Bellevue rain conditions and during the time frame of this study.

### *Particle Size Distributions*

The particle size distributions of stormwater at different locations in an urban area greatly affect the ability of different source area and inlet controls in reducing the discharge of stormwater pollutants. A series of U.S. EPA funded research projects has examined the sources and treatability of urban stormwater pollutants (Pitt, *et al.* 1995). This research has included particle size analyses of 121 stormwater inlet samples from three states (southern New Jersey, Birmingham, Alabama; and at several cities in Wisconsin) that were not affected by stormwater controls. .

Particle sizes were measured using a Coulter Counter Multi-Sizer IIe and verified with microscopic, sieve, and settling column tests. Figures 1 through 3 are grouped box and whisker plots showing the particle sizes (in µm) corresponding to the $10^{th}$, $50^{th}$ (median) and $90^{th}$ percentiles of the cumulative distributions for the three areas. If 90 percent control of suspended solids (by mass) was desired, then the particles larger than the $90^{th}$ percentile would have to be removed, for example. In all cases, the New Jersey samples had the smallest particle sizes (even though they were collected using manual "dipper" samplers of the cascading water and not automatic samplers that may miss the largest particles), followed by Wisconsin, and then Birmingham, Alabama, which had the largest particles (which were collected using automatic samplers and had the largest rain intensities). The New Jersey samples were obtained from gutter flows in a residential neighborhood that was xeroscaped, the Wisconsin samples were obtained from several source areas, including parking areas and gutter flows mostly from residential, but from some commercial areas, and the Birmingham samples were collected from a long-term parking area. The median particle sizes ranged from 0.6 to 38µm and averaged 14µm. The $90^{th}$ percentile sizes ranged from 0.5 to 11µm and averaged 3µm.

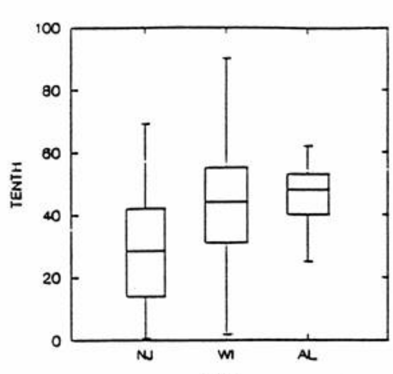


**Figure 1. Tenth percentile particle sizes for stormwater inlet flows (Pitt, *et al*. 1997).**

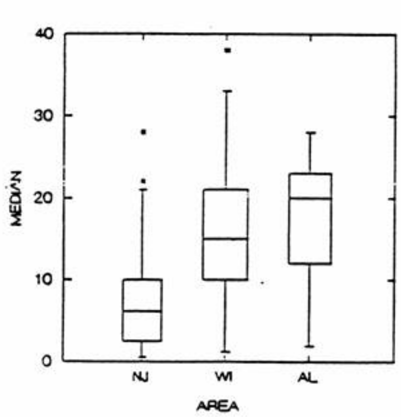


**Figure 2. Fiftieth percentile particle sizes for stormwater inlet flows (Pitt, *et al*. 1997).**

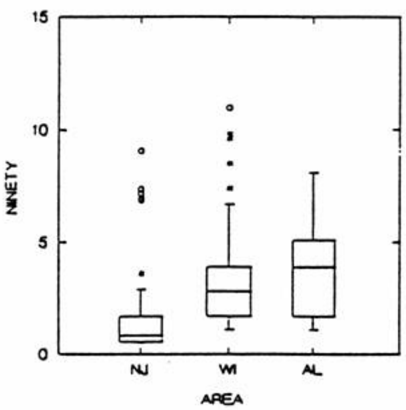


**Figure 3. Ninetieth percentile particle sizes for stormwater inlet flows (Pitt, *et al*. 1997).**

Stormwater particle size distributions typically do not include bed load components because the sample line velocities in automatic samplers may not be high enough to collect the largest material, plus the line diameter may be smaller than some of the bed load material. During the Monroe St. (Madison, WI) detention pond monitoring, the USGS and WI DNR installed special bed load samplers that trapped the bed load material for analysis. The bedload samplers were liter-sized wide mouth containers which were placed in bored holes in the bottom of the enclosed flat bottomed concrete small box channels right before the pond. Three units were placed in the channel bottom, each having different width slots cut in their lids. The mass of material trapped was directly related to the ratio of the width of the slot to the width of the channel. The material was removed, dried, sieved, and weighed. This particle size distribution was combined with the flow-weighted particle distributions obtained for the runoff events monitored during the same exposure period. Practically all events were monitored, with little flow not represented in these analyses. Figure 4 shows the measured particle size distributions for 16 seasonal samples (each having several runoff events), also including the bedload particle size distributions. The bedload samplers were in place for several weeks at a time in order to accumulate sufficient sample for analyses. The bedload material was comprised of the largest material represented on this figure (generally about 300 or 400 µm and larger particulates) and was about 10 percent of the annual total solids loading, but ranged from about 2 to 25 percent for individual periods. The bed load component in Madison was most significant during the early spring rains when much of the traction control sand that could be removed by rains was being washed from the streets. This is not a large fraction of the solids, but it represents the largest particle sizes flowing in the stormwater and it can be easily trapped in most detention ponds or catchbasins.

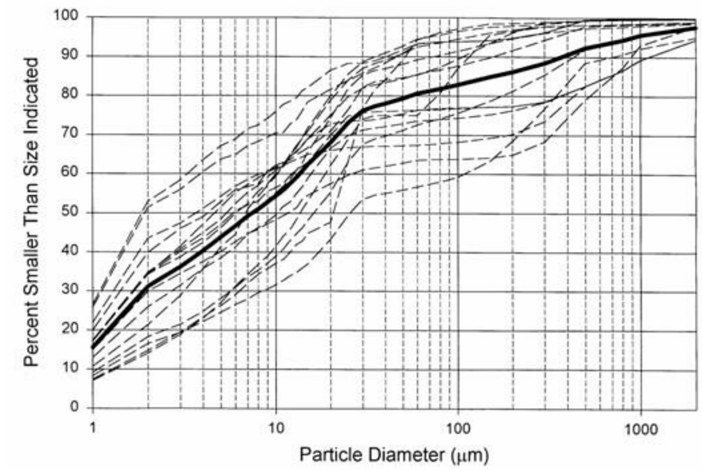


**Figure 4. Inlet particle size distributions observed at the Monroe St. wet detention pond.**

Figure 5 shows a typical "delta" of large material immediately near the influent to a wet detention pond, along with an accumulation of material along the invert of the corregated steel drainage pipe. This photo was taken at a pond in Snowmass, CO, in an area of heavy sand applications for traction control in the winter. The bedload sediment material in this photo is quite large, several mm in diameter, and near the upper range of the particle size distribution shown previously. This drainage pipe is relatively short, connected to an adjacent parking area. It is rare for this large material to be transported great distances in drainage systems. If it does enter a pond, or any type of sediment device, it is easily trapped near the inlet. However, small to moderate sized particulates can easily be transported quite some distance in the drainage system and be deposited well away from the inlet when discharged into wet detention ponds. The use of forebays, or small pre-settlement ponds, will trap much of the moderate-sized particulates (down to about 25μm) within a relatively small area for easier removal during maintenance operations. The finer particulates (down to just a few μm), containing most of the pollutants, would be trapped in the main pond area. The sediment depth is relatively small when spread over a large pond area.

**Figure 5. Bedload sediment accumulation in sewerage and near inlet to pond (Snowmass, CO).**

Additional data obtained by Pitt, *et al.* (1997) for the USEPA described particle sizes from many different source flows in the Birmingham, Alabama, area. These data did not indicate any significant differences in particle size distributions for different source areas or land uses, except that the roof runoff had substantially smaller particle sizes than the other areas sampled. Also, the source area particle size distributions indicated that larger particles were much more likely to be present at source areas than at outfalls. The larger particles appear to be trapped in the flow paths and drainage system before they reach the outfalls.

