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Critical appraisal of piping phenomena in earth dams

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Abstract This paper presents a comprehensive review of published literature on soil piping phenomena. The first tools to design earth dams to resist piping were developed during 1910–1935. Filter criteria for dispersive soils was refined in the 1970's. Piping phenomena are generally defined as: (1) heave, (2) internal erosion, (3) backwards erosion, although other modes are possible. Recent work on piping highlights the limitations of the occurrence of piping and the role that design and construction may play in a large percentage of piping failures. Standardized laboratory procedures are available to assess piping potential in cohesive materials, but no such methods exist for non-cohesive soils. However, methods are available for evaluation of self-filtration potential. Recent advances in computer technology have facilitated the evaluation of seepage and deformation in embankments but computational methods for evaluation of piping potential are currently limited.

Keywords Earth dams · Piping · Internal erosion · Heave · Suffosion

Résumé L'article présente une analyse générale de résultats publiés sur les phénomènes de suffosion des sols. Les premiers outils visant à dimensionner les barrages en terre contre la suffosion ont été développés durant la période 1910–1935. Les critères de filtres destinés à éviter ces phénomènes ont été améliorés dans les années 1970. Les phénomènes de suffosion sont généralement associés à (1) du gonflement, (2) de l'érosion interne, (3) de l'érosion régressive, mais d'autres processus sont possibles. Des travaux récents sur la suffosion mettent en lumière les techniques permettant de limiter les risques d'apparition de ce phénomène. Ils montrent aussi le rôle des principes de conception et des techniques de construction sur beaucoup de situations de rupture initiées par des phénomènes de suffosion. Des essais de laboratoire standardisés existent pour évaluer la susceptibilité à la suffosion de sols cohérents, ce qui n'est pas le cas pour des sols non cohérents. Cependant, des méthodes sont disponibles pour évaluer la capacité d'auto-filtration d'un matériau donné. Des avancées récentes dans le domaine de la simulation numérique ont facilité l'évaluation des ècoulements et des déformations dans les structures de barrage en terre, mais il faut noter que ces méthodes numériques restent impuissantes pour mettre en évidence les conditions d'apparition de la suffosion.

Mots clés Barrages en terre · Suffosion · Erosion interne · Gonflement

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Introduction

Based on the history of earth dam failures in the nineteenth and twentieth centuries, it is likely that piping failures in dams have occurred since the earliest dams were constructed around 2900 BC. Early methods of construction did not consider the effects of seepage or proper zonation of materials to provide adequate filters in earth dams. As experience grew with the successful construction of dams on a variety of foundation materials, empirically successful dam designs emerged by the first millennium AD as evidenced by the 2,000 year service life of the Proserpina Dam constructed by the Romans (Jansen 1983, p. 16). Shortly after Henry Darcy (1856) recognized the relationship between head, length of flow path and fluid velocities in granular media, methods were developed to evaluate piping potential from the length of flow path under dams (Bligh 1910, 1911a, b, 1913; Lane 1934). With the advent of modern soil mechanics, Terzaghi (1922, 1925) developed a method for the evaluation of heave. Methods were also developed to aid in the design of defensive piping measures in the form of filters (Terzaghi 1922, 1939, 1943; Terzaghi and Peck 1948; Bertram 1940; Sherman 1953; Sherard and Dunnigan 1989), and evaluation of hydraulic gradients for design (Forcheimer 1886; Casagrande 1937; Cedergren 1977). This paper provides a critical review of piping literature and methods currently available for the evaluation of piping- related problems in existing dams.

Definitions

Due to the large body of work that has been completed on piping research, and the fact that the work is the product of international and multi-discipline study, there are a number of definitions in the literature regarding piping phenomena. It has been common for practicing engineers to lump all these definitions under the generic term "piping". This practice makes it difficult to determine the root cause of piping failures when researching case histories. For clarity, definitions are necessary before proceeding with this review.

(a) Terzaghi (1939), Lane (1934), and Sherard et al. (1963, p. 115) present a model of piping in which particles are progressively dislodged from the soil matrix through tractive forces produced by intergranular seeping water. The mobilizing tractive forces are balanced by the shear resistance of grains, weight of the soil particles and filtration. The erosive forces are greatest where flow concentrates at an exit point and once soil particles are removed by erosion the magnitude of the erosive forces increases due to the increased concentration of flow. This view of piping is the classic backwards-erosion style of piping. "Backwards erosion" is generally produced where a roof of competent soil or some other structure allows the formation of a bridged opening. The tractive force causing this type of erosion is directly proportional to the velocity of intergranular flow. Lutz (1934) presented a model where the physico-chemical properties of soils are the primary factor in soil erosion. In the case of "backwards erosion" piping, resistance to the removal of grains of soil is dependent on the hydraulic gradient through the soil (which is required to overcome this resistance) as well as the state of stresses around the opening being formed by the erosion of material.

(b) "Internal erosion" (as used here) is similar to backwards erosion piping in that tractive forces remove soil particles. However, internal erosion is due to flow along pre-existing openings such as cracks in cohesive material or voids along a soil-structure contact. By this definition, internal erosion is not due to the dynamics of intergranular flow and the hydraulics of the problem are quite different than for backwards erosion (Lane 1934). Rather than being initiated by Darcian flow at an exit point, internal erosion is initiated by erosive forces of water along a pre-existing planar opening. Hence, it may be expected that internal erosion would initiate in accordance with the cubic law of flow for planar openings (Franco and Bagtzoglou 2002; Louis 1969; Worman and Olafsdottir 1992). Internal erosion is expressed as a tractive load along the length of the opening. This contrasts to the case of backwards erosion, where the erosion is occurring at the exit point. Due to the nature of hydraulic conductivity at a soil-structure boundary, fluid velocities may be more erosive for a given hydraulic gradient due to higher velocity flows. Also, as the hydraulic conductivity tends to be slightly greater at a soil structure contact, this is often the first place that increasing hydraulic gradients may express themselves through erosion.

(c) "Tunneling" or "jugging" are common features observed in dispersive soils caused by rainfall erosion and is discussed by a number of Australian and New Zealand workers (Jones 1981). Tunneling occurs within the vadose zone and is due to chemical dispersion of clay soils from rainwater passing through open cracks or natural conduits. By definition, tunneling does not occur in the phreatic zone. In extreme cases, tunneling or jugging in dispersive soils can lead to dam failures similar to the tunneling activities of animals or penetration by tree roots.

(d) East European researchers coined the term "suffosion" (Jones 1981; Burenkova 1993; Pavlov 1898; Kral 1975; Galarowski 1976) to describe the gradual migration of fine materials through a coarse matrix leading to failure (McCook 2004; Kovaks 1981; Kral 1975; Galarowski 1976; amongst others). This process can result in a loose framework of granular material with relatively high seepage flows that leads to collapse of the soil skeleton (McCook 2004). In non-cohesive materials, suffosion leads to zones of high permeability (and water transmission), potential outbreaks of increased seepage, increased erosive forces and potential collapse of the skeletal soil structure. Suffosion can be a much slower process than is commonly observed where piping occurs along a concentrated leak. Hence, suffosion may be related to long-term seepage problems that exhibit increasing seepage quantities over a period of years. Gradual loss of finer matrix materials in a soil supported by a coarser grained skeleton is termed suffusion, which may lead to a more general collapse and loss of soil structure, termed suffosion (Kézdi 1979; McCook 2004).

(e) The phenomenon of “heave” was discussed by Karl Terzaghi (1922, 1943), who developed an equation for assessing heave in sheet pile cofferdams. As reported by Terzaghi, heave occurs when a semi-permeable barrier overlies a pervious zone under relatively high fluid pressures. One significant aspect of heave is if the fluid pressures in the pervious zone increase, such as during a flood, a point may be reached where the uplift at the base of the semi-permeable barrier exceeds the vertical effective stress of the overlying barrier. This form of failure occurs at a hydraulic boundary where the migration of water through the barrier is at a slower rate than the rate of increase in pressure due to some transient event. This mode of failure is somewhat different from piping by backwards erosion or internal erosion. Failure by either of these methods requires seepage velocity sufficient to remove individual soil particles and is not as dependent on the stability of the soil mass.

From the above definitions, it can be seen that quite different mechanisms are at work for the various modes of piping and that engineering analysis specific to each is required to assess these different modes of piping.

Statistics of piping failures

Although Sherard et al. (1963, p. 124) indicate that most serious problems from piping are the result of progressive backward erosion of concentrated leaks, the historic record of dam failures due to piping indicates there may be other factors involved. Review of the statistics of dam failures has shown that a very large percentage of piping cases are due to internal erosion, inadequate filter design or improper maintenance. If conduits were properly designed or avoided altogether, the number of piping failures would drop significantly. Piping data taken from Jones (1981) and Lane (1934) is summarized in Table 1, with the addition of more recent data taken from the National Performance of Dams Program (http://www.npdp.stanford.edu/index.html). Based on this accumulation of 267 dam piping failures, the piping cases have been divided into four broad categories;

(I) Foundation related piping failures.
(II) Conduit and internal erosion piping failures.
(III) Possible backwards erosion and suffosion piping failures.
(IV) Piping failures induced by biological activity.

As can be seen from the examples of Category III piping failures shown in Table 1, nearly one-third (31.1%) of all piping failures could be associated with the classic backwards erosion model of piping or suffosion. However, it is quite possible that if more specifics were known about these failures the percentage could be significantly lower. Unfortunately, when dams fail by piping the evidence is often washed away with the dam. Hence, the statistics are very rough and some engineering judgement is necessary to classify piping failures. The majority of piping failures may be attributed to a variety of other causes, such as piping along conduits, other structures and internal erosion (49.8%) into or along foundations or abutments (15%), or piping due to biological activity (4.1%). It is interesting to note that the dams that failed by biological activity are commonly less than 9 m in height and that failure by piping into the foundation tends to occur in large dams.

von Thun’s 1985 report indicates that up to 26% of piping failures could be attributed to poor filter design (Bonala and Reddi 1998). Foster et al. (2000a, p. 1005) summarized the failure statistics of dams taken from ICOLD and other studies and found 46% of all dam failures can be attributed to some form of piping. A breakdown on specific modes of piping revealed that 30.5% of all dam failures were due to piping through the embankment, 14.8% were due to piping through the foundation and 1.6% were from piping into the foundation. They reported 35% of piping failures through the embankment occurred more than 5 years after first filling and 59% occurred during the first 5 years (Foster et al. 2000a, p. 1017). Of the incidents of dam failure due to piping through the embankment, they found 46% were due to piping at or near a conduit or other structure.

