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Soil Conservation Service, Fort Worth, TX

Prepared for:

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Abstract: Technical sessions consisted of pavement and horizontal construction, in situ stabilization, geosynthetics/soil reinforcement systems, dams and water conveyance systems, erosion control waste management, waste/byproducts stabilization/utilization, and soil stabilization with contaminated soils. To encourage input from the large number of attendees, a special workshop format was used.

PROCEEDINGS

SECOND INTERAGENCY SYMPOSIUM ON STABILIZATION OF SOILS AND OTHER MATERIALS

SPONSORED BY

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NOVEMBER 2-5, 1992
METAIRIE, LOUISIANA

PROCEEDINGS

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Sponsored by

U.S. Army Corps of Engineers
Bureau of Reclamation
Soil Conservation Service
Federal Highway Administration
Environmental Protection Agency
Naval Facilities Engineering Command

November 2-5, 1992
Metairie, Louisiana

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ACKNOWLEDGMENTS

The organizing committee for the Second Interagency Symposium on Stabilization of Soils and Other Materials (Charles McElroy, Soil Conservation Service; Newell Brabston and Norman Francingues, U.S. Army Corps of Engineers; Ed Gray and Theresa Casias, Bureau of Reclamation; Donald Fohs, Federal Highway Administration; Michael Jones, Naval Facilities Engineering Command; Trish Erickson, Environmental Protection Agency) would like to thank the following sponsoring agencies for their cooperation and publicity for the symposium:

Naval Facilities Engineering Command
Environmental Protection Agency
Federal Highway Administration
U.S. Army Corps of Engineers
Soil Conservation Service
Bureau of Reclamation

Special thanks go to the U.S. Army Corps of Engineers for making the local arrangements; to the Soil Conservation Service for printing and distributing the initial symposium announcements; to the Bureau of Reclamation for processing registrations, printing and distributing brochures, and preparing the workshop proceedings; and to the Environmental Protection Agency for preparing mailing labels. In particular, the efforts of the following Reclamation employees are appreciated: Barbara Prokop for preparing the proceedings and Leticia Jacobo for handling the administrative details of the workshop. Without Leticia's help, the symposium would not have been possible.

We would also like to thank the moderators, invited speakers, and authors. Finally, we would like to thank the attendees and participants in the workshop.

INTRODUCTION

Based on the excellent response from those attending the First Interagency Workshop in Denver, Colorado, on November 7-8, 1989, an organizing committee met in New Orleans, Louisiana, to determine interest in a second symposium. The committee consisted of Theresa Casias, Bureau of Reclamation Denver Office; Newell Brabston, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi; Charles McElroy, Soil Conservation Service in Fort Worth, Texas; Clayton Ormsby, Federal Highway Administration, McLean, Virginia; Trish Erickson, Environmental Protection, Cincinnati, Ohio; and Robert Becker and Charles Rome, New Orleans District, U.S. Army Corps of Engineers.

Through contacts with other Federal agencies and prospective speakers, the committee determined that there was sufficient interest in a Second Interagency Symposium. The New Orleans District Corps of Engineers agreed to host the workshop at the Landmark Hotel in Metairie, Louisiana, November 2-5, 1992.

The workshop format was established, speakers were contacted, and agreements were made for lodging and conference facilities. The organized committee finalized the details in late August in Denver, Colorado.

WORKSHOP FORMAT

Technical sessions consist of pavement and horizontal construction, in situ stabilization, geosynthetics/soil reinforcement systems, dams and water conveyance systems, erosion control waste management, waste/by-products stabilization/utilization, and soil stabilization with contaminated soils.

To encourage input from the large number of attendees, a unique workshop format is being used. Each technical session will begin with a 25-minute presentation by invited authors followed by a 20-minute discussion period where special problems and research needs will be identified. Each attendee will be asked to fill out a response card for each session, ranking the problems and research needs according to their own individual priority.

OBJECTIVES

The symposium is designed to meet the following objectives:

- Provide a forum for the exchange of soil stabilization technology among Federal, State, and local agencies, academia, and the private sector.
- Explore the various areas of application.
- Identify the stabilizers being used.
- Discuss various construction methods and procedures.
- Identify the laboratory and field tests used for design, QC (quality control), and QA (quality assurance).

- Identify problem areas and research needs, many of which are common to the various applications.

These proceedings will be distributed at the symposium. Following the symposium, all participants will receive a proceedings supplement by mail, which will include:

- A summary of each discussion period, which includes a list of problems and research needs and their respective priorities.
- Evaluation summary.
- Post-proceedings technical information.
- A list of participants so that technology exchange can continue.
- A reference list of applicable publications.
- Future plans.

AGENDA

SECOND INTERAGENCY SYMPOSIUM ON STABILIZATION OF SOILS AND OTHER MATERIALS

Monday, November 2, 1992

Registration 5:00 p.m. to 7:00 p.m.

Day 1 - Tuesday, November 3, 1992

Registration	7:00 a.m to 5:00 p.m.
8:00 - 8:10	**Introduction/Welcome**
8:10 - 8:30	**Keynote Speaker - Harvill Eaton - Louisiana State University**

Technical Session 1 - Pavements and Horizontal Construction
Moderator - Newell Brabston - U.S. Army Corps of Engineers

8:30 - 8:55	"Improvement of Marginal Materials by Stabilization," Raymond S. Rollings, Jr., U.S. Army Waterways Experiment Station, Vicksburg, MS.
8:55 - 9:20	"Recent Developments in Sulfate-Induced Heave in Treated Expansive Clays," D. N. Little, Texas A&M University. and Thomas M. Petry, University of Texas-Arlington, Arlington, TX.
9:20 - 9:40	**Break**
9:40 - 10:05	"Full Depth Reclamation of Asphalt Roads With Calcium Chloride," James B. Pickett, General Chemical Corporation, Parsippany, NJ.
10:05 - 10:30	"Lime-Fly Ash Aggregate Base and Subbase Courses," Alfred B. Crawley, Mississippi State Highway Department, Jackson, MS.
10:30 - 10:50	**Discussion**
10:50 - 12:00	**Lunch**

Technical Session 2 - In Situ Stabilization
Moderator - Michael Jones - Naval Facilities Engineering Command

12:00 - 12:25	"In Situ Ground Modification," Joseph P. Welsh, Hayward Baker Co., Odenton, MD.
12:25 - 12:50	"Liquefaction at Naval Station Treasure Island and Design of Mitigating Measures," Maurice Power, J. H. Egan, and M. L. Traubenik, Geomatrix Consultants, San Francisco, CA, and Richard Faris, Western Division, Naval Facilities Engineering Command, San Bruno, CA.
12:50 - 1:10	**Break**
1:10 - 1:35	"In Situ Testing Performed at Jackson Lake Dam and Mormon Island Auxiliary Dam," Michael G. Stevens, Lawrence Von Thun,and Jeffery Farrar, Bureau of Reclamation, and Matthew G. Allen, Sacramento District, Corps of Engineers, Sacramento, CA.
1:35 - 2:00	"Current Technologies in Ground Treatment and In-Situ Reinforcement," Donald A. Bruce, Nicholson Construction of America, Bridgeville, PA.
2:00 - 2:20	**Discussion**

Technical Session 3 - Geosynthetics/Soil Reinforcement Systems
Moderator - Charles McElroy - Soil Conservation Service

2:20 - 2:45	"History of Reinforced Walls in the Forest Service," John E. Steward, USDA-Forest Service, Washington, DC.
2:45 - 3:10	"Principles, Applications, and Testing of Geosynthetic Clay Liners," Walter E. Grube, Jr., Clem Environmental Corporation, Fairmount, GA.
3:10 - 3:30	**Break**
3:30 - 3:55	"Using Geogrids for Overflow Protection During Construction," Wendell Scheib, USDA-Soil Conservation Service, Chester, PA.
3:55 - 4:20	"Current Research on Geosynthetics," Barry R. Christopher, Polyfelt, Incorporated, Atlanta, GA, and Robert D. Holtz, University of Washington, Seattle, WA.
4:20 - 4:40	**Discussion**
4:40 - 4:50	**Announcements**

Day 2 - Wednesday, November 4, 1992

Technical Session 4 - Erosion Control
Moderator - Edward Gray - Bureau of Reclamation

8:00 - 8:25	"Basic Mechanisms of Erosion," Peter Bosscher, University of Wisconsin-Madison.
8:25 - 8:50	"Case History Streambank Protection - Bioengineering Evaluation, Southwest Washington," Leland Saele, Soil Conservation Service, Portland, OR.
8:50 - 9:10	**Break**
9:10 - 9:35	"The Expanding Role of Geosynthetics in Erosion and Sediment Control," Marc S. Theisen, Synthetic Industries Chattanooga, TN.
9:35 - 10:00	"Revegetation and Geosynthetic Studies at Ocean Lake to Control Shoreline Erosion," John E. Boutwell and Alice I. Comer, Bureau of Reclamation, Denver, CO.
10:00 - 10:20	**Discussion**

Technical Session 5 - Dams and Water Conveyance Systems
Moderator - Theresa Casias - Bureau of Reclamation

10:20 - 10:45	"Soil Stabilization Used as Remedial Measures for Water Retention and Conveyance Systems," Donald Snethen, Oklahoma State University, Stillwater, OK.
10:45 - 11:10	"Lime Stabilization of Levee Slopes," Robert L. Fleming, George Sills, and Edwin S. Stewart, U.S. Army Corps of Engineers, Lower Mississippi Valley, Vicksburg, MS.
11:10 - 11:30	**Break**
11:30 - 11:55	"Construction Control of RCC and Soil-Cement Using Heat of Neutralization Test, Nuclear Moisture Density Gauge, Vibrating Compaction Hammer, and VEBE," E. Kunzer and A. Benavidez, Bureau of Reclamation, Denver, CO.
11:55 - 12:20	"The Use of Posttensioned Anchors on the Arch Portion of Stewart Mountain Dam, Arizona: A Case Study Involving Precision Drilling," Donald Bruce, Nicholson Construction, Bridgeville, PA, and Robert Bianchi, Bureau of Reclamation, Denver, CO.
12:20 - 12:40	**Discussion**

12:40 - 2:15	Lunch/Luncheon Speaker: John Metcalf, Freeport McMoRan Chaired Professor, Institute for Recyclable Materials, Louisiana State University, Baton Rouge, LA.

Technical Session 6 - Waste Management
Moderator - Norman Francingues - Corps of Engineers

2:15 - 2:40	"Accounting for Boundary Layer Effects in the Modeling of Leaching from Monolithic Waste Forms," G. D. Allen and W. W. Pitt, Texas A&M University, TX.
2:40 - 3:05	"Retrofitting Perimeter Leachate Collection Systems for Existing Sanitary Landfills: A Challenging Problem," T. R. West, Purdue University, West Lafayette, IN.
3:05 - 3:25	**Break**
3:25 - 3:50	"Basic Research on S/S Mechanisms: Implications for Practice," Frank K. Cartledge and Marty E. Tittlebaum, Louisiana State University, Baton Rouge, LA.
3:50 - 4:15	"Environmental and Geotechnical Testing in Support of Waste Stabilization," Terry M. McKee, Browning-Ferris Industries, Houston, TX.
4:15 - 4:30	**Stretch**
4:30 - 4:55	"A Proposed Protocol for Evaluation of Solidified Wastes," Julia A. Stegemann, Wastewater Technology Centre, Burlington, Ontario, Canada, and Pierre L. Côté, Zenon Environmental Inc., Burlington, Ontario, Canada.
4:55 - 5:20	**Discussion**
5:20 - 5:40	**Announcements**

Day 3 - Thursday, November 5, 1992

Technical Session 7 - Waste By-Product Stabilization/Utilization
Moderator - Donald Fohs - Federal Highway Administration

8:00 - 8:25	"Assimilation of Wastes and By-products into the Highway System: Status Report and Regulatory Influences," Robert J. Collins, R. J. Collins & Associates, Springfield, PA.
8:25 - 8:50	"Recycled Rubber in Highway Construction and Maintenance," Jon A. Epps, University of Nevada, Reno, NV, and Amy L. Epps, University of California, Berkeley.
8:50 - 9:05	**Break**
9:05 - 9:30	"EPA Federal Procurement Guidelines and the Impact of RCRA Reauthorization," Barbara M. Scharman, Science Application International Corporation, Falls Church, VA.
9:30 - 9:55	"Florida Resource Recovery and Management Act," Lawrence L. Smith, Florida Department of Transportation, Tallahassee, FL.
9:55 - 10:15	**Discussion**
10:15 - 10:25	**Stretch**

Technical Session 8 - Soil Stabilization with Contaminated Soils
Moderator - Trish Erickson - Environmental Protection Agency

10:25 - 10:50	"Landfill Soil Stabilization - Implications With Leachate, Combustible Gases, and Physical Constraints to Revegetation," Thomas W. Hilditch, Gartner Lee Limited, Markham, Ontario, Canada.
10:50 - 11:15	"A Protocol for Evaluating Proprietary Solidification/Stabilization Technology Using Laboratory Leach Data," Mark Bricka and Norman R. Francingues, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
11:15 - 11:35	**Break**
11:35 - 12:00	Treatability of Metals in Soils by Solidification/Stabilization," P. M. Erickson, EPA, Cincinnati, OH.
12:00 - 12:20	**Discussion**
12:20 - 12:30	**Summary, Conclusions, Future Plans:** Newell Brabston, U.S. Army Waterways Experiment Station, Vicksburg, MS.

TECHNICAL SESSION 1

Pavements and Horizontal Construction

IMPROVEMENT OF MARGINAL MATERIALS BY STABILIZATION

By Raymond S. Rollings[1]

Engineers commonly consider stabilization as a means to enhance the structural strength of a pavement. However, an equally valuable application of stabilization is to upgrade an otherwise structurally unsuitable material to a degree that allows its use in a pavement structure or to improve its physical characteristics to enhance its constructability. Where the structural strengthening of the pavement is paramount (i.e., emphasis is on a stronger or stiffer structure that allows a reduction in pavement thickness), the emphasis is on use of reasonably good quality materials to allow a significant strength gain. On the other hand, stabilization of marginal materials is trying to enhance undesirable characteristics of existing natural material so that they can be used in pavement construction rather than incurring the cost and time required to import superior conventional materials.

This presentation will describe ongoing research at the Waterways Experiment Station investigating the use of stabilization techniques to upgrade marginal materials to allow their use in pavements. Among the topics of interest in this investigation are:

1. Stabilization of wet soils to allow construction.

2. Improvement of weak aggregate to allow their use with conventional asphalt and portland cement binders.

3. Overcoming adverse effects of excess plasticity, poor grading, excess fines, water sensitivity, and low strength of paving materials.

[1] USAE Waterways Experiment Station, Vicksburg, MS.

RECENT DEVELOPMENTS IN SULFATE-INDUCED HEAVE IN TREATED EXPANSIVE CLAYS

By D. N. Little[1] and T. M. Petry[2]

Abstract: While many applications of calcium-based stabilizers have not resulted in sulfate-induced heave or buckling, those which have cause substantial repair or replacement project costs. This paper presents the latest results of research into the solubility of sulfates, the determination of sulfate levels in soils, the types reactions that occur in treated materials, and successful treatment options for those with low sulfate levels. It explains why a 10:1 water to soil extraction is best, why levels of sulfates above 0.2 percent are potentially dangerous, and that double applications of lime can be used to treat some sulfate bearing clays.

INTRODUCTION

During the last decade there have been numerous incidents of sulfate-induced heave, or buckling of roadway pavement sections, both reported in literature and unreported. These have occurred in clay soils which have been stabilized using calcium based stabilizers of lime, Portland cement, or fly ash and lime fly ash mixtures. The basics of the deleterious sulfate-induced heave are discussed by Petry and Little (1991). What has not been so reported is the large number of projects where clays have been treated with these stabilizers and no sulfate induced heave has occurred. It is curious that, in many cases, these successful projects have been constructed in areas where the amounts of "soluble sulfates" were at levels thought to be potentially hazardous. Therefore, even though much has been written about the phenomenon of sulfate induced heave, much is left to be discovered and explained. The authors are heavily involved in the areas of better identification of potentially problematic clay soils and how to best stabilize these materials.

This report should be viewed as an update on the progress of research and findings concerning this problematic phenomenon. It includes information about the amounts of sulfates which are soluble, the methodologies of extraction and measurement of soluble sulfates, the results of recently completed testing of stabilization techniques in clays with relatively low levels of sulfates, X-ray diffraction and scanning electron microscope evaluation on specially treated clays, and rates of reaction studies currently underway.

DETERMINATION OF SOLUBLE SULFATES

The term "soluble sulfates" should be self explanatory, but a number of different techniques are used to extract these sulfates from the soil. It has been one of the tasks of ongoing research conducted by the authors to determine the best of these methods from the standpoint of how well it represented actual field conditions during stabilization and how well it consistently defines the potential for problematic behavior of treated materials.

In order to best represent the field situation, it is believed necessary to use water as an extraction fluid. The use of distilled-demineralized water has been selected since it does not introduce unknowns in the process and is fairly representative of the natural water conditions. The use of the same water which will

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[2] Professor of Civil Engineering, University of Texas at Arlington and member ASCE.

be used during stabilization would be best, but the source of this water is often unknown at the time of design and testing. Using ratios of water to soil which are close to field situations is necessary to provide proper representation. Ratios which have been included in recent studies are 1 to 1, 1 to 5 and 1 to 10.

Recent studies also have been conducted to determine how long the water should be in contact with the soil before extraction. Although these studies are at this time incomplete, they indicate that a contact time of between 1/2 to 3/4 hour provides essentially the same results as a 24-hour contact. In fact, there is some indication that longer contact time provides lower levels of measured sulfates. In all cases, the amounts of sulfates have been measured using the standard water Gravimetric Method with Drying or Residue to Determine Sulfates, as outlined in the 17th Edition of *Standard Methods for the Examination of Water and Wastewater* as edited by Clesceri, Greenberg, and Trussell and published in 1989.

The results of sulfate recovery level studies are shown in Table 1. Sulfate bearing compounds were added to a soil with low sulfate content and were then extracted using the water to soil ratios discussed above, and the values given in the table have been corrected for the amounts of sulfates occurring naturally in the soil. The soil was a mixture of 25 percent bentonite and 75 percent Eagle Ford clay soil. The levels of sulfate extraction for both the sodium and potassium compounds are very similar and those for the calcium sulfate compound follow the same trends but at a lower level. Two important trends can be seen in Table 1. There is a general relationship between the sulfates added and those measured. This means that the extraction methods identify the relative amounts of sulfates in the soil. The other fact shown in this table is that the 1 to 10, soil to water, ratio provides the most consistent and representative results.

Table 1. - Recovery rates for sulfates from clay soil.

p/m SO_4 in compound	1:1 extraction ratio (p/m)	1:5 extraction ratio (p/m)	1:10 extraction ratio (p/m)
Na -1350	73	134	469
Na -4730	203	2630	4114
Na -33810	1640	17734	23874
K -1100	75	102	562
K -5510	284	2494	4457
K -27570	1893	18335	24603
Ca -1410	35	213	981
Ca -4940	338	2890	4737
Ca -35280	779	7951	15466

STABILIZATION RESEARCH RESULTS

In order to provide a stable subgrade when treating a clay soil with calcium-based stabilizers, two very important reactions are needed. The first is modification of the clay so that it acts like a silty or sandy material. It has been determined that this most certainly happens when clays containing significant levels of sulfates are treated. This beneficial change of behavior does not, however, provide a stable material in these cases. The second, and perhaps most important reaction is the pozzolanic reaction. This reaction provides further improvement of the clay and, more importantly, "glues" the newly formed flocks of material together into a stable mass. It has been determined by researchers of the sulfate induced heave phenomenon, that when ettringite is formed, the pozzolanic reaction does not take place, or is at least substantially inhibited. In these cases the beneficial product which holds the material together is missing and the detrimental product which splits the material apart is formed.

It is, therefore, central to the proper stabilization of these clays with significant levels of sulfates that the pozzolanic reaction be promoted. Currently there are three concepts by which this may be accomplished. The first of these concepts employs the pretreatment of the clay with chemicals which tie up and precipitate the sulfates out of the pore water system. The second concept employs a double treatment of stabilizer. The first treatment is envisioned to use up the sulfates in the development of ettringite. An appropriate time is allowed to pass and then the second treatment is applied. The delay period between the first and second treatment affords appropriate time for development of the expansive mineral while the treated soil is still in the uncompactive state. The second treatment is employed to enhance the pozzolanic reaction necessary for long-term stability. The third technique includes the use of an additive or additives to promote the pozzolanic reaction and thereby reduce or prevent the formation of ettringite.

The second of the treatment techniques described above, that of double application, has reportedly been used for some time with good success by the TxDOT (Texas Department of Transportation). The problem associated with building on these successes is that the facts of the cases have not, for the most part, been recorded or reported. Recent uses of this method on highway subgrades in North Central Texas appear to be working, but the exact methodologies used are quite variant and project specific. In one case the material in question was finally treated three times and the total lime applied was 21 percent.

Research recently completed has included investigation of the parameters of the double lime application methodology and the use of additives to promote pozzolanic reaction in lien of ettringite formation. The applications were evaluated using a three-dimensional swell test. The test procedure starts with clay material pulverized to meet field criteria, treated with the stabilizing agents and additives, and compacted, using standard Proctor effort (ASTM D698), into a 6-inch-diameter, 4-1/2-inch-high cylinder. This cylinder is placed on a porous stone and wrapped horizontally with a wicking geotextile filter. The outside of the sides and top of the specimen are then sealed with a triaxial membrane and plastic wrap. This assembly is placed into a deep dish and put into a 120 °F oven for 2 days of curing. Following this curing step water is introduced into the bowl and maintained at a level slightly above the porous stone. The height and circumference of the specimen are monitored daily. Selected results of swell testing are shown in Table 2.

The soil utilized for this part of ongoing research was provided from the new Denver Airport area. Using a 1:10 extraction procedure the soluble sulfates ranged from 951 to 5,987 p/m, with a mean value of 2,775 p/m. This material has average properties, untreated, of a liquid limit of 68.9, a plastic index of 41.7 and linear shrinkage (by the Texas bar shrinkage method) of 28.1 percent. When treated these properties are 54.4, 16.6, and 7.1 percent, respectively, after a 24-hour cure. The treatments applied to this clay soil included hydrated lime of 6, 8, and 10. Double treatment combinations used were 3, 6 and 8 percent lime followed by 3, 4, and 5 percent, with delays between treatments of 7, 14, 21, and 35 days. Additives used with the lime percentages of 5 and 6 percent were potassium hydroxide, signified in Table 2 with "K," and a proprietary additive which is signified by an "A" in the table. The percentages of these additives where held to small amounts to represent realistic field treatments.

As indicated in Table 2 the Denver clay when untreated has a potential swell of almost 10 percent, and when treated with 6 percent lime it is almost 7 percent. The swell becomes over 9 percent when 10 percent lime is added. This is obviously not what one would expect with normal stabilization and is indicative of sulfate induced swell. However, it should be noted that even with the sulfate swell that occurs with higher concentration of lime, the swell is less than that in the natural clay. Among the double lime treatments results shown in Table 2, one will note that the optimum time between treatments is approximately 21 days, since the swell does not change as this delay time is increased. This material can, evidently be stabilized using an initial application of as little as 3 percent lime followed by a 3 percent, second application. Other double application rates are not as favorable.

It is important to note that the philosophy of a double application is to provide sufficient moisture and calcium to produce any deleterious product which may develop during the "delay" period prior to compaction. Ettringite forms rapidly, and if sufficient quantities of water and lime are added initially and mixed well, formation during a delay period of only a few days should occur. A danger in this approach is that too little water could be added initially only partially solubilizing the available sulfates. If this occurs, a low-sulfate form of calcium-sulfate-aluminate-hydrate (C$\bar{S}$AH) could be formed. Upon release of high levels of sulfate, such as during a subsequent heavy rain, the low-sulfate C$\bar{S}$AH may transform into a high-sulfate form of C$\bar{S}$AH which can result is substantial expansion (Petry and Little, 1991).

The second application of lime following the delay period may be necessary to "knit" the soil together through the development of a pozzolanic matrix. This second application may not be necessary or even advisable if the required delay period is short (less than 7 days). The only way to know the proper delay period and dual application rate is through testing.

The addition of potassium hydroxide, to enhance pH, does some good by itself but does not produce a very stable product. The use of the proprietary additive, however, appears to be quite promising, since it is necessary to use only 0.5 percent to reduce swell while using 1.5 percent is only somewhat better. The combination of lime, potassium hydroxide, and the additive is also very promising and may well be the best solution to reduce or eliminate horizontal volume increase in this particular soil. These methodologies provide several possible stabilization techniques yet to be tested in the field.

X-RAY DIFFRACTION AND SCANNING ELECTRON MICROSCOPY STUDY OF DENVER SOIL

The Denver soil previously discussed was stabilized with various combinations of lime and additives and was evaluated using x-ray diffraction (XRD) and scanning electron microscopy (SEM). The purposes of this examination were to (1) determine if evidence of mineralogical changes exists due to lime reaction with the clay mineral, (2) determine, to the degree possible, the composition of the "products" of the reaction between the lime and the clay mineral, and (3) evaluate the extent of the lime-soil reaction as influenced by the presence of sulfates and the amount of lime.

X-ray diffraction spectra such as those shown in Figures 1a and 1b demonstrate that a mineralogical change does occur in the Denver clay upon stabilization with lime. In Figure 1a the XRD spectra shows the strong peak at approximately 14.4 angstroms which is definitive of the d-spacing of a smectite-type mineral. The presence of smectite is further substantiated by the presence of the 17.6 angstrom peak upon saturation with ethylene glycol. The presence of the highly polar ethylene glycol expands the layers from approximately 14.4 angstroms to approximately 17.6 angstroms. Final substantiation of the presence of the smectite mineral is that the smectite peak disappears upon heating at 550 °C.

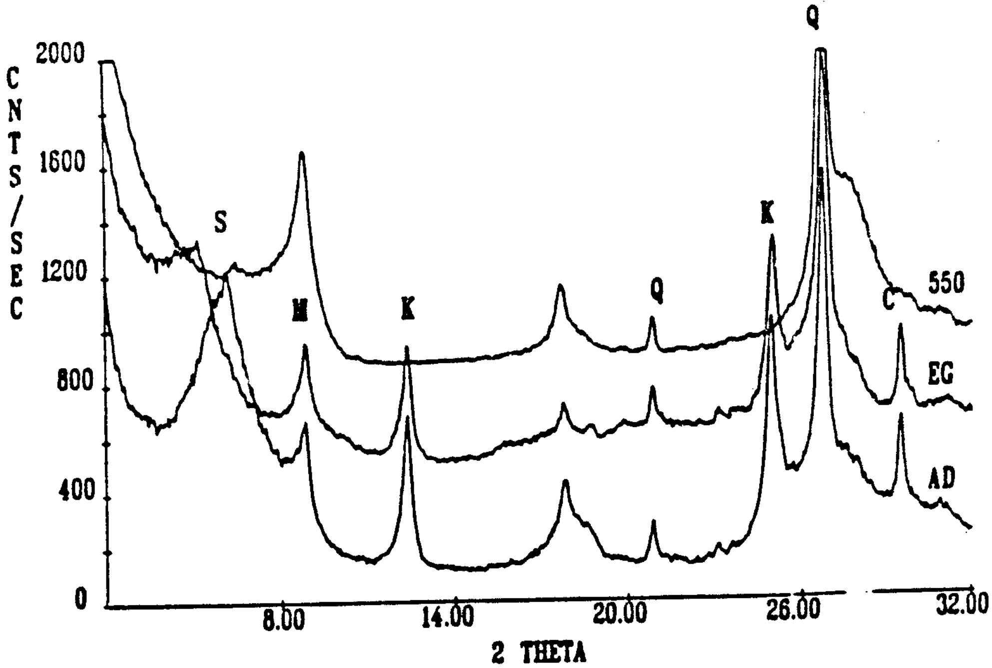


Figure 1a. - XRD spectra for clay of Denver soil (AD = air dry, EG = ethylene glycol solvation, 550 = dried at 550 °C, S = smectite, M = mica, K = kaolinite, Q = quartz, and C = calcite).

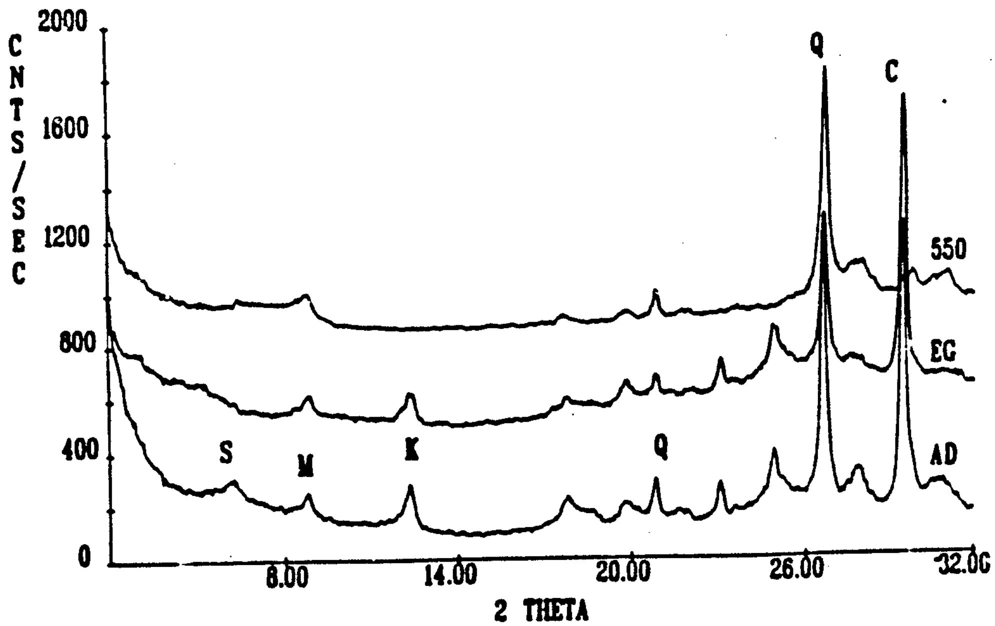


Figure 1b. - Denver soil clay fraction) following stabilization with 6 percent lime.

Table 2. - Swell test results for Denver soil.

Treatment	Duration days	Max. vert. heave (%)	Days	Max. hor. heave (%)	Days
Natural	47	9.96	22	7 77	12
6% Lime	47	6.93	34	3.19	12
8% L	47	8.80	41	3.59	12
10% L	47	9.33	34	3.79	30
3L+3L-7D	3S	3.91	25	2.00	14
6L+3L-7D	39	3.82	25	2.20	15
3L+3L-14D	48	1.60	29	0.40	3
6L+3L-14D	46	3.02	26	1.00	26
3L+3L-21D	55	1.87	21	1.00	16
6L+3L-21D	56	2.93	26	1.20	17
3L+3L-35D	69	0.89	16	0.40	2
6L+3L-35D	69	2.49	15	1.00	28
3L+4L-14D	48	2.76	21	1.00	8
3L+4L-21D	55	1.78	21	0.60	20
3L+4L-35D	69	1.60	23	0.60	26
6L + 2K	47	2.76	21	3.59	15
5L + 0.5A	39	0.27	19	0.60	8
6L + 0.5A	39	0.36	12	0.80	18
5L + 1.5A	39	1.07	12	0.60	32
6L + 1.5A	39	1.60	12	1.00	22
5L+4K+.5A	39	0.89	13	0.60	21
6L+4K+.5A	39	0.71	13	0.40	20
5L+4L+1.5A	39	0.53	12	0.00	0
6L+4K+1.5A	39	0.89	12	0.20	1

*D = Delay

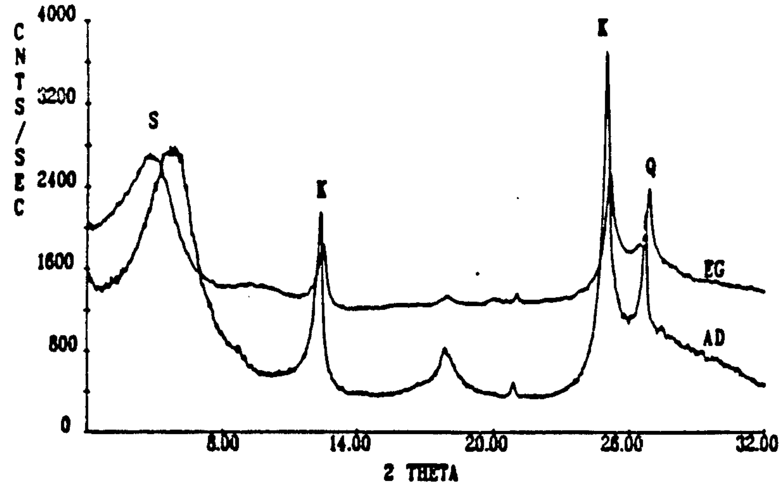
When the clay fraction of the Denver soils is evaluated following the addition of 6 percent lime, Figure 1b, it is clear that the reaction between the soil and lime has resulted in the collapse of the smectite peak. A subtle peak exists in the air dry sample; however, this peak is only about 20 percent of the intensity of the clay fraction of the untreated Denver soil.

The difference between the two XRD spectra of the untreated soil and the lime-treated soil could be due to (1) a true mineralogical change in the clay due to lime stabilization, (2) a surface coating of perhaps calcite which could mask the clay mineral, or (3) the effect of the lack of orientation of the clay mineral due to the effects of cation exchange and flocculation/agglomeration. In order to evaluate the potential of a coating effect, the lime-soil mixture was subjected to an acid wash with hydrochloric acid. The XRD following the acid wash showed an increase in the intensity of the smectite peak but only to about 30 percent of the original level. Figure 2 summarizes the XRD spectra for a lime-treated Beaumont, Texas, clay. The same sequence of events is seen. The strong smectite peak is essentially eradicated by lime stabilization and the peak intensity is only partially recovered after acid washing. Since the XRD calcite peak was fully eradicated with the acid wash for both the Denver and Beaumont soils and the smectite peak was only partially recovered, it was concluded that the calcite coating of the clay minerals accounts for only a small part of the reason for the collapse of the smectite peak due to the addition of lime. It was further concluded that a mineralogical change does indeed occur due to a pozzolanic-type reaction between the clay mineral and the calcium hydroxide.

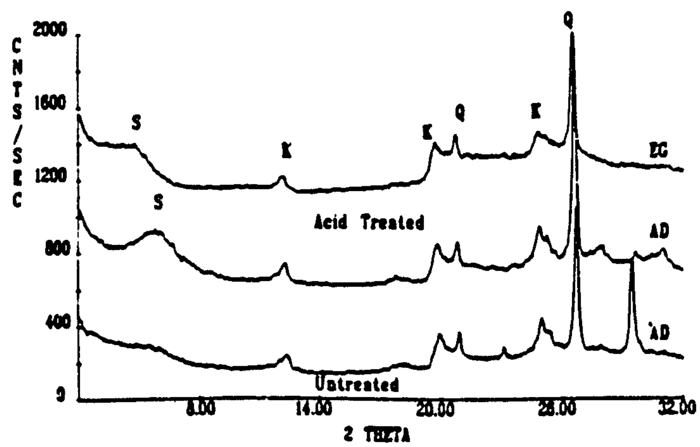
The reaction between the clay mineral and the lime was further evaluated using aqueous chemistry. In this analysis the concentrations of silicon, aluminum, calcium, bicarbonate, and carbonate ions was measured as a function of lime content and based on the presence of other additives. The study demonstrated that the solubility levels of silica and alumina do not change significantly enough in the aqueous solution to account for a substantial strength gain due to the development of pozzolanic-type cementitious products.

As a follow-on SEM analysis was performed on the lime stabilized Denver soil. This analysis clearly demonstrated the visual presence of semi-crystalline to crystalline-type product emanating from the surface of the clay. Energy dispersion spectra (EDS) was used in concert with the SEM to identify the elements present in the product of the lime-soil reaction. This EDS analysis showed the presence of silicon, aluminum, sulfur, and calcium. Most of the product evaluated was comprised of silicon, aluminum, and calcium, whereas about 30 percent of the time the product contained some levels of sulfur. The authors realize that it is not possible to definitively identify the minerals presence due to the background effect of the clay which is rich in all the elements mentioned except calcium. In the EDS evaluation every effort was made to concentrate on crystalline to semicrystalline product which formed at edges of fracture faces and extended away from the clay surface. Thus the product evaluated should have been minimally affected by the clay background. EDS was used only to try to differentiate between reaction product containing sulfur and product without sulfur.

The conclusion of the SEM, EDS, aqueous chemistry XRD analysis is that the reaction between the calcium hydroxide and the clay is to form calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) product. Occasionally these reaction products contain sulfur. Obviously, the presence of sulfur in conjunction with alumina, calcium and water could indicate the presence of calcium-sulfo-aluminate-hydrate minerals. When these minerals occur in the monosulfate or trisulfate form or transition back and forth between these two forms they can be very expansive and disruptive (Petry and Little, 1991). However, Diamond and Kinter (1965) and Mitchell and Herzog (1963) have shown that various levels of sulfur are often present in the pozzolanic or cementitious product of lime-stabilized soil and portland cement-stabilized soil. Thus the CAH and CSH minerals may be quite variable in terms of morphology and in terms of chemical composition. This point is evident in terms of the variable degree of crystallinity of these reaction products. Although the ettringite mineral has a definitive chemical composition, it is possible that amorphous compounds between the traditionally accepted pozzolanic reaction products of CSH and CAH the ettringite-type minerals exist.



(a)



(b)

Figure 2. - Beaumont, Texas, clay (a) natural soil, (b) lime-stabilized soil (6 percent lime).
The spectra labeled untreated means lime-treated but no acid (HCl) treatment. Acid-treated means HCl wash.

The pozzolanic reaction which is responsible for the strength gain in lime-stabilized soils and for much of the consistency changes in these soils is a surface based reaction. This is not a new determination but supports the previous research of Diamond and Kinter (1965), Eades (1962), etc. The surface reaction between the lime and the clay in the high pH environment is substantiated by XRD, EDS, SEM, and aqueous chemistry information. Probably the most correct scenario for the reaction between the lime and the clay surface is that either a monolayer of calcium hydroxide molecules reacts with the clay surface or a critical concentration of calcium cations (cation crowding) is attracted to the surface of the clay, and these cations react with the soil-silica and/or soil-silica released from the clay in the high pH environment induced in the lime-saturated pore water environment.

Whether the clay surface-lime reaction is the result of the interaction between calcium hydroxide molecules and soil-silica and soil-alumina as proposed by Diamond and Kinter (1965) or whether the interaction is between the soil-silica and soil-alumina and calcium ions as proposed by Ho and Handy (1963) and others, is not of great consequence here. Of prime importance is that the reaction occurs at the surface, probably continues into the clay mass as the reaction products "peal off" and release new, unreacted clay and is high dependent on the efficiency of mixing. This surface reaction between lime and clay under solution attack in the high pH environment obviously occurs to a considerable degree in the Denver soil.

XRD analysis did not reveal the presence of ettringite. Perhaps this is because the percentage of ettringite necessary for detection is in the range of 5 to 10 percent. In order to detect the presence of pozzolanic product as well as ettringite and other forms of calcium-alumino-sulfate-hydrate minerals, SEM scanning was used. The SEM analysis was performed on the natural soil and soils with 6 percent lime, 8 percent lime, 10 percent lime, and double combinations of lime with mellowing or delay periods of from 3 to 35 days. Although the SEM visual technique is certainly subjective in terms of the ability to identify the presence and quantity of pozzolanic product and other reaction products, it was obvious in this analysis that as the amount of lime increased, the amount of visible reaction product increased. Since the reaction product did in a significant number of instances include sulfur and thus may have been some form of calcium-aluminate-sulfate-hydrate or calcium-silicate-sulfate-hydrate, too much of this product could result in expansion.

As a result of the subjective SEM scanning approach, the following conclusions are drawn:

1. As more lime is used, the amount of reaction product increases noticeably. This may be good if the increase in the amount of the product improves strength ar ' durability or it <u>may</u> be deleterious if the product contains sulfate to the level that expansion occ. In the case, the optimum lime content should be defined as that at which unconfined compressive strength peaks - representing maximum pozzolanic strength development.

2. Based on the morphology of the reaction product seen during the SEM scanning analysis, the predominate reaction product is calcium-silicate-hydrate (CSH) which occurs at the surface of the clay during a surface reaction between the silica-alumina-rich smectite mineral and the calcium hydroxide in the high pH environment (fuzzy or burry and long-slender substance). However, a substantial amount of stubby, stalk-like product is also evident. Perhaps this is ettringite or a form of calcium-aluminate-sulfate-hydrate mineral or calcium-silicate-aluminate-sulfate-hydrate mineral.

Figure 3 is an SEM micrograph of the Denver clay. Notice that morphology of the clay is consistent with the XRD spectra indicating a significant amount of the smectite clay mineral. Figures 4, 5, 6, and 7 illustrate SEM micrographs of lime-stabilized Denver clay. In Figures 4 and 5 the fuzzy and needle-shaped reaction product seems to emanate from the clay surface which is "attacked" by the calcium hydroxide in the high pH environment. The CSH and CAH seems to grow from the clay surface and ultimately knit the clay surfaces together to form a cemented matrix, adding strength and durability. Figures 6 and 7, the shorter, stalk-like crystals of the sulfur containing reaction product is shown. This may be responsible for the expansion noted in Table 2 with increased lime content.

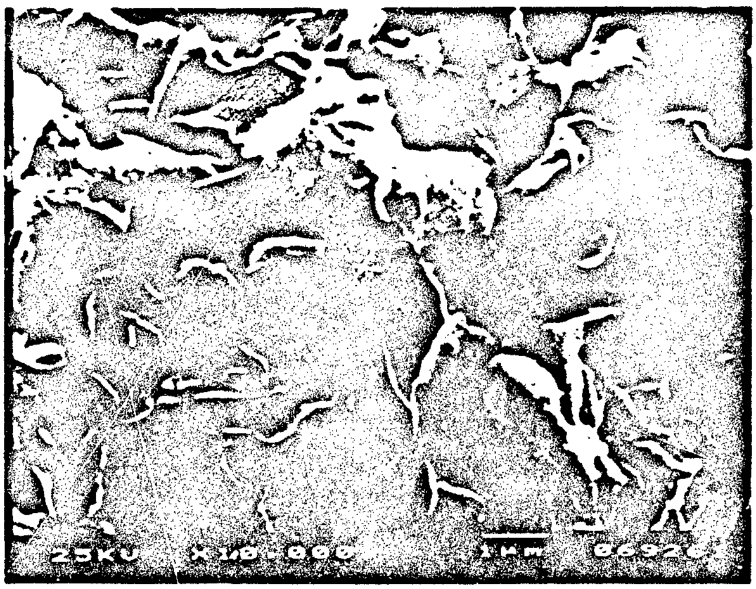


Figure 3. - SEM micrograph of Denver clay.

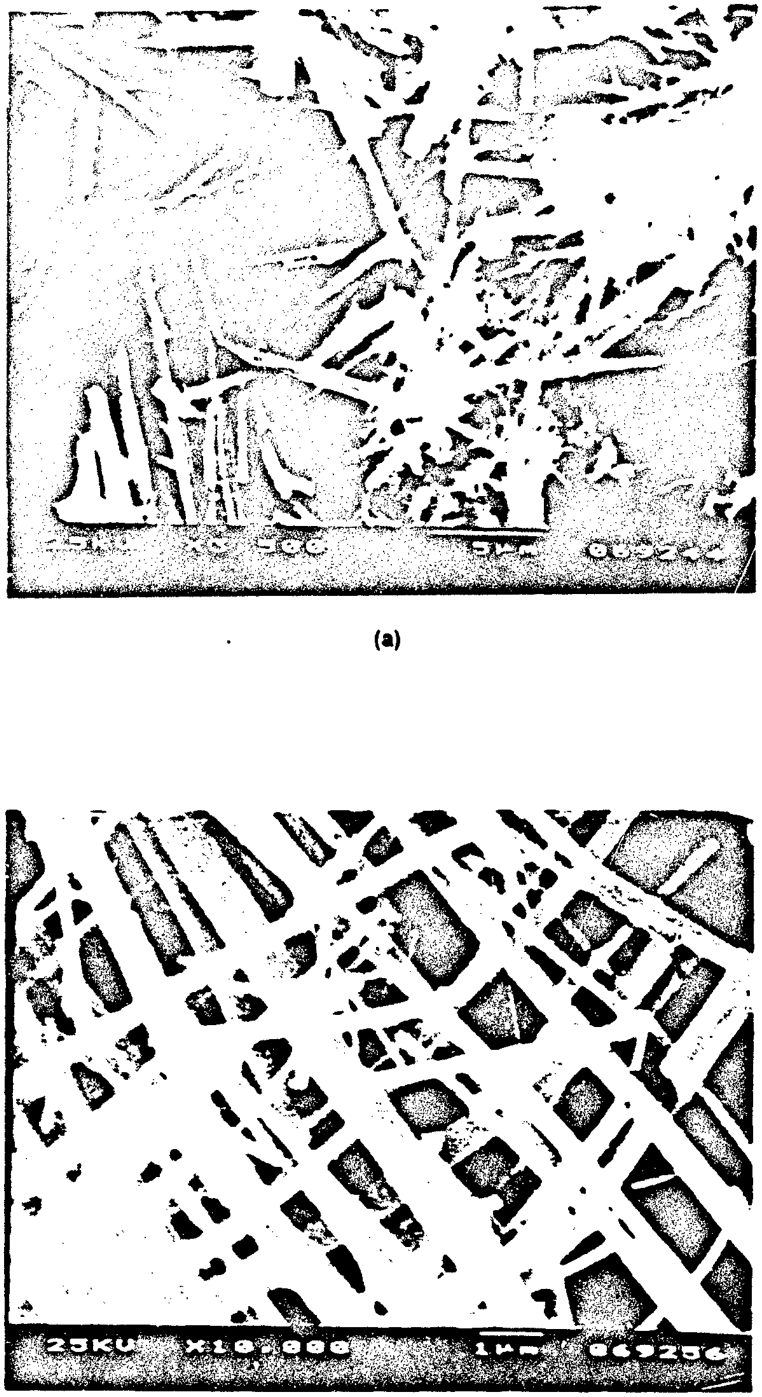
COMPRESSIVE STRENGTH OF DENVER SOILS STABILIZED WITH LIME

The lime-stabilized Denver soil with an approximate soluble sulfate content of between 0.3 and 0.5 percent demonstrates all the evidence of pozzolanic reaction based on XRD, SEM, and EDS analysis. However, the proof of a significant pozzolanic reaction is based on the degree of compressive strength gained through lime-stabilization. The degree of benefit gained through strength gain through lime-stabilization becomes complex and difficult to evaluate when sulfates are present due to the potential development of the expansive calcium-sulfo-aluminate minerals or some of the amorphous compounds which may be intermediate forms of calcium-silicate-sulfate-hydrates.

(a)

(b)

Figure 4. - SEM micrograph of pozzolanic product due to lime stabilization of Denver clay.



(a)

(b)

Figure 5. - SEM micrograph of stubby crystals indicative of calcium-alumine-sulfate minerals which are potentially expansive.

A hypothesis is that the CSH and CAH pozzolanic products produce considerable compressive strength and help develop a matrix locking the stabilized clay aggregates together in a fashion which helps prevent swell due to any portion of the unstabilized clay which may take on water or due to the formation of and hydration of any expansive calcium-sulfo-aluminate hydrates. Evidence in Table 2 and form unconfined compressive strength testing from the Denver soils indicates that these soils are pozzolanically reactive to a considerable degree and that this pozzolanic reaction binds the soil matrix together in a fashion the successfully resists expansion as long as the degree of development of the expansive calcium-sulfo-aluminate-hydrate minerals are held in check. Apparently from Table 2, this reaction is optimized with a total lime content of about 6 percent applied in two applications of 3 percent each with a delay time between applications of at least 21 days.

Even when the lime is applied in one application, the swell is substantially less than for the natural soil without lime. This fact together with the fact that considerably high compressive strengths are achieved with lime stabilization of the Denver soil indicate that the majority of the lime reacts to form pozzolanic product which stabilizes the soil through mineralogical alteration and development of a cementitious matrix. Very little reaction product with expansion potential is apparently produced. With an additive rate of 6 percent lime, the Denver soil achieved an unconfined compressive strength of approximately 450 lb/in^2 following 28 days of moist curing at 77 °F (25 °C).

CONCLUSIONS

1. Sulfates should be extracted using distilled-demineralized water as a fluid and the ratio of soil to water should be 1:10.

2. The level where soluble sulfates should be considered a potential problem is as low as 2,000 p/m. However, the potential for deleterious expansion also depends on the percentage of clay in the soil. Soils with relatively high levels of sulfates but relatively low clay content may not be problematic.

3. When the level of soluble sulfates is below 5,000 p/m, a double application of lime can often be effectively used to stabilize as long as the percentage of lime is relatively low. The delay period between applications should be determined for each individual soil. The delay period can range from as little as 3 days to over 21 days.

4. The proprietary additive utilized in this research was found to be very effective in reducing sulfate-induced heave to very low levels in the Denver soil tested. In addition, may be some benefit in adding potassium hydroxide.

5. Determination of an optimum percentage of lime is important to insure adequate stabilization in terms plasticity reduction in clays and development of strength due to pozzolanic reaction. It is also important in sulfate-bearing soils not to use more than the optimum amount of lime as the excess stabilizer could fuel the development of potentially expansive calcium-sulfo-aluminate-hydrate minerals.

6. When employing the double application technique, it is important that sufficient quantities of water are used to solubilize the sulfate to react with the soluble aluminate from the clay and with the calcium from the lime to form ettringite during the delay period. If this is not done transitions between the monosulfate and trisulfate form of the calcium-sulfate-aluminate-hydrate could prove disruptive.

7. Certain sulfate-bearing soils can and have been successfully stabilized with lime using proper construction technique.

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Full Depth Reclamation of Asphalt Roads With Calcium Chloride

By James B. Pickett[1]

Abstract: By reusing the asphalt dug up during reconstruction of deteriorating roads and blending it with the gravel base and liquid calcium chloride, states, cities, and towns are able to rebuild their roads at a 50-percent savings.

The implementation of a full depth reclamation program with calcium chloride is a viable, cost-effective solution to the deterioration of the roads. Many of the asphaltic roadways are in the advance stages of deterioration due to aging, base problems, and drainage. Some roads have been maintained with the application of a periodic seal coat or an overlay. Because overlays only last a limited time before cracking begins to show through and because of insufficient funding to completely reconstruct the roadways, full depth reclamation with calcium chloride is helping states, counties, cities, and towns.

The process consists of pulverizing the existing deteriorated surface and gravel base from a depth of 4 to 12 inches. The mass of asphalt, stone, gravel, and dirt is then distributed evenly along the road. A distributor truck then applies the liquid calcium chloride at a rate of 0.75 gallon per square yard. After the application of liquid calcium chloride, the road is repulverized, graded, and rolled. The distributor truck then applies liquid calcium chloride at a rate of 0.25 gallon per square yard to prevent raveling.

Some important savings in full depth reclamation of asphalt roads with calcium chloride when used in lieu of traditional reconstruction methods are:

1. Natural resources - Utilizes in situ materials such as asphalt surfaces courses, pulverizing it with the gravel base course.
2. Energy - No oil used.
3. Time - Average time to do 1 mile is 2 to 3 days.
4. Uniform moisture control.
5. Increased density.
6. Controlled curing for increased stability.
7. Dust-free surface.
8. Frost protection.
9. It is economical.

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INTRODUCTION

Full depth reclamation (FDR) - a method which is what recycling is all about -- savings. A savings in time, a savings in materials, a savings in the use of additives.

FDR is a reclamation technique in which the full flexible pavement structure and a predetermined portion of the underlying base materials are uniformly crushed, pulverized, blended or sized resulting in a stabilized base course. This method will significantly extend the life of the road and the amount of road work done with a budgeted dollar. It is with this objective in mind that we look to the use of calcium chloride as a stabilizer by recycling it with asphalt surfaces and base materials dug up during reconstruction. This combination of a crushed surface and base material with the addition are mixed to facilitate the road builder's oldest form of stabilization: thorough mix and uniform compaction.

In the field of highway engineering the stabilization is recognized as including all procedures for improving the performance of the asphalt, soils and aggregates used in road construction and maintenance. A highway material may be considered stable if it exhibits a high degree of durability or permanence under traffic, moisture fluctuations and frost action in colder climates.

The concept of stabilization involves improvement of soil and aggregate material by one or more of the following procedures: drainage corrections, compaction, gradation changes, use of additives.

The objective of the highway superintendent employing any combination of the various stabilization procedures can be divided into categories:

1. Improve the bearing capacity of existing subgrade soil.

2. Modify the physical properties of an unsuitable or questionable base course material.

3. Obtain maximum performance from suitable base course material.

4. Reduce the total pavement thickness for a given traffic load.

5. Provide a satisfactory wearing surface for low traffic volume secondary roads.

The properties of calcium chloride make it a particularly useful additive in the stabilization field. Four physical properties are instrumental in calcium chloride performance in stabilization applications:

1. Calcium chloride's attraction for moisture.

2. Calcium chloride's low vapor pressure which enables the chemical to resist evaporation.

3. A calcium chloride solution has a high surface tension, providing an ability to bind aggregate particles together.

4. A calcium chloride solution has a strong moisture film, the lubrication of the aggregate particles help in the compaction. The result is greater density through more effective compaction.

The calcium chloride attraction for moisture and its low vapor pressure maintain uniform moisture content, while the recycled material is graded, rolled, and cured. Maintaining the optimum moisture content in the base is the chemical's greatest contribution to the stability of the pulverized material.

Most FDR of asphalt roads with calcium chloride is done on roadways with advanced stages of deterioration, alligator cracks, rutting, frost heaves, potholes, etc. The work consists of pulverizing the existing surface and blending the crushed surface with its gravel base to a desired depth, adding 0.75 gallon of liquid calcium chloride per square yard, repulverizing it, grading it, rolling it, adding 0.25 gallon of liquid calcium chloride per square yard to prevent raveling and letting it cure. Work has been done with Bomag MPH 100 recyclers, Caterpillar R.R.250, CMI R5500, and Barber Greene RX-40 Dynaplane machines with savings in time, material, additives, and money.

In 1978 the town of Niskayuna, New York, stumbled onto this concept for one of their most serious road maintenance problems. Niskayuna's Highway Department had always saved and reused old blacktop to create a new aggregate base and had been using calcium chloride for dust control for a number of years. It was a natural next step to combine the two operations. When Niskayuna combined the two operations, the results were impressive. Niskayuna's biggest budget was blacktop, but the budget did not keep pace with the price of blacktop. It was a losing situation until they hit upon the calcium chloride FDR solution. Niskayuna scarified out blacktop and the gravel base and passed it through a crusher and redeposited this homogeneous mass of crushed asphalt and gravel over the sub-base. After the homogeneous mass was laid out, 35 percent liquid calcium chloride was applied at the rate of 0.60 gallon per square yard. Three days later they applied liquid calcium chloride again, applying it in two passes at a rate of 0.25 gallon to per square yard during each pass. Niskayuna then graded and rolled the road. Within a few days the road started to set up. A finished surface course of 1-1/2 inches of plant mix topped the road.

The following years, 1979 and 1980, Niskayuna did one mile each year with similar results. In 1981, Niskayuna did 5 miles. By scarifying, combining the blacktop and the gravel base, crushing them, then adding the liquid calcium chloride to bind the aggregate, they were getting reconstructed roads at half the material cost, and they were staying within their budget.

In 1983, the town of Colonie, New York, used a Bomag MPH 100 recycler to a depth of 12 inches, 6 to 8 inches of which was asphalt which was pulverized with 4 inches of gravel base. On the second pass with the Bomag recycler, as asphalt emulsion was added through the Bomag metering system, problems started to develop in the application of the emulsion. How much emulsion to add? They were constantly stopping the operation of the Bomag machine to adjust the amount of emulsion to add. Either too much was added to the recycled material which made it blend through, or not enough to bind the recycled aggregate. With the success that its neighboring Town of Niskayuna had with liquid calcium chloride, a decision was made to try the liquid calcium chloride. After the asphalt road was pulverized to a depth of 12 inches, graded and rolled, liquid calcium chloride was applied to the road through an Etnyre distributor at a rate from 0.60 to 1 gallon per square yard. Within a few days the road started to set up. One month later the road looked just like a paved road. Later that summer that road was paved. The ADT of that particular road is 1,500 to 2,000.

After the Colonie, New York, work, several towns in the area tried out the liquid calcium chloride. In the town of East Greenbush, New York, they substituted the liquid calcium chloride for an emulsion additive. The day the Bomag MPH 100 recycler started to grind up the road, they had assurances that the liquid calcium chloride could do the job and would cost about sixty percent less than the emulsion. During the recycling, the road was kept open and it was watched carefully. The Director of Public Works was surprised that the road set up so hard. They waited four weeks before applying a double seal of oil and stone. During the summer of 1983, due to the success of their test road, the Town of East Greenbush, New York, reconstructed an additional 3 miles of roads with the Bomag MPH 100 recycler and liquid calcium chloride. Over the last several years the town has done 20 miles. Some of the roads were not surfaced, and to this day they are still not surfaced, but remain a smooth aggregate road.

For over 15 years the town of Sempronius, New York, experienced problems with a heavily traveled oil and stone road. Every year they were faced with repairing potholes, alligator cracks and ruts. The road surface always remained a problem because they were not convinced they had a solid base along the entire length of the road. At first they thought of tearing up the whole road, but the cost was too prohibitive. An alternative was to continue filling potholes and washed-away areas. Trueing and leveling would not cure the problem. It cost more in the long run and the road would still be in bad shape. A cost of $30,000 was estimated to pave the road. This compared with $18,000 to recycle the road with liquid calcium chloride, including a sealing of oil and stone. The decision was made to try FDR. This included pulverizing with a Barber Greene RX 40, grading, shaping and rolling, plus two applications of 0.40 gallon per square yard of 35 percent liquid calcium chloride. If it had to be done the old way, it would have required over two weeks of labor. The Barber Greene RX 40 made those passes starting at the edge of an 18-ft-wide road and traveled about 40 ft per minute. On each pass the machine pulverized the road to a width of about 6 ft and to a depth of 6 inches. The machine ground up the 1 inch of oil and stone surface and 5 inches of gravel base. The homogeneous mass of oil and stone, dirt and gravel was then distributed evenly along the road. A grader reshaped the road, forming a crown with an elevation of 1/4 inch per foot. At this point, an asphalt distributor truck applied liquid calcium chloride at a rate of 0.40 gallon per square meter. After the second application of the same amount, the road was then rolled. It was the town's intention to seal the road with oil and stone; however, the road was so hard and was standing up to traffic so well, they decided not to put on a wearing course of oil and stone. After a summer of heavy traffic by 10-wheel tractor trailers, commuters and service vehicles, plus heavy rains, the road remained hard and dust free. The motorist thought the road was paved.

This stabilization and recycling of existing roadways is not new. There are different procedures, using different types of equipment, such as scarifiers, hammermills, mix pavers and pulvi-mixers. The newer reclaimers, such as the Bomag MPH 100, Caterpillar 225 and Barber Greene's RX 40, make it economically attractive for in situ FDR roads.

Several factors need to be considered when the road is set up for FDR. Mix design, which includes chunk sizes and material gradation, as well as binder type and amount, must be determined. Lay-down requirements must be decided. An economic analysis should be carried out to compare the cost and savings of this method of pavement rehabilitation, with alternative pavement maintenance strategies. In every instance, when all the factors were considered, FDR of an asphalt road with calcium chloride was tried. The results were successful.

Most in-place asphalt stabilization and recycling projects consist of a series of operations:

1. Ripping or scarification of the existing pavement layers and gravel base.

2. The reduction in size of the asphalt treated aggregate particles and gravel base.

3. The mixing in of the new asphalt binder with treated aggregate particles and base material.

4. Spreading the recycled material, and

5. The compaction of the recycled material.

Let us consider the factors in the recycling series of operations which include the depth of the road to be recycled and the depth of the asphaltic material in the road.

1. A scarifier, a hammermill and grader are needed for these operations. A Bomag or a Caterpillar reclaiming machine can do this series of operations in one pass.

2. Mixing in of the new asphalt binder. What binder? How much binder? How much asphaltic surface is going to blend with the new binder? How will the gravel absorb the binder? Will the gravel absorb too much? Not enough to bind? This operation is the most critical in the recycling process.

> "Any type of asphalt material -- asphalt cement, foamed asphalt, cutback asphalt, asphalt emulsion or recycling agent can be added through the recycler from the tank on an asphalt distributor. In recent years, emulsified asphalt has been the primary binding agent used in most cost in-place recycling projects. The primary decisions to be made during the mixing operation revolve around the type of asphalt binder to be added and the amount to be used. Again, depending on job conditions, one or more passes of the recycler may be required to properly distribute and mix the asphalt binder with the reclaimed material. Because of this multi-pass operation and because of the variability of this binder addition process, the uniformity of the binder distribution is sometimes poor." (Scherocman, Record 898)

Mixing in liquid calcium chloride into the reclaimed material can be done easily and with less of a margin of error than asphalt emulsion. The addition of liquid calcium chloride can be done through a distributor on the surface of the pulverized road material and then repulverized to the desired depth.

The spreading or grading of recycled material with calcium chloride is done in the conventional way with a grader.

The compaction is done with the conventional compaction equipment, a static steel roller, a vibratory roller, or a rubber-tired roller to provide the desired density to the cold recycled mixture.

The amount of the 35 percent liquid calcium chloride does not vary and with less margin for error than asphalt emulsion. For example: in a desired cut from 6 to 8 inches we recommend 0.75 gallon after the first pass and 0.25 gallon after rolling of 35 percent liquid calcium chloride per square yard. For a depth of 4 to 6 inches we recommend the same amount.

There are engineers who say that because of unknown factors of mix design which can significantly alter the level of performance of the full depth reclaimed material, a wearing surface should always be placed over the recycled mixture. This is not the case at all. We have had wearing courses of a single and double surface treatment, a layer of cold mix asphalt, a layer of asphalt concrete, or a wearing course of just the full depth reclaimed material that has just been treated with the addition of liquid calcium chloride. Twenty percent of the full depth reclaimed asphalt roads with calcium chloride remain without a surface course.

There are certain factors to be considered in the recycling with calcium chloride:

1. The asphaltic surface must always be blended with the gravel base course.

2. The gravel base course must be free of 4-inch bones and cobbles, large boulders, rocks, tree stumps, etc.

3. Limitations as far as gradation is concerned with the reclaiming machine. There are chunks of asphaltic material over 2 inches. The percentage of -200 mesh is sometimes less than 3 percent. If the depth of cut is 4 inches, sometimes fines must be added. It is extremely difficult to achieve 100 percent passing through a 1-inch screen. This is a typical specification of materials for a calcium chloride recycling project.

Materials

The materials shall be a mixture of bituminous concrete and existing gravel base course material pulverized to conform to the following gradation:

Sieve Designation	% by Weight Passing
2 inches	100
1 inch	30-65
Number 200	3-12

Allowances must be made in the specifications for the inherent variability of the full depth reclaimed material. In most projects the above specification is met after the second recycling run, mixing the calcium chloride and the recycled material to the desired depth.

CONCLUSIONS

FDR of asphalt roads with calcium chloride when used in lieu of traditional construction method saves:

1. Money
2. Natural resources
3. Energy
4. Time

Money - With a reclaiming machine the cost of pulverization of the road runs from $1.00 to $2.00 per square yard, depending on the depth of the cut. The cost of the 35 percent liquid calcium chloride averages $0.75 per gallon.

Natural resources - Utilizes in situ materials such as asphalt surface courses, pulverizing and mixing it with the gravel base courses.

Energy - No oil used.

Time - Average time to do 1 mile is 2 days.

The concept of using calcium chloride for an additive in reclamation has been in use since 1978.

BENEFITS OF CALCIUM CHLORIDE AS AN ADDITIVE

- **Uniform Moisture Control** - The most important factor in obtaining maximum density in a well graded mixture is the maintenance of the optimum moisture content. Because of its low vapor pressure, calcium chloride in solution resists evaporation, even in periods of low humidity and high temperature.

- **Increased Density** - Increases the surface tension. Moisture film of calcium chloride solutions are stronger than plain water. The treated aggregate attains a greater density than the untreated similar materials.

- **Less Compactive Effort Required** - Less rolling is required. The accelerated compaction permits earlier completion of work.

- **Less Binder Material Required** - Because calcium chloride aids soil fines in maintaining moisture film, it proves an adequate bond for the aggregate.

- **Surface Uniformity** - The ultimate aim is a smooth riding surface free from long transverse and longitudinal variations, so detrimental to smooth riding and easy driving. Moisture retained in the road permits the base course to be carried as an open surface for an indefinite period before priming with bituminous materials without excess wear and deterioration due to traffic.

- **Controlled Curing for Increased Stability** - The results show that calcium chloride used in the mix ensures a high structural stability, for it controls the rate of drying in both the compaction and curing period.

- **Dust-Free Surface.**

- **Improved Bond** - It is an aid to the absorption of bituminous materials. Priming materials are readily absorbed and there is no block of bituminous materials due to dust film.

- **Adaptable to Stage Construction** - Because the calcium chloride aids in keeping the aggregate in place. Due to budgets, engineers have been inclined, especially in rural areas, to recommend building roads in stages to check the grade, drainage and the selection of the surfaces.

- **Extends the Road Recycling Season** - Due to the low freezing point of calcium chloride, recycling work can begin just after the frost is out of the soil and extend into late November.

- **Frost Protection** - Small percentages of calcium chloride are effective in reducing detrimental frost action. Work done by Dr. Floyd Slate of Purdue University concluded that calcium chloride, in a stabilized mixture, prevented detrimental frost heaving (Slate, Record 422)).

- **Calcium Chloride Does Not Impose Any Environmental Threats** - George Momberger, a Senior Engineer Technician with the New York State Department of Environmental Conservation, said if calcium chloride does leak into a stream, it will carbonate out and leave the water.

- **It Is Economical** - The average price of 35 percent liquid calcium chloride is $0.75/gallon furnished and applied. A total of one gallon per square yard is recommended.

REFERENCES

Scherocman, J. A., "Cold In-Place Recycling of Low Volume Roads," *Transportation Research* (Record 898).

Slate, F. O., "Use of Calcium Chloride in Sub-grade Soils for Frost Protection," *Transportation Research*. (Record 422).

LIME-FLY ASH-AGGREGATE BASE AND SUBBASE COURSES

By Alfred B. Crawley[1]

Abstract: This paper describes the use of lime-fly ash-aggregate (LFA) base and subbase courses for highway pavements. This type pavement course is considered as an alternate to cement-treated bases (CTB) to provide strength characteristics similar to CTB without the cracking and resulting poor durability of flexible pavements constructed over CTB.

INTRODUCTION

With the advent of the Interstate era the Mississippi State Highway Department (MSHD) began extensive use of cement-treated bases (CTB) and subbases for highway pavements. Both mix-in-place and central plant mixing were used to construct base and subbase courses for both flexible and rigid pavements. The aggregates used were select, bank-run granular materials falling generally into the categories of sand clay topping and sand clay gravel which can be used as granular courses when untreated but were cement-treated to provide a high strength, generally non-erodible base course. The drawback to the use of CTB was the extensive crack pattern which developed shortly after construction and reflected through flexible pavement. This reflection cracking severely impacted the durability of the pavement. The cracks allowed for water infiltration, infiltration of incompressibles, pumping of subgrade soils, spalling of the crack faces and other problems which contributed to the creation of unacceptably rough riding surfaces. Expensive rehabilitation was required to restore an acceptable riding surface. A more durable base course material was sought that combined the desirable strength properties of CTB without the accompanying cracking.