After the stormwater particulates enter the storm drainage, they will tend to settle as they flow towards the outfall to the receiving water. If they settle slowly, such as occurs for small particles, they will remain suspended and not become part of the bed load or sediment in the sewerage. However, if they settle to the bottom of the pipe before reaching the outfall, they may become part of the bed load which will bounce along the pipe bottom, or become trapped with other settled debris. When the flow stops, the sediments will tend to dry and become more consolidated. The next runoff event may cause some of this settled material to become resuspended and may move towards the outfall. Therefore, there are three phases to particulate transport in storm drainage systems: 1) settling of the particulates in the flowing water, 2) movement as bed load during the event, 3) accumulating as sediment and potentially subsequent scour. The following discussion describes these sediment transport phases.

### Settling of Particulates in Flowing Water in Storm Drainage Systems

The settling velocities of discrete particles are shown in Figure 6, based on Stoke's and Newton's settling relationships. Probably more than 90% of all inlet stormwater particulates are in the 1 to 100 µm range, corresponding to laminar flow conditions, and appropriate for using Stoke's law. This figure also illustrates the effects of different specific gravities on the settling rates. In most cases, stormwater particulates have specific gravities in the range of 1.5 to 2.5. This corresponds to a relatively narrow range of settling rates for a specific particle size. Particle size is much easier to measure than settling rates and it is generally recommended to measure particle sizes using automated particle sizing equipment (such as a Coulter Counter Multi-Sizer) and to conduct periodic settling column tests to determine the corresponding specific gravities and shape effects. If the particle counting equipment is not available, then small-scale settling column tests (using 50 cm diameter Teflon™ columns about 0.7 m long) can be easily used.

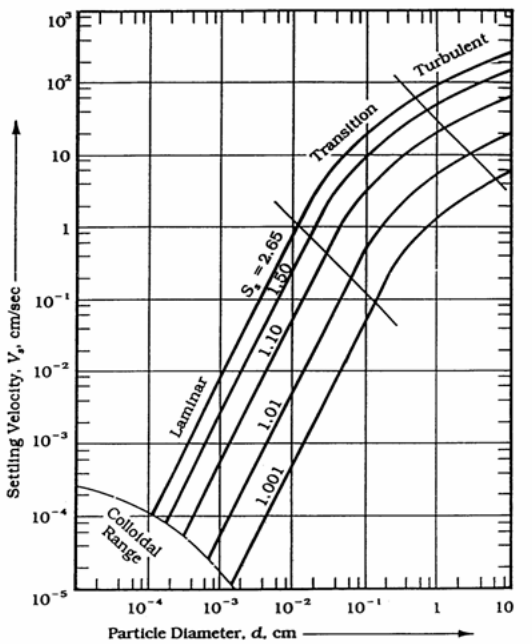


**Figure 6. Type 1 (discrete) settling of spheres in water at 10° C (Reynolds 1982).**

Pisano and Brombach (1996) obtained solids settling curves for numerous stormwater and CSO samples. They are concerned that many of the samples analyzed for particle size are not representative of the true particle size distribution in the sample. As an example, it is known that automatic samplers do not sample the largest particles that are found in the bedload portion of the flows. Particles having settling velocities in the 1 to 15 cm/sec (100 to 1,000 µm in size) range are found in grit chambers and catchbasins, but are seldom seen in stormwater samples obtained by automatic samplers. It is recommended that bedload samplers be used to supplement automatic water samplers in order to obtain more accurate particle size distributions (Burton and Pitt 2002). Selected US and Canadian settling velocity data are shown in Table 1. The CSO particulates have much greater settling velocities than the other samples, while the stormwater has the smallest settling velocities. The corresponding "Stoke's" particle sizes for the geometric means are about 100 µm for the CSOs, about 50 µm for the sanitary sewage, and about 15 µm for the stormwater.

**Table 1. Settling Velocities for Wastewater, Stormwater, and CSO (Pisano and Bromback 1996)**

| Samples | Geometric Means of Settling Velocities Observed (cm/sec) | Range of Medians of Settling Velocities Observed (cm/sec) |
|---|---|---|
| CSO | 0.22 | 0.01 to 5.5 |
| dry weather wastewater (sanitary sewage) | 0.045 | 0.030 to 0.066 |
| stormwater | 0.011 | 0.0015 to 0.15 |

More than 13,000 CSO control tanks have been built in Germany using the ATV 128 rule (Pisano and Bromback 1996). This rule states that clarifier tanks are to retain all particles having settling velocities greater than 10 m/hr (0.7 cm/sec, or about 100 µm), with a goal of capturing 80% of the settleable solids. Their recent measurements of overflows from some of these tanks indicate that the 80% capture was average for these tanks and that the ATV 128 rule appears to be reasonable for combined sewerage.

Table 2 presents settling conditions for particulates moving in pipes ranging from 1 to 5 ft in diameter, and for flow depths ranging from 10% to 100% of the pipe diameters. Pipe slopes ranging from 0.1 to 2% and particles from 1 to 10,000 µm, all with specific gravities of 2.5, are used in these calculations. This table shows the distances the particles would travel before they would settle to the bottom of the pipe, if starting from the surface of the flow, using Manning's equation to calculate the stormwater velocity, and the combination of Stoke's and Newton's laws for settling rates. A particle settling to the pipe bottom doesn't imply that the particles would be permanently trapped as sediment, but the particles may move (relatively slowly) as part of a mobile bedload. Obviously, the flow distances required for settling for the smallest particles are very long and would remain suspended. Some of the 100 µm particle flow conditions are shown to result in relatively short settling distances, while most of the 1,000 and 10,000 µm particles would most certainly settle to the pipe bottom before reaching the outfall. If the flow and associated bed load movement stops before the particles reach the outfall, they will form a more compact sediment requiring more energy during subsequent rains to scour.

**Table 2. Settling Distance (ft) for Particles Flowing in Pipes having Various Diameters and Slopes (n=0.013)**

| | | 1 µm particles (2.5 specific gravity) | | | | 10 µm particles (2.5 specific gravity) | | | |
|---|---|---|---|---|---|---|---|---|---|
| Pipe Diameter (ft) | flow depth/pipe diameter ratio | 0.001 slope | 0.005 slope | 0.01 slope | 0.02 slope | 0.001 slope | 0.005 slope | 0.01 slope | 0.02 slope |
| 1 | 0.1 | >10,000 | >10,000 | >10,000 | >10,000 | 180 | 390 | 550 | 780 |
| 1 | 0.3 | >100,000 | >100,000 | >100,000 | >100,000 | 1000 | 2300 | 3200 | 4600 |
| 1 | 0.5 | >100,000 | >100,000 | >100,000 | >100,000 | 2200 | 4895 | 6900 | >10,000 |
| 1 | 0.8 | >100,000 | >100,000 | >100,000 | >100,000 | 4000 | 8929 | >10,000 | >10,000 |
| 1 | 1 | >100,000 | >100,000 | >100,000 | >100,000 | 4400 | 9790 | >10,000 | >10,000 |
| 1.5 | 0.1 | >10,000 | >10,000 | >100,000 | >100,000 | 350 | 770 | 1100 | 1500 |
| 1.5 | 0.3 | >100,000 | >100,000 | >100,000 | >100,000 | 2000 | 4500 | 6400 | 9000 |
| 1.5 | 0.5 | >100,000 | >100,000 | >100,000 | >100,000 | 4300 | 9629 | >10,000 | >10,000 |
| 1.5 | 0.8 | >100,000 | >100,000 | >100,000 | >100,000 | 7900 | >10,000 | >10,000 | >10,000 |
| 1.5 | 1 | >100,000 | >100,000 | >100,000 | >100,000 | 8600 | >10,000 | >10,000 | >10,000 |
| 2 | 0.1 | >10,000 | >100,000 | >100,000 | >100,000 | 560 | 1300 | 1800 | 2500 |
| 2 | 0.3 | >100,000 | >100,000 | >100,000 | >100,000 | 3300 | 7300 | >10,000 | >10,000 |
| 2 | 0.5 | >100,000 | >100,000 | >100,000 | >100,000 | 7000 | >10,000 | >10,000 | >10,000 |
| 2 | 0.8 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >10,000 | >10,000 | >10,000 |
| 2 | 1 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >10,000 | >10,000 | >10,000 |
| 3 | 0.1 | >100,000 | >100,000 | >100,000 | >100,000 | 1100 | 2500 | 3500 | 4900 |
| 3 | 0.3 | >100,000 | >100,000 | >100,000 | >100,000 | 6400 | >10,000 | >10,000 | >10,000 |
| 3 | 0.5 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >10,000 | >10,000 | >10,000 |
| 3 | 0.8 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >10,000 | >10,000 | >100,000 |
| 3 | 1 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >10,000 | >10,000 | >100,000 |
| 5 | 0.1 | >100,000 | >100,000 | >100,000 | >100,000 | 2600 | 5800 | 8100 | >10,000 |
| 5 | 0.3 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >10,000 | >10,000 | >10,000 |
| 5 | 0.5 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >10,000 | >100,000 | >100,000 |
| 5 | 0.8 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >100,000 | >100,000 | >100,000 |
| 5 | 1 | >100,000 | >100,000 | >100,000 | >100,000 | >10,000 | >100,000 | >100,000 | >100,000 |