Cedergren (1977, p. 8) classified seepage failures into two categories; (1) failures caused by migration of particles to free exits or into coarse openings, (2) failures caused by uncontrolled saturation and seepage forces. Any opening, such as large pores in gravels or cobbles, open joints in rock, cracks caused by earthquakes, biological activity, or deteriorated/broken conduits may result in migration of particles. Other common causes of piping are due to poor construction of the embankment, poor compaction adjacent to outlet pipes or other structures, poor treatment of the foundation, from settlement cracks in the embankment, or conduits (Sherard et al. 1963, p. 124, 126; von Thun 1985).

Table 1 Piping failure case histories (after Jones 1981, with recent data added from the NPDP generated list of piping dam failures)

	Dam and location	Dates				
		Built	Failed	Height m (ft)	Type	Reason
I. Foundation related piping failures						
1	Allen, LA	–	1981	–	–	Piping at foundation
2	Ashley Dam, MA	–	1909	–	–	Piping within foundation
3	Ashti, India	–	1883	17.7 (58)	Rolled Earth	Foundation seepage
4	Avalon, NM	1893	1904	17.7 (58)	Earth and Rock	Piping into rock
5	Bad Axe Structure No. 12, WI	–	1978	22.2 (73)	–	Piping into abutment foundation joints (stress relief cracks) dam failed when pool reached new high
6	Baldwin Hills Reservoir Dam, CA	–	1963	48.8 (160)	Rolled Earth	Piping into foundation from fault movement
7	Black rock, NM	–	1909	3.0 (9.9)	–	Piping under lava rock in abutment
8	Blandon, MN	–	1948	6.4 (21)	Timber Crib	Piping of foundation
9	Brooklyn, NY	1893	1893	–	–	Foundation seepage
10	Cedar Lake, OK	–	1986	13.7 (45)	–	Piping in abutment, fissures in foundation
11	Clear Creek No. 2, AZ	–	1980	3.6 (12)	–	Piping due to natural springs in abutment rock, failed during construction
12	Coon Rapids, MN	–	1917	–	–	Foundation Piping under dam
13	D.T. Anderson, CO	–	1974	–	–	Piping at foundation
14	Dalton, NY	–	1912	8.8 (29)	Earth and concrete core	Foundation piping
15	Dansville Reservoir Dam, NY	–	1909	16.5 (54)	–	Foundation piping—undermining
16	Green Lick Run, PA	–	1904	–	–	Piping through and along abutment
17	Jennings Creek No. 3, TN	–	1963	–	–	Foundation piping (possible karst?)
18	LaFruta, TX	1930	1930	18.6 (61)	Rolled Earth	Foundation piping
19	Fairview, MA	–	1922	9.1 (30)	Rolled Earth	Foundation piping undermining
20	Lake Lanier Dam, SC	–	1926	16.8 (55)	–	Abutment undermining
21	Lake Toxaway, NC	1902	1916	18.9 (62)	Rolled Earth	Piping into rock fissures
22	Lake Seneca Dam, OH	–	1974	9.8 (32)	–	Piping in foundation through cutoff
23	Lebanon, PA		1893	12.2 (40)	Rolled Earth	Piping along foundation
24	Long Tom, ID	1906	1915	15.2 (50)	Earth and Puddle Core	Piping to outlet tunnel in rock abutment
25	Narragninep, CO	1908	1928	24.1 (79)	Rolled Earth	Abutment leakage
26	Martin Plant, FL	–	1979	–	–	Piping in foundation soils
27	Mellville, UT	1907	1909	11.0 (36)	Rolled Earth	Piping through foundation
28	Newton Gulch, CO	–	1973	12.8 (42)	–	Piping through foundation
29	Pleasant Valley, UT		1928	19.2 (63)	Earth and Rock	Piping through settlement cracks
30	Port Angeles, OR	–	1912	–	–	Piping of foundation
31	Quail Creek, UT	–	1988	63.7 (209)	–	Piping through foundation and up an observation pipe, failure occurred within 24 h of first notice
32	Schoefield, UT	1926	1927	18.9 (62)	Earth and Rock	Piping into rock/transverse crack
33	Stockton Creek, CA	1950	1950	24.4 (80)	Rolled Earth	Piping along abutment
34	Swift No. 2, WA	–	2002	25.3 (83)	–	Piping into rock foundation
35	Teton Dam, ID	1975	1976	93.0 (305)	Rolled Earth	Piping through abutment
36	Troy, ID	–	1995	13.1 (43)	–	Piping "blowout" of right abutment
37	Vaughn Creek, NC	–	1926	–	–	Piping into foundation abutment
38	Wishkah Reservoir No. 2 Dam, WA	–	1997	14.3 (47)	–	Piping, sinkhole in abutment and hole under timber crib dam

Table 1 continued

	Dam and location	Dates		Height m (ft)	Type	Reason
		Built	Failed			
39	Wesley Raley	–	1978	–	–	Piping through foundation
40	Zuni, NM	1907	1909	21.3 (70)	Earth and Rock	Piping through abutment
II. Conduit and internal erosion piping failures (includes piping induced by conduit failure, and structure contact piping)						
1	Ansonia Canal, CT	–	–	5.5 (18)+	–	Line of creep failure
2	Apishapa, CO	1920	1923	35.0 (115)	Rolled Earth	Piping through settlement cracks
3	Alto Pass Reservoir Dam, IL	–	1965	13.7 (45)	–	Piping along CMP conduit on first filling
4	Anita Dam, MT	–	1997	–	–	Piping along outlet pipe upon first filling
5	Annapolis Mall Swim Pond, MD	–	1993	9.1 (30)	–	Piping of fill along CMP during heavy rainfall
6	Ansonia, CT	–	1894	–	Rolled Earth	Piping along outlet
7	Arrowhead Lake, PA	–	1994	5.5 (18)	–	Piping under spillway, occurred during repair to apron
8	Arrowhead Lake (Stone) Dam, NC	–	2002	7.0 (23)	–	Piping under spillway slab
9	Baker Pond, VT	–	1956	5.5 (18)	–	Piping at pipe spillway
10	Beaver Brook, CT	–	1894	–	–	Piping along outlet
11	Beaver Lake Dam, IL	–	1983	3.0 (10)	–	Piping along spillway pipe
12	Beech Lake Dam, NC	–	2002	6.7 (22)	–	Piping, ruptured pressurized conduit
13	Beldon Pond Lake Dam, OH	–	1999	–	–	Piping into corroded conduit
14	Bent Tree Dam, TN	–	1999	–	–	Piping above conduit
15	Bergeron Dam, NH	–	1996	11.0 (36)	–	Piping immediately beneath spillway slab
16	Big Bay Lake, MS	–	2004	17.4 (57)	–	Piping through French drains (failed within 24 h of observed seepage)
17	Black Rock Estates Pond, MD	–	1992	–	–	Piping along conduit after heavy rainfall, shortly after construction
18	Blairtown, WY	–	1888	–	Rolled Earth	Piping along outlet
19	Blanch Park, CO	–	1984	–	–	Piping at outlet
20	Bradford, England	–	1896	27.4 (90)	Rolled Earth	Piping along outlet
21	Brindley Dam, MI	–	2003	–	–	Piping failure near outlet structure
22	Carninal Club Pond Dam, NC	–	2002	3.7 (12)	–	Piping, voids around conduit
23	Castlewood, CO	–	1897	21.3 (70)	–	Blowout of pipe under dam
24	Cedar Hills Lake Dam, NC	–	1977	6.7 (22)	–	Piping around conduit
25	Centennial Narrows Dam, AZ	1950's	1997	–	–	Piping near center of dam along transverse crack
26	Clarke Apple Orchard Lake Dam No. 1, GA	–	2002	–	–	Piping around outlet pipe
27	Conshohaken Hill, PA	–	1873	–	Rolled Earth	Piping through broken lining
28	Coon Rapids, MN	–	–	14.6 (48)+	–	Line of creep failure
29	Corpus Christi, TX	–	–	11.3 (37)+	–	Line of creep failure
30	Cranberry Creek, WI	–	1996	–	–	Piping at CMP outlet during high flows
31	Crane Creek, ID	1910	1928	19.2 (63)	Earth and Puddle core	Piping into outlet
32	CSC Orchards, Frost Protection Pond	–	1995	–	–	Piping along outlet conduit
33	Dale Dyke, England	–	1864	29.0 (95)	Earth and Puddle core	Piping along outlet
34	Davis Reservoir, CA	–	1914	11.9 (39)	Rolled Earth	Piping along outlet
35	Deoha, India	–	–	3.7 (12)+	–	Line of creep failure
36	Dry Creek, MT	1938	1939	14.0 (46)	Rolled Earth	Piping along outlet (dam failed three times)
37	Dolgarrog, N. Wales	–	–	9.1 (30)+	–	Line of creep failure