The MSHD was introduced to the concept of using fly ash and lime to chemically treat granular materials for use as base and subbase courses in 1981 when the Federal Highway Administration began promoting the use of fly ash in Demonstration Project No. 59, "The Use of Fly Ash in Highway Construction." One of the features of this material that caught the attention of the MSHD was the property of autogenous healing, whereby cracks forming in the material would re-cement over time due to continued chemical reaction. At that time the MSHD was designing the pavement structure for a highway project located adjacent to a coal-fired electric generating station from which fly ash was available. The decision was made to initiate an evaluation of lime-fly ash-aggregate (LFA) mixtures on this project in hopes of finding an alternate to CTB that would be more compatible with flexible pavement. Another desirable attribute was providing an environmentally acceptable use for this by-product of coal combustion.

One of the basic goals of highway agencies is to construct quality pavements for the lowest life cycle cost. If LFA courses would provide acceptable strength properties without the cracking problems of CTB, expanded use could be made of locally available aggregates which would result in lower costs.

APPLICATIONS

The MSHD adopted a three phase program in the evaluation of LFA mixtures for base and subbase construction. All the work involved the use of fly ash and lime to chemically treat soils and granular materials. The first phase was the least cost, least risk usage: in-place subgrade stabilization. The second step involved central plant mixing with a high quality bank-run sand gravel aggregate. The third phase

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utilized road-mixing a bank-run, low plasticity sand topping. All the LFA courses were used for base or subbase under flexible pavement and were 150 mm thick. Through this program we hoped to determine which of these operations, if any, was best able to meet our needs.

The phase one project was built in 1982-83 on SR 63 in Jackson County. Mississippi Power Company's Plant Daniel is located at the south end of this project. The main purpose of the soil stabilization was to create a good working platform on which to place the pavement. The stabilized subgrade would also act as a subbase if sufficient strength was obtained. Fly ash and powdered quick lime were spread on the subgrade, mixed in place with the addition of sufficient water and compacted without difficulty. Percentages of lime and fly ash were 4 and 12, respectively. The flexible pavement thickness was 180 mm. Performance has been excellent. Deflection testing of the finished roadway and compression testing of cores indicate strength characteristics similar to CTB. Reflection cracking is essentially nonexistent. Cracking is occurring in the flexible pavement at this time (1992) but it is due to aging of the asphalt. Economic analysis showed significant savings, approximately $12,430 per two-lane km, over the conventional treatment. It became apparent that this product offered considerable promise as a high quality base course material.

The phase two project was built near Natchez in southwest Mississippi. This project involved both construction of a four lane divided facility and adding two new lanes parallel to the existing two lanes for a portion of US 84/98. The aim here was to see if it was cost effective to manufacture the LFA base in a central mixing plant. Central plant mixing generally produces a more uniform, higher quality material than does road-mixing.

This project was constructed in 1985-86. A bank-run sand gravel aggregate was used. Lime and fly ash percentages were 4 and 12, respectively. Flexible pavement thickness was 100 mm. The LFA base course construction quality was poor due to several factors, primarily mechanical malfunctions of the central mixing plant and the contractor's inability to maintain the proper moisture content in the LFA base. Shrinkage cracks in the LFA base were common in some areas of the project and absent in other areas. The most likely cause of the variability was widely varying proportions of hydrated lime and fly ash in the mixture. The use of an LFA base course resulted in a savings in initial cost of about $21,750 per two-lane km as compared to a hot mix bituminous base.

This project was rehabilitated in 1991 with a hot mix asphalt overlay. An investigation revealed only isolated areas of poor quality LFA base with most of the pavement distress resulting from saturated subgrade conditions and prematurely oxidized hot mix asphalt pavement.

The next phase of the evaluation involved road-mixing lime and fly ash with a low plasticity sand topping. Once again 4 percent lime and 12 percent fly ash were used. The thickness of the flexible pavement was 150 mm. Performance of this project so far is excellent. A savings of about $24,860 per two-lane km was achieved.

A review of these three projects indicated that road-mixing produced excellent results and was very cost effective. It was decided that the higher quality (generally) available with central plant mixing was not cost effective for this usage. Strength characteristics of the LFA courses were as good as CTB. Cracking of the road-mixed LFA courses was essentially nonexistent. Bank-run sand gravel and sand topping both were deemed very suitable for use in LFA courses. Best results were achieved with low plasticity aggregates.

These three projects all used Class F fly ash. A decision was made to use only Class F fly ash until more experience was gained with LFA bases. In 1989 the use of Class C fly ash was permitted on a project in north Mississippi. No problems were encountered. At least three subsequent projects have, at this writing, utilized Class C fly ash and performance is excellent. Dry fly ash has been used on all but one project, where the contractor proposed using conditioned fly ash from a landfill. Conditioned fly ash refers to ash where sufficient water is added to permit easy handling in a landfill operation. Since Class F fly ash does not undergo any chemical reaction without the presence of lime, conditioning with water does not preclude future use in applications like those described in this paper. Problems were experienced due to contamination of the conditioned fly ash. The power plant used powdered limestone to remove sulfur dioxide from the stack gases. The resulting high sulfate-content scrubber sludge was disposed of in the same general area as the fly ash landfill and it is believed the conditioned fly ash was contaminated with the scrubber sludge. The contaminated ash caused unexpected expansion in the LFA base course and resulted in numerous blowups. The LFA base course in the area of each blowup was removed and replaced with hot mix asphalt. About 4 years have elapsed since this LFA base was constructed and it appears that the base has stabilized. No significant distresses have developed since the completion and opening to traffic. An investigation revealed that excess water drives the reaction resulting in expansion and the material should remain relatively stable if the moisture content does not fluctuate very much. The project may well develop additional problems in later years when aging cracks appear in the flexible pavement if water is allowed to penetrate to the base course. The use of landfill ash in LFA has been suspended until it can be assured that such problems will not reoccur.

Hydrated lime was used on all but 2 of 12 projects built thus far. Powdered quick lime was used on the first project and granulated quick lime was used to make a lime slurry for another project. Quick lime is no longer allowed for dry application due to its highly caustic nature.

SPECIFICATIONS

Fly Ash

MSHD specifications require that all fly ash must be obtained from an approved source. Different classes of fly ash or different sources of the same class can not be mixed without special approval. The temperature of the bulk fly ash can not exceed 74° C at the time it is incorporated. The fly ash must meet the requirements of AASHTO Designation: M 295 except for loss on ignition, with a maximum of 6 percent, and the deletion of the requirement for pozzolanic index. This AASHTO specification parallels ASTM C-618.

Lime

Hydrated lime and granular or pelletized quick lime must have a minimum of 90 percent calcium and magnesium oxides with a maximum of 7 percent carbon dioxide. Quick lime must have 100 percent passing a 20 mm sieve and 0-30 percent passing the No. 4 sieve.

Spreading and Mixing

It is required that fly ash be spread and mixed into the granular material prior to spreading the lime. Lime can be applied either dry or in slurry form. Continuous mixing is required until a homogeneous mixture is attained with a minimum of 60 percent of the dry weight, exclusive of gravel and stone, passing a No. 4 sieve.

Compaction

Compaction must begin immediately after mixing is complete and be completed in the same work day. The specified density is 96 percent of Standard Pr · or (AASHTO T-99). The only moisture requirement is that the mixture have the proper moisture co ent needed to attain the specified density.

Finishing and Curing

The surface of the LFA course must be smooth and conform to the lines, grades and cross sections shown on the plans. Vertical tolerances are plus or minus 13 mm. The surface must be kept continuously moist until the placement of the subsequent pavement course. The contractor can defer placement of the subsequent course for up to 21 days by placing a curing seal over the LFA course within 2 days after finishing the course. In practice, the contractor almost always applies a curing seal. The LFA course must be kept continuous moist until the curing seal is applied.

DESIGN

Pavement Design

All pavement structures are designed using the AASHTO Guide. Standard practice for equivalencies between LFA courses and hot mix bituminous base course is 150 mm of LFA equals 100 mm of hot mix bituminous base. A layer coefficient of 0.34 is used for bituminous base which yields a layer coefficient for LFA of 0.23. Where an LFA base course is deemed appropriate, the general practice is to offer alternate designs. The minimum thickness of hot mix asphalt pavement placed above an LFA base is 100 mm. In each of the six projects where alternate designs have been offered the LFA alternate has always been selected. The economics are so clear cut that the LFA alternate is usually the only one that contractors bid.

Mix Design

The MSHD mix design procedure involves preparing trial batches of a limited factorial of lime and fly ash contents with the granular material to be treated. Weight percentages of these two ingredients usually range from 2 to 4 percent for lime and from 6 to 15 percent for fly ash. Specimens at the various lime and ash contents are prepared. Laboratory procedures generally follow AASHTO T-99. Replicate specimens are cured both in a fog room and in a 38 °C hot room. The specimens are tested in unconfined compression at 7, 14, and 28 days. After reaching their maximum load, each specimen is unloaded and placed in the fog room for an additional 28 days and then retested. Most retests result in higher strengths due to the autogenous healing property of the LFA material. Table 1 gives typical laboratory values for these retests. Specimens represented by these retests were initially cured for 28 days at 38 °C before the first break and subsequently placed in a fog room at 21 °C for 28-day intervals before retesting.

The selection of the job mix formula is based largely on the 28-day breaks of the cylinders cured in the hot room. Strengths between 2750 and 4150 kPa are desired. The other strength data (7 and 14 days) is used to evaluate strength gain characteristics, autogenous healing capability and other items peculiar to a given area. It is important to note that this strength data is not considered to be an estimate of the in-place strength of the LFA material at 28 days. The in-place strength gain is largely a function of temperature, with very little chemical reaction taking place below 10 °C. These criteria help ensure that the material has sufficient strength to perform its function. Based on limited coring of LFA materials, an ultimate core strength, generally reached by 1 year of age, between 2750 and 6200 kPa will perform adequately without excessive cracking.

Table 1. - Typical laboratory results for autogenous healing*

Lime/Fly Ash Content %	Initial strength at 28 days kPa	Rebreak #1 at 56 days kPa	Rebreak #2 at 84 days kPa
2/6	765	903	-
2/6	1014	1089	-
2/9	2489	2289	1917
2/9	2310	1993	-
2/12	3130	3179	2889
2/12	2703	2737	2393
3/9	2344	2200	2055
3/9	2255	2406	2055
3/12	4365	4316	3813
3/12	3075	3158	2620
3/15	4089	4454	3889
3/15	3909	3834	2917

* Unconfined compression testing on 101.6- by 116.6-mm specimens.

One of the perplexing problems encountered in designing LFA courses is the inability to get good correlation between strengths of molded cylinders and cores cut from the in-place material. The problem appears to be due either to the inability to extrude LFA samples from the mold without damaging them or significant differences in the moisture-density properties of the materials. Continuing investigation of this problem is being done.

CONSTRUCTION

The equipment used in the construction of LFA courses is the same as that used for in-place lime or cement stabilization. The lime and ash are generally spread pneumatically directly from the transport tank. Mixing is done with single transverse shaft rotary mixers. Three passes are usually made to complete the mixing. Water is applied through water trucks that are either gravity flow or pressurized. Compaction usually begins with a self-propelled padfoot roller and is finished with a pneumatic tire roller. Shaping and blading are done with a motor grader.

One of the most undesirable parts of the construction is the dust clouds that envelop the immediate area during the spreading operation. Since almost all of the work in Mississippi has been done in rural areas this has not presented any major problems. If this type work is done is urbanized areas or adjacent to heavily traveled roadways, it may be necessary to apply the lime and fly ash in slurry form.

One outstanding advantage of the LFA projects constructed in Mississippi is the very forgiving nature of the material. The reaction time of the LFA mixture is usually measured in days or weeks instead of hours, as is generally the case with CTB. If the proper density is not achieved because of improper moisture content, there is ample opportunity to add additional water or aerate the mixture to get the moisture content corrected. Compaction to the specified density is not overly difficult as long as the moisture content is within 1 or 2 percent of optimum.

The item stressed in construction inspection is that the LFA material must be kept continually moist until the curing seal is applied or the next course of pavement is constructed. When weather conditions are sunny and/or windy, a fog of water needs to be applied to the LFA base several times during the day. If the top of the LFA base is allowed to get too dry, a crust will develop which will delaminate and reduce the effective thickness of the course and may cause shoving problems later. There is no substitute for diligence on the part of the inspector to insure the LFA material is kept moist.

PERFORMANCE

Environmental Cracking

The relatively slow rate of strength gain acts to retard shrinkage (environmental) cracking. Shrinkage cracks begin at the top surface of the stabilized course and their propagation through the depth is a function of the drying rate (George, 1973). The top fiber shrinkage stress is a function of the "modulus" and, in turn, the strength of the material. The low modulus, and, in turn, reduced slab action, retard initiation of shrinkage cracking. Despite the low strength (in a given point in time) resulting from slow cementation processes, it can be shown that these base courses exhibit numerous fine cracks instead of the relatively wide cracks spaced further apart in stiff slabs. Fine, light cracks are least likely to reflect through the pavement surface, promoting better performance and longevity.

Autogenous Healing

The continued chemical reaction of LFA mixes plays a crucial role in healing the shrinkage microcracks mentioned in the previous section. It is hypothesized that some or all of the microcracks are healed due to this autogenous healing trait before they become visible to the naked eye. This hypothesis is substantiated by the condition surveys done on the LFA projects that indicate minimal cracking in these base courses. The combination of slow strength gain and self-healing due to continued chemical reactions are the major reasons for the general absence of cracking.

Field Performance

The best measure of performance is ride quality and the absence of major distresses in the finished pavement. Pavement roughness surveys and visual surveys for cracking and rutting are conducted and cores are taken to evaluate the performance of LFA courses. Considering the road-mix projects constructed with dry fly ash that have been open to traffic at least 2 years, cracking has been insignificant and rutting has been generally less than 6 mm. Pavement ride quality, as measured by a response-type road roughness device, is excellent with Pavement Serviceability Ratings (PSR) above 4.0. Deflection testing has also been conducted on the projects and indicates a gradual decrease in deflection over the first year, indicating increased rigidity of the pavement system as the LFA base gains strength. Of the five projects that are complete and open to traffic, three are rated excellent and two are rated good. One of those rated good is the project constructed with the central mixing plant which was discussed earlier. The other project rated good is the project built with conditioned fly ash and it appears the blowups have stabilized.

ECONOMIC ANALYSIS

Economic analyses have been conducted on all of the LFA projects. Most of the projects have been awarded on the basis of alternate base course designs, which indicates that the LFA design is the most economical. Overall, the MSHD has saved over $3,050,000 through the use of LFA bases on the 12 projects built to date. These projects account for approximately 180 two-lane km. The amounts were calculated by comparing the cost of the other alternate on those projects offering alternates and by comparing the cost of the conventional treatment on the two projects where the LFA course was specified. The amounts saved on the individual projects range from a low of $6315 to a high of $25,106 per two-lane roadway km.

SUMMARY

This paper describes the use of LFA base course mixes as an alternative to CTB. The primary disadvantage of CTB is the environmental (shrinkage) cracking that leads to the infiltration of roof water and incompressibles, pumping of subgrade soils, spalling of the crack faces and other problems that adversely effect pavement performance. It has been shown that the slow strength gain associated with LFA leads to a condition where the microcracks in the material are generally healed before they can develop into macrocracks. Continued chemical reactions, known as autogenous healing, within the mix are largely responsible for this behavior. Case studies have shown that the LFA mixes develop strength properties over time that are similar to CTB. Excellent performance for 10 years on the oldest project indicate good durability in the mild climate of Mississippi.

REFERENCE

George, K. P., Mechanism of Shrinkage Cracking of Soil-Cement Bases, HRR 442, 1973.

TECHNICAL SESSION 2

In Situ Stabilization

IN SITU GROUND MODIFICATION

by Joseph P. Welsh, P.E., Fellow, ASCE[1]

Abstract: This paper will update the State of the Practice for the following six techniques of in situ ground modification: Vibroflotation, Stone Columns, Dynamic Deep Compaction, Compaction Grouting, Chemical Grouting, and Jet Grouting. Included is a description of each of the techniques with their histories, current and new uses, environmental uses if applicable, limitations, relative cost, references, research needs, and significant case histories.

INTRODUCTION

The last 15 years have seen dramatic acceptance of in situ ground modification by the design-construction team. Potential savings of construction cost and time have been the owner's motivation. The better understanding of the principles of modifying the ground for new or remedial construction have been the engineer's motivation, while the entire team is driven by the potential benefit to cost ratio.

Others have been motivated by the environmentally sound philosophy of improving existing ground to meet site-specific requirements, rather than removing the problem and creating another problem by disposing of the inadequate material.

The six principal categories of ground modification can be classified as follows:

Densification
Adhesion
Reinforcement
Selective excavation and replacement
Physical chemical alteration
Biological transformation

The first four categories are mature ground modification systems used on an ever expanding basis to improve the ground for new and remedial construction project. The physical chemical alteration and biological transformation categories consist of emerging technologies being designed to clean up waste from the soil and groundwater, and will not be discussed in this presentation. To address the purpose of this session of the symposium, I have selected the following six ground modification techniques from three of the mature categories to discuss:

Vibrocompaction - densification
Stone columns - excavation replacement
Dynamic deep compaction - densification
Compaction grouting - densification
Chemical grouting - adhesion
Jet grouting - excavation and replacement

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The description and history of each of the above techniques will be discussed along with existing and new uses and, where appropriate, environmental applications. Relative cost and research needs will be included and references will be cited on both case histories and significant recent publications.

VIBROFLOTATION

Description and History

Depth vibrators have been used for over 50 years to improve loose, noncohesive sands. The utilization of a depth vibrator for densification and compaction of loose sands was developed in Germany in the mid-1930's and was introduced to the United States in Cape May, New Jersey in 1948 (Brown, 1977). The vibrations are induced by rotating eccentric weights mounted on a shaft and driven by a motor located in the upper part of the vibrator casing. The soils are subject to accelerations of up to 3 g's, which causes liquefaction in the loose sands and improvement in the relative density as high as 85 percent. Sand can be fed down the vibrator to make up for the densification of the in-place sand, or an entire site can be dropped in elevation. The typical vibrator is between 10 to 15 ft long (3 m to 4.5 m) and weighs approximately 7 tons (1814 kg). Depending upon the nature of the soil, the diameter of influence of the vibrator is up to 14 ft (4.5 m); however, depending upon the degree of relative density improvement required, the normal probes are spaced on a triangular pattern 6 to 12 ft (1.85 to 3.7 m) on center. Utilizing this technique, sands have been improved to depths of 115 ft (35 m). The major limitation of the vibroflotation system is the grain size of the soil; only sands can be densified and the efficiency of the technique decreases as the cohesion increases. The normal limitation is a maximum of 12 percent silt and clay, with the clay being less than 3 percent.

New Developments

One of the problems with seismic research has been creating an earthquake to verify design assumption. Nature will not cooperate and shake tables and centrifuges are expensive research tools with scale effect problems. Research studies ongoing at the University of Southern California show that, by observing a piezo-cone as the vibroflot is installed into the ground, you can determine if the soil is liquefiable and when liquefaction actually occurs.

Patents are pending on the use of the vibroflotation system to assist in the removal of contaminants from soils to speed up the normal pump and treat method. By adapting the vibrator so that a surfactant, or other fluids, are injected into the soil as it is liquefied or disturbed by the vibrator, the fluids with the contaminants will flow to the surface under a protective hood and can receive the necessary decontamination treatment.

Relative Cost

The mobilization for a vibrator for densification of soil with the auxiliary equipment and supervision normally runs from 10 to 20 thousand dollars. Although many factors effect the cost of vibrocompaction, the typical cost per linear meter of vibratory probe insertion would be 25 dollars.

STONE COLUMNS (VIBROREPLACEMENT, VIBRODISPLACEMENT)

Description and History

In order to expand the capabilities of the vibratory probe in improving the finer grained soils, the stone column concept was developed in the late 1950's. Where stratas of cohesive soils were encountered, these soils were removed by the jetting action of the vibratory probe and replaced by stone as the backfill material in lieu of sand. Thus, the columns of stone replaced the cohesive soils and any granular soil in the stratification were also densified (Barksdale and Bachus, 1983). The next major development was the dry vibrodisplacement technique; instead of jetting the vibrator in place and feeding the stone around the perimeter of the vibrator, the stone was installed to the tip of the vibrator through an auxiliary pipe attached to the vibrator. As the soil is displaced rather than replaced, normally smaller diameter elements are formed by the vibrodisplacement system. However, the lack of jetting water has minimized the fines laden water causing an environmental problem. The first use of the vibrodisplacement method in the United States was for the Steel Creek Dam Foundation at the Savannah River Plant in South Carolina (Keller et al., 1987; Dobson, 1987).

Uses

The Michigan DOT is building a facility in Port Huron, Michigan which will be the largest use of stone columns in North America. Three thousand stone columns were installed 65 ft (20 m) deep and form a 3-ft (1-m) diameter stone column on 9.5-ft (2.9-m) centers to allow the construction of a reinforced earth embankment for a new Customs facility adjacent to an existing U.S.-Canada bridge. The expanded use of stone columns has many reasons, one of which is the availability of more technical information to assist the consultants in design, construction and inspection of stone columns. Two of these publications are "Design and Construction of Stone Columns" (Barksdale and Bachus, 1983) for the Federal Highway Administration and ASTM Proceedings "Deep Foundation Improvement; Design, Construction and Testing" (Esrig and Bachus, 1991).

New Developments

The aforementioned research project at the University of Southern California will result in the ability to design stone columns, not only for densification, but also to take advantage of the drainage aspects of the stone columns to dissipate excess pore water pressure in case of an earthquake.

Cost

The mobilization/demobilization cost for stone columns is similar to that of vibroflotation but, as the stone backfill is more expensive than sand backfill, the cost of stone columns starts at 45 dollars per linear meter; being higher for vibrodisplacement projects.

DYNAMIC DEEP COMPACTION

Description and History

Dynamic deep compaction can be defined as a densification of soil deposits by means of repeatedly dropping a heavy weight onto the ground surface. This system began commercially in the United States in the early 1970's and, due to its relatively economical cost, has made rapid progress. "Dynamic Compaction for Highway Construction" (Lukas, 1986) is a 230 page bible on dynamic compaction which discusses the soil types that can be improved, methods of contracting, energy required in different formations, etc.

Uses

Dynamic deep compaction is currently applicable to solve the following problems:

1. Reduction of foundation settlement due to loose or low density soils
2. Protecting soil from liquefaction in seismic zones
3. Permitting construction on fills
4. Densifying sanitary landfills for highway construction
5. Densifying mine spoils
6. Pretreatment of potentially collapsible soils
7. Increasing the capacity of sanitary landfills

Limitations

The major limitations of dynamic deep compaction in improving soils are as follow: a) the energy of the weight on the ground creates shear, compression and Raleigh waves which can cause problems with adjacent structures and a safety problem to personnel from flying debris; b) the depth of improvement is a square root function of the weight times the height of drop. With commercially available cranes, 20 tons dropped from 100 ft (30 m) is the maximum energy available, which limits the densification depth of improvement to approximately 35 ft (10.7 m).

Relative Cost

The cost of mobilization of dynamic deep compaction starts at 35 thousand dollars and can double or even triple that amount for some of the larger cranes. The cost of densification depends upon how much energy is applied, but normally ranges between 7 and 15 dollars per square meter of surface area plus any additional fill and geotechnical testing requirements.

Environmental Impact

More and more applications are becoming available for dynamic deep compaction to increase the capacity of sanitary landfills. The closure cap can now be placed on a densified medium with less chance of differential settlement. The cost of the densification program can be offset by the fees collected (tippage fees) from the additional volume of landfill space generated (8 to 20 percent). Schexnayder and Lukas (1992) reported on the utilization of dynamic deep compaction to densify low level radiation waste and allow these wastes to be properly capped.

COMPACTION GROUTING

Description and History

Compaction grouting was initially developed in the 1950's in the United States as a method of rectification of settlement problems (Graf, 1969). It evolved due to the lack of control of slurry grouting for solving these types of problems. The nuance of compaction grouting was the injection of low slump, cementitious grouts under high pressure to densify problem soil stratas.

This soil improvement technique was next used to counteract the settlement caused by soft ground and mixed faced tunnelling. First used on the Baltimore Subway System, it has subsequently been successful on many other major soft-ground tunnelling operations (Baker et al., 1983). On the Seattle, Washington Light Rail Transit, a major innovation was the use of noncementitious grouts, to allow re-injection through the grout pipes, if any additional movement occurred (Robinson et al., 1991).

A large-scale, fully instrumented test program verified that compaction grouting could densify potentially liquefiable soil beneath an existing, operating dam and developed design criteria, quality assurance, and a quality control program for the actual utilization beneath this and other potentially liquefiable sites (Salley et al., 1987).

Improving the soil for new construction was the next plateau of compaction grouting; it was combined with dynamic compaction to improve the soil beneath two new fossil plants in Jacksonville, Florida to allow construction on shallow footings (Schmertmann et al., 1986). The bottom-liner system of a new landfill was protected against local loss of support due to sinkhole action by creating a compaction grout "mat" (Schmertmann and Henry, 1992). The rational design theory was backed up by laboratory experiments and a field verification testing program. This procedure gave all parties involved confidence that the possibility of local loss of support for the landfill liner and subsequent potential for groundwater contamination was greatly reduced. The resulting design procedure will be one of the standards for future compaction grouting programs.

Uses

The 40 years of compaction grouting has resulted in thousands of successful projects in the United States. Due to space limitation, a listing of only the category of problems solved with significant or recent case history references are given for assistance on future problems.

1. Tunnel Settlement Problems (Babendererde, 1991)

2. Controlling Settlement Due to Soft Ground Tunneling (Robinson et al., 1991)

3. Correction of Liquefaction and Seismic Problems (Salley et al., 1987, Mitchell and Wentz, 1991)

4. Solving Limestone Related Problems (Henry, 1986) (Henry, 1987) (Welsh, 1988) (Partos et al., 1989)

5. Prior to New Construction (Partos et al., 1982) (Hussin and Ali, 1987) (Chastanet and Blakita, 1992)

Relative Cost

Like all grouting programs, the cost of compaction grouting can be broken down into 3 basic elements: mobilization/ demobilization, grout pipe placement, and injection of compaction grout.

Mobilization/Demobilization - The cost can start under 10 thousand dollars and reach 50 thousand dollars when multidrill rigs and large batch plants are required.

Grout Pipe Placement Cost - Three-inch grout pipes are normally installed on a primary/secondary grid pattern at costs between 50 and 60 dollars per linear meter.

Compaction Grout Injection Cost - The volume of grout injected normally ranges between 5 and 20 percent of the total mass volume being treated and the injected cost start at 200 dollars per cubic meter of grout injected.

CHEMICAL GROUTING

Description and History

"Chemical Grout System: any mixture of materials used for grouting purposes in which all elements of the system are pure solutions (no particles in suspension)" (ASCE Grouting Committee, 1980).

Chemical grouts are used for two basic purposes: structural and water control. Structural chemical grouting is designed to give cohesion to sand - in effect, make sand into sandstone. Water control chemical grouting is designed to form a water barrier in granular soil formations or seal off water infiltration in existing structures.

Of the five major grouting systems, slurry, chemical, compaction, jet and fracture grouting, chemical grouting is the most researched. This is mainly due to the Federal Highway Administration recognizing, in the 1970's, the potential value of chemical grouting in their proposed, major subway construction program across the United States. The following research reports are of value to consultants planning to utilize chemical grouting;

1. "Design and Control of Chemical Grouting" Vol. 1, "Construction Control" (Baker et al., 1982); Vol. 2, "Materials Description Concepts" (Krizek and Baker, 1982); Vol. 3, "Engineering Practice" (Baker, 1982); Vol. 4, "Executive Summary" (Baker, 1982)

2. "Grouting in Soils" (Hendron and Lenehan, 1976), Vols. 1 and 2

3. "Chemical Grouts for Soils" (Tallard and Carson, 1977)

Other publications of significant value are:

1. "Planning and Performing Structural Chemical Grouting" (Baker, 1982)

2. "Ground Control for Soft Ground Tunnels Using Chemical Stabilization - A Case History Review" (Clough et al., 1979)

3. "Chemical Grouting" Second Edition (Karol, 1990)

Uses and New Developments

The largest chemical grouting project in the United States also resulted in a unique test of chemical grouting and verified its value on any future, soft ground tunnel through granular soil. The horizontal grouting of the twin 21-ft (6.4-m) diameter tunnels under the 10 lanes of the Hollywood Freeway in Los Angeles, California utilized over 7.5 million liters of Geloc-4 sodium silicate grout (Gularte et al., 1991) (Gularte et al., 1992). These papers describe this project in detail including the design, drilling, injection, field monitoring and the finite element study performed. After the twin tunnels had been successfully bored through the grouted soil with negligible freeway movement, a fire in one of the tunnels destroyed the lagging and the loss of support caused an ungrouted section in an adjacent parking lot to collapse; however, the chemical grouts soil, designed only to assist tunneling, continued to support the Freeway with only minor settlement.

Cost

The cost of a chemical grout program can be roughly calculated by assuming that the grout take will be 30 percent of the volume of soil to be injected and the cost per gallon injected will range from 50 cents to 1 dollar per liter. Sleeve port grout pipes are normally placed on a 1 to 1.5 meter grid pattern and their installation cost is about 75 dollars per linear meter. Mobilization/ demobilization cost starts at 10 thousand dollars and will depend upon the amount and type of drilling and grouting equipment required by the specific project.

JET GROUTING

Description and History

Jet grouting is the most versatile of all grouting techniques because of its adaptability to almost any type of soil. Developed in Japan in the 1970's, it was further refined in Europe in the early 1980's and has been gaining favor in the United States since the mid-1980's. Three systems of jet grouting have evolved: the single rod, double rod, and triple rod. The single rod horizontally injects cement grout under high pressure as the rod is rotated and withdrawn to form a cylinder of soil cement. The major limitation of this system is that variations in soil stratification will result in variations in the jet grout element. The double rod system is similar, except a sheath of air protects the grout being injected which produces larger diameter, but still has the soil variation limitation. The triple rod system is the most sophisticated of the jet grouting techniques; using high pressure water protected by air to remove the soil as the void created is filled with a predesigned grout called soilcrete. The problem solving uses in the United States of the triple rod jet grouted system have been mainly underpinning, excavation support, groundwater control, and bottom sealing of excavation. The use of jet grouting has resulted in the greatest economy when jet grouting simultaneously solves more than one problem on a project. Typical combinations have been excavation support, underpinning, and groundwater control. "Jet Grouting - Uses for Soil Improvement" (Burk and Welsh, 1991) describes 17 case histories of singular and multiused jet grouting by the triple rod system.

Environmental Applications

The jet grouting system has been used to form a horizontal barrier to seal off polluted groundwater. This, coupled with a cement bentonite cutoff wall reinforced with sheet piles and tied back with anchors, created a bathtub to allow construction of a coal unloading facility without continuous dewatering. The first utilization of the jet grouting for a soil washing was a test program described by Grisham and Sondermann (1990). This project in Hamburg, Germany is currently ongoing, with the contaminant being removed from beneath the structures without damaging the existing buildings.

Cost

Due to the sophisticated drilling and computerized grouting equipment, the mobilization cost of Jet grouting begins at 35 thousand dollars. The cost of an element one meter in diameter is approximately 250 dollars per linear meter. This excludes the handling of the waste created by the soil removal in the jet grouting operation.

SUMMARY

The past 15 years have seen rapid growth in the field of in situ ground modification and each project performed and reported to the industry enhances the continuous growth of these environmentally sound construction techniques.

After the Loma Prieta earthquake in October 1989 in California, Professor Mitchell examined 12 structures that had the ground improved prior to this magnitude 7 earthquake and reported no distress in these structures, while adjacent structures sited on unimproved ground had problems (Mitchell, 1991). The methods utilized for ground improvement and the frequency of use are as follows:

Stone columns	3
Dynamic deep compaction	3
Vibrocompaction	2
Compaction grouting	1

Chemical grouting	1
Sand compaction piles	1
Nonstructural timber piles	1

The report cautions that in no case was the Loma Prieta earthquake of maximum design magnitude, peak ground acceleration or duration.

CONCLUSIONS

The predicted future of ground modification will be in three major areas:

1. Improving the soil for new construction

2. Improving the soil for infrastructures and remedial purposes

3. In situ waste cleanup

The former two areas will continue to be benefit/cost ratio driven, while the latter will be regulatory driven.

Upgraded codes and more recognition of seismic hazard will result in soil improvement being employed more for new and remedial seismic protection programs.

New and remedial construction will see a continuing upswing in the volume of in situ ground modification using the four mature categories - Densification, Adhesion, Reinforcement and Selective Excavation and Replacement, while the waste cleanup will cause many emerging technologies to be researched, developed, and refined, in the emerging categories of physical chemical alterations, and biological transformations.

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LIQUEFACTION AT NAVAL STATION TREASURE ISLAND AND DESIGN OF MITIGATING MEASURES

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Abstract: During the 1989 Loma Prieta Earthquake, liquefaction occurred in hydraulically placed sandfill comprising Treasure Island, a manmade island in San Francisco Bay. The island settled and spread laterally several inches during the earthquake. Locally, settlements as great as 2 ft (0.6 m) were documented. Damage to Naval Station buildings on the island was generally minor but some buildings experienced greater damage. The recorded level of ground shaking on the island, occurrence of liquefaction, and magnitude of liquefaction-induced compaction settlements were reasonably consistent with postearthquake analyses using state-of-the-practice methods. Residual strengths for the liquefied sand were inferred from back analyses of observed lateral spreading displacements. There is potential for ground failure effects on Treasure Island to be much more severe during future earthquakes than experienced during the relatively distant Loma Prieta earthquake. Vibroreplacement was determined to be a cost-effective method for mitigating liquefaction-induced lateral spreading at Treasure Island. However, before implementing this method, consideration should be given to the potential for lateral movements in soft clays underlying the liquefiable sands. Deep soil mixing was examined as a method to improve the performance of soft clays beneath the causeway to Treasure Island.

INTRODUCTION

U.S. Naval Station Treasure Island is situated on a manmade island created in the mid 1930's by hydraulically placing sandfill over soft sediments in San Francisco Bay. A location map of the island is shown in Figure 1. During the Loma Prieta earthquake of October 17, 1989, the sandfill liquefied. Liquefaction is a soil behavior in which relatively loose sandy soils below the ground water table lose a substantial amount of strength due to high pore water pressures generated by strong earthquake ground shaking. As a result of liquefaction, relatively small amounts of lateral spreading (generally less than 1 ft [0.3 m] of horizontal movement) occurred around the perimeter of Treasure Island. The interior of the island experienced a general subsidence of several inches. The U.S. Navy conducted a study (Geomatrix Consultants, 1990a, 1990b) to investigate the island's behavior during the Loma Prieta earthquake as well as evaluate its expected behavior during potential future earthquakes that could occur closer to Treasure Island and/or be of larger magnitude than the Loma Prieta earthquake. This paper focuses on aspects of that study that describe the observed behavior of the island during the Loma Prieta earthquake and compare the observations with predictions using current analytical methods and correlations. The behavior of the island during future earthquakes and ground improvement measures to mitigate potential lateral spreading hazards are also addressed. Further details regarding the behavior of Treasure Island during the Loma Prieta earthquake and assessments of behavior during future earthquakes are presented in Power and others (1992).

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SUBSURFACE CONDITIONS AT TREASURE ISLAND

A typical cross section through the perimeter of Treasure Island is shown in Figure 2. A rockfill section on the perimeter slope face provides retainment and wave protection for the sandfill comprising the island. The sandfill varies from clean sand to silty sand with occasional clayey sand zones. Because the sand was placed by hydraulic-filling methods without compaction, it is generally in a loose condition. Beneath the fill over parts of the island is a layer of natural shoals sands, which vary from clean to clayey sands with clay layers. The shoals sands are loose to medium dense. The combined thickness of the sandfill and shoals sands varies from approximately 30 to 50 ft (9 to 15 m), except for a 400-ft (122-m) long area along the north side of the island where a key was dredged below the bay bottom. In this localized area, the sandfill is approximately 70 ft (21 m) thick.

Underlying the sandfill and shoals sands is a layer of young, normally consolidated, soft to stiff clay, known locally as Bay Mud, which varies in thickness from 10 to 120 ft (3 to 36 m) across the island. Beneath the Bay Mud, older bay sediments (predominantly stiff to very stiff clays) extend to bedrock. The estimated depth to bedrock below the island ground surface varies from approximately 100 to 400 ft (30 to 122 m).

BEHAVIOR OF TREASURE ISLAND DURING LOMA PRIETA EARTHQUAKE

Ground Shaking

The Loma Prieta earthquake, of moment magnitude (Mw) 7, occurred on or adjacent to a segment of the San Andreas fault in the Southern Santa Cruz Mountains approximately 50 miles (80 km) south of Treasure Island. Ground shaking was recorded on the island, as well as on Yerba Buena, a rock island located adjacent to Treasure Island (Figure 1). The rock motion had a peak acceleration of 0.06 g, whereas the ground motion on Treasure Island had a peak acceleration of 0.16 g. This amplification of ground motion on the island relative to rock was typical of amplifications observed at several locations situated around the San Francisco Bay margin that are underlain by Bay Mud.

To examine site amplification effects observed at Treasure Island for the Loma Prieta earthquake, site response analyses were conducted using the SHAKE computer program (Schnabel et al., 1972) in which nonlinear soil behavior is modeled using the equivalent linear method. The accelerograms recorded on Yerba Buena Island during the earthquake were used as rock input motions for the SHAKE analyses. At the time analyses were conducted for this study, site-specific measurements of dynamic soil properties were not available; therefore, properties for the analyses were estimated on the basis of published correlations of shear wave velocity or shear modulus with penetration resistance data for sands (Sykora, 1987) and with undrained shear strength for clayey soils, (Egan and Ebeling, 1985) as well as values measured for similar soils at other bay margin sites (Fumal, 1978). The estimated shear wave velocity profile used in the analyses is illustrated in Figure 3. Measurements of shear wave velocity made at Treasure Island subsequent to the analyses conducted for this study are also shown in Figure 3 for comparison and illustrate that the estimated velocity profile was reasonable.

Ground surface motions computed by a typical analysis is illustrated in Figure 4 for the east-west horizontal component, and is compared with the recorded east-west motion recorded on Treasure Island during the Loma Prieta earthquake. The input rock motion is also shown. The computed and recorded ground surface motions show generally similar amplitudes and strong-shaking duration. After the strong-shaking portion of the recorded motion, the amplitude of shaking for the motion is greatly reduced; as may be seen in Figure 4, the computed amplitudes exceed the recorded amplitudes for this later part of the motion. The change of shaking intensity in the recorded motion is believed to correspond to the onset of liquefaction in the loose sandfill, because liquefied sands cannot transmit as much energy from bedrock to the ground surface, lower accelerations result. The time at which liquefaction is thought to have occurred is indicated on the recorded motion in Figure 4. Such softening behavior of the sands due to liquefaction cannot be modeled well with SHAKE analysis, thus postliquefaction motions are overestimated by the analysis. In analyses conducted subsequent to this study using a nonlinear, effective stress computer program DESRA (Lee and Finn, 1985), the postliquefaction reduction in motion was simulated, and the entire time history was quite similar to the recorded motion. Peak ground-surface accelerations obtained from several SHAKE analyses modeling the various subsurface conditions across the island were in the range of about 0.14 g to 0.18 g, indicating that the level of ground shaking probably did not vary greatly at different locations on Treasure Island.

Liquefaction

Sand boils were observed at many locations across the island, substantiating that liquefaction of the subsurface sands had occurred during the Loma Prieta earthquake. To assess the occurrence of liquefaction in these materials, standard penetration test (SPT) and cone penetration test (CPT) data obtained during the Geomatrix (1990a, 1990b) study, as well as during many previous geotechnical investigations on Treasure Island, were used in conjunction with the Seed-Idriss empirically-based method for assessing liquefaction potential (Seed and Idriss, 1982; Seed et al., 1985). Peak ground acceleration, total and effective overburden pressures at the point of interest, and SPT blowcounts are needed for the assessment. CPT data were converted to equivalent SPT blowcounts using a site-specific correlation. The standardized blowcount used in the method is $(N_1)_{60}$, which represents the SPT blowcount to advance a 2-inch (51-mm) O.D. split spoon sampler 1 ft (0.3 m) at a 60-percent hammer energy efficiency, with correction to a 1 ton/square ft (98 kPa) effective overburden pressure. The Seed-Idriss method is based on the empirical correlation between cyclic stress ratio (computed from the peak ground surface acceleration or from site response analyses, e.g., SHAKE analyses) and $(N_1)_{60}$ that differentiate the observed occurrence or nonoccurrence of liquefaction in sand deposits during historical earthquakes. The basic correlation presented by Seed et al. (1985) is illustrated in Figure 5, which was developed for magnitude 7.5 earthquakes for sandy materials with different fines contents, but may be adjusted to other magnitude earthquakes using correction factors given in Seed et al. (1985). The set of curves appropriate to a magnitude 7 (i.e., Loma Prieta) earthquake would be obtained by multiplying the ordinates of the curves illustrated in Figure 5 by a 1.06 correction factor. Blow counts for silty sands were converted to equivalent clean sand blow counts using Figure 5.

For a given value of the peak ground surface acceleration (a_{max}), in units of g, and the total and effective overburden pressures at the depth of interest (σ_o and $\bar{\sigma}_o$, respectively), a value of the average induced cyclic stress ratio ($\tau_c/\bar{\sigma}_o$) can be computed using the expression (Seed and Idriss, 1971):

$$\frac{\tau_c}{\bar{\sigma}_o} = 0.65\,\frac{a_{max}}{g}\,\frac{\sigma_o}{\bar{\sigma}_o}\,r_d$$

in which r_d is a stress reduction factor that decreases from a value of 1 at the ground surface to a value of 0.9 at a depth of about 35 ft (10.7 m). Thus, given a value of cyclic stress ratio and curves such as shown in Figure 5, a critical level of $(N_1)_{60}$ can be determined, such that those materials having an $(N_1)_{60}$ value greater than the critical level would be likely to not liquefy and those having a value less than the critical level would be likely to liquefy. By comparing the critical $(N_1)_{60}$ level with the measured $(N_1)_{60}$ of the sand, an expectation of liquefaction occurrence or nonoccurrence can be assessed.

Figure 6 illustrates the comparison of the critical $(N_1)_{60}$ level for clean sands with the equivalent clean sand $(N_1)_{60}$ data compiled from SPT and CPT measurements during this and previous studies on Treasure Island. Figure 6 is for a portion of the perimeter of the island, and plots showing similar results were prepared for other portions of the perimeter. A substantial number of the $(N_1)_{60}$ data fall below the critical level, indicating that much of the sand below the ground-water table liquefied during the Loma Prieta earthquake, which is in agreement with the observations of sand boils. However, because some of the data below the ground-water table also lie above the critical level, it is likely that there were zones within the fill and shoals where liquefaction did not occur.

Ground Settlement

Ground surface settlement was quite obvious across Treasure Island, especially differential settlement adjacent to or inside of buildings, localized sinkholes, or vertical slumping associated with lateral spreading. Observations of ground surface settlement adjacent to piled structures and/or old pilings indicated that a magnitude of settlement associated with shaking-induced compaction of the subsurface materials was generally up to approximately 6 inches (150 mm). Limited pre- and post-earthquake ground survey data confirm that the surface of the island generally subsided by approximately 2 to 6 inches (50 to 150 mm), variably across the island. These settlements presumably occurred in the days following the earthquake as excess pore water pressures in the sands dissipated and the sands compacted to some degree. Near the island perimeter, settlements were locally greater due to slumping accompanying bayward lateral spreading. Where thicker fill existed at the north end of the island, up to 2 ft (0.6 m) of settlement occurred based on comparison of pre- and post-earthquake aerial topographic survey photos of the perimeter area.

To develop a more complete picture of the settlement variation across the island than provided by the limited survey data, ground surface settlements associated with shaking-induced compaction were estimated using the compiled $(N_1)_{60}$ data, the observed Loma Prieta earthquake ground motions, and an approach for estimating the magnitude of earthquake-induced compaction settlement of sandy materials modified from Tokimatsu and Seed (1987). The Tokimatsu and Seed (1987) procedures were found to overestimate the shaking-induced compaction settlement for the Loma Prieta earthquake. A modified approach was developed that utilized the simplified framework of Tokimatsu and Seed (1987) as a basis and incorporated the influence of grain-size on postcyclic volumetric strain described by Lee and Albaisa (1974). Examination of the empirical data utilized by Tokimatsu and Seed (1987) to develop their volumetric strain relationships indicates that the soils from the various sites/studies had similar grain-size characteristics (i.e., D_{50}). Laboratory testing results presented by Lee and Albaisa (1974) indicate that postliquefaction volumetric strains of sandy soils is dependent on grain size, such that a soil with a smaller D_{50} would experience smaller strains than coarser-grained soils. Based on the Lee and Albaisa (1974) data, D_{50} -dependent adjustments were made to the volumetric strains computed from the Tokimatsu and Seed (1987) procedures for a given layer/zone of soil. Because the grain sizes of the sands at Treasure Island are typically finer than the soils of Tokimatsu and Seed (1987), the net effect was a general reduction of settlement computed by the Tokimatsu and Seed (1987) procedures.

The results of this analysis are illustrated in Figure 7, showing contours of estimated ground surface compaction settlement associated with the Loma Prieta earthquake. Elevation changes at survey monuments or reference point locations determined from the available ground survey measurements are also shown in Figure 7. There is reasonably good agreement between surveyed elevation changes and the settlement estimated from nearby SPT/CPT data. Figure 7 indicates that there are zones for which compaction settlements may have been as large as 8 to 10 inches (200 to 250 mm). These areas typically correspond to areas where the thickest zones of liquefied fill/soil were expected to have occurred based on examination of cross sections through the island.

Lateral Spreading

The perimeter areas of Treasure Island experienced lateral (bayward) movement, with associated cracking and slumping. A detailed mapping of the cracking is presented in Geomatrix Consultants (1990b). Lateral spreading movements were manifested by ground cracks parallel to the island perimeter. These cracks extended as far inland from the perimeter slopes as 550 ft (168 m), but in most areas cracking did not extend beyond 200 ft (61 m) from the perimeter slope. Summation of horizontal crack widths indicates a maximum amount of bayward lateral spreading of about 1 ft (0.3 m). Pre- and post-earthquake horizontal coordinates data for a benchmark indicated approximately 10 inches (250 mm) of bayward movement. In the local area at the north end of the island where up to 2 ft (0.6 m) of settlement occurred, lateral spreading movements may have exceeded 12 inches (300 mm) but did not manifest themselves in the lawns that cover most of this area.

Postearthquake observations also indicate that the lateral movement occurred during the ground shaking and did not continue after the earthquake. These observations are consistent with the phenomenon of lateral spreading in which earthquake ground-shaking affects the stability of slopes containing potentially liquefiable soils by seismic inertia forces within the slope, and by shaking-induced strength reductions in the liquefiable soil materials. Temporary instability due to seismic inertia forces are manifested by lateral (in this case, bayward) movements of the slope and such movements can potentially involve large land areas. For the duration of ground shaking associated with the Loma Prieta earthquake, there could be several such occurrences of temporary instability, producing an accumulation of down-slope (bayward) movement. The Loma Prieta earthquake was of unusually short duration for a magnitude 7 earthquake. Had the duration been longer, lateral spreading movements would have been larger.

The primary objective of the analysis of lateral spreading movement was to evaluate the residual strength exhibited by the sandfill and shoals material in a state of earthquake-induced liquefaction. To achieve this objective, slope stability and deformation analyses were conducted for several cross-sections through mapped lateral spreading zones, given the Loma Prieta ground shaking conditions and observed deformation patterns.

Based on observations from a number of historical liquefaction failure sites, it has been postulated that liquefied sands have a small residual postliquefaction undrained shear strength (Seed and Harder, 1990). Figure 8, from Seed and Harder (1990), shows a relationship between $(N_1)_{60-cs}$ and a range of values for the residual undrained shear strength based on a number of case histories. $(N_1)_{60-cs}$ is a "clean-sand" corrected equivalent blowcount, which accounts for the residual strength of a sand based on its density and fines content. The range of values of $(N_1)_{60-cs}$ for the liquefied sands at Treasure Island is typically between 6 and 15 (Figure 6). Based on Figure 8, for such blowcount values the undrained residual strengths of liquefied sands are estimated to generally be in the range of about 50 to 700 psf (2 to 34 kPa).

To determine a representative site-specific value of undrained residual shear strength for the liquefied Treasure Island sand materials during the Loma Prieta earthquake, deformation analyses were made. The method of analysis was based on Newmark's (1965) approach which was modified by Makdisi and Seed (1978) to calculate seismically induced ground displacements for a slope. Several cross sections were analyzed, and it was found that values of residual shear strength between about 375 and 450 psf (18 and

22 kPa) were necessary to have the calculated displacement compatible with the observed. However, because liquefaction may not have occurred completely, it is also possible that the strengths mobilized in nonliquefied or partially liquified zones exceeded these values while residual strengths of fully liquefied sand were less than these values.

Damage to Buildings and Utilities

The effects of the Loma Prieta earthquake on U.S. Naval Station Treasure Island facilities varied across the island and appeared to correlate reasonably well to areas of more significant ground distress. Most of the buildings on the Naval Station reportedly suffered no or insignificant damage or distress, many experienced minor cracking or differential settlement; and several experienced greater damage. The buildings that suffered greater damage were generally situated near the perimeter and in areas of mapped significant ground distress, primarily lateral spreading. In addition, some buildings in the greater damage category were located in areas of relatively larger estimated settlements.

Most buildings on Treasure Island are supported on shallow footing foundations; a few are supported on piles. The pile-supported buildings performed well; however, these buildings were not located in areas of apparent significant lateral spreading. Interior non-pile-supported floor slabs and shallow-footing supported buildings attached to pile-supported buildings experienced large differential settlements relative to the pile-supported structures and consequent severe damage.

Underground utilities were also affected by lateral spreading movements and ground settlement associated with liquefaction. Forty-four utility line breaks were reported. Many of the breaks occurred in the area adjacent to the perimeter dike and are probably primarily due to lateral spreading movements. It is not clear from the available data what amount of lateral movement was required to cause breaks in different types of utility lines, although at some locations, spreading cracks on the order of an inch are mapped near the breaks. Some of the breaks also appeared to occur as a result of differential vertical (slumping) movement on lateral spreading cracks. In the interior area of the island, breaks cannot be associated with lateral spreading, but rather appear to be generally associated with ground settlement and/or areas where liquefied fill materials are at shallow depths below the ground surface. Typically, breaks occurred in areas with estimated ground settlements of at least 6 to 8 inches (150 to 200 mm).

Performance of Ground-improved Areas

There are several areas where ground improvement of different types had been implemented at Treasure Island prior to the earthquake. Ground improvement methods included compaction piles, Terraprobe, and vibroflotation/stone columns. Generally, buildings founded on improved ground performed well. No major structural damage was detected in Building 450 (sand compaction pile densification), and 452 and 453 (nonstructural timber pile densification), although there was a concrete spall at the end of one wing of Building 453. The area of Buildings 487, 488, and 489 was densified using vibroflotation; these buildings suffered little to no damage. At Pier 1, the densified areas exhibited no signs of ground movements, while sinkholes and sand boils developed in immediately adjacent areas. A benchmark monument is located on the shore near Pier 1. This monument did not undergo significant horizontal movement due to the earthquake, which is consistent with the lack of ground distress in this area.

The medical/dental facility situated in the south-central part of the island was under construction at the time of the earthquake. At this location, vibroreplacement had been completed using gravel size crushed rock backfill to improve loose soil conditions to a depth of about 20 ft. Below 20 ft, increased fines content limited the success of the densification procedure. The improved ground zone at the facility apparently performed well during the earthquake; there was no evidence of liquefaction or other ground-failure-related distress at the surface, and only minor differential settlement was observed. Sand flows did occur within 22-ft (6.7-m) deep elevator shaft excavations which were open at the time of the earthquake.

ESTIMATED BEHAVIOR OF TREASURE ISLAND DURING FUTURE EARTHQUAKES

As is the case for most of the San Francisco Bay Area, Treasure Island could experience much stronger ground shaking than occurred during the Loma Prieta earthquake. A maximum earthquake occurring on segments of either the Hayward fault (magnitude 7) or the San Andreas fault (magnitude 8) closest to the island would result in estimated values of peak ground acceleration of about 0.4 g as compared to 0.16 g experienced during the Loma Prieta earthquake. For the Hayward event, the duration of strong shaking could be twice as long as for the Loma Prieta earthquake and for the San Andreas event, could be five times as long. The behavior of the island was assessed for these events.

The critical $(N_1)_{60}$ blow counts for liquefaction is compared with the $(N_1)_{60}$ blow count data for both earthquakes in Figure 9, from which it can be seen that complete liquefaction of the sands would be expected. The most severe potential effect of the liquefaction and strong shaking would be lateral spreading. Using the simplified Newmark-type deformation analysis along with values of residual sand strength inferred from the Loma Prieta earthquake analysis discussed earlier, it was estimated that lateral displacements could be on the order of 4 ft (1.2 m) and 10 ft (3 m) for the earthquakes on the Hayward fault and San Andreas fault, respectively. Because of uncertainties in the estimation methods, these displacements are not upper bound values and could well be exceeded. The lateral spreading movements could extend several hundred feet into the island and as a result cause severe damage to facilities in the affected areas.

Beyond the zone of lateral spreading, the main ground failure hazard would be settlement. Using the modified Tokimatsu and Seed (1987) method discussed earlier for the Loma Prieta earthquake analysis, it was estimated that compaction settlements would generally range from 8 to 16 inches (200 to 400 mm) over the island and locally exceed 20 inches (500 mm). The differential settlements accompanying these settlements would be damaging to many buildings and facilities. However, overall, compaction settlements would tend to vary smoothly over the island (similar to the pattern of Figure 7) and have much less severe effects on facilities than those caused by several feet or more of lateral spreading. Locally, ground loss due to sand boils could produce more severe differential settlements.

MITIGATION OF LATERAL SPREADING HAZARD

During the initial post Loma Prieta earthquake studies, several methods were considered for mitigating the damaging effects of future earthquakes on Treasure Island. In consideration of the more significant impacts to facilities resulting from lateral deformations as compared to general area settlement, evaluation of improvement procedures was directed at reducing the risk of liquefaction-induced spreading hazards. In this scenario, unimproved fills on the landward side of a stabilized perimeter zone would be expected to liquefy and settle, but be contained by the improved zone so that damages due to lateral deformations would be minimized. Improvements were evaluated which would contain the liquefied soils under static loadings with a factor of safety of at least 1.3. The degree of lateral deformation that would be induced in the stabilized zone due to cyclic seismic loading was then evaluated.

The improvement methods considered included various forms of fill soil densification, soil improvement by cement or chemical stabilization, and in-water rock fill placement to contain/stabilize the slope. It was found that the most cost-effective means of mitigating the hazard was to create a buttressing, stabilized region of sand adjacent to the perimeter slope by the vibroreplacement method. In this method, the sand would be densified by vibroflotation at approximately 7 ft (2.1 m) on-centers and stone columns (approximately 3 ft (1 m) in diameter) would be constructed at the vibroflotation locations. The width of the stabilized region was designed to be great enough to statically retain fully liquefied sand and control dynamically induced displacements behind the stabilized zone. It was found that if the stabilized zone had a width of approximately 50 ft (15 m) extending back from the top of the perimeter slope, it should effectively retain the liquefied sands farther landward and limit lateral displacements behind the stabilized zone to a maximum of approximately 1 ft (0.3 m) during a maximum earthquake on the San Andreas fault and 1/2 ft (150 mm) for a maximum earthquake on the Hayward fault. The vibroreplacement stabilized zone is illustrated in plan in Figure 10 and in cross section in Figure 11.

Stability and deformation evaluations of these densified sections indicated that it would also be important to assess the potential for movements in the underlying Bay Mud, which is a relatively soft soil. With the sand-stabilization measures in place, the weak zone for lateral movements then shifts from the stabilized sand into the Bay Mud (see Figure 11). Potentially, larger movements might occur on sliding surfaces that pass through the Bay Mud below the stabilized sand buttress than would occur on surfaces that pass through the buttress. Within the scope of studies performed, it was not possible to assess in detail the potential for such movements through the Bay Mud around the perimeter of Treasure Island. Preliminary analyses using Newmark-type methods indicated that movements of several feet might occur and that therefore more detailed consideration of this potential hazard would be desirable. Preliminary evaluations indicated that a deep soilcrete stabilization process extending 30 ft into the Bay Mud layer could be effective in limiting this form of deep seated deformation.

Causeway Stabilization

The causeway is a narrow strip of filled land connecting Treasure Island and Yerba Buena Island (Figure 1). Available data indicate that the causeway was constructed using methods similar to those employed for Treasure Island. The subsurface conditions include liquefiable sandfill, Bay Mud, and deeper bay sediments above bedrock. A cross section through the causeway illustrating these conditions is shown in Figure 12.

During the Loma Prieta earthquake, liquefaction-induced lateral spreading of the causeway occurred in a manner similar to that described previously for the perimeter of Treasure Island. The stability and postearthquake integrity of the causeway was deemed to be particularly important because the causeway provides the only means of land access to and exit from Treasure Island. Thus the Navy conducted a study to assess means for stabilizing the causeway (Geomatrix Consultants, 1991). The objective of stabilization was to provide two 12-ft (3.7-m) wide traffic lanes in the central part of the causeway that would be passable (although with some distress) following postulated future maximum earthquakes.

Initial design effort was directed at a cellular stabilization method using deep soil mixing to improve the future performance of both the sandfill/shoal materials and the underlying soft Bay Mud layer. Evaluations of the preliminary design indicated that liquefaction within the stabilizing cells could result in unacceptable postearthquake deformations and a combination of methods was proposed. It was concluded that, from a technical standpoint, the preferred stabilization method for the causeway consisted of a combination of: vibroreplacement to the depth of the sandfill/shoal materials in areas most susceptible to liquefaction damage; and deep soil mixing within the sands and extending below the fill to just beneath the base of the Bay Mud layer along the full length of the causeway profile susceptible to damage. Figures 13 and 14 show the combined stabilization scheme in plan and cross section respectively.

In the deep soil mixing stabilization method, soil and cement grout are mixed in place to form "soilcrete" columns. The process involves penetration into the ground by large diameter (24- to 36-inch, 0.6- to 0.9 m) augers to the depth of required treatment. Cement grout is pumped down through the auger shafts during penetration and withdrawal of the augers. The augers and mixing paddles mix the soil and cement grout in place to form the soilcrete columns. Typically, a dual- or triple- auger rig is used. The soilcrete columns are typically constructed in a honeycomb configuration as shown in Figure 13 to create cells that would be effective in resisting loads applied along potential failure surfaces. The cellular configuration shown in Figure 13 is similar to that used at the U.S. Bureau of Reclamation project at Jackson Lake Dam in Wyoming (Pujol-Ruis et al., 1989) to stabilize potentially liquefiable sandy soils. In order to better resist lateral earth and pore water pressures that could develop within stabilizing cells when perimeter slopes deform, the outside edges of the cells are oriented to form an arch. A buttress, consisting of three soil-mixed columns, is located at the end of each arch to resist the compression load carried by the arch. During further evaluation of this configuration it was found that high stresses would develop in the soilcrete columns due to liquefied soil contained in the soil-mixed cells. To prevent these high stresses, vibroreplacement was incorporated to prevent liquefaction of those areas where the sandy, saturated soils exist near the surface of the causeway.

The deep soil mixing process has been reported to achieve unconfined compressive strengths of soilcrete columns as high as 1000 lb/in^2 (6900 kPa) in sandy soils. For clay soils, strengths would be lower; it was estimated that unconfined compressive strengths of at least 70 lb/in^2 (480 kPa) could be achieved in the Bay Mud with shear strengths equal to approximately one-third of the compressive strengths.

For the combined vibroreplacement/deep soil mixing stabilization scheme shown in Figures 13 and 14, simplified deformation analyses indicated that roadway deformation should not exceed 2 to 4 ft (0.6 to 1.2 m) for the postulated San Andreas fault earthquake and 1 to 2 ft (0.3 to 0.6 m) for the postulated Hayward fault earthquake. It was also assessed that these estimates were probably conservative.

An alternative mitigation scheme for the causeway was also developed that consisted solely of vibroreplacement within the sandfill. The vibroreplacement locations were 7 ft (2.1 m) on centers and extended over a central 60 ft (18 m) of the causeway. It was assessed that for this alternative, deformations to the roadway could significantly exceed those for the combined soil mixing/vibroreplacement scheme and that the performance was more uncertain. However, this alternative was estimated to be considerably less costly to construct. For stabilizing of a 1000-ft (305 m) length of the causeway, the estimated cost of the technically preferred scheme was $7.1 million; whereas the estimated cost of the alternative scheme was $2.4 million.

The assessments of mitigation measures described herein for Treasure Island have pointed to the need for: (1) methods for estimating lateral displacements with greater confidence; and (2) case histories where ground stabilization methods to mitigate lateral spreading have been tested by strong earthquake shaking. It is considered that finite element analyses might provide additional insight into the displacements that might occur for situations of stabilized sands overlying soft clays such as depicted in Figure 11.

CONCLUSIONS

The behavior of Treasure Island during the Loma Prieta earthquake again illustrated the vulnerability to liquefaction of sandfills placed through water without compaction. Significant ground failure and consequent damage occurred during the Loma Prieta earthquake, but effects could have been much more severe had the earthquake been closer and/or larger, as potentially could affect the island, other parts of the San Francisco Bay Area, and similar near shore waterfront fill in seismic areas throughout the United States in the future.

Most buildings on the island performed satisfactorily during the Loma Prieta earthquake. Thus liquefaction does not necessarily imply severe effects to construction; the effects depend on the amount and types of ground movement resulting from liquefaction and the nature of the design and the construction. Compaction settlements of several inches caused relatively minor damage overall. Lateral spreading movements of several inches were more damaging; potentially much larger lateral spreading movements during larger, closer earthquakes could be very damaging.

The recorded level of ground shaking on the island during the Loma Prieta earthquake was in reasonably good agreement with that predicted using current ground response analysis methods. The occurrence of liquefaction on the island was consistent with behavior that would be predicted using the Seed-Idriss method of liquefaction potential evaluation. Applying a modified version of the Tokimatsu and Seed (1987) method resulted in estimates of compaction settlement in reasonably good agreement with observed settlements. Residual strengths of liquefied sand inferred from analysis of the lateral spreading behavior on the island fall within the range reported by Seed and Harder 1990.

The vibroreplacement ground stabilization method was assessed to be the most cost-effective method for mitigating the liquefaction-induced lateral spreading hazard at Treasure Island. This method could be applied to protect selected areas or critical facilities. However, before it is implemented, a thorough assessment of the potential for movement in the underlying relatively soft clays (Bay Mud) should be completed. A mitigation scheme developed for the causeway to Treasure Island combined the deep soil mixing method to the base of the Bay Mud with the vibroreplacement method in the sands. However, this scheme is relatively expensive in comparison to vibroreplacement alone, and a more detailed evaluation of the improved soil strengths and the expected performance of the combined materials would be warranted before proceeding.

Current uncertainties with regard to future Navy and Department of Defense facility requirements in the Bay Area make the near term implementation of these improvements at Treasure Island unlikely. However, localized improvements for selected facilities may be possible in concert with ongoing seismic measurement and geotechnical evaluation studies. In addition, information with regard to observed soil performance at Treasure Island has broad implications relative to expected performance of other waterfront fill sites in seismic regions. The Navy, and other government agencies, should draw upon this experience to aid in further developing methods for predicting the expected seismic performance of such sites and applying soil improvement processes to mitigate the effects of liquefaction-induced lateral spreading.

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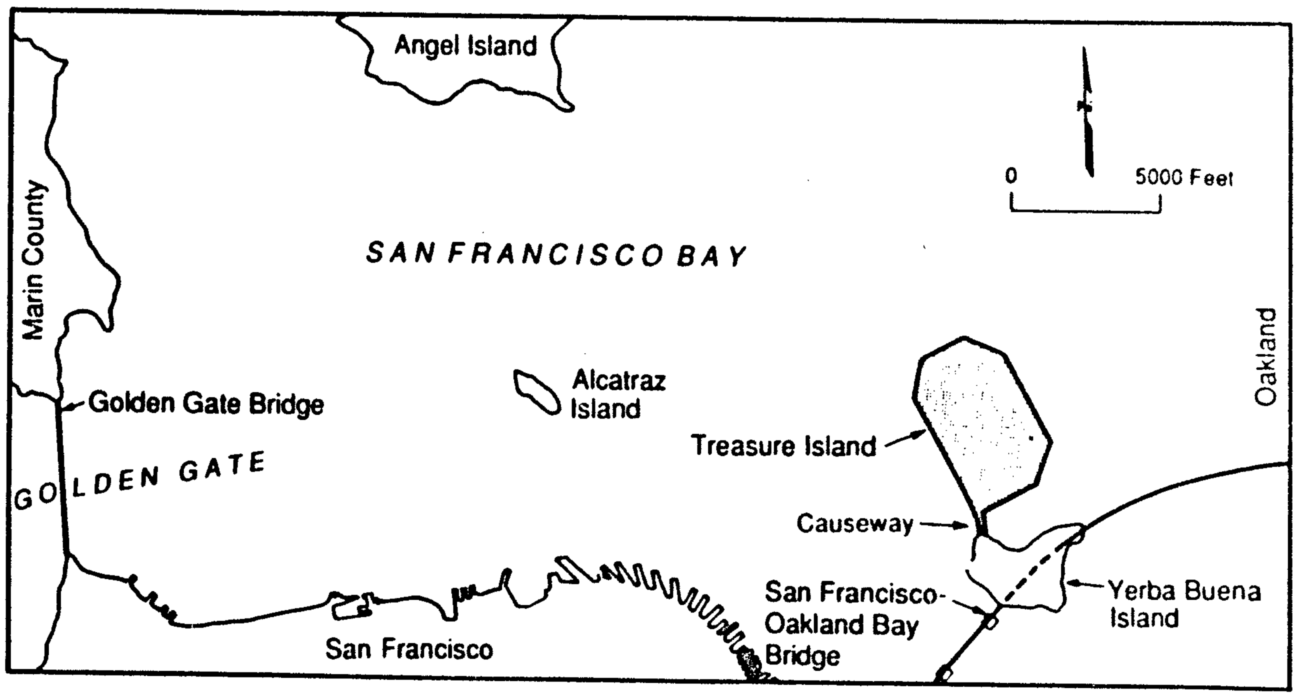


Figure 1. - Location map.