**Table 2. Settling Distance (ft) for Particles Flowing in Pipes having Various Diameters and Slopes (n=0.013) (cont.)**

| | | 100 µm particles (2.5 specific gravity) | | | | 1,000 µm particles (2.5 specific gravity) | | | | 10,000 µm particles (2.5 specific gravity) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pipe Diameter (ft) | flow depth/pipe diameter ratio | 0.001 slope | 0.005 slope | 0.01 slope | 0.02 slope | 0.001 slope | 0.005 slope | 0.01 slope | 0.02 slope | 0.001 slope | 0.005 slope | 0.01 slope | 0.02 slope |
| 1 | 0.1 | 1.9 | 4.4 | 6.2 | 8.7 | 0.1 | 0.2 | 0.3 | 0.4 | 0.0 | 0.0 | 0.1 | 0.1 |
| 1 | 0.3 | 11 | 25 | 36 | 51 | 0.5 | 1.1 | 1.6 | 2.3 | 0.1 | 0.2 | 0.3 | 0.5 |
| 1 | 0.5 | 24 | 54 | 77 | 110 | 1.1 | 2.4 | 3.5 | 4.9 | 0.2 | 0.5 | 0.7 | 1.0 |
| 1 | 0.8 | 44 | 99 | 140 | 200 | 2.0 | 4.5 | 6.3 | 8.9 | 0.4 | 0.9 | 1.3 | 1.8 |
| 1 | 1 | 49 | 110 | 150 | 220 | 2.2 | 4.9 | 6.9 | 10 | 0.4 | 1.0 | 1.4 | 2.0 |
| 1.5 | 0.1 | 3.8 | 8.6 | 12 | 17 | 0.2 | 0.4 | 0.5 | 0.8 | 0.0 | 0.1 | 0.1 | 0.2 |
| 1.5 | 0.3 | 22 | 50 | 71 | 100 | 1.0 | 2.3 | 3.2 | 4.5 | 0.2 | 0.5 | 0.6 | 0.9 |
| 1.5 | 0.5 | 48 | 110 | 150 | 210 | 2.2 | 4.8 | 6.8 | 10 | 0.4 | 1.0 | 1.4 | 1.9 |
| 1.5 | 0.8 | 87 | 200 | 280 | 390 | 3.9 | 8.8 | 12 | 18 | 0.8 | 1.8 | 2.5 | 3.5 |
| 1.5 | 1 | 96 | 210 | 300 | 430 | 4.3 | 10 | 14 | 19 | 0.9 | 1.9 | 2.7 | 3.9 |
| 2 | 0.1 | 6.2 | 14 | 20 | 28 | 0.3 | 0.6 | 0.9 | 1.2 | 0.1 | 0.1 | 0.2 | 0.2 |
| 2 | 0.3 | 36 | 81 | 120 | 160 | 1.6 | 3.6 | 5.2 | 7.3 | 0.3 | 0.7 | 1.0 | 1.5 |
| 2 | 0.5 | 77 | 170 | 250 | 350 | 3.5 | 7.8 | 11 | 16 | 0.7 | 1.6 | 2.2 | 3.1 |
| 2 | 0.8 | 140 | 320 | 450 | 630 | 6.4 | 14 | 20 | 28 | 1.3 | 2.8 | 4.0 | 5.7 |
| 2 | 1 | 160 | 350 | 490 | 690 | 7.0 | 16 | 22 | 31 | 1.4 | 3.1 | 4.4 | 6.2 |

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3 | 0.1 | 12 | 27 | 39 | 55 | 0.5 | 1.2 | 1.7 | 2.5 | 0.1 | 0.2 | 0.3 | 0.5 |
| 3 | 0.3 | 71 | 160 | 230 | 320 | 3.2 | 7.2 | 10 | 14 | 0.6 | 1.4 | 2.0 | 2.9 |
| 3 | 0.5 | 150 | 340 | 480 | 680 | 6.9 | 15 | 22 | 31 | 1.4 | 3.1 | 4.3 | 6.1 |
| 3 | 0.8 | 280 | 620 | 880 | 1200 | 13 | 28 | 40 | 56 | 2.5 | 5.6 | 7.9 | 11 |
| 3 | 1 | 310 | 680 | 960 | 1400 | 14 | 31 | 43 | 61 | 2.7 | 6.1 | 8.7 | 12 |
| 5 | 0.1 | 29 | 64 | 90 | 130 | 1.3 | 2.9 | 4.1 | 5.8 | 0.3 | 0.6 | 0.8 | 1.2 |
| 5 | 0.3 | 170 | 370 | 530 | 750 | 7.5 | 17 | 24 | 34 | 1.5 | 3.4 | 4.8 | 6.7 |
| 5 | 0.5 | 360 | 800 | 1130 | 1600 | 16 | 36 | 51 | 72 | 3.2 | 7.2 | 10 | 14 |
| 5 | 0.8 | 650 | 1500 | 2100 | 2900 | 29 | 66 | 93 | 130 | 5.9 | 13 | 19 | 26 |
| 5 | 1 | 710 | 1600 | 2300 | 3200 | 32 | 72 | 100 | 140 | 6.4 | 14 | 20 | 29 |

## Resuspension of Settled Particulates in Storm Drainage

This discussion presents some particulate transport information that can be used to predict if settled particles forming a sediment deposit in a pipe may be resuspended or scoured during subsequent events. This information does not allow predictions to be made concerning the accumulation of particulates in the sediment, only the likelihood that previously settled material may scour.

### *Allowable Velocity and Shear Stress*

**Allowable Velocity Data**

The concept of allowable velocities for various soils and materials dates from the early days of hydraulics. Table 3 is an example of allowable velocities from U.S. Bureau of Reclamation research (Fortier and Scobey 1926, reprinted by McCuen 1998) that also shows the corresponding allowable shear stresses and Manning's roughness values. If these velocities are exceeded for an extended period, it is assumed that the channel lining material can become unstable. These values are not directly applicable to pipe flows, but the typically used maximum velocity of about 3 ft/sec for storm drainage design is similar to the values for stiff clays and silts. It is interesting that clays can withstand higher velocities than sands.