Table 1 continued

	Dam and location	Dates		Height m (ft)	Type	Reason
		Built	Failed			
38	Durance, France	–	–	6.7 (22)+	–	Line of creep failure
39	East Liverpool, OH	–	1901	–	–	Break over pipe through dam
40	East Peoria Dredge Disposal Facility, IL	–	1994	–	–	Piping at contact with sheet pile weir
41	Eagle Lake Dam, NY	–	2001	–	–	Piping into spillway barrel
42	East Purington Lake, IL	–	1983	–	–	Piping along spillway pipe
43	Eight Trout Club, VT	–	1990	–	–	Piping at interface between spillway and embankment
44	Emery, CA	–	1966	16.2 (53)	–	Piping-corrosion of outlet pipe
45	Empire, CO	1906	1909	9.1 (30)	Rolled Earth	Piping along outlet
46	E.R. Jahna—Independent North Sand Mine Tailings	–	1999	–	–	Piping at internal culvert
47	Fairfield Swamp Pond, VT	–	1980	4.6 (15)	–	Piping under core wall and pipe spillway
48	Faulkner Lake, MS	–	2004	9.1 (30)	–	Piping under spillway slab
49	Fergus Falls, MN	–	–	7.0 (23)+	–	Line of creep failure
50	Flederborn, Germany	–	–	7.0 (23)+	–	Line of creep failure
51	Flood Detention Dam, KS	–	1950	–	–	Piping along sewer line
52	Forsythe, UT	1920	1921	19.8 (65)	Rolled Earth	Piping under/along structure
53	Forsyth Rerservoir, GA	–	1997	6.1 (20)	–	Piping, undermined spillway during high flows
54	Frenchman Creek, CO	–	1995	7.3 (24)	–	Piping along spillway foundation, also caused sinkhole to appear in abutment
55	Galbreath Sediment Dam, WA	–	1997	–	–	Piping, probably along conduit
56	Gunnison, CO	–	1890	6.1 (20)	Rolled Earth	Piping along outlet
57	Halls Lake, VT	–	1984	–	–	Piping through spillway section
58	Harmon Park, OH	–	–	1.8 (6)+	–	Line of creep failure
59	Hatchtown, UT	1908	1914	19.8 (65)	Rolled Earth	Piping along outlet
60	Hazel Lake, WI	–	1995	2.1 (7)	–	Piping at aluminum outlet pipe
61	Hein Coulee Structure	–	1988	–	–	Piping between conduit and abutment bedrock
62	Hematite, KY	–	1998	4.0 (13)	–	Piping along contact between embankment and concrete sluice
63	Henry, CO	–	1996	–	–	Piping adjacent to outlet conduit
64	Hester Lake Dam, MO	–	1991	–	–	Piping beneath spillway
65	Holmdel Park Dam, NJ	–	1989	–	–	Piping at spillway culvert
66	Humboldt Lake, TN	–	1955	–	–	Piping around/along spillway on first filling
67	Ireland No. 5, CO	–	1984	6.1 (20)	–	Piping, erosion under spillway
68	Juniper Creek, OR	–	1953	–	–	Piping around outlet
69	Khanki, India	–	–	3.7 (12)+	–	Line of creep failure
70	Kitcha Bye, India	–	–	2.4 (8)+	–	Line of creep failure
71	Knodle (Hurdsfield) Dam, ND	–	1993	–	–	Piping under spillway weir and at abutments, after years of erosion
72	Lake Flamingo Dam	–	2001	8.2 (27)	–	Piping under conduit
73	Lake Francis (old dam), CA	1899	1899	15.2 (50)	Rolled Earth	Piping along outlet
74	Lake Francis, CA	1899	1935	23.5 (77)	Rolled Earth	Piping under spillway
75	Lake Latonka, PA	–	1966	10.4 (34)	–	Piping under concrete spillway
76	Lake Lynn Dam, NC	–	1995	2.1 (7)	–	Piping along spillway
77	Lake Runnemede, VT	–	1998	4.6 (15)	–	Piping under spillway

Table 1 continued

	Dam and location	Dates				
		Built	Failed	Height m (ft)	Type	Reason
78	Laramie, CO	–	1983	–	–	Piping along outlet
79	Lawn Lake, CO	–	1982	7.3 (24)	–	Piping around outlet works (or through embankment)
80	Lancaster, PA		1894	9.1 (30)	Rolled Earth	Piping along outlet
81	Lower Latham, CO	–	1973	8.2 (27)	–	Piping between fill and spillway
82	Lynde Brook, MA		1876	8.2 (27)	Rolled Earth	Piping along outlet
83	MacDonalton, PA	–	1911	–	–	Piping under spillway
84	Maquoketa, IA	1924	1927	6.1 (20)	Rolled Earth	Piping along structure
85	Mendham Reservoir Dam, NJ	–	1996	–	–	Piping under and adjacent to spillway structure
86	Millsboro Pond, DE	–	1979	3.7 (12)	–	Piping around culverts during high flows
87	Nadrai Escape Fall, India	–	–	6.1 (20)+	–	Line of creep failure
88	Nagels Mill Pond, MD	–	1999	4.9 (16)	–	Piping along concrete box spillway
89	Narora, India	–	1898	4.0 (13)+	–	Line of creep failure
90	New Bedford, MA	1866	1868	7.6 (25)	Rolled Earth	Piping along conduit
91	Owl Creek Site 13, OK	–	1957	8.5 (28)	–	Piping along conduit
92	Partridge Lake, WI	–	1993	–	–	Piping along outlet pipe
93	Pioneer Monument State Park, UT	–	1978	–	–	Piping along outlet
94	Pittsfield, MA	–	–	10 (33)+	–	Line of creep failure
95	Pittsfield Dredge Disposal Pond Dam, IL	–	1999	10.7 (35)	–	Piping along conduit, failure occurred within 2 h of observed seepage
96	Plattsburg, NY	–	–	10.4 (34)+	–	Line of creep failure
97	Port Angeles, WA	–	–	24.1 (79)+	–	Line of creep failure
98	Portland, ME	1889	1893	13.7 (45)	Rolled Earth	Piping along conduit
99	Pruett, Calif.	–	1937	–	–	Piping around outlet
100	Puentes, Spain	–	–	(143)+	–	Line of creep failure
101	Reservoir No. 1, UT	–	1961	–	–	Piping around outlet gate structure on first filling
102	Ridgewood Avenue Dam (Lake Apopka Dam)	–	1997	–	–	Piping along pipe
103	Rolling Green Community Lake, MD	–	1999	–	–	Piping, collapse of CMP
104	Roundy, UT	–	1973	3.0 (10)	–	Piping along CMP failed 8 years after reconstruction from previous incident
105	Roxboro Municipal Lake Dam, NC	–	1984	10.0 (33)	–	Piping under paved spillway, problem was noted months before failure occurred
106	Royal Oaks, MS	–	2002	7.6 (25)	–	Piping at spillway structure
107	Saddle Lake Dam, NY	–	1974	7.0 (23)	–	Piping into CMP joints
108	Sand Creek, CO	–	1915	7.3 (24)	–	Piping side of outlet
109	Sarnia Dam, ND	–	1978	6.1 (20)	–	Piping around conduit
110	Scofield, UT	–	1928	23.8 (78)	–	Piping through settlement cracks near abutment
111	Shale Creek, MT	–	2004	4.6 (15)	–	Piping, corroded CMP
112	Sky Lake No. 1, TN	–	1987	–	–	Piping at spillway
113	Spencer Estates Detention Basin, NJ	–	1999	–	–	Piping along culvert
114	Spring Lake, RI	1887	1889	5.5 (18)	Earth and Rock	Piping along outlet
115	Spruce Lake, VT	–	1969	7.3 (24)	–	Piping at pipe spillway
116	Staffordville, CT		1887	6.1 (20)	Earth and Rock	Piping along outlet
117	Stewart, VT	–	1980	–	–	Piping at pipe spillway

Table 1 continued

	Dam and location	Dates		Height m (ft)	Type	Reason
		Built	Failed			
118	Stoney River, WV	–	–	11.9 (39)+	–	Line of creep failure
119	Simpson Dam, ND	–	1986	5.0 (16.5)	–	Piping at low level outlet pipe after heavy rainfall
120	Swanson, VT	–	1991	–	–	Piping at interface between spillway and embankment
121	Toreson, CA	–	1953	16.8 (55)	–	Piping—outlet pipe corrosion
122	Tupelo Bayou Site 1, AR	–	1973	14.6 (48)	–	Piping at spillway conduit (from differential settlement) failed during construction
123	Tupper Lake, NY	1906	1906	5.5 (18)	Rolled Earth	Piping along outlet
124	Turlock Irrigation, CA		1914	17.1 (56)	Rolled Earth	Leakage around outlet
125	Unnamed Dam, SC	–	1990	–	–	Piping along spillway pipe on first filling
126	Upper Red Rock Creek Site 20, OK	–	1986	9.4 (31)	–	Piping, internal erosion through embankment (dispersive soils??)
127	Vertrees, CO	–	1998	8.2 (27)	–	Piping, outlet damaged
128	Walter Bouldin Dam, AL	1967	1975	51.8 (170)	Earth and Rock	Piping into downstream shell adjacent to intake structure??
129	Weisse, Czech.		1916	12.8 (42)	Rolled Earth	Piping along outlet
130	Wilmington, DE	1887	1900	3.7 (12)	Rolled Earth	Piping along outlet
131	Woodward, NH	–	–	9.1 (30)+	–	Line of creep failure
132	Worcester, MA	1871	1876	12.5 (41)	Rolled Earth	Piping into conduit
133	Wyoming Development Co. No. 1, WY	–	1969	14.9 (49)	–	Piping above outlet works
III. Possible backwards erosion and suffosion piping failures (uncertain/unknown cause piping failures are also lumped into this category)						
1	Bischel, WI	–	1988	3.7 (12)	–	Piping
2	Boyd Reservoir, NV	–	1995	9.8 (32)	–	Piping through embankment after rain and snowmelt
3	Bridgefield Lake Dam, MS	–	2001	7.6 (25)	–	Piping induced sloughing/slope failure
4	Browder, TN	–	1972	8.2 (27)	–	Piping leak
5	Camp Ritchie, MD	–	1929	–	–	Piping
6	Caney Coon Creek Site 2, OK	–	1964	–	–	Two pipe tunnels appeared simultaneously at downstream toe (dispersive soils ??)
7	Cold Springs, CO	–	1912	15.2 (50)	Rolled Earth	Embankment seepage
8	Corpus Christi Dam, TX	–	1930	18.6 (61)	–	Piping through cutoff
9	Costilia, NM	1920	1924	38.1 (125)	Rolled Earth	Embankment seepage
10	Crump Reservoir, OR	–	1980	4.6 (15)	–	Piping
11	Crystal Lake, CT	–	1961	15.2 (50)	–	Piping after long history of leakage (100 + years)
12	Del Rio Creek, TN	–	1984	–	–	Piping during heavy rainfall
13	Desabia Forebay, CA	1903	1932	16.2 (53)	Rolled Earth	Piping through embankment
14	Dexter Creek, CA	–	1973	–	–	Piping
15	Dresser No. 4 Dam, MO	–	1975	32.0 (105)	–	Piping
16	Earth Resources Co. Nacimento, NM	–	1973	–	–	Piping
17	East Head Pond Dam, MA	–	1997	4.6 (15)	–	Piping
18	Eblen No. 2, ID	–	1977	4.3 (14)	–	Piping failed suddenly
19	Echo Lake, CT	–	1958	5.5 (18)	–	Piping in repaired section
20	Edwards, TN	–	1979	5.8 (19)	–	Piping
21	Eureka Holding, MT	–	1995	12.2 (40)	–	Piping through dike after heavy rainfall
22	Fertile Mill Dam, IA	–	1979	3.4 (11)	–	Piping or seepage induced slope failure