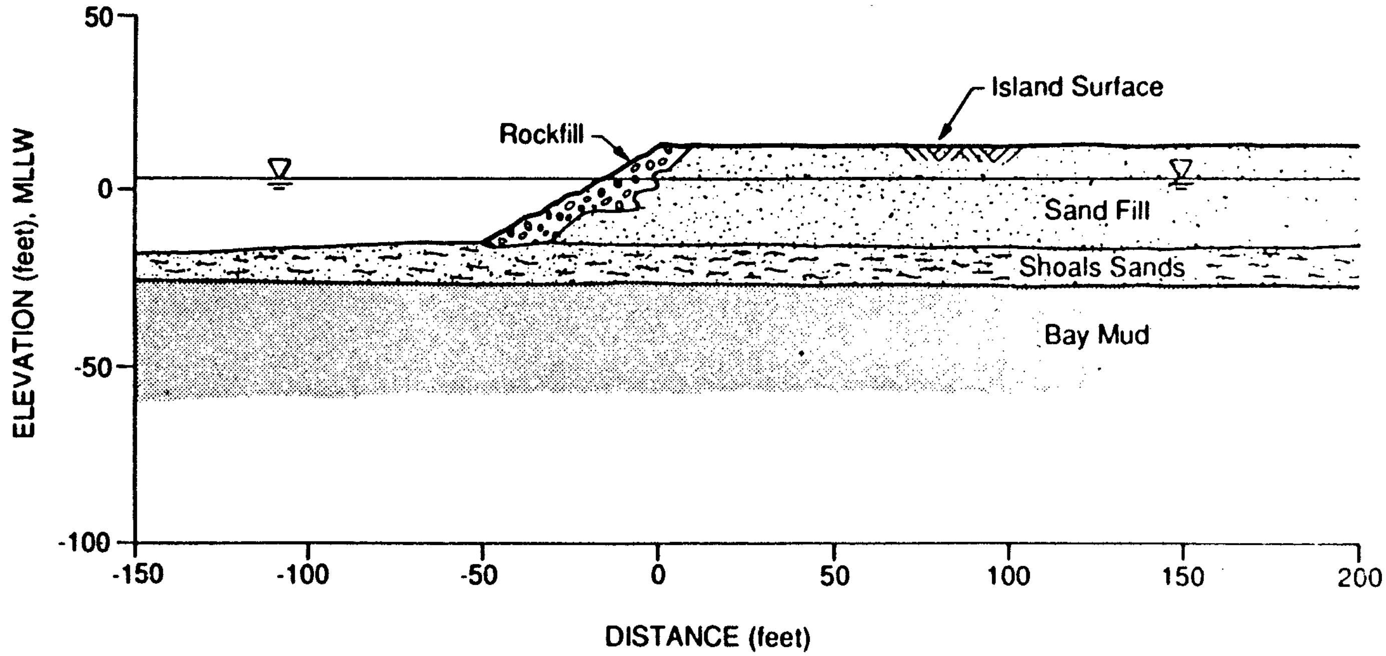
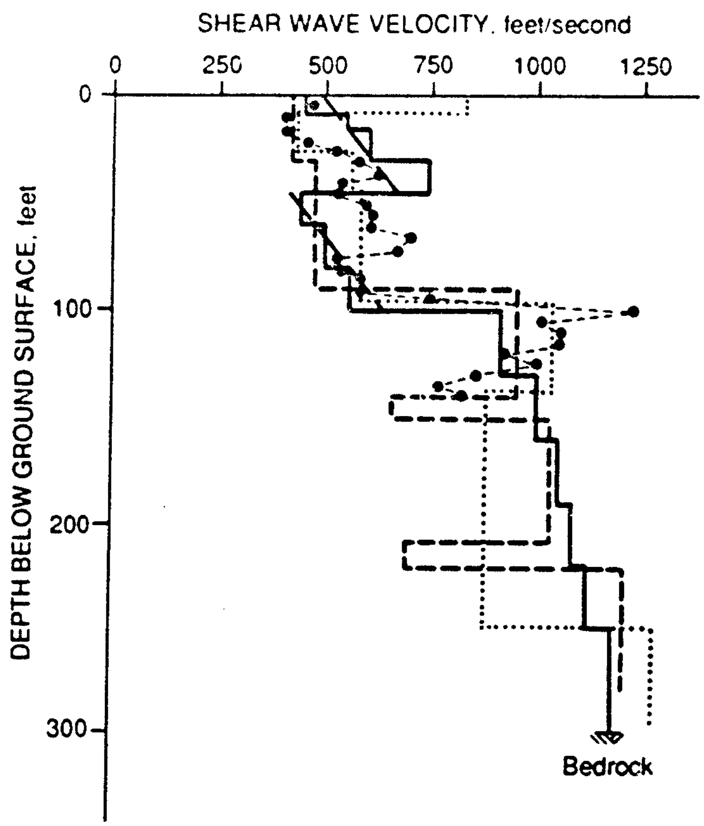


Figure 2. - Typical cross section through Treasure Island perimeter.



EXPLANATION

Measured (de Alba et al., in press)

Measured (Gibbs et al., 1992)

Measured (Hryciw et al., 1991)

Measured Redpath Geophysics, 1991)

Estimated (Geomatrix Consultants, 1990)

Figure 3. - Shear wave velocity profiles.

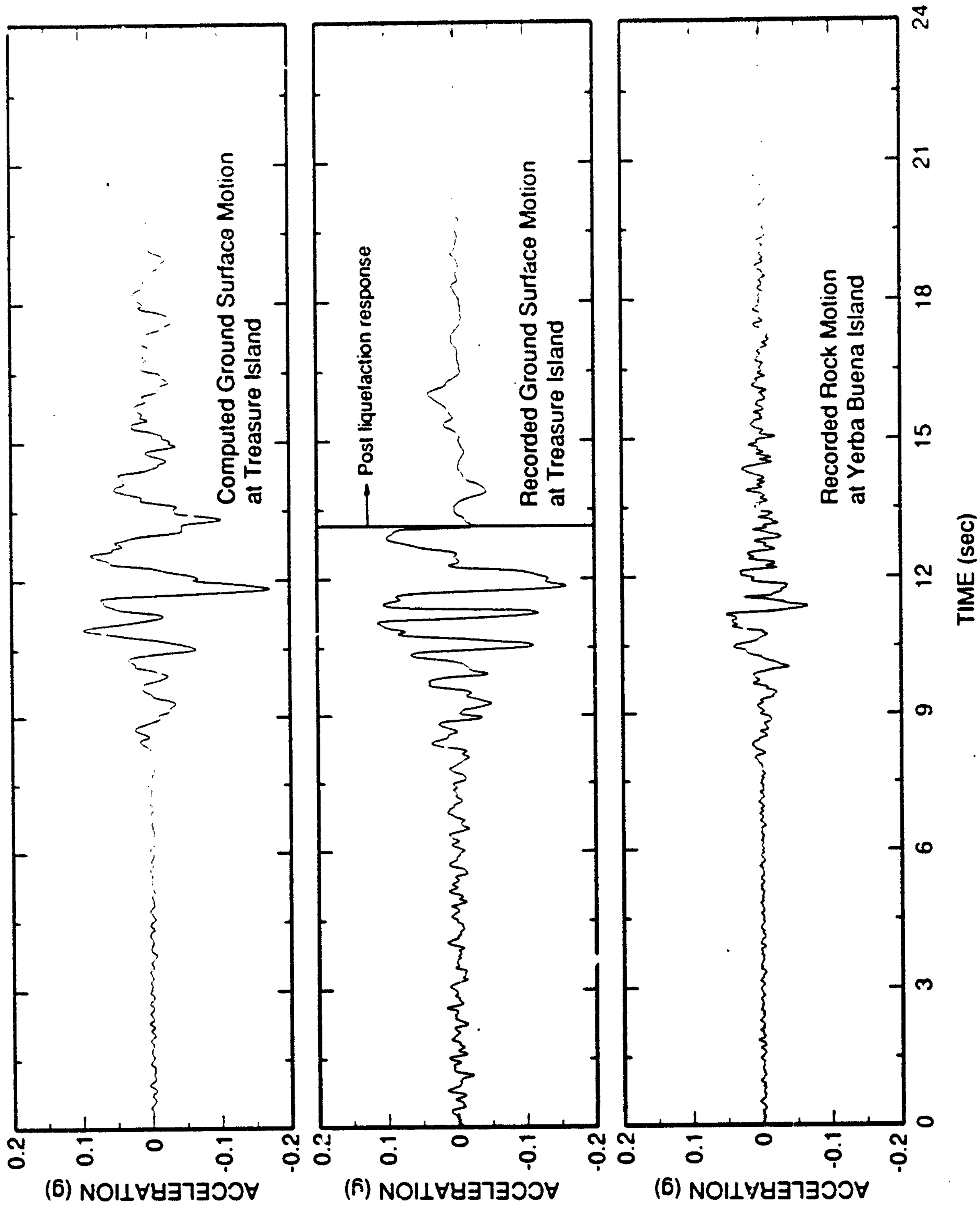


Figure 4. - Acceleration time histories.

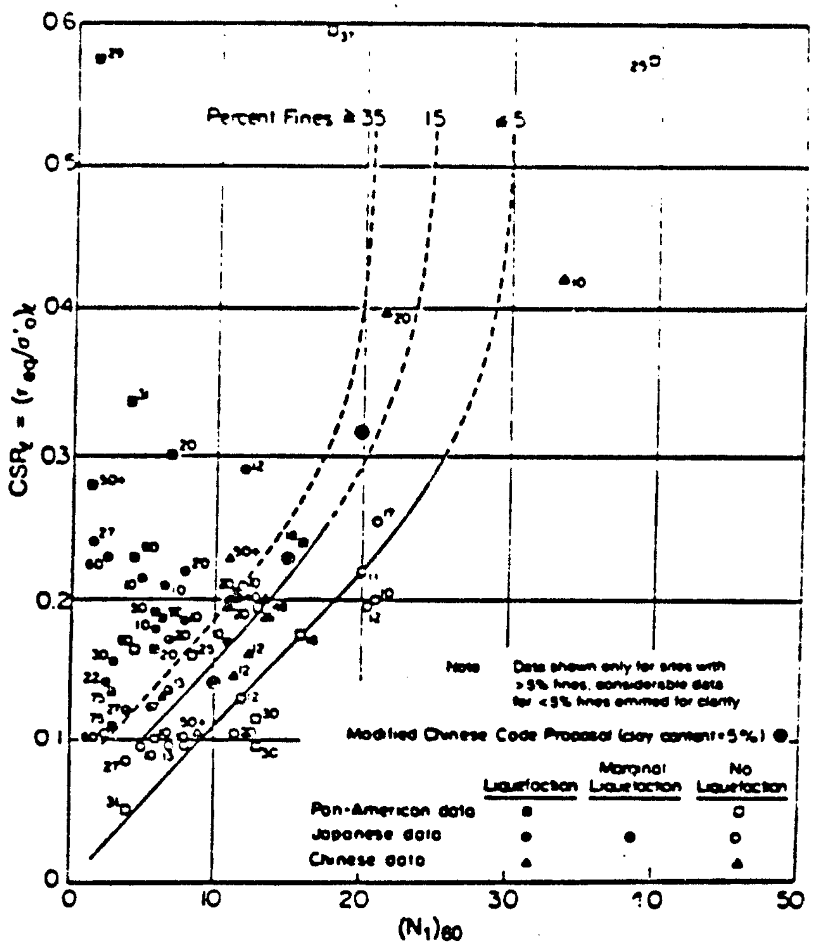


Figure 5. - Relationship between cyclic stress ratio causing liquefaction and $(N_1)_{60}$ values for M = 7.5 earthquake (after Seed et al., 1985).

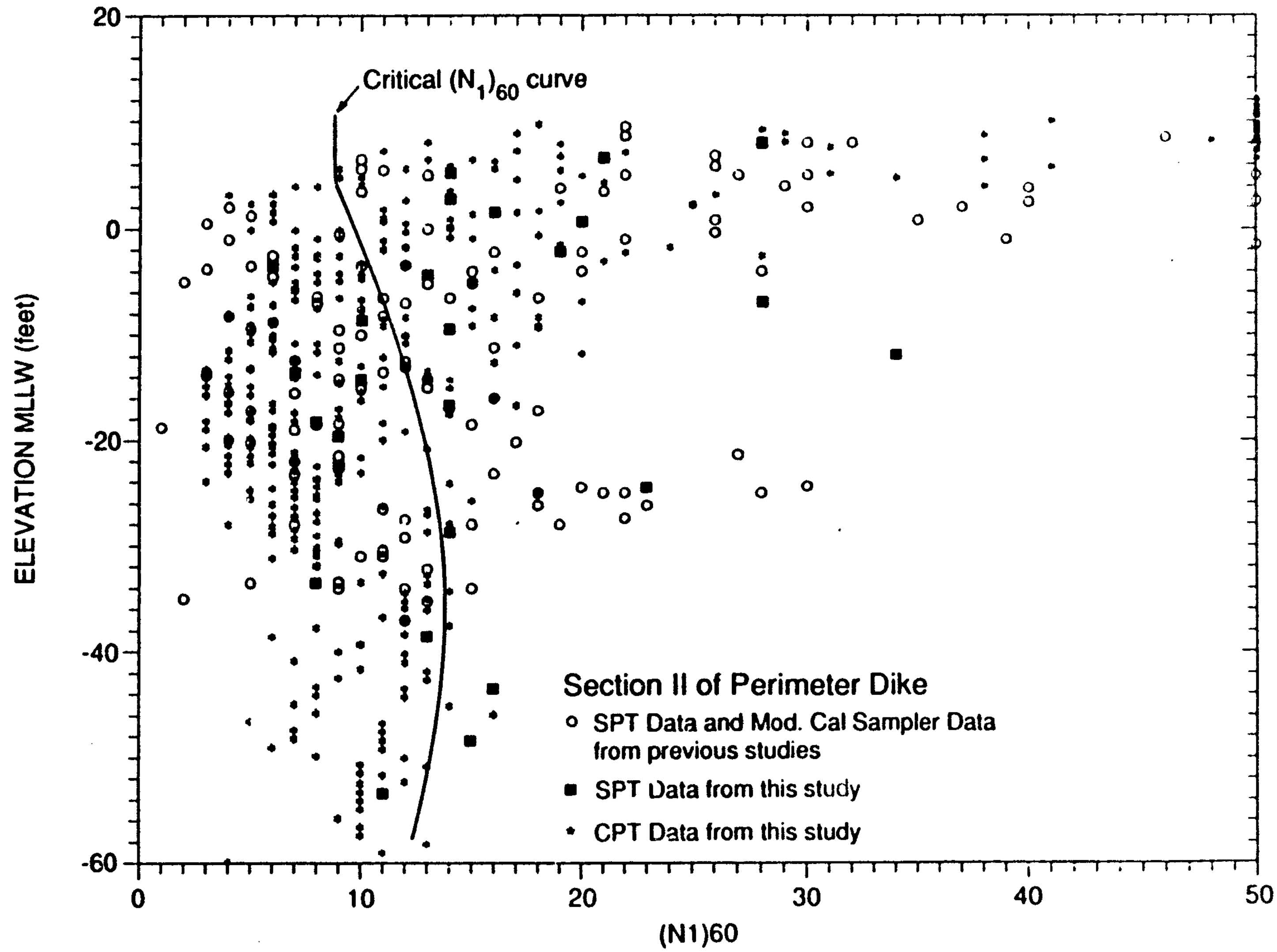


Figure 6. - Comparison of critical $(N_1)_{60}$ blowcount for liquefaction with $(N_1)_{60}$ data for Loma Prieta earthquake.

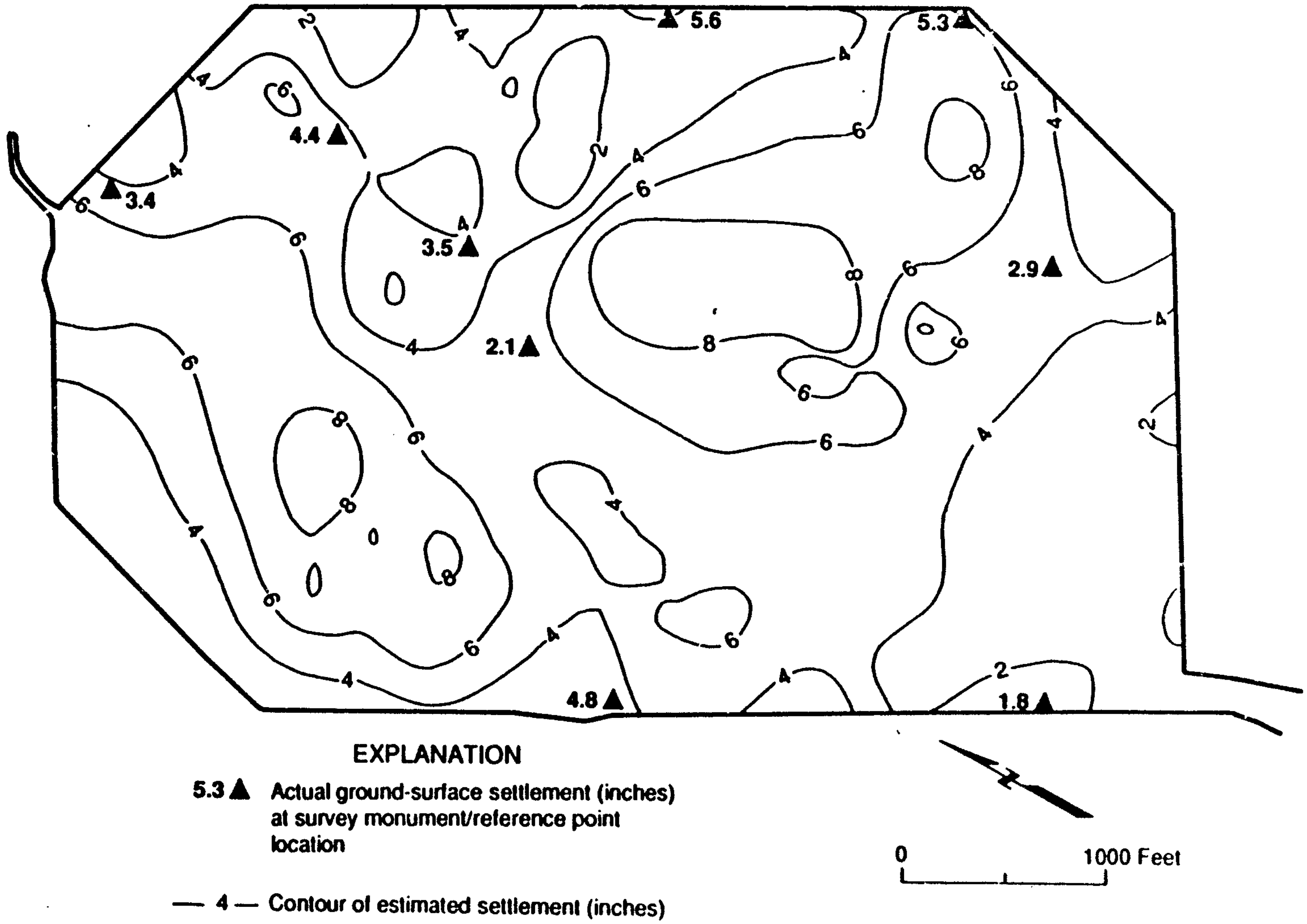


Figure 7. - Shaking-induced compaction settlement associated with Loma Prieta earthquake.

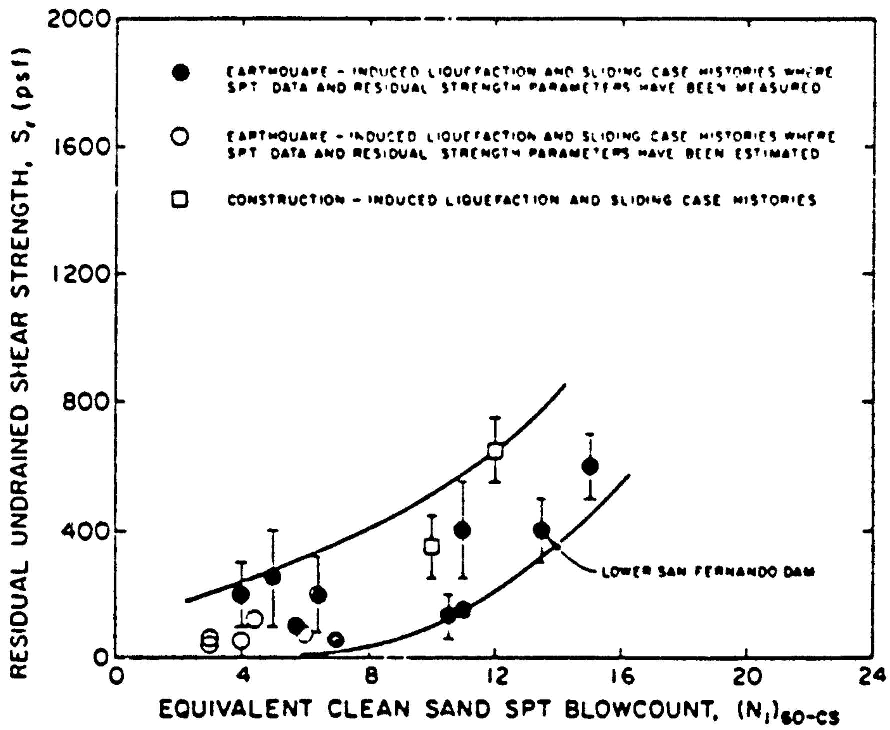


Figure 8. - Relationship between corrected "clean sand" blowcount $(N_1)_{60\text{-}cs}$ and undrained residual strength (S_r) from case studies (after Seed and Harder, 1990).

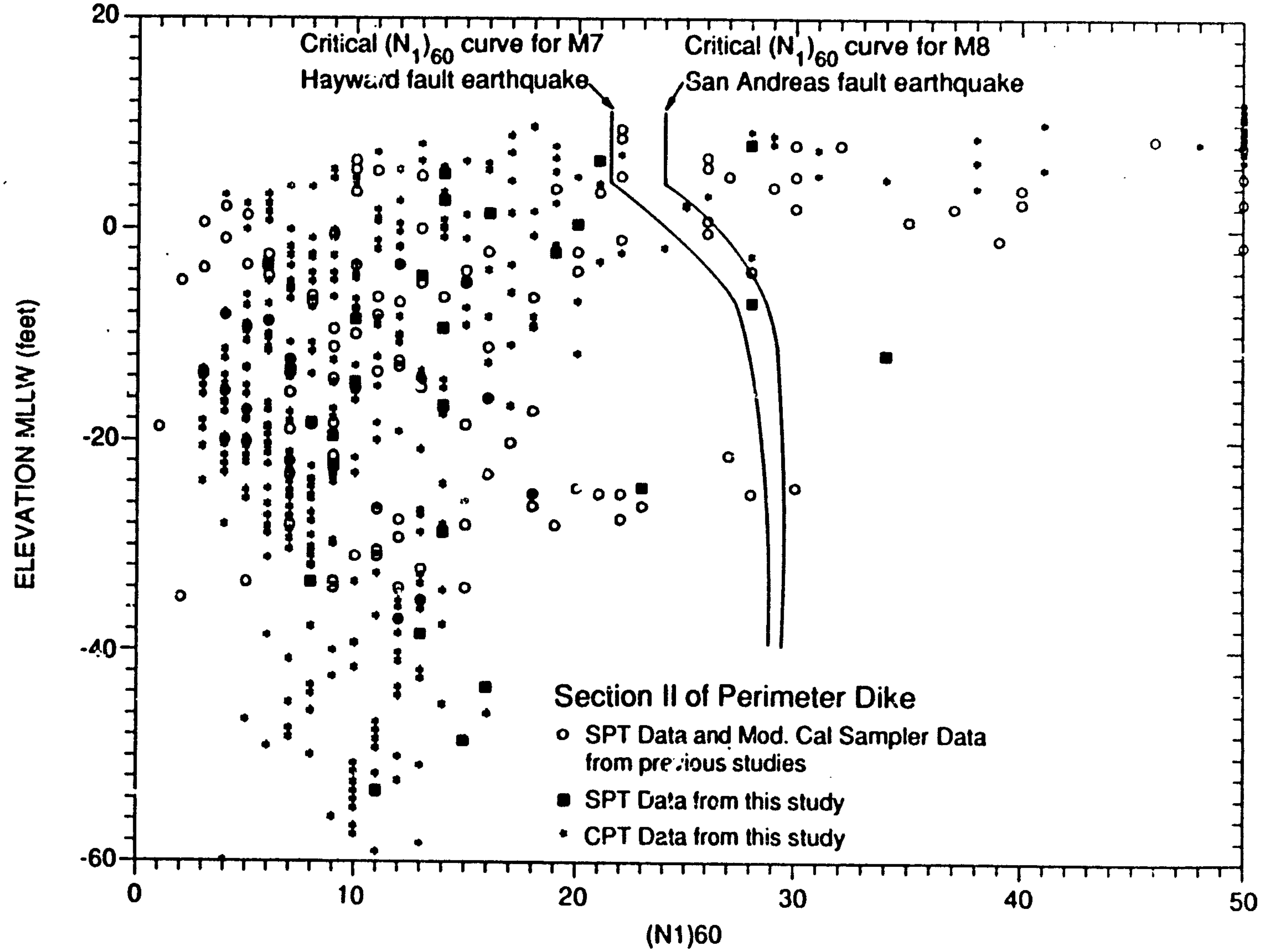


Figure 9. - Comparison of critical $(N_1)_{60}$ blowcount for liquefaction with $(N_1)_{60}$ data for maximum earthquakes.

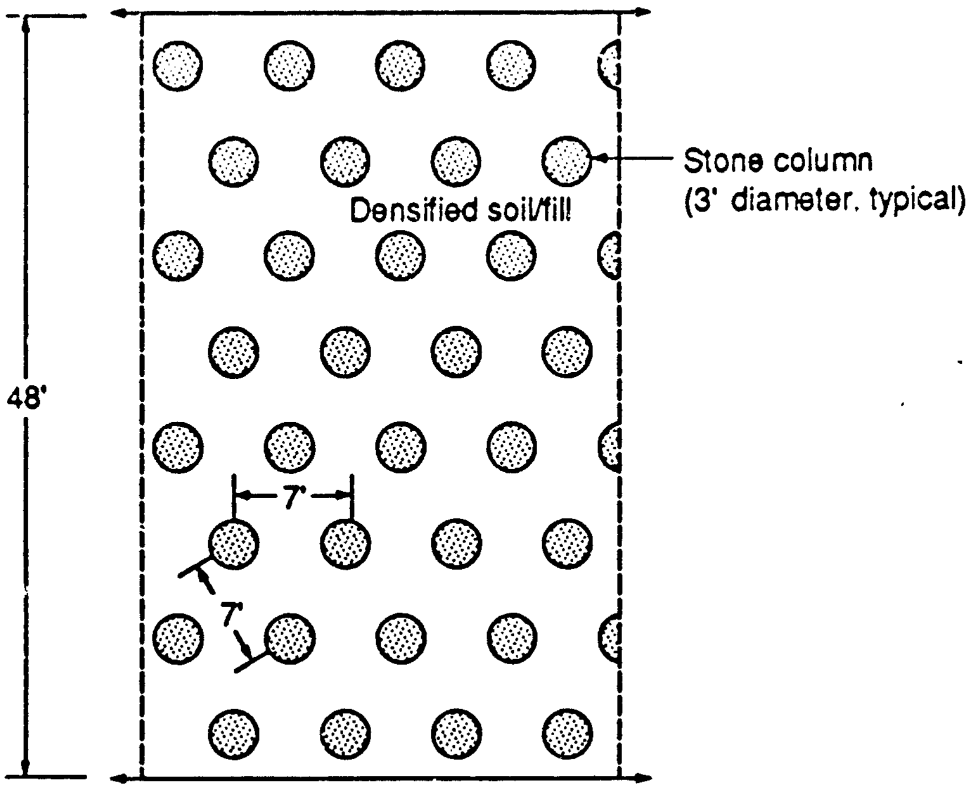


Figure 10. - Plan view of vibroreplacement configuration to mitigate liquefaction-induced lateral spreading at Treasure Island.

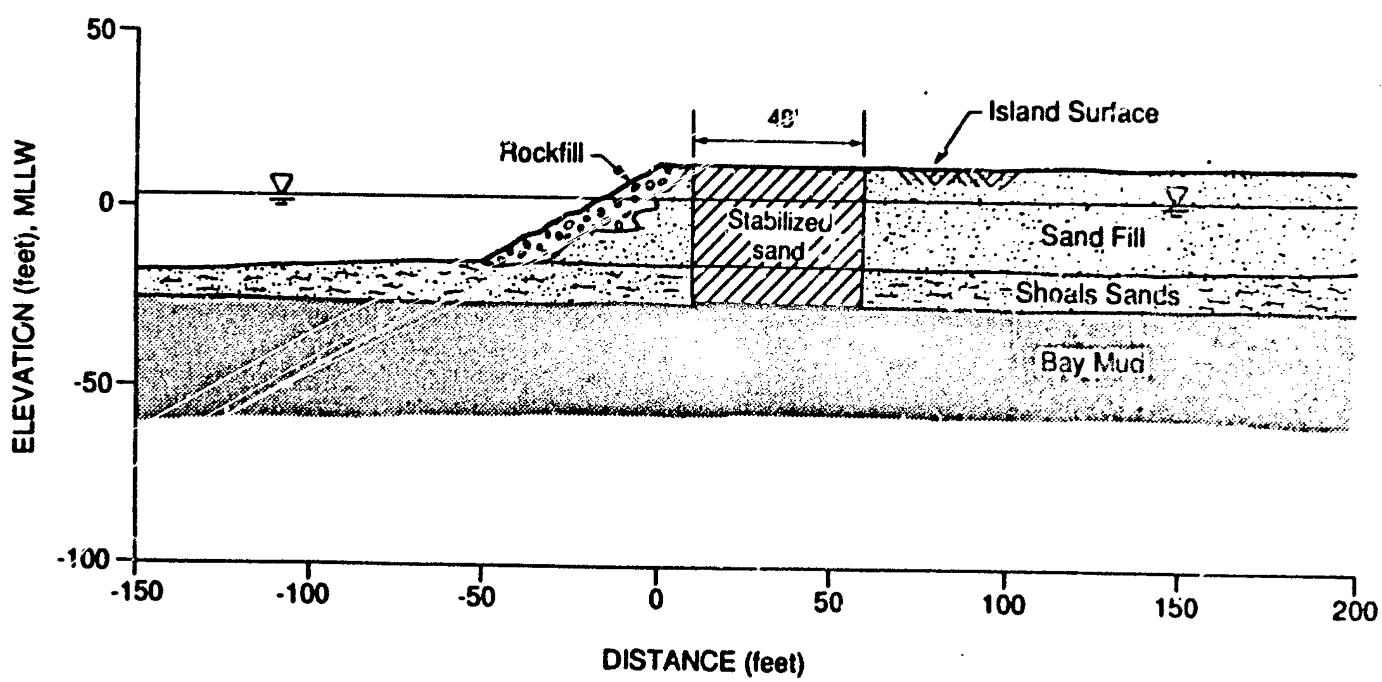


Figure 11. - Cross section of vibroreplacement extent to nitigate liquefaction-induced lateral spreading at Treasure Island.

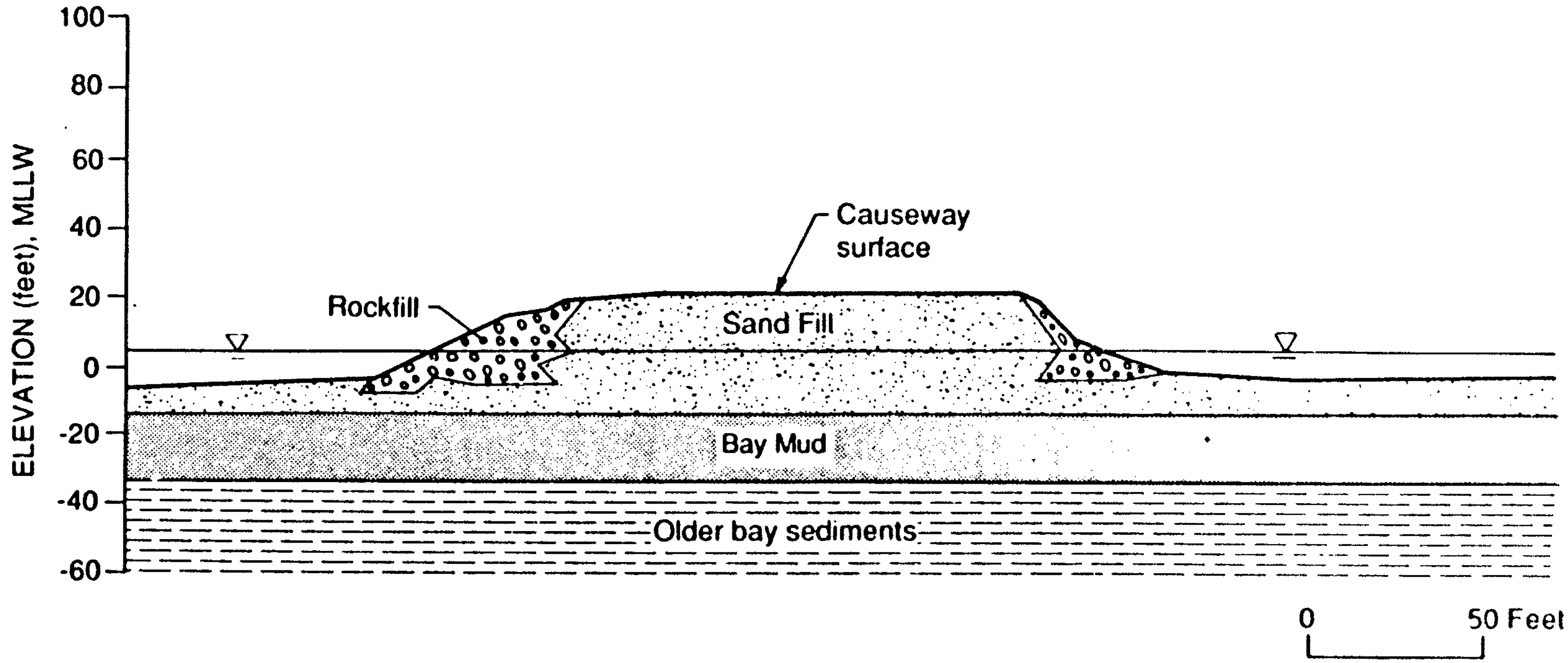


Figure 12. - Cross section through causeway to Treasure Island.

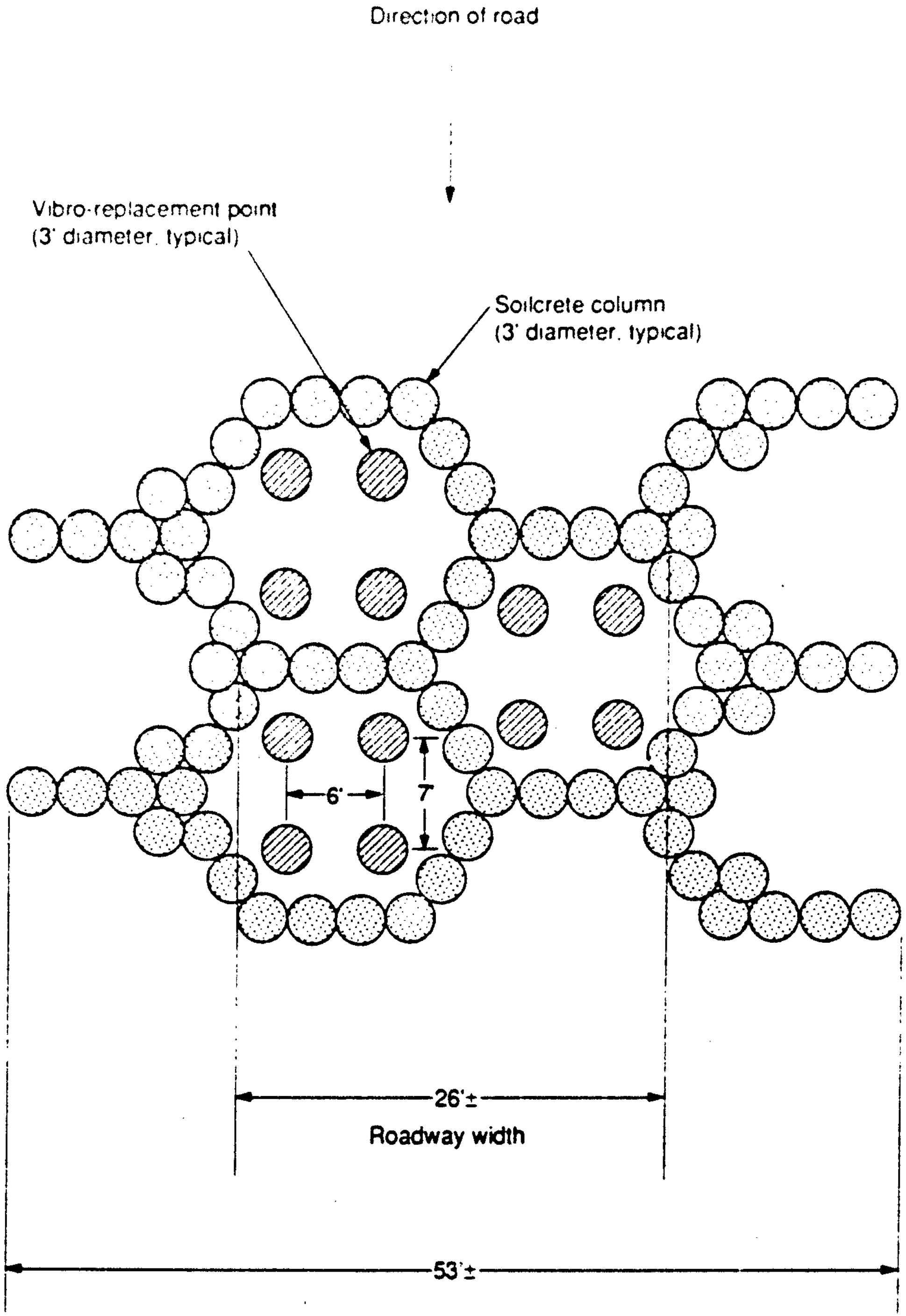


Figure 13. - Plan view of vibroreplacement/deep soil mixing configuration for causeway stabilization.

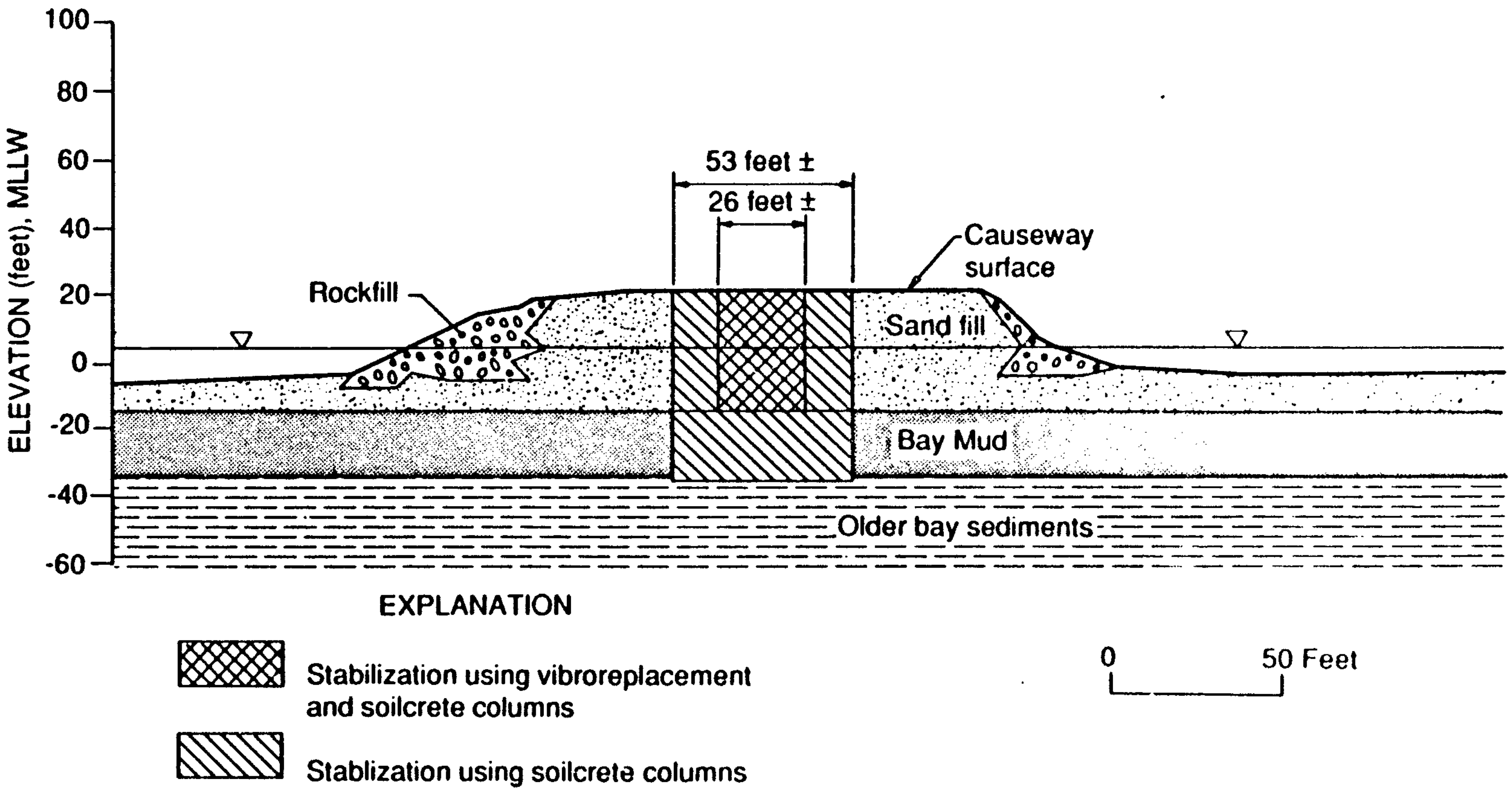


Figure 14. - Cross section of vibroreplacement/deep soil mixing extent for causeway stabilization.

IN SITU TESTING PERFORMED AT JACKSON LAKE DAM AND MORMON ISLAND AUXILIARY DAM

By Michael G. Stevens[1], J. Lawrence Von Thun[2], Jeffery A. Farrar[3], and Matthew G. Allen[4]

Abstract: In situ testing methods used to evaluate remedial treatment methods implemented to preclude liquefaction at Jackson Lake Dam and Mormon Island Auxiliary Dam are discussed. Multiple in situ measurement methods were evaluated for potential implementation at Jackson Lake Dam. Almost every type of in situ test known to be commonly available was investigated to some extent. The Standard Penetration Test (SPT) was selected as the most dependable and effective measurement tool for quantifying remedial treatment by densification. Some specialty testing techniques showed promise for future applications but require further development work before they can be useful for construction control. Dynamic compaction was selected to remediate apparently coarse-grained dredged alluvium foundation materials at Mormon Island Auxiliary Dam. The Becker Hammer Penetration Test (BPT) was selected as the primary tool for empirically estimating liquefaction potential before, during after remediation construction. Additionally, Standard Penetration Testing was performed in conjunction with installation of vibrating wire piezometers to further delineate material properties and stratigraphy, monitor pore pressure development during the compaction process and provide data for a site-specific SPT to BPT correlation. The BPT allowed for collection of a large data base for evaluation of compaction effectiveness. A unique method of data reduction and analysis was performed to visually depict the effectiveness of treatment at various treatment depths within the remediation zone. Preliminary evaluations were encouraging on treatment effectiveness. Further testing after completion of construction raised the question of quantifying friction effects on the Becker Hammer drive steel during testing. Results of preliminary SPT to BPT correlation testing, compaction effectiveness determinations and measurements performed in an attempt to quantify frictional characteristics of the BPT are summarized for discussion.

IN SITU TESTING EXPERIENCE AT JACKSON LAKE DAM

Introduction

Jackson Lake Dam in Grand Teton National Park in northwestern Wyoming was originally constructed in the early 1900s. Under Reclamation's Safety of Dams Program, the north embankment and embankment foundation were found to be susceptible to earthquake-induced liquefaction. Structural modifications were performed in two stages from 1986 through 1988 to remediate for stability concerns. Remediation construction included replacement of the north embankment and foundation treatments to preclude liquefaction.

Site investigations showed that the north embankment was founded on a complex interbedded fluvial/lacustrine deposit. The complex interbedding of gravel, sand, silt, and clay was caused by

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progradation of the Pilgrim Creek alluvial fan into paleo Jackson Lake during tectonic downdropping and rotation associated with the Teton Fault zone on the west side of the valley. A massive lacustrine clay and silt sequence underlies the fluviolacustrine soils at depth. A Reclamation seismotectonic study judged the Teton fault zone to be capable of a magnitude 7.5 earthquake at an epicentral distance of 4 miles.

Compaction piles were considered for foundation improvement and a test section was constructed in 1981. Results of various types of in situ testing, which were investigated to monitor this type of remediation for liquefaction and their potential for use in monitoring large scale construction, are summarized below. Detailed description of the compaction pile construction technique and results of testing to monitor treatment effectiveness are reported by Luebke and Young (1982). Although results of treatment by this method were encouraging, cost and time limitations prevented implementation to fully remediate the site. Final foundation treatments implemented included dynamic compaction and the Soil Mixing Wall (SMW) method to construct a deep mixed-in-place soil cement reinforcement grid under the upstream and downstream toes of the new embankment. SMW was also used to construct a cutoff wall along the upstream toe of the new embankment. Detailed reports on the modifications at Jackson Lake Dam can be found in reports to the U.S. and International Committees on Large Dams (Luebke et al., 1991; Farrar et al., 1990).

In situ testing played an important role in foundation investigations. Due to the complex nature of the foundation, investigation programs stressed the use of simple penetration and geophysics testing to determine areal extent of liquefiable deposits. The primary methods implemented for evaluation of liquefaction potential were Standard Penetration tests (SPT), Cross Hole Shear Wave Velocity tests, and Cone Penetration tests (CPT). Results of these investigations delineated potentially liquefiable sands, silty gravels, silty sands, and low plasticity silts both within the embankment and at depths of up to 100 ft within the foundation.

Cone penetration tests provided excellent information on stratigraphy including mapping of remnant lacustrine layers in the foundation. However, cone penetration tests were severely hampered by a significant amount of coarse gravels associated with fluvial deposits which had variably cut lacustrine deposits. The gravel deposits necessitated the use of drill rig preboring and resulted in reduction of thrust capacity due to rod bending. Reclamation developed some innovative methods for sand backfilling of prebores to partially increase thrust capacity. The fluvial deposits were extremely variable laterally and often boreholes located 5 to 10 ft apart would not intersect the same soils at equivalent elevations.

Attempts were made to correlate penetration and shear wave velocity testing to engineering properties obtained from several "specialty testing" methods where undisturbed samples were obtained within the compaction pile test section area and where remediation by dynamic compaction had taken place. Almost every type of in situ test known to be commonly available was investigated to some extent for potential application at Jackson Lake Dam. The following specialty testing techniques were investigated:

Borehole Shear Test (BST)
Menard Pressuremeter Test (MPT)
Rock Dilatometers
Stepped Blade Test (SBT)
Self Boring Pressuremeter Test (SBMPT)
Flat Plate Dilatometer (DMT)
Spectral Analysis of Surface Waves (SASW)
Nuclear Borehole Geophysics
Seismic Tomography

Borehole Sonic Logging
Stress Captors

Although some of the specialty testing techniques investigated showed promise for future applications, as may have become more evident upon further development and verification, the SPT was selected as the primary monitoring tool for empirically evaluating liquefaction potential just prior to, during and after remediation construction at this site.

Compaction Pile Test Section Monitoring

Compaction piles were tested for their ability to improve potentially liquefiable soils in a series of test sections performed in 1981. The compaction pile is similar to extrusion-type Franki piles where cohesionless gravel backfill is extruded through a 20-inch tube with 10,000- to 15,000-lb hammers (Technical Report of Construction, 1981). Two test sections (A and B) were performed containing hexagonal grids of 60-ft-deep compaction piles spaced at 6-, 8-, and 10-ft spacings. Investigations consisted of undisturbed sampling, SPT, CPT, and Shear Wave Velocity determinations performed before and after construction. SPT and shear wave velocity tests were the best indicators of liquefaction resistance (Luebke and Young, 1982). SPT and crosshole shear wave determinations were performed in drill holes located in the center of a triangle defined by three piles. Test section site B contained significant gravels and there were only a few sets of SPT N-value data for evaluation. At the sites containing gravels the shear wave velocity was found to be the best indicator of improvement.

Efforts were made to measure increases in horizontal stresses from compaction pile improvement. The data have not been published but is available for inspection at the Reclamation's Denver Office Research Laboratories. Stress captors were installed at several elevations at site B in the center of a triangle defined by three piles. The stress captors consisted of rubber membranes similar to pressuremeter cells which were pressurized to stresses slightly exceeding in situ horizontal stress prior to compaction. The captors were monitored for a period of 1 year after compaction. In one case, a series of captors were monitored during pile driving. An adjacent piezometer was also monitored which allowed for evaluation of effective stress path during pile driving and extrusion. Results of the captor installation indicated K_o values increased from 1.0 to 2.5 with only minor relaxation after 1 year. The data are felt to be fairly reliable due to dramatic yielding of soils surrounding the captors. The captors however still suffer from modulus incompatibility, which is a major problem when measuring in situ stresses by known methods. Efforts continued during dynamic compaction construction to measure in situ stress changes with stress captors but were abandoned because of poor installation and the lack of soil alteration around the probe, i.e. captors could not handle dynamic compaction shock and at rest stress changes were smaller.

At compaction pile test site A there was a significant effort to quantify the liquefaction susceptibility of soils by the "steady state" approach advocated by Poulos et al. (1985). At this site, a 3- to 5-ft-thick clean sand layer was available for testing in addition to more prominent silty sands and low plasticity silts. Detailed fixed piston samples were taken with extensive stroke recovery measurements for accurate determination of void ratio. A detailed description of this testing and summary of results is provided in a report prepared for Reclamation by Geotechnical Engineers, Inc. (1985).

Reclamation also evaluated electrical methods in an attempt to measure in situ state or dilativeness index through determination of soil type and porosity. A testing program was performed using an electrical formation factor probe and dielectric probe developed by Arulanandan (1985). Results of this study, and other in situ measurements taken during and following test section construction, are summarized on figure 1. The data show that the formation factor probe, shaped like a 3-inch Shelby tube, was not consistent in measuring liquefaction resistance in the treated denser sand due to disturbance during insertion of the probe. The dielectric probe did predict porosity well, and does have the potential for implementation as a rapid in situ testing tool for estimating this parameter (Farrar et al., 1988).

Dynamic Compaction Monitoring

The dynamic compaction program consisted of dropping a 30-ton weight from a height of 100 ft for a specified number of drops on a specified pattern. The drop grid had primary impact points spaced on 40-ft centers with 20 drops at each impact point. The treatment was then completed with 15 secondary and 10 tertiary drops at 40- and 20-ft closure spacings.

Remediation construction was performed under two contracts. Differences in construction and monitoring methods needed to be considered during treatment adequacy determinations. A total of approximately 500,000 ft^2 of dynamic compaction was performed in Stage I contract and about 230,000 ft^2 in Stage II. The dynamic compaction treatment area was divided into 14 sectors, A through Q, for control purposes. Additional information on the dynamic compaction program can be found in cited references (Luebke et al., 1991; Farrar et al., 1990).

Principal methods for evaluating improvement by dynamic compaction were SPT and crosshole shear wave velocity measured at selected sites within the remediated area. Contract cone penetration testing was performed in Stage I but the program was abandoned after it became evident that the presence of gravel layers slowed testing to the point that it effected dynamic compaction production. Approximately 20 CPT soundings and 5 DMTs were performed by Reclamation for precompaction and postcompaction comparisons. Data have not been published to date but are available for review at Reclamation's Denver Office Research Laboratories. A considerable amount of specialty testing was also performed to evaluate improvement.

Primary monitoring of Stage I was performed by drilling 168 SPT drill holes before and after compaction. For Stage II monitoring 153 SPT drill holes were drilled. SPT holes were located in regular patterns of 50- to 80-ft spacings. Samples were taken at 2.5-ft intervals to depths of 35 to 50 ft. Drilling was performed by contractors. SPT testing was accomplished generally within 1 to 2 weeks after completion of the dynamic compaction phase to be studied. Reclamation drilled an additional 25 SPT holes approximately 9 months after dynamic compaction to evaluate aging effects.

There were very significant procedural differences in performing SPTs among Stages I and II contractors. The Stage I SPTs were performed using safety hammers by the rope and cathead method using tricone rotary rock bits and drilling mud. The Stage II contractor performed SPTs using a CME (Central Mine Equipment Company) automatic hammer and were advanced with rotary casing advancer. The most significant difference between Stages I and II SPTs was the energy delivered to the sampler. Seed et al. (1984) have shown the significance of SPTs with regard to liquefaction analysis. Reclamation performed energy measurements in accordance with ASTM (American Society for Testing and Materials) test designation D 4633 (ASTM, 1986). Only 10 series of tests of approximately 200 blows of data were collected for both the Stages I and II contractors. The best estimate of drill rod energy ratio, ER_i, was 73 and 97 percent for Stages I and II contractors, respectively (Farrar, 1991). Over 4,000 SPT tests were analyzed to evaluate energy differences after construction (Farrar and Yasuda, 1991). The analysis showed that there was a 50 percent energy difference between contractors and that the SPT N-value was inversely proportional to energy regardless of soil types. The disagreement in apparent energy differences point to the difficulties with energy measurements and the need to consider drilling methods as an additional variable in correcting SPT data to acceptable sand-equivalent $(N1)_{60}$ values.

During construction, a direct comparison approach was used to evaluate improvement. In the direct comparison approach, neighboring boreholes within 5 to 10 ft are drilled before and after different phases of compaction. SPT N-values were converted to sand-equivalent $(N1)_{60}$ values through procedures developed by Seed et al. (1984). Also, a clean sand equivalent N offset, determined from earlier site investigations, was used to evaluate soils not directly applicable to the standard SPT liquefaction correlation. The direct comparison approach allowed for real time evaluation to ensure that design criteria were being met and also had the advantage of delineating spatial distribution of improvement, clarifying effects of stratigraphy and detecting potential planes or zones of weakness. In many cases, comparison of critical sand layers at the same elevations was not possible due to the complex fluvial stratigraphy. In Stage I, SPTs were performed after specific phases of compaction. At a single location where soil conditions were critical there were two and sometimes three postcompaction SPT borings. In other noncritical locations there were no postcompaction borings. Through analysis of improvement by phase, it was determined that the complete four phases of dynamic compaction were necessary for required improvement. In Stage II, each precompaction boring was matched with a postcompaction boring drilled after the final phase of compaction. A summary of improvement in critical liquefiable soils using the direct comparison approach is given in tables 1 and 2. The data indicate that significant improvement is realized in critical soils and generally larger improvement was realized in Stage II construction. Details of direct comparison analysis techniques can be found in the references by Von Thun (1988, 1989).

Another approach to SPT data evaluation is from a statistical analysis standpoint. An analysis of over 4,000 SPT data points was accomplished by incorporation into a database (Farrar and Yasuda, 1991). The database includes laboratory gradation and moisture content information. While this method is not entirely feasible to apply during construction, it does provide powerful insight into the effects of dynamic compaction treatment adequacy. Through the statistical approach, the following effects could be more clearly analyzed; improvement by soil type, effects of SPT energy and drilling method, improvement by phase of dynamic compaction, and effective depth of improvement. With regard to liquefaction improvement Reclamation evaluated the effects of gravel content, importance of no-recovery tests and the "Clean Sand Equivalent Offset" proposed by Seed (1987). Table 3 summarizes improvement in raw N-value for predominant soil types from Stage II by the statistical approach. At first glance one may conclude that the soils are not potentially liquefiable. However, the data for sands is influenced by gravels and additional analysis was required to determine N-values in sands not influenced by gravels. Also, the material with no recovery appears most liquefiable in precompaction data. Effective depth of improvement is summarized for Stages I and II on figures 2 and 3. The figures show that effective depth of treatment is from 35 to 45 ft. Some important conclusions of the statistical analyses were that gravels had a significant influence on evaluating liquefaction resistance, that the clean sand equivalent offset values recommended by Seed (1987) need further evaluation, and that clean sand equivalent offset values may be a function of soil density and stress-related conditions on a site-specific basis.

Two special studies were performed during dynamic compaction in an effort to develop and/or consider less costly methods of evaluating improvement and to evaluate in situ stress changes created during and following treatment. Foundation shear wave velocity was evaluated by surface wave and crosshole techniques at three sites before and after dynamic compaction. Increase in shear wave velocity at the three sites is summarized on figure 4, as taken from Sirles (1988). Results indicate the less costly surface approach may have merit in evaluating dynamic compaction at other sites, but the complex layering and lack of lateral homogeneity in the Jackson Lake foundation caused difficulties in data interpretation (Sirles, 1988).

Attempts were made to measure in situ stress changes using pressuremeters, stepped blades, and flat plate dilatometers along with SPT, CPT, and undisturbed sampling and testing at two sites; one being primarily low plasticity silts (lacustrine deposit) and the other being silty sands. Results indicate increases in in situ horizontal stresses from 50 to 100 percent (Woodward and Clyde Consultants, 1987). An example of typical test results is shown on figure 5 for precompaction and postcompaction results within Sector C. Electrical measurements were also performed. Again, it is apparent that measurement of in situ horizontal stresses using different methods result in a wide variation of values. Interpretation of the different in situ test results at Sector C were complicated by the difficult stratigraphy of the Lacustrine deposits which contained alternating thin layers of soil of differing drainage properties. Presence of thin silty sand layers in lacustrine deposits will cause variable drainage conditions for different in situ tests.

SMW Testing

The SMW process is a construction method which mixes a hardening or stabilizing agent with soil in place to form columns of treated material. At Jackson Lake Dam it was desired to form reinforcing cells constructed of columns with shear strength of approximately 200 lb/in^2. Cement slurry grout is pumped from a batch plant through hollow-stemmed earth augers as they penetrate the ground. Two and three 36-inch diameter auger machines drilled overlapping columns in hexagonal grid and line patterns to depths of up to 110 ft. The deep treatment grid pattern was constructed over a total surface area of 216,000 ft^2 with an average depth of 71 ft, forming a block volume of treated foundation material of approximately 568,000 yd^3. Approximately 427,000 lin ft of column was constructed using 38,900 tons of cement for a total cost of approximately $14.25 million. Approximately 248,312 ft^2 of cutoff wall, having a length of 3,985 ft and an average depth of 62.3 ft, was constructed using 8,239 tons of cement for a total cost of approximately $4 million. Primary methods for quality control were wet sample testing, coring of hardened columns and related laboratory strength testing of samples.

Limited in situ testing was performed to evaluate strength and modulus of soil cement columns. Use of in situ tests allowed for determination of engineering properties in areas of poor recovery. The Menard pressuremeter was used to evaluate modulus of deformation. The rock borehole shear test was used to estimate shear strength of cures SMW columns. Since shear strength was a major concern, the borehole shear test results could be very important for predicting shearing resistance in areas of poor recovery. The advantage of using this device is that rapid in-hole Mohr Coulomb stress paths can be obtained. Unfortunately testing with the rock bore shear testing device was attempted late in the program. Figure 6 illustrates results of our only comparison testing. The predicted compressive strengths in this comparison are weaker than that of surrounding cores. The explanation was that the borehole had not been tested until 54 days after coring which resulted in weakening of material in the borehole wall. The borehole shear device may prove quite valuable if further developed to estimate in situ strength of mixed inplace soil cements. Nuclear downhole Geophysical testing was also performed in core holes.

Reclamation has employed seismic tomography for delineating cutoff wall continuity and geologic properties in two dimensions on several major projects. Seismic tomography was performed along two rows of SMW columns by using a string of receivers to record multiple ray paths between drill holes spaced at 10 and 25 ft. Two-dimensional images of P wave velocity and attenuation were obtained (Wong and Nelson, 1989). An example of seismic tomography imaging could not be presented as obtained at this site, but, if proven economical, could be advantageous for monitoring cutoff wall continuity at other sites. Additional work is being completed to evaluate shear modulus and develop correlations with engineering properties such as shear strength and permeability.

IN SITU TESTING AT MORMON ISLAND AUXILIARY DAM

Introduction

Mormon Island Auxiliary Dam (MIAD) is the largest earthfill dike on the Folsom Dam and Reservoir Project, a key feature of California's Central Valley Project (CVP), located near Folsom, California. MIAD is 4,820 ft long and is 165 ft high at the maximum section. The structure was designed and constructed by the U.S. Army Corps of Engineers (COE) in the late 1940s and early 1950s. Ownership was transferred to Reclamation upon completion for operation and maintenance.

Portions of the embankment are founded on alluvial materials that were dredged and processed for gold on several occasions in the first half of the century. The dredging process redeposited the alluvium in a very loose condition. Preliminary Safety of Dams investigations (COE, 1990a; COE, 1990b) indicated the dredged alluvium foundation consisted primarily of gravels with cobbles and sand, sandy gravels, cobbles with gravel and sand and gravelly sands. Due to the apparent coarse nature of these materials, the Becker Hammer Penetration Test (BPT), as defined by Harder and Seed (1986), was selected as the primary tool for empirically estimating liquefaction potential based on penetration resistance. It was concluded from the BPT and related investigations that extensive liquefaction was expected and that catastrophic loss of reservoir may result under earthquake loading.

Drought conditions and low reservoir levels in 1990 provided an opportunity to perform remedial treatment for liquefaction upstream of the embankment. Densification by dynamic compaction was selected based on time available for construction, technical feasibility and overall cost. Additional studies (Ledbetter et al., 1986) defined required treatment to full depth of the dredged alluvium. To address the probability of not treating below a 40-ft depth by dynamic compaction, a working platform berm was constructed to allow access for future remediation to depth without lowering of the reservoir.

BPTs were performed within the remediation zone before (precompaction), during (intermediate or postphase 2 compaction) and after (postcompaction, postberm and cased/uncased investigations) dynamic compaction. Additionally, Standard Penetration Testing (SPT) was performed in conjunction with installation of vibrating wire piezometers to (1) further delineate material properties and stratigraphy; (2) monitor pore pressure development during compaction; and ultimately (3) provide data for a site-specific SPT to BPT correlation. SPT investigations performed along the upstream side of the embankment showed great quantities of clean sands with fine gravel, clean sands and silty sands than indicated by preconstruction explorations. Finer grained materials were particularly evident within the bottom 15 to 30 ft of the zone requiring treatment.

In addition to penetration testing, crosshole shear wave velocity, spectral analysis of surface waves (SASW) and nuclear downhole geophysical testing were performed to evaluate treatment. The remainder of this paper concentrates on presentation of analyses performed related to penetration resistance testing.

COMPACTION PROGRAM

Dynamic compaction was performed on a grid pattern consisting of primary, secondary and tertiary tamper impact points. The grid had primary impact points on 50-ft centers with 30 drops specified at each impact point (Phase 2) and was completed with 30 secondary (Phase 3) and 15 tertiary (Phase 4) drops at closure spacings. The disturbed compaction working surface was densified with 2 drops of the tamper edge-to-edge (Phase 5). A 50-ft primary drop spacing was selected in an attempt to achieve a 50-ft treatment depth based on the assumption that the majority of materials requiring treatment were free-draining gravels with cobbles and sand. The specified remediation treatment zone width of 100 ft was increased in width to 150 ft during construction based on SPTs indicating finer grained materials were present than had been estimated from preconstruction investigations. The treatment length was decreased from about 900 to 800 ft based on BPTs showing nonliquefiable undredged materials were present at the east and west edges of the specified treatment area. A plan view of the treatment area and locations of in situ testing are shown on figure 7.

IN SITU TESTING SUMMARY

Thirty BPTs, seven SPTs, and one crosshole shear wave velocity triplet were completed from the compaction working surface, elevation 375 ft, to bedrock prior to performing dynamic compaction. Seventeen BPTs were performed after completing Phase 2 of the dynamic compaction program. SASW testing was completed during Phase 5 compaction. Forty-two BPTs and five side-by-side SPTs were performed within 2 weeks following compaction construction. Five to six months after compaction, seven BPTs and three side-by-side SPTs were performed through the berm (postberm) in an attempt to quantify the aging effects on treatment and obtain additional BPT to SPT comparison data. Work through the berm added about 55 ft to the depth of drilling compared to explorations performed before, during and immediately after compaction construction (postcompaction data). Nuclear geophysical testing was also completed during this time period. About 1 year after dynamic compaction, four cased and uncased BPTs with side-by-side SPTs were completed through the berm. This work was done to determine whether or not friction on the Becker Hammer casing being driven through the berm and/or the heavily-compacted upper layers of the treated zone played an important role in evaluating the blowcount increases indicated by BPT data collected immediately after compaction. Two of the BPT locations were cased through the berm to the top of the remediation zone and two were cased to about a depth of 30 ft within the remediation zone. Followup crosshole testing will be completed in the near future at 3 locations within the remediated zone. A plan view of the remediation zone and locations of BPTs and SPTs are shown on figure 7.

The BPT method was very productive, allowing efficient evaluation of dynamic compaction. Performing SPTs was very limited due to difficult drilling conditions. Significant wear on SPT equipment took place and resulted in unforeseen additional cost. During postcompaction testing forty-two 60- to 70-ft BPT soundings were completed in 10 working days, providing data for every 1-ft interval. In about a 2-week period five SPTs were completed with sampling and were performed at 5-ft intervals within the bottom 20 to 30 ft of the treatment area. It should be noted, however, that even if considered costly and difficult to perform, the SPT data are invaluable in refining and ultimately finalizing a BPT to SPT correlation.

REDUCTION OF RAW BPT DATA TO $(N1)_{60}$ SAND-EQUIVALENT VALUES

As stated, the BPT quickly produces large amounts of data. Experience with this method is somewhat limited, and therefore it is customary to adjust BPT blowcounts to equivalent SPT blowcounts for further analyses. For this project, the blowcount and bounce pressure data collected in the field were adjusted to equivalent SPT N_{60} values (Harder and Seed, 1986), and further refined as recommended by Seed et al. (1984; 1987) to $(N1)_{60}$ sand equivalent penetration resistance values. Simply stated, BPT data collected before, during and after compaction were adjusted for the following factors (Harder and Seed, 1986):

1. Variable internal combustion conditions effecting the efficiency of the diesel pile driving hammer,

2. The size of drive casing,

3. The type of Becker Hammer drill rig,

4. Published (Farrar et al., 1990) correlation between BPT and SPT N_{60} values,

5. Overburden pressure, C_n, and

6. Effect of sloping ground on C_n.

Reducing the large amounts of data as per recommended and adopted procedures (Seed et al., 1984; Seed, 1987; Harder and Seed, 1986) can become long and laborious. In an effort to expedite the process, all but one of the necessary corrections to raw data were mathematically modeled in a spreadsheet. Field personnel familiar with the system were able to provide printouts of the spreadsheets with calculated $(N1)_{60}$s, all intermediate values within the spreadsheet and a printed plot of $(N1)_{60}$s vs. depth for any BPT, in the time it took the drill crew to remove drive casing and relocate to another drilling location, about 20 to 30 minutes in most cases. In this way, the large quantity of data produced by the BPT method was managed and made available for review during construction.

PRELIMINARY EVALUATION OF PENETRATION RESISTANCE TESTING

As depicted on figure 8, comparison of average precompaction and postcompaction BPT data indicated a high level of compaction effectiveness was achieved in the upper 30 to 40 ft of the treatment zone. Below about 40 ft a reduced increase in penetration resistance was shown to exist, although marked increases in average baseline precompaction blowcount values were still present. The lower 20 ft of the remediation zone was the area likely to exhibit $(N1)_{60}$ values less than the established criteria for a factor of safety against the occurrence of liquefaction (FSL) of 1.0. To provide an FSL > 1.0, $(N1)_{60}$ values were required to be above about 23 at the compaction working surface and transitioned, generally linear, to a value of about 17 at the full depth of about 60 ft. Further evaluation of this data indicated that, generally, the middle one-third of the 150-ft-wide treatment zone showed improvement to levels above an FSL = 1.0 in terms of penetration resistance to full depth. Areas within the treatment zone which did not achieve desired levels of remediation were along the undredged/dredged alluvium contacts, or "abutments," and at depth along the upstream and downstream edges of the treatment zone.