**Table 3. Maximum Permissible Velocities and Corresponding Unit Tractive Force (Shear Stress) (U.S. Bureau of Reclamation research, Fortier and Scobey 1926)**

| | | Clear Water | | Water Transporting Colloidal Silts | |
|---|---|---|---|---|---|
| Material | n | V (ft/sec) | $\tau_o$ (lb/ft$^2$) | V (ft/sec) | $\tau_o$ (lb/ft$^2$) |
| Fine sand, colloidal | 0.020 | 1.50 | 0.027 | 2.50 | 0.075 |
| Sandy loam, noncolloidal | 0.020 | 1.75 | 0.037 | 2.50 | 0.075 |
| Silt loam, noncolloidal | 0.020 | 2.00 | 0.048 | 3.00 | 0.11 |
| Alluvial silts, noncolloidal | 0.020 | 2.00 | 0.048 | 3.50 | 0.15 |
| Ordinary firm loam | 0.020 | 2.50 | 0.075 | 3.50 | 0.15 |
| Volcanic ash | 0.020 | 2.50 | 0.075 | 3.50 | 0.15 |
| Stiff clay, very colloidal | 0.025 | 3.75 | 0.26 | 5.00 | 0.46 |
| Alluvial silts, colloidal | 0.025 | 3.75 | 0.26 | 5.00 | 0.46 |
| Shales and hardpans | 0.025 | 6.00 | 0.67 | 6.00 | 0.67 |
| Fine gravel | 0.020 | 2.50 | 0.075 | 5.00 | 0.32 |
| Graded loam to cobbles when noncolloidal | 0.030 | 3.75 | 0.38 | 5.00 | 0.66 |
| Graded silts to cobbles when noncolloidal | 0.030 | 4.00 | 0.43 | 5.50 | 0.80 |
| Coarse gravel, noncolloidal | 0.025 | 4.00 | 0.30 | 6.00 | 0.67 |
| Cobbles and shingles | 0.035 | 5.00 | 0.91 | 5.50 | 1.10 |

Note:
- an increase in velocity of 0.5 ft/sec can be added to these values when the depth of water is greater than 3 ft.
- a decrease in velocity of 0.5 ft/sec should subtracted then the water contains very coarse suspended sediments.
- for high and infrequent discharges of short duration, up to 30% increase in velocity can be added

**Allowable Shear Stress Data**

By the 1930's, boundary shear stress (sometimes called tractive force) was generally accepted as a more appropriate criterion than allowable velocity for channel stability. The average boundary shear stress in uniform flow is calculated by

$$\tau_o = \gamma RS \quad \text{(lb/ft}^2\text{)}$$

where:

$\gamma$ = specific weight of water (62.4 lbs/ft$^3$)

R = hydraulic radius (ft)
S = hydraulic slope (ft/ft)

Flow characteristics predicting the initiation of motion of sediment in noncohesive materials are usually presented in nondimensional form in the Shield's diagram (Figure 7). This diagram indicates the initial movement, or scour, of noncohesive uniformly graded sediments on a flat bed. The diagram plots the Shield's number (or mobility number), which combines shear stress with grain size and relative density, against a form of the Reynolds number that uses grain size as the length variable. The ASCE *Sedimentation Manual* (1975) uses a dimensionless parameter, shown on Figure 7, to select the dimensionless stress value. This value is calculated as:

$$\frac{d}{\nu}\left[0.1\left(\frac{\gamma_s}{\gamma}-1\right)gd\right]^{0.5}$$

where:

d = particle diameter (meters)
g = gravitational constant (9.81 m/sec$^2$)
ν = kinematic viscosity (1.306 x 10$^{-6}$ m$^2$/sec for 10$^o$C)
$\gamma_s$ = specific gravity of the solid
$\gamma$ = specific gravity of water

A series of parallel lines on Figure 7 represent these calculated values. The dimensionless shear stress value ($\tau_*$) is selected where the appropriate line intersects the Shield's curve. The critical shear stress can then be calculated by:

$$\tau_c = \tau_*(\gamma_s - \gamma)d$$

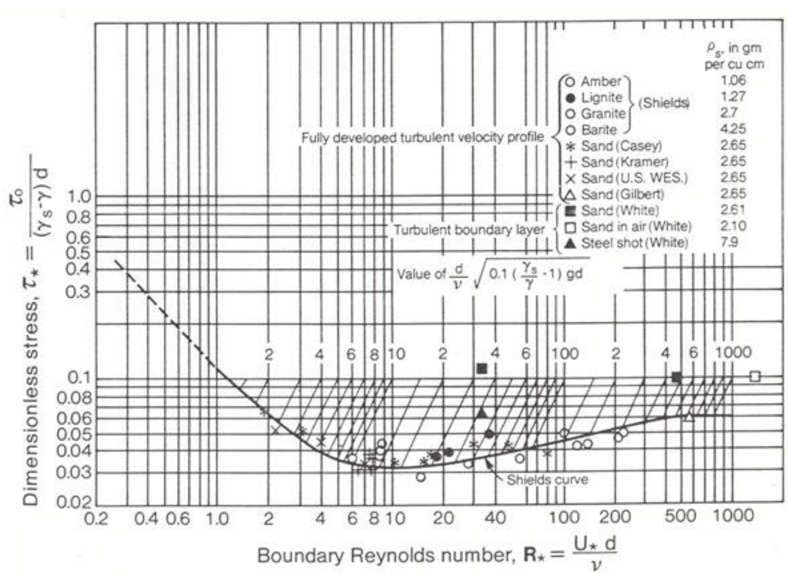


**Figure 7. Shield's diagram for dimensionless critical shear stress (COE 1994).**

An example evaluation is given by the COE (1994) in their assessment manual. In their example, the use of the Shield's diagram is shown to likely greatly over-predict the erodibility of the channel bottom material. The expected reason they give is that the Shield's diagram assumes a flat bottom channel and the total roughness is determined by the size of the granular bottom material. The actual Manning's roughness value is likely much larger because it is largely determined by bed forms, channel irregularities, and vegetation, and not grain size. They recommend, as a more realistic assessment, that empirical data based on field observations be used. In the absence of local data, they present Figure 8 (from Chow 1959) for applications for channels in granular materials. This figure shows the permissible unit tractive force (shear stress) as a function of the average particle diameter, and the fine sediment content of the flowing water.

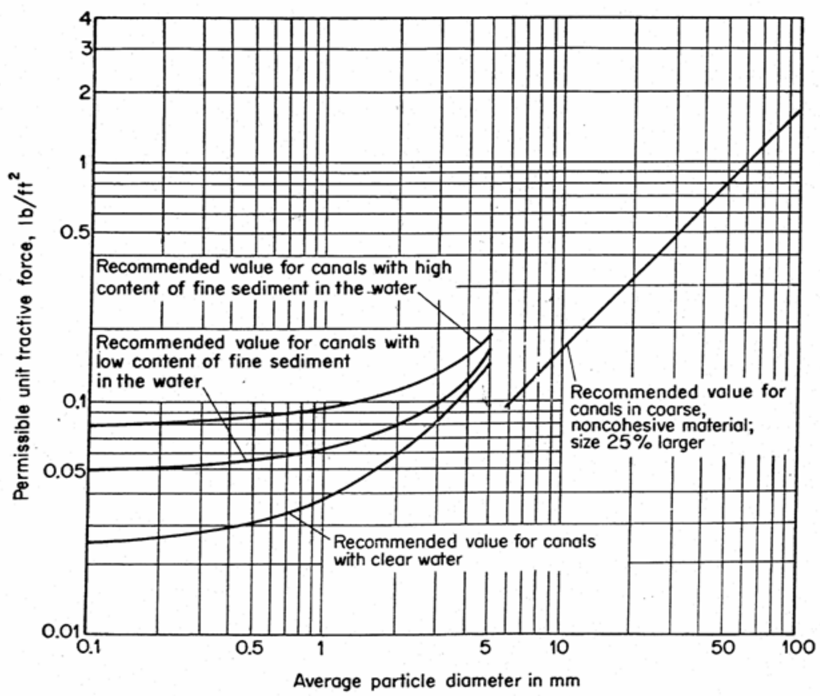


**Figure 8. Allowable shear stresses (tractive forces) for canals in granular materials (U.S. Bureau of Reclamation, reprinted in Chow 1959).**

Table 4 shows calculated shear stresses, velocities, and discharge quantities for various pipe conditions. Also shown are the estimated maximum particles sizes that would not be scoured during these flow conditions. For the smallest slopes, almost all settled particles would likely remain and not be scoured, while almost all unconsolidated sediments would be scoured for pipe greater than 1% in slope. This table shows that pipe conditions resulting in at least 3 ft/sec stormwater velocities would also have shear stresses of at least 0.08 lb/ft$^2$. This shear stress would likely cause scour of particles up to 2,000 µm in size for water having a low content of fine sediment. If the water had a high content of fine sediment, the maximum particles likely to be scoured from the sediment may be only about 100 µm in size. If the sediment was somewhat consolidated (as expected to occur during dry periods between runoff events), than the necessary shear stress to cause sediment scour would be substantially greater, as noted in the following discussion.