Table 1 continued

	Dam and location	Dates		Height m (ft)	Type	Reason
		Built	Failed			
23	Haas Pond Dam, CT	–	1984	4.0 (13)	–	Piping
24	Hebron, NM	1913	1914	17.1 (56)	Rolled Earth	Piping through embankment
25	Holland Dam Site A, TX	–	1997	4.0 (13)	–	Failed either by undermining near center of dam or by piping through desiccation cracks
26	Horse Creek, CO	1911	1914	17.1 (56)	Rolled Earth	Piping
27	IMC-AGRICO Hopewell Mine, FL	–	1994	–	–	Piping
28	Jackson Creek Watershed, SC	–	1977	–	–	Piping
29	Julesburg Jumbo, CO	1905	1910	21.3 (70)	Rolled Earth	Seepage
30	Lake Avalon, NM	1894	1904	14.6 (48)	Rolled Earth	Seepage
31	Lake Francis (old dam), CA	1899	1914	15.2 (50)	Rolled Earth	Piping
32	Lake Gary Dam, MS	–	1995	12.2 (40)	–	Piping
33	Lake Nora Dam, GA	–	2001	18.3 (60.2)	–	Piping
34	Lake Paran, VT	–	1852	7.6 (25)	–	Piping
35	Lake Venita Dam, MO	–	1997	9.1 (30)	–	Piping, water flowed from toe prior to failure
36	Lambert, TN	–	1963	16.5 (54)	–	Piping-small leak increased leading to breach
37	Little Washita River Site 13, OK	–	1987	10.7 (35)	–	Piping through gypsiferous soils, failed 10 years after construction
38	Little Wewoka Creek Site 17, OK	–	1960	7.3 (24)	–	Piping (dispersive soils??) Small leak at 8 AM developed into tunnel by night
39	Littlefield, NV	–	1929	–	–	Piping/Seepage induced slide
40	Longwalds Pond, MA		1922	9.1 (30)	Earth and Concrete core	Piping
41	Lyman, AZ	1913	1915	19.8 (65)	Rolled Earth	Piping through core (?)
42	Magic, ID	1910	1911	39.6 (130)	Rolled Earth	Piping through embankment
43	Mahonoy City, PA		1892	–	Rolled Earth	Piping
44	Mann Creek Dam, OR	–	1982	–	–	Piping
45	Marshall Lake, CO	1908	1909	21.3 (70)	–	Seepage
46	Masterson, OR	1950	1951	18.3 (60)	Rolled Earth	Piping dry fill
47	Mill River, MA	1865	1874	13.1 (43)	Earth and Concrete core	Seepage
48	Mohawk, OH		1913/1915	5.5 (18)	Rolled Earth	Seepage
49	Montreal, QC, Canada		1896	5.5 (18)	Earth and Rock	Seepage
50	Mud Point, MA	1873	1886	4.6 (15)	Earth and Rock	Piping
51	Name Unknown, VT	–	1998	–	–	Piping
52	Nebraska City, NE	1890	1890	5.2 (17)	Earth and Rock	Seepage
53	Noonan, VT	–	1986	–	–	Piping
54	Norton Brook, VT	–	1942	11.0 (36)	–	Piping
55	Otter, TN	–	1978	6.1 (20)	–	Piping during flood
56	Penn Forest, PA	–	1960	46.0 (151)	–	Piping-sinkhole developed on upstream face
57	Pine Ridge Dam, PA	–	1969	–	–	Piping
58	Pinkston, MO	–	1978	21.3 (70)	–	Piping static liquefaction
59	Riddel Pond, VT	–	1990	–	–	Piping
60	Rinse, VT	–	1996	2.4 (8)	–	Piping
61	Rocky Ford, UT	1914	1915/1950	21.3 (70)	Rolled Earth	Seepage
62	Roxborough, PA	1894	1894	–	Rolled Earth	Piping

Table 1 continued

	Dam and location	Dates		Height m (ft)	Type	Reason
		Built	Failed			
63	Saint John, ID	–	1980	11.9 (39)	–	Piping, sinkholes on upstream slope—second case of piping failure at this dam
64	Sauk River Melrose, MN	–	2001	8.2 (27)	–	Piping sloughing, slope failure during high water
65	Scottdale, PA		1904	18.3 (60)		Piping
66	Seymour Reservoir Dam, IA	–	1976	7.3 (24)	–	Piping
67	Sheltan, CT		1903	6.1 (20)	Earth and Rock	Piping
68	Snow Bird Lake Dam, NY	–	1980	5.5 (18)	–	Piping
69	Southern Clay Co. Dam No. 2, TN	–	1989	–	–	Piping (sloughing on upstream and downstream slopes)
70	Sunrise Lake Dam, PA	–	1962	–	–	Heave and piping
71	Swansen, Wales	1867	1879	24.4 (80)	Earth and Rock	Piping
72	Timber Creek Watershed Dam 1, KS	–	1967	9.4 (31)	–	Piping thorough embankment
73	Toliver, ID	–	1984	–	–	Piping, sudden failure
74	Towanda, PA	–	1939	–	–	Piping/seepage induced sloughing
75	Upper Moore Pond, VT	1900	1973	5.8 (19)	–	Piping
76	Upper Red Rock Creek Site 48, OK	–	1964	7.0 (23)	–	Piping erosion tunnel (dispersive soils??) Breached within 24 h of first leak
77	Vance Lake, MS	–	1979	7.3 (24)	–	Piping
78	Vernon Marsh-Ref. Flowage, WI	–	1996	2.1 (7)	–	Piping, sunny day failure
79	Wallace Lake Dam, NC	–	1988	4.3 (14)	–	Piping
80	Washington County Lake Dam, IL	–	1962	7.9 (26)	–	Piping on first filling
81	West Julesburg, CO	1905	1910	16.8 (55)	Rolled Earth	Piping
82	Wister, OK	1951		27.4 (90)	Rolled Earth	Piping
83	Worcester, CO	1912	1951	20.7 (68)	Rolled Earth	Concentrated seepage
IV. Piping failures induced by biologic activity						
1	Big Sand Creek Str Y032032, MS	–	2002	7.6 (25)	–	Piping, pipe formed 5 to 6 feet above normal pool, possible animal activity
2	Dennery Lake, MS	–	2005	6.7 (22)	–	Piping, biological growth
3	Eleva Roller Mill, WI	–	1994	4.3 (14)	–	Piping, tree roots
4	Johny Stewart Pond, MS	–	2003	–	–	Piping along rotted tree roots
5	Johnston City Lake Dam, IL	–	1981	4.3 (14)	–	Piping in poorly maintained embankment
6	Lower Stichcomb, GA	–	1978	4.2 (13.8)	–	Piping (muskrat hole into foundation)
7	Mallard Lake, TN	–	1996	6.4 (21)	–	Piping from animal activity, or possible instability
8	Prospect Reservoir Dam, CO	–	1980	–	–	Piping via animal burrows, in area with crack
9	Smith River Lumber Co. Pond, OR	–	2002	8.2 (27)	–	Piping halfway up side of dam, may be due to trees, burrows (failed within hours of first observed seepage)
10	Udall, AZ	–	1997	7.3 (24)	–	Piping aided by tree roots
11	Upper Lebanon Reservoir No. 1, AZ	–	1978	–	–	Piping through embankment (tree roots)

Early work

One of the earliest references to the piping process was made by Von Richthofen in 1886 as it applied to landforms in loess (Jones 1981). The term piping was used by Bligh in 1910 to describe the removal of soil along the foundation of masonry dams; the mechanical process of this form of piping was being researched in laboratory studies some-time around 1895 in India (Bligh 1910; Clibborn 1902). Col. Clibborn predicted the collapse of the Narora Dam on the Ganges River, India in 1898, which is apparently the first incident where piping became an engineering concern (Jones 1981, p. 27). Subsequently, Bligh (1910; 1911a, b, 1913), who developed most of his theories while working in India, first recognized a possible connection between the length of flow path and the tractive forces available to move soil particles. His theory is termed the line-of-creep theory. It is an empirically derived method to evaluate the piping potential along the contact between structures and soils. Bligh's theory contrasts with the short-path theory, which assumes a molecule of water would travel the shortest distance between a headwater entrance and exit point, which had been employed prior to the line-of-creep theory. The short-path theory is a straight line simplification of a flow-tube, which could be more accurately estimated by construction of a flow net.