To develop the conclusions briefly described above, a unique method of data reduction and analysis was performed to visually depict the effectiveness of treatment at various depths within the remediation zone. BPT values were averaged on 10-ft intervals in the upper 30 to 35 ft of the remediation zone where field-developed plots of penetration resistance with depth showed high levels of compaction. Beneath the highly-compacted area, BPT values were averaged on 5-ft intervals to an elevation represented by undredged alluvium or rock based on resistance value increases. Both preconstruction and postcompaction penetration values were reduced in this manner. These average values were then entered into Reclamation's 3-dimensional computer-aided drafting and design (CADD) system to generate "contours" of penetration resistance, as typically depicted by the plan and sections of figure 9. The combination of the BPT providing large amounts of data throughout the remediation zone and state-of-the-art computer applications in reducing the data to manageable and visually descriptive levels, provided an extremely effective means for determining the nature and suitability of the treatment.

Preliminary evaluations of compaction effectiveness, as completed throughout the remediation zone by the analysis method depicted on figure 9, were quite encouraging. Since some areas showed only marginal improvement, additional BPTs and SPTs were performed in March 1991 to evaluate aging effects. Aging effects have been observed in several ground improvement programs (Von Thun, 1988; Mitchell, 1984).

If quantifiable, the potential added resistance from aging may have resulted in reduced future remediation requirements. As shown on figure 8, significant blowcount increases above precompaction and postcompaction values were obtained through the working platform berm. Increases of this magnitude were considered unlikely due to aging alone. As a result, the effect of friction on the BPT values obtained to date was in question.

The side-by-side SPTs completed to match the March 1991 BPT work were performed to increase the data base believed necessary at this time in finalizing BPT to SPT $(N1)_{60}$ sand-equivalent blowcount reduction procedures. "Side-by-side" data from SPTs were obtained within a 5- to 15-ft distance from the BPT locations. Due to the highly variable and generally coarse nature of the dredged deposit, it was anticipated that significant scatter of data would result and a fairly large database may be required, from a statistical standpoint, to finalize data reduction procedures. Plots of BPT versus SPT $(N1)_{60}$ values obtained prior to the March 1991 work, as shown on figures 10 and 11, show anticipated variances when comparing SPT results with BPTs calculated by established procedures (Harder and Seed, 1986). However, the database collected and evaluated at this time could not definitively depict potential friction effects in comparing precompaction to postcompaction penetration resistance values. Therefore, cased and uncased BPTs were performed through the berm in September 1991 in an attempt to quantify the effects of friction on precompaction, postcompaction, and postberm BPT results. Additional side-by-side SPTs and related materials sampling and testing were being completed at these locations.

Preliminary results of cased and uncased BPT investigations are summarized on figures 12 through 16. Potential areas of interest for discussion are summarized as follows:

1. First and perhaps most important from an evaluation of treatment adequacy standpoint, is the difference in postcompaction BPT $(N1)_{60}$ values for the bottom 20 to 30 ft of the treatment zone to that obtained from BPTs performed in a drill hole cased through the highly treated upper portion of the compaction zone, as depicted on Figure 12. These results show a significant frictional effect may need to be accounted for, and possibly at varied magnitudes with depth, in reducing postcompaction BPT data. However, it is considered by Harder (1991) that friction is an inherent aspect of the BPT test and related adjustments to obtain equivalent SPT N-values. This would suggest that BPT values obtained in cased holes would underestimate the actual available SPT penetration resistance. This theory generally proves out when comparing the BPT and SPT data depicted on figures 13 and 14, as collected at two of the four cased/uncased investigation locations, but is in question when comparing the BPT and SPT data of figure 15. A possible explanation for the SPT penetration resistance values being so much lower than the cased BPT values at the latter location is that materials present here contained a high fines content with little or no gravel. This would suggest that BPT to SPT correlations, as estimated under existing procedures, may not be as valid in materials having a high fines content and a small percentage of coarser fraction materials.

2. Comparison of uncased BPT data collected through the berm in March 1991 to the uncased BPT data collected through the berm in September 1991, as depicted on figure 16, shows a definite decrease in penetration resistance for the September work. Possible issues for consideration in discussions on these results are summarized as follows:

   a. Redistribution of stresses within the highly compacted berm materials after reservoir levels had inundated the berm during the summer of 1991;

b. Reduced friction levels within the berm and foundation materials which had not seen water during the March investigations and were inundated by the reservoir prior to the September work;

c. Reduced friction as provided by water from reservoir levels near elevation 417 ft in September to those ranging from 384 to 404 during the March testing.

3. Comparison of the precompaction BPT values obtained from work along the downstream side of the embankment and those obtained along the upstream side of the embankment just prior to treatment may suggest that friction is also a factor in estimating baseline penetration resistance at this site. This determination is considered critical in estimating the residual shear strength(s) of these materials as required for stability studies that will determine if additional remediation is required. Preconstruction BPT work performed downstream of the embankment suggested an average BPT $(N1)_{60}$ value of about 7 in the free field downstream of the embankment. An overall average $(N1)_{60}$ of around 12 was obtained for all precompaction testing performed along the upstream side for the embankment. The upstream average was calculated from values below approximately elevation 365 ft within the treatment zone, or 10 ft below the compaction working surface elevation of 375 ft. Differences in downstream to upstream preconstruction averages could in part be attributed to any one and/or all of the following:

a. The larger database collected along the upstream side is more representative than that collected prior to construction along the downstream side.

b. The groundwater elevation on the downstream side of the embankment fluctuated between approximately elevation 365 to 370 during preconstruction investigations and was at or below elevation 355 during the precompaction work. As a result, less friction may have been encountered in the downstream work than precompaction investigations.

c. Different drill rig types used during testing may have differing combustion efficiencies than have been previously documented.

d. A 5-ft compaction pad was constructed with coarse zone 1 embankment materials, compacted by equipment travel, from elevations 370 to 375 ft prior to performing preconstruction BPTs. This area may have increased frictional effects above that encountered in the upper 10 ft of the investigations performed along the downstream side of the embankment.

e. Some of the precompaction data were collected in the area that had been loaded by the upstream slope of the embankment prior to removal for foundation compaction. Higher average blowcounts may be partially the result of embankment loading from tests performed in this area.

f. The possibility, although considered small, that looser materials actually exist along the downstream side of the structure. This is conceivable if based on an assumption that a different dredging process was implemented in the two areas.

The above results suggest that further research and related investigations may be necessary to provide the desired level of confidence in reducing BPT data for use in empirically estimating liquefaction potential using this procedure.

Conclusions Relative to In situ Testing at Jackson Lake Dam

In situ testing played a major role in the investigation and control of ground improvements at Jackson Lake Dam. Due to the complex stratigraphy at the site, the use of multiple simple penetration tests was employed to delineate and evaluate potentially liquefiable soils. SPT and shear wave velocity testing were the primary methods for evaluating ground improvements by dynamic compaction. Cone penetration testing was useful for determinations of stratigraphy but was hampered by gravels in the foundation.

Evaluation of SPT data to estimate dynamic compaction effectiveness can be made by direct comparison during construction and by more detailed statistical analyses following construction. Both approaches in data interpretation were successful at this site. Considerable difficulty was encountered in evaluating data due to complex ground stratigraphy and the occurrence of gravels.

Energy transmission and drilling methods are important considerations in evaluating SPT data. The use of an automatic hammer during Stage II construction simplified evaluation procedures.

Reclamation applied major effort to measure ground stresses before and after ground improvement. Stress captors can be a useful technique for determining stresses from column treatment methods such as compaction piles and vibroflotation. Efforts to measure in situ ground stress changes from dynamic compaction through insertion instruments such as steeped blade, flat dilatometer, and pressuremeters were complicated by highly variable ground stratigraphy, groundwater and drainage conditions.

Efforts were made to improve prediction of in situ void ratio and state by performing high quality undisturbed sampling for correlation to penetration tests and by using electrical methods. Reduction of information collected for the compaction pile test section illustrates how data could be used for site-specific correlations. Electrical methods showed promise for the dielectric probe to be used for porosity prediction.

In situ tests consisting of pressuremeter, borehole shear and seismic tomography were used for SMW quality evaluations. The borehole shear device can be extremely useful to evaluate column shear strength in areas of poor core recovery. Tomography was very helpful in providing qualitative two dimensional pictures of cutoff wall continuity.

Conclusions from In situ Testing at Mormon Island Auxiliary Dam

Becker penetration testing (BPT) values obtained just following dynamic compaction (postcompaction), as well as baseline (precompaction) BPT values, may require an adjustment of the combined effect of the frictional resistance of the materials through which the Becker Hammer drill steel has just advanced, water levels, site-specific material properties and stress-related conditions. The magnitude of this adjustment may be dependent on what is already considered an inherent part of present data reduction procedures.

Preliminary evaluations of compaction effectiveness consisting of "contouring" average BPT values on 5- to 10-ft increments is an extremely effective means for determining premodification conditions and the nature and suitability of applied treatment. The highly efficient BPT method economically provided penetration resistance information throughout the entire zone requiring and receiving remediation such that analyses of this type could be completed with a high degree of confidence.

Currently the Standard Penetration Test (SPT) remains the key control measure to which other procedures must be correlated when making critical decisions of ground stability under earthquake loading.

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Table 1. - Average change in SPT blowcount by sector and material type - matched intervals - Stage I construction (Von Thun, 1988).

Sector	Material classification				All data
	GP and SP	SM	ML	SM-ML	
B	12.5(10)	7.2(5)	4.4(18)		7.7(33)
C	12.3(15)	9.4(5)	7.4(11)	11.8(4)	10.3(35)
D	10.7(10)	6.2(5)	5.3(9)		7.7(24)
E	15.6(8)	11.3(21)	5.5(17)		9.9(46)
F	11.1(10)	14.5(6)	7.3(10)	7.2(2)	10.2(28)
G	14.0(8)	10.0(3)	7.0(2)		12.0(13)
H	19.0(19)	10.5(8)	6.3(3)		15.5(30)
Summary	14.0(80)	10.4(53)	5.8(70)	10.4(6)	10.3(209)

(**) = Number of sample intervals

Table 2. - Average change in SPT blowcount by sector and material type - matched intervals - Stage II construction (Von Thun, 1988).

Sector	Material Classification			Overall Average Change*
	GP and SP	SM (SM-ML, SP-SM)	ML(ML-CL)	
K	58(8)	20(26)	11.0(9)	22.0
L	51(9)	27(26)	28.0(13)	24.5
M	49(9)	27(24)	16.0(11)	26.0
N	67(7)	19(19)	9.0(7)	20.5
P	43(5)	18(25)	10.5(11)	10.0
Q	Insufficient data for comparison (6 matched samples)			
Summary	53.9(38)	22.3(120)	16.0(51)	20.6

(**) = Number of sampled intervals.

* Note: Approximately 170 intervals per sector.

Table 3. - Summary of improvement in raw N value for predominant soil types (Farrar and Yasuda, 1991).

USCS soil type	Stage II precompaction (B2)			Stage II postcompaction (A2)			Percent increase
	# PTS	N-AVG.	SD	# PTS	N-AVG.	SD	(A2-B2)/B2
CL	22	4	3	27	9	4	132
ML	174	7	5	93	13	6	97
GM	22	17	14	26	34	22	101
SM-ML	3	12	7	2	15	6	25
SM	328	16	8	260	25	11	60
NR	101	18	11	109	48	36	167
SW-SM	16	23	8	13	31	10	335
SP-SM	159	21	9	139	33	9	60
SP	17	21	9	5	33	13	58
GW-GM	12	23	14	14	49	19	116
GP-GM	17	26	11	19	48	20	83
GW	5	34	17	2	60	12	73
GP	189	22	11	214	45	20	101
Totals	1065			923			85

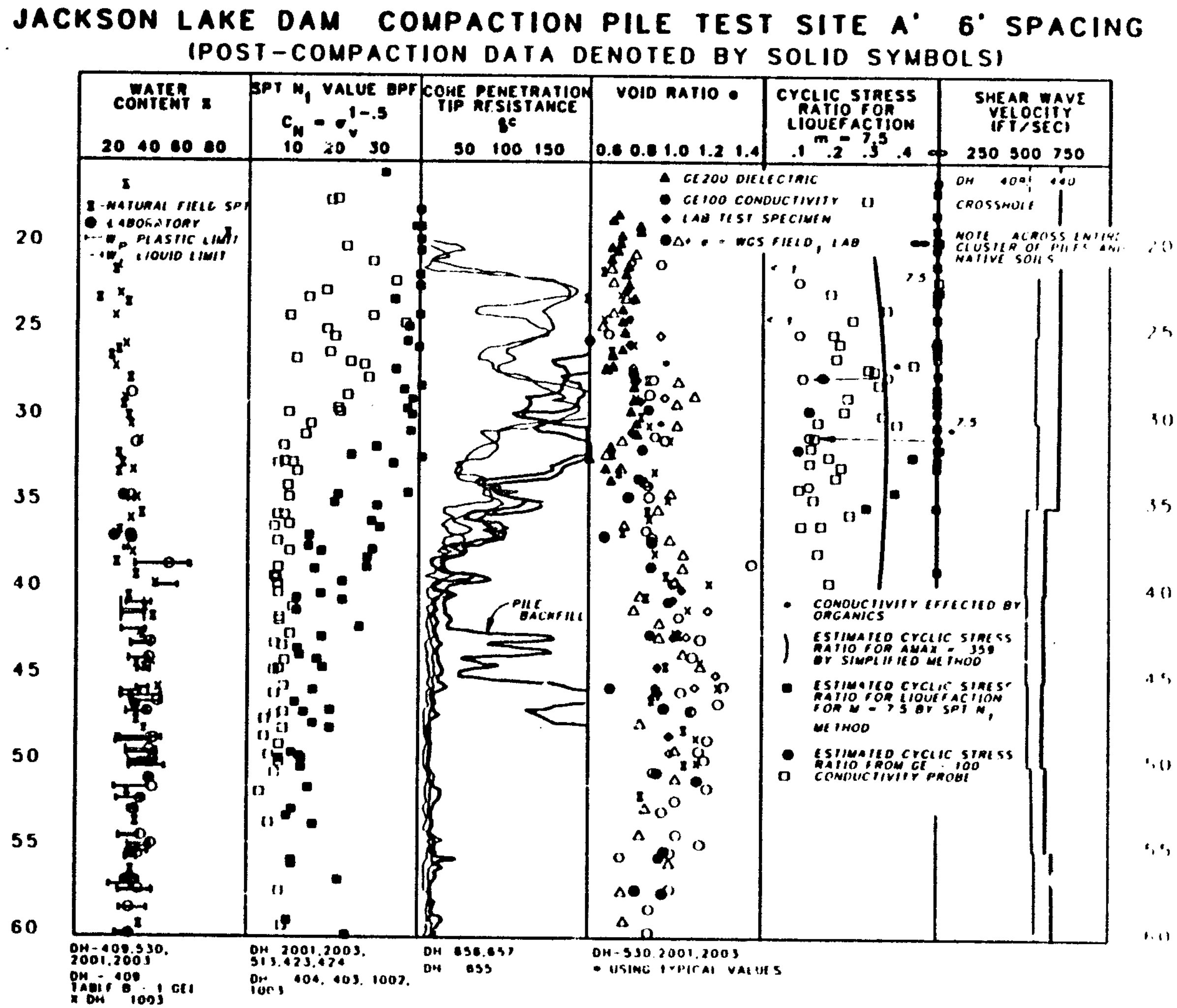

Figure 1. - Summary of in situ testing at compaction pile test section A (Farrar et al., 1988).

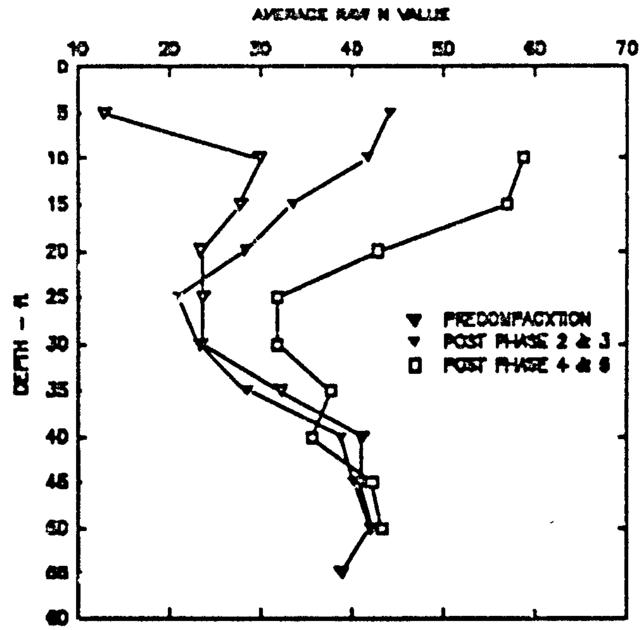

Figure 2. - Average raw N value of all data versus depth - Stage I (Farrar and Yasuda, 1991).

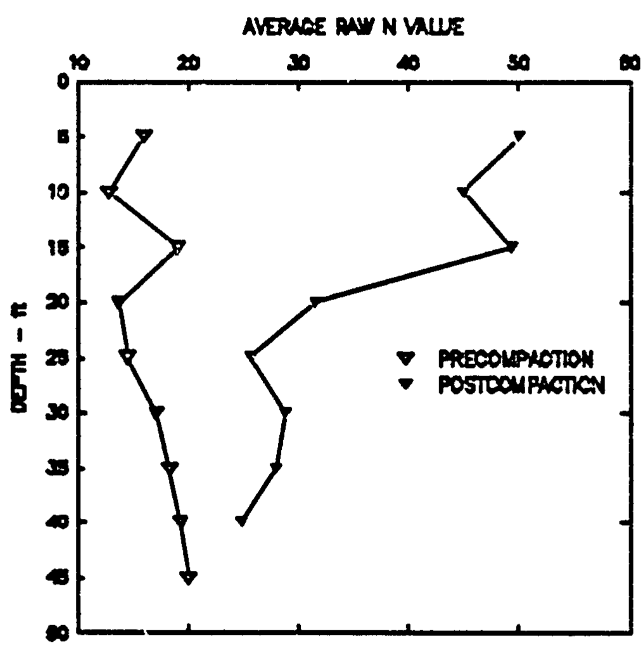

Figure 3. - Average raw N value of all data versus depth - Stage II (Farrar and Yasuda, 1991).

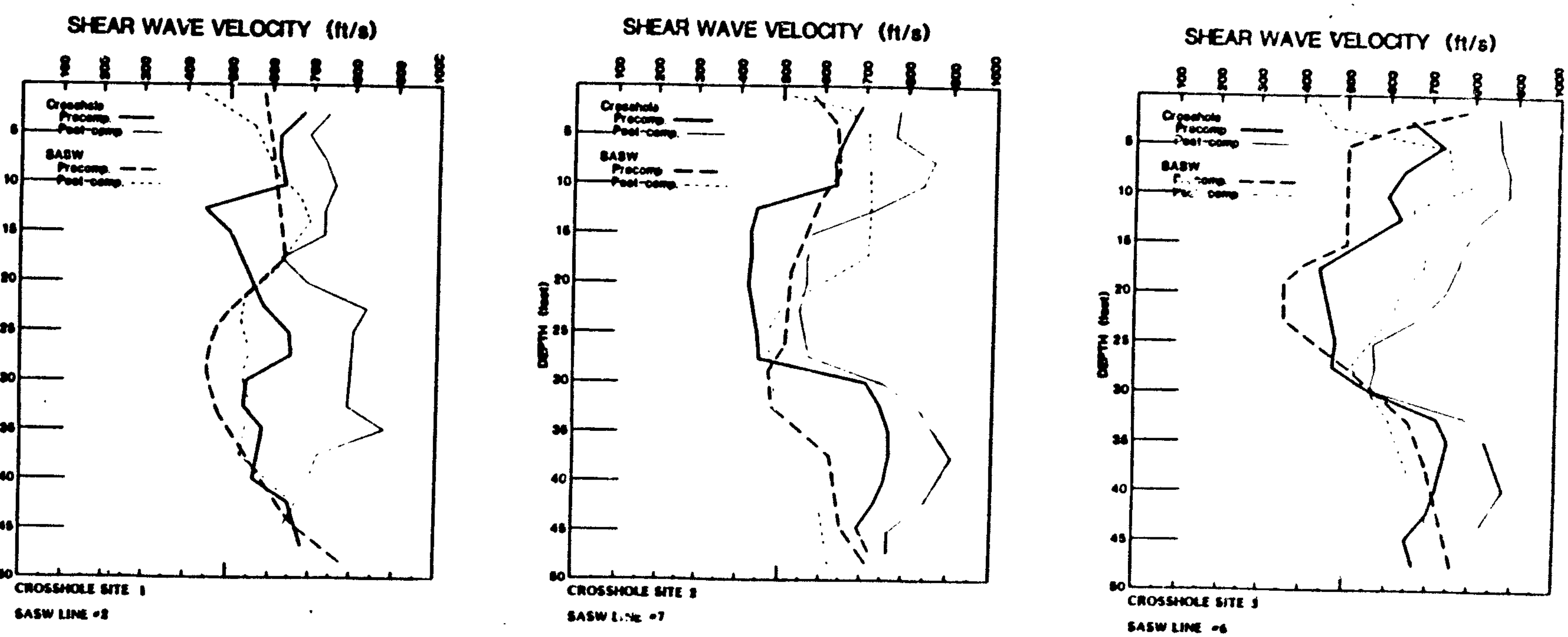

Figure 4. - Summary of increase in shear wave velocity by SASW and crosshole methods (Sirles, 1988).

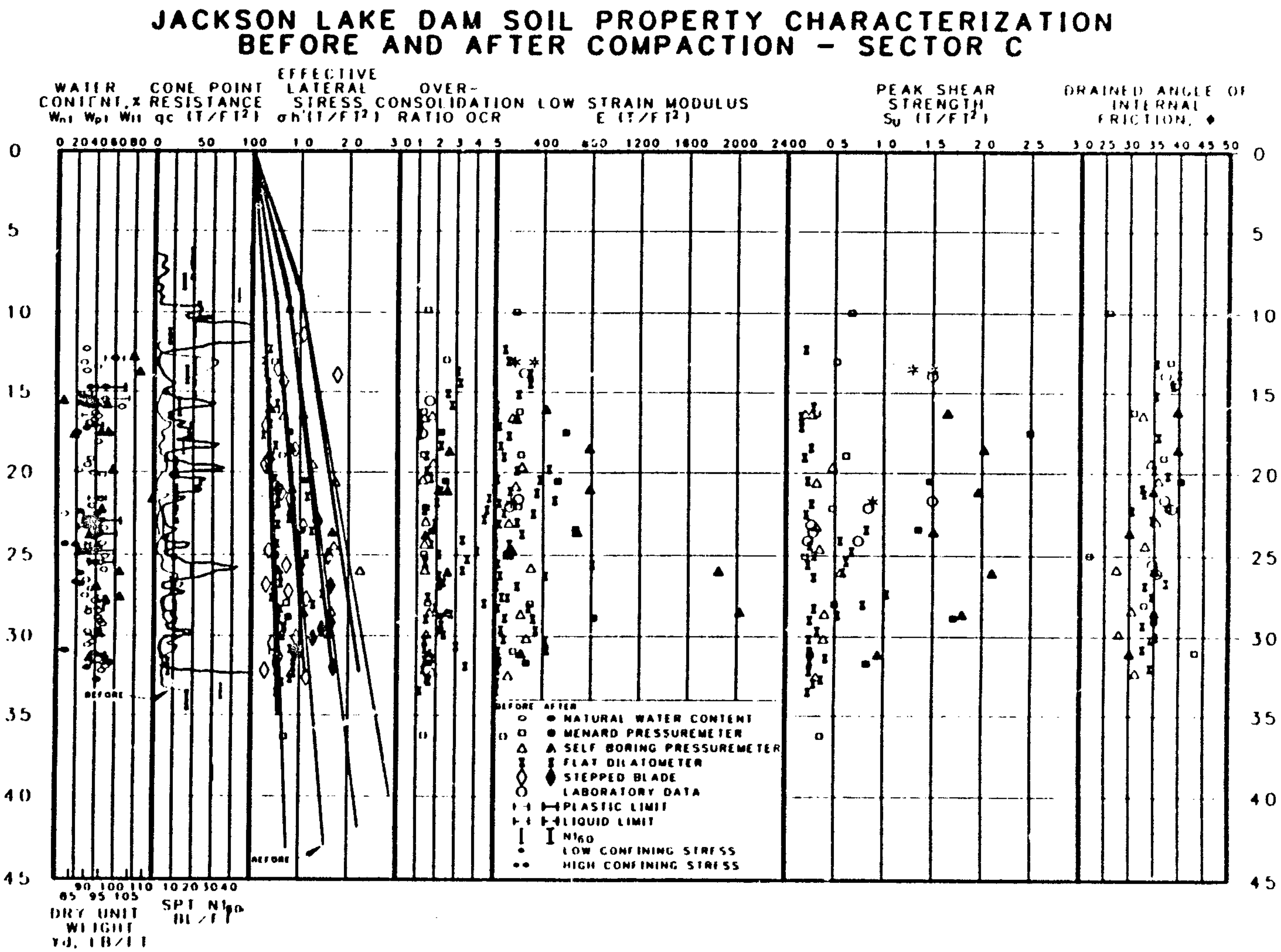

Figure 5. - Summary of stress change measurements from dynamic compaction sector C (Woodward Clyde Consultants, 1987).

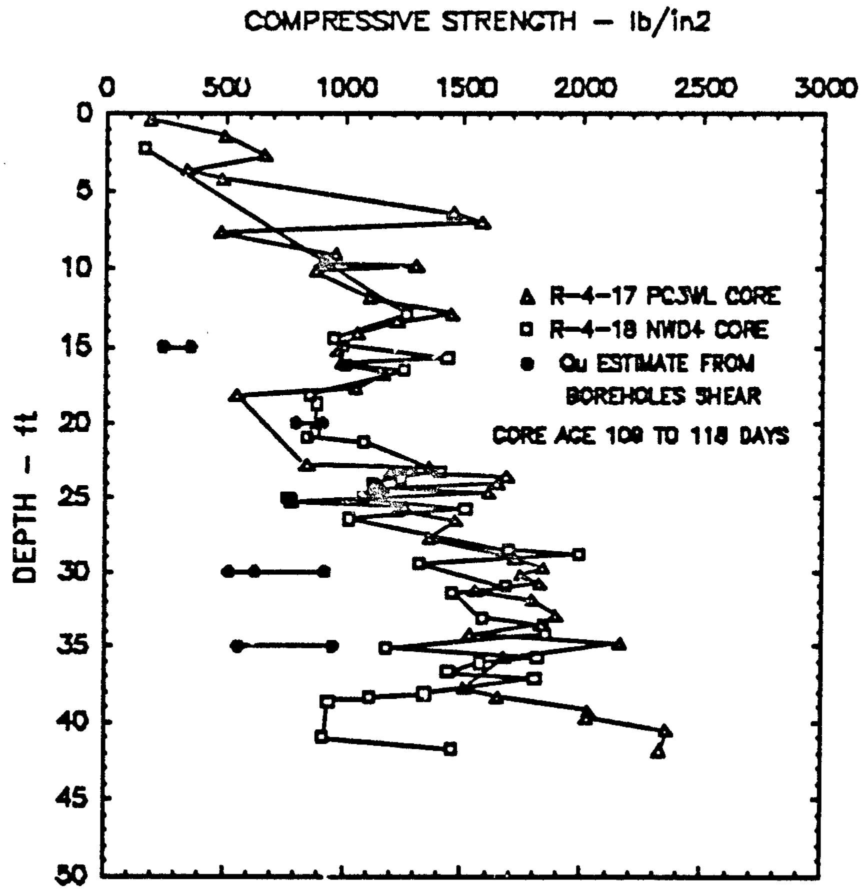

Figure 6. - Comparison of borehole shear and core compressive strength testing in SMW columns.

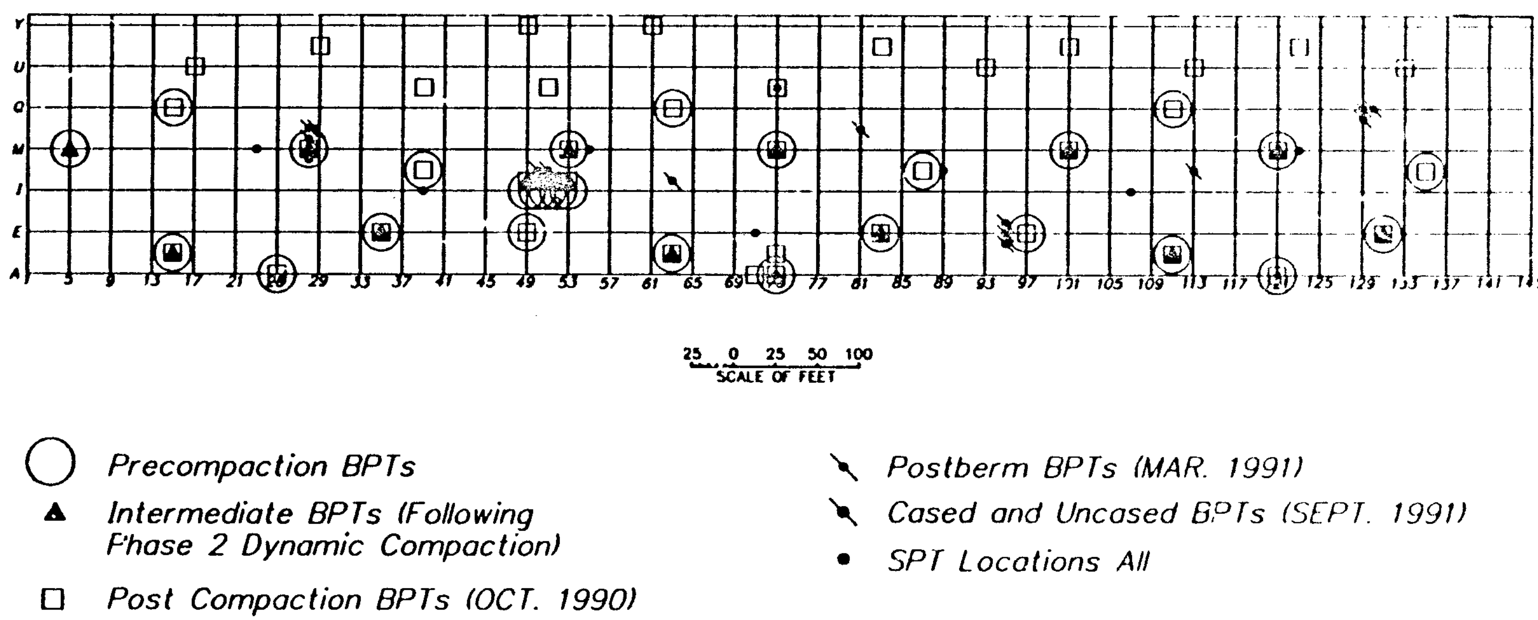

Figure 7. - Plan view of remediation zone with BPT and SPT locations on dynamic compaction grid.

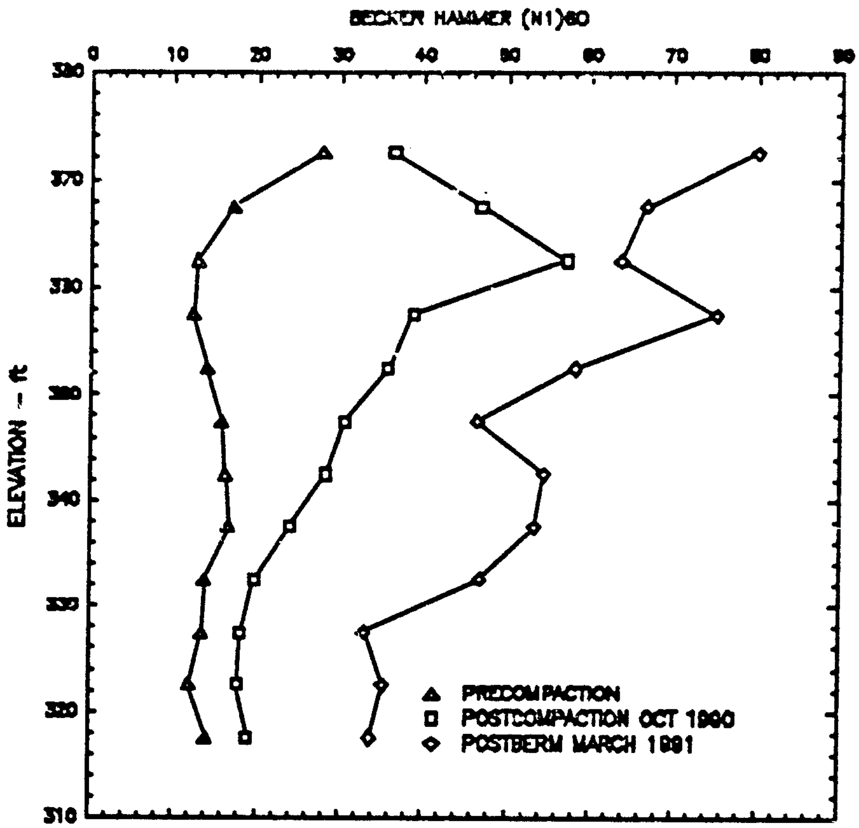

Figure 8. - Average BPT (N1)60S versus depth - precompaction, postcompaction, and postcompaction through berm.

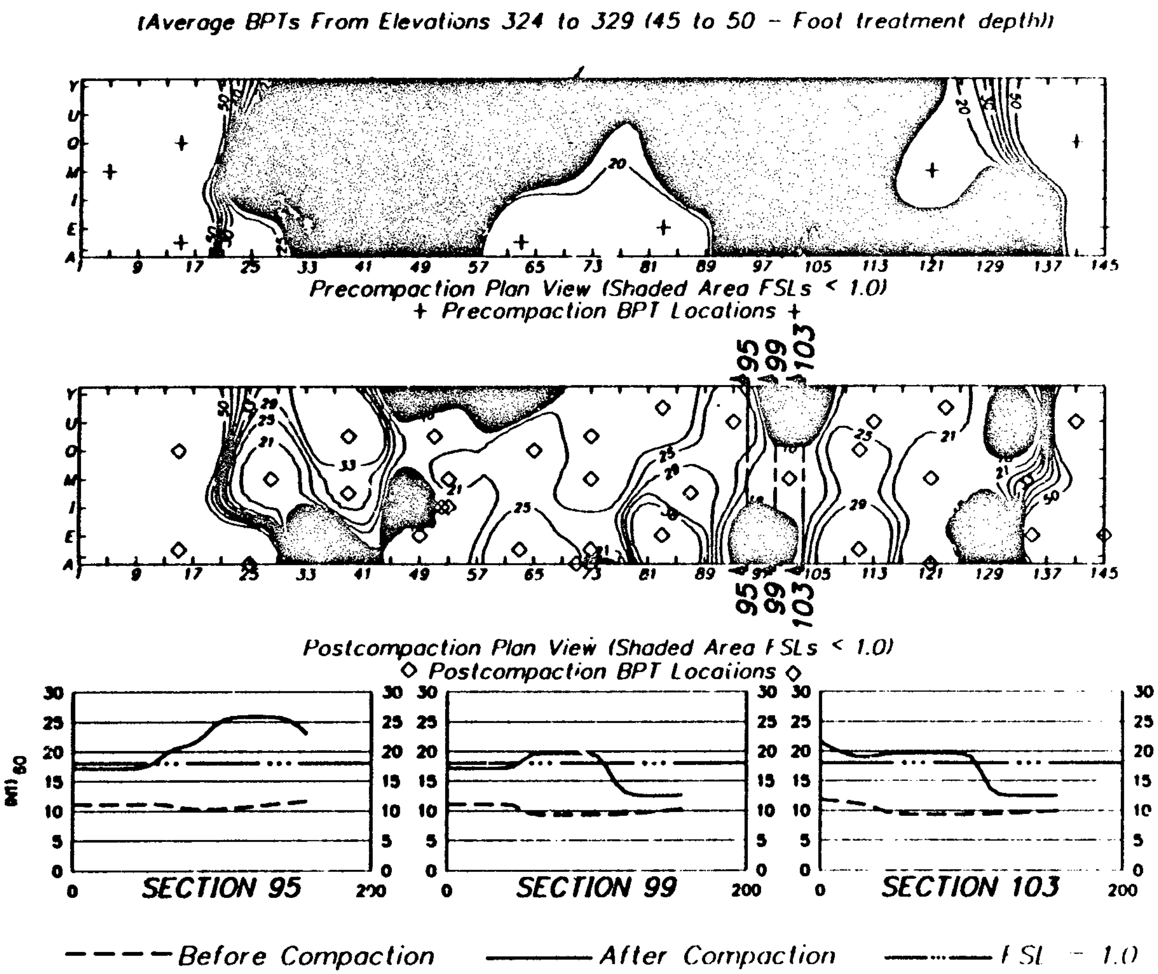

Figure 9. - Typical "contouring" analysis plan view.

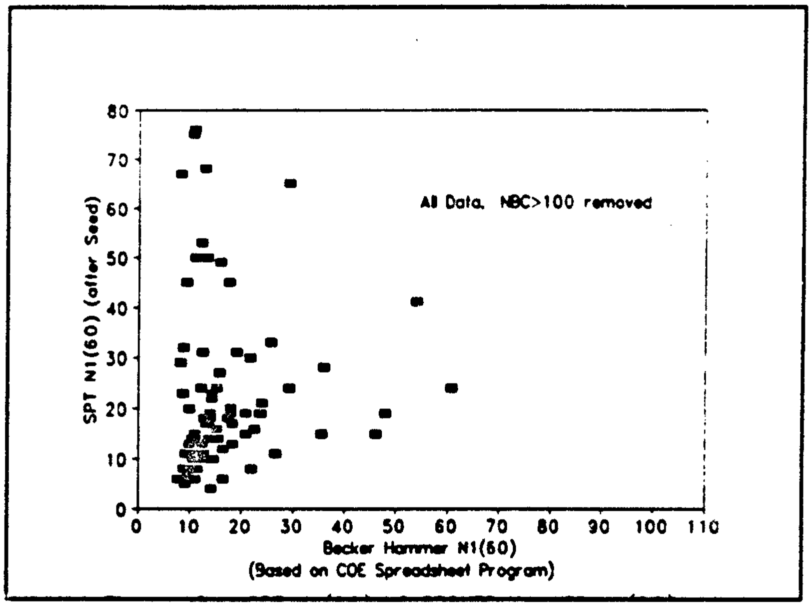

Figure 10. - SPT N1(60) versus Becker Hammer N1(60) - all data - before April 1991.

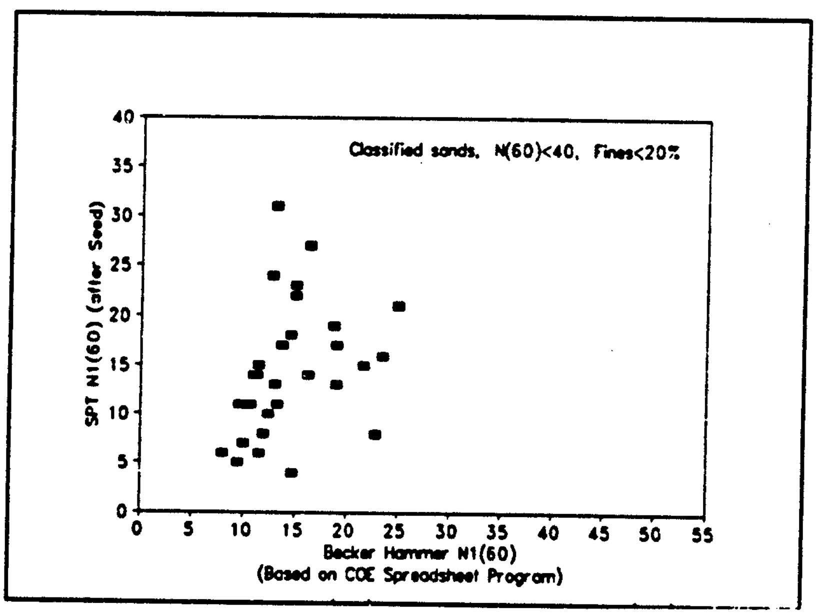

Figure 11. - SPT N1(60) versus Becker Hammer N1(60) -

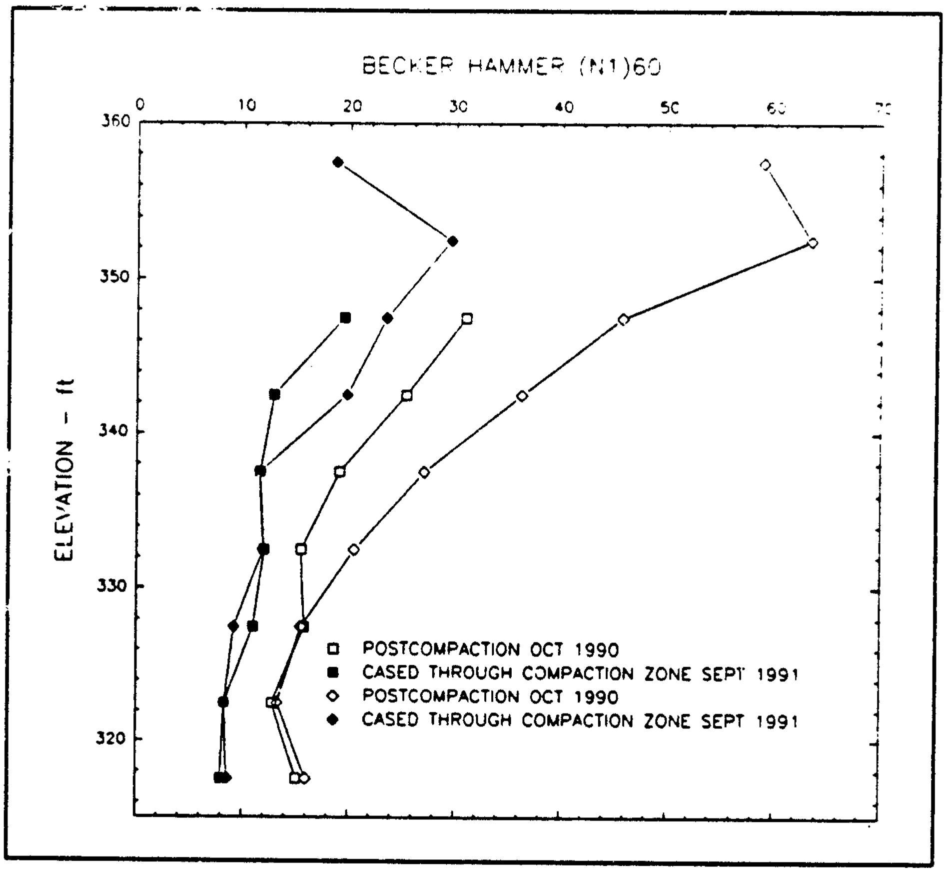

Figure 12. - Average BPT (N1)60s versus depth - direct comparison of postcompaction drill holes - cased through compaction zone.

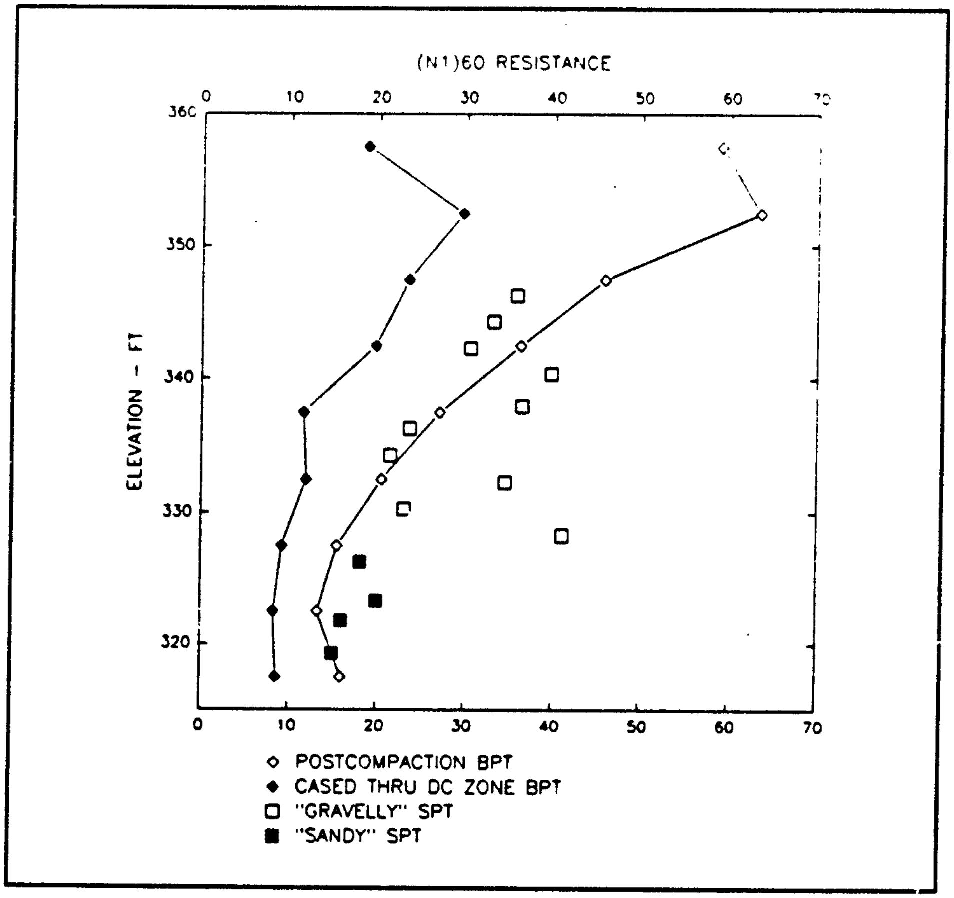

Figure 13. - Comparison of postcompaction BPT and cased BPT with gravelly and sandy SPT.

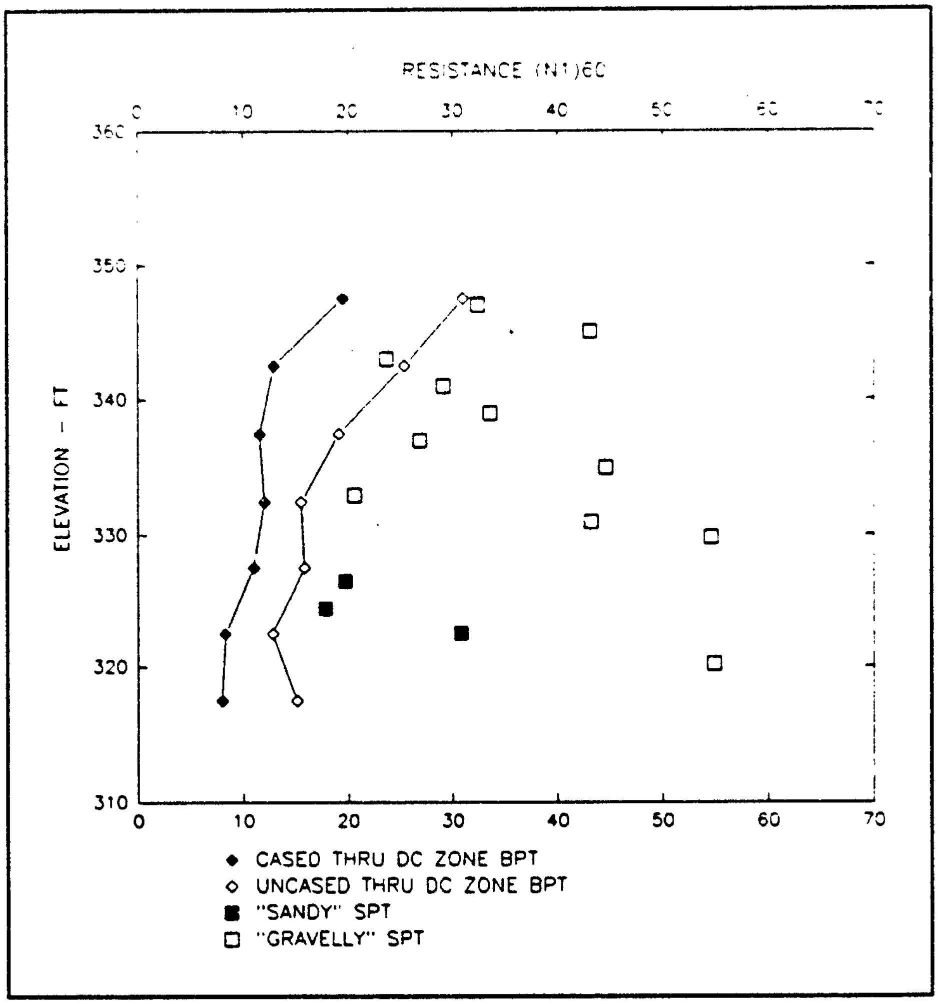

Figure 14. - Comparison of postcompaction BPT and cased BPT with gravelly and sandy SPT.

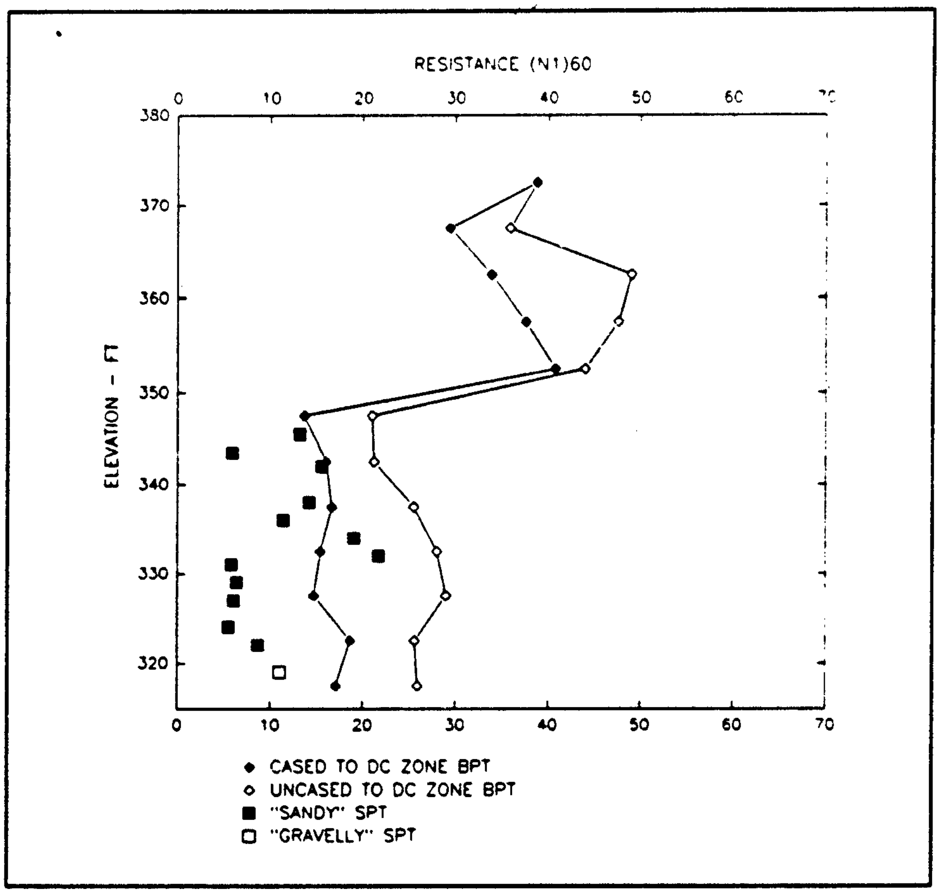

Figure 15. - Comparison of cased and uncased BPT with SPT in area with high fines content.

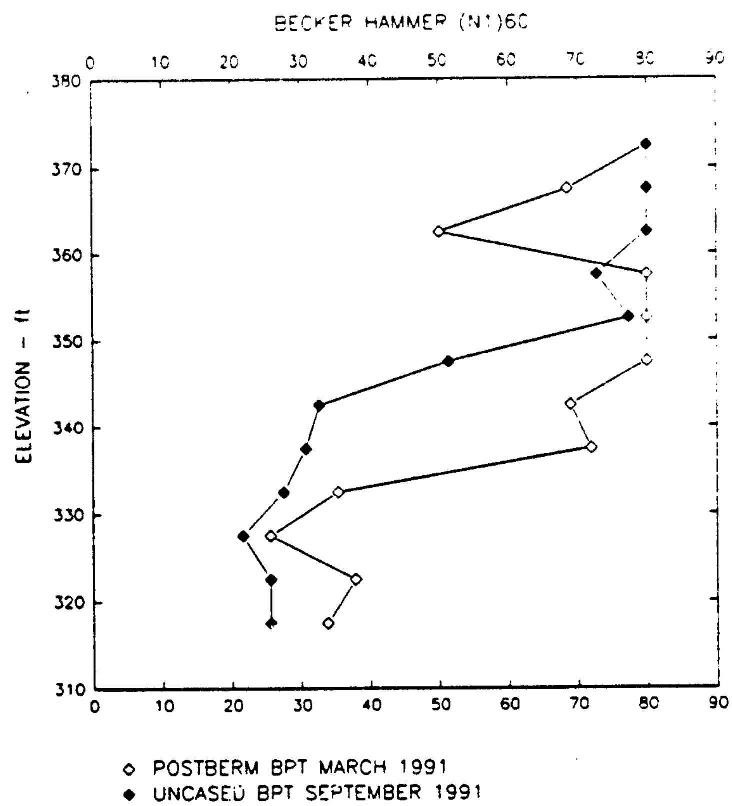

Figure 16. - Comparison of uncased BPTs before and after reservoir inundation.

CURRENT TECHNOLOGIES IN GROUND TREATMENT AND IN SITU REINFORCEMENT

By Dr. Donald A. Bruce, Member, ASCE[1]

Abstract: Ground treatment by grouting has been conducted in the United States since the nineteenth century, although most of the techniques used for soils have been imported much more recently. The principles of in situ ground reinforcement featuring the use of small diameter soil nails or pinpiles have only been exploited for the last 25 years of so, but already provide a potent tool for the foundation engineer. This paper summarizes the state of practice in both ground treatment and reinforcement at a time when significant developments are occurring in each category, aided by the evolution of innovative contracting and procurement processes.

INTRODUCTION

According to Weaver (1991), the first application of rock grouting for consolidation was at New Croton Dam, New York, in 1893, while the first hydraulic cutoff was executed at Hinkston Run Dam, Pennsylvania, in 1901. By the mid-1930s, major works were being carried out [e.g. at Hocver (Boulder) Dam] under specifications and practices which "quickly became the unofficial grouting standards" (Karol, 1990) and have in part persisted to the present day.

Soil grouting by permeation with chemicals only truly emerged in the 1950s, by which time compaction grouting had been conceived by Warner and co-workers (Warner, 1982). Jet grouting was imported in the early 1980s followed by mechanical mix-in-place methods such as SMW Seiko. Most recently hydrofracture grouting has been heavily promoted, especially on the West Coast.

Despite this long history of usage, and the impressive scale and complexity of many of the works, grouting in some quarters still has a less than flattering reputation, totally at odds with its standing in other countries as a valued and respectable engineering tool. Frequently one meets owners who feel duped by grouting contractors, whom, of course they have elected to pay by the bag mixed and not the end result achieved. One hears contractors who have lost heavily on certain projects as a result of the rigid application of obsolescent specifications by hamstrung inspectors. One reads of projects where "we tried grouting - it didn't work," after the engineer had turned to it when all else had failed and the situation had totally deteriorated, both technically and contractually. In short, one can summarize this poor reputation as arriving from bad conception, poor execution and inappropriate contracting and procurement practices.

Construction activities in the United States are now becoming much more amenable to the benefits of grouting, as the market expands towards urban, industrial, and infrastructure development and redevelopment. Many of these activities have to be conducted in areas of difficult soil and hydrogeological conditions, restricted access, and severe performance criteria. There is therefore a rapidly growing demand for innovative techniques and methods, often offered by specialty contractors, backed by European or Japanese resources. These new approaches, aided by more appropriate contracting and procurement practices, are helping elevate the status of grouting, so that it is being more widely perceived as a reliable engineering tool from the onset, rather than a last resort when all else has been tried and failed. This paper provides an overview of current practice in each of the major grouting methodologies.

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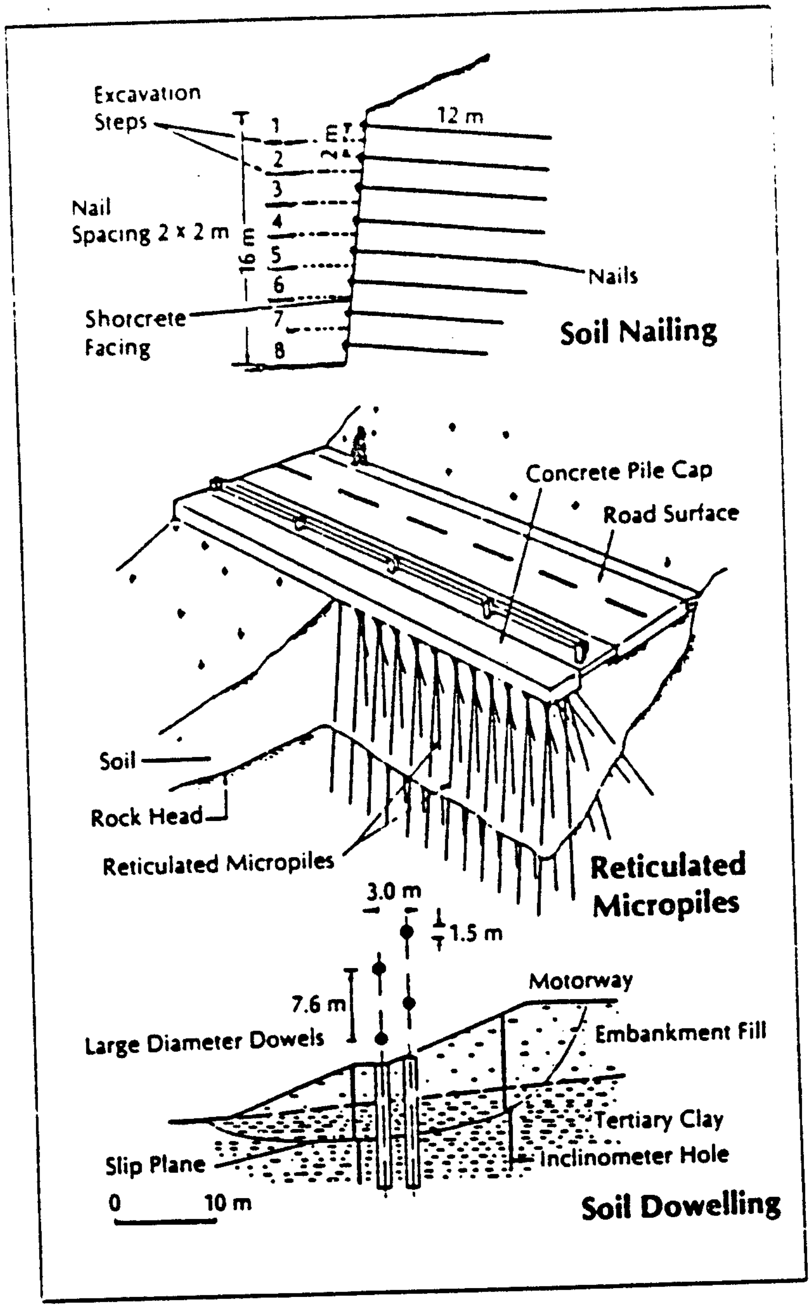


Figure 1. - The family of in situ soil reinforcement techniques (Bruce and Jewell, 1986).

Another aspect of ground engineering technology is the principle of in situ earth reinforcement. As identified by Bruce and Jewell (1986) three basic classes can be recognized (Figure 1):

(1) Soil nailing refers to reinforcing elements installed horizontally or subhorizontally into the cut face, as top down staged excavation proceeds. The inserts improve the shearing resistance of the soil by being forced to act in tension.

(2) Reticulated pinpiles are similar inserts, but steeply inclined in the soil at various angles, both perpendicular and parallel to the wall face. The overall aim is to provide a stable block of reinforced soil to act as a gravity retaining structure, holding back the soil behind.

(3) Soil dowelling is applied to reduce or halt downslope movements on well defined shear surfaces. The principle exploits the large lateral surface bearing area and high bending stiffness of the dowels that are of far larger diameter than nails or pinpiles. The use of soil dowelling is rare in urban environments, although it can prove attractive when combined with linked deep drainage in arresting massive land movements (e.g., in eastern Italy and southern California). (Bianco and Bruce, 1991).

Soil nailing is primarily used in excavation or cut slope support and has become very popular, well documented and well researched (ASCE, 1987) in the United States. Dowelling is equally as well known, although has a less frequent application. Reticulated pinpiles, however, are a relatively recent development. These bored, cast-in-place elements are from 4 to 8 inches in diameter. The composite soil-pile structure then constitutes an in situ barrier to arrest actual or potential slope movements.

When this structure is considered in cross section, it somewhat resembles a letter 'A', and so is popularly referred to as Type A Wall. Important advances are being made (Bruce, 1992b; Pearlman et al., 1992) towards creating more rational design methodologies. Given the considerable potential of Type A Walls, they are therefore the subject of the latter part of this paper.

GROUND TREATMENT

Rock Grouting

As inferred above, rock grouting practice largely follows traditional lines although within the last few years it would seem that publications by such as Houlsby (1990) and Weaver (1991) have had a refreshing and innovative impact. Their moves towards change, coupled with a wider appreciation of overseas developments have been aided by the international flavor of many of the annual short courses (e.g., at University of Missouri - Rolla, and University of Wisconsin - Milwaukee), the active contributions of foreign specialists in domestic industry, and the experiences shared with United States grouting consultants in foreign works (Anthony et al., 1992). In addition, the technical demands of grouting new sites of difficult geology (Aberle et al., 1990) and the increasing amount of remedial grouting at existing sites (Bruce, 1990) has forced challenges to old paradigms. In general the following broad statements can be made to reflect typical current practices.

- Drilling is still largely conducted by rotary methods, although the insistence on diamond drilling (including full coring) is no longer so prevalent. Top drive rotary percussion is growing in acceptance in certain quarters - with the increasing availability of diesel hydraulic crawler rigs, as long as water flush is used. Somewhat surprisingly, certain consultants are beginning to allow air flushed down-the-hole hammers to be used for routine grout hole drilling. Even when the air is "misted" with some inducted water, most specialists believe that this medium has a detrimental effect on the ability of fissures to subsequently accept grout (Houlsby, 1990; Bruce et al., 1991a).

- Water testing is not so rigorously or intensely conducted as, for example, Houlsby would advocate, and in the vast majority of cases, stage water tests are run at a single, relatively low, excess pressure and results are expressed in units of cm/sec as opposed to Lugeons.

- Grout mixes have traditionally been "thin" by European standards and composed of only cement and water, but, again, change is evident. For example certain Government agencies (USCOE, 1984; USBR, 1984, 1987) have been systematically experimenting with fluidifiers and plasticizers, while work continues with pozzolans and silica fume and other modifiers. The systematic use of stable, bentonitic grouts, in accordance with the current European theories (Deere and Lombardi, 1985) is not yet widespread.

- Grouting equipment has changed little, except that tighter controls are being exercised at batching stations over mix proportioning. Grouting pressures remain conservative by foreign standards - although often exceeding the old "one psi per foot" rule - and "constant pressure" progressive cavity pumps such as Moynos are specified over "fluctuating pressure" piston or ram pumps. Grout consumptions still tend to be recorded in "sacks per foot." There are two areas especially where major change is evident, and where rock grouting practice has undergone rapid changes: parameter recording and staging philosophies.

- Parameter recording by electronic means has become standard practice on all federal jobs and on most others also. This may range from a simple "in the field" chart recorder, to the telemetric system, devised by the Bureau of Reclamation at their massive New Waddell Dam project in Arizona (Aberle et al., 1990). There, electronic pressure transducers, magnetic flowmeters and density meters in the field constantly relay data via a Remote Telemetry Unit to a Central Telemetry Unit (CTU), where all the grouting parameters are displayed in real time. Graphical data consist of flow rate, pressure, bag rate, and water-cement ratio. Numerical data include hole and stage number, target pressure, volume, density, w/c ratio, take rate, depth, cumulative take, date and time. Numerical data from six stages can be monitored instantaneously. The field inspector is in constant communication via radio with the CTU office to exchange information and instructions. Data are stored for future technical analyses and reports, and also for payment purposes. Aberle et al. concluded that these systems are extremely valuable and greatly help to direct and optimize the grouting. This is to be warmly applauded given their earlier statement that "in Reclamation, drilling and grouting is the most thoroughly inspected construction which is performed on a dam project."

- Regarding staging practices, the competent rock available and selected for past sites was ideally suited to ascending stage operations, and this method has become the traditional standard. Descending stage grouting is becoming more common, reflecting the challenges posed by more difficult site conditions in the remedial and hazardous waste markets. The work described by Weaver et al. (1992) relating to the sealing of dolomites under an old industrial site at Niagara Falls, New York, represents a statement of the best of American practice.

In some cases of extremely weathered and/or collapsing bedrock, even descending stage methods can prove impractical, and two recent projects illustrate innovative trends. Firstly, at Lake Jocassee Dam, South Carolina, a remedial grouting project was conducted (Bruce et al., 1992) to reduce major seepages through the Left Abutment of the dam. *Given the scope of operating within innovative contracting procedures*, the contractor was able to vary his methods in response to the extremely variable ground conditions actually encountered. Some holes permitted ascending stages, others needed descending stages, while the least stable had to be grouted through the rods during their slow withdrawal.

A second example is the grouting of poorly cemented hard rock backfill 2800 ft below ground level in a copper mine in Northern Ontario, Canada (Bruce and Kord, 1991). This medium proved so difficult to drill that none of the conventional grouting methods could be made to work. Instead, the first North American application of the Multiple Packer Sleeved Pipe (MPSP) system, devised by Rodio, in Italy, was called for. The MPSP System is similar to the sleeved tube (tube à manchette) principle in common use for grouting soils and the softest rocks (Bruce, 1982). The sleeve grout in the conventional system is replaced by concentric polypropylene fabric collars, slipped around sleeve ports at specific points along the pipe (Figure 2). After placing the pipe in the hole, the collars are inflated with cement grout, via a double packer, and so the grout pipe is centered in the hole, and divides the hole into stages. Each stage can then be grouted with whatever material is judged appropriate, through the intermediate sleeved ports. Considerable potential is foreseen in loose, incompetent, or voided rock masses, especially karstic limestones (Bruce and Gallavresi, 1989).

As a final note, there remains considerable activity in bulk infill, principally associated with older, shallower mining operations in the Appalachians and Wyoming. Rotary and rotary percussive drills, often of water well drilling type, are common, with the void filling (either partial or total) being executed with cementitious grouts or concrete prepared in large scale site batching plants. Innovations are restricted to improved automated parametric recording and the development of special foamed grouts intended to extinguish mine fires.

Soil Grouting

Five fundamental categories of soil grouting methodologies are being used in the United States to various extents and the industry is rapidly evolving. Technological advances are being made by chemists, physicists and geotechnical engineers on the one hand, and are being prompted by the increasingly severe demands made by structural engineers, environmentalists and property developers on the other. Such has been the pace of recent developments that soil grouting is fast achieving the status of the "design tool, as it should be from the onset" (Clough, 1981) instead of a final remedial option when "conventional" techniques have failed.

Permeation grouting. - Probably the oldest and most widely used principle, covering a wide range of applications, materials and injection methods (Figure 3). Much of the smaller, simpler work is executed by end of casing injection (or lancing: Bruce, 1989) using cement based grouts. However, largely through the efforts of a limited number of specialty contractors, there has survived an important if sporadic market in sophisticated chemical grouting using the tube à manchette system (Karol, 1990). This has been executed principally in association with new Metro systems, and the major work conducted to prevent run-ins and control settlements during the subsequent excavation of the twin 20-ft-diameter tunnels under the Hollywood Freeway in Los Angeles is a fine example of the state of practice (Gularte et al., 1992). On this project, incidentally, a fire which occurred in the lining of the tunnel during its construction provided a unique (and successful) test of the surrounding treated ground.