**Table 4. Calculated Shear Stress and Particle Scour (yellow high-lighted values associated with velocities of at least 3 ft/sec)**

| | | shear stress lb/ft$^2$), velocity (ft/sec), and discharge (ft$^3$/sec) for different slopes: | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Pipe Dia. (ft) | flow depth/pipe dia. ratio | shear at 0.001 slope | Max. particle size not scoured (µm) | velocity at 0.001 | discharge at 0.001 | shear at 0.005 slope | Max. particle size not scoured (µm) | velocity at 0.005 | discharge at 0.005 | shear at 0.01 slope | velocity at 0.01 | discharge a 0.01 |
| 1 | 0.1 | 0.0041 | <100 | 0.57 | 0.028 | 0.020 | <100 | 1.3 | 0.063 | 0.041 | 1.8 | 0.0 |
| 1 | 0.3 | 0.011 | <100 | 1.1 | 0.21 | 0.053 | 250 | 2.5 | 0.48 | 0.11 | 3.5 | 0. |
| 1 | 0.5 | 0.016 | <100 | 1.4 | 0.55 | 0.078 | 1,500 | 3.2 | 1.29 | 0.12 | 4.5 | |
| 1 | 0.8 | 0.019 | <100 | 1.6 | 1.2 | 0.094 | 2,800 | 3. 7 | 2.6 | 0.20 | 5.2 | |
| 1 | 1 | 0.016 | <100 | 1.4 | 1.1 | 0.078 | 1,500 | 3.2 | 2.5 | 0.12 | 4.5 | |
| 1.5 | 0.1 | 0.0061 | <100 | 0.75 | 0.083 | 0.030 | <100 | 1.7 | 0.19 | 0.061 | 2.4 | 0. |
| 1.5 | 0.3 | 0.016 | <100 | 1.5 | 0.63 | 0.080 | 1,500 | 3. 3 | 1.4 | 0.16 | 4.6 | |
| 1.5 | 0.5 | 0.023 | <100 | 1. 9 | 1.63 | 0.12 | 3,500 | 4.2 | 3.6 | 0.23 | 6.0 | |
| 1.5 | 0.8 | 0.028 | <100 | 2.1 | 3. 4 | 0.14 | 5,000 | 4.8 | 7.6 | 0.28 | 6. 8 | |
| 1.5 | 1 | 0.023 | <100 | 1. 9 | 3.3 | 0.12 | 3,500 | 4.2 | 7.4 | 0.23 | 6.0 | |
| 2 | 0.1 | 0.0081 | <100 | 0.91 | 0.18 | 0.041 | <100 | 2.03 | 0.40 | 0.081 | 2.9 | 0. |
| 2 | 0.3 | 0.021 | <100 | 1. 8 | 1.4 | 0.11 | 3,300 | 4.0 | 3.1 | 0.21 | 5.6 | |
| 2 | 0.5 | 0.031 | <100 | 2.3 | 3.5 | 0.16 | 4,500 | 5.1 | 7.98 | 0.31 | 7.2 | |
| 2 | 0.8 | 0.038 | <100 | 2.6 | 7.3 | 0.19 | >5,000 | 5.8 | 16 | 0.38 | 8.2 | |
| 2 | 1 | 0.031 | <100 | 2.3 | 7.2 | 0.16 | 4,500 | 5.1 | 16 | 0.31 | 7.2 | |
| 3 | 0.1 | 0.012 | <100 | 1.2 | 0.53 | 0.061 | 700 | 2. 7 | 1.2 | 0.12 | 3.8 | 1 |
| 3 | 0.3 | 0.032 | <100 | 2.3 | 4.0 | 0.16 | 5,000 | 5.2 | 9.0 | 0.32 | 7.4 | |
| 3 | 0.5 | 0.047 | <100 | 3.0 | 10 | 0.23 | >5,000 | 6.7 | 23 | 0.47 | 9.5 | |
| 3 | 0.8 | 0.057 | 450 | 3.4 | 22 | 0.28 | >5,000 | 7.7 | 48 | 0.57 | 11 | |
| 3 | 1 | 0.048 | <100 | 3.0 | 21 | 0.23 | >5,000 | 6.7 | 47 | 0.47 | 9. 5 | |
| 5 | 0.1 | 0.020 | <100 | 1.7 | 2.1 | 0.10 | 3,000 | 3.8 | 4.6 | 0.20 | 5.4 | |
| 5 | 0.3 | 0.053 | 400 | 3.3 | 16 | 0.27 | >5,000 | 7.4 | 35 | 0.53 | 10 | |
| 5 | 0.5 | 0.078 | 1,500 | 4.2 | 41 | 0.39 | >5,000 | 9.4 | 92 | 0.78 | 13 | 1 |
| 5 | 0.8 | 0.094 | 2,800 | 4.8 | 84 | 0.47 | >5,000 | 114 | 190 | 0.94 | 15 | 2 |
| 5 | 1 | 0.078 | 1,500 | 4.2 | 83 | 0.39 | >5,000 | 9.4 | 190 | 0.78 | 13 | 2 |

assuming n=0.013

water with low content of fine sediment in water (if high content, then require higher shear stress)

**Erodability of Previously Settled Material after Consolidation**

Figure 9 is an example of allowable shear stresses for a range of cohesive materials having a range of void ratios corresponding to varying amounts of consolidation. Again, the COE recommends that local field observation or laboratory testing results be given preference. If the void ratio was about 0.4, corresponding to a compact sediment, the shear stress would have to be greater than about 0.3 lb/ft$^2$ to affect particles larger than clays. This shear stress may only occur for larger and full-flowing pipes having 2% slopes (and about 7 ft/sec velocities).

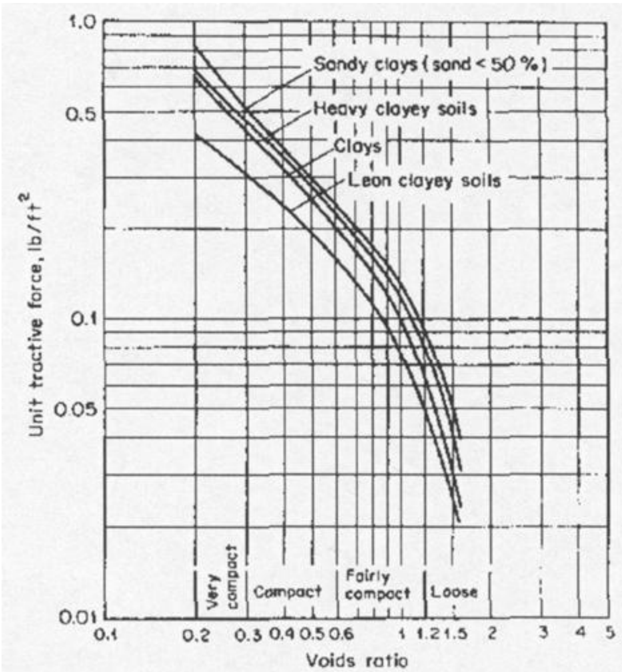


**Figure 9. Example of allowable shear stresses (tractive forces) for cohesive materials (COE 1994) (a Leon clayey soil is hardpan. Hardpan is a condition of the soil or subsoil in which the soil grains become cemented together by such bonding agents as iron oxide and calcium carbonate, forming a hard, impervious mass).**

## *Criteria to Ensure Self-Cleaning in Sewerage*

Pisano, *et al*. (1998) described the mechanisms associated with deposition and scour of sediment in sewerage. The following paragraphs are summarized from that report.