Flow nets were initially developed by Phillip Forchheimer around 1900 and later formalized by Arthur Casagrande in 1937. This method was an improvement to the short-path method that came after Bligh developed his line of creep theory (1910) and has been used extensively in modern times to predict exit velocities. An important aspect of the line of creep theory is that the preferential flow path is not Darcian, but that it utilizes Darcy's law that discharge (hence seepage velocity) is directly proportional to the hydraulic head and inversely proportional to the length of flow path. In Bligh's theory, the flow path is described as the sum of the vertical and horizontal distances measured along the structure/soil contact. There was much discussion in the early 1900's about whether seepage traveled along the structure contacts or via intergranular flow (Lane 1934). Bligh's (1910) line of creep method became the accepted tool for evaluation of masonry or concrete structures founded on soils. Bligh's equation is shown below:

$$L = cH \text{ and } c = L/H \tag{1}$$

where
L = required safe flow length (or actual flow length)
c = percolation factor
H = hydrostatic head across the structure.

The above equation illustrates that flow gradients have a direct influence on the piping potential of a structure. Bligh measured L as the sum of the vertical and horizontal distances along the base of the structure. He developed guidelines for assessing a safe percolation coefficient from his equation by empirical correlation with a number of dams that had failed and classified foundation soils into five classes (Bligh 1910) as shown in Table 2.

Lane (1934) also used the term piping to describe the removal of soil along the foundation of masonry dams but drew a clearer distinction between flow along structural contacts and diffuse flow through granular media (Lane 1934, p. 937). Terzaghi defined piping as the progressive backward erosion of particles from an exit point of concentrated leakage, along with another mechanism termed heave (Terzaghi 1922, 1939, 1943). Work by Lane (1934) bolstered Bligh's earlier work with more extensive case histories and some adjustments were made to take into account the anisotropic conditions that govern fluid flow in stratified materials. This improvement was sparked by a realization while working on the Prairie du Sac Dam in Wisconsin, USA (Lane 1934, p. 949), although Lane credits Griffith (1913) with first developing the concept. Lane's equation, known as the weighted-creep-method, is shown below:

$$Ln = cH \text{ and } c = Ln/H \tag{2}$$

where
Ln = minimum safe flow length
c = safe weighted creep ratio
H = hydrostatic head across the structure.

Lane's empirical correlation is very similar to Bligh's, although as the flow paths are handled differently in the two methods, the guidelines for assessing piping potential are not comparable. Lane's method assumes anisotropic flow and provides an arbitrary reduction of 1/3 to the length of horizontal flow paths. Using this equation, Lane developed guidelines based on a study of over 200 dams. His well-documented empirical correlation quickly replaced Bligh's line-of-creep method for the evaluation of structures founded on soil. Lane recommended the safe weighted creep ratio values shown in Table 3.

These safe weighted creep ratios do not take into account the possible presence of dispersive soils in the case histories used to develop the safe weighted creep ratio. The equation of Bligh, later improved by Lane, provides an empirically derived basis for estimating piping potential. Based on these equations, increased piping potential is directly proportional to hydraulic head and inversely proportional to the length of seepage path. It is generally applied at soil structure boundaries and is still in common use today. However, from a theoretical view, flow along soil structure boundaries does not adhere to Darcian flow rules and may be more accurately modeled by the cubic law rule that governs flow along planar openings (Franco and Bagtzoglou 2002; Louis 1969; Worman and Olafsdottir 1992). Hence, the size of the opening would play a critical role (Franco et al. 2002, p. 3). There is currently no method in practice that accounts for this apparent discrepancy in

Table 2 Bligh's line of creep recommended values for piping stability

Class	Soil	Required c
A	Fine silt and sand	18
B	Fine micaceous sand	15
C	Coarse sand	12
D	Gravel and sand	9
E	Boulders, gravel and sand	4–6 (increased to 6–9 in 1913)

the theoretical basis behind Lane's weighted creep method, and no one has attempted to assess if Lane's safe weighted creep ratios adequately account for dispersive/non-dispersive soils.

Lane recognized that two distinct forms of piping failure may exist; failure due to line-of-creep flow, or piping due to the shortest path (intergranular) flow. He believed that both the weighted creep and short path failure modes should be evaluated to ensure safety in the design of dams (Lane, 1934 pg. 938). However, in current practice, it is not uncommon for engineers to neglect assessing one or the other of these two failure modes (US Bureau of Reclamation 1987; USACE 1993; FERC 2005). Lane also indicated that there may be a single method that could determine the path of least resistance between line-of-creep versus shortest path methods (Lane 1934, p. 935). But the state-of-the-art that existed in Lane's time had not developed to an extent to resolve this problem, and it has not yet been solved today.

Terzaghi (1939, 1943), Terzaghi and Peck (1948), and Bertram (1940) provided the equations for filter criteria. Terzaghi's equations do not predict whether self-filtration will occur in a homogeneous mass of soil. This development came much later (Kenney and Lau 1985; Åberg 1993). However, the contributions of Terzaghi were invaluable in the design of defensive measures to protect against piping failures. Many subsequent workers (Bertram 1940; Sherman 1953; Sherard et al. 1984a; Sherard and Dunnigan 1989) performed additional studies to refine and confirm Terzaghi's original filter criteria. The evolution of filter design criteria is shown in Table 4 (after Sherman 1953).

Table 3 Lane's weighted creep recommended values for piping stability

Soil	Required c
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel and cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay	1.6

Another early advancement was the theory of heave, first proposed by Terzaghi (1922, 1939, 1943). Jones (1981, p. 28) reports that while Terzaghi and Peck noted that most piping failures appeared to be due to subsurface erosion (Terzaghi and Peck 1948, p. 506), it was the simple heave theory that was most commonly discussed by engineers (Casagrande 1937; Legget 1939; Bertram 1940, 1967; Glossop 1945; US Bureau of Reclamation 1947; Harr 1962; Sherard et al. 1963; Lambe and Whitman 1969; and others). In fact, other than those derived from the line of creep theory, most modern equations for factors of safety against piping are some form of the original heave theory.

Terzaghi (1922, 1943, p. 257) originally presented his method for calculating piping potential in the case of boils in a cofferdam cell. The problem is specifically for upward vertical flow of groundwater into the floor of an excavated and dewatered cofferdam. Terzaghi (1922) performed model tests of this problem and found that the sand floor remains stable up to a critical value of hydraulic head outside the cofferdam. Once this head is exceeded "...the discharge increases more rapidly than the head, indicating an increase of the average permeability of the sand". He termed this phenomenon "piping", which is commonly observed as boils. Terzaghi's (1922) description of the heave phenomenon bears some similarity to the critical state theory of soil in that a critical maximum void ratio appears to be obtained at the state of failure. Increasing hydraulic loads beyond this point do not appear to affect the hydraulic conductivity and by extrapolation the void ratio.

Terzaghi (1943) determined a factor of safety against such piping, which is simply the effective weight of a prism of soil in the area of expected heave, divided by the excess hydrostatic pressure beneath it. The critical factor of safety was found by trial and error using a number of potential depths for the prism. The factor of safety is expressed as;

$$Gs = (W'/Ue) \tag{3}$$

where:
Gs = Factor of Safety against piping
W' = Effective weight of the most critical sand prism

Table 4 Summary of the development of Filter Design Criteria (after Sherman, 1953), where D_{15} is diameter of particles at 15% passing (for filter material), and d_{85} is diameter of particles at 85% passing (for base soil), d_{15} is diameter of particles at 15% passing (for base soil), and C_u is the coefficient of uniformity of base soil (d_{60}/d_{10})

Investigators	Base material	Filter material	Criteria developed
Terzaghi (1922)	Uncertain if criteria was based on experiments or conservative reasoning		$D_{15}/d_{85} < 4 < D_{15}/d_{15}$
Bertram (1940)	Uniform quartz and Ottawa sands	Uniform quartz and Ottawa sands	$D_{15}/d_{85} < 6$ $D_{15}/d_{15} < 9$
Newton and Hurley (1940)	Well-graded gravelly sand	Natural bank gravels. Finer sizes successively screened out. Fairly uniform filters	$D_{15}/d_{50} < 15$ $D_{15}/d_{15} < 15$
Waterways Experiment Station (1941), (1948)	Random material types. Fine to coarse sand	Random types including natural pit-run gravels	$D_{15}/d_{85} < 5 D_{15}/d_{15} > 4$, < 20 $D_{15}/d_{50} < 25$ Gradation of filters should be more or less parallel to base. Filter should be well graded.
Office Chief of Engineers	All types	Concrete sand and coarse aggregate generally recommended	$D_{15}/d_{85} < 5$ $D_{15}/d_{15} > 5$
US Bureau of Reclamation (1947)	Artificially blended materials of various ranges including uniform material	Artificially blended uniform filters Artificially blended well-graded filters	$D_{50}/d_{50} > 5, < 10$ $D_{50}/d_{50} > 12, < 58$ $D_{15}/d_{15} > 12, < 40$
Waterways Experiment Station (1942)	All types	Certain general types recommended	Filter design curve C_u of base vs D_{15}/d_{15}
Sherman (1953)	Vicksburg loess and screened loess	Mixtures of natural sands (Corp of Engineers standard gradation for concrete sand) and gravels (Corp of Engineers standard gradation for concrete gravel)	$D_{15}/d_{85} \leq 5$ $D_{15}/d_{15} \leq 20$ $D_{50}/d_{50} \leq 25$ But for very uniform base materials: ($C_u <$ 1.5) $D_{15}/d_{85} \leq 6$ And for widely graded base materials: ($C_u >$ 4) $D_{15}/d_{15} \leq 40$ gap graded or widely graded materials are not recommended for filters

Ue = Uplift pressure at the base of the prism (determined from a flow net).

Terzaghi and Peck (1948, p. 229) identified two processes that can cause failure by piping;

(1) Scour or subsurface erosion that starts at an exit point near the downstream toe and proceeds upstream along the base of the structure or within the foundation.
(2) Sudden rise of a large body of soil adjoining the downstream toe of the structure.