Applications for dam grouting have been far less frequent, with the work described by Karol (1990) at Rocky Reach Dam, Washington, in the late 1950s apparently remaining the largest. Smaller applications in remedial works are summarized by Bruce (1990, 1992a).

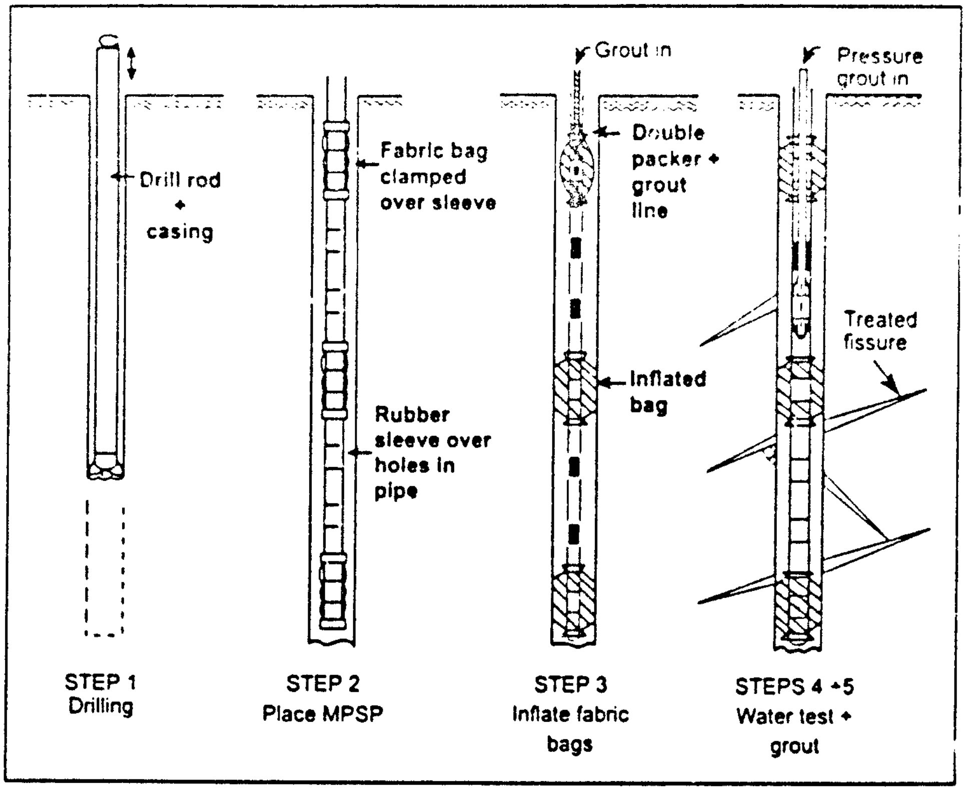


Figure 2. - The MPSP grouting method.

RHEOLOGICAL CLASS	PARTICULATE SUSPENSIONS (BINGHAMIAN FLUIDS)			SOLUTIONS (NEWTONIAN FLUIDS)			GASEOUS EMULSIONS	
	UNSTABLE	STABLE		COLLOIDAL SOLUTIONS (EVOLUTIVE)		PURE SOLUTIONS (NON-EVOLUT)		
MAIN TYPES OF GROUTS	CEMENT ONLY	CEMENT WITH BENTONITE OR CLAY	DEFLOCCU-LATED BENTONITE	CHEMICAL GROUTS			SWELLING GROUTS	
				BASED ON SODIUM SILICATE		BASED ON ORGANIC RESINS	BASED ON CEMENT	BASED ON ORGANIC PRODUCTS
				HIGH STRENGTH	MEDIUM-LOW STRENGTH			
FIELDS OF APPLICABILITY	FISSURED ROCK AND MASONRY			MICRO-FISSURED AND POROUS ROCK			LARGE VOIDS OR CAVITIES	CAVITIES WITH FAST FLOWING WATER
		GRANULAR SOILS						
		PREVAILING GRAVEL	COARSE SANDS	MEDIUM-FINE SANDS		FINE SILTY SANDS (SANDY SILTS)		
COEFFICIENT K OF PERMEABILITY (m/s)		$> 5.10^{-4}$	$> 5.10^{-5}$	$> 5.10^{-5}$	$> 1.10^{-5}$ ①	$> 1.10^{-6}$ ②		
SPECIFIC SURFACE S_s (m^2/N)		< 0.5	< 1.5	< 1.5	< 4	< 10		
BASIC INJECTION PRINCIPLES	HIGH PRESSURE	CONTROLLED QUANTITY AND PRESSURE					LOW PRESSURE (FILLING)	

① LIMIT GOVERNED BY VISCOSITY/TIME EVOLUTION ② NORMAL LIMIT FOR UNIFORM IMPREGNATION

Figure 3. - Classification of grouts related to groutable media (Gallavresi, 1992).

Compaction grouting. - This "uniquely American" process has been used since the early 1950s and is still attracting an increasing range of applications. In summary, very stiff, "low mobility" grouts (Warner, 1992) are injected at high pump pressures (up to 5000 psi) in predetermined patterns to increase the density of soft, loose or disturbed soil. When appropriate materials and grouting parameters are selected, the grout forms regular and controllable volumes, centered on the point of injection. Near surface injections may result in the lifting of the ground surface and associated structures, akin to the principle of slabjacking described by, for example, Bruce and Joyce (1983).

Indeed, the earlier applications were largely for leveling slabs and light buildings on shallow foundations (ASCE, 1977; Warner, 1982). Prior to the pivotal Bolton Hill Tunnel project (Baker et al., 1983) compaction grouting had been used on such subway projects to compensate tunnel induced settlements *after* completion of the tunnel. The philosophy changed fundamentally at that time, however, so that grouting was executed *during* the excavation of the tunnel at locations just above the crown: soil decompressions were therefore prevented from migrating up to cause surface settlements. This principle has been adopted for more recent major tunnelling schemes including those in Phoenix (Lyman et al., 1988) and currently on the Los Angeles Metro.

The popularity of the technique continues to grow, in no little way due to the active preachings of the "founding fathers," such as Warner (1992) and Graf (1992), and the lucid case histories presented openly by contemporary contractors such as Bandimere (Sealy and Bandimere, 1987), Berry (Berry and Grice, 1989), Welsh, and their co-workers. The technique has now been exported to Japan and to Europe and so is the only native American grouting technique to be so recognized.

New important fields of application include the mitigation of liquefaction potential for dams (Salley et al., 1987), the combatting of sinkhole damage in karstic limestone areas (Welsh, 1988), and talus slope stabilization (Weaver, 1989).

Whereas the ASCE grouting conference in 1982 largely provided an overview of the past, the corresponding conference in 1992 provided insights into the future. For example, Schmertmann and Henry (1992) unveiled a new design theory for constructing "compaction grout mats" in karstic conditions. Warner and colleagues (1992) presented accounts of fundamental field and laboratory research into the basics of compaction grout, and the conclusions are regarded in certain circles as revolutionary. For example, they conclude that the "control of slump alone is not a valid means to ensure adequate low mobility grout," and further that "irrespective of slump or pumpability" criteria, grouts that are too mobile can result in hydraulic fracturing of the soil and loss of control over the operation. High mobility can result from excessive clay and/or water, whereas the addition of coarse aggregate has been observed to be advantageous to rheology. They also found that injection rates should be maintained at less than 10 gallons/minute to enhance the development of regularly shaped bulbs.

It is against this backdrop of opportunity, challenge and discovery that compaction grouting expands into its fifth decade of applications.

Hydrofracture grouting. - The concept is that stable, high mobility cementitious grouts are injected at relatively high rates and pressures to deliberately fracture the ground. The lenses, ribbons and bulkheads of grout so formed are conceived as increasing total stresses, filling unconnected voids, locally consolidating or densifying the soil and providing a framework of impermeable membranes. It has been rare to find this principle deliberately exploited outside the French grouting industry, although there is no doubt that the effects have often been achieved, unintentionally, in the course of other methods of grouting. Warner, as noted above, has identified the possibility in compaction grouting operations, while

Tornaghi et al. (1988) note that hydrofracture *naturally* occurs with conventional cement-based grouts in soils with a permeability of less than 10^{-3} m/sec.

Graf (1990) has described recent tests conducted in the United States towards rationalizing certain parameters. Apparently polypropylene fibers have been incorporated into the grout to provide a degree of tensile and flexural strength to the grout bodies after setting. In California especially, certain contractors are actively promoting the application of "controlled fracture" grouting for applications involving slope stabilization, loose fill consolidation, expansive soil treatment and soft ground tunneling. Despite the potential, the term "controlled fracture" remains nevertheless for many American grouting engineers a contradiction in terms.

Most recently, however, tube à manchette techniques were used to reconstitute the clay core of Mud Mountain Dam, Washington (Eckerlin, 1992). Loose zones and voids had developed as defects in the core which then experienced severe hydraulic fracturing by the bentonite slurry being used in the attempted construction of a 450-ft-deep diaphragm wall through the dam. Over 5000 yd^3 of slurry were rapidly lost into the core while excavating the early panels, and the dam was longitudinally split. A phase of gravity grouting was first undertaken to fill the voids and fissures caused by the bentonite slurry. A program of "recompression" grouting was then undertaken to recompact the core and improve the soil stress conditions. "The recompression technique created soil cracks in multiple directions by hydraulically fracturing with grout forming structures that provided cohesion and resistance to further fracturing." Cement bentonite grouts were used with sodium silicate added to vary setting time from 2 to 60 minutes. Over 5000 yd^3 of grouts were injected into over 19,000 ft of grout holes, and this remedial program, during which the drilling and grouting parameters were electronically monitored, "practically eliminated" slurry losses during the remainder of the diaphragm wall work, intended to seal the core.

Jet grouting. - The tremendous upsurge in jet grouting throughout the world since the late 1970s has not been reflected by its rather subdued market volume in the United States. This is despite the excellent effort put forward by certain specialty contractors (Burke et al., 1989; Welsh and Burke, 1991), independent authorities (Kauschinger et al., 1992), Federal agencies such as the Corps of Engineers and the Bureau of Reclamation (Paul, 1988), and educators at short courses.

Both the one-fluid (i.e., cement) and the three-fluid (i.e., cement, water, and air) methods have been used successfully in a range of applications including water cutoffs, structural underpinning (probably the most common), hazardous waste containment (Gazaway and Jasperse, 1992), pile support (Andromalos and Gazaway, 1989), and tunnel presupport (Kauschinger et al., 1992). In the last named application, two significant case histories have to date been recorded: on the D.C. Metro, and on an Atlanta Metro tunnel under an active interstate highway. In Canada (Imrie et al., 1988), jet grouting was even conducted through the core of an existing embankment dam as part of a seismic retrofit program.

There are many obstacles in the path of universal application and acceptance.

- Firstly, it must be admitted that there have been disappointing experiences to set against the successes. These have been perpetuated by some contractors who have allowed certain operational subtleties to escape them in the translation from the original German, Italian, or Japanese; by other contractors whose advantage in high-pressure grouting equipment has alone not been a match for the vicissitudes of low bid geotechnical contracting; and by certain engineers who have simply, but unfortunately, specified the wrong technique.

- Secondly, and as referred to in the Introduction, it is doubtful if the state and direction of the construction industry truly needs the particular advantages of jet grouting on a large scale.

- Thirdly, it would seem that most of the benefits which jet grouting can impart, can be supplied by other techniques (such as pinpiles or Soil Mixed Wall) at a considerably lower cost.

From an American viewpoint, possibly the single biggest attraction of jet grouting is probably that it has the opportunity to be "designer driven." This would give it a unique position in an industry where experience and "feel" are key elements, and most of the knowledge - to universal suspicion - lies in the hands of the specialty contractors. In short, it could become a "by the book" technique, greatly reducing economic, technical, and operational risk, and providing a certain predictable level of reliability in the final product, even in the poorest soils.

It will be fascinating to see the outcome of this debate, for the market remains small but expectations and awareness remain high. The future could well be decided on the outcome of one major, high profile application: as grouters we trust it will be an extravagant success.

Mechanical mix in place. - By convention, this method typified by proprietary names such as SMW (Soil Mixed Wall) and DSM (Deep Soil Mixing) is not regarded as soil grouting, even though its origins are over 30 years old (Jasperse and Ryan, 1992). However, it does fulfill certain criteria for inclusion in this review: it uses conventional cement based grouts; it certainly improves the mechanical and hydraulic properties of the treated soil; and, importantly, it is challenging conventional grouting methods in a wide range of applications. The fact that it does not feature injection, sensu strictu, into the soil is not sufficiently overbearing to delete it from discussion.

The method features the introduction of cementitious grouts down the stems of large diameter (22 to 40 inches) discontinuous flight augers as they are rotated to target depth (Figure 4). Each rig may have multiple augers (up to a maximum of four), although the role of the central units is often just to encourage breakup of the soil by injecting air or water. A smaller amount of grout is placed during withdrawal of the auger. The result is the formation of soil-cement columns, which by proper selection of equipment and sequencing can be combined into continuous in situ walls. Developments are being made with the injection of dry materials which react in place, e.g., the RODEM[5] method (Rodio, 1992).

Applications in the United States include support of excavation structural walls (when appropriately reinforced), waste containments, and hydraulic cutoffs for dams (Cushman Dam, Washington) and levees (Sacramento, California). The single largest example to date was for the seismic retrofit of Jackson Lake Dam, Wyoming. Here over 430,000 lin ft of columns were installed in a cellular, sexagonal pattern to improve the liquefaction resistance of a major dam foundation and a 230,000-ft^2 curtain to a depth of 105 ft was similarly formed (Figure 5).

Mix in place methods are proving extremely competitive in appropriate conditions. Less attractive circumstances include a) very dense, bouldery or obstructed overburden, b) low headroom, difficult access, c) depths over about 100 ft (although 200 ft is claimed as the maximum), and d) projects of limited scope.

The advantages of the concept have been further exploited in the sister technique of SSM (Shallow Soil Mixing) wherein larger diameter mixing heads are used for fixing hazardous materials to depths of 7 to 25 ft (Jasperse and Ryan, 1992). This system permits the use of dry reagents and an effective vapor collection apparatus. It can be used with cementitious, chemical or even biological reagents as required. One variant uses steam or hot air to extract volatile pollutants from the subsoil.

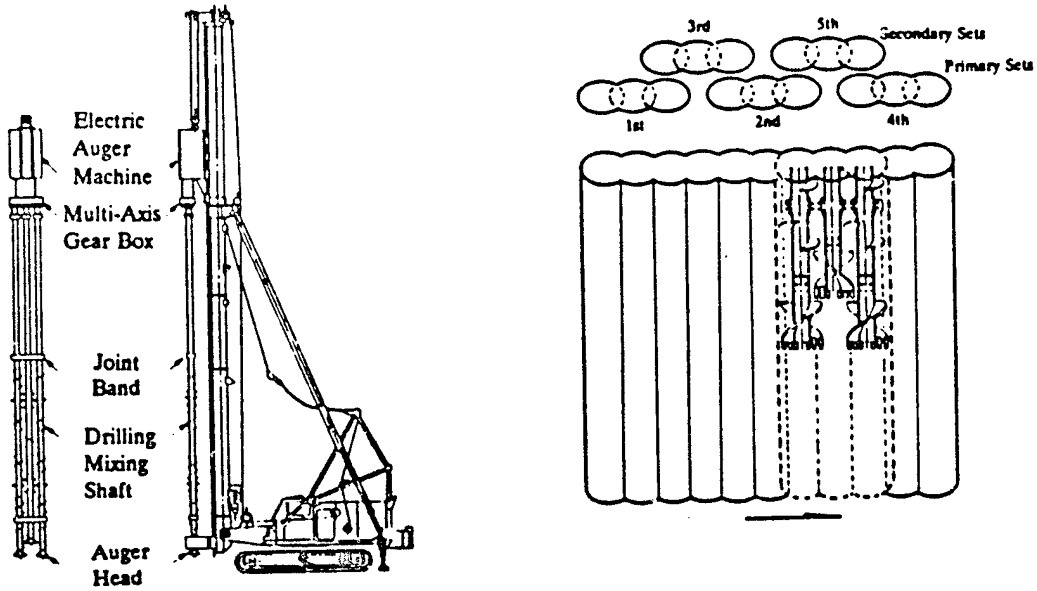


Figure 4. - Equipment and procedure for SMW system (Taki and Yang, 1991).

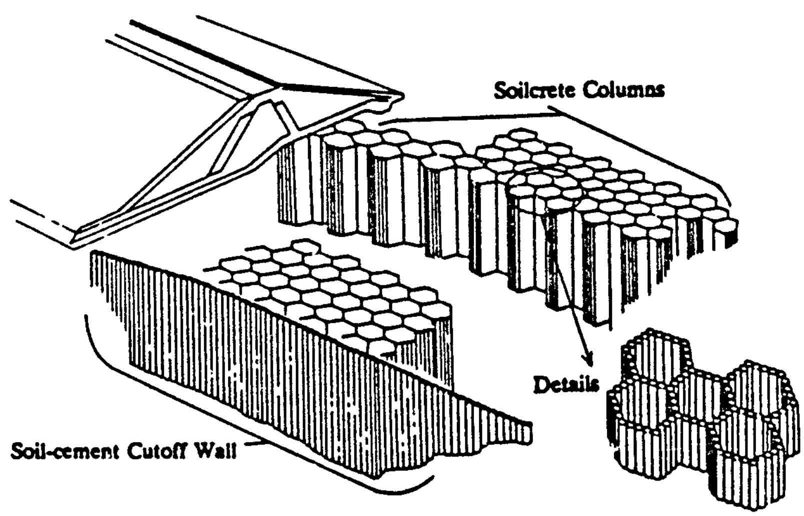


Figure 5. - Soil-cement mixing work for Jackson Lake Dam modification project (Taki and Yang 1991).

Outside the environmental market, however, there is considerable potential for the SMW technique, for it seriously threatens the former preserves of diaphragm walling, conventional "beams and lagging" support, jet-grouted cutoffs, and a whole range of ground improvement technologies (including compaction grouting) which may be considered for liquefaction control.

Miscellaneous Trends and Developments

There are many other aspects of the drilling and grouting market which are undergoing rapid and important development. As detailed at the ASCE Specialty Conference on Grouting, Soil Improvement and Geosynthetics, held at New Orleans, LA, in February 1992, and as summarized by Bruce (1992c) these can be categorized as follows:

- Improvement in the various types of overburden drilling equipment and methods (Table 1), and a greater inclination amongst the drilling community to free themselves from local or traditional paradigms in selecting the most apposite approach to each site's demands.

- Microfine *cements* have been imported into the States since 1984 and have been well marketed (Clarke et al., 1992) and researched (e.g., Schwarz and Krizek, 1992). There remain, however, certain problems associated with timely availability (in large quantities), handling, preparation and cost. Much favorable attention has recently been focused on an alternative principle.

The Cemill[R] technology (DePaoli et al., 1992a) permits microfine grouts to be produced, on site, from normal cement grouts, in a wet regrinding process (Figure 6). Excellent grain size characteristics are produced (Figure 7), resulting in enhanced penetrability characteristics (Figure 8). Yet to be exploited in the United States, this method is proving highly successful - technically and economically, in Italy.

Equally attractive to the U.S. market is the concept of improving the penetrability of cementitious grouts by fundamentally examining their rheological and internal stability characteristics. The Mistra[R] series of grouts (DePaoli et al., 1992b) has already been successfully exploited in Europe (Mongilardi and Tornaghi, 1986) and provides extremely stable mixes with greatly reduced cohesion (Figure 9). Both these features generate major technological and economical benefits, and the concept is attracting favorable interest in the United States

Regarding chemical grouts, sodium silicate bases remain the most popular for general purpose. Other materials such as phenoplasts, aminoplasts, chrome lignins, and acrylamides are well known in the United States (Karol, 1990) but are not very common due to environmental concerns, and, simply, cost. Urea formaldehydes have been used (Graf et al., 1985) but require meticulous preparation and may not always be permitted by "regulatory circumstances" (Weaver, 1991). Several specialty formulators are promoting a variety of polyurethane grouts, and water reactive prepolymers, but to date their application has been somewhat limited by cost to small (albeit very challenging) applications.

The Environmental Protection Agency is considering a ban on acrylamides and methylolacrylamide grouts currently used extensively in rehabilitation of sewerlines and manholes, while according to McIntosh (1992), a possible acrylate monomer replacement, AC-400, "has essentially been rejected by the industry" despite attracting the interest of excellent research efforts (Schwarz and Krizek, 1992). The use of epoxy resins has been limited to the structural repair of concrete structures (Bruce and DePorcellinis, 1991) while there remains a sporadic market (Bruce, 1990) for hot asphalt injection for the interim sealing of fast and large seepages.

Table 1. - Summary of overburden drilling methods.

DRILLING METHOD	PRINCIPLE	COMMON DIAMETERS AND DEPTHS	NOTES
1. Single Tube Advancement a) Drive Drilling	Casing, with "lost point" percussed without flush.	2-4" TO 100'	Hates obstructions or very dense soils.
b) External Flush	Casing, with shoe, rotated with strong water flush.	4-8" to 150'	Very common for anchor installation. Needs high torque head and powerful flush pump.
2. Rotary Duplex	Simultaneous rotation and advancement of casing plus internal rod, carrying flush.	4-8" to 200'	Used only in very sensitive soil/site conditions. Needs positive flush return. Needs high torque.
3. Rotary Percussive Concentric Duplex	As 2, above, except casing and rods percussed as well as rotated.	3-1/2 -7" to 120'	Useful in obstructed/bouldery conditions. Needs powerful top rotary percussive hammer.
4. Rotary Percussive Eccentric Duplex	As 2, except eccentric bit on rod cuts oversized hole to ease casing advance.	3-1/2 -8" to 200'	Obsolescent, expensive and difficult system for difficult overburden. Largely restricted to water wells.
5. "Double Head" Duplex	As 2 or 3, except casing and rods rotate in opposite senses.	4-6" to 200'	Powerful, newer system for fast, straight drilling in worst soils. Needs large hydraulic power.
6. Hollow Stem Auger	Auger rotated to depth to permit subsequent introduction of tendon through stem.	6-15" to 100'	Hates obstructions, needs care in cohesionless soils. Prevents application of higher grout pressures.

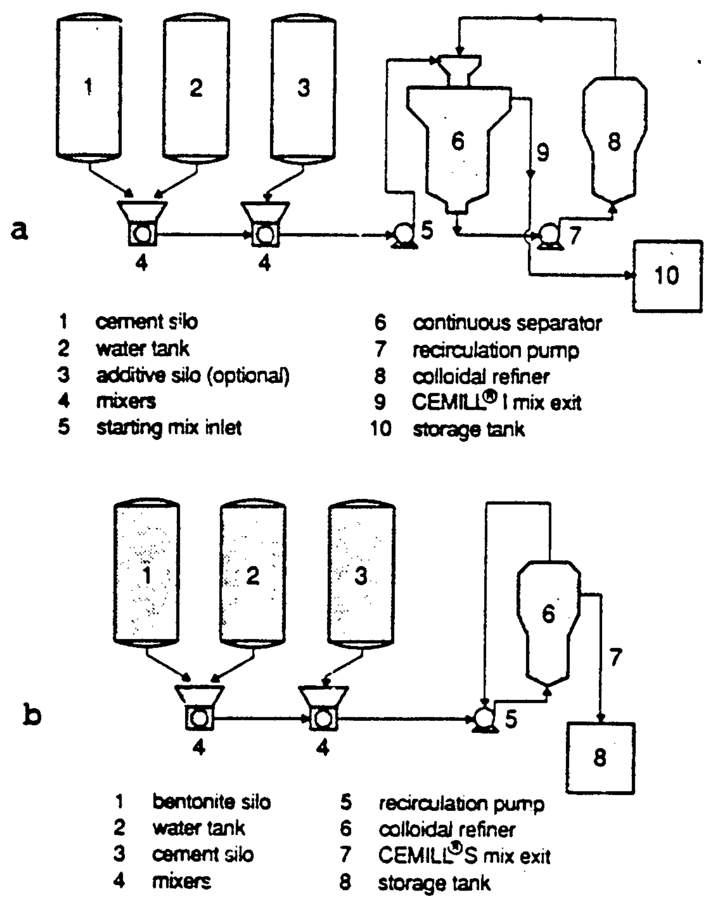


Figure 6. - Layout of production plants for (a) CEMILL-I (unstable grouts) and (b) CEMILL-S (stable grouts) (DePaoli et al., 1992a).

	grain size (μmm)					
	D 95	D 85	D 60	D 50	D 15	D 10
CEMILL® 6	15.0	9.0	6.0	5.0	1.3	0.9
CEMILL® 9	9.0	5.5	3.5	2.5	0.6	0.4
CEMILL® 12	6.0	4.0	3.0	2.2	0.4	0.3
ONODA MC-500	8.0	60.0	4.5	4.0	2.5	2.0
Portland 525	40.0	22.0	11.0	8.0	2.5	2.0
bentonite	60.0	40.0	15.0	10.0	1.7	1.2

ⓐⓑⓒ sands for injection tests
ⓐ $\gamma = \gamma_{max} = 1.713$ g/cm^3
ⓑ $\gamma = \gamma_{max} = 1.701$ g/cm^3
ⓒ $\gamma = \gamma_{max} = 1.690$ g/cm^3
ⓓ bentonite
ⓔ Portland 525 cement
ⓕ ONODA MC-500 cement
ⓖⓗⓘ CEMILL® mixes

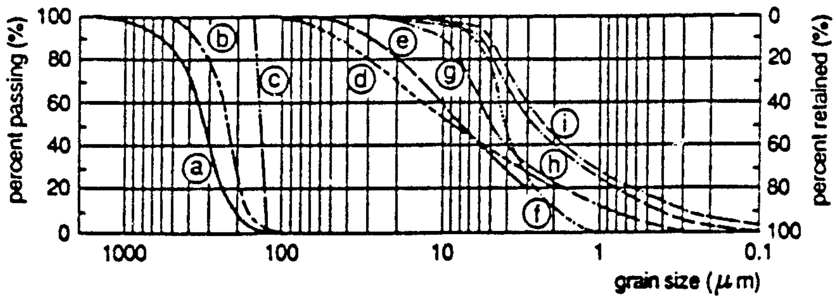
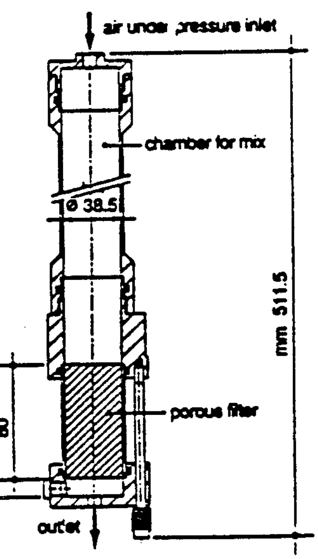


Figure 7. - Grain-size distribution curves for sands, dry materials, and grouts (DePaoli et al., 1992b).

filter no.	permeability (m/s)		grain size (μm)					porosimetry (μm)							specific surface cm^2/g	retaining capacity (μm)
	theoretical Hazen (C = 1.45)	experim. (permeam.)	D 95	D 60	D 15	D 10	U	theoretical (Kozeny) D 80	D 50	D 30	experim. (Hg porosimetry) D 95	D 85	D 15	D10		
07	$5.9 \cdot 10^{-3}$	$3.8 \cdot 10^{-3}$	1500	900	700	640	1.41	300	240	150	380	300	170	160	28	70
06	$2.3 \cdot 10^{-3}$	$8.3 \cdot 10^{-4}$	750	620	450	400	1.55	160	133	90	360	260	130	124	37	60
04	$7.7 \cdot 10^{-4}$	$4.5 \cdot 10^{-4}$	700	480	250	230	2.09	110	90	60	300	140	70	64	56	40
01	$2.8 \cdot 10^{-4}$	$1.6 \cdot 10^{-4}$	400	230	160	140	1.64	58	49	32	120	64	46	44	111	10
005	$1.4 \cdot 10^{-4}$	$9.5 \cdot 10^{-5}$	180	120	110	100	1.20	35	25	18	90	46	32	30	125	5

a

b



c

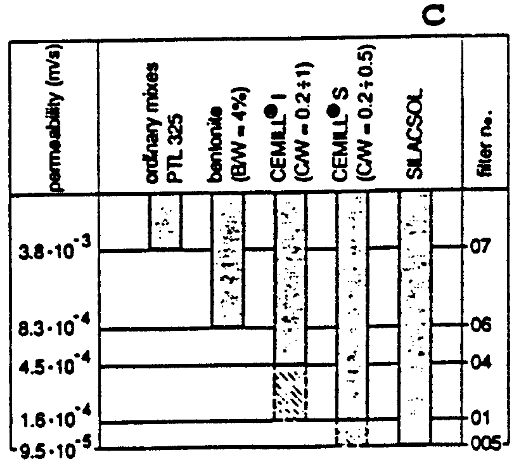


Figure 8. - Injection test details (a) porous stone filter characteristics, (b) penetrability limit of different mixes into filters (DePaoli et al., 1992b).

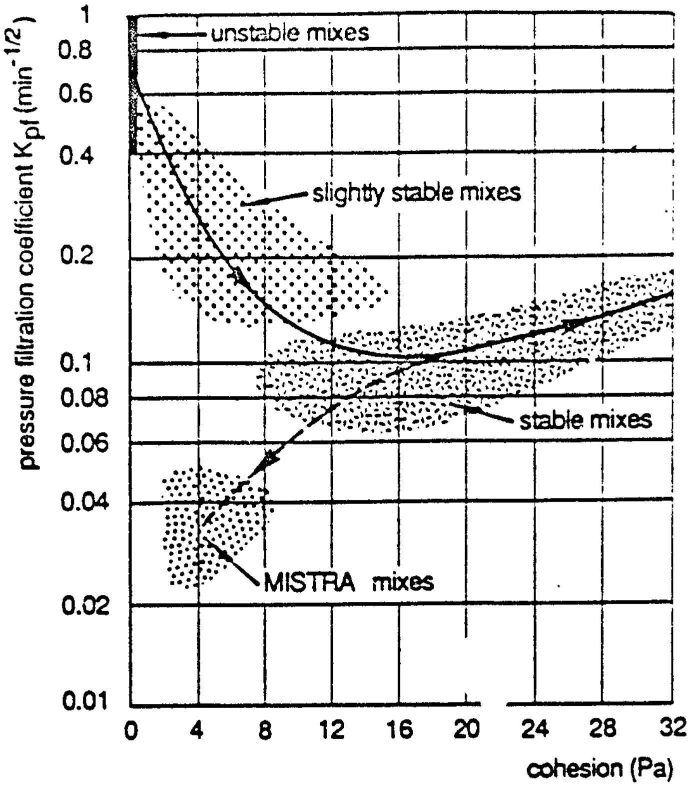


Figure 9. - Relationship between stability under pressure and cohesion for the different types of mixes (DePaoli et al., 1992a).

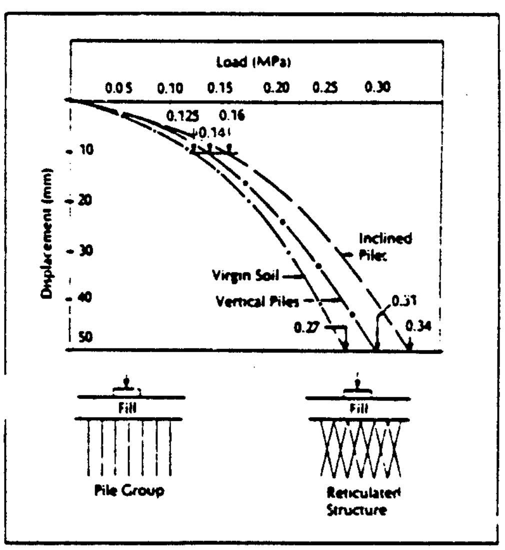


Figure 10. - Field test data for different minipile arrangements (Plumelle, 1984)

IN SITU EARTH REINFORCEMENT: TYPE A WALL

General Features

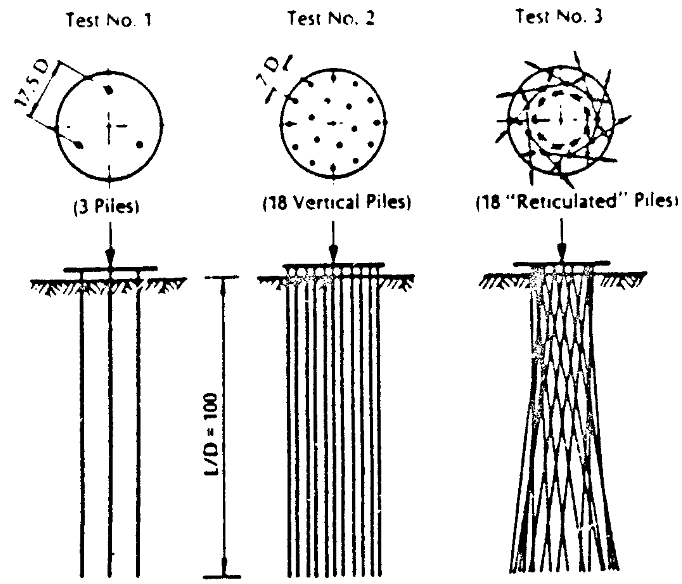
Early applications of conventional, axially loaded pinpiles indicated, surprisingly, a positive "group effect," thought to be due to beneficial soil-structure interaction (Figures 10 and 11). This advantage was then exploited in slope stability applications in Western Europe and later - but infrequently -in the United States. In urban environments similar groups of pinpiles (or "INSERTS" in this context) can be used in cut and cover, as well as bored tunnel, construction. There the concept is to create protective structures in the ground to separate the foundation soil of the building from the zones that are potentially subject to disturbance (Figure 12). All these INSERT structures rely for their effectiveness on soil/pile interaction, and not on intergranular soil cementation. This composite structure - referred to as a "Type A Wall" because of its distinctive cross sectional appearance - is intended to stop loss of soil from behind it, and to prevent sliding along potential failure planes passing through it.

Design approaches continue to lag behind other aspects of the technology, but several instrumented field programs have confirmed that reinforcement stresses and overall wall movements in service are minimal, and that most probably designs have been highly conservative. Even their original proponent - Fernando Lizzi - confirmed in 1982 that "it is not yet possible to have at our disposal an exhaustive means of calculation ready to be applied with safety and completeness." In addition, an ASCE Committee (1987) also alluded to the great reliance placed on soil/pile interaction, the safe exploitation of "which is still subject to experience and intuition."

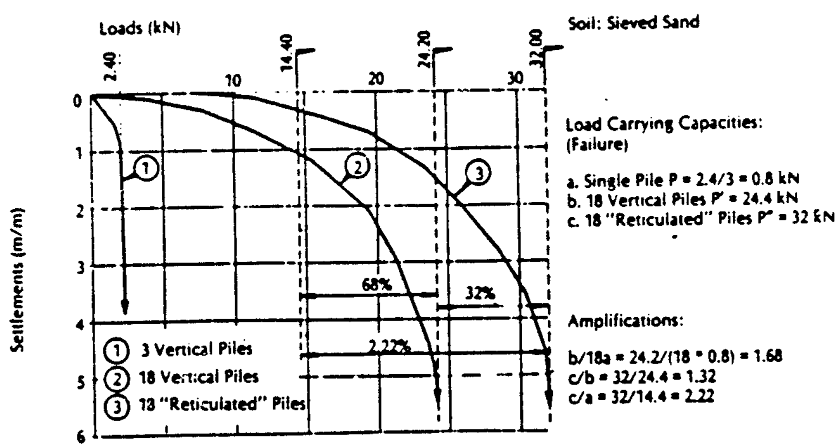
The typical approach to design is, of course, relatively simple, and involves standard basic steps:

- Estimating loads (active and passive) on the wall;
- Conducting a stability analysis to determine the shear force needed to maintain a required factor of safety;
- Determining the number of INSERTS needed to provide the required shear resistance;
- Calculations (similar to those for a conventional gravity wall) to check stability against overturning, sliding and bearing failure at the base of the wall.

Usually the INSERTS are extended into bedrock where economically possible, but, in any event, always below the potential failure plane). INSERT Walls can be constructed in close proximity to existing buildings in relatively tight access locations without the need to excavate or underpin, and without causing any decompression of the foundation soil. Given their mode of construction, as detailed below, they can be installed in any type of ground, including through boulders, old foundations or other obstructions with no constructional limitation on hole inclination or orientation.



Arrangement of Piles in Model Test



Load Test Results for Piles in Coarse Sieved Sand

Figure 11. - Model test data for different minipile arrangements in coarse sieved sand (Lizzi, 1978).

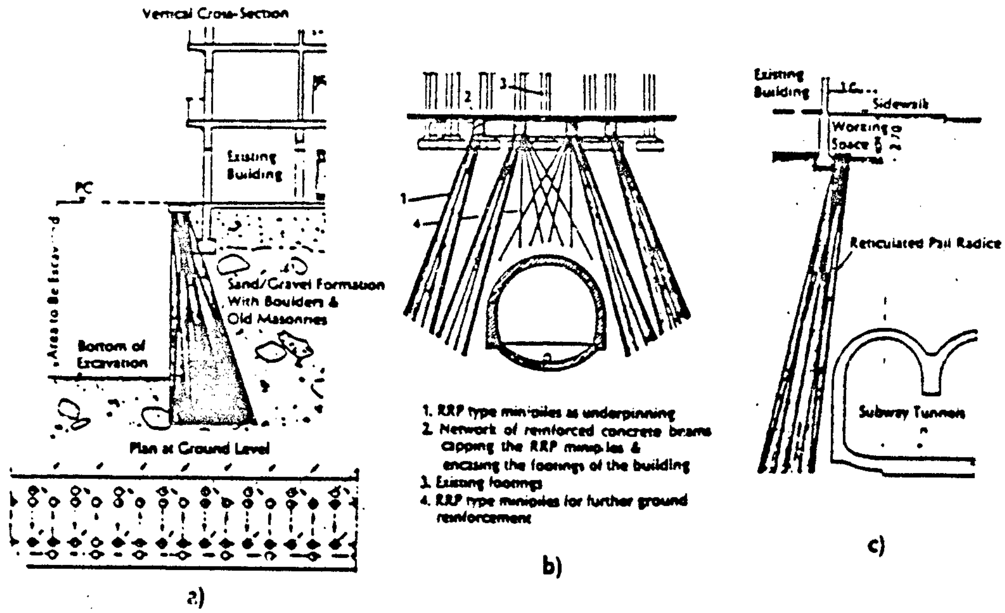


Figure 12. - Applications of INSERTS a) for cut and cover and b) and c) around bored tunnels (Bruce, 1989).

Construction Aspects

The successive steps involved in the construction of a Type A Wall are illustrated in Figure 13. The capping beam may be installed before or after the INSERTS are formed, although field evidence suggests that the latter option allows for an earlier benefit from the reinforcement. The drilling method is chosen to ensure minimal disturbance or upheaval to the soil. Of the six generic methods of overburden drilling, the most common method is rotary drilling with water flush, either via a single casing or by the duplex method, depending on ground conditions. Once the casing has been advanced to target depth it is filled with a stable, high-strength cementitious grout, and the permanent reinforcement is placed. This may be a solid high-strength steel bar, typically 1 to 2 inches in diameter, or a steel pipe of suitable dimensions, as dictated by the structural design requirements. The drill casing is then withdrawn from the hole as grout continues to be injected under pressure. The effect of the pressure grouting is three-fold in most conditions:

- It ensures all voids or drilling related disturbances to the soil are filled;
- It permeates a little into sands and gravels;
- It compacts somewhat soil around the pile that is too fine to be permeated.

Individual piles are oriented in different directions in each plane to promote the most effective soil/pile network. After installation of the INSERTS, the capping beam is simply graded over, or it can form the base of a guard rail or similar: the whole wall is thus wholly out of sight and maintenance free.

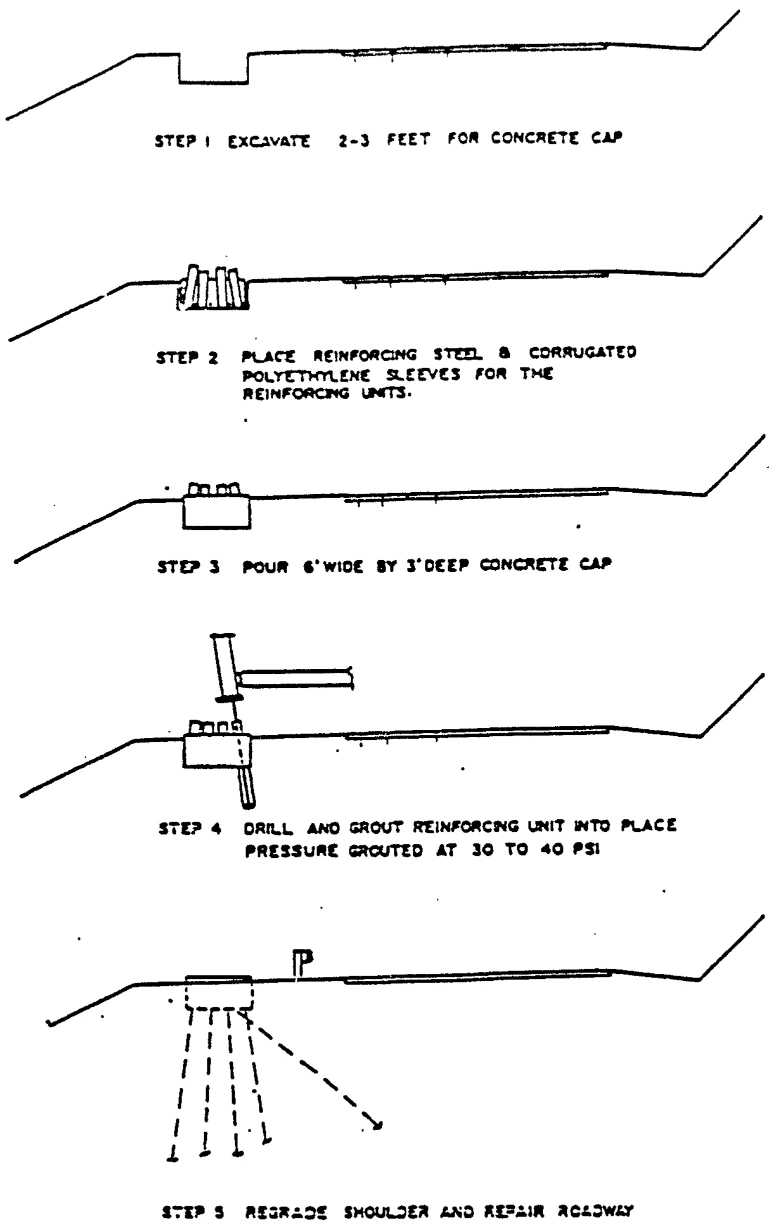


Figure 13. - Typical steps in INSERT wall construction.

Case History: Road in Armstrong County, Pennsylvania

Portions of State Route 4023 north of Kittanning, Pennsylvania, were constructed on a slope adjacent to the Allegheny River. A 240-ft-long section of the two-lane road, and the railroad tracks located upslope, experienced damage caused by slope movements toward the river. In June, 1988, and January and February 1989, the owner conducted a subsurface exploration program and installed slope inclinometer casings to monitor the slope movements. The inclinometers indicated that a slip-plane was located approximately 26 to 36 ft below the roadway and that the slope was moving at a rate of up to 0.75 inch per month downwards toward the river.

Boreholes showed that a significant amount (20 to 30 ft) of fill had been placed at the site apparently during the construction of the roadway and/or railroad tracks. The fill consisted of intermixed loose to medium dense rock fragments and medium stiff silty clay. Underlying the fill was a 5- to 10-ft-thick layer of stiff colluvial clay with rock fragments, in turn overlying a 3- to 20-m layer of weathered claystone. Competent rock was encountered at about 50 ft below the roadway, and generally consisted of medium hard siltstones and sandstones.

The owner designed a repair of the failed section using an anchored caisson wall extending into competent rock. The earth pressures used for the design were based on the results of stability analyses, for which the soil along the slip-plane was assigned a residual friction angle of 17°. This design provided a minimum factor of safety with regard to the overall slope stability equal to 1.5 and 1.2 for the normal and rapid drawdown conditions, respectively. A row of 3-ft-diameter caissons were foreseen at a center-to-center spacing of 4.5 ft and located immediately downhill of the roadway. The caissons were to be connected at the top by a cast-in-place reinforced concrete cap which was to have 90-ft-long prestressed rock anchors extending underneath the roadway at 7-ft lateral intervals.

In 1989, the contractor proposed and the owner accepted an alternative design employing an INSERT Type "A" Wall. The wall consisted of four rows of pinpiles extending across the slip-plane and into competent bedrock. It comprised two equal length sections designated as Wall A and Wall B. Wall A contained a higher density of piles than Wall B, because the top of weathered rock dipped to a lower elevation in the area of Wall A which resulted in a larger volume of soil to be stabilized in this area. In general, Wall A contained 1.3 piles per lin ft, and Wall B contained 0.9 pile per lineal foot. Besides providing a significant economic savings over the original design, the selection of the INSERT Wall allowed for one lane of roadway to remain open during construction (February to May, 1989). The wall was constructed as described above, with the cap poured after pinpile installation for practical reasons.

To monitor the INSERT Wall performance, two sections of the wall were instrumented with strain gauges, inclinometers, telltales, and survey pins. The inclinometers yielded the most useful information regarding the performance of the wall. The data for inclinometers located relatively close to and within the wall indicated that up to 1.5 inches of horizontal movement occurred during the 75-day construction period, but that a maximum of 0.3 inch of movement occurred in the 7-month period following the completion of the wall.

Overall, the inclinometer data indicated that the wall performed as expected, and had effectively stopped the slope movements at the site. These data also confirmed that some deflection of the relatively flexible INSERTS may be required to mobilize their lateral resistance.

CONTRACTING PRACTICES

Weaver (1991) addressed the elements necessary for a successful grouting project: they are equally valid for INSERT works

- A design accommodating the site geological conditions;
- Specifications that allow or facilitate modifications to the works as the site conditions are revealed;
- An "experienced, competent, cooperative and honest" contractor;
- Appropriate materials, equipment and techniques;
- Knowledgeable inspection staff, and
- An effective quality assurance program.

While reviewing the history of grouting, in particular, in the United States, however, it is clear that rarely have these elements been simultaneously in place. The author believes that there are two fundamental reasons: inflexible specifications and "low bid" procurement systems.

Regarding specifications, these must "be tailored to the project in hand and to the objectives to be accomplished." Instead, successive generations of specifications have been cobbled together from sections lifted from previous documents, and often contain "boiler plate" sections which may be contradictory and always perpetuate the use of outmoded procedures and/or inappropriate materials. Specifications of this nature have dissuaded domestic contractors from innovating and have discouraged foreign specialists from competing.

The procurement system has proved equally stifling: the low bidder on a tightly specified job invariably wins the award, although he then operates as little more than a broker of labor, equipment, and materials. However, in recent years there have been encouraging signs that a more enlightened approach is surfacing.

As a first step, stronger prequalification criteria are being applied to prospective bidders and their personnel. Specifications are being changed to "performance" types, to encourage bidders to be creative and innovative, and, most significantly, awards are being made not just on the basis of a low bid (Nicholson, 1990). In addition, many owners, including Federal agencies, are promoting the concept of having "partnering" agreements between all the involved parties. This concept is a recognition that every contract includes an implied covenant of good faith. The process attempts to establish working relationships through a mutually developed formal strategy of commitment and communication. It tries to create an environment where trust and teamwork prevent disputes, improve quality, promote safety, and continue to facilitate the execution of a successful project. Significantly, it is wholly endorsed by the Associated General Contractors of America, a group which has not always favored the more innovative procurement procedures.

Two recent examples can be cited to illustrate the operation and benefits of these newer contracting practices. The first example is the rock anchoring project recently completed at Stewart Mountain Dam, Arizona (Bruce et al., 1991b). This very delicate but critical dam stabilization was studied by the owner and the Bureau of Reclamation for many years, during which time they interviewed various specialists from all facets of the anchorage industry. The result was a very challenging specification which set well-defined targets but allowed the bidders a great deal of scope for original thought. Each bidder had to submit a very detailed Technical Proposal, which was closely graded by a Government team of specialists. A separate Price Proposal was submitted, but this was adjudicated by an independent group. The results were then combined, with a heavy weighting placed on the score from the Technical Proposal As a result, the best qualified responsive contractor was chosen, having been encouraged to write and price an individual and extremely detailed method statement. In every respect the project was a stunning success, and was completed within program, under budget and without a hint of litigation.

The second illustration is a much smaller remedial grouting operation, also undertaken by Nicholson, at Lake Jocassee Dam, South Carolina (Bruce et al., 1992). Seepage through the left abutment of this high embankment structure had to be addressed by the owner, Duke Power following an intervention by the Federal Energy Regulatory Commission. Again, a performance specification was set and a small number of prequalified contractors were permitted to bid. Again, a strong technical proposal proved crucial to securing the award. Using the new approach of "Responsive Integration℠," the seepage was greatly reduced and the grouting deemed a major, and (in the light of previous local experience) surprising success.

Similar contracting and procurement principles have also recently been exploited at major remediation projects at Horse Mesa Dam, Arizona, and at the United Grain Terminal, Port Vancouver, Washington - to mutual advantage.

FINAL REMARKS

In the fields of ground treatment by grouting, and in situ reinforcement by Type A Walls, it is a period of considerable dynamism. In each field there is a long history of application in the United States but close examination reveals that the experience has not always been wholly satisfactory in either the technical or the contractual spheres. In recent years, various factors have conspired to improve the current situation and offer immense promise for the future. There factors include the impact of foreign technologies, the emergence of native "points of light," the changing demands of the American construction industry, and the heartfelt desire to move towards innovative contracting procedures and partnerships.

With these prospects in place, one can understand the new optimism of those involved in each corner of the business of ground treatment and reinforcement. With these people in place, one can foresee a new identify for our national efforts on the world stage.

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TECHNICAL SESSION 3

Geosynthetics/Soil Reinforcement Systems

HISTORY OF REINFORCED WALLS IN THE USDA FOREST SERVICE

By John E. Steward[1], Member ASCE

INTRODUCTION

The USDA Forest Service manages activities on 123 national forests and 19 national grasslands covering 191 million acres. Accessing and managing these lands requires an extensive transportation system. Retaining walls and reinforced soil concepts have been used extensively to build and maintain a functional transportation system. The rugged terrain, remote areas, and secondary nature of the roads require innovative and adaptable designs.

Forest Service personnel have been innovative in adapting the theories and practice of soil reinforcement since the early 1970's. The first known geotextile wall built in the United States was built on the Siskiyou National Forest in southwestern Oregon in 1974. The second geotextile wall was built on the Olympic National Forest in western Washington in 1975. Since 1974 and 1975, the Forest Service has built a wide variety of structures using a variety of reinforced soil materials. Geotextiles, tires, lightweight wood backfill materials, chainlink fencing, fiberglass roving, manure, straw bales, and geocomposite drains have all been used in retaining structures.

This paper presents the development of soil reinforcement uses by the Forest Service. Topics include basic principles and guidelines developed during the 1970's, creative engineering and innovation in the 1980's, and a look to future developments for the 1990's.

The 1970's were marked by trial uses of new materials and methods and development of basic principles and guidelines for the use of soil reinforcement. Revision of the *Retaining Wall Design Guide* (Driscoll, 1979) will benefit the Forest Service by capturing and sharing the innovations of the 1980's. Development and adoption of new methods and technologies such as soil nailing will continue in the 1990's.

THE 1970'S – DEVELOPMENT OF BASIC PRINCIPLES AND GUIDELINES

Many geotechnical engineers and engineering geologists were hired by the Forest Service in the early 1970's. Their skills proved essential for locating and building a stable transportation system in rugged terrain. Until the early to mid-1970's, standard design gravity retaining structures were the norm. Steel and aluminum bin walls, concrete and wood crib walls, and wire gabion walls were the predominate retaining structures built during this period. The geotechnical personnel, with their understanding of soil mechanics and slope stability, began to develop custom designs for local conditions.

The geotechnical personnel readily adopted the concepts of soil reinforcement when introduced in the early 1970's. The introduction of geotextiles as soil reinforcement in the mid-1970's presented many opportunities. Table 1 lists some key developments with geotextiles and soil reinforcement in the Forest Service. By 1979, the basic principles and guidelines of soil reinforcement were established for use with low-volume roads. These principles and guidelines were captured, along with basic good design information, in *Retaining Wall Design Guide* (Driscoll, 1979).

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The first known geotextile-reinforced wall built in the United States was built on the Siskiyou National Forest in 1974. This wall, shown in Figure 1, was built using non-woven, needle-punched, polypropylene geotextiles. The wall was built to verify laboratory model tests performed by Professor J Richard Bell at Oregon State University. The wall face was formed by bags filled with sand. The face of the wall was covered with gunite (shot concrete) for protection from sunlight and vandalism. Plastic pipes were inserted in the wall for drainage.

Figure 2 shows the second geotextile wall built in 1975 near Shelton, Washington. This wall, about 176 ft long by 19 ft high, was fully instrumented for internal and external movement. Non-woven, needle-punched polyester and polypropylene geotextiles were used to compare their strength and elongation characteristics. There was much concern at the time about the potential for long-term creep in geotextiles under constant tensile load. Less than 1 inch of horizontal or vertical movement was measured in the wall during the first 6 months after construction. Monitoring over the next 3 years indicated no further movement within the wall (Mohney and Steward, 1977).

Temporary steel L-braces and wood boards were used to support the face during construction. This method continues to be used today for the construction of geotextile retaining walls. The surface of this wall was sprayed with the emulsified asphalt for protection from sunlight. Very little degradation of the geotextile face had occurred when the wall was recoated with emulsified asphalt about 1985.

About 1976, Bruce Vandre, then Geotechnical Engineer on the Siskiyou National Forest in Oregon, designed a soil wall using chainlink fencing for the reinforcement. Tensile tests, soil pullout tests (Figure 3), and corrosion tests were performed on the metal fabric to develop design information. The chainlink wall, example shown in Figure 4, appears to be the basis for the commercial development of the welded wire wall shown in Figure 5. The welded wire wall has proven to be economical and easy to construct, resulting in extensive use of this wall type throughout the United States.

Several innovative reinforced soil or partially reinforced soil wall types were designed and constructed in the early 1970's. Figure 6 shows a culvert pipe wall with facing restrained by strap anchors. The wall in the photo, located on Mary's Peak near Corvallis, Oregon, was instrumented for performance. This wall, which is nearly 60 ft high, has performed very well.

The wide variety of wall types developed and being used in the early to mid-1970's presented a need for retaining wall design guidelines (Driscoll, 1979). The *Retaining Wall Design Guide* documented design methods and provided practical guidance for design and construction of retaining walls for low-volume roads.

The *Retaining Wall Design Guide* provided design guidance for most wall types in use at the time. Table 2 lists the wall types contained in the design guide by design method and probable behavior of the wall. The walls are further classified in the guide as standard or nonstandard as shown in Table 3.

Retaining walls are designed for internal and external stability. Table 4 presents recommended factors of safety for external stability analysis.

Important features for internal design of soil-reinforced geotextile walls is shown in Figure 7.

Design equations related to Figure 7 are:

Vertical Spacing - x

$$x = \frac{Sf}{(FS)(\tau_h)}$$

Embedded Length - Le

$$Le = \frac{S_f}{(FS_f(2d\delta\tan\phi_f)}$$

or 3.0 ft min.

Overlap - Lo

$$Lo = \frac{hKx\ (FS_o)}{2d_f\tan\phi_f}$$

or 3.0 ft min.

where

S_f	=	Strength of the geotextile
FS	=	Desired factor of safety
γ_h	=	Horizontal stress at midpoint of layer.
FS_f	=	Factor of safety for the geotextile, generally in the range of 1.50 to 1.75
δ	=	Unit weight of the backfill
ϕ_f	=	Soil-geotextile friction angle generally taken as 2/3 f
FS_O	=	Factor of safety of the overlap

The basic approach to the design of geotextile-reinforced walls presented here has been used for most soil-reinforced walls designed by Forest Service personnel since the mid-1970's. One factor that has changed is the geotextile strength used for the design.

Initially, the working strength was based on limiting the loading to minimize creep. The working load was limited to 35 to 50 percent of the wide width (8-inch-minimum sample width) tensile strength of the geotextile. Later, some walls were designed for the minimum of the strength at 10 percent elongation or 30 to 50 percent of the wide width tensile strength. Recent work by the Federal Highway Administration (FHA) (Christopher et al., 1990) indicates other strength properties may be more appropriate.

THE 1980'S -- INNOVATIVE ENGINEERING APPLICATIONS

The 1980's were a period of innovation and creativity in the design of soil-reinforced walls using the concepts developed in the 1970's. Walls were constructed using local and lightweight materials for backfill, and a variety of facing and internal reinforcement materials. Innovative reinforced soil walls developed or used include:

Chainlink Fabric Walls (Figure 4.)

Welded Wire Walls (Figure 5.)

Concrete Block-Faced Reinforced Walls (Figures 8a and 8b)

Geotextile-Reinforced Soil Walls (Figure 9)

Tire-Faced Reinforced Soil Walls (Figures 10a and 10b) (Keller and Cummins, 1990)

Wood-Faced Reinforced Soil Walls (Figures 11a and 11b)

Manure-Hay Reinforced Soil Walls (Figures 12a and 12b)

Fiberglass Roving Reinforced Soil Walls (Figures 13a and 13b)

Local Backfill Materials: Local backfill has been used for several walls and buttresses as shown in Figures 10, 11, 12, 13, and 14 (Burke, 1988; Keller, 1989 and 1990; and McNemar, 1989).

Lightweight Backfill Reinforced Walls (Figure 15)

Table 5 developed by Gordon Keller, Geotechnical Engineer on the Plumas National Forest, summarizes soil properties and performance of several walls using local materials (Keller, 1990). Positive internal drainage to prevent hydrostatic pressures is a key feature in the design and construction of each of these walls. Geocomposite drainage materials have been used extensively in retaining walls in the last 5 to 7 years.

Wood materials such as sawdust, wood chips, and "hog fuel" (small pieces of wood produced by processing wood scraps through a machine called a "hog") have been used for retaining wall backfill (Figure 15). The wood backfill, which weighs 40 to 60 lb/ft^3, wet weight, is about 50 percent of the weight of soil and rock backfill materials, minimizing loading in potentially unstable areas. Shredded tires, with a unit weight of about 35 lb/ft^3, would also be a suitable lightweight backfill material.

Table 6, also developed by Gordon Keller (Keller, 1990), shows a comparison of various internally reinforced walls. Costs of these walls, updated to 1992, appear reasonable, especially considering the remote locations. Driven H-pile walls, commonly used in steep terrain, typically cost in the range of $30 to $60 per square foot of front face.

Table 7 lists Forest Service personnel responsible for many of the case histories cited in this paper. These contacts are provided for readers wanting to "try these walls at home."

THE 1990'S -- WHAT IS AHEAD?

An effort is just beginning, in cooperation with the FHA, Coordinated Technology Implementation Program (CTIP), to rewrite and update the 1979 *Retaining Wall Design Guide* (Driscoll, 1979). Planned modifications include: 1) updating reinforced soil wall design criteria and facing systems, 2) updating risk analysis methodology, 3) standard designs for low height walls, especially for geotextile reinforced walls, and 4) reinforced soil slopes.

The wide variety of facing materials and systems available now and in the future will increase the aesthetic acceptability of geotextile-reinforced soil walls. Long-term durability testing of geotextiles under the leadership of the FHA will increase the reliability and predictability of long-term designs. These developments, aided by improved and simplified design methods and favorable costs, will lead to increased use of reinforced soil walls by the Forest Service and others.

An example of new technology for the 1990's is the use of launched soil nails for stabilizing slopes in lieu of retaining walls. The soil nail launcher literally shoots 1- and 1-1/2-inch diameter steel nails up to 20 ft long into the ground using one shot of compressed air. The soil nail launcher is being demonstrated in the Western United States during July and August 1992.

SUMMARY

Forest Service employees have developed, adopted, and provided basic concepts and criteria for design and construction of soil-reinforced walls and slopes during the 1980's and 1990's. Innovation has included a wide variety of facing, backfill, and internal reinforcement methods and materials. The use of local and lightweight materials for backfill has demonstrated the adaptability of these methods to build low-cost durable walls.

Soil-reinforced walls and slopes are adaptable to many sites and conditions. Use of these walls types is expected to increase in the future.

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Table 1. - Historical use of geotextile and soil reinforcement by the Forest Service.

1973	Used for filter layers (drainage) for roads.
1974	First Geotextile Reinforced Wall constructed on the Siskiyou National Forest (NF) near Cave Junction, OR.
1975	Second Geotextile Reinforced Wall constructed on the Olympic NF near Shelton, WA.
1976	Geotextile test road constructed on the Olympic NF near Quinault, WA.
1976	Report: "Use of Fabrics in Construction and Maintenance of Low-Volume Roads."
1977	FHWA republished 1976 report.
1979	*Retaining Wall Design Guide* published.

Table 2. - Wall classification (Driscoll, 1979).

Gravity	Anchored
1. Bin walls	1. Vertical culvert pipe
a. Rectangular	2. Horizontal sheet pile
b. Circular	3. H-Pile, timber lagged
c. Cross tied	4. Vertical sheet pile
2. Concrete crib	5. Stack sack
3. Timber crib	6. All gravity structures
4. Gabions	
5. Concrete gravity	
6. Concrete cantilever	
Reinforced backfill	**Cantilever piles**
1. Reinforced earth	1. Vertical sheet piles
2. Fabric	2. H-Pile, Timber lagged
3. Stack sack	

Table 3. - Standard and nonstandard wall designs (Driscoll, 1979).

Standard walls	Nonstandard walls
1. Bin walls	1. H-piles, lagged
a. Rectangular	2. Vertical culvert pipe
b. Circular	3. Geotextile
c. Cross tied	4. Horizontal sheet pile
2. Reinforced earth	5. Vertical sheet pile
3. Timber crib	6. Anchored (tied back)
4. Concrete crib	7. Stack sack
5. Gabions	
6. Concrete gravity	
7. Concrete cantilever	

Table 4. - Recommended factors of safety

	Bearing capacity	Overturning	Sliding at base	Slope stability
Normal highway loadings	2.0-3.0*	1.5-2.0*	1.5-2.0*	1.2-1.5
Occasional heavy Transient loading	1.5-2.0	1.2	1.2	1.2

* Upper range of factor of safety refers to silt and clay backfill or foundation soil.

Table 5. - Typical "local" soil used in structures (Keller, 1990).

Site	Wall type	USC[1]	% minus 200	PI[2]	Phi' deg	c' psf	Comments
Goat Hill	Welded wire	SM SC	21 20	5 8	34 31	200 300	Some face settlement
L. North Fork	Reinforced fill (1:1)	SM ML	38 55	2 3	34 33	100 150	Slight slope ravel
B. Longville	Welded wire	CL	50+	-	26	200	Poor foundation
Grave	Geotextile	SM	26	NP	35	850	Irregular face
Butt Valley	Tire-faced	SC	38	8	26	400	10% face settlement
Klamath	Timber-faced	SM	27	NP	30	0	Minimal settlement
Willamette	Wood chips+ geotextile	GP	0	NP	32	0	5% total settlement

Note: Phi' (internal angle of friction) and c' (cohesion) are from Consolidated-Undrained tests at 95 percent of T-99 Density. 1 psf = 0.0479 kN/m^2

1. Unified Soil Classification.
2. Plasticity Index.

Table 6. - Summary of alternative earth retaining structures (Keller, 1990).

Type of structure	Height (ft)	Cost* (per ft^2)	Advantages/Disadvantages
Reinforced fills	15-50	$5-$14	Less expensive than walls where they fit; slope typically 1:1; 1/2:1 slope with extra measures.
Tire-faced walls	10	$14-$20	Significant face settlement; visually questionable.
Timber-faced walls	18	$16-$22	Optimum wall considering cost, durability, and aesthetics; easy to construct.
Geotextile-faced walls	1-20	$15-$29	Temporary structures; irregular and non-durable face unless covered or treated.
Lightweight walls	28	$16-$25	Special geotextile walls suited for landslide terrain; moderate settlement with sawdust.
Chainlink fencing walls	22	$23-$29	Require a custom design; accommodates face settlement.
Welded wire walls	6-30	$23-$34	Most commonly built FS wall; good construction support from manufacture; standard designs available.
Block-faced walls	5-30	$14-$25	Masonry block facing; very aesthetic and durable; standard designs available; used for landscaping

Note: 1 ft = .3048 m; *1992 costs, typically including drainage, excavation, and backfill. Total wall cost can increase significantly depending on wall size, site difficulty, and other road repair work.

Table 7. - Names, addresses, and phone numbers of Forest Service authors

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Richard VanDyke	Geotechnical Engineer Westside Engineering Zone II Siskiyou National Forest 93976 Ocean Way Gold Beach OR 97444	(503) 247-7026
Michael Burke	Geotechnical Engineer San Juan National Forest 701 Camino Del Rio Room 301 Durango CO 81310	(303) 385-1271
Ron McNemar	Supervisory Civil Engineer Daniel Boone National Forest 100 Vaught Road Winchester KY 40391	(606) 745-3100
John Mohney	Pacific Northwest Region Regional Geotechnical Engineer 333 SW First Street Portland OR 97204-3440	(503) 326-2738

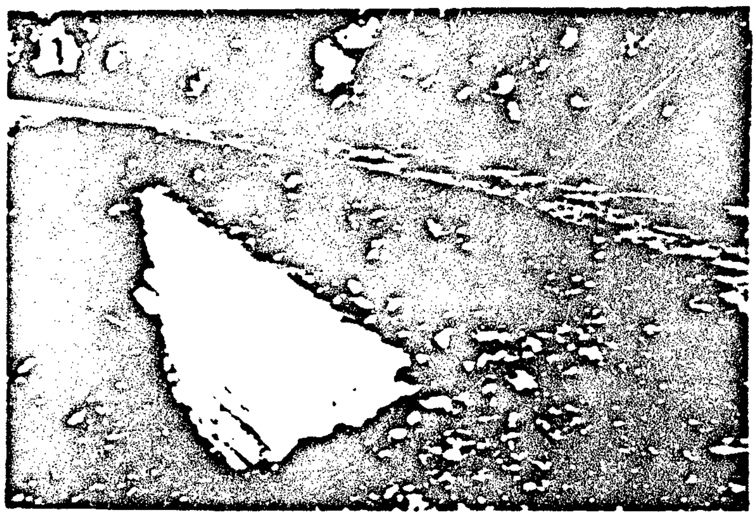

Figure 1. - The first geotextile wall in the United States before covering with gunite facing, Siskiyou National Forest, 1974.

Figure 2a. - The second geotextile reinforced wall built in the United States being coated with emulsified asphalt to prevent damage from ultraviolet radiation; Olympic National Forest, 1975.

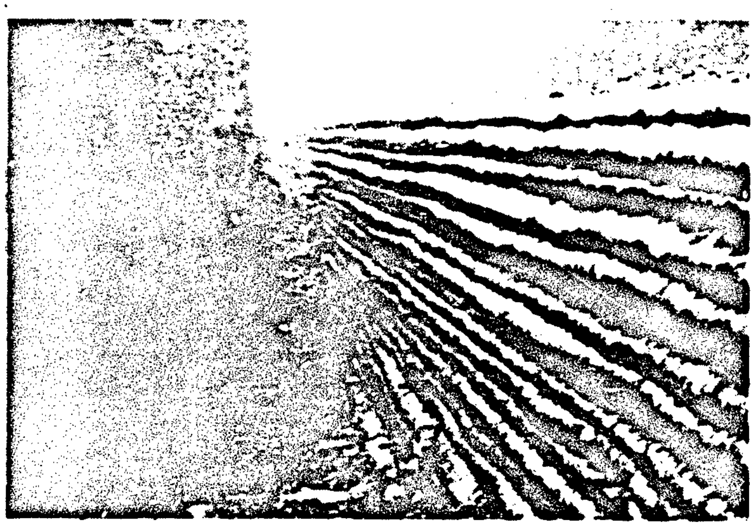

Figure 2b. - Completed wall with emulsified asphalt coating.