Not all of the particles of a given size at the flow/sediment boundary dislodge and move at the same time because the flow is turbulent and contains short term fluctuations in velocity. The limiting condition, below which sediment movement is negligible, known as the threshold of movement, is usually defined in terms of either the critical bed shear stress or the critical erosion velocity.

Once sediment is entrained in flowing water, it may travel in the sewer in one of two basic ways: 1) Finer, lighter material tends to travel in suspension, while 2) heavier material travels in a rolling, sliding mode as bedload. In the transport of suspended sediment, there is a continuous exchange between particles settling out and those being eroded and re-entrained back into the flow. Under certain conditions, fine grained and organic particles can form a highly concentrated mobile layer of “fluid mud” near the pipe invert (Ashley and Crabtree 1992).

If the flow velocity or turbulence level decreases, there will be a net reduction in the amount of sediment held in suspension. Below a certain limit, the sediment will form a bed, with transport occurring only in the surface layer. If the flow velocity is further decreased, sediment transport will cease completely. The flow conditions necessary to prevent deposition depend on the pipe size and on properties of the sediment, mostly particle size and specific gravity. Flow velocities needed to entrain sediment tend to be higher than those at which deposition occurs.

**Sewer Self Cleansing Criteria**

Average shear stress is an important parameter in the criteria for sewer self-cleansing. The average shear stress is the amount of force the fluid exerts on the wetted perimeter of the pipe. Another important parameter is the bed shear stress which is the amount of force the fluid exerts on the bed of sediment in the pipe. Bed shear stress is related to bed load scour and movement.

Historically, the design of drainage systems has included the prevention of deposition of sediments in pipes. These conditions are based on a minimum velocity of flow, or a minimum shear stress that the flow should exert on the walls of the pipe to maintain self-cleansing conditions. The minimum velocity of flow, or the minimum shear stress, corresponds to a particular depth of flow, or with a particular frequency of occurrence. Available design criteria were reported by the Construction Industry Research and Information Association (CIRIA), and a summary of these, outlined by Nalluri and Ab Ghani (1996), are shown in Table 5 and Table 6.

**Table 5. Minimum Velocity Criteria**

| Source | Country | Sewer Type | Minimum Velocity | | Pipe Conditions |
|---|---|---|---|---|---|
| | | | (m/s) | (ft/sec) | |
| American Society of Civil Engineers (1970) | USA | Sanitary | 0.6 | 2.0 | Full/Half-full |
| | | Storm | 0.9 | 3.0 | Full/Half-full |
| British Standard (1987) | UK | Sanitary | 1.0 | 3.3 | Full |
| | | Combined | 1.0 | 3.3 | Full |
| Bielecki (1982) | Germany | Not noted | 1.5 | 5.0 | Full |

**Table 6. Minimum Shear Stress Criteria**

| Source | Country | Sewer Type | Minimum Shear Stress | | Pipe Conditions |
|---|---|---|---|---|---|
| | | | ($N/m^2$) | ($lb/ft^2$) | |
| Lysne(1969) | USA | Not noted | 2-4 | 0.04 – 0.08 | Not noted |
| ASCE(1970) | USA | Not noted | 1.3-12.6 | 0.027 – 0.26 | Not noted |
| Yao (1974) | USA | Storm | 3.0-4.0 | 0.061 – 0.083 | Not noted |
| | | Sanitary | 1.0-2.0 | 0.021 – 0.042 | Not noted |
| Maguire | UK | Not noted | 6.2 | 0.13 | Full/Half-full |
| Bischof (1976) | Germany | Not noted | 2.5 | 0.052 | Not noted |

These criteria take no account of the characteristics of the sediment, of the suspended sediment concentration, the bed load, or of any cohesion between the sediment particles. Nonetheless, minimum velocity and minimum shear stress levels of 1 m/s (3.3 ft/sec) and 2 $N/m^2$ (0.04 $lb/ft^2$) are commonly accepted criteria. However, recent work by many researchers has shown that a single value of minimum velocity or shear stress cannot adequately describe the self cleansing conditions in all pipes of different size, roughness and gradient for a range of sediment characteristics and flow conditions.

In practice, sewer pipes will not be maintained self-cleansing at all times. The diurnal pattern of the dry weather flow and the temporal distribution and nature of sediments found in sewer flows may result in the deposition of some sediments at times of low flow and the subsequent erosion and transport of these sediments, either as suspended load or bed-load, at times of higher flow. The deposited sediments will exhibit additional strength due to cohesion and, provided that the peak dry weather flow velocity or bed shear stress is of sufficient magnitude to erode these sediments, the sewer will maintain self cleansing operation at times of dry weather. May, *et al.* (1996) presented a definition to describe a self cleansing sewer as “an efficient self-cleansing sewer is one having a sediment-transporting capacity that is sufficient to maintain a balance between the amounts of deposition and erosion, with a time-averaged depth of sediment deposit that minimizes the combined costs of construction, operation and maintenance.” To achieve such self-cleansing performance, the following criteria apply:

1. Flows equaling or exceeding a limit appropriate to the sewer should have the capacity to transport a minimum concentration of fine-grain particles in suspension (applicable for all types of sewerage systems).

2. The capacity of flows to transport coarser granular material as bed-load should be sufficient to limit the depth of deposition to a specified proportion of the pipe diameter. This criteria generally relates to combined and stormwater systems. Limit of deposition considerations, i.e., “no deposition” generally applies to sanitary sewer designs. In this context, there must be sufficient shear in sanitary systems to avoid deposition of large particles.

3. Flows with a specified frequency of occurrence should have the ability to erode bed particles from a deposited granular bed that may have developed a certain degree of cohesive strength (applicable to all systems).

To meet these criteria, new guidelines have been developed by CIRIA (Ackers, *et al*. 1996), and are being adopted throughout Europe for the design of sewers to control sediment problems. Design criteria for the transport of fine grained material in suspension, the transport of coarser sediments as bed load, and the erosion of cohesive sediment deposits and guidelines on the minimum flow velocity and pipe gradient for different types and sizes of sewer are outlined. To account for the effects of cohesion (Criterion 3), the design flow condition should produce a minimum value of bed shear stress of 2.0 $N/m^2$ (0.04 $lb/ft^2$) on a flat bed for a rough concrete pipe (Ackers, *et al*. 1996).

The third criterion is of specific interest to the problem of cleansing accumulated mature sediment beds. Various researchers have studied the flow conditions required to release particles from a deposited bed, which has developed a degree of cohesion. Summaries of investigations forming much of the basis for Criterion 3 are as follows:

- Nalluri and Alvarez (1992), whose laboratory studies used synthetic cohesive sediments, concluded that there were two ranges of bed shear stress at which erosion occurred: 2.5 $N/m^2$ (0.05 $lb/ft^2$) applying for the weakest material, comprising a surface layer of fluid sediment, and 6 to 7 $N/m^2$ (0.12 to 0.14 $lb/ft^2$) for the more granular and consolidated material below. It was found that, after erosion, the synthetic cohesive sediments behaved very much like non-cohesive material.

- Ristenpart and Uhl (1993) found in field tests that during dry weather an average bed shear stress of 0.7 $N/m^2$ (0.015 $lb/ft^2$) was required to initiate erosion, increasing to an average of about 2.3 $N/m^2$ (0.05 $lb/ft^2$) during wet weather, or to 3.3 $N/m^2$ (0.068 $lb/ft^2$) after a prolonged period of dry weather and presumably, consolidation of the deposited bed.