They termed the first failure mode as "failure by subsurface erosion", and the second failure mode "failure by heave". Terzaghi et al. (1996) indicated that the first type of piping defies a theoretical approach. They state "...in reality, most piping failures occur at hydraulic heads h_c' much smaller than the head h_c computed on the basis of theory..." and they report the ratio h_c'/h_c decreases rapidly with decreasing grain size (Terzaghi et al. 1996, p. 475). They state that piping failures can occur from a few to many years after first filling. This is confirmed by a review of cases of piping failures, which in some cases occur decades after first filling (Crystal Lake Dam: 100 years, Upper Moore Pond Dam: 73 years, NPDP database). Terzaghi et al., (1996, p. 478) also state "theory and experience lead to the following conclusions. Most of the piping failures on record have been caused by subsurface erosion involving the progressive removal of materials through springs; this condition invalidates the theory of piping by heave. The factor of safety with respect to piping by subsurface erosion cannot be evaluated by any practicable means".

Terzaghi et al. (1996, p. 475) indicate that most piping failures are caused by a process that reduces the factor of safety gradually and inconspicuously until the point of failure is reached. They also state, and it is a common misperception, that this process cannot occur in a homogeneous body of cohesionless sand. Cohesionless embankments commonly experience sinkholes due to piping or failure by piping along conduits. It is also possible for pipes to develop in otherwise cohesionless embankments when minor inhomogeneites in embankment construction allow for zones of cohesion or cementation. Heterogeneities in foundations may also support the formation of pipes. Hence, potential piping in cohesionless embankments should not be completely ignored and must be evaluated as with any other potential failure mode. The type of piping to which Terzaghi et al. were referring cannot be evaluated adequately with current engineering tools.

Dispersive soils

Jones (1981, p. 96) credits Aitchison (1960) and Aitchison et al. (1963) as among the first to suggest that piping processes involved dispersion of clays. Ritchie (1963) developed a method for determination of the dispersivity of soils, called the Dispersion Index method. Ritchie (1963) defined 33 percent of the soil fraction less than 0.004 mm dispersing after 10 min of shaking in water as indicative of potential failure by tunneling for earth dams in Australia. A symposium was held by Australia's Commonwealth Scientific and Industrial Research Organization (CSIRO) in 1964 to discuss the failure of a number of small earth dams. Researchers began to study this problem in subsequent years (Arulanandan et al. 1975; Alizadeh 1974; Kandiah 1974). The initial discovery of how dispersive soils were affecting Australian dams was followed eight years later by a similar round of dam failures in the US (Sherard 1971; Sherard et al. 1972). Research into the US dam failures led to the subsequent proceedings at the 79th ASTM annual meeting in Chicago, IL, and the issue of ASTM Special Technical Publication No. 623 (Stapledon and Casinader 1977; Sargunan 1977; Villegas 1977; Rosewell 1977; Ryker 1977; Heinzen and Arulanandan 1977; Sherard et al. 1977; and others), which dealt specifically with the problems of dispersive soils and brought together the knowledge base of Australian and US workers.

A number of tests were developed for dispersive soils; amongst these are the SCS Laboratory Dispersion Test (Decker and Dunnigan 1977; Sherard et al. 1972), the Triple Hydrometer Test (Coumoulos 1977), Pinhole Test (Sherard 1976; ASTM D4647-93), Modified Pinhole Test (Heinzen and Arulanandan 1977, p. 206, 216), Emerson Crumb Test (Emerson 1967), Free Swell Test (Ladd 1960), Floyd's Sticky Point Test (Crouch 1977), Rotating Cylinder Test (Riley and Arulanandan 1972; Sargunan 1977), Sodium Adsorption Ratio (SAR) Test (Sherard et al. 1972), ESP Test (Sherard et al. 1972; Jones 1981, p. 55), SCS Field Test (Decker and Dunnigan 1977, p. 102), Modified Hydrometer Test (Forsythe 1977, p. 153), Dielectric Dispersion Test (Arulanandan et al. 1973; Heinzen and Arulanandan 1977, p. 210; Appendix), and Flume Test (Kandiah 1974; Heinzen 1976).

Heinzen and Arulanandan (1977, p. 204) consider Sherard's pinhole test a qualitative test, whereas the rotating cylinder test (Riley and Arulanandan 1972) and flume tests are quantitative. According to Heinzen and Arulanandan (1977, pp. 205–206) qualitative methods for the evaluation of dispersivity are;

(1) SCS dispersion test (Decker found that critical values of dispersion ratio depend to some extent on soil type).
(2) Emerson's crumb test (considerable difference may exist between the results from the crumb test versus the SCS dispersion test).
(3) Pinhole test (where dispersive soils fail under a 50 mm head, intermediate soils erode slowly under 50 or 150 mm head, and non-dispersive soils produce no erosion under 380 or 1,020 mm head. Shear stress can be estimated assuming laminar flow and the diameter of the hole.)

Heinzen and Arulanandan performed pinhole tests with a turbidity meter and found a continuous turbidity value of 30 Jackson turbidity units (JTU) indicative of colloidal suspension that could be used to calculate the critical shear stress. Use of a turbidity meter with the standard pinhole test provides a more accurate determination of piping initiation.

Forsythe (1977, p. 153) found the pinhole test to be the most reliable method for the identification of dispersive soils. The percent sodium of total soluble salts was also a good indicator for Forsythe, but he reported some of his sample areas did not show as good a correlation as the pinhole test. In 281 samples collected from around the world, approximately 85% of those with dispersive characteristics tested 30–40% dispersion as determined by the SCS dispersion test (Decker and Dunnigan 1977, p. 104). Similar accuracy was obtained when comparing the pore water sodium versus total salt content. Decker and Dunnigan (1977) conclude that their data confirmed that the SCS dispersion test is reliable for predicting dispersiveness of soils, provided they are maintained at the natural moisture content (p. 107). However, a number of other workers have reported problems with the SCS Laboratory Dispersion Test. From a review of the literature, it becomes apparent that there is no single diagnostic test for dispersive soils. It is recommended practice (Marshall and Workman 1977) to perform a number of tests and use considerable engineering judgment when dealing with dispersive soils. A significant problem in identification of dispersive soils is the common inhomogeneity of dispersive characteristics. Very frequent sampling and testing are recommended and there are a number of field scale tests that should be utilized during construction.

None of the experimental methods in use (dispersion test, crumb test, pinhole test, rotating cylinder test, flume test) measure the internal resistance due to seepage forces (Bonala 1997, p. 16). This, combined with other heterogeneities in soil chemistry and fabric, probably account for the discrepancies commonly noted when dispersive soil surveys are performed. Table 5 lists some of the accepted criteria for dispersive soil tests (Jones 1981; Middlebrooks 1953).

Sherard et al. (1972) reviewed a number of Australian studies and concluded that the quantity of dissolved sodium cations in the pore water relative to other cations was one of the key parameters governing piping susceptibility. They summarized the empirical data from case histories of failures that showed good correlation to the concentration of sodium in the saturation extract versus the total soluble salts in the saturation extract (Na(100)/(Ca + Mg + Na + K). Based on this relationship, they divided the case histories into zones. Examples of similar plots from Cole Tatanasen and Maiklad (1977) are reproduced in Figs. 1 and 2.

Sargunan (1977, p. 396) plotted extrapolated SAR and total cation concentration corresponding to $\tau_o = 0$ to determine threshold limits between flocculated and deflocculated states based on the fluid chemistry. As one would expect, it was found that the boundary varies depending on whether the sample was illite or kaolinite, hence the boundaries are unique to the clay mineralogy of the sample. Sargunan (1977, p. 397) concluded that chemical control of piping failure can be achieved by modification of either the total cation concentration of the percolating water, or the SAR, or both. Figure 3 shows the general relationship of SAR and total cation concentration resulting in piping.

Table 5 Published criteria for susceptibility of soils to piping (modified from Jones 1981)

Charman (1969, 1970a, b)	Ritchie dispersal index < 3 = susceptible, DI < 3 and volume expansion > 10% = highly susceptible (later changed to > 20%)
Crouch (1976)	Dispersal Index < 3 (Ritchie 1963)
	Permeability < 1 cm/h (Fletcher et al. 1954; Brown 1962; Heede 1971)
	ESP > 0.1 (lowest noted by N.O. Jones 1968) or more commonly 12 (after Fletcher and Carroll 1948; Heede 1971)
Decker and Dunnigan (1977)	Percent dispersion by volk test:
	> 25% for inorganic low plasticity silt (ML) and Sand–clay (SC) soils
	> 35% for inorganic clayey soils of low to medium plasticity (CL)
	> 40% for inorganic clays of high plasticity (CH)
Ingles (1968)	Permeability ≥ 10^{-2} cm/s for silt or silt-size aggregates
	Permeability ≥ 10^{-5} cm/s for dispersive clay soils
	ESP > 12% indicating dispersive clay soils, but dispersivity depends on exchange cations, salt concentrations of water, and clay minerals
Stocking (1976)	ESP > 15% indicating dispersive clay soils

Advances in filter design/internal stability

Batereau (1993) provides a recent update of latest filter criteria. The discovery of the potential problems with dispersive clays ultimately led to a re-evaluation of filter designs for protection of clays (Honjo and Veneziano 1989; Sherard et al. 1977, p. 473, 1977, 1984b; Sherard and Dunnigan 1989; Åberg 1993; Arulanandan and Perry 1983; amongst others). Bonala and Reddi (1998, p. 49) discuss the limitations of modern filter criteria for filtering clay soils. In particular, the phenomenon of clogging is not adequately addressed in filter criteria and the long term performance of such filters could become an issue over time. In Bonala's (1997, p. 128) discussion of the role of critical shear stress in particle transport, he states "...progressive accumulation of the particles in filters may lead to buildup of excessive pore pressures leading to instability of geotechnical structures such as retaining walls and dams". This is especially true in cases where filters have been designed for dispersive soils. In such cases, migrating fines may form filter cakes on the surfaces of the filter, which significantly decrease the hydraulic conductivity of the filter (Sherard et al. 1977). Bourdeaux and Imaizumi (1977, pp. 16, 17) performed a series of sand filter tests on compacted sand filters and dispersive clays. They found that compacted filters designed with conventional filter criteria would control leaks through dispersive clay. Hydraulic gradients between 0.5 and 4.0 had little influence on the rate of sealing. They concluded that silt sized particles from the dispersive clays helped to form a skin on the upstream surface of filters and decreased water flow.