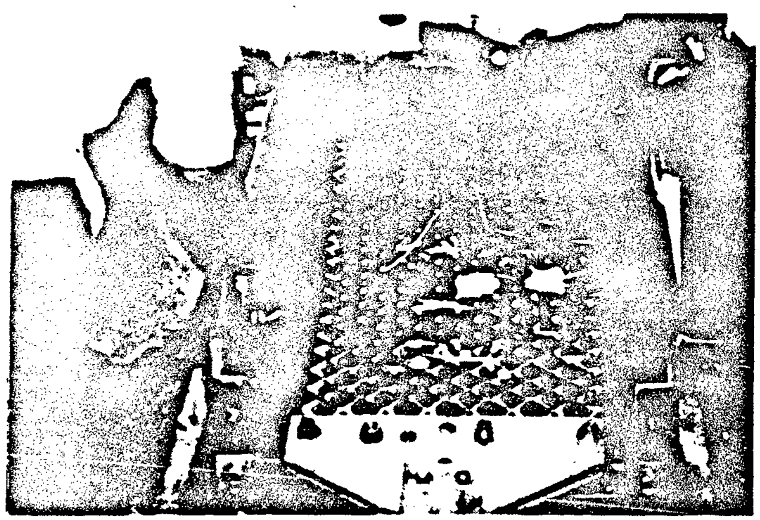

Figure 3. - Pullout test for metal chainlink material; about 1976.

Figure 4a. - Chainlink fabric reinforced wall under construction.

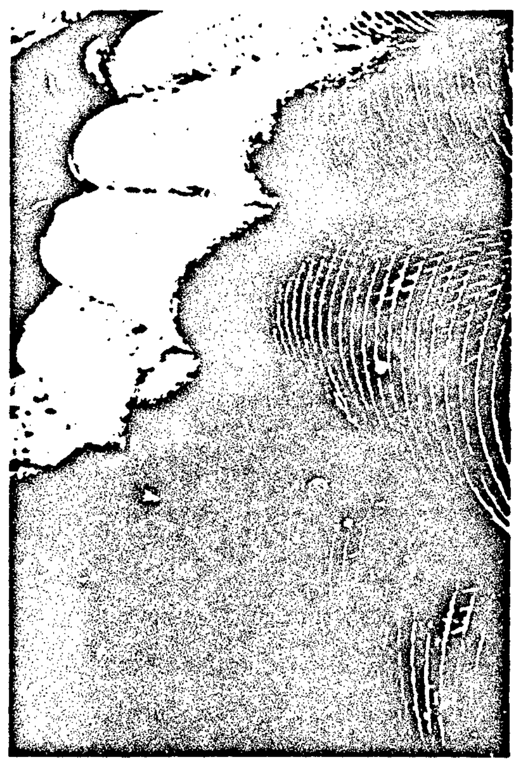

Figure 4b. - Closeup view of a completed chainlink fabric reinforced wall.

Figure 5a. - Unstable slope between two roads on the Plumas National Forest.

Figure 5b. - Slope in Figure 5a after repair using a welded wire wall.

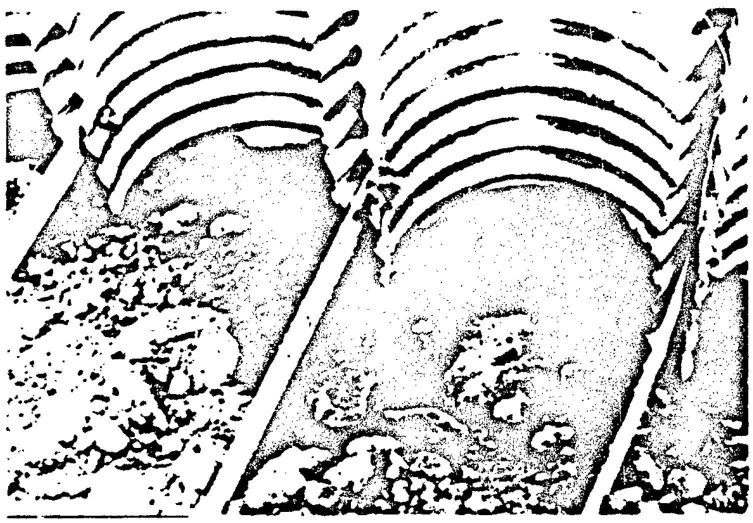

Figure 6a. - Closeup view of the connection inside the wall between the culvert pipe wall facing and the metal straps.

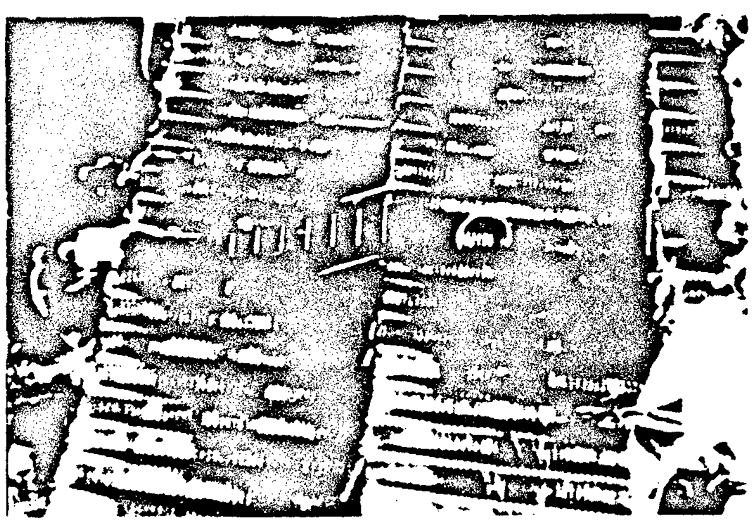

Figure 6b. - Placing instruments in the 60-ft-high wall at about the 40-ft-high stage; Siuslaw National Forest about 1974.

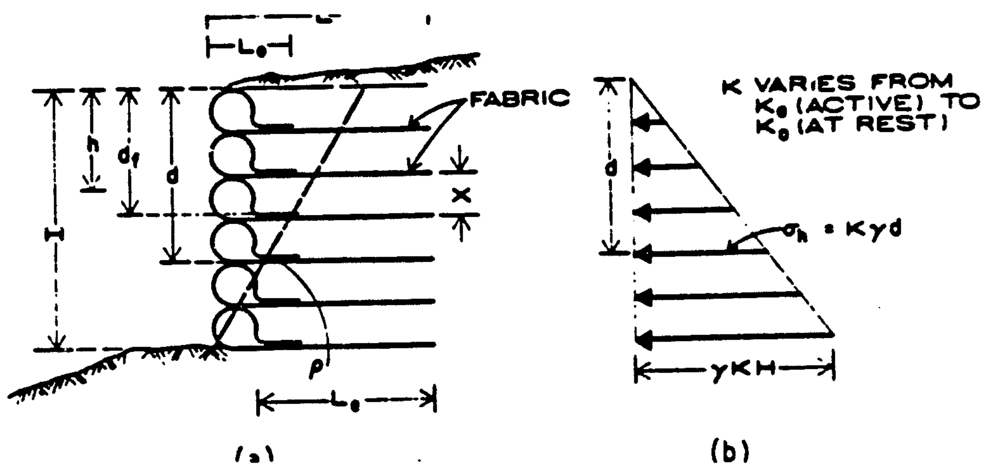

Figure 7. - Geotextile Wall–Earth pressure distribution (Driscoll, 1979, and Steward et al., 1977).

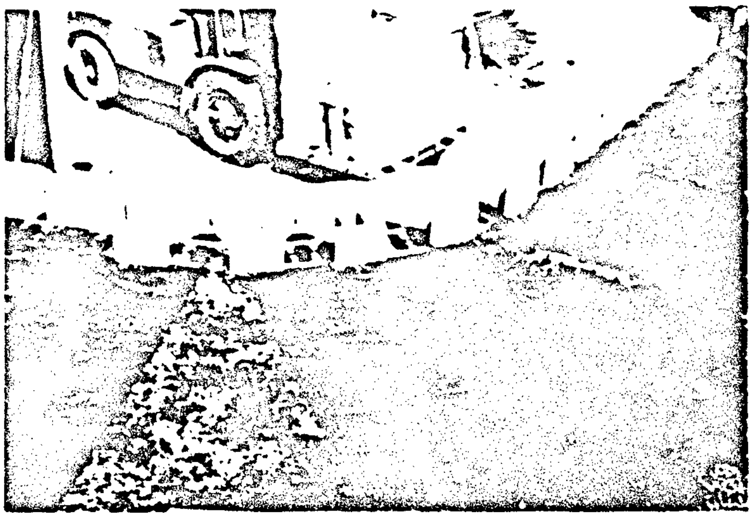

Figure 8a. - Top view of concrete block faced reinforced soil wall under construction.

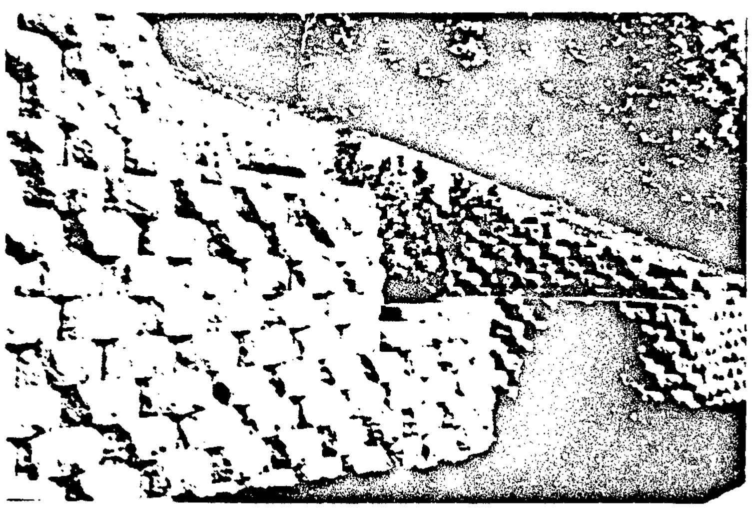

Figure 8b. - View of a completed concrete block faced reinforced soil wall; Siuslaw National Forest about 1990.

Figure 9. - Road repaired using a geotextile faced reinforced soil wall/slope; Siuslaw National Forest, 1987.

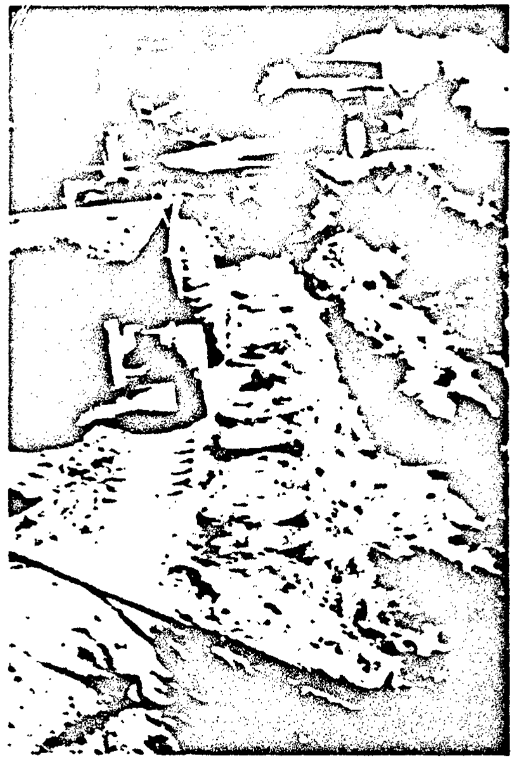

Figure 10a. - Placement of soil backfill in tire facing for tire-faced reinforced soil wall; Plumas National Forest, 1992.

Figure 10b. - View of the completed tire-faced wall.

Figure 11a. - Construction view of the wood-faced wall.

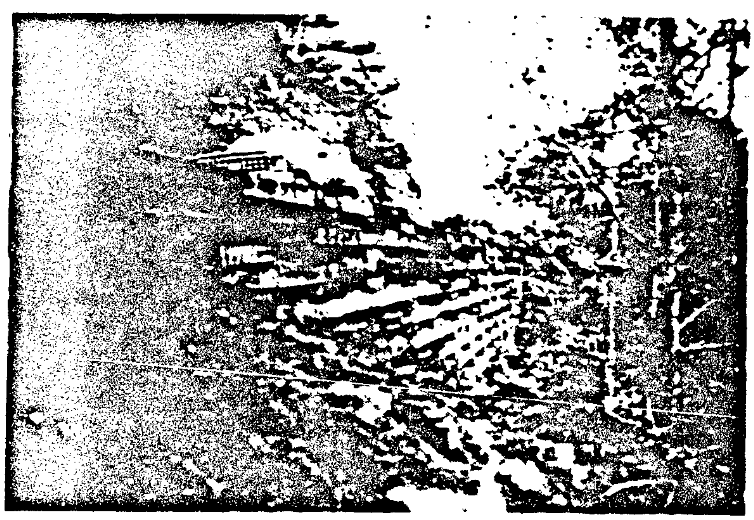

Figure 11b. - View of the completed wood-faced reinforced soil wall.

Figure 12a. - Workers placing internal reinforcement material.
Note: the geocomposite drain behind the backfill.

Figure 12b. - A view of a completed "manure-faced" wall after one growing season.

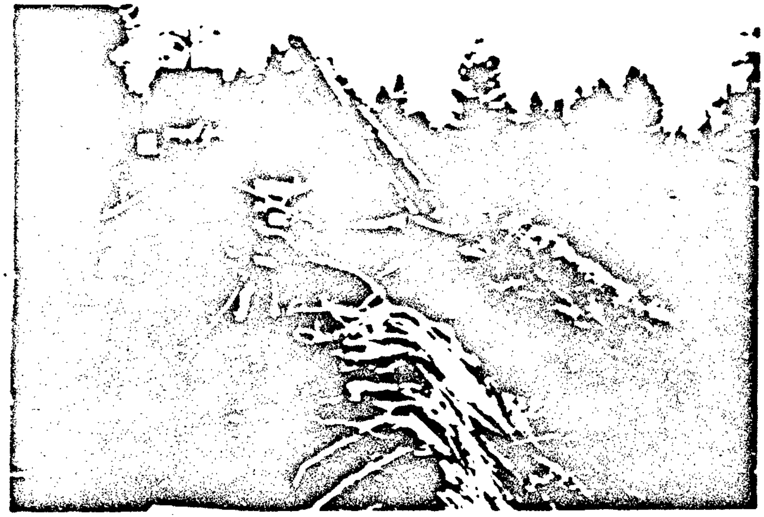

Figure 13a. - Placing fiberglass roving material on the face of a reinforced soil wall on the San Juan National Forest.

Figure 13b. - Internal reinforcement in place over the fiberglass roving material.

Figure 13c. - View of the completed wall.

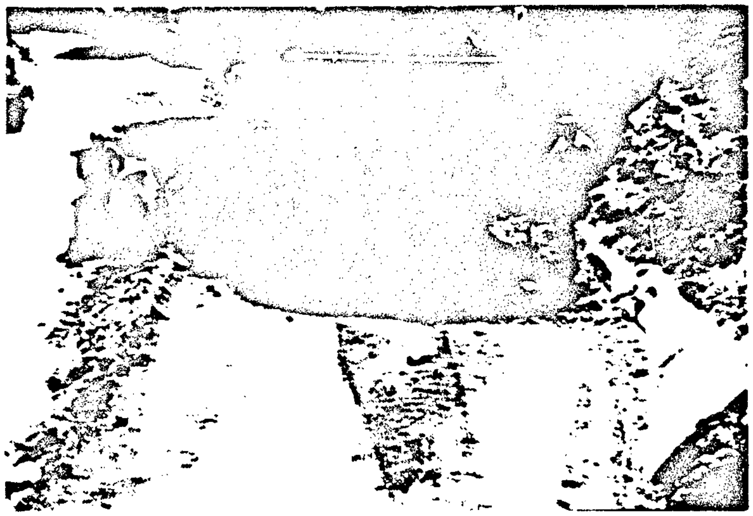

Figure 14a.- Placement of local soil backfill for a reinforced soil buttress on the Plumas National Forest. Note geocomposite drain on right side of photo.

Figure 14b. - View of the completed reinforced soil buttress.

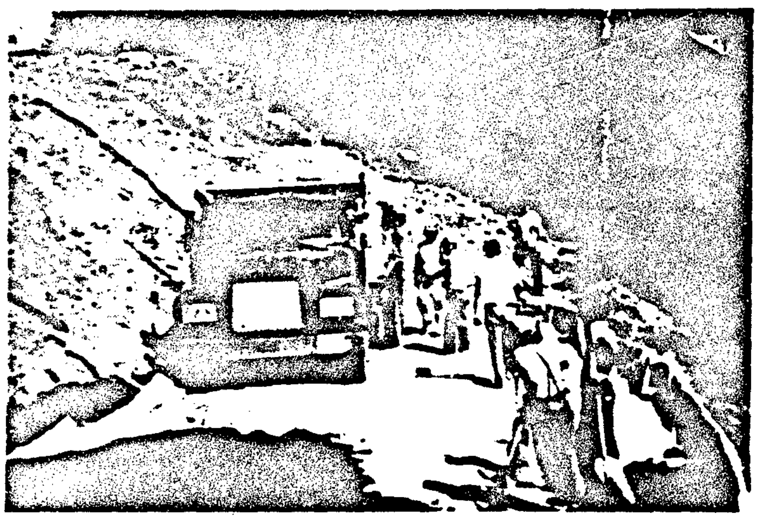

Figure 15.- Construction of a geotextile reinforced wall using lightweight wood materials for backfill.

PRINCIPLES, APPLICATIONS, AND TESTING OF GEOSYNTHETIC CLAY LINERS[1]

By W. E. Grube, Jr.[2]

Abstract: The history of manufactured Geosynthetic Clay Liners (GCL) is presented. The basis for use of bentonite clay, a pure sodium montmorillonite, in terms of this clay's chemical and physical properties is explained. Applications of GCL's as landfill liners, cover system barriers, pond liners, and tank farm secondary containments are discussed. Performance requirements include very low permeability, seam integrity, and uniform manufactured quality. Quality control areas of importance include raw materials, GCL production processes, product testing, and user acceptance testing. Research has intensified in recent years over the few hydraulic studies conducted 3 to 4 years ago. Engineering design parameters, liner/leachate compatibility, seam verification, and regulatory acceptance factors are areas of current agency and research attention. Twenty-eight references are included.

INTRODUCTION

Prefabricated liquid containment liners manufactured with a composite of bentonite clay and geosynthetic fabrics were originated in 1982 (Simpson, 1990). Sodium bentonite clay was incorporated because this mineral possesses a natural property of significant swelling upon hydration with water. Hydrated sodium bentonite has long been shown to exhibit extremely low permeability (Ran and Daemen, 1991). Granulated bentonite clay is confined within geotextiles, either mechanically, or with the aid of adhesives, as the prefabricated liner is produced into rolls several meters in width and tens of meters in length. This provides a uniform distribution of the water-sealing material, bentonite, over the area of the final manufactured product.

BACKGROUND

Following the introduction and commercial acceptance of the first product, several additional similar materials have entered the marketplace since 1982. This has resulted in civil engineers active in the geosynthetics industry recognizing them as "geosynthetic clay liners."

Commercially useful clays dominated by the mineral, montmorillonite, are commonly called "bentonite". Definitions of "bentonite," "montmorillonite," and related "smectite" have been somewhat interchanged by industrial users, but have been clearly stated by academic, industrial, and professional geologic nomenclature committees (Hosterman et al., 1985). Deposits of montmorillonite clay are found worldwide, ranging from a millimeter to about a meter in thickness. However, most of these are dominated by calcium or a mixture of cations on the active ion-exchange surfaces of the mineral. Although the montmorillonite clay mineral is found worldwide in both soils and concentrated in geologic deposits, its occurrence in bentonite is of particular commercial value. Bentonite clay is a naturally occurring geologic deposit, and is the primary commercial source of the clay mineral, montmorillonite

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(Ross and Hendricks, 1945). Wyoming bentonite is recognized worldwide as the purest sodium montmorillonite commercially available from natural geologic formations. These geologic deposits occur in a small region comprised of northeastern Wyoming, southeastern Montana, and northwestern South Dakota. Some of the clay beds consist almost entirely of sodium smectite (Elzea and Murray, 1990).

Numerous and extensive studies have documented the effectiveness of bentonite as an absorbent, buffer, barrier, or sealing agent against solute and liquid migration (Farmer and Tardy, 1990). The impact of swelling of soils with significant amounts of montmorillonite minerals has been extensively studied by both agricultural scientists and geotechnical engineers.

THEORY

Bentonite mineralogy, the physics and chemistry of its various reactions and properties, and application of both the pure and chemically amended clay have been extensively studied by academic and industrial scientists and engineers over many decades. The unique composition of this mineral results in a negative charge on its surface, allowing it to act as an ion-exchange medium for positively charged cations. In addition, when fully dispersed in water, the mineral disintegrates into very small particles, in the size range of ten-thousandths of a millimeter, resulting in a relatively large surface area per unit weight of material. The combination of large specific surface area and high cation exchange capacity is highly dominated by the sodium cation in Wyoming bentonite. This ion, in a aqueous environment, is surrounded by a more extensive cloud of water molecules than any other cation, such as calcium, magnesium, aluminum, etc. Water surrounding a particular ion in aqueous solution is called hydration. Thus sodium bentonite is more highly hydrated than other cation types. This extensive hydration separates individual montmorillonite mineral particles, filling microscopic void spaces and forming a uniform mass of numerous small mineral particles which present a restrictive obstacle to hydraulic flow. Physical chemists and soil mechanics scientists have referred to a sodium-rich bentonite mass as a "dispersed" system. A calcium-rich system wherein the clay particles are less hydrated and exist as agglomerations of individual mineral particles is referred to as a "flocculated" system (Mitchell, 1976). Clearly, a flocculated system consists of larger particulate masses, with concurrently larger void spaces among masses ("particles"), allowing more room for water flow, with resultant higher hydraulic conductivity than a sodium bentonite.

Sodium montmorillonite taken from a mine, dried and pulverized, readily disperses into a colloidal gel or suspension, depending upon the amount of water and degree of mixing applied. A suspension of 5 to 10 percent in water provides a density and viscosity which the petroleum industry has found to provide an ideal drilling mud. This use has consumed large quantities of bentonite clay for over 70 years (Hosterman et al., 1985). Similarly, the plasticity properties of bentonite were recognized in the 1920's, and led to its continuing use as a foundry sand binder (Hosterman et al., 1985). Literally hundreds of other commercial uses of smaller amounts have developed from this clay's unique physical and chemical properties (listed in bentonite vendors' literature).

In its purest form, this mineral will absorb nearly 5 times its weight in water, and swell to nearly 15 times its dry volume, forming a thick gel (of the consistency of ordinary toothpaste) which has a very low permeability to water. The physical chemistry of the mineral/solute/water interactions which take place to form a nearly impervious medium has been extensively studied and described (Williams and Daemen, 1987). The swelling and sealing property of bentonite is utilized in manufacturing geosynthetic clay liners as a uniform product which can serve the same hydraulic barrier purpose as a much larger thickness of engineered compacted soil.

Prefabricated bentonite clay liners were invented and first commercially introduced in 1982 as a substitute for bentonite/soil admixtures. For the soil to be amended and sealed, difficulties in uniform application, mixing, and quality control for applications of a 2 to 4 pounds of bentonite powder per square foot were overcome by this manufactured product. In 1986 the first geosynthetic clay liner product was incorporated into the primary liner system of a hazardous waste landfill.

APPLICATIONS OF GEOSYNTHETIC CLAY LINERS

Since its introduction, over 130 million ft^2 of GCL's have been installed. Four manufacturers have GCL products in the world market. These GCL's are being installed in the base of new landfills, in cover systems for landfill closures, as pond liners for a variety of water containments, as secondary containment structures at bulk liquid storage facilities, and to seal structural foundations against groundwater seepage.

LANDFILLS

Geosynthetic clay liners are finding rapidly increasing acceptance by civil engineers in the primary liner system of double composite liners required for hazardous waste landfills permitted under the United States Environmental Protection Agency's (USEPA) Subtitle C (Struve, 1990; Daniel and Koerner, 1991). Major advantages over several feet of compacted clay soil liners in this application include construction factors and absence of consolidation water.

Manufacturers' records show over 11 million ft^2 of GCL installed as solid waste landfill liners in the past several years. Recent passage of Subtitle D regulations by the USEPA includes allowance for liner system design alternatives to the standard design of a geomembrane overlying 2 ft of compacted clay liner. 40 CFR 258.40, Section 4.2.1(a) states that

> "New MSWLF units and lateral expansions shall be constructed: (1) In accordance with a design approved by the Director of an approved State or as specified in Section 258.40(e) for unapproved States. The design must ensure that the concentration values listed in Table 1 will not be exceeded in the uppermost aquifer at the relevant point of compliance as specified by the Director of an approved State under paragraph (d) of this section, or . . ."

Section 4.3.3 emphasizes the low hydraulic conductivity required to minimize leachate flow. It further presents numerous construction issues and quality control requirements essential to achieve an acceptable compacted clay soil liner. The manufacturing quality control applied to GCL production and testing, along with the very low hydraulic flow rates achievable with GCL's, are particular advantages to these materials in helping a MSWLF meet regulatory requirements. Economic and construction advantages have also been listed by other authors (Grube, 1991; Daniel and Koerner, 1991; Trauger, 1992).

A recent in-depth technical and economic analysis of 7 barrier layer options was reported (Koerner and Daniel, 1992). Every option which included a GCL outperformed those which included a compacted clay soil. These authors concluded that:

"• compacted clay liners by themselves are <u>not</u> the general barrier system of choice,
- geomembranes or geosynthetic clay liners are better overall choices technically and on the basis of benefit/cost ratio than compacted clay liners, and

- the barrier system of choice for most municipal solid waste landfill covers should be a geomembrane (GM) or a geomembrane over a geosynthetic clay liner (GM/GCL)."

FINAL COVER SYSTEMS

Final cover systems are recognized as essential hydraulic control structures to be included in closure of waste management sites. A major objective is to restrict infiltration of water into underlying waste. Other requirements include isolation of waste from surface exposures, control of gas release from the waste, and control of surface water runoff.

Reports by the U.S. Army Corps of Engineers (Bennett et al., 1991) to the U. S. Nuclear Regulatory Commission extensively describe the soil materials useful in soil cover systems. These authors recognize the effectiveness of manufactured geosynthetic materials, but point out the need for regulatory acceptance and technical issues important on a case-by-case basis. Manufacturers' records show that over 3.5-million square ft. of GCL have been installed as infiltration barriers in landfill cover systems. These installations include hazardous and solid waste landfills, mine tailings containing low-level radioactive wastes, ash disposal sites, and Superfund remediation sites containing specific waste types.

An important aspect of GCL incorporation into cover systems includes the demonstrated capability of a manufacturer to produce barrier products with particular geotextile properties to meet specific site design objectives. These may include tensile strength specifications to accommodate expected subsidence, porous fabrics designed for gas or vapor transmission, or clay adaptation to limit flow of gases or vapors. The production quality control available with GCL's has also been found to be more favorable than with compacted soil liners by some review agency offices.

POND LININGS

Geosynthetic clay liners have been installed as liners in industrial water and decorative ponds for many years. In these applications, most manufacturers recommend that compatibility tests be conducted during design. Such testing will ensure that chemically reactive solutes, especially wastes from industrial processes, will not interfere with the hydration and sealing of the bentonite clay in the GCL.

BULK LIQUID STORAGE FACILITIES

A rapidly growing application of GCL's is as secondary containment structures at bulk liquid storage facilities. This growth is based on successful installations of over 7 million ft^2. The petroleum and liquid agricultural fertilizer industries comprise the bulk of these applications. Testing by a soil corrosion engineering laboratory has shown that one type of hydrated GCL has a low electrical resistance, which allows operation of cathodic protection circuits installed with steel storage tanks. Compatibility studies have shown that the hydrated GCL maintains its very low permeability even against strong salt solutions and hydrocarbons for time periods much longer than are needed to contain a tank leakage until it can be properly cleaned up.

GCL PERFORMANCE REQUIREMENTS

The principal requirement for any type of liner, infiltration barrier, or waterproofing material is to restrict fluid and/or solute movement. For waste management landfills or similar facilities, there is a clear regulatory mandate to limit pollutant migration so that groundwater concentration is less than MCL (maximum concentration limit) at a specified monitoring point. Thus, a very low permeability, as measured by hydraulic conductivity through the barrier material, is needed. Data presented by both GCL manufacturers and independent testing laboratories confirm hydraulic conductivities as low as 10^{-10} cm/sec. Various published values range up to 100 times higher, depending on particular products and degree of confinement and consolidation of the GCL.

COMPOSITE LINERS

Currently accepted designs for landfill liner systems emphasize the composite liner---geomembrane underlain by several feet thickness of soil compacted to exhibit a hydraulic conductivity less than 1×10^{-7} cm/sec. The compacted soil liner serves primarily as an insurance backup to stop seepage from a compromised overlying geomembrane. Where an individual State's solid waste permit program has been approved under Section 4005 of RCRA, that State's permit officials may approve low permeability alternatives to several feet of compacted soil.

Use of GCL beneath a geomembrane presents several advantages to all parties concerned. Advantages and disadvantages have been listed (Grube and Daniel, 1991). Many of the disadvantages that were cited resulted from the youth of the GCL industry and consequent paucity of widely published performance data. Current GCL manufacturers, users, and academic research facilities are generating extensive databases with characteristic and performance information.

A current concern, both technical and regulatory, with respect to composite liner systems is the planar intimate contact between a geomembrane and underlying soil liner. Research studies have demonstrated that extensive lateral migration of seepage from a damaged geomembrane occurs unless the underlying soil liner directly contacts the geomembrane above. The flexibility of a GCL and the expansion of hydrated bentonite into the damaged area of an overlying geomembrane provide a much higher degree of intimate contact than is likely to be achieved with a compacted soil surface. This close connection of the two materials effectively prevents lateral flow of seepage from a damaged geomembrane above.

GCL SEAMS

A typical liner system may contain many miles of seams of geosynthetic materials. For a representative GCL, 13.5-ft width and 100-ft length, with 6-inch seam overlaps, one-acre of liner contains about 3/4 mile of seams. Thus, seam integrity is a major performance requirement. Seams between adjacent sheets of GCL are normally completed with a degree of sheet overlap specified by the manufacturer. Independent laboratory tests have shown that manufacturers' recommendations are either adequate or conservative. The primary seam integrity requirement consists of ensuring that the permeability of a seamed area is no greater than that of the GCL itself. The means of achieving seam integrity varies among GCL products, and each manufacturer's specifications should be followed.

Construction damage and other injuries by foreign objects is cause for continuing concern by waste management structure engineers. The presence of high quality bentonite in a GCL enables a self-healing quality, and sealing around penetrating objects, that is not possible with either geomembranes or compacted soil liners.

SPECIFICATIONS FOR GCL

GCL characteristics that remain popular with specification writers include physical dimensions. Although early GCL's were considered to be "1/4-inch thick," this dimension both disregarded geotextile contribution and included no tolerance. Likewise, the quantity of bentonite clay per unit area was claimed in early literature to be "one pound per square foot." No allowance for an interaction between bentonite quality and amount present was presented. Where a product specification includes a requirement for a specific quantity and quality of bentonite, the active ingredient in a GCL, it is easy to see the shortcomings of simple dimensional requirements. Current GCL product properties that accurately characterize the low-permeability material should be stated in terms of both quantity and quality of the actual bentonite.

GCL QUALITY CONTROL

A significant aspect of GCL manufacture, installation, and performance is a broad area of quality control. Both user customers and oversight regulatory agencies are continually increasing their requirements for testing and analytical certifications.

An example of a current GCL QC Program can be illustrated from the activities of a major manufacturer, based on evolutionary development over 10 years' production and installation of over 120 million ft^2 of GCL.

Raw materials

- Clay
  - Supplier lab testing
  - Supplier Certificate of Analysis (COA)
  - Manufacturer's lab verification testing
- Geosynthetic fabric(s)
  - Supplier lab testing
  - Supplier COA
  - Manufacturer's lab verification testing

Production

- Material application quality checks
- Controls on production equipment
- Inspection at production equipment material exit
- Inspection at rollup
- Labelling and warehousing
- Shipping, invoices
- GCL manufacturer's COA

Post-production QC

- Permeability
- Dimensions
- GCL swell

Clay properties

User QC

Permeability
GCL Swell
Dimensions
Fabric properties
Clay properties

RESEARCH

Daniel and Estornell (1991) summarized the proceedings of the first known meeting (June 7-8, 1990, in Cincinnati, Ohio) specifically organized to assemble technical information on GCL's. Researchers, design engineers, academicians, and regulatory agency participants contributed a broad array of experience and some data to this effort. A followup meeting has been scheduled for July 1992, with the goal of reporting extensive recent research and addressing newly recognized technical and regulatory issues. The results of this conference will be published by the USEPA's Office of Research and Development.

Examination of the historical studies into liner material performance shows the persistence of several issues which point to a site-specific data requirement.

Liner/leachate compatibility procedures for geomembrane and compacted soil liners have been developed and published following many years study by the USEPA. However, there remains little uniformity in data interpretation; with the judgment of individual permit approval officials providing decisions on a site-by-site basis. Manufacturers and users of GCL's have depended upon independent testing laboratories to provide data which confirm leachate/GCL compatibility as individual site questions arise. Since there has not been a long historic study of these materials by researchers with governmental support, the laboratory methods have not been well verified and prescribed in a uniform manner. Indeed, the test methods currently in use consist of a blend of approaches based on soils engineering and clay mineral characterization.

Engineering design parameters in use for other types of geosynthetic materials - geofabrics, geotextiles, geomembranes - are generally applied to GCL's directly. This application is significant because GCL's must be viewed as a simple binary system of materials, comprising the hydraulic barrier properties of a soil, with the structural properties largely governed by the geotexiles which enclose the bentonite clay. An important exception is the internal shear strength of hydrated bentonite in GCL's which have no connecting fibers between the sandwiching geotextiles. Thus, tensile strength in longitudinal and transverse directions, puncture resistance properties, and internal and interfacial shear resistance represent the primary GCL properties which an engineer must know to develop an adequate design which incorporates a GCL. Research areas which have not been developed include the adequacy of overlying and underlying materials to a GCL which may enhance or restrict the utility of the GCL.

Studies underway at a few universities are examining the extent and effects of in-plane flow, within the geotextiles which confine the bentonite clay. Since the various commercially available GCL's are manufactured with different fabrics enclosing the bentonite clay, different fabric properties among the GCL's can be expected.

Seam integrity with adjacent overlapping GCL's cannot be verified using techniques useful with geomembranes. In addition, the actual seaming technique varies among each manufacturer's product. Estornell and Daniel, 1992, reported on several manufacturers' product seam effectiveness using large steel tanks. Different overlap systems, all with a soil cover to simulate a full-scale installation were, flooded and permeability results collected. At least one manufacturer is conducting regular permeability testing of GCL seam overlaps. The Construction Quality Assurance program for each job site remains the major point of inspection and approval of the seams in GCL installations.

At the present time, only a few research institutes are known to be examining GCL's. These include Drexel University, University of Texas-Austin, University of Hanover-Germany, and several private geotechnical testing laboratories.

A Task Group has been formed within the American Society for Testing and Materials (ASTM), under Committee D-35 with support of Committee D-18, to develop a standard of practice for GCL's and to prioritize development of standard test methods for GCL's.

CONCLUSIONS

Wyoming Bentonite clay possesses a unique water-sealing capability that has been proven by both academic studies and industrial practice. Purer forms of this clay, containing higher percentage of sodium montmorillonite, are most effective, and most desirable when this clay is manufactured into blanket-type products for extensive field-scale installation.

Over 10 years of experience, with over 130 million ft^2 manufactured and installed, have proven the utility of this method of constructing a liner or water-sealing layer over large areas. The technical and economic superiority over compacted clay soil liners in meeting many regulatory and technical requirements are being increasingly recognized.

Selection of the appropriate GCL must be based on specified technical performance requirements. These include raw material properties, production quality control for product uniformity, product permeability and corollary properties, and manufacturer's quality certification.

Quality control must include all aspects of raw materials, production, testing, installation, and independent laboratory verifications. Relevant test methods need further development and standardization.

Hydraulic containment properties of GCL's are quite well established. Design and installation factors must be examined on a site-by-site basis, but general guidance for generic structures need to be provided. Seam integrity, in-plane hydraulic flow in geotextiles, and regulatory acceptability are areas of current active investigation.

As was concluded at a USEPA Workshop in 1990, developers of performance data must strive to publish their experience in widely available technical outlets.

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USING GEOGRIDS FOR OVERFLOW PROTECTION DURING CONSTRUCTION

By Wendell L. Schelb, P.E.[1]

Abstract: Geogrid used to protect and reinforce the downstream surface of an earth and rock embankment dam during construction. The dam is being constructed in four phases in a narrow mountain valley. Three of the four phases could experience overtopping by storm runoff from the 77-square-mile drainage area.

INTRODUCTION

Dam No. WV-3, Wheeling Creek Watershed, is a high hazard dam located on Dunkard Fork at its confluence with Wheeling Creek in the northern panhandle of West Virginia. Runoff from the 77-square-mile mountainous drainage area above the dam flows westerly out of Pennsylvania into Wheeling Creek and has caused extensive flood damage in the city of Wheeling, West Virginia. This is the last of seven dams built to control flooding in the city of Wheeling, West Virginia.

The 85-ft-high dam is a conventional earth and rock embankment with a primary (principal) spillway and an auxiliary (emergency) spillway. The top of dam is set at the elevation required to pass the runoff from a 24-hour duration, probable maximum precipitation (PMP) storm. The primary spillway consists of a reinforced concrete riser, a 84-inch-inside-diameter concrete pressure pipe, and a reinforced concrete impact basin outlet structure. The primary spillway controls the runoff from a 100-year frequency, 1 day/10-day storm event and regulates the outflow from the dam. The auxiliary spillway is an open channel excavated through rock in the left abutment which discharges into Wheeling Creek. The embankment consist of a central clay to sandy clay compacted earthfill zone with compacted rockfill zones upstream and downstream of the central earthfill zone. The rockfill zones consist of moderately weathered to unweathered siltstone and sandstone having a maximum stone size of 18 inches. The siltstone and sandstone rock is expected to break down some during hauling and placement resulting in a free draining rockfill with a large percent of the voids filled with sand and gravel size particles. The rockfill and earthfill zones are separated by a rockfill transition zone upstream and a granular chimney drain downstream.

A hydrologic and hydraulic analysis of the dam for the various phases of construction indicated that the partially constructed embankment has a high probability of being overtopped during construction or during the winter shutdown periods. Flow depths up to 5.0 ft over the embankment could be anticipated. Concerns for public safety in the downstream areas, for environmental concerns related to sediments that could result from erosion of the partially constructed embankment, and for the economic impact on the owners and contractors of a breach or damage to the embankment, prompted the planning and design staff to develop a design approach that would minimize downstream damages and economic impacts to the owner and contractor. The result was an innovative design using geogrid to protect and reinforce the downstream face of the embankment during the various construction phases.

Dam No. WV-3 is now in the second phase of construction. The total construction cost of the dam is $16 million.

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GEOGRID SPECIFICATIONS

A generic contract specification was developed for the geogrid to foster open competition in the marketplace. This specification described in general terms the environment in which the geogrid must perform and the expected performance ranges. The specification also listed specific salient features and properties that the geogrid must meet. The selection of these salient features was based upon the calculated minimum strength, the performance needed, and the environment in which it must perform. An excerpt from the specification for the salient features is in Appendix A.

The geogrid installed in the first phase of construction would be subjected to 3 years of exposure to the local climatic conditions (sunlight, freezing temperatures, hot temperatures, rain, snow, etc.). To minimize the reduction of strength and flexibility due to exposure to sunlight, a minimum of 2 percent carbon black was specified.

Installation of the geogrid could extend into cold weather periods. Flexibility of the geogrid material in cold temperatures was needed to allow it to be bent to a 22° inside angle and to be worked against without cracking or breaking. A copolymer polypropylene provided the best flexibility at cold temperatures and was specified.

The tensile strength of a geogrid is directly related to the amount of strain allowed in the geogrid during its operation. For this short-term application with exposure to the local climatic conditions, a strain of up to 5 percent was considered appropriate. For a calculated minimum tensile load of 608-lb-per-ft width at 5 percent strain in the geogrid, a secant modulus of 12,160-lb-per-ft width or greater was needed. Since 2 percent strain was most commonly tested at that time and most commonly cited in literature on the geogrids, a secant modulus at 2 percent strain was specified in the contract specification. This was based upon material available on the market at the time which could provided the minimum secant modulus at 5 percent strain needed for the work. The tensile strength of the geogrid can be determine for any strain; however, the most common strains tested presently are 2 and 5 percent. In retrospect, the geogrid should be specified either for the tensile strength at the allowable strain for the design or at the commonly tested strain (2 or 5 percent) that best defines the product needed.

To provide reinforcement of the embankment and resistance against pullout, the geogrid must interact with the soils in which it is placed. This interaction consists of the friction between the soil and geogrid and the ability of the soil to interlock with itself through the geogrid and with the geogrid. This interlocking with the soil is a function of the aperture (openings) of the geogrid. In this case, the geogrid had to interact with rockfill. To allow for some interlocking to take place, penetration of the rock through the geogrid must take place. The specified aperture size was based on manufacturing and material restrictions and engineering judgement.

GEOGRID DESIGN AND INSTALLATION

An analysis of the downstream 2.5:1 (2.5 ft horizontal to 1 ft vertical) face of the embankment during overflow events indicated that the rockfill zone could not withstand the flows without eroding and leading to a subsequent breach of the partially constructed embankment. A sudden release of impounded water resulting from a breach could cause flooding downstream and impose a hazard to property and life. Sediment resulting from such a breach was determine to be environmentally detrimental to the downstream areas.

A shared risk approach was considered the best alternative. To prevent erosion during anticipated overflow events and to reduce damages on very large storm events, the downstream face of the embankment must be protected during construction. This lead to the innovative approach of using geogrid to protect and reinforce the downstream slope of the embankment.

The design objectives were to optimize the construction sequence while keeping the length of unprotected slope to a minimum and reducing the time that any part of the downstream slope would be vulnerable to overflow damage. Progressive placement of five foot high geogrid capsules provided the best solution to meet all the objectives.

The geogrid must withstand the stresses (forces) imposed on the geogrid capsule during the construction period. Two conditions were analyzed.

The critical condition for the geogrid results from flow over the embankment and down the downstream face of the embankment. The critical forces on the geogrid occur where the overflow starts down the downstream slope of the embankment. These forces come from the drag of the water passing over the geogrid and from the momentum change in the flow as it accelerates from subcritical to supercritical flow on the downstream face of the embankment. The forces from the flowing water imposed on the geogrid can be determine by applying the impulse-momentum method of analysis as is done in determining the dynamic action of water jets in engineering mechanics. In this case, it was determined that the force acting along the geogrid would be 608-lb-per-ft width of the geogrid. From this force, the minimum tensile strength of the geogrid and the anchoring force needed to resist pullout of the geogrid can be determined.

A second condition considers water entering the rockfill and creating hydrostatic pressures on the face of the embankment where the internal flow surfaced. For the free draining downstream rockfill zone, it was determined that the rockfill with the anticipated internal water forces from overflow events would be stable within itself. Also, it would not impose forces on the geogrid greater than those forces imposed by the overflowing water on the surface.

Each 5-ft-high geogrid capsule must remain stable by itself immediately after completion of the capsule. No credit can be taken for subsequent capsules since they would be vulnerable to lose by erosion until completion. Anchoring resistance by interlocking of the rockfill through the geogrid could not be accurately determine without full scale testing. The minimum anchor length was determined by assuming only friction resistance between the geogrid and the rock fill with the active and passive forces on both sides of the geogrid. Based upon friction of adjacent materials only, a 5-ft length of geogrid placed on a 1:1 (45°) upstream side of the capsule and a 10-ft length at the downstream toe of the capsule was required to develop the anchoring strength needed to assure the geogrid would stay in place during an overflow event. The upstream anchor was secured by immediately placing a 14-ft-wide fill section against the upstream side of the geogrid capsule.

The top width of the capsule was adjusted to accommodate commonly manufactured widths of geogrid. By designing the capsule to utilize the geogrid width, the length of the rolls could be placed across the embankment. This reduced the geogrid end joints, reducing labor for placement and joining, and increased the efficiency of the installation.

A seven-step installation sequence was developed and placed in the contract drawings for the contractor to follow. Appendix B summarizes the seven-step installation sequence shown on the contract drawings.

In the last phase of construction when the risk of overtopping is no longer a concern, all visible geogrid will be removed from the embankment surfaces to allow for the embankment slopes to be vegetated. The geogrid exposed to the local climatic conditions is expected to ultimately break up and not be aesthetically pleasing and may interfere with routine maintenance. Since the rockfill will resemble a rocky earth slope and the dam and reservoir will have some secondary recreational benefits and be highly visible from a county road, vegetation of the embankment slopes was highly desirable.

SUMMARY AND CONCLUSIONS

Where site conditions are such that storm runoff can not be safely passed around or through an earth and/or rock embankment dam during construction and the probability of the embankment overtopping during construction is moderate to high, the design must address how the overtopping can be handled in a safe and environmentally sound manner during construction. The use of geogrids to reinforce the surface of the embankment affords a economical and efficient method of assuring the integrity of the embankment during construction for overflow events. Similar approaches have historically been used in the industry to protect embankments against overflow events during construction. These approaches have used wire mesh attached to metal anchor bars embedded in the embankment, blanketing the embankment with rock-filled wire baskets (gabions), covering the embankment with a butyl rubber membrane, and others.

The selection of the geogrid was based primarily upon availability of materials, ease of installation, ease of removal, and projected reliability. In the selection process, generalized costs of components were compared, but a detailed cost analysis was not made for cost comparison of alternative methods. The unit cost of the installation of the geogrid ranged from $3.60 to $10.00 per square yard (sy) in the bids received. The unit cost in the contract awarded was $5.00/sy for 30,000 square yards in the base period of the contract and $10.00/sy for 10,000 square yards in the option period of the contract.

Geogrid can afford a short-term protection system for use during construction. I do not recommend at this time that geogrids be used for long-term protection where they are exposed to sunlight or other detrimental climatic or environmental conditions.

Dam No. WV-3 is in the second phase of construction. Installation of the geogrid has progressed without any major problems. To date, there has not been a storm that has caused an overflow of the partially constructed embankment.

APPENDIX A.

EXCERPT FROM SPECIFICATION FOR SALIENT FEATURES

The geogrid shall conform to the requirements listed in Table No. 1.

TABLE 1

PROPERTY	TEST METHOD	UNITS	VALUE
Interlock			
aperture size[1]	I.D. Calipered[2]		
MD		in	1.8 (nom)
CMD		in	2.0 (nom)
open area	Measured[3]	%	75 (min)
thickness	ASTM D 1777-64		
ribs		in	0.07 (nom)
junctions		in	0.20 (nom)
Reinforcement			
flexural rigidity	ASTM D 1388-64		
MD		mg-cm	750,000 (min)
CMD		mg-cm	1,000,000 (min)
tensile modulus	GRI GG1-87[4]		
MD		lb/ft	21,500 (min)
CMD		lb/ft	22,500 (min)
junction strength	GRI GG2-87[5]		
MD		lb/ft	1,350 (min)
CMD		lb/ft	1,440 (min)
junction efficiency	GRI GG2-87[5]	%	90 (min)
Material			
copolymer polypropylene	ASTM D 4101 Group 2/Class 1/ Grade 1	%	97 (min)
carbon black	ASTM D 4218	%	2.0 (min)
Dimensions			
roll length		ft	41 (min)
roll width		ft	8 (min)

APPENDIX A.

EXCERPT FROM SPECIFICATION FOR SALIENT FEATURES

Notes:

1. MD dimension is along roll length. CMD dimension is across roll width.
2. Maximum inside dimension in each principal direction measured by calipers.
3. Percent open area measured as recommended by the manufacturer.
4. Secant modulus at 2% elongation measured by Geosynthetic Research Institute test method GGI-87 "Geogrid Tensile Strength". No offset allowances are made in calculating secant modulus.
5. Geogrid junction strength and junction efficiency measured by Geosynthetic Research Institute test method GG2-87 "Geogrid Junction Strength".

APPENDIX B.

SEVEN STEP CONSTRUCTION SEQUENCE
FOR THE
INSTALLATION OF THE GEOGRID
SUMMARIZED FROM THE DRAWINGS.

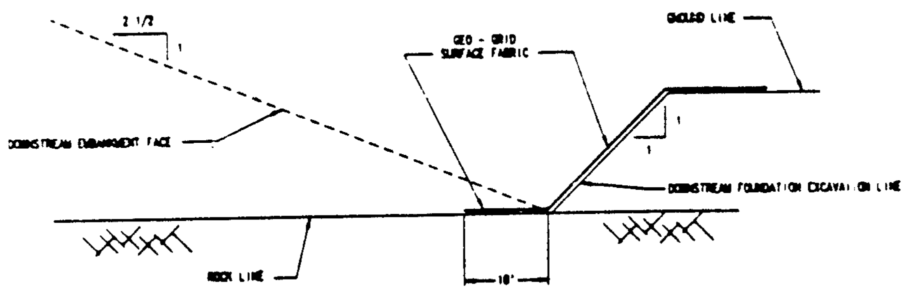
CONSTRUCTION DETAILS

1. The toe drain shall consist of Type III drain fill. The toe drain and Zone III shall be installed in accordance with Construction Specifications 24 and 25.

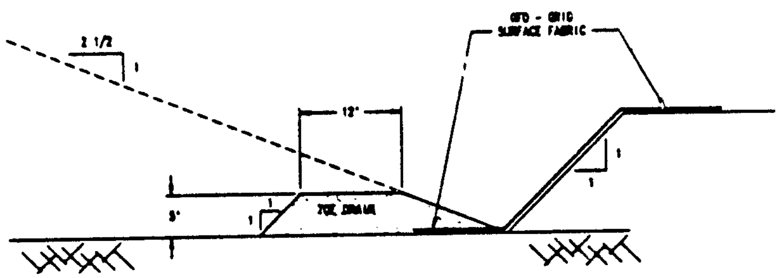
2. Construction Sequence
   Step 1 - Layout the fabric.
   Step 2 - Install toe drain.
   Step 3 - Pull surface fabric taunt over toe drain and anchor fabric.
   Step 4 - Place Zone III anchor increment to hold previous surface fabric increment. Layout surface fabric for next Zone III increment.
   Step 5 - Install Zone III increment.
   Step 6 - Pull surface fabric taunt over Zone III and anchor fabric.
   Step 7 - Place Zone III anchor increment to hold previous surface fabric increment. Layout surface fabric for next Zone III increment.

   Continue to install surface fabric and Zone III increments to Elevation 865.0 as shown on Phase 1, 2, and 3 drawings.

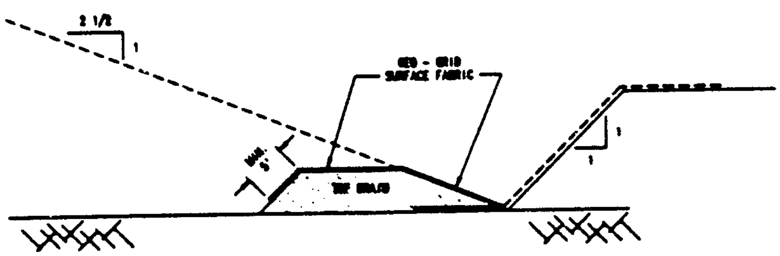
APPENDIX B. (Cont'd)



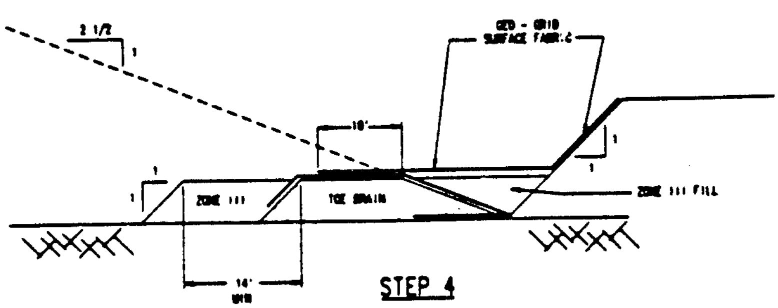
STEP 1



STEP 2

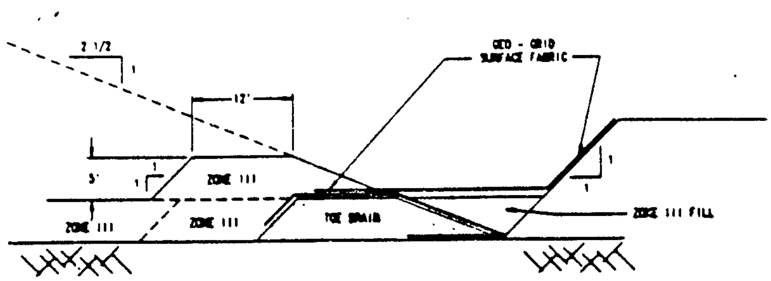


STEP 3

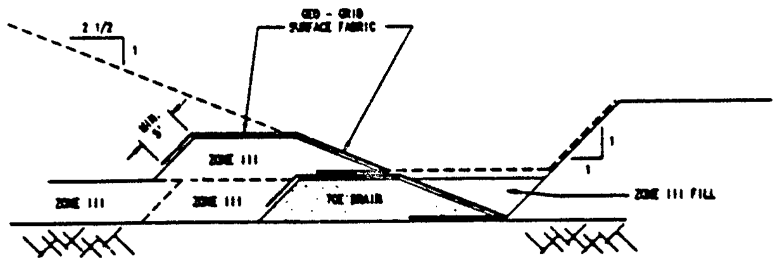


STEP 4

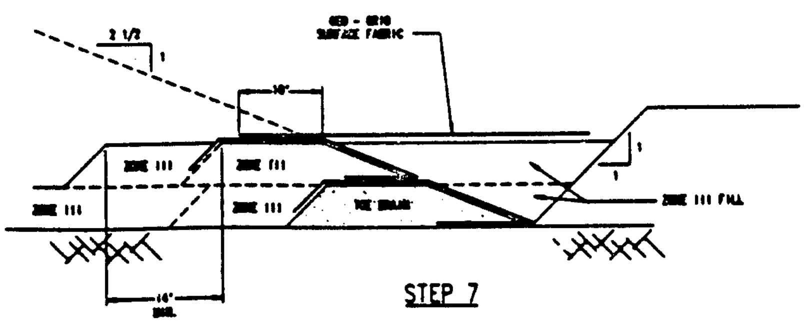
APPENDIX B. (Cont'd)



STEP 5



STEP 6



STEP 7

CURRENT RESEARCH ON GEOSYNTHETICS

By Barry R. Christopher, P.E.[1]
Robert D. Holtz, Jr., P.E., PhD[2]

INTRODUCTION

The use of geosynthetics in geotechnical engineering practice continues to develop and increase. Because of their performance advantages, improved economy, consistent properties, and ease of placement, geosynthetics have been successfully used to modify or even replace conventional geotechnical materials. Application areas include filters for drainage and erosion control systems, drainage media for drains, temporary armor and grass reinforcement for erosion control applications, stabilization and reinforcement of soft subgrade for road and railway construction, reinforcement of soils for construction of steep slopes and vertical retaining walls, and membranes for water retention and waste containment systems.

As the use of geosynthetics continues to grow, advancements are continually being made in the design, application, and materials technology. This paper presents a review of much of the current ongoing research on geosynthetics for the application areas given above. Implications of current research results on future directions and additional research needs are also discussed.

DRAINAGE AND FILTRATION APPLICATIONS

Background

The first application of geotextiles over 30 years ago was as filters for erosion control systems along the coast of Florida. Since those early days, geotextiles have been used successfully as filters in a vast number of applications and they have proven to be a cost effective and generally superior alternative to conventional graded granular filters. Even so, failures still occur and improvements in practice and materials are still possible.

The main challenges for improvement of current practice relates to three key areas (Christopher et al., 1992) required to advance the state of practice:

1. Design methods need to be theoretically rather than empirically based and less reliant on performance testing using site-specific soils.

2. Design methods should be applicable to any geotextile, soil, and hydraulic conditions.

3. Design methods should permit the optimization of materials to enable manufacturers to improve their products as required.

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Filtration Research

Organizations currently working in this or related areas include the Geosynthetics Research Institute (GRI), Syracuse University, Purdue University, University of Washington, Ecole Polytecnique (Montreal), and University of Illinois.

At GRI, four related programs are ongoing to evaluate filtration performance. The first program is a National Cooperative Highway Research Program sponsored field study to assess drain/geotextile filter systems and prefabricated geocomposite drainage systems in highway applications (GRI, 1992). To date, they have evaluated the performance of drainage systems at over 91 field sites throughout the United States. The work has included collection of both intact and disturbed samples of the geosynthetic along with upstream and downstream soils.

In a companion study the long-term performance of filters are being evaluated using long-term flow tests (GRI, 1992). The focus of the program is on the fine particulate clogging potential of four geotextile types (woven monofilament, nonwoven heat bonded, and light and heavy weight needle-punched nonwoven geotextiles) with four overlying conditions (sand, silt, silty sand mixture and geotextile alone). Up to 3000 hours of data have been obtained for each condition, and the results are currently being analyzed.

Another study underway at GRI is a reevaluation of the fine fraction filtration (F^3) test procedure for design (Sansone and Koerner, 1991). In this method the fine soil fractions are scalped from the soil, mixed into a slurry and allowed to flow though the geotextile. The test was developed some 15 years ago by CALTRANS (Hoover, 1982) but it has not commonly been used in practice. The test appears to provide rapid prescreening of geotextiles for either retention or clogging potential.

Another GRI study is focused on dynamic filtration evaluation in which fines are dynamically forced though the geotextile (Narejo and Koerner, 1991). The intent of the study is to develop a test appropriate for dynamic field conditions such as under faulted pavements.

A major study at Syracuse University is focused on an evaluation of the pore size characterization of geotextiles. This research involves comparing six methods of pore size characterization: dry sieving/apparent opening size (AOS) from ASTM D-4751; hydrodynamic sieving/filtration opening size (FOS) from the Canadian Standards method; wet sieving (D_w) from the Franzius Institute; and pore size and porosity distribution methods such as mercury intrusion, bubble point (capillary), and image analysis. The objectives of this study are to:

1. Describe what the various tests are actually measuring;
2. Compare measured values of opening size by the different methods;
3. Relate other geotextile properties to opening size; and,
4. Make conclusions as to which is the best method to use to determine the pore size.

The test evaluation phase of the Syracuse program is anticipated to be completed by the fall of 1992. Other phases will focus on the pore size and filtration performance relationships.

The Syracuse project is being done in cooperation with Ecole Polytecnique in Montreal, where extensive research on the FOS has been carried out in the past and work continues with an emphasis on actual field performance based on field exhumation studies.

Related research currently underway at the University of Washington is focused on the relationship between pore size distribution and filtration performance with the intent of establishing a more complete, theoretically supported design method. Initial work will evaluate the relations proposed by Fisher et al. (1990) and will utilize pore size characterization techniques such as Mercury intrusion porosimetry and image analysis. A related study involves hydraulic conductivity and long-term filtration tests on various geotextiles with local internally unstable glacial till soils commonly found in the Puget Sound region. This study should be completed by early 1993.

Geocomposite Drains

Geocomposite drains, consisting of a geotextile filter and an extruded polymeric drainage core, are under evaluation by several research groups.

The University of Illinois has for several years conducted research on geocomposites for pavement edge drains, focussing on the dynamic filtration evaluation of geotextiles. They have also recently completed a study of the relation between micro pore structure and filtration performance.

Already mentioned is the GRI field exhumation study, a portion of which is to evaluate geocomposite edge drains used for highway drainage retrofit projects. Initial findings emphasize the importance of proper drain installation, including placing the geotextile in intimate contact with the adjacent soil, as well as the necessity for an appropriate evaluation of geotextile filtration compatibility with the surrounding soil in order to predict effective performance.

ASTM is also currently working on test methods for evaluation of geocomposite core materials including long-term compressibility and hydraulic evaluation.

A field trial has been conducted by the Soil Conservation Services (SCS) in which a geocomposite drain was used for seepage control in an upstream watershed dam (Ryker et al., 1988). The geocomposite, consisting of a geotextile filter and geonet drainage core, was designed and installed as a combination embankment and foundation drain. Construction was completed in February, 1987. SCS is continually monitoring the project including periodic exhumation and evaluation of the geotextile filter and drainage core.

Erosion Control

The University of Illinois and the Corps of Engineers is currently evaluating the use of geotextiles with erosion control methods.

A study of the effectiveness of various temporary and semipermanent geosynthetic erosion control methods is underway by the Texas DOT.

GRI has completed construction of a laboratory rainfall/erosion simulator system, and they are currently evaluating a variety of geosynthetic erosion control systems under a range of conditions. (Weggel and Rustom, 1991).

ROADWAY STABILIZATION

Background

Geotextiles have been successfully used to stabilize soft subgrades for roadway construction for more than two decades. In this application the geotextile provides stabilization by both separating the base course aggregate from the subgrade, preventing material intermixing and promoting drainage of the subgrade and pore pressure dissipation during construction and under subsequent loading. The first successful use of textiles in roadways dates back to the 1920's when the Bureau of Public Roads and U.S. Department of Agriculture cooperated to install woven cotton fabrics in test sections in four States. Studies conducted by the U.S. Forest Service in the early 1970's verified the effectiveness of geotextiles as separators when they are correctly installed (i.e., without construction damage). Even so, there is a need for long-term performance data and prediction methods and there is considerable interest in improving roadway performance by reinforcement of the base course with geogrids.

Stabilization Research

Several research groups are currently working on roadway stabilization. In cooperation with the Washington State Department of Transportation (DOT), The University of Washington has two ongoing studies, both focused on the long-term performance evaluation of stabilization geotextiles. The first study consist of an extensive performance evaluation of geotextiles installed in permanent roadways which have been exhumed up to 15 years after installation. During Phase I, 8 sites in eastern Washington were studied, and up to 15 sites are planned for investigation in western Washington in Phase II.

The second study is a full-scale field test conducted on a State highway in Washington to evaluate the ability of different geotextiles to stabilize a soft subgrade during construction and to look at their influence on the long-term performance of the pavement. The site chosen for the test had a long history of poor performance. Five different geotextiles were selected based on their ability to survive construction and their drainage and filtration characteristics. The site was subdivided into 12 instrumented sections with two control or "no geotextile" sections. Different lift thicknesses were used during construction and construction trafficking tests were conducted. Long-term evaluation will be based on actual pavement performance, instrumentation monitoring, and exhumation of some specific test areas. The initial construction was completed in the spring of 1991 and long-term monitoring is planned for at least 5 years.

The U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory has also constructed a full-scale field test section to evaluate a different stabilization problem, frost heave. The test section has been designed to evaluate the ability of heavier weight nonwoven needlepunched geotextiles to reduce frost heave and other related problems. Previous laboratory studies have indicated that geotextiles made from hydrophobic polymers (i.e., polypropylene) can provide a capillary "break" to help mitigate frost heave and ice lens formation. The intent of the field study is to verify these laboratory results. The field section was constructed in state New Hampshire in cooperation with the New Hampshire DOT on a roadway that was under reconstruction due to significant frost damage. The test section is subdivided into 10 subsections with geotextiles located at different depths within the subgrade and 5 control or "no geotextile" areas. Subsurface instrumentation includes thermocouples, soil moisture sensors, water level monitoring stations and surface level survey. Construction of the test section was completed in the summer of 1991 and the site will be monitored for three freeze-thaw seasons.

The U.S. Army Corps of Engineers Waterways Experiment Station is performing research on the use of geogrids to reinforce and potentially reduce the base course thickness. A literature review and test section design have been completed and are reported in FAA report number DOT/FAA/RD-90/28. The test section has been constructed and full-scale traffic tests have been performed to validate the geogrid base reinforcement potential for flexible pavements used to support light aircraft traffic. The report is anticipated to be published by the FAA later this year.

REINFORCEMENT APPLICATIONS

Background

One of the most exciting application areas is the use of geosynthetics as reinforcing elements to improve the structural characteristics of the soil. Reinforcement enables engineers to build embankments over extremely soft foundations. Steep slopes and even vertical retaining structures can be reinforced with geosynthetics, and these are usually significant cheaper than conventional unreinforced, flat slopes, and structural retaining walls.

In connection with the necessity to build dikes for flood control and dredged spoil retention, the U.S. Army Crops of Engineers probably has done the most research and development work on geosynthetic-reinforced embankments on very soft foundations. We have learned much from their willingness to publish their early failures, their design methods, and their recommended construction procedures. Corps research in this area has been in cooperation with GRI, The University of Delaware and Oklahoma State University, among others. Other groups working on reinforced embankments include the University of Western Ontario (UWO), Purdue University, and the University of Maine. This latter work has involved primarily analytic work, although some (UWO) have carried out full scale instrumented field tests.

For reinforced slopes and retaining walls, a major research effort has been the Federal Highway Administration (FHA) project "Behavior of Reinforced Soil," which began in the early 1980's. The initial focus was to unify the design approaches for reinforced slopes and retaining walls such that they could be incorporated into standard practice. The initial part of the study was an extensive literature review published as NCHRP Report 290 (Mitchell and Villet, 1987). This report formed the basis for an experimental program consisting of small and field scale static models, numerical models, and small- and large-scale centrifuge models. The work was completed in 1990 with the publication of a design manual (Christopher et al., 1990).

Research on Reinforced Embankments Over Soft Foundations

The U.S. Corps of Engineers are continuing their efforts to expand applications in this area. The Waterways Experiment Station is currently reevaluating the use of hydraulically filled geosynthetic tubes. The tubes are filled with sand or dredge spoil by pumping methods and range in length from 100 to 1000 ft with a typical circumference of 24 to 32 ft. The tubes used in the current evaluation program are manufactured from high strength woven geotextiles and, in one case, lined with a needle punched nonwoven. Principle application areas under investigation include: dredge containment dikes using the dredge spoil for dike construction, minimizing deployment problems for construction of embankments over soft foundations, coastal erosion control and construction of jetties and levies.

Research on Reinforced Slopes and Walls

Much ongoing research in this area appears to be focused on improved design methods and characterization of geosynthetic reinforcement properties. Although polymers are generally recognized as being very durable, we simply do not have sufficient long-term experience as to their durability for use in applications where project design lives often exceed 75 years. The FHA and cooperation with a group of State DOT's has a comprehensive study underway to evaluate the durability and life expectancy of reinforcing geosynthetics. The work is divided into four main complementary thrusts:

1. Location and recovery of field aged geosynthetic samples from actual structures and the characterization of their exhumed conditions and chemical/stress regimes.

2. Concurrently, unaged samples from manufacturers covering the broad range of commercially available reinforcing products will be characterized fully, to provide baseline conditions.

3. Development of laboratory protocols for testing of representative samples for aging, attributed to oxidative or hydrolytic processes. The idea is to be able to extrapolate short-term data to the required in-service life expectancy requirements.