- Ashley and Crabtree (1992) suggested that the bonds between particles at the surface of a deposited bed are weakened by the presence of the water, so that surface layers can be successively stripped away by the flow. Measurements in the Dundee, Scotland, sewers indicated that it began to move at a fluid shear stress of about 1 $N/m^2$ (0.02 $lb/ft^2$), with significant erosion of a deposited bed occurring at bed shear of 2 to 3 $N/m^2$ (0.04 to 0.06 $lb/ft^2$). Taking account of a review of work by other researchers, Ashley and Crabtree concluded that most deposits should be eroded at a shear stress exceeding 6 to 7 $N/m^2$ (0.12 to 0.14 $lb/ft^2$).

Pisano, *et al.* (1998) conducted a “desktop” analysis of the sedimentation and scour problems associated with the 3.05 m (10 ft) South Ottawa Tunnel, in Ottawa, Canada. This procedure analyses deposition on a daily basis, determining accumulations of inorganic and organic deposits as well as scour and erosion of prior deposites over a year’s period. The particles in this combined sewerage had settling velocities ranging from 0.1 to 30 cm/sec, corresponding to 75 µm fine sand to 3/8 inch gravel. If the average shear stress is smaller than the critical shear stress for that sieve size and density, deposition will occur. Erosion of the bed load is based on maximum hourly shear stress. If the maximum shear stress is great than 2 N/m$^2$, then erosion of the bed load occurs. Table 7 shows the full range of depositon and scour that may occur for this large tunnel for a variety of discharge, velocity, and shear stress conditons.

**Table 7. Assessment of 3.05 Meter Tunnel (Pisano, *et al.* 1998)**

| Discharge | | Velocity | | Fluid shear | | Deposition or erosion potential |
|---|---|---|---|---|---|---|
| (m$^3$/sec) | (ft$^3$/sec) | (m/sec) | (ft/sec) | (N/m$^2$) | (lb/ft$^2$) | |
| 2.6 | 93 | 0.36 | 1.2 | 0.27 | 0.0056 | Severe deposition |
| 3.5 | 130 | 0.47 | 1.5 | 0.48 | 0.0099 | Moderate to severe deposition |
| 4.4 | 160 | 0.60 | 2.0 | 0.74 | 0.015 | Mild to moderate deposition |
| 5.3 | 190 | 0.72 | 2.4 | 1.04 | 0.021 | Slight to mild deposition |
| 6.1 | 220 | 0.84 | 2.8 | 1.43 | 0.029 | Skims juvenile sediments |
| 7.0 | 250 | 1.06 | 3.5 | 1.86 | 0.038 | None to slight erosion top layer |
| 8.8 | 320 | 1.20 | 4.0 | 2.85 | 0.059 | Slight to mild erosion of consolidated beds (2-5%) |
| 10.5 | 380 | 1.44 | 4.7 | 4.15 | 0.085 | Mild erosion of consolidated beds (5-15%) |
| 13.2 | 470 | 1.80 | 5.9 | 6.47 | 0.13 | Moderate erosion of consolidated beds (15-25%) |
| 15.8 | 570 | 2.17 | 7.2 | 9.3 | 0.19 | Substantial erosion (25-35%) |
| 17.5 | 630 | 2.41 | 7.9 | 11.5 | 0.24 | Substantial erosion (35-50%) |

Note: $K_b$ = 1.2 millimeters = rough concrete

## Accumulation of Sediment in Bellevue Inlet Structures and Storm Drainage System

An important part of the Bellevue NURP project was the measurement of the sediment accumulating in the inlet structures (Pitt 1985). The storm drainage system inlets were cleaned and surveyed at the beginning of the project. The 207 inlet structures were then surveyed nine times over two years to determine the depth of accumulating material (from December 1979 through January 1981). The first year rate of accumulation was relatively steady (based on 3 observation periods), while the sediment loading remained almost constant during the second year. During the second year, there was about twice as much contaminated sediment in the separate storm drainage system at any one time as there was on the streets. The scouring of the sewerage sediments out of the drainage systems was not found to be significant during the project period. There was a period of heavy rains in October of 1981 (about 100 mm of rain during a week, very large for Bellevue) during the second year when the accumulated material did not decrease, based on observations made before and after the rain (August 1981 and January 1982). The lack of sediment movement from catchbasin sumps was also observed during earlier tests conducted in San Jose by Pitt (1979). During that study, an idealized catchbasin and sump were constructed based on Lager, *et al.* (1974) and was filled with clean material having the same particle sizes as typical sump material, along with fluorencent tracer beads. During a year, freezing core samples were obtained and the sediment layers were studied to determine any flushing and new accumulations of material. The sediment material was found to be very stable, except for a very thin surface layer.

The first year accumulation rates (L/month per inlet) ranged from 1.4 in Lake Hills to 4.8 in Surrey Downs, as shown on Table 8. The catchbasins and inlets had sumps (the catchbasin sumps were somewhat larger), while the manholes were much larger, with more volume available for accumulation sediment. The stable volume that occurred during the second year was about 60% of the total storage volumes of the catchbasins and inlets (sump volume below the outlet pipe). If the sumps were very shallow, the maximum sediment depth was only about 12 mm, while the deeper sumps had about 150 mm of accumulated sediment. Individual inlet structures had widely varying depths, but the depth below the outlet appeared to the most significant factor affecting the maximum sump volume available. This “scour” depth generally was about 300 mm. If the sumps were deeper, they generally were able to hold more sediment before their equilibrium depth was reached and would therefore would require less frequent maintenance. About 100 L/ha/yr accumulated in Surrey Downs, while only about 2/3 of this value accumulated in Lake Hills. Nine of the most heavily loaded catchbasins in the first summer inventory in Surrey Downs were located very near two streets that did not have curbs and had extensive nearby sediment sources (eroding hillsides). These few catchbasins (about 10% of the total catchbasins) accounted for more than half of the total Surrey Downs sediment observed during that survey. They also represented about 70% of the observed increased loadings between the first winter and summer inventories.

**Table 8. Accumulation Rate of Sediment in Inlet Structures in Bellevue, WA (Pitt 1985)**

| | **Number of structures** | | **Sediment accumulation (L/month)** | | **Approx. months to stable volume** | **Stable volume (L)** | |
|---|---|---|---|---|---|---|---|
| | **total** | **per ha** | **per ha** | **per unit** | | **per ha** | **per unit** |
| Surey Downs (38.0 ha) | | | | | | | |
| Catchbasins | 43 | 1.1 | 5.3 | 4.8 | 13 | 68 | 62 |
| Inlets | 27 | 0.7 | 2.0 | 2.8 | 20 | 40 | 57 |
| Manholes | 6 | 0.2 | 0.8 | 4.0 | 19 | 15 | 76 |
| Average | 76 total | 2.0 total | 8.1 | 4.2 | 15 | 123 total | 62 |
| Lake Hills (40.7 ha) | | | | | | | |
| Catchbasins | 71 | 1.7 | 2.4 | 1.4 | 18 | 43 | 25 |
| Inlets | 45 | 1.1 | 1.5 | 1.4 | 14 | 22 | 20 |
| Manholes | 15 | 0.4 | 1.6 | 4.0 | 23 | 36 | 90 |
| Average | 131 total | 3.2 total | 5.5 | 1.7 | 18 | 101 total | 31 |

Besides inlet sediment surveys, pipe surveys were also conducted during the study. Very few storm drain pipes in either test area had slopes less than one percent, the assumed critical slope for sediment accumulation. In Lake Hills, the average slope of the 118 pipes surveyed was about 4 percent. Only 7 percent of the Lake Hills pipes had slopes less than 1 percent. The 75 pipes surveyed in Surrey Downs had an average slope of 5 percent, and 12 percent had slopes less than 1 percent. A pipe sediment survey was conducted in October of 1980. Very little sediment was found in the storm drains in either study area. The pipes that had significant sediment were either sloped less than 1-1/2 percent, or located close to a source of sediment. The characteristics of the pipe sediments were similar to the characteristics of the sediment from close by inlets and catchbasins, indicating a common source, and the eventual movement of the inlet sediments. The volume of sediment found in the Lake Hills pipes was about 1-1/2 m$^3$, or about 0.04 m$^3$ per ha, or about 40% of the total sediment in the inlet structures (about 0.1 m$^3$ per ha stable volume). This was equivalent to about 70 kg of sediment/ha. In Surrey Downs, much more sediment was found in the storm drainage: more than 20 m$^3$ of sediment was found in the pipes, or about 0.5 m$^3$/ha or 1,000 kg/ha. Most of this sediment was located in silted-up pipes along 108$^{th}$ St. and Westwood Homes Rd. which were not swept and were close to major sediment sources.