Sherard and Dunnigan (1989) performed a series of tests for filters for use downstream of impervious soils such as the cores of dams. Their investigation confirmed that the filter criteria currently in use can prevent erosion of impervious core material, for each of the four groups of soils they investigated. Their recommendations for filter boundaries are shown in Table 6. D_{15b} is the "boundary" filter size, which is the size that prevents erosion in the base soil. If the D_{15} of the filter was greater than this, erosion of the base soil took place in their experiments. If the D_{15} of the filter was less than or equal to this size, no erosion took place. Their study did not consider the long-term effect of clogging.

The conclusion one may draw from these studies is that filters designed for fine grained base soils may not facilitate drainage. This is probably not a significant problem, as clay base soils should transmit little water. However, if lowered

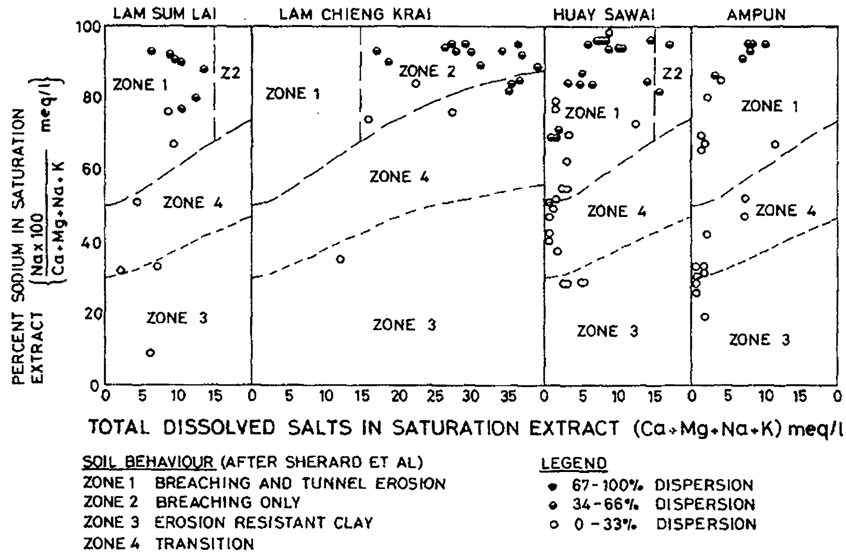

Fig. 1 Chemical zones and plots of data from Cole et al. (1977)

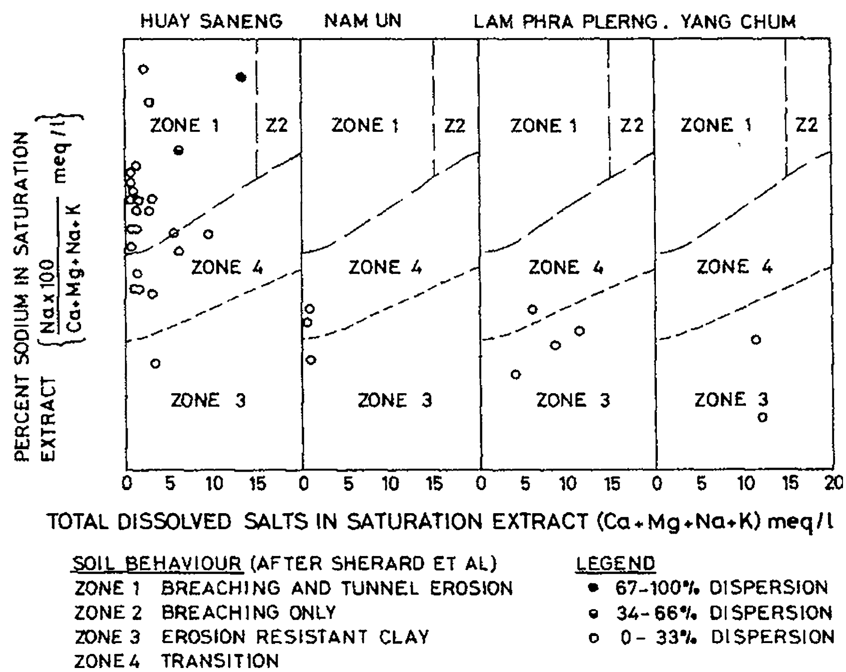

Fig. 2 Examples from other dams (from Cole et al. 1977)

phreatic levels are necessary for stability reasons, standard filters may not suffice for fine grained base soils. The issue of clogging in these types of filters has not yet been adequately addressed.

Moffat and Fannin (2006, p. 273) cite Kézdi (1979) and Kenney and Lau (1985) who quantify the potential for internal instability based on grain size distribution, and Skempton and Brogan (1994) who verify the applicability of the geometric factors elucidated by the earlier workers. Skempton and Brogan (1994, p. 449) refer to the methods of Kézdi (1979) for assessing the internal stability of soils. The general concept is that a soil is divided into its fine and coarse fractions and treated as a filter and base soil. Terzaghi's (1939) filter criteria must be satisfied for the soil to be internally stable. Permeation tests by Skempton and Brogan (1994) and Adel et al. (1988) indicated that failure of internally unstable soils can occur at hydraulic gradients much lower than predicted by theory. Skempton and Brogan (1994) theorized that the piping was triggered at lower hydraulic gradients due to a lower effective stress acting on the finer fraction. As Moffat and Fannin (2006) elucidated "...hydromechanical influences governing the onset of instability are not well understood". The stability of non-cohesive materials and influence of stress is not well developed.

Skempton and Brogan (1994, p. 452) reviewed vertical flow tests conducted by Kenney and Lau (1985) and note

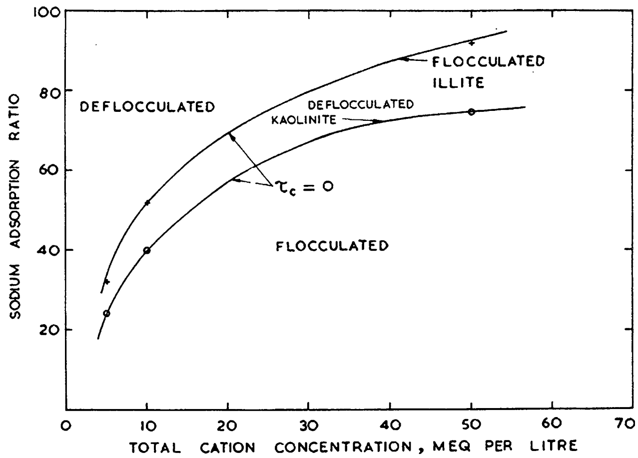

Fig. 3 Soils with low total salt concentrations require less sodium percentage to become dispersive (Sargunan 1977)

that there was a question remaining regarding the critical gradient at which fines began to move (the paper by Kenney and Lau did not provide enough information to determine the critical hydraulic gradients). As reported by Skempton and Brogan (1994), follow-up tests were conducted by Adel et al. (1988) using an apparatus with horizontal flow. The Adel et al. tests reportedly resulted in critical hydraulic gradients required to initiate piping from 0.16 to 0.7, depending on the degree of internal stability (Skempton and Brogan 1994, p. 452). Skempton and Brogan noted that there are no comparable data for the case of upward flow and subsequently carried out the upward flow tests. They reported that the vertical tests resulted in hydraulic gradients that were higher than the horizontal flow tests of Adel et al. but that the differences were small. A comparison of the two sets of data are reproduced in Table 7 (Skempton and Brogan 1994, p. 452).

Some interesting differences in the data not discussed in Skempton and Brogan (1994) are the apparent discrepancies between the i_c for the stable material, and the fact that the Adel et al. (1988) results for the unstable material show an i_c approximately half that of Skempton and Brogan (1994). Terzaghi's (1922) laboratory work on heave was performed with vertical flow, which yielded the critical gradient of 1.0. More work is required to assess the importance of the orientation of the test apparatus in these types of permeation tests. Skempton and Brogan's (1994) and Adel et al. (1988) results apparently indicate that the critical hydraulic gradient in stable materials can vary from 0.7 to 1.0, depending on the direction of fluid flow. However, for unstable materials the critical hydraulic gradient could be roughly 1/3 (vertical flow case) to 1/6 (horizontal flow case) of the normally anticipated threshold of 1.0, based on the theory of Terzaghi (1922).

Table 6 Filter boundaries for four soil groups (modified from Sherard and Dunnigan 1989)

Soil Group	Fine content by no. 200 sieve (%)	Filter boundary (D_{15b}) determined by test
1	85–100	D_{15b}/d_{85} = 7–12 (average value 9)
2	40–80	D_{15b} = 0.7–1.5 mm
3	0–15	D_{15b}/d_{85} = 7–10
4	15–40	Intermediate between groups 2 and 3, depending on fines content

Skempton and Brogan (1994, p. 457) proposed that there is a reduction factor, depending on the amount of stresses carried by an unstable fraction of soil, that reduces the critical hydraulic gradient necessary to commence piping;

$$i_c = \alpha\gamma'/\gamma_w \qquad (4)$$

where:
i_c = critical hydraulic gradient
α = reduction factor
γ = effective unit weight of soil
γ_w = unit weight of water.