4. Establish a confined stress-strain test procedure to be used for the other phases of the work.

The research, which should be completed by 1995, is anticipated to provide design engineers with credible methods for specifying and predicting the long-term performance of geosynthetic reinforcement.

One of the two essential design input parameters is the pullout resistance of the reinforcement. Pullout research is underway at several places including Louisiana State University, University of British Columbia, and the Geosynthetics Research Institute. ASTM also has developed a draft standard for pullout evaluation.

Standard techniques have not yet been established to accurately determine the in-soil tensile strength and modulus of geosynthetics in soil. Several institutions are currently working on developing laboratory test methods to determine the confined stress-strain characteristics of geosynthetics; these include the Massachusetts Institute of Technology, the Geosynthetics Research Institute, University of Washington, University of Colorado at Denver, and Brooklyn Polytechnic University. GRI has developed a standard procedure for confined tension evaluation of geotextiles. This method along with several others are also being considered in the FHA durability study mentioned above.

Recent research at the University of Colorado at Denver has involved a numerical model study of the influence of reinforcement deformation response on reinforced wall performance. A group at the University of California at Berkeley is working on the influence of differential settlement on reinforced structures.

Several studies are ongoing to evaluate reinforcement of marginal soils. For example, the University of California at Berkeley is evaluating the influence of drainage and pore pressure relief in conjunction with reinforcement for improving the slope stability of high water content soils. The work consist of a theoretical review of drainage requirements, and the influence of reinforcements with enplane drainage potential. Also under investigation are the design modifications necessary to include the influence of in soil drainage and associated pore pressure dissipation (e.g. nonwoven needle punched geotextiles).

In a similar study, The University of Maryland is continuing their work using centrifuge models to evaluate the use of nonwoven geotextiles to reinforce cohesive soil backfills. Their latest work focuses on the effects on wall stability of different foundation soils, of different wall face batter angles, of the addition of lime to the cohesive soil, of pore pressure dissipation through the geotextile and of the length of the reinforcement. The ultimate outcome will be the development of simple analytical tools for the design of retaining walls and slopes in areas where granular soils are not readily available. As part of the research program, a full scale structure is planned to fully evaluate the proposed design technique.

Two large-scale instrumented model walls using both cohesive and granular backfill and nonwoven geotextile reinforcement have been load tested at the University of Colorado at Denver as part of an international prediction symposium. Fifteen researchers and engineers, in advance of loading, made predictions of strains in the reinforcement, lateral earth pressure on the timber facing, and the short-term and creep deformation of the face when the walls were surcharged and loaded to failure. Proceedings of the Symposium will be published later this year.

CONTAINMENT APPLICATIONS

The fastest growing application of geosynthetics is in waste containment systems. The growing value of containment space has led to the incorporation of geosynthetics as the standard of practice for the design of such systems. Geosynthetics find application within all three components of a waste containment system, the liner, the waste itself and the cap as follows:

Containment Liner System

- Site stabilization and access for construction of the liner.
- Gas venting to prevent natural gas from collecting beneath the liner.
- Primary and secondary geomembrane liners
- Protection of the liner from potential puncture damage.
- Intermittent drainage in leak detection between geomembrane sheets.
- Leachate collection system drains and filters.

Within Body of Waste

- Daily cover materials during placement of waste.
- Gas collection within between intermediate layers of waste.

Caps and Covers

- Stabilization of soft or otherwise unstable landfill mass to facilitate cap construction and minimize subsidence related problems.
- Geocomposite gas vent layers.
- Geotextile gas transfer media for low gas production landfills.
- Pore water dissipation and friction improvement between geomembranes and cohesive soil.
- Primary geomembrane liner.
- Protection of geomembrane liner from potential puncture damage.
- Geocomposite drainage layers above the hydraulic barrier.
- Separators between biotic barriers and surface topsoil layers.
- Filters beneath erosion control armor stone on the cap surface and in drainage swell and spillways.
- Reinforced grass for improved erosion control.

Along with the proliferation of applications, there is an equal proliferation of research which is too numerous to cover in detail in this paper. The research tends to focus on four broad areas:

1. Durability of geomembranes and the geosynthetic components;
2. Leak prevention including puncture protection of the liner using geotextiles;
3. Leachate control including influence of biological activity on long-term filtration performance;
4. Design evaluation.

The Geosynthetic Research Institute in cooperation with the US EPA currently have research ongoing in all of the areas mentioned above. The specific activities at GRI were published in the June issue of Geotechnical News (Koerner, 1992).

PUBLICATIONS AND INFORMATION

As most of the research we have just described is currently ongoing, it is expected that specific results and conclusions will be available in the near future in the usual geotechnical and geosynthetics journals and conference proceedings.

Common journals are the ASCE *Journal of Geotechnical and GeoEnvironmental Engineering*, *Canadian Geotechnical Journal*, *Geotechnical Fabrics Report*, the ASTM *Geotechnical Testing Journal* and the IGS journal *Geotextiles and Geomembranes*. The next (biennial) North American geosynthetics conference is *Geosynthetics '93* which will be in Vancouver, B.C. The next international geosynthetics conference will be in Singapore in 1994.

THE FUTURE?

May we make some predictions about the use of geosynthetics in the next few decade.

1. The use of geosynthetics in civil engineering will continue to increase, especially in transportation and waste containment applications.

2. Graded granular filters will become the exception and geotextile filters will become the accepted standard of practice in drainage and erosion control systems.

3. Embankment slopes and retaining structures reinforced with geosynthetics will become the standard rather than the exception, because of economics, esthetics, and seismic resistance.

4. Future geotechnical testing laboratories will also have the capability for testing geosynthetics, as required for design and construction.

5. Analysis for reliability and risk assessment will be developed and standardized for all critical reinforced soil structures and waste containment systems.

6. Specialty geocomposite and related products will continue to be developed to take advantage of the many unique properties and advantages of geosynthetics.

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TECHNICAL SESSION 4

Erosion Control

BASIC MECHANISMS OF EROSION

by Peter J. Bosscher[1]

DEFINITIONS

Soil erosion is the removal of surface layers by the agencies of wind, water, and ice. Soil erosion involves a process of both particle detachment, entrainment, and transport. Erosion is initiated by drag, impact, or tractive forces acting on individual particles of soil at the surface.

NATURE OF SOIL EROSION

The major agents producing erosion or degradation of soil are water, wind, ice, and gravity with the two most common agents being water and wind. This paper will focus on describing water and wind erosion. These agents act on areas affected by manmade activity to create sediments that pollute streams and fill reservoirs. Valuable plant nutrients are also lost with erosion. The amount of soil erosion currently occurring in the United States is significantly higher than occurred during the 1930s even though government subsidies, educational efforts, and erosion practices have been instigated.

PART I. WATER EROSION

The water erosion process involves the detachment, entrainment, transport, and deposition of soil particles or aggregates as they move through the water system. We often think of this process as beginning on upslope land and terminating in a lake or other water body. Actually the cycle may take place many times as soil moves from a point of origin to ultimate deposition. As the soil moves through the system, the eroded material may cause triple damages. First, it may reduce the value and utility of the originating land. Second, the eroded material may cause a reduction in water quality as it moves through the system, and third the sediment laden flow may damage the transport route and/or the deposition site. Loss to the originating land is referred to as on-site damages; while damages that occur while in transit or at the point of deposition are commonly referred to as off-site damages.

OVERVIEW

Detachment

Let us look at each of the subprocesses separately. Detachment of erodible sized particles from the soil mass may occur by rain drop impact or by shear of flowing water. Rainfall in Wisconsin may impact soil with energy levels as great as 60 million ft-pounds per acre each year. Unless this energy is absorbed before it strikes the soil surface, large quantities of soil can be dislodged from the soil mass. Therefore, soil detachment due to raindrop impact is often appreciable. The second method of detachment is the shear of the runoff water. Whenever the shear force becomes large enough to overcome the cohesive, friction and gravity forces that hold the soil in place, aggregates will break from the soil mass. The relative importance of the raindrop detachment and the shear detachment depends upon many things. Detachment

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by drops is a function of the drop characteristics, soil characteristics, and surface condition. Detachment by shear is primarily a function of soil resistance and flow characteristics.

Entrainment

Entrainment is the process whereby detached particles of an erodible size are picked up by the transporting water. Erosion initiated by excessive shear usually results in immediate entrainment. Particles sheared from the soil mass immediately enter the transport phase. This is not necessarily true of particles detached by raindrop impact. Such particles may be dislodged from the soil mass, splashed into the air, and then fall back to the soil surface, thereby moving only a short distance. The detached particles may lay on the surface until they are splashed into the air again by another raindrop. This process may be repeated many times until particles fall into an area with sufficient flow to begin the transport process or until a sufficient quantity of overland flow occurs to begin moving the particle.

Transport

The third phase of the erosion process is the transport phase. This is the movement of the soil particles primarily due to runoff water. Transport should be divided into two phases. The first is the inter-rill stage; the second, the rill and gully stage. In the inter-rill phase, the flow is primarily that of a thin film of water flowing overland. The carrying capacity of this thin film flow is limited and usually insufficient to shear particles. It may have sufficient energy to move particles previously dislodged by raindrops. Thus, detachment by raindrops combined with transport by inter-rill flow results in relatively uniform soil removal from the soil surface.

As the water begins to move down the slope, it tends to concentrate and reaches a point where the velocity is large enough to begin shearing soil particles from the soil mass. Rills begin to develop. Rills are small channels on a hillside where the water has concentrated and begins to detach soil due to increasing shear. Rills begin to develop whenever water becomes channelized. The rate of channelization depends on factors such as variations in soil type, topographic conditions, tillage marks, and random irregularities in the soil surface due to micro-topographic changes. Once the rill begins, it tends to increase in size due to increasing flow concentration and often progresses upslope due to headcutting.

A rather artificial distinction exists between rills and gullies. In general, we have accepted the definition that rills may be removed by normal tillage methods, and that gullies are channels that are of such a size that they can not be easily crossed by equipment or removed by tillage operations. The development process of rills and gullies are much the same. Gullies develop by more massive soil movement due to the greater concentration of runoff. Gullies may have additional soil loss due to slumping of sidewalls. Often excess moisture in the gully walls weakens the soil structure and causes soil to slide into the bottom of the gully where it is removed by subsequent runoff. Gullies may also proceed upslope through headcuts.

Deposition

Deposition of soil may occur at almost any point along the flow path. Deposition occurs whenever the carrying capacity of the water is reduced to the point where it cannot carry the entire sediment load. This decreasing carrying capacity is often due to a decrease in slope steepness or perhaps an increase in vegetative cover. Either reduces the velocity of flow and decrease its ability to carry sediment. Deposition is a very selective process with the larger, more dense particles and aggregates being deposited first and the finer particles carried further through the flow system. Particles and stable aggregates which are sand size and larger will settle quite rapidly as the carrying capacity is reduced. On the other hand, clay will remain in suspension for a long period of time even in relatively quiet water. This property of deposition is important as we think about conservation practices designed to reduce sediment load by trapping.

PRINCIPLES OF CONTROL

It is important to recognize that the amount of soil eroded from a given area is dependent upon two factors. The first is the availability of particles of a transportable size. If erodible sized particles are not available the soil will not erode. Some conservation practices are designed to minimize the amount of detachment that takes place, thereby reducing the material available for transport. The second factor is the transport capacity of the runoff water. If the velocity of the runoff can be maintained at a small enough level and the amount of water in a concentrated area can be minimized, the carrying capacity of the water will not be sufficient to transport available material. In this case, particles may be detached but transport capacity will limit erosion. These two points are very important in considering erosion control practices. Erosion control can be accomplished by controlling both or either of these factors.

First, the soil can be protected to inhibit the rate of breakdown of soil structure. This limits the amount of soil available for transport. Second we can design measures which prevent or minimize the concentration of water, thereby keeping flow quantities and velocities low. This reduces the ability of the water to transport soil.

DETERMENANTS OF RAINFALL EROSION

In 1958, Wischmeier and Smith published the Universal Soil Loss Equation (USLE) which has served as a guide for predicting long term erosion rates from upland areas. Since that time many refinements have been made to the equation; however, it still stands as the most widely used equation for erosion prediction. A Revised Universal Soil Loss Equation (RUSLE) has recently been published as well as another predictive tool, the Water Erosion Prediction Project (WEPP) procedure. These two methods have also been computerized and though they are more precise in their predictions than the USLE, each method also requires additional data which may need to be estimated.

For our purposes, the USLE equation will be used as a basis for describing the effects of controlling parameters on erosion rates. Wischmeier and Smith identified six parameters for predicting erosion. These six factors may be lumped into four - Rainfall Erosivity Factor (R), Soil Erodibility Factor (K), Topographic Factor (LS), and a Cover-Management- Practice Factor (CP). First, each of these factors will be examined to see how they influence soil erosion. An example of the use of the Universal Soil Loss Equation to predict average annual erosion rates will then be described.

Rainfall Erosivity Factor (R)

Wischmeier and Smith demonstrated that the single most important factor in describing the erosive power of a rainstorm is the product of the rainfall energy and the maximum 30-minute intensity for the storm. The product of these two terms divided by 100 has been designated as the Rainfall Erosivity Factor. While R values for individual storms and years and various return periods (Table 1) are available, average annual values are more readily available and most useful for predictive purposes (Figure 1).

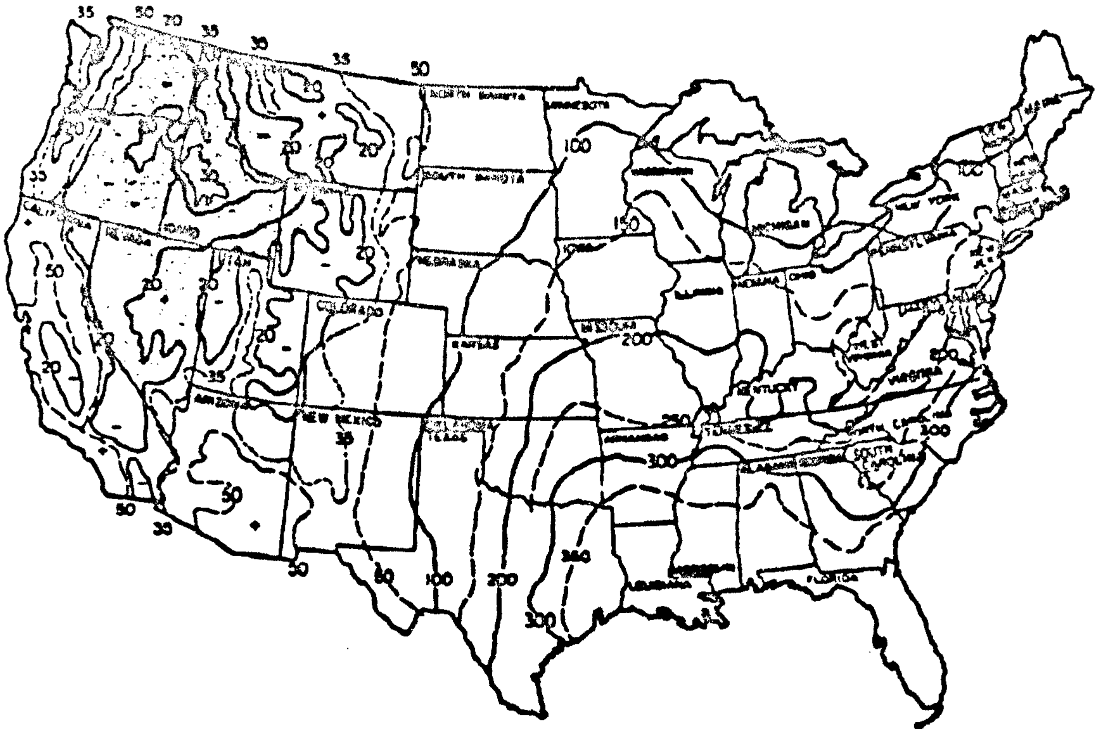

Figure 1. - Rainfall and Runoff Erosivity Index, R, by geographic location (from USDA and EPA, 1975)

As we shall see in the discussion of the cover-management-practice factor, the distribution of the erosivity factor throughout the year (Figure 2) is also important in determining expected erosion rates. If periods of high erosivity correspond to times of the year when soils are without cover, large erosion rates can be expected; however, when similar period of high rainfall erosivity occur and soils are well covered with vegetation, erosion rates will be less.

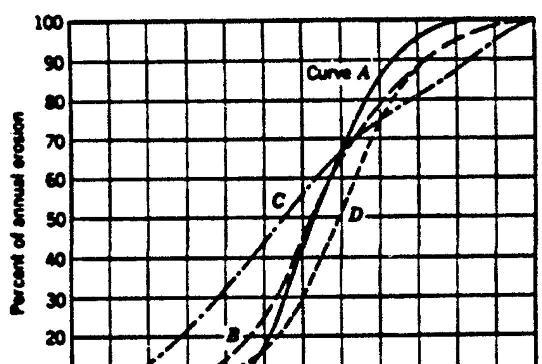

Figure 2. - Monthly distribution of the rainfall and runoff erosivity index: Curve A, northwestern Iowa, northern Nebraska, southeastern South Dakota. Curve B, northern Missouri, and central Illinois, Indiana, and Ohio. Curve C, Louisiana, Mississippi, western Tennessee, and eastern Arkansas. Curve D, Atlantic Coastal Plains of Georgia and the Carolinas. (Redrawn from Smith and Wischmeier, 1962.)

Table 1. - Examples of the range of annual values and single storm R values.

Location	Observed 22-year range (annual R)	Single storm erosion index values normally exceeded once every				
		1	2	5	10	20
				years		
Alabama, Montgomery	164-780	62	86	118	145	172
Connecticut, Hartford	65-315	23	33	50	61	79
Florida, Jacksonville	283-900	92	123	166	201	236
Georgia, Atlanta	116-549	49	67	92	112	131
Illinois, Springfield	38-315	36	52	75	94	117
Indiana, Indianapolis	60-349	29	41	60	75	90
Kentucky, Lexington	54-396	28	46	80	114	151
Louisiana, New Orleans	273-1366		73	99	121	141
Maryland, Baltimore	50-388	41	59	86	109	133
Michigan, East Lansing	35-161	19	26	36	43	51
Minnesota, Minneapolis	19-173	25	35	51	35	78
Missouri, Columbia	98-449	43	58	77	93	107
New Hampshire, Concord	52-212	18	27	45	62	79
New York, Rochester	22-180	13	22	38	54	75
North Carolina, Raleigh	152-569	53	77	110	137	168
Pennsylvania, Harrisburg	48-232	19	25	35	43	51
South Carolina, Columbia	81-461	41	59	85	106	132
South Dakota, Huron	18-145	19	27	40	50	61
Tennessee, Nashville	116-381	35	49	68	83	99
Texas, Austin	59-669	51	80	125	169	218
Virginia, Roanoke	78-283	23	33	48	61	73
Wisconsin, Madison	58-251	29	42	61	77	95

Several important conclusions can be drawn from these figures and tables.

1. The average annual R value varies greatly across the United States (Figure 1). Values as great as 500 exist in the southeastern portion of the United States while in the Midwest values of about 150 are common. This would tell us that if all other factors were equal, erosion in the southeast would be about 2.5 times greater than in the Midwest on an average year.

2. Annual R values vary greatly from year to year at a given location. Note in Table 1 the wide range in annual R values for a given location. At some locations there is a factor of 8 or more between the maximum and minimum annual value observed. This means that year to year rainfall variations can cause a 10-fold variation in erosion even at the same location.

3. Not only does the magnitude of the R-value vary with location but so does the distribution of rainfall erosivity throughout the year.

Soil Erodibility (K)

The soil erodibility factor, K, in the USLE is a quantitative description of the inherent erodibility of a soil. Field determination of K values is time consuming and laborious. However, guideline values have been determined as illustrated in Table 2. In addition, the nomograph shown in Figure 3 may be used to estimate K factors provided the particle size analysis, organic matter, infiltration characteristics and soil structure are known.

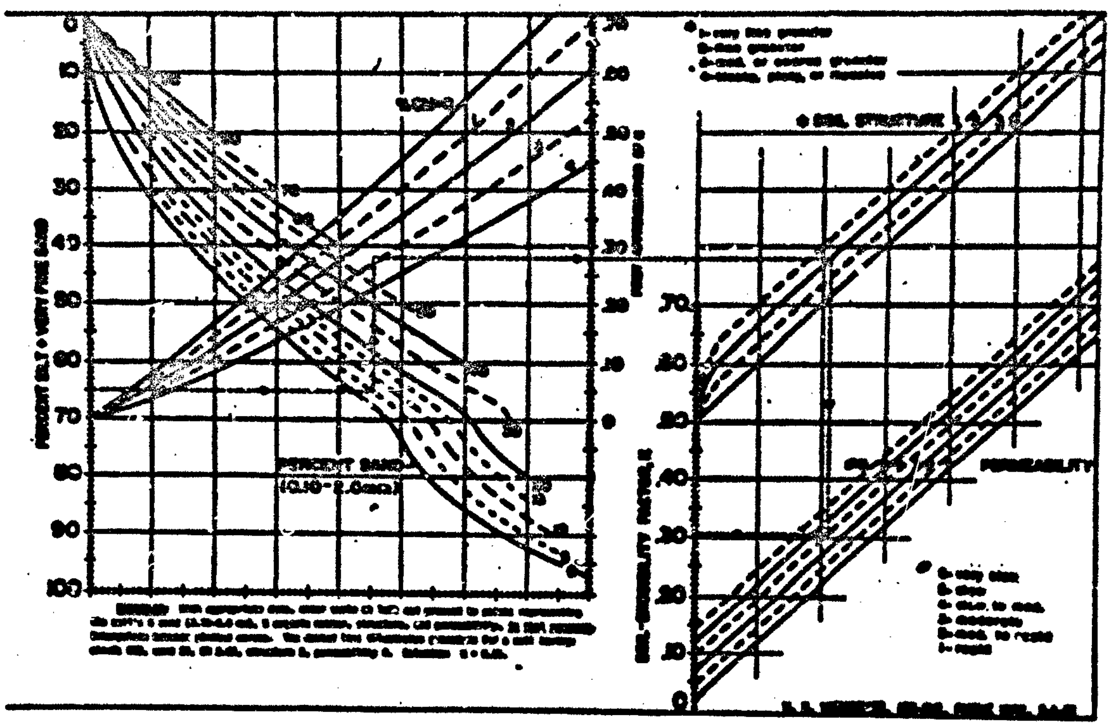

Figure 3. Soil erodibility nomograph.

It is difficult to develop guidelines for estimating soil erodibility.

In general:

1. **Erodibility decreases as the infiltration rate increases.**

2. **Erodibility decreases as the particle or stable aggregate size increases. Not that soils classified as silts have the greatest K values. Soils very high in silt tend to be very poorly aggregated and silt**

sized particles are highly transportable. While soils high in clay have smaller soil particles, these particles tend to form stable aggregates and therefore not erode as easily as silt.

3. Erodibility decreases as the organic matter of the soil increases.

Table 2. - Indication of the general magnitude of the soil erodibility factor (K).

Texture class	Organic matter class		
	<0.5%	2%	4%
Sand	0.05	0.03	0.02
Fine sand	0.16	0.14	0.1
Very fine sand	0.42	0.36	0.28
Loamy sand	0.12	0.1	0.08
Loamy fine sand	0.24	0.2	0.16
Sandy loam	0.27	0.24	0.19
Fine sandy loam	0.35	0.3	0.24
Loam	0.38	0.34	0.29
Silt loam	0.48	0.42	0.33
Silt	0.6	0.52	0.42
Sandy clay loam	0.27	0.25	0.21
Clay loam	0.28	0.25	0.21
Silty clay loam	0.37	0.32	0.26
Sandy clay	0.14	0.13	0.12
Silty clay	0.25	0.23	0.19

Values shown are estimated averages of specific soil values.

Topographic Factor (LS)

The effects of slope length and gradient is represented by L and S, respectively; however, they are often evaluated as a single parameter, LS. Slope length is defined as the length of slope from the point of origin of overland flow to the point where the slope decreases sufficiently for deposition to occur or to the point where sediment enters a well defined channel. Slope gradient is the slope segment slope. Figure 4 is a graphical representation of the LS value for various lengths and gradients.

The factors obtained from Figure 4 are for slope segments of uniform shape. The shape of the slope, however, affects the erodibility of the slope. Onstad et al. (1967) and Foster and Wischmeier (1974) present a methodology for computing LS on irregular shaped slopes.
General conclusions from LS considerations:

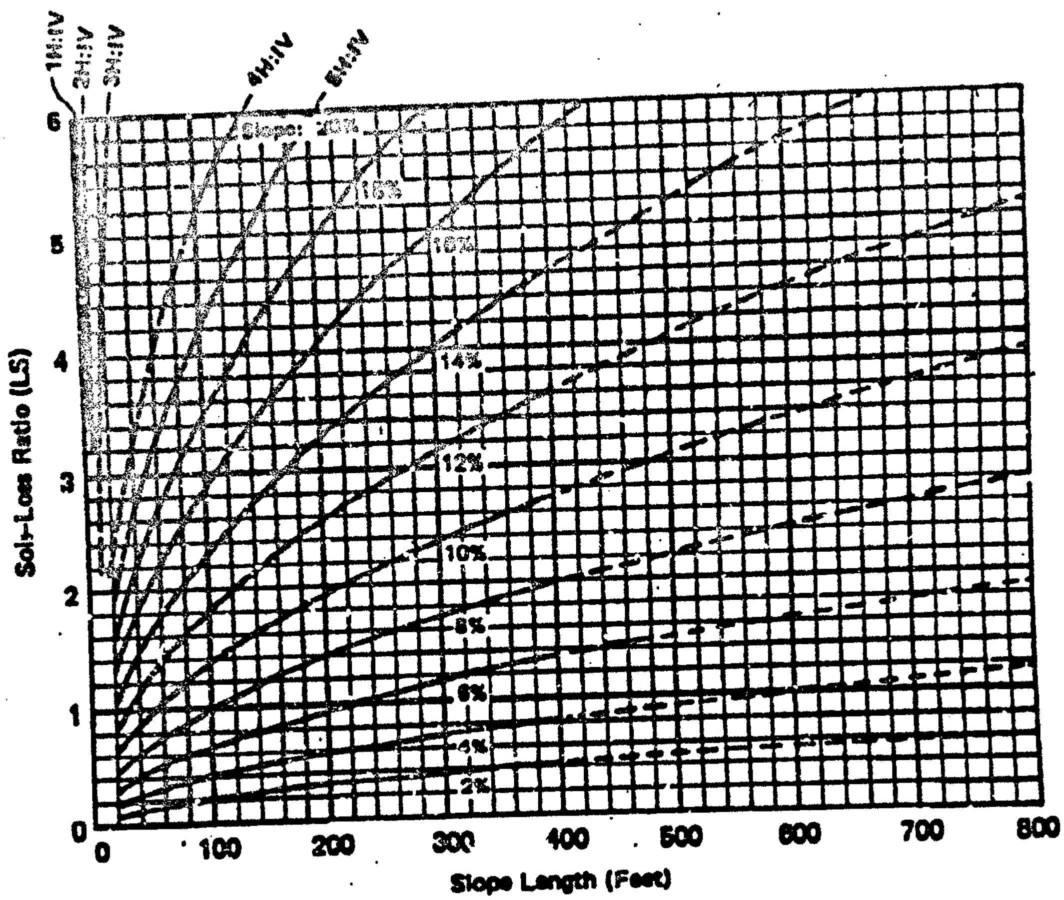

Figure 4. - Slope length and steepness factors.

1. Potential erosion rates increase with increasing slope length or steepness

2. Potential erosion rates can be modified by changing the slope geometry. Concave-shaped slopes will have less erosion than convex slopes of the same length and average gradient.

Cover-Management-Practice Factor (CP)

The USLE was originally developed to predict erosion rates from agricultural land with C and P being divided into a cropping management factor (C) and a conservation practice factor (P). The cropping management factors described the effect of rotations and tillage practices. The practice factor described practices such as contouring and strip cropping. In nonagricultural areas, it is often possible to combine the concepts of C and P into one factor that describes the cover and management of an area. A CP value of 1 is defined as the erosion from a continuously fallow area with no conservation practices. Typical CP values for various land use activities are presented in Table 3.

Most often one is interested is obtaining the CP values for a series of activities scheduled over a period of time. To illustrate these computations consider the following example. In this example a forested area near Boston is to be cleared for construction. The transition is to take place over a 1-year period as shown in Table 4 below. What is be the CP value for the years activities?

Table 3. - Examples of various CP values for various cover and management practices.

Condition	CP factor
Bare compacted soil	
Compacted bulldozer scraped up and down	1.3
Compacted fill	1.2-1.7
Seeding	
Establishment period	0.40
Permanent well established	0.01
Crushed stone	
Application rate 6.2 lb/sq ft	0.05
Application rate 11 lb/sq ft	0.02
Woodchips	
Application rate 1 lb/sq ft	0.02

Table 4. - Construction schedule.

Date	Activity
January 1 to March 31	Forested
April 1 to June 30	Cleared
July 1 to September 30	Construction
October 1 to December 31	Vegetated

Table 5. - Computation of CP value.

Time period	Activity	Percent R	Activity CP	Weighted CP
1/1 to 3/31	Forested	11	0.001	0.000
4/1 to 6/30	Cleared	21	0.30	0.063
7/1 to 9/30	Construction	45	1.00	0.450
10/1 to 12/31	Vegetated	24	0.05	0.012
			CP value	0.525

Percent R used as the weighting factor.

To illustrate activity scheduling affects on CP values consider the same sequence of activities conducted under a different time schedule as shown in Table 6.

Table 6. - Computation of CP under a second time schedule.

Time period	Activity	Percent R	Activity CP	Weighted CP
1/1 to 3/31	Construction	11	1.00	0.11
4/1 to 6/30	Vegetated	21	0.05	0.011
7/1 to 9/30	Forested	45	0.001	0.001
10/1 to 12/31	Cleared	24	0.30	0.072
			CP value	**0.194**

Note that the only difference between Table 5 and Table 6 is the scheduling of the activities. The year is still divided into four equal time periods. Since the same geographical location was used, there is no change in the R distribution. The difference is that in the first schedule (Table 5) the major erosive activity is scheduled during the period of greatest R, while in the second schedule this activity was scheduled during the period of smallest R. This change in scheduling results in a decrease in CP from 0.525 to 0.194. Therefore, all other factors being equal operating under the second schedule would result in greatly reduced average erosion rates.

APPLICATION OF THE UNIVERSAL SOIL LOSS EQUATION

To illustrate the use of the Universal Soil Loss Equation let's extend the example used to determine CP. Assume:

Location: Boston
  R = 140
Soil: Silt Loam with 4% Organic Matter
  K = 0.42

Slope Characteristics:
  Concave LS = 2.9
  Convex LS = 5.0

Cover-Management/Practice: (As previously determined)
  Schedule 1 CP = 0.525
  Schedule 2 CP = 0.194

Concave/Schedule 1	140 x 0.42 x 2.9 x 0.525	89
Concave/Schedule 2	140 x 0.42 x 2.9 x 0.194	33
Convex/Schedule 1	140 x 0.42 x 5.0 x 0.525	154
Convex/Schedule 2	140 x 0.42 x 5.0 x 0.194	57

Note that the average annual erosion rate under the various conditions we have described could vary from 33 to 154 tons/acre/year depending upon the slope and schedule used.

Part II. WIND EROSION

WIND EROSION PRINCIPLES

The basic causes of wind erosion and the principles of its control are easily understood. Wind erosion is most likely to occur when:

1. The soil is fine grained or highly organic;
2. The soil surface is bare, smooth, loose, and dry;
3. The area is open and free of obstructions and
4. Windspeeds are high.

Wind erosion can be reduced or controlled by:

1. maintaining the soil surface in a nonerosive condition and/or
2. preventing the erosive winds from coming in contact with the soil surface.

While the principles of wind erosion and its control are relatively simple, the challenge is to find that combination of practices which is compatible with management plans yet provide protection against the erosive wind forces.

FACTORS AFFECTING WIND EROSION

Soils Characteristics

Most serious wind erosion problems are often associated with sandy soils or highly organic soils. Because of their loose, single-grain structures which tend to dry rapidly, sandy soils are highly susceptible to wind erosion. Dry organic soils are also highly erosive because of their low bulk density.

Wind-eroded soils move in three ways. The smallest and lightest particles - silt, clay and organic particles - move through suspension. Suspended particles appear as dust clouds and may occur high in the air. They may travel many miles before being deposited. Suspended material is of concern because it reduces visibility, creates nuisance dust problems, and may carry nutrients and pesticide residues.

Saltation is the term used to describe intermediate sized particles bouncing from the soil surface to a height of up to 3 or 4 ft and then falling back to the earth's surface. The kinetic energy of saltating grains of sand may break nonerosive sized aggregates into erodible sized particles. Saltating particles may injure or kill young plants reducing yields or forcing replanting of crops and may be deposited in drainage or road ditches forcing costly cleanout or causing reduced drainage. Some particles are too large to be bounced into the air in a saltating motion. These large particles may move through surface creep. Such particles are rolled along the soil surface only occasionally rising a few inches above the surface. This movement may occur due to impact of saltating particles or by direct force of the wind. Particles moving by surface creep may have sufficient energy to cut off or abrade young plants. Soil moving by surface creep may also be deposited in fence rows or drainage ways.

Aggregates, clods, and particles larger than about 0.08 inch in diameter are usually not susceptible to movement by wind unless the aggregate is first broken into erodible sized particles. These large particles tend to stay in place and help protect the soil against wind erosion.

The amount of soil moving by surface creep, saltation, and suspension is dependent upon soil type and condition as well as wind speed. Dry organic soils and dust sized particles are highly susceptible to movement by suspension. Sands, on the other hand, tend to move primarily through saltation and surface creep. In general, particles moving by saltation play a major roll in wind erosion. Often saltation is the dominant form of movement. The impact of saltatin g particles break stable aggregates into erodible sized particles, thereby, increasing the soil available for transport.

Soil Surface Conditions

Soil surfaces that are bare, smooth, loose and dry tend to be highly erodible. Soils moistened by rainfall or irrigation water tend to be much less erosive than dry soils. Unfortunately, strong winds dry the soil surface rapidly and may lead to wind erosion even when the soil profile has abundant water.

A soil's susceptibility to wind erosion is also affected by the roughness of the soil surface. Surface ridges such as these produced by tillage implements tend to reduce the wind's velocity between the ridges reducing the soils susceptibility to erode. In addition, soil particles that move from the ridge tops are often caught between the ridges where they are protected from the wind. Sorting of the particles on the ridge tops creates armor plating of the ridges by larger, less-erodible particles further reducing erosion. One must distinguish between surface ridge roughness and irregular topography. While ridge roughness tends to reduce erosion, knolls scattered throughout a field are subject to high erosion rates because the erosive force of the wind is much greater on the knoll top than on a level field.

Cover conditions also play a major role in determining if wind erosion will occur. Soils covered by stable mulches or growing vegetation are much less erosive than bare soils. The effectiveness of a mulches or growing plants is influenced by the plant species or mulch type, plant population or mulch rate, and orientation with respect to the prevailing wind. Because young plants are susceptible to blowout and abrasion, care must be taken to protect young plants against erosive forces until their foliage is sufficiently developed to provide the necessary protection.

Unprotected Length

The length of the exposed area in the direction of the wind also greatly affects erosion rates. Wind has a finite capacity to transport soil which is a function of its energy. As a wind moves across an exposed area it begins to pick up soil. The shorter the exposed distance the less opportunity to reach its transport capacity and the lower the erosion rate.

Obstructions placed along the edge of exposed areas help decrease the unprotected length and to reduce erosion by temporarily reducing the windspeed and therefore the transport capacity.

Wind Characteristics

Winds are characterized by their velocity, duration, and direction. Not all winds have sufficient velocities to cause soil movement. The wind velocity necessary to initiate soil movement is the "threshold velocity" and is dependent upon the size, density and condition of the soil particle. The threshold velocity for an organic soil may be as low as 8 miles per hour; for sandy soils the threshold velocity is usually considered to be about 12 miles per hour when measured 1 ft above the soil surface. The velocity of wind increases with height above the soil surface. Since most published wind data are for heights greater than 1 ft, threshold velocities must be adjusted upward before comparison with published data. For example, data available from the Stevens Point Municipal Airport in Wisconsin are collected at a height of 35 ft. A threshold velocity of 12 mph at 1 ft would correspond to approximately 18 mph at the 35-ft height.

Combined Effects

The previous discussion has identified critical soil, cover and wind conditions for wind erosion. The total wind erosion potential is a combination of these three factors. Critical components identified were:

Soils: Sandy and highly organic soils

Cover: Most erosive when bare and smooth. This is most apt to occur in the spring between tillage and crop canopy development and from harvest to snow cover.

Plant susceptibility: First few weeks after emergence.

Wind: During the spring and late fall months.

During the winter months the erosivity of the wind is relatively high, but if soils tend to be snow covered or frozen preventing soil movement, the wind erosion potential is relatively low. During the summer months wind erosion potential is relatively low and soils are often covered with growing vegetation. This combination of low wind velocities and good vegetative cover means that the potential for wind erosion is very low. The remaining periods of spring and fall are the times of concern. In the spring, the erosive energy of the wind is at its greatest. Soils also tend to be in their most erosive state since this is a time of tillage with little vegetative growth. While the wind conditions are somewhat less erosive in the fall, some soils are left exposed after harvest. These exposed soils and increasing wind conditions produce a second erosive period.

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CASE HISTORY
STREAMBANK PROTECTION - BIOENGINEERING EVALUATION
Southwest Washington

By Leland Saele[1]

Abstract: Soil Conservation Service (SCS) experience with application of bioengineering methods on three streambank protection projects in southwest Washington

Working with the Washington Department of Fisheries and the Washington Department of Ecology, the SCS has participated in the installation and monitoring of bank protection measures to stabilize actively eroding streambanks and improve fish and wildlife habitat along the Newaukum River in southwest Washington. The bank treatments selected for these trial sites rely primarily on combinations of woody vegetation and rock for stability. The instream rock placement was completed during normal low flow in the fall of 1989 followed by vegetative bank treatments in early spring of 1990. Winter storms during 1990 and 1991 resulted in overbank flow at all three sites. This provided a real test of bank stability during the initial establishment and monitoring period. The combination of a rock toe and woody vegetation on the bank shows promise for replacing traditional SCS methods of rock riprapped bank treatments. The continuous rock toe protection may be necessary to assure stability during the establishment period. Extensive loss of woody vegetation from insect damage is a problem to be reckoned with.

INTRODUCTION

The Soil Conservation Service (West National Technical Center and Washington State) entered into a cooperative agreement with the Washington Department of Fisheries (WDOF) and Washington Department of Ecology (WDOE) for developing, implementing, and evaluating biotechnical methods for streambank stabilization. This agreement was initiated in 1988 and currently three sites have been constructed. All three sites are located in southwest Washington on the Newaukum River in a climatic zone of mild wet winters and temperate, generally dry, summers. The design and drawings were prepared by the WDOF and reviewed by WDOE and the SCS. Construction was performed by WDOF in two phases: Phase I, in the fall of 1989, for instream and low bank work and Phase II, in early spring 1990, for vegetative measures and upper bank shaping. The general features of each site and comments on performance to date are noted below.

SITE FEATURES - reference figures 1, 2, and 3.

Nygard Site

a. Phase I - August 1989, low flow in river - 1) installed toe rock (205-ft reach) to elevation of ordinary high water line; and 2) installed rock and brush (R&B) trenches at 25-ft intervals (450-ft reach).

b. Phase II - May and June 1990 - 1) shaped top of bank (985-ft reach); 2) live willow staking on banks, 2- by 2-ft grid (950-ft reach), was completed in May*; and 3) fencing (1560-ft reach).

* Live willow staking was intended to be completed during dormant season; however, this was not accomplished. Inspection in July 1990 indicated approximately 50 percent survival.

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Fitzgerald Site

a. Phase I - August 1989, low flow in river - 1) installed rock and brush trenches at 30-ft intervals (410-ft reach), rock projected from the toe of trench to form minijetty at toe of bank.

b. Phase II - March 1990 (dormant season) - 1) Shaped bank between rock and brush trenches; 2) live willow staking on banks, 2- by 2-ft grid, installed in late March and early April*; and 3) fencing.

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* Inspection in July 1990 shows 100 percent survival.

Teitzel Site

a. Phase I - August 1989, low flow in river - 1) installed toe rock along entire reach (510 ft), top of rock at ordinary high water line; and 2) experimentally installed six 1-ft-wide trenches at 8-ft intervals approximately 8 ft into bank near the midpoint of the project reach. Two trenches were filled with rock and four trenches were filled with willows between layers of soil.*

b. Phase II - Mid-March 1990 - 1) shape upper bank, slope at 1:1 to 1.5:1 from top of toe rock; 2) install live willow fascine in bank at top of toe rock; and 3) live willow staking on banks, approximately 550 ft on a 2- by 2-ft grid**; and 4) fencing.

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* Willow filled trenches were swept clean by winter flood flows. Design revised to eliminate trenches.
** Inspection in July 1990 shows excellent stand with 100 percent survival.

COMMENTS

a. General. - Two back-to-back storms during the winter of 1989-90 (estimated at 70- to 100-year events) provided a severe test of the Phase I bank treatment measures. A field review of the sites in March of 1990 indicated initial good success with the R&B trenches at two sites. The narrow R&B trenches at the Teitzel site did not perform well and the design was altered to include a live fascine above the toe rock in place of the 12-in-wide R&B trenches.

Live willow stakes were obtained from local sources and included several species, the most predominant being Sitka. Live stakes were generally 4 ft long, installed with half the length below the ground surface. Initial survival of the willow stakes was reflective of the dormancy period at the time of installation. Those installed in early spring (March) had high initial success whereas those cut and installed later did not fare well. Surprisingly the willows in the R&B trenches, installed in late July and August had good survival. Overall performance for the first season was considered excellent for those installed in the fall or during the early spring dormant period.

The winter of 1990-91 also proved to be a severe test with storms in November 1990 and March 1991 that resulted in overbank flow at all three sites. This was followed by a cool wet spring and a very dry summer and fall. Inspection of the sites in November 1991 indicated a high percentage of mortality from an infestation of the Willow Borer at two of the sites.

April 1992 - Western Washington experienced a mild winter without flood flow on the Newaukum River. All sites look relatively good. O&M has been performed to replace willows lost to the Willow Borer and desiccation.

SITE-SPECIFIC COMMENTS

Nygard. - A site visit in March 1990 indicated no significant change in bank conditions in that portion of the reach where R&B trenches had been installed. However, downstream of this reach for a distance of several hundred feet, bank sloughing did occur. Prior to the Phase I work the entire reach was unstable with the upper portion, where the R&B trenches were installed, as the most unstable. It is apparent that the R&B trenches located at 25-ft intervals were effective, including the reach without toe rock. The rock and projecting brush from each trench appears to be sufficient to slow near bank velocity while deflecting the flow away from the bank for the short interval between trenches. The erosion downstream of the R&B trenches is indicative of what could have occurred over the entire reach had the R&B trenches not been installed.

The rock riprap installed along the toe of the bank appears stable. Observations during the installation of the toe rock in August 1989 noted that the gradation did not meet the specified size. There was considerable oversize material and a lack of middle size rock for a uniform gradation. There was no evidence that rock was displaced during the winter floods; however, long-term stability is a concern.

Site conditions in October 1990 show a good stand of willows at each R&B trench. The live willow staking survival rate between the trenches varies from 60+/- percent on the lower bank to 15+/- percent on the upper banks. Although volunteer grass and weeds are thick in some areas there are spots without ground cover. These bare spots are susceptible to erosion if exposed to high velocity flow. The soils at this site are plastic and this will help. Also, the willow stakes, even though dead, will retard the near bank flow and minimize the effect of high water flows during the winter months as long as they remain in place.

The willow stakes that have died are replaced with live willow stakes in March of 1991. General site condition is good at this time. This site was not inspected in the fall of 1991.

Inspection in April 1992 reveals isolated, relatively small areas of high willow mortality, mostly upper bank. The Willow Borer and desiccation are suspect.

Fitzgerald. - Considerable exposed raw bank between the R&B trenches was observed during a site visit in March 1990, prior to bank shaping and installation of the live willow stakes. Although the reach looked bad, the bank was quite irregular before the trenches were installed and it was difficult to determine actual erosion caused by flood flow during this first winter.

Site conditions in July 1990 show an excellent stand of willows that were installed the last week of March. Survival was estimated at nearly 100 percent. The bank shaping that was done just prior to live staking resulted in a more positive projection of the R&B trench. This was believed to improve the retardance factor and more effectively deflect flow away from the bank. The projection of large rock at the base of the trench was very much evident during low flow stage. Smaller size material may have been displaced during the winter flows.

Continued erosion between the R&B trenches is evident in the spring of 1991. The soils at this site are more stratified than first indicated. The high-stage flows are washing fines from sand and gravel layers in the bank near the toe of slope, resulting in substantial erosion. In some areas, more than 24 inches of bank material has been lost, exposing the roots of willow stakes over the entire embedment length.

Inspection of the site in November 1991 reveals an infestation by the willow borer. High mortality of the willow stakes and willows in the R&B trenches is evident. The dry summer and fall is believed to be a contributing factor, particularly along the upper bank.

Teitzel. - A site visit on March 13 revealed that winter flood flows had completely removed the brush layering from a number of the 1-ft-wide trenches. The rock filled trenches were still intact. Because of the poor performance of the narrow trenches, the design was altered and a live fascine was installed just above the toe rock along the entire reach. The banks were shaped and live willow stakes installed.

A site visit the following July showed an excellent stand of willows growing from the willow stakes installed in mid-March. No dead willows were found. The willow fascine was also very much evident by a line of willows at the top of toe rock.

This site also experienced significant loss of willows during the summer and fall of 1991. Loss was attributed to the Willow Borer and plant stress due to dry summer and fall. The willow fascine did not appear to be affected by the Willow Borer infestation.

CONCLUSIONS

a. R&B layered trenches, 4 to 6 ft wide, installed in the bank, normal to streamflow and spaced at intervals of approximately 25 ft, appear effective in protecting the intervening bank from high stage flows prior to the establishment of other vegetation.

b. Initial survival of willow brush layering (R&B trenches) installed in late summer (August) and fall has been successful. Saturation of earth fill during installation is necessary to prevent desiccation.

c. Initial success with live stakes using 4-ft willow whips installed to a depth of 2 ft was good. Extended dry periods during the first several years is a factor. Live stakes should be installed to a depth that will ensure tip contact with moist soil during the dry season.

d. Live willow staking in late spring after the dormant season was not successful.

e. Seeding of shaped banks may be necessary for development of good ground cover, particularly in more erosive low plastic soils. Competition from weeds and desiccation will be a factor during establishment.

f. Placement of rock or other mechanical means to protect the toe may be critical to success, particularly if the bank materials are non plastic. Filters and bedding for rock, in accordance with standard engineering practice are recommended for long-term stability.

g. Frequent maintenance should be anticipated.

The initial success of this project has demonstrated that applying biotechnical methods in conjunction with standard engineering practice will work in western Washington. It has been referred to as Bioengineering but is still mostly "trial and error" engineering. We hope to develop a more scientific approach by continued monitoring of these jobs and others. We believe this project is a good start and that eventually a true bioengineering approach for streambank protection will be developed.

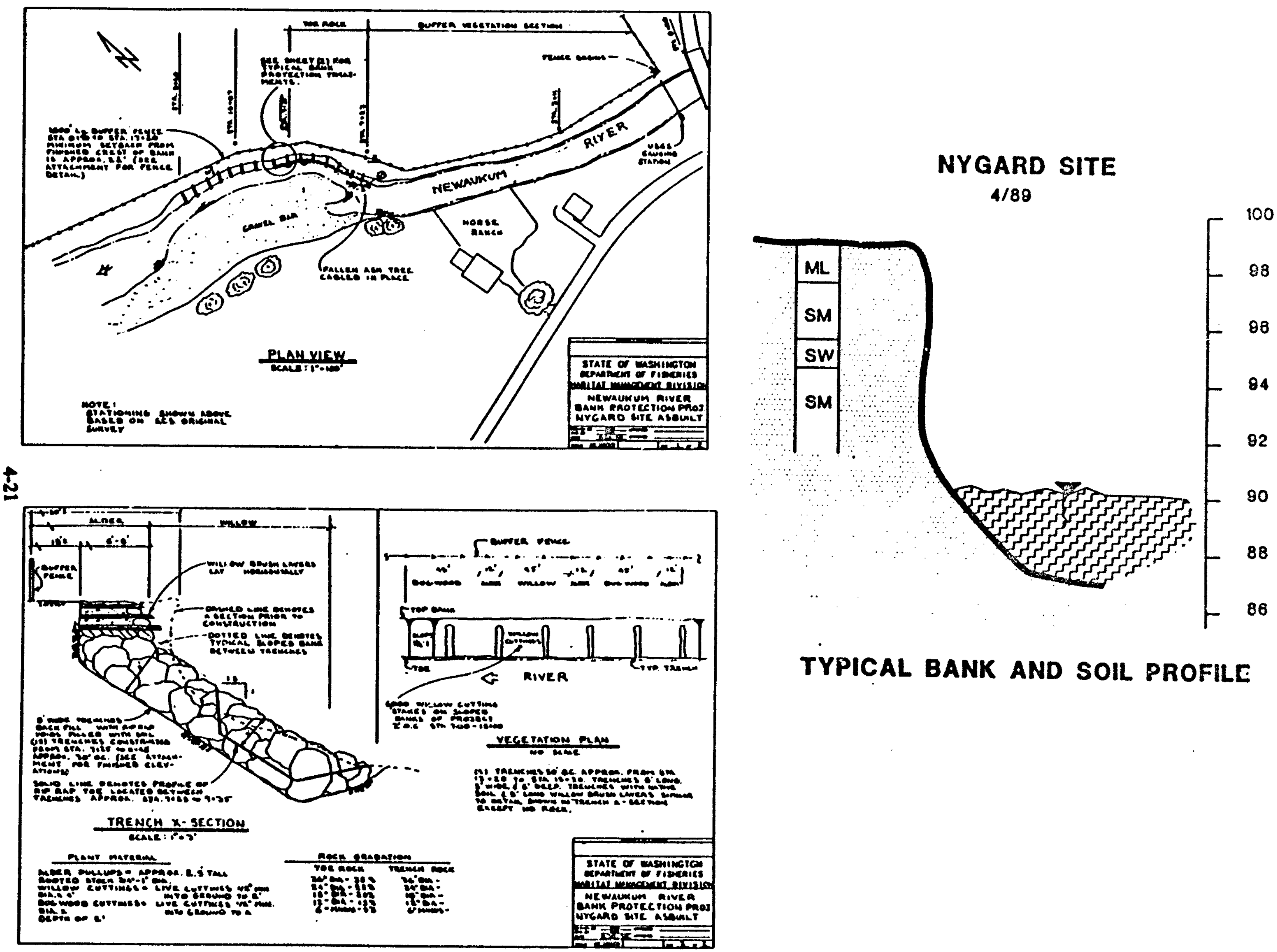


Figure 1. - Nygard site - typical bank and soil profile.

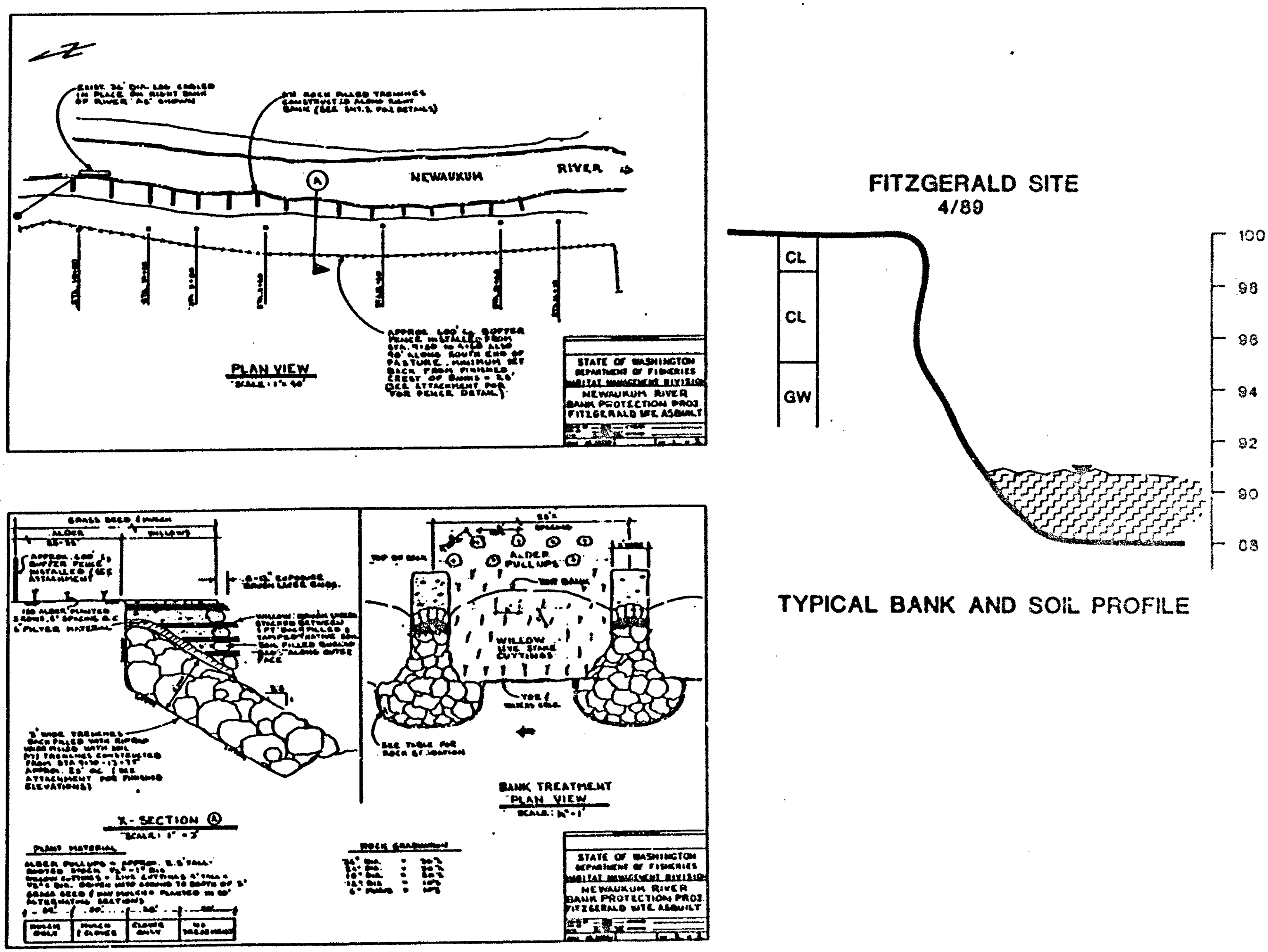
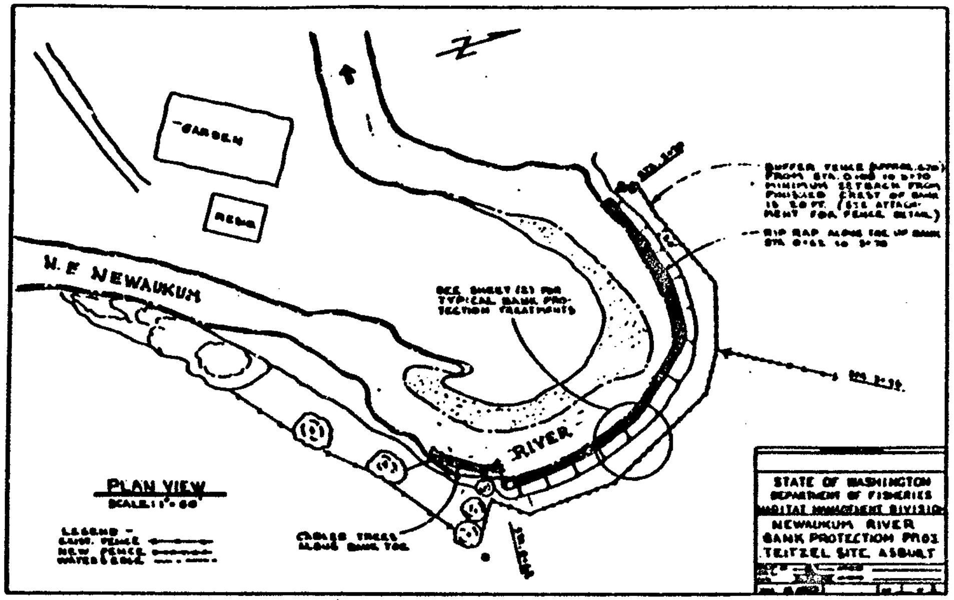
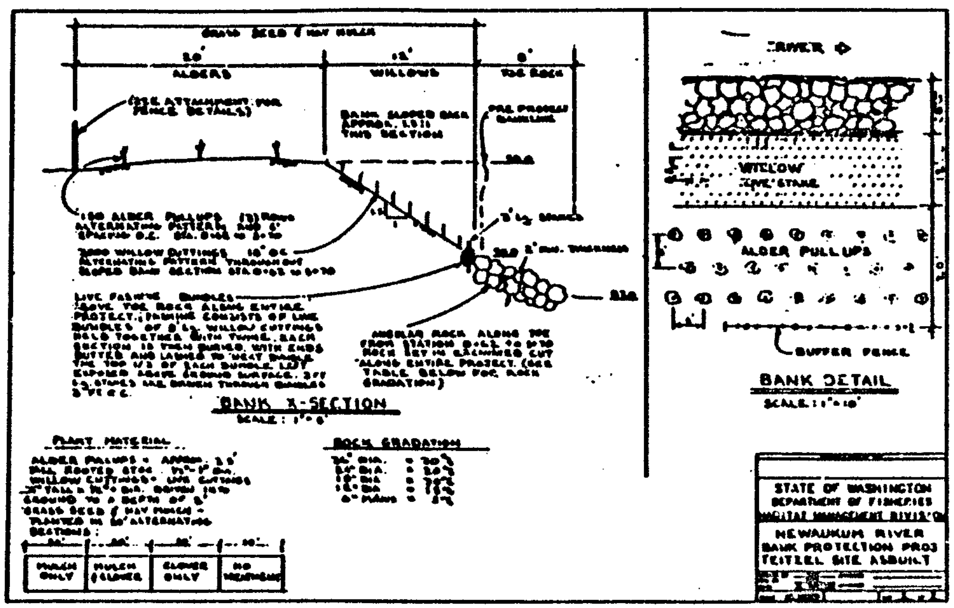


Figure 2. - Fitzgerald site - typical bank and soil profile.





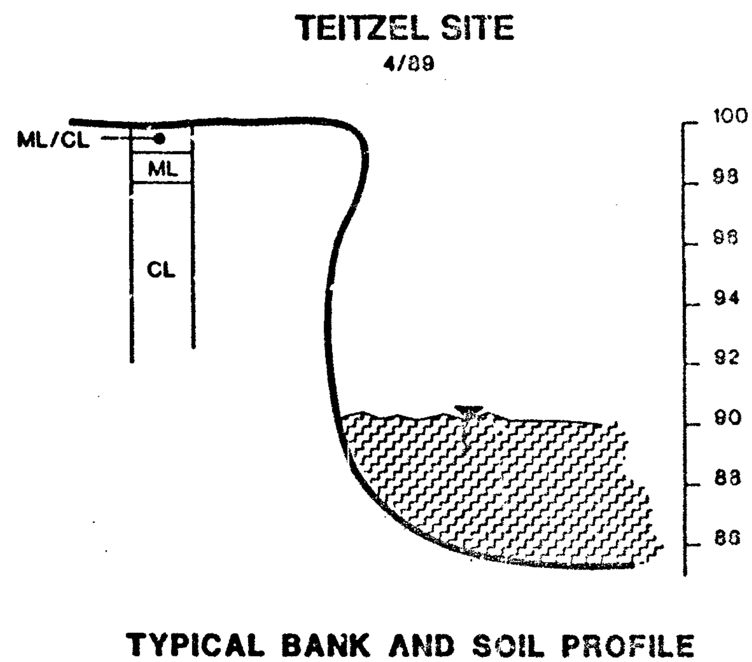


Figure 3. - Teitzel site - typical bank and soil profile.

THE EXPANDING ROLE OF GEOSYNTHETICS IN EROSION AND SEDIMENT CONTROL

By Marc S. Theisen[1]

Abstract: The use of geosynthetic erosion and sediment materials continues to expand at a rapid pace. From their early beginnings in the late 1950's, geosynthetic materials today are the backbone of the erosion and sediment control industry. Geosynthetic components are an integral part of erosion and sediment materials ranging from temporary products such as hydraulic mulch geofibers, plastic erosion control meshes and nettings, erosion control blankets, and silt fences to high-performance turf reinforcement mats, geocellular confinement systems, erosion control geotextiles, fabric formed revetments and concrete block systems. This paper provides a brief overview of these materials and concepts, and how they may be designed and incorporated into cost effective applications.

INTRODUCTION

We are entering a new environmental era where concern for the protection of our planet's natural resources will reach global proportions. Continued technological advances have led to improved monitoring of Earth's vital signs. As such, prior theoretical modeling of environmental concerns such as the "Greenhouse Effect," ozone depletion, rising sea levels, deforestation, drought, accelerated erosion, sediment loading of waterways, species extinction and the eventual downfall of mankind appear chillingly realistic.

Slogans such as "Think Globally, Act Locally," "Love Your Mother," and "Someone Always Lives Downstream," are spearheading the efforts of numerous preservation groups. With the continued demise of oppressive governments, optimism for world peace and an unprecedented feeling of global unity, a spirit of environmental cooperation is beginning to prevail.

The term "nonpoint pollution" hopefully is heading toward obsolescence with "watch dog" groups such as Stream Watch sloshing their way up muddy creeks to pinpoint sources of unchecked sediment. Improved methods to detect and monitor rates of erosion and sedimentation via high tech satellite imagery or even the actions of the Stream Watchers of the world lends credence to the old saying "you can run but you can't hide." Generators of sediment and other pollutants can and will be identified.

Cumulative research suggests excessive sediment in our waterways **is the planet's most prevalent contaminant.** The amount of world wide erosion is staggering. The Worldwatch Institute has placed the annual rate of soil erosion from crop lands at approximately 27.5 billion metric tons or 25 billion US tons (Brown and Wolf, 1984). Agriculture accounts for roughly 60 to 70 percent of the annual soil loss with construction and urban runoff accounting for about 15 percent of the erosion "pie." Mining and forestry combined may contribute up to 10 percent. All sources combined, the annual rate of erosion in the United States alone probably exceeds 3.3 billion metric tons/year (Northcutt, 1992).

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Sediment accounts for more than two-thirds of all pollutants entering U.S. waterways. It is estimated up to $13 billion per year are spent in the United States to directly mitigate the offsite impacts of erosion and sediment. Increscent economic and social losses from reductions in arable farmland, timber production, fishery yields, species diversity and navigable waterways exceed those caused by pollutants in the public eye such as nuclear and hazardous wastes, oil spills, air pollution, or ground water contamination. Worse yet, the problem is exasperated as one moves downstream toward our coastlines and population centers.

Recently a number of laws have been mandated in the United States to combat excessive erosion. Such legislation ranges from local erosion and sediment control ordinances to numerous State and Federal agricultural, waste containment and surface mining acts, to the broadly encompassing Environmental Protection Agency's 1972 Clean Water Act and The Farm Bill administered by the Department of Agriculture.

In October 1992 The Clean Water Act will mandate projects disturbing more than 2 hectares (5 acres) of land to obtain a National Pollutant Discharge Elimination System (NPDES) permit to help identify and quantify release of pollutants into our watersheds. This act is up for further review in 1992. With environmental groups pushing for numerical standards for sediment discharge, next year inspectors may be measuring the turbidity of your runoff in parts per million, just like heavy metals or hazardous waste. Landfills, surface mines, commercial real estate developments, even our public agencies such as DOT's, county and municipal entities will be scurrying to develop erosion and sediment control plans or face stiff Federal fines. These actions are only the tip of the iceberg as more and more Government agencies and entities get with the program.

Just what is erosion control? To control erosion is to curb or restrain (not completely stop) the gradual or sudden wearing away of soils. We have all seen extreme examples of excessive erosion such as gullied hill slopes or stream channels choked with debris, but often erosion goes unchecked on flat to moderately sloping terrain. Soil loss is a continually occurring process in natural ecosystems as well as successfully reclaimed sites--without it our scenery would be very boring. The goal of any revegetation or erosion control project should be to stabilize soils and manage erosion in an economical manner (Theisen, 1988).

In this era of shrinking budgets, decision makers are hard pressed to reclaim disturbed sites at minimum costs. Given site conditions such as slope angles, climate, runoff, soil profile and ultimate land use; a specifier must select with confidence a technique she (he) feels will perform up to expectations at the lowest cost. Over the past 25 years the erosion control industry has experienced rapid growth and is becoming more sophisticated. Materials developed for erosion and sediment control (E&SC) are becoming increasingly effective. Improved design and installation guidelines are directing the use of E&SC products toward more specific and cost effective applications. The industry has evolved from the seed drills, straw blowers, hydroseeders, excelsior, jute, concrete channel liners and riprap of the 1960's into a diverse hierarchy of techniques and materials. It seems as if every month a new product is introduced to control erosion and sediment in more specific situations. Numerous materials have come and gone in the survival of the fittest, most cost competitive products.

Historical Perspective

Geosynthetics may broadly be defined as synthetic materials or components used with soil, rock, earth or other geotechnical engineering related materials as an integral part of a man made product, structure or system. Benefits include reinforcement, stabilization, separation, drainage, filtration, containment, and erosion control. Related materials include geotextiles, geogrids, geomembranes, geomeshes, geonets, geomats, geofibers, geocomposites, and the newest term, "geoappurtenances" to cover the myriad of materials being developed for geotechnical applications.

Many of us perceive the use of geosynthetic materials for erosion and sediment control as a new horizon. However, geosynthetics have played a major role in the E&SC industry for over 30 years, particularly in the case of rolled goods. In 1958, a geosynthetic component was incorporated into an erosion control system which has changed the course of slope, channel and embankment protection. A "plastic cloth" was used in lieu of a granular filter to prevent sand from washing out behind concrete blocks used for shoreline protection (Richardson and Koerner, 1990). The significant cost savings realized when a 0.4-millimeter-thick plastic filter cloth could replace up to a meter of soil peaked the interest of the U.S. Army Corps of Engineers. Subsequent successful installations of woven plastic cloth filters in coastal structures led to the birth of the geotextile industry as is practiced today. Through the years tens of millions of square meters of woven and nonwoven geotextiles have been installed as a critical component of hard armor systems.

Another geosynthetic breakthrough was initiated about 10 years later. In the mid-1960's only one type of erosion control blanket existed. A state soil conservationist discovered the material used for wrapping cotton bales could be used to prevent soil erosion. The material was jute, a woven mesh of thick natural yarns, which when applied on the soil surface provided thousands of tiny check dams to help keep soil from washing away. Jute blankets allow vegetation to become established on steeper slopes and in higher flowing swales than traditional hydraulic straw and hay mulches. A similar material remains in use today.

However, jute has drawbacks: its open weave construction leaves soil exposed, the organic material tends to shrink and swell under changing moisture conditions, and it is extremely flammable. To achieve optimum results straw or hay mulch still must be placed beneath the jute.

What was needed was a one step, roll out mulch blanket. The first attempts involved a very dense mat of curled, barbed aspen wood (excelsior) fibers. The material stayed together but was too dense to allow vegetative growth. Next, a twisted kraft paper net was placed above a thinner mat of excelsior fibers. Vegetation grew through the blanket but performance of the paper netting was very inconsistent; often breaking down too quickly and being lifted by the vegetation or worse yet, allowing the blanket to be washed away before vegetative establishment. A stronger, nonmoisture sensitive, more durable netting was needed. Polypropylene netting was the answer.