The chemical quality of the captured sediment was also monitored. Tables 9 and 10 show the sediment quality for Surrey Downs inlet structures sampled between January 13 and June 17, 1981. The sediment quality shown on this table is very similar to the street dirt chemical quality that was simultaneously sampled and analyzed. It is interesting to note that the COD values increase with increasing particle sizes, likely corresponding to increasing amounts of organic material in the larger material. The nutrients are generally constant with size, while the metal concentrations are much higher for the smaller particles, as expected for street dirt. As indicated on the table, the lead values were likely much higher when these samples were taken compared to current conditions. Current outfall lead concentrations are now about 1/10 of the values they were in the early 1980s.

**Table 9. Chemical Quality of Bellevue, WA, Inlet Structure Sediment (mg constituent/kg total solids) (Pitt 1985)**

| Particle Size (μm) | COD | TKN | TP | Pb* | Zn |
|---|---|---|---|---|---|
| <63 | 160,000 | 2,900 | 880 | 1,200 | 400 |
| 61-125 | 130,000 | 2,100 | 690 | 870 | 320 |
| 125-250 | 92,000 | 1,500 | 630 | 620 | 200 |
| 250-500 | 100,000 | 1,600 | 610 | 560 | 200 |
| 500-1,000 | 140,000 | 1,600 | 550 | 540 | 200 |
| 1,000-2,000 | 250,000 | 2,600 | 930 | 540 | 230 |
| 2,000-6,350 | 270,000 | 2,500 | 1,100 | 480 | 190 |
| >6,350 | 240,000 | 2,100 | 760 | 290 | 150 |

* these lead values are much higher than would be found for current samples due to the decreased use of leaded gasoline since 1981.

**Table 10. Annual Calculated Accumulation of Pollutants in Bellevue, WA, Inlet Structures (Pitt 1985)**

| | Solids L/ha/yr | Solids kg/ha/yr | COD kg/ha/yr | TKN kg/ha/yr | TP kg/ha/yr | Pb kg/ha/yr | Zn kg/ha/yr |
|---|---|---|---|---|---|---|---|
| Surrey Downs | 96 | 147 | 37 | 0.17 | 0.25 | 0.49 | 0.10 |
| Lake Hills | 66 | 100 | 7.5 | 0.07 | 0.07 | 0.07 | 0.02 |

These accumulations are substantial. As an example, the total suspended solids discharged at the outfalls was about 100 to 200 kg/ha/yr, with the amount accumulating in the catchbasin inlets about the same quantity.

## Conclusions and Recommendations

To achieve self-clean in a drainage system, Pisano, *et al.* (1998) listed the following criteria:

1. Flows equaling or exceeding a limit appropriate to the sewer should have the capacity to transport a minimum concentration of fine-grain particles in suspension (applicable for all types of sewerage systems).

2. The capacity of flows to transport coarser granular material as bed-load should be sufficient to limit the depth of deposition to a specified proportion of the pipe diameter. This criteria generally relates to combined and stormwater systems. Limit of deposition considerations, i.e., "no deposition" generally applies to sanitary sewer designs. In this context, there must be sufficient shear in sanitary systems to avoid deposition of large particles.

3. Flows with a specified frequency of occurrence should have the ability to erode bed particles from a deposited granular bed that may have developed a certain degree of cohesive strength (applicable to all systems).

Ashley and Crabtree (1992) made sediment movement measurements in the Dundee, Scotland, sewers and found that sediment began to move at a fluid shear stress of about 1 N/m$^2$ (0.02 lb/ft$^2$), with significant erosion of a deposited bed occurring at bed shear of 2 to 3 N/m$^2$ (0.04 to 0.06 lb/ft$^2$). Taking account of a review of work by other researchers, they concluded that most deposits should be eroded at a shear stress exceeding 6 to 7 N/m$^2$ (0.12 to 0.14 lb/ft$^2$). This is likely to occur for most flow conditions in pipes at least 2 ft in diameter at a slope greater than 0.5%.

For the smallest slopes (<0.5% slopes), almost all settled particles would likely remain and not be scoured, while almost all unconsolidated sediments would be scoured for pipe greater than 1% in slope. Pipe conditions resulting in at least 3 ft/sec stormwater velocities would also have shear stresses of at least 4 N/m$^2$ (0.08 lb/ft$^2$). This shear stress would likely cause scour of particles up to 2,000 µm in size for water having a low content of fine sediment. If the water had a high content of fine sediment, the maximum particles likely to be scoured from the sediment may be only about 100 µm in size.

**Required Sample Line Velocities to Minimize Particle Sampling Errors**

The collection of stormwater samples representing the complete range of likely particle sizes can be challenging. Manual samples can be obtained as the flowing water drops into an inlet, or as the water cascades out of an outfall. These samples, if frequent during an event, should well represent the complete particle size distribution. However, if an automatic sampler using a pump system is used, there may be some losses of particles during the sampling operation. Typical sample lines are Teflon™ lined polyethylene and are 10 mm in diameter. The water velocity in the sample line is about 100 cm per second, enabling almost complete sampling of a wide range of particle sizes. However, bed load sampling equipment is usually needed to adequately collect samples of bed load at outfalls. Table 11 shows the particle sizes that would be lost in vertical sampling lines at a pumping rate of 30 and 100 cm/second (Burton and Pitt 2001).

**Table 11. Losses of Particles in Sampling Lines (Burton and Pitt 2001)**

| | 30 cm/sec flow rate | | 100 cm/sec flow rate | |
|---|---|---|---|---|
| | Critical settling rate (cm/sec) | Size range (µm, for ρ = 1.5 to 2.65 g/cm$^3$) | Critical settling rate (cm/sec) | Size range (µm, for ρ = 1.5 to 2.65 g/cm$^3$) |
| 100% loss | 30 | 2,000 - 5,000 | 100 | 8,000 - 25,000 |
| 50% loss | 15 | 800 - 1,500 | 50 | 3,000 - 10,000 |
| 25% loss | 7.5 | 300 - 800 | 25 | 1,500 - 3,000 |
| 10% loss | 3.7 | 200 - 300 | 10 | 350 - 900 |
| 1% loss | 0.37 | 50 - 150 | 1 | 100 - 200 |

A water velocity of 100 cm/sec (about 3 ft/sec) would result in very little loss of stormwater particles. Particles of 8 to 25 mm would not be lifted in the sample line at all at this velocity, but these sized particles would not fit through the openings of the intake or even fit in most sample lines. They are also not likely to be present in stormwater, but may be a component of bedload in a stream, or gravel in the bottom of a stormdrain pipe, requiring special sampling. Very few particles larger than several hundred micrometers occur in stormwater and these should only have a loss rate of 10% at the most. Most particles in stormwater are between 1 and 100 µm in diameter and have a density of between 1.5 and 2.65 g/cm$^3$. Even at 30 cm/sec, these particles should experience insignificant losses. A pumping rate of about 100 cm/sec would add extra confidence in minimizing particle losses. ASTM (1995) in method D 4411 recommends that the sample velocity in the sampler line be at least 17 times the fall rate of the largest particle of interest. As an example, for the 100 cm/sec example above, the ASTM recommended critical fall rate would be about 6 cm/sec, enabling a particle of several hundred micrometers in diameter to be sampled with a loss rate of less than 10%. This is certainly adequate for most stormwater sampling needs. Bed loads comprised of particles of up to several mm can account for about 5 to 10% of the sediment load under some conditions, and special sampling would be needed to collect these larger particulates. In many cases, this larger material would not move through the pipe, but accumulate as a semi-permanent sediment deposit.

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