The above equation would be applicable for internally unstable, gap graded materials prone to suffusion of matrix soil through a coarser grained, load-bearing skeleton. An important conclusion from this work is that the distribution of internal stresses may influence piping potential.

Kézdi (1979), Kenney and Lau (1985) and Åberg (1993) provide means to test soils for internal stability. A rule suggested by Skempton and Brogan (1994, p. 452) is that of Kovacs (1981), which states that materials having a uniformity coefficient of less than 10 are self filtering and materials with a uniformity coefficient of more than 20 are probably unstable. Soils that are gap graded may be prone to suffusion. Skempton and Brogan (1994, p. 452) termed this phenomenon- "segregation piping". If enough of the matrix grains are lost, collapse of the soil skeleton, sinkholes, or piping may ensue (McCook 2004; Kovacs 1981). Due to the lack of electro-chemical forces, segregation piping (suffusion) may be a more common phenomenon in noncohesive materials.

Recent work

Recent workers have re-emphasized Lane's (1934) distinction between piping and internal erosion, to help differentiate between the phenomena of flow through granular media versus flow through cracks or structural contacts (McCook 2004). Other recent work into piping phenomena has focused on the development of predictive mathematical

Table 7 Horizontal and vertical flow tests (Skempton and Brogan 1994). H/F is ratio of coarse to fine components

State	Horizontal flow (Adel et al. 1988)		Upward vertical flow (Skempton and Brogan 1994)	
	$(H/F)_{min}$	i_c	$(H/F)_{min}$	i_c
Unstable	0.25	0.16	0.14	0.20
	0.36	0.17	Not tested	Not tested
	0.50	0.17	Not tested	Not tested
	Not tested	Not tested	0.98	0.34
Stable	1.3	0.70	1.6	1.0
	Not tested	Not tested	2.8	1.0

models for particle transport and filter clogging (Reddi et al. 2000; Bonala 1997; Reddi and Kakuturu 2006a, b; Kakuturu 2003; Locke et al. 2001). This work stems from environmental applications in facilitated transport remediation techniques. Other continuing work is field research into dispersive soils and how this impacts natural soils. This is potentially a growing area as more development takes place in areas of unstable dispersive soils.

Jones (1981) provided an extensive review of literature related to piping. Although his primary interest was in soil piping from the soil geomorphology and hydrological viewpoints, his review covers many aspects of engineering related piping. Jones presents an argument that piping in natural soil has a significant role in geomorphological development in terms of drainage channel and valley network development, and plays a significant role in hydrological processes. His view is supported by the work of a number of recent workers (Valentin et al. 2005; Carey and Woo 2000; Derbyshire 2001; Holden and Burt 2002; Huddart and Bennett 2000; amongst many others). Jones' (1981) model for natural soil piping is shown in Fig. 4.

The formation of pipes, tunnels and jugs has been well documented in areas with dispersive soils. Jones (1981, p. 9) differentiates tunneling phenomena from classical piping phenomena on the basis that tunneling starts with water entering surface cracks "with velocities measured in cm/s and rarely in m/sec whereas piping occurs deep within a dam without necessarily following cracks and in flows of mm/s". He states that tunneling is only really effective in dispersive soils (Sherard, Dunnigan and Decker 1976). "Tunnelling begins upstream, with saturation settlement in poorly compacted soil creating a combined crack and discontinuity in permeability near the phreatic line as opposed to piping which begins by hydrodynamic pressure lifting particles at the downstream exit of seepage" (Jones 1981, p. 10). Hence, on the basis of this definition it appears tunneling is a vadose zone process.

Other recent work includes the collection of empirical data concerning dam failures by Foster et al. (2000b) and assessing the specific failure modes related to piping. This work, along with the statistical data of Foster et al. (2000a) is shedding some light on the causes of piping and causing engineers to reassess the failure mechanisms and genetic characteristics of piping (McCook 2004). For example, Foster et al. (2002) found that the hydraulic gradient is not nearly as important as other factors that lead to piping failures such as potential seepage paths along conduits and structures, or some other defect. This is a return to the earlier observations of Terzaghi (1929) that minor details in the foundation or a structure can trigger dam failures. These recent observations also support the earliest views of piping developed in India by Col. Clibborn and juxtapose the concept of backwards erosion piping later fostered by Terzaghi. The backwards erosion model is not as significant a cause of dam failures as the minor details that contribute to piping failures by internal erosion. Review of the database contained in Table 1 confirms this to be the case.

Recent work has also been conducted with respect to estimating the time of development of piping (Fell et al. 2003; Annandale et al. 2004). Numerical and laboratory investigations are currently being developed to come up with methods to better predict the timing of pipe development.

There is little work that has been completed with respect to constitutive models of piping. Tomlinson and Vaid (2000) performed experiments to determine the effects of confining pressure and seepage forces on piping and found confining pressures influence the critical hydraulic gradient to some extent. A mechanistic approach was taken by Kakuturu (2003), Reddi and Kakuturu (2006a, b) that predicts progressive erosion versus self healing in dams or other structures with soil filters, and is based on the rate of particle release, concentration of suspended particles and probability of entrapment using a capillary tube model of porous media. These models are applicable to evaluation of filter performance and in cases where the performance of environmental liners is to be evaluated. Yamamuro and Lade (1997a, b) developed a modified Single Hardening Model for static liquefaction of silty sands under low confining stresses that may have some application to piping research. However, to the writers' knowledge, no one to date has attempted to develop a constitutive model of piping that could be used in a continuum model to predict the piping behavior of dams.

Recommendations

Piping has been described in the literature as occurring within the vadose zone. The mechanism described for piping in the vadose zone precludes piping from forming in

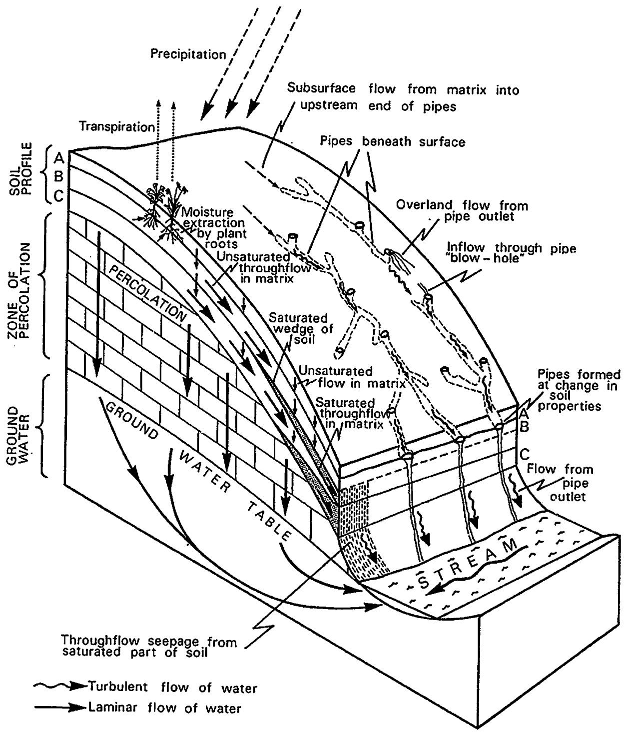

Fig. 4 Tunneling/piping phenomena common in the vadose zone of dispersive soils (Jones 1981)

the phreatic zone. Piping has also been described in the literature as strictly a phreatic condition due to adverse hydraulic gradients. However, recent literature has indicated that piping is common in areas where the hydraulic gradients are not that great. Engineering guidance on piping has taught engineers that hydraulic gradients less than 1.0 are generally safe; yet, there are some instances of piping where it has occurred at gradients as little as 0.17. Obviously, the term piping is describing a number of different phenomena, some which are only remotely related. More work is needed to differentiate these various causes of piping. Engineers need to have a common language and understanding of piping when discussing and researching piping phenomena and understand the various modes of piping and how to assess piping potential for each mode.

A worldwide clearinghouse of dam failure data, similar to the University of Stanford's National Performance of Dams Program should be developed. It is imperative that enough details regarding dam failures be recorded in one database, to allow thorough assessments of dam failure modes. We recommend that reports of dam failures should include;

(1) Date of first construction.
(2) Date of failure.
(3) Foundation characteristics.
(4) Dam characteristics (including if dispersive soils are involved).
(5) Exact location of first piping (or other failure) and its association with project structures (e.g. piping along conduit, piping under/along a structure, or piping into the foundation, etc.).
(6) Timing of the event if it can be reconstructed.
(7) Identify if the incident was caused by structural failure of a project feature. For example, if piping is associated with a conduit, if possible, the report should indicate if it is the result of conduit failure.

Unlike the detailed US National Transportation Safety Board (NTSB) investigations into aircraft failures, reports of dam failures are often not as rigorous. However, such thorough studies would greatly improve the overall safety of dams by increasing our understanding of the causes of dam failures.

Piping phenomena have been studied by a large number of researchers and the list of publications on this topic is voluminous. However, more research is still needed to fill in the gaps in our understanding of piping. Many existing structures were constructed without filters or with inadequate filters. Current methods for evaluation of piping potential are based on theories that were developed almost 100 years ago, which have proven to be inadequate when one considers the range of mechanisms that fall under the heading "piping".

Conclusions

Most of the previous work has focused on piping of cohesive materials, in particular dispersive clays, piping in natural soils, or filter criteria. Some preliminary work has been conducted on the statistics of piping failures, which hopefully will improve as dam failure reporting requirements develop further. Excluding the advances in filter engineering, there have been few significant advances with respect to piping in non cohesive soils since Lane's weighted creep method was published in 1934. Other than Moffat and Fannin (2006) and Åberg (1993), very little recent work has been done with respect to piping in cohesionless soils. Although some headway is being made with the recent focus on related failures, dams are still failing by piping and more work remains to be done in this field. Even in spite of the number of advances in our understanding of piping phenomena, there are still a large number of incidents that occur due to concentrated leakage or formation of sinkholes in embankment dams. This reflects a need for increased attention to the prevention of these potential failure modes.

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