Combining a dense mat of excelsior with a plastic netting lead to the first successful excelsior erosion control blankets. Field trials with various nets, fiber lengths and glue patterns resulted in essentially the same blankets we see today. The key to the improved performance of excelsior over jute blankets is the plastic net backbone of the product.

Biaxially Oriented Process Nets

Biaxially oriented process (BOP) nets are typically manufactured from polypropylene or polyethylene resins. BOP nets are extremely versatile in that composition, strength, elongation, aperture size and shape, color, and ultraviolet stability can easily be designed into the product for specific site requirements. Because they do not absorb moisture, these nets do not shrink and swell like kraft paper nets and jute blankets. BOP nets have proven to be so adaptable they are being used to create more complex products and are even used alone to anchor loose fiber mulches such as straw, hay and wood chips. The lightweight nettings placed over mulches come in rolls which are 3 to 4-1/2 meters in width, weigh only about 55 kilograms and will cover 0.4 hectare (1 acre) or more. Installation of these products is less labor intensive than traditional netting products.

Erosion Control Meshes

A step up from BOP nettings are woven polypropylene geotextile erosion control meshes. In fact, the newer twisted fiber erosion control meshes can provide comparable performance to natural fiber erosion control blankets. These photobiodegradable, natural looking, high-strength polypropylene meshes protect the soil surface from water and wind erosion while accelerating vegetative development. Four-meter lightweight rolls facilitate installation on slopes and channels. Erosion control meshes may be used alone, with dry mulches, or as a stabilizing underlay for sod reinforcement. They also show promise as an open weave geotextile facing for fostering vegetation on geosynthetically reinforced steepened slopes or bioengineering installations where establishment of woody plant species is desired. Displaying rapid photobiodegradation in one direction, these meshes allow woody vegetation to freely sprout and emerge through the installation with little potential of girdling.

Erosion Control Blankets (ECB's)

BOP nettings or woven meshes of varying characteristics are now placed on one or both sides of finely tuned erosion control blankets adapted to anticipated site conditions. These 1- to 2-meter-wide biodegradable fiber erosion control blankets (ECB's) are composed of straw, excelsior, cotton, coconut, polypropylene or blends thereof. Nettings or meshes may contain UV stabilizers for controlled degradation or long chain interrupters to accelerate photodegradation. Colors vary from clear, tan, green to black. Methods of holding the fibers in place range from glues and glue strips to more superior parallel lock stitching with cotton, polyester or polyolefin threads. Applications for the wide variety of blankets range from protection of gradual to steep slopes to low or moderately flowing channels. The top of the line blankets may provide temporary resistance to short duration flow velocities of up to nearly 3 meters per second.

Finally and perhaps of most concern to the environment, these meshes and nettings may ultimately become biodegradable. As photodegradation progresses the plastic chains are cut into shorter and shorter segments down to a plastic "sand" which becomes part of the soil. These short segments become biologically degradable and are attacked by soil microorganisms and converted to carbon dioxide and water (Guillet, 1974). It is unfortunate that emotional, uninformed anti-plastic stigmas sometimes preclude the use of these extremely cost effective temporary materials in lieu of costly exotic fibers or hard armor solutions.

TERMS VS. PERMS

At this point an important distinction must be presented regarding the intended use of E&SC materials. For many installations vegetation alone will provide adequate long-term erosion protection. However, getting vegetation established requires a variety of techniques. Materials of a temporary nature which facilitate vegetative establishment, then degrade, may be termed **TERM's or temporary erosion and revegetation materials.**

Basically TERM's consist of degradable natural and/or synthetic components which provide temporary erosion control and aid in the growth of vegetation. In only a few instances are TERM's "totally organic". Remember vital geosynthetic components often include netting, stitchings and adhesives. These short term materials degrade leaving only vegetation for long-term low to medium flow resistance. Table 1 lists various TERM techniques.

Table 1. - General term techniques

Straw, hay, and hydraulic mulches
Tackifiers and soil stabilizers
Hydraulic mulch geofibers
Erosion control meshes and nets (ECMN's)
Erosion control blankets (ECB's)
Fiber roving systems (FRS's)

Site conditions requiring the higher performance of reinforced vegetation or revetment systems will require PERM's or **permanent erosion and revegetation materials.** PERM's may be subdivided into **biotechnical composites** when vegetation is reinforced or **hard armor systems** when nonvegetated inert materials are installed.

Biotechnical Composites are composed of nondegradable materials which furnish temporary erosion protection, accelerate vegetative growth and ultimately become synergistically entangled with living plant tissue to extend the performance limits of vegetation. This reinforced vegetation provides "permanent" medium to high flow resistance provided Biotechnical Composites are protected from sunlight via shading by vegetation and soil cover. Table 2 outlines examples of Biotechnical Composites.

Table 2. - Biotechnical composites (perm's)

UV stabilized fiber roving systems (FRS's)
Erosion control revegetation mats (ECRM's)
Turf reinforcement mats (TRM's)
Soil and sports turf geofibers
Vegetated geocellular containment systems (GCS's)
Vegetated concrete block systems

Hard armor systems generally employ inert materials used to provide high to maximum flow resistance where conditions exceed performance limits of reinforced vegetation systems. Listed in Table 3 are systems used to provide permanent erosion protection of areas subject to high flows, wave action and/or scour attack.

Table 3. - Hard armor systems (perm's)

Geocellular containment systems (GCS's)
Fabric formed revetments (FFR's)
Concrete block systems (CBS's)
Gabions
Riprap
Composites and hybrids

Fiber Roving Systems (FRS's)

Fiber roving systems (FRS's) are another geosynthetic concept providing moderate erosion protection. Developed in the late 1960's, rovings are applied in a continuous strand for protection of drainage swales and slopes.

Fiberglass roving is a material formed from fibers drawn from molten glass and gathered into strands to form a single ribbon. Polypropylene roving is formed from continuous strands of fibrillated yarns wound onto cylindrical packages such that the material can be fed continuously from the outside of the package. Use of fiberglass roving has been declining due to its carcinogenic properties and is being displaced by more versatile "environmentally friendly" polypropylene roving.

Erosion control roving is unique because of the flexibility of application, allowing for any width or thickness of material to be applied (Agnew, 1991). Other erosion control materials, such as blankets or mats require the user to apply the width or thickness of material supplied. Fiber rovings may be viewed as an "in situ" erosion control geosynthetic with reduced labor and material costs over traditional blanket materials. The continuous strand concept provides ease of installation with minimal waste factors from overlap.

Using compressed air, roving is rapidly applied through a nozzle over the seeded surface and then anchored in place using emulsified asphalt or other natural or synthetic soil stabilizers. Photobiodegradable polypropylene roving may be used for temporary applications (TERM) or when UV stabilized is appropriate for extended use situations (PERM). In addition, these polypropylene roving systems may be colored to match substrates or improve visual aesthetics.

The use of fiber roving systems is rapidly expanding. Key markets include highways, surface mines and landfills. The future in FRS's lies in the development of a one step application apparatus which will further accelerate installation efficiency. The concept of developing an on site mat or blanket is certainly appealing and extremely cost effective.

TRM VS. ECRM

Turf reinforcement is a method or system by which the natural ability of plants to protect soil from erosion is enhanced through the use of geosynthetic materials. A flexible three-dimensional matrix retains seeds and soil, stimulates seed germination, accelerates seedling development and most importantly, synergistically meshes with developing plant roots and shoots. In laboratory and field analyses, biotechnically reinforced systems have resisted flow rates in excess of 4 meters per second for durations of up to 2 days, **providing twice the erosion protection of unreinforced vegetation** (Carroll et al., 1991). Such performance has resulted in the widespread practice of turf reinforcement as an alternative to concrete, riprap and other armor systems in the protection of open channels, drainage ditches, detention basins and steepened slopes.

Permanent geosynthetic mattings are composed of durable synthetic materials stabilized against ultraviolet degradation and inert to chemicals normally encountered in a natural soil environment. These mattings consist of a lofty web of mechanically or melt bonded polymer nettings, monofilaments or fibers which are entangled to form a strong and dimensionally stable matrix. Polymers include polypropylene, polyethylene, nylon, and polyvinyl chloride.

Geosynthetic mattings generally fall into two categories: **turf reinforcement mats (TRM's)** or **erosion control revegetation mats (ECRM's)**. Higher strength TRM's provide sufficient thickness and void space to permit soil filling/retention and the development of vegetation within the matrix. TRM's are installed first, then seeded and filled with soil. Seeded prior to installation, ECRM's are denser, lower profile mats designed to provide long-term ground cover and erosion protection. By their nature of installation TRM's can be expected to provide more vegetative entanglement and long-term performance than ECRM's. However, denser ECRM's may provide superior temporary erosion protection. Geosynthetic mattings occupy one of the fastest growing niches of the erosion and sediment control industry.

Geocellular Containment Systems (GCS's)

Geocellular containment systems work in a unique fashion in that strength or stabilization by confinement is achieved by a series of three-dimensional cells up to 20 centimeters deep. When expanded into position, the polyethylene or polyester cells have the appearance of a large honeycomb, one of nature's most efficient structures. The cells are then backfilled with soil, sand, or gravel depending upon application. For revegetation, the soil-backfilled cells are seeded, fertilized and covered with a variety of TERM or PERM techniques. The mulches provide surface protection while the cells greatly reduce the chances of subsurface failure and act as a deeper rooted biotechnical composite. Shallow lateral root development is precluded by the nearly impermeable geocell walls. As such vegetated GCS's are limited to flow velocities of 2 to 3 meters because of the tendency of the cells to sustain scouring under high flow velocities or shear conditions (Chen and Anderson, 1986).

For higher flow conditions GCS's may act as an easy to install form which is filled with concrete or grout to create a hard armor system. Typically a geotextile will be placed beneath the expanded web to provide separation and/or filtration. Erosion control applications for GCS's are many including steep slope revegetation, channel liners, shoreline revetments, retaining walls, boat ramps, and low flow stream crossings.

Fabric-Formed Revetments (FFR's)

Fabric forming systems are mattresses typically constructed of water permeable, double layer woven geotextiles which are positioned on the area to be protected and filled with a pumpable fine aggregate concrete (structural grout). The two layers of geotextile are joined at discrete points to create a form which when filled with grout will conform to most subsoil conditions. Thickness and geometry are determined by internal spacer threads woven into the upper and lower sheets of fabric. In many cases the mattresses may be installed for less cost than conventional armor systems since all construction is conducted in place with no heavy equipment or skilled labor required (Richardson and Koerner, 1990).

FFR's are generally available in three styles. Filter point mats are formed with a double-layer woven fabric, joined together by interwoven filter points which relieve hydrostatic pressure. Uniform section mats are formed with a double-layer woven fabric, joined together by spacer cards on closely spaced centers. Relief of hydrostatic uplift pressure may be provided by inserting plastic weep tubes through the mat at specified centers. Articulating block mats are formed with a double-layer woven fabric, joined together into a matrix of rectangular compartments each separated by a narrow perimeter of interwoven fabric. High-strength cables may be threaded between the two layers of fabric to interconnect the concrete filled compartments (blocks), and provide for block articulation. Hydrostatic pressure relief is achieved by slits cut between adjacent blocks and/or inserting plastic weep tubes. A filtration geotextile is recommended beneath all fabric formed revetments.

Installation of FFR's consists of four basic steps:

1. Site preparation
2. Geotextile and panel placement/field assembly
3. Structural grout pumping
4. Inspection of field seams, zipper connectors, and lap joints

Concrete Block Systems (CBS'S)

Concrete block systems consist of prefabricated concrete panels of various geometries which may be attached to and laid upon a woven monofilament or nonwoven geotextile. Bending and torsion are accommodated by having the concrete blocks articulated with joints, weaving patterns or connection devices. Concrete block systems may be subdivided into three groups: nontied interlocking blocks, cable-tied blocks, or in situ concrete (Hewlitt et al., 1987).

Concrete block revetments incorporate cellular concrete blocks, either open or closed, and are underlain with a properly designed filtration geotextile. The blocks are held on the slope by anchors placed at the top of the slope and/or by friction between the slope and the blocks. The blocks can be assembled into fabricated mats either at the factory or onsite. Sections of precabled concrete blocks may be placed by using a special spreader bar, which may lower costs on large projects. Or the blocks may be handplaced with or without the cable subsequently installed.

Articulating concrete block revetment systems combine the favorable aspects of lightweight blankets and meshes, such as porosity, flexibility, vegetation encouragement, habitat enhancement, and ease of installation, with the nonerodibility, self weight, and high tractive force resistance of rigid linings. These specially designed interlocking precast concrete grids are a proven cost-effective, aesthetic, and functional alternative to dumped stone riprap, gabions, structural concrete, and other heavy-duty, durable channel protection systems. Additionally, these systems offer enhanced flow efficiencies, nurturing of vegetative cover and safe access (Koutsourais and Sprague, 1992).

Gabions

Gabions are compartmented rectangular containers made of galvanized steel hexagonal wire mesh or rectangular plastic mesh and filled with hand-sized stone. Cells of equal capacity are formed by factory-inserted plastic or wire netting diaphragms or partitions which add strength to the container and help maintain its shape during the placement of stone. In highly corrosive conditions a polyvinyl chloride coating is used over the galvanized wire.

Advantages of gabions include flexibility, durability, strength, permeability and economy versus rigid structures. The growth of native plants is promoted as gabions collect sediment in the stone fill. A high percentage of installations are underlain by woven monofilament and nonwoven geotextiles to reduce hydrostatic pressure, facilitate sediment capture and prevent wash out from behind the structure.

Riprap

Riprap consists of stone dumped in place on a filter blanket or prepared slope to form a well graded mass with a minimum of voids. Stone used for riprap is hard, dense, durable, angular in shape; resistant to weathering and to water action; and free from overburden, spoil, shade and organic material. The riprap material is generally placed on a gravel bedding layer and/or a woven monofilament or nonwoven geotextile fabric.

Performance of Erosion Control Materials

Several test procedures have been proposed to quantify performance of erosion control materials. Initial concern for vegetated systems is temporary erosion protection prior to and during seed germination and seedling development. Typically, this level of performance is measured by the material's ability to minimize soil loss when subjected to various flow rates and/or rainfall amounts. Temporary erosion protection is important but the long-term goal of any vegetated erosion control matrix is to provide permanent erosion protection via permanent vegetation and/or subsequent root reinforcement. The more rapidly vegetation becomes established the more rapidly long-term erosion control may be accomplished. Thus, the material's ability to facilitate vegetative establishment is equally important. Too much emphasis on an erosion control product's temporary protection may inhibit the growth of newly emerging seedlings.

Perhaps the most critical parameter in an engineering design is flow resistance before, during and long after vegetative establishment. Some erosion control materials may be washed away before the vegetation takes hold while others may temporarily exhibit excellent flow resistance only to lose their effectiveness as they degrade or decompose over time. Specifiers must take into account immediate and long-term flow resistance based upon longevity of the material when designing grassed slopes and waterways.

Two basic design concepts are used to evaluate and define a channel configuration that will perform within accepted limits of stability. These methods are defined as the permissible velocity approach and the permissible tractive force (shear stress) approach. Under the permissible velocity approach the channel is assumed stable if the adopted velocity is lower than the maximum permissible velocity. The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and the materials forming the channel boundary (Chen and Cotton, 1988).

The permissible velocity approach uses Manning's Equation where with given depth of flow, D, the mean velocity may be calculated as:

$$V = 1.49\, R^{2/3}\, S^{1/2} / n$$

where

- V = average velocity in the cross section;
- n = Manning's roughness coefficient;
- R = hydraulic radius, equal to the cross-sectional area, A, divided by the wetted perimeter, P; and
- S = friction slope of the channel, approximated by the average bed slope for uniform flow conditions.

The tractive force approach uses a simplified shear stress analysis which is equal to:

$$\tau = \gamma D S$$

where

- τ = tractive force;
- γ = unit weight of water;
- D = maximum depth of flow;
- S = average bed slope or energy slope.

Design criteria based on flow velocity may be limited because maximum velocities vary widely with channel length (L), shape (R), and roughness coefficients (n). In reality it is the force developed by the flow, not the flow velocity itself, that challenges the performance of erosion control systems. Tractive forces caused by flowing water over the ground surface create shear stresses which can be used as a design parameter independent of channel shape and roughness. Moreover, the higher stresses developed in channel bends or other changes in stream channel geometry can be quantified by simplified shear stress calculations, providing a higher level of design confidence than otherwise possible (Chen and Cotton, 1988).

Critical shear stress determinations are meant to be used with velocity calculations for prescreening of channel lining designs. Manning's Equation remains the primary hydraulic research and design tool. However, as everyday practice has determined, a simplified screening criteria such as maximum shear stress is necessary to ensure properly engineered design of channel lining erosion control systems. Figure 1 combines cumulative research for several erosion control materials and attempts to group categories of erosion control materials into their cost effective design niches (Theisen, 1992).

Maximum permissible velocities for vegetative techniques are illustrated under vegetated and nonvegetated conditions. Thus a designer will have performance guidelines from the time a material is installed to when it becomes fully vegetated. As additional data becomes available from field and laboratory analysis perhaps a design guide chart utilizing maximum permissible shear stress may be developed. The reader must be cautioned, velocity and tractive force are not directly proportional. Under certain conditions, a decrease in velocity may increase depth of flow, thereby increasing shear stress.

Flow Duration

Of key importance is the significance of duration of flow. Note from Figure 1 that allowable flow velocities decrease with flow duration. This is a critical point. Manufacturers of both organic, natural and synthetic erosion control products often express the erosion resistance of their materials in terms of maximum allowable flow velocity or permissible shear stress. Though unstated, these flow limits are typically for very short durations (minutes rather than hours). They do not reflect the potential for severe erosion damage that results from moderate flow events over a period of several hours. Ironically, many manufacturers, designers and users do not consider duration of flow when evaluating and selecting erosion control measures (Theisen and Carroll, 1990).

Typically, a major precipitation event will produce significant flow velocities with durations lasting hours or days . . . not minutes. The two day design duration was selected because in grass waterways, high-velocity flow events should be no longer than about 2 days duration, following which grass recovery and subsoil drainage should be able to take place (Hewlett et al., 1982). As Figure 1 illustrates, duration of flow will reduce the erosion resistance of a grassed surface. It is critical that design of grassed waterways take this into account.

Flow values for the various temporary erosion control mulches, blankets, meshes and rovings have been truncated and placed into a gray area because extended flow duration trials for these materials have not been reported. Their long-term performance may either go up or down as vegetation becomes established and ultimately will fall into the niche for natural vegetation as the product degrades. Short term performance of fully vegetated surfaces is impressive at nearly 4 meters/second. However, as duration of flow progresses, long-term performance drops off sharply to 2 meters/second (6 ft/second) with 100 percent vegetative cover to only 1 meter/second (3 ft/second) with poor cover.

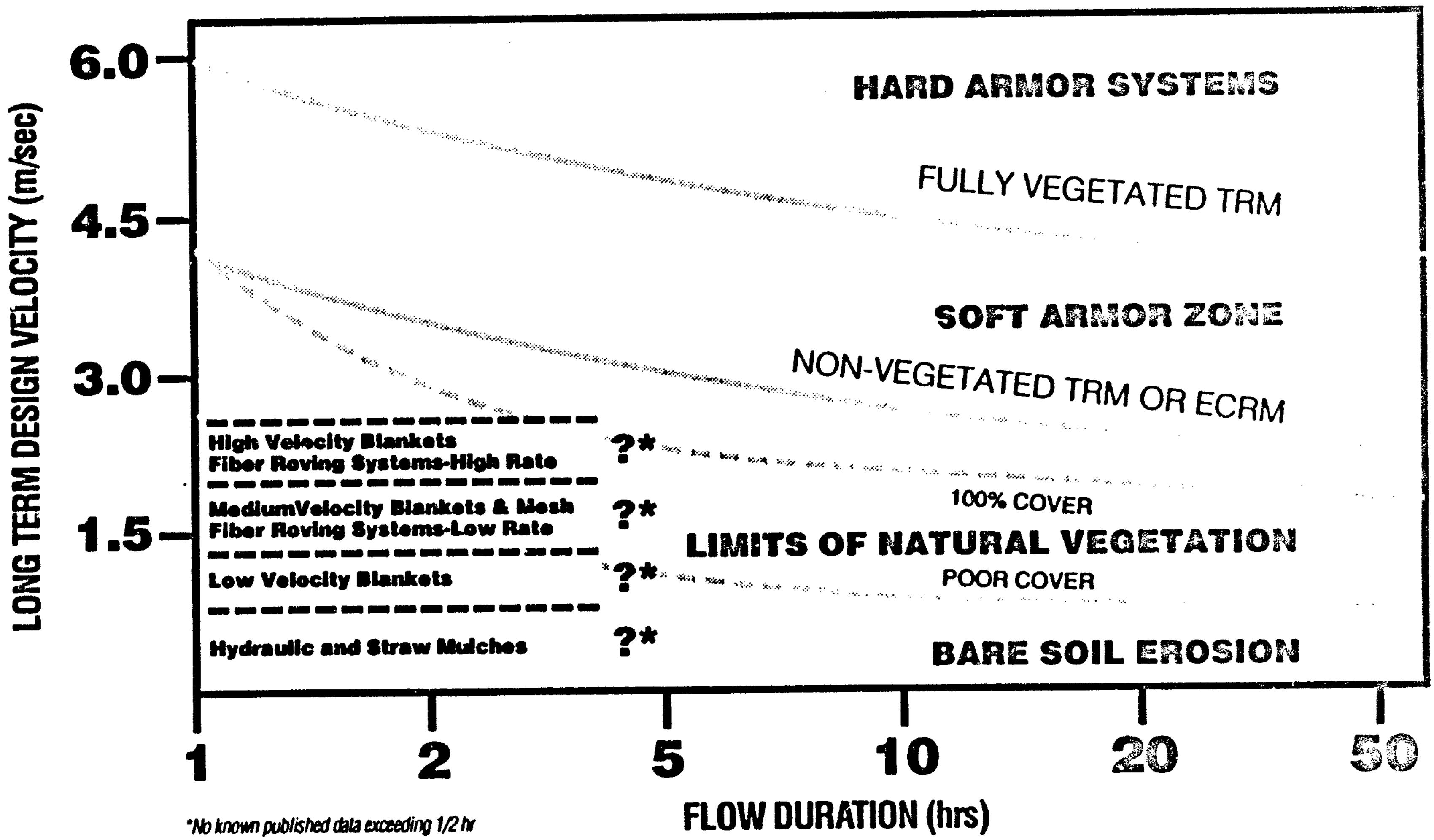

Figure 1. - Long-term performance guidelines.

The "Soft Armor Zone" begins just above the limits of natural vegetation. Performance data for reinforcing mats ranges from unvegetated TRM's and ECRM's (which exceed performance of natural vegetation) to the upper curve which delineates maximum recommended design velocities obtained from field and laboratory evaluation of vegetated TRM's (Carroll et al., 1991; Hewlett et al., 1987; Theisen and Carroll, 1990; Western Canada Hydraulic Laboratories, 1979; Hoffman and Adamsky, 1982; Theisen, 1992). Fully vegetated, geosynthetic mattings may withstand short-term (1/2 hour) flow velocities of 6 meters/second and flow rates of in excess of 4 meters/second for durations of up to 2 days.

The upper end of the graph is comprised by the niche for hard armor materials. The graph is not intended to establish performance limits for these materials, but rather to define the upper limits of "soft armor" (reinforced vegetation). Performance for hard armor materials will be considerably higher and upper limits are beyond the scope of this paper.

Sediment Control

Going hand in hand with aggressive erosion control measures should be a well conceived sediment control plan. Erosion control measures are an offensive strategy to attack potential sedimentation while sediment control practices are a stop gap defensive strategy. In erosion and sediment control planning, the old sports axiom that a strong offense is the best defense is certainly apropos. Vegetation is clearly the finest sediment control product on the planet!

Geosynthetic silt fences have become a standard construction practice over much of the United States replacing straw and hay bales, brush layers and rock check dams. Silt fences are generally installed at the beginning of the construction project and usually consist of woven slit tape geotextiles mounted on prefabricated fence.

A well designed silt fence must initially screen silt and sand particles from runoff. A soil filter is formed adjacent to the silt fence and reduces the ability of water to flow through the fence. This leads to the creation of a pond behind the fence which serves as a sedimentation basin to collect suspended soils from runoff water. To meet such needs, the geotextile must have properly sized openings to form the soil filter and the storage capacity of the fence must be adequate to contain the volume of water and sediment anticipated during a major storm (Richardson and Koerner, 1990).

Porous sediment control structures are one of the newest geosynthetic approaches to sediment control. A three-dimensional moldable mass of crimped polypropylene fibers may be placed in rills or gullies to provide passive sediment control. Placed by hand with its size and shape determined by the installer, applications include rill and gully repair, ditch checks, sediment traps, and perimeter berming. Moreover, the fibers may be encapsulated in a polypropylene mesh to create prefabricated check dams for swale and ditch protection during new construction. Table 4 lists a few sediment control techniques.

Table 4. - Examples of sediment control techniques

Vegetation
Straw and hay bales
Brush layers
Silt fences
Porous sediment control structures (PSCS's)
Rock check dams
Sediment traps, basins and ponds

The Importance of Geosynthetic Materials

While accurate numbers are difficult to come by, sales of erosion control products and services are estimated at $500 to $750 million per year (Northcutt, 1992). The Industrial Fabrics Association International (IFAI) estimates that during 1992, "organic" erosion-control materials (including mulches, mats, tackifiers and emulsions) will compose 55 percent to 65 percent of the erosion control industry. Synthetic mats will make up the remaining 35 percent to 45 percent of the total market of 65 million square meters (Jagielski, 1992).

Assuming a market share of 40 percent for synthetic mattings at an average selling price of $6.00 per square meter, sales of these materials would total $156 million per year. Considering only "organic" erosion blankets to comprise the remaining 60 percent of the market and selling at an average price of $0.60 per square meter, only $23 million in annual sales would be generated. Add the annual sales of erosion control geotextiles, fiber roving systems, fabric formed revetments etc. to the equation and the disparity becomes even larger.

Then consider all but two or three types of degradable blankets utilize geosynthetic components. At most only 10 percent of the "organic" side of the market really is completely organic. Accurate assessment of geosynthetic materials used for E&SC quickly becomes an extremely complicated endeavor. The author personally believes that market share of synthetic mattings is over estimated. The point is not to belabor numbers but to identify the expanding role of geosynthetics in erosion and sediment control. Without the plethora of "geo" materials available, the rapidly evolving E&SC industry would be pretty "slim pickins."

Other Geosynthetic Opportunities

Ideas for geosynthetic erosion and sediment control materials abound. Certainly new ideas have been omitted or are being developed at the time of this publication. Odds are high that geosynthetics will work their way down the ladder into more traditional applications such as hydraulic mulches and degradable erosion control blankets. Hydraulic mulch geofibers (which improve the tenacity of wood fiber and recycled paper mulches) and recycled plastic fiber blankets and mats have already entered the market. Geofibers are being used as part of sports turf systems in major athletic stadiums providing both ground stabilization and a root reinforcing matrix.

The future of geosynthetics for E&SC lies partly in the recycling of waste plastics generated from other applications. Polymer specifications for E&SC applications may not be as stringent as other geosynthetic materials such as geomembranes, geogrids and geotextiles. It's an environmentally friendly gesture in an environmentally friendly industry. And recycled plastics are cost effective.

The Resource Conservation and Recovery Act (RCRA, amended in 1984) requires the EPA to designate items which can be produced with recovered materials, then prepare procurement guidelines to assist Federal agencies in complying the Section 6002 of RCRA. Section 6002 requires that agencies using Federal funds to procure those items must revise their specifications and purchase such items containing the highest percentage of recovered material practicable. Currently, the EPA is studying the feasibility of developing procurement guidelines for construction products, including materials used as erosion control materials.

More research on erosion and sediment control effectiveness of the myriad of materials is mandatory. Questions regarding resistance to extended flow durations and long-term performance for all materials must be answered. Systems must be developed for standardizing product descriptions, sanctioning uniform test and evaluation procedures, and creating a market reporting system to insure broad acceptance of E&SC industry. Organizations such as the International Erosion Control Association (IECA), Erosion Control Technology Council (ECTC), American Society for the Testing of Materials (ASTM), Industrial Fabrics Association International (IFAI), and the Geosynthetics Research Institute (GRI) must lead the way.

Another hurdle to overcome or trail to blaze, depending upon how you look at it, is the issue of biodegradable plastics for short term applications. This is an area of very important research. Members of the industry as well as the general public must be educated. The performance advantages of man made fibers over natural fibers is recognized in many sectors of the textile industry. The E&SC industry is quite possibly a sleeping giant for man made fibers. Keep your ear to the ground and your eyes wide open because "you ain't seen nothin' yet." The future of geosynthetics in erosion and sediment control is bright!

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REVEGETATION AND GEOSYNTHETIC STUDIES AT OCEAN LAKE TO CONTROL SHORELINE EROSION

By John E. Boutwell[1]
Alice I. Comer[1]

A cooperative erosion control project involving the Midvale Irrigation District, Wyoming Game and Fish Department, and Reclamation is currently underway at Ocean Lake. Ocean Lake is a 2469-surface-hectare lake located 32 kilometers northwest of Riverton, Wyoming. It is experiencing shoreline erosion due mostly to wave action, and ice scouring, as well as freeze/thaw disturbance. Of the approximate 48 kilometers of shoreline, 3.2 kilometers have been designated as severely eroded (vertical or cut banks greater than 3 meters in height) and 29 kilometers, minor to moderately eroded. The shoreline is very susceptible to erosion because of the sandy soils and soft shale that surround the reservoir. Prior to this study, riprapping of the eroded shore was the only erosion control measure being practiced along the sloughing banks.

Study plots using several different geosynthetic materials, vegetation, and riprap as well as combinations of these materials were established. These plots are being compared to riprapped areas, previously placed along the shoreline, as well as to each other. The geosynthetic materials used in this project were donated by Synthetic Industries of Chattanooga, Tennessee. Grass seed selection and fertilization were made using recommendations received from the U.S. Soil Conservation Service located in Riverton, Wyoming.

Four different geosynthetic materials were used in this project. Three of these materials consisted of a three-dimensional web of polyolefin fibers positioned between two biaxially oriented nets and mechanically bound together by parallel stitching. These three materials fit the category of erosion mats and shall be referred to in the presentation as 1, 1A, and 2. Material No. 1 is the lofty black polyolefin material, material No. 1A is the same polyolefin material having a thin nonwoven geotextile fabric sewn to the underside, and material No. 2 is a dense green polyolefin material. These three materials were designed to be used in conjunction with the seeding of plant material. The fourth material used was a woven monofilament geotextile which was used as a filter fabric, separating the sandy soil of the contoured slope from the riprap material, which was placed on top of it. This material will be referred to as material No. 3.

These materials were anchored 1 meter above the crest of the slope in a V-shaped trench that exceeded 30 centimeters in depth, as well as top width. A similar trench was used for anchoring the material at the toe of the slope. Riprap was placed on top of the buried geosynthetic materials, at the toe of the slope, for additional protection from uplifting due to wave and ice action.

In addition to anchoring the material to the top and toe of the slope, all geosynthetic materials were anchored across the slope using 30 centimeters steel staples. Subsequent runs of geosynthetic material, which were placed side by side down the fall line of the contoured slope, were overlapped by 15 centimeters and stapled to prevent uplifting by the wind.

Besides these four geosynthetic materials, there was a plot composed only of vegetation, a plot with riprap applied to the newly resloped bank and the original riprapped area were no resloping was done.

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Grasses used to revegetate these shoreline plots were a native mixture comprised of Western Wheatgrass, Indian Ricegrass, Streambank Wheatgrass, Prairie Sandreed, and Sheep Fescue. In addition to these native grasses, Garrisons Creeping Foxtail was used to seed the shore area immediately adjacent to the water's edge.

Cultured plant material, in the form of sod, used in the vegetation plot included, Garrison Creeping Foxtail, *Juncus gerardii*, and a grass collected from the shore of Tiber Reservoir located in north-central Montana. Cattails and bulrush which were collected a short distance from the study plots were also planted along the waters edge of the vegetation plot.

A chemical fertilizer containing a blend of 18 percent nitrogen, 22 percent phosphorus, and 6 percent potassium was applied to all of the seeded areas and a 16-month, slow-release, polymer-encapsulated fertilizer was used when transplanting the vegetative material.

In May 1992, with the help of the Midvale Irrigation District, approximately 100 meters of Ocean Lake shoreline was contoured to a 2 to 1 slope.

Plots, approximately 120 square meters in size, were set up along the contoured shoreline. Those to be seeded were fertilized with 5.5 kilograms of chemical fertilizer per plot. Erosion mats 1 and 1A were seeded after the material had been secured to the slope, whereas erosion mat 2 was seeded prior to the placement of the erosion mat. Seeding of these plots with the five native grass species was done by hand broadcasting the area at a rate of 2 kilograms/plot. Garrisons Creeping Foxtail was hand seeded along the immediate shoreline at a rate of 1/2 kilogram/plot.

These study plots will be evaluated for a period of at least 3 years for effectiveness in controlling erosion, aesthetic and wildlife habitat value, and cost estimates of each method will also be made. Preliminary results of each study plot, materials, and techniques used will be presented during the symposium.

TECHNICAL SESSION 5

Dams and Water Conveyance Systems

SOIL STABILIZATION USED AS REMEDIAL MEASURES FOR WATER RETENTION AND CONVEYANCE SYSTEMS

By Donal' .. Snethen, Ph.D., P.E.[1]

INTRODUCTION

Water retention and conveyance systems serve various social and economic functions, such as: domestic water supply, irrigation and drainage, hydroelectric power, flood control, navigation, and recreation, to name the more important ones. Proper design, construction, and maintenance of these facilities assure, within practical limits, that the water retention or conveyance structure will perform according to expectations for the duration of its design life. Unfortunately, design, construction, and maintenance procedures may not anticipate or otherwise account for all situations which could damage the water retention or conveyance structure. In such cases, remedial repair of the structure becomes necessary. This paper deals with the use of soil stabilization, specifically chemical modification/stabilization, to treat or repair damaged water retention or conveyance structures. For the purpose of these discussions, water retention or conveyance systems are limited to dam and levee embankments (i.e., constructed earth fills) and canal or channel slopes (i.e., excavated slopes). The discussions will consider: the problems associated with embankments and slopes used in water retention or conveyance systems, remedial soil chemical stabilization (as well as some new construction) measures that can be used to address these problems, and some case studies where these remedial measures have been used. This paper does not include any consideration of in situ foundation stabilization (i.e., vibrocompaction, reinforcement or grouting) or any consideration of reinforcement of embankments or slopes. Both of these topics are covered in other technical sessions in this symposium. In addition, the concept of drainage and its influence on the strength and stability of embankment or slope materials is not addressed herein.

THE PROBLEM

Embankments and slopes in water retention or conveyance systems are subject to the same type of problems experienced by any embankment or slope, namely, slope instability (i.e., slides or slumps) and erosion (i.e., internal such as piping and external such as surface runoff or wave action). Slides or slumps are mass movements in which the motion of the moving ground (failure mass) is parallel to the surface of rupture and the moving ground remains reasonably intact. The movement consists of shear strain or displacement along one or several surfaces and is initiated by either increased shear stress or decreased shear strength. The various types of slides, classification of materials and motion, and description of causative factors are discussed by various authors (Terzaghi, 1950; Varnes, 1958; Transportation Research Board, 1978). Damage from slides and slumps can vary from simple esthetics to failure of the embankment cross section. The major economic loss is typically the cost of repair of the slides which requires, at a minimum, removal and replacement of the failed materials. Extreme damage may require construction of an offset (i.e., levee) or complete reconstruction of the embankment. Erosion, whether internal or external, is dependent on the type of soil present in the embankment or slope. Certain soils exhibit characteristics that increase the potential for surface erosion or piping. These materials are generally referred to as dispersive soils. They pose a major hazard to the performance of dams, levees or excavated slopes (McElroy, 1987, 1989; Ryker, 1977, 1987; Sherard et al., 1972a, 1972b; Sherard, 1986). Damage from dispersive or other erosive soils can vary from gullies or washes on the embankment

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or slope to complete failure of the embankment due to piping. The repair of damage caused by the presence of dispersive soils has been a major problem for agencies involved in dam, levee, and canal construction. Typically, treatment of dispersive soils involves chemical modification of the soil to reduce dispersivity. For other erosive soils, slope protection generally reduces the potential erosion problem. Surface erosion from wave action is an example of an erosion problem that has been approached using soil stabilization methods during construction. Specifically, soil cement slope protection has been used successfully by several agencies to protect embankment dam slopes.

SOIL STABILIZATION OPTIONS

Slide Repair

Options available for repair of slide damage to embankments or slopes include:

1. Pushing failed material back into failed area and dressing slope.

2. Removal of failed material and replacement with better soil placed with compaction control (may also include reducing slope angle).

3. Removal of failed materials treatment with lime or fly ash, recompaction with compaction control, and

4. Lime or lime-fly ash injection to strengthen embankment or slope materials in-place.

Option 1 is obviously the least desirable because the same material that failed is being used in the repair. Considering the size and number of slides, the simplest remedial response may be economically justified, however, the permanence of this option is limited. Option 2 is also somewhat limited with respect to permanence; that is, unless the replacement soil is good quality material the embankment or slope may fail in the same location. The reduced slope angle will require more plan area for reconstruction which may pose additional limitations. Based on experience from major dam, levee, or canal construction agencies, option 3 appears to provide the best balance of constructability, permanence, and economics. Although from an economic perspective it is probably the most costly option, it does provide greater assurance with respect to permanence because the soil properties are chemically modified and the chemical reaction usually results in the development of cementation bonds. In other words, it improves the weakened soil by modifying high activity minerals and bonds the particles together. Lime or lime-fly ash injection is a variation on option 3 since the chemical is applied "in-place." However, chemical injection has some significant limitations with respect to the amount of chemical applied and the distribution of the chemical in the soil mass. The amount of chemical applied is generally controlled by the number of injection passes carried out during the stabilization process, generally two or more passes are used in most applications. The distribution of the chemical is a function of the number of fractures or fissures (either pre-existing or created by the injection process) available for the chemical to move through.

To better describe the use of soil stabilization in embankment or slope repair, two examples - one on levee restoration using double lime modification and the other on levee stabilization using lime-fly ash injection - are briefly discussed in the following paragraphs.

Levee Restoration. - The levee reach on which double lime modification was used to repair a slide failure was described by Alvey (1992) and is located in East Central Missouri near Cape Girardeau along the Big Muddy River (Illinois side). Procedures used to select lime percentages, estimate the amount of strength increase obtainable, and obtain various construction and quality control parameters were based on Townsend's (1979) recommendations. Essentially Townsend's recommendations for selection of lime percentage and verification of strength improvement are summarized in Figure 1. Construction and quality control items are discussed in the text of Townsend's report. Numerous slides had occurred along this reach of levee, which was constructed of high plasticity clays with liquid limits in the mid-90s, plastic limits in the mid-20s and plasticity indexes in the 70s. Some of the repair methods that had been used in the area were:

1. Push failed material back into place,
2. Backfill slide depression with new material,
3. Mix (disc) lime into surface materials, and
4. Construct drainage layer in levee slope.

None of these repair methods were very successful or permanent, especially in the high plasticity clays. The St. Louis District of the Corps of Engineers specifies double lime modification when:

1. Soil Plasticity Index (PI) is greater than 40,
2. Soils are anticipated to be difficult to break down,
3. Single application of lime exceeds 4 percent (i.e., mixing difficulties increase),
4. Uniform soil-lime blend is desired.

The construction procedure followed by the St. Louis District starts with removal and stockpiling of the top soil over and around the slide area (typically about 12 in). An inspection trench is dug to locate the failure surface of the slide. The failed soil mass is excavated and placed in a stockpile/mixing area. 10-in lifts are spread in the mixing area and half the required hydrated lime is applied, mixed, and cured. The bottom of the excavated area is treated with lime. The remaining half of the hydrated lime is applied to the soil in the mixing area, the soil-lime blend is mixed then spread and compacted in thin lifts in the excavation until final grade is reached. The topsoil is replaced, fertilized, seeded, and mulched. Experience to date using double lime modification has been very good.

Levee Stabilization. - The levee reach on which lime-fly ash injection was used to stabilize the levee embankment constructed of slide-prone materials was described by Holloway (1992) and is located near Brunswick, Missouri, along the Missouri River. Numerous slides had occurred along a 1300-ft section of levee constructed of high plasticity montmorillonitic clays which were placed with minimal compaction. In all, approximately 15,000 ft of levee had been constructed using the same material and procedures. Because of the amount of material involved, treatment by removal, chemically modifying, and replacing the slide-prone soils was not economically feasible. The decision was made to use lime-fly ash injection to treat the problem soils in-place before slides could occur. Specially equipped crawler tractors with four injection probes were used to treat both sides of the levee to a depth of 10 ft below the ground surface. A primary and secondary injection process was used. Experience to date with the process has been successful.

Erosion Repair/Prevention

Options available for repair/prevention of damage to embankment or slopes from erosive soils, particularly dispersive soils, include:

1. Avoid the material (i.e., relocate the structure or do not use the soil for construction),
2. Alter the properties of the soil with chemical admixture,
3. Provide protective structures or coverings to accommodate the erosive soil.

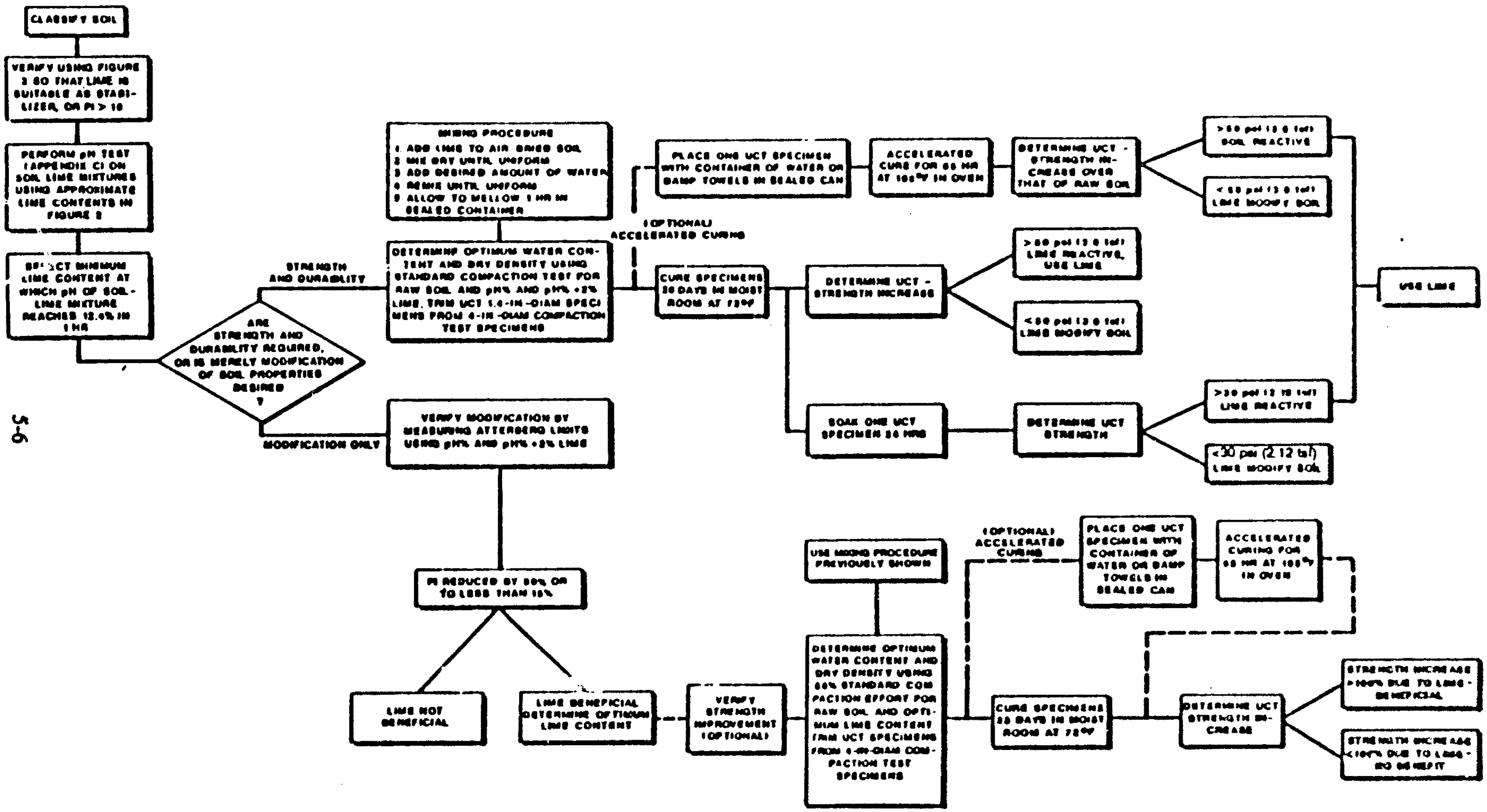

Figure 1. - Flow diagram for assessing use of lime for levee restoration (Townsend, 1979).

Option 1 has its merits but requires preconstruction characterization of the soils and alternative locations for achieving the purpose of the geotechnical structure. Often neither of these requirements can be met. For the context of this paper this option is included only for completeness of the discussion.

Option 2 has been shown to be the most successful and economical approach for repair of damaged embankments or slopes and for prevention of damage to new structures. Lime is the most often used chemical admixture with high-calcium fly ash used where its available. For damaged embankments or slopes, the repair process involves removal of soil from around damaged areas to a depth exceeding the bottom of gullies or jugs, treatment of the soils with hydrated lime or fly ash, and recompaction of the treated soil to establish the original grade. Essentially the treated soil forms a protective layer over the surface of the embankment or slope. For new construction, the lime- or fly ash-treated soil is placed over the surface of the embankment or slope as a protective cover. In both cases, topsoil is placed over the treated soil layer and the surface is seeded or sodded.

Option 3, which is not actually a form of soil stabilization used in the context of this paper, involves the use of protective structures such as riprap layers, articulating concrete block mats (cabled or noncabled), or gabions. All of these protective structures are generally used with geotextiles to provide resistance to erosion for the fine-grained portions of the soils.

The use of chemical modification to repair or prevent erosion damage from dispersive soils is discussed in two case studies described in the following paragraphs. Both examples are taken from records on USDA Soil Conservation Service Flood Control Structures in Oklahoma. Over the past two decades the Soil Conservation Service in Oklahoma has used hydrated lime or fly ash modified soil on 35 to 40 small dams using the procedures described in these case studies.

Erosion Repair. - Cane Creek Watershed Project is a flood control dam located in East Central Oklahoma near Muskogee. The embankment is approximately 3,600 ft long and 30 ft high at the maximum section. In 1988, numerous jugs (i.e., vertical portion of underground erosion tunnels in which the base is at least as large or larger in diameter than the top) were noted in the downstream slope of the embankment near the crest. These problems appeared in spite of a good protective vegetative cover. The decision was made to alter the dispersive properties of the surface soils using hydrated lime (quick lime is allowed but requires more safety considerations). The percent lime used in the repair work was determined from a series of laboratory tests on specimens prepared at different lime percentages (i.e., Atterberg limits, shrinkage limit, dispersion, and compaction). The criteria used for lime percentage selection were based on the percentage that raised the shrinkage limit to near the saturation moisture content based on the design compacted density. For this project 3 percent hydrated lime was selected. The construction process used for the lime modification and associated repair work was:

1. Six in of topsoil was removed from the surface of the embankment and stockpiled. Then 12 in of embankment fill material was excavated and moved to a designated mixing area (or stockpiled for later mixing),

2. Jugs deeper than 1.5 ft were cleaned of loose materials to their full depth and backfilled with lime-treated soil,

3. Three percent hydrated lime (dry powder form) was added to the soil along with the required amount of water and the materials were thoroughly mixed using disc plows and rotary mixers,

4. The lime-treated soil was cured for 72 hours while maintaining consistent moisture conditions,

5. The lime-treated and cured soil was placed in 6-in lifts on a scarified and watered embankment surface and compacted using a crawler tractor (weight greater than 40,000 lb and two passes) until a 12-in-thick layer (perpendicular to the surface) was obtained.

6. Topsoil was placed over the treated layer and seeded.

The treated embankment cross section is shown in Figure 2. To date no additional problems with the embankment have occurred.

Erosion Prevention. - Stillwater Creek Watershed Project is a flood control dam located in central Oklahoma within the city of Stillwater. A new spillway was constructed to meet dam safety requirements. The embankment was removed and reconstructed because of its age and condition and the removal of the old spillway structure. The main embankment is 45 ft high at the maximum section and approximately 750 ft long. Because of the potential for erosion damage, the new embankment was constructed with a 12-in-thick lime-treated protective layer over the crest and downstream slope. Three percent hydrated lime, applied in slurry form, was used for the lime-soil blend. The same mixing, curing, and placement procedures previously described were used for the lime-treated soil layer construction. The cross section of the new embankment is shown in Figure 3. At this time the lake has not been filled because of local roadway construction; however, several rainfall events have occurred since completion with no signs of distress.

Slope Protection

Upstream slopes of embankment dams require protection against the destructive action of waves. Typically, surface protection for embankments is provided by riprap. Experience has shown that dumped riprap is the most practical and economical means for slope protection. Where large quantities are required and/or where suitable sources of rock for riprap are limited, alternatives to riprap have to be used. A commonly used alternative for slope protection is soil cement placed in a stair-stepped horizontal layer configuration on the slope. A good embankment foundation is necessary so that deformation after placement of the soil cement layers is minimal. No special design requirements are required for the use of soil cement slope protection. Normal embankment construction procedures are used, with the soil cement slope protection added as a surface treatment on the upstream slope. The required minimum thickness of the soil cement protective layer is 24 in measured perpendicular to the embankment surface. For typical embankment slopes between 2:1 and 4:1, a horizontal layer between 6 and 8 ft wide provides the minimum thickness. Each succeeding step (layer) is set back a distance equal to the product of the compacted layer thickness (in feet) times the embankment slope. The stepped layers are placed and compacted using standard highway construction equipment. Soil cement can be made from a variety of soils; however, coarse sandy and gravely soils with less than 25 percent passing the U.S. No. 200 sieve are considered the best. Mixing of the soil and cement is generally done in a central mix plant with the soil-cement blend delivered to the construction area using trucks and conveyor systems.

The Bureau of Reclamation has had extensive experience with soil cement slope protection (Casias and Howard, 1984; Casias 1989). Casias (1989) described the performance of 14 major embankments constructed by the Bureau of Reclamation since 1963 on which soil cement slope protection was used. In Oklahoma and Texas, the Tulsa District, U.S. Army Corps of Engineers has used soil cement slope protection on two recently constructed projects: Truscott Brine Dam and Arcadia Dam.

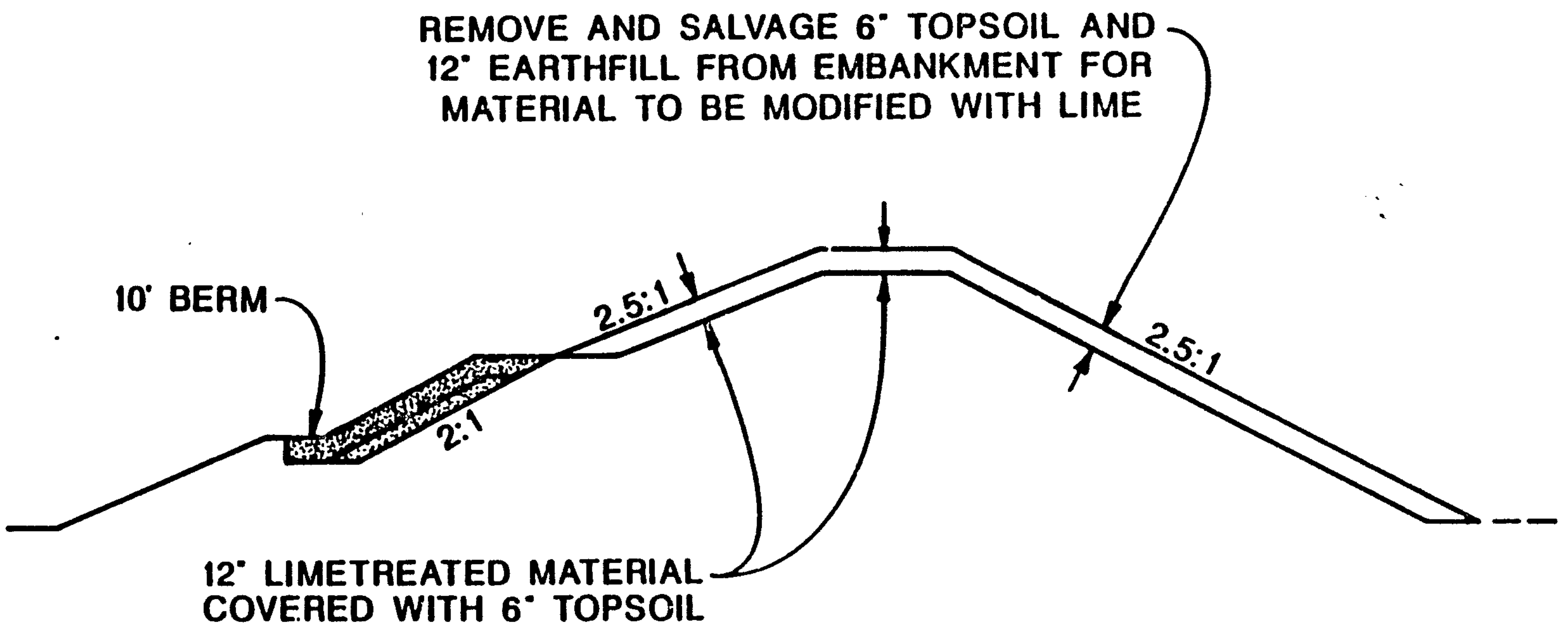

Figure 2. - Typical section used for repair of ercsion damage to Cane Creek embankment.

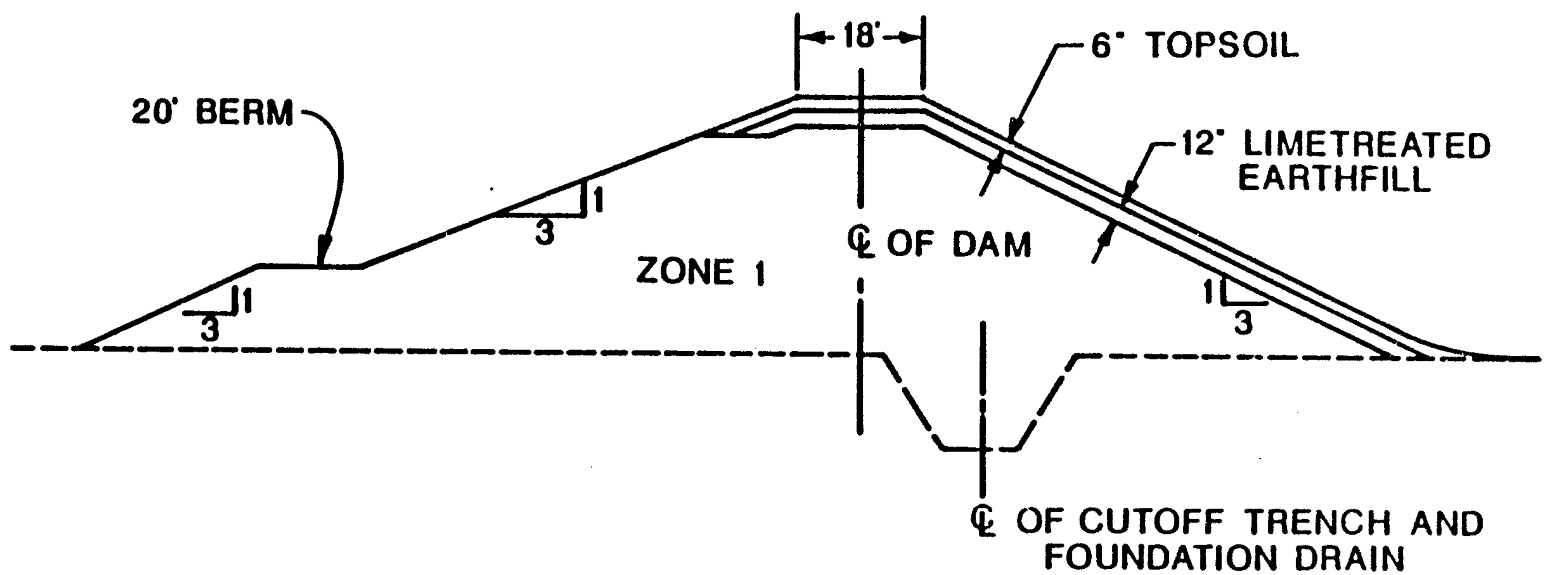

Figure 3. - Typical section used for preventive treatment of erosive soils at Stillwater Creek (Boomer Lake) embankment.

Subsidence Damage to a Canal

Subsidence caused by hydrocompaction of collapse-susceptible soils is a serious problem for agencies involved with operation and maintenance of canals. Subsidence results in local ponding of water in the canal invert, deformation of canal linings, and occasionally the failure of canal slopes. Hydrocompaction is the settlement (collapse) of certain fine-grained soils (predominantly silts) due to loading and saturation. If recognized prior to construction, the soils can be removed (over-excavation) or pre-collapsed by flooding or dynamic compaction. However, collapse-susceptible soils are sometimes difficult to recognize and structures like canals are built over them. The change in stress caused by the construction combined with the readily available source of water leads to subsidence problems.

Remedial repair procedures (Cast, 1992) generally involve overexcavation of the soils in the canal cross section, placement of some type of membrane, and reconstruction of the lining with products like fabric-formed concrete. This is a rather expensive repair procedure. Cast (1992) described an alternative procedure used by the Bureau of Reclamation to repair damaged canals and prevent future problems. The procedure involves the injection of silt into the subsided area to stabilize the foundation conditions (i.e., "compact" collapsible soils) and act as a "sealant" in the settlement cracks. Following silt injection, the damaged lining panels are removed, the slope and invert grade re-established, and new panels constructed.

In one specific case (Cast, 1992), the soil profile consisted of:

0 - 25 ft	Compacted embankment
25 - 35.3 ft	Upland Valley Alluvium (silt)
35.3 - 52 ft	Peorian Loess (silt)
52 - 58 ft	Mixed layers of poorly graded sand, silt, and clay.

The alluvium was the layer considered most collapse-susceptible. The silt injection program used conventional grouting procedures and equipment and open hole procedures between the end of the injection rod (about 21 ft deep) and a depth of 53 ft. The silt mixture (55 percent silt and 45 percent water) was injected until grout leaks appeared or surface heaving occurred or minimal grout take and high pressure occurred. Based on grout takes, surface responses, and some cone penetration testing, the collapse-susceptible soil was densified and the potential settlement was minimized. The damaged lining panels were removed, the invert and slopes were excavated and recompacted for a depth of $3\pm$ ft, and new canal lining panels were installed.

SUMMARY

Remedial repair of water retention and conveyance systems is similar to repair of any geotechnical structure; that is, there is a variety of options available to make the repairs and selection of the "best" option is a judgment-based decision dependent on the particular situation. Some of the factors that should be considered in the decision process are the type and condition of soil in the embankment or slope, the required degree of permanence of the repair, and the cost of the repair.

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LIME STABILIZATION OF LEVEE SLOPES

By Robert L. Fleming, Jr.[1] M.ASCE,
George L. Sills[2] M.ASCE, and
Edwin S. Stewart, Jr.[3]

Abstract: The U.S. Army Engineer District, Vicksburg, has developed a procedure for using lime in repairing slough slides that occur in the riverside slope of the Main Line Mississippi River Levees. Since 1964, approximately 450 main line levee slides have been repaired. After studying the cause and occurrence of these slides between 1968 and 1982 a procedure was developed for using lime to repair the slides. The history and results of those slide studies, the description of the repair procedures, and the performance of repaired slides to date are discussed in this paper.

INTRODUCTION

The U.S. Army Engineer District, Vicksburg, has approximately 1670 miles of levee to maintain. Of this, 460 miles are a part of the main line Mississippi River Levee system located in the states of Arkansas, Louisiana, and Mississippi. The levees in the main line system range in height from 25 to 40 ft. In addition to the main line system, there are approximately 1,210 miles of backwater and headwater levees that range in height from 5 to 25 ft. All of the main line levees and a large percentage of the backwater and headwater levees were constructed with riverside slopes that consist of clays (CH) and (CL) to limit through seepage. Shallow slough slides have been occurring along the riverside slopes of these main line levees for the last 60 years. Between 1964 and 1982, approximately 200 to 225 slides were repaired. In 1979 alone, a total of 41 slides were repaired and at that time represented the largest number of slides to occur in a single year.

The typical slough slide usually develops in the riverside slope which is normally steeper than the landside slope. Most riverside slopes are a 1V on 4H slope. The typical slide can be defined as a shallow slide whose maximum depth to the slip plane varies between 4 and 8 ft and whose failure is triggered by heavy rainfall after an extended period of weathering. Weathering results from desiccation and causes strains to be induced during the seasonal shrinking and swelling process. The zone of weathering that develops usually extends to a depth of 5 to 7 ft. The process and its effect on slide development is described in detail later in this paper. The slides occur primarily between the riverside crown and a point midway down the slope and range in length from 100 to 300 ft along the levee. A typical slide is shown in section and plan on Figure 1.

This paper describes the efforts to identify the causes of these slides and the methods, both past and present, that are used to repair them.

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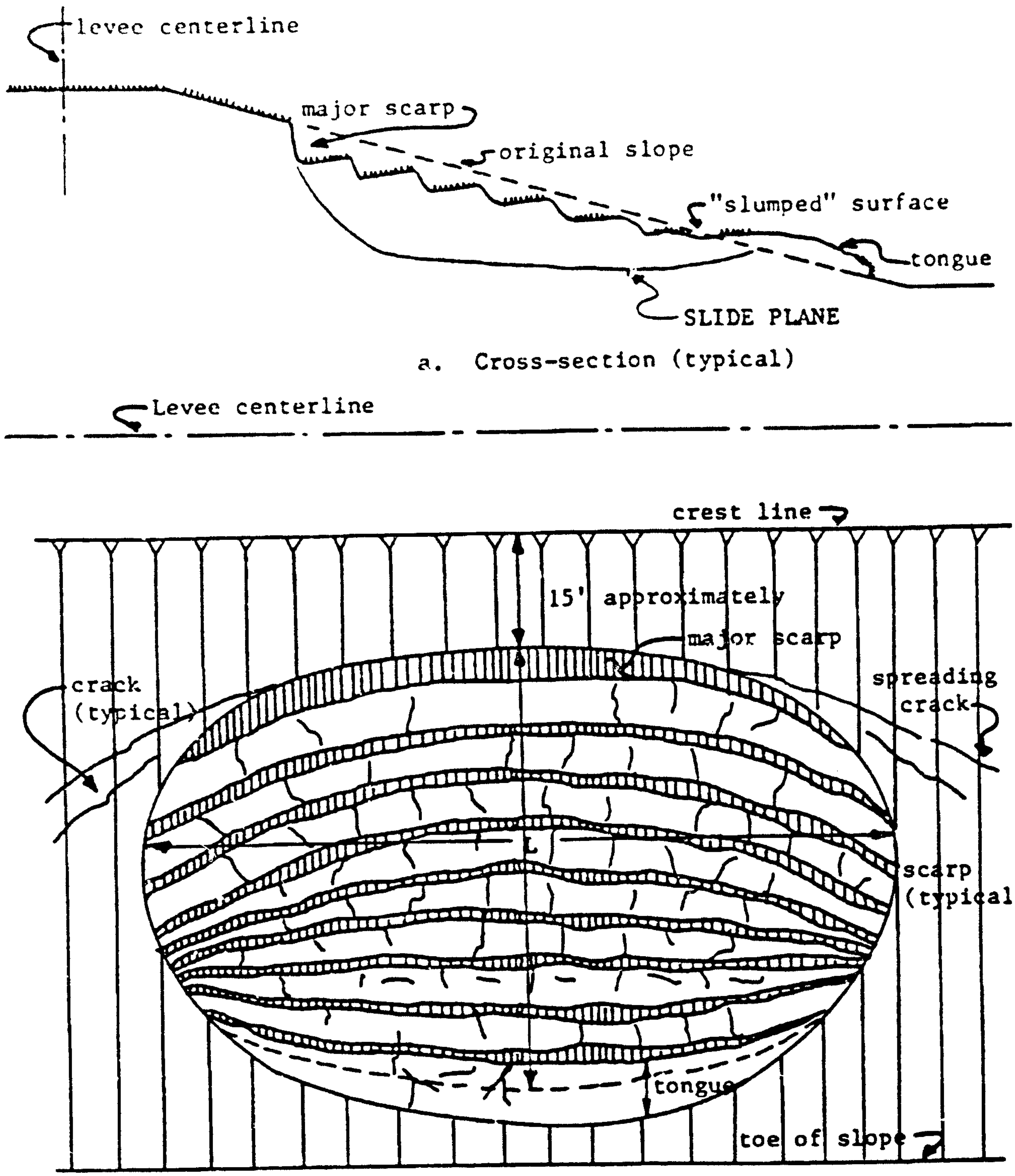

Figure 1. - Typical slough slide: (a) cross section; (b) plan view.

BACKGROUND

Prior to the early 1980's, the method of repairing these slides normally consisted of excavating the failed material and rebuilding the slopes on a flatter angle with new material from a borrow source. In some cases a berm was used to repair the slides. Although not required by the 1947 Mississippi River Commission Levee Code, it had become the policy of the Vicksburg District to construct a standard 40-ft-wide riverside buttress berm on all levees 25 ft or greater in height. This policy was unofficially adopted in 1962 in an effort to prevent slough slides. In the late 1960's and 1970's, the slides seemed to become more numerous. Also, obtaining rights-of-way and additional sources of borrow was becoming more and more costly and time consuming. It was clear that a more cost effective method of repairing the slides was needed.

The first organized effort to evaluate slough slide failures began in 1968. Several slides were trenched and a limited laboratory testing program was initiated in an attempt to determine soil parameters or site conditions unique to the areas experiencing slough slide failures. The results of these observations were summarized in an unpublished report by Larry Cooley, who at the time was Chief of the Foundation and Materials Branch. Following this report, the Vicksburg District trenched several more of these slides to obtain "undisturbed" soil samples and to observe fracture planes, crack distribution, and material composition. Observed similarities between materials and macrostructure in these slides and those described in the literature relating to long-term failures in cuts in stiff, fissured, highly plastic clays seemed to indicate that a time dependent weakening of these levee materials was occurring due to seasonal shrinking and swelling. Skempton (1978) suggested that failure would occur just before reaching the value of fully softened strength, which is equal to the peak strength of the remolded normally consolidated clay.

George Sills published a Master's Thesis (1981) entitled, "Study of Long-Term Failure in Mississippi River Levee Material." This helped to add emphasis to find a solution. Using Cooley's unpublished observations and Sills' thesis as a starting point, the Vicksburg District embarked upon an extensive field and laboratory study to relate slide susceptible areas to some soil property that designers could use in levee enlargement design to minimize the occurrence of these slides and to also use in determining a repair for these slides that would surely occur on the 460 miles of existing main line levees.

MECHANISMS OF SLOUGH SLIDE DEVELOPMENT

It was evident from data collected that the slough slides occurring along the levee systems were a result of long-term reduction in shear strength. Very little documentation existed concerning long-term failures in compacted highly plastic clays. However, similarities between failures in compacted highly plastic clays and cut slopes in stiff fissured clays suggests meaningful comparisons of the mechanism of failure can be made.

The reduction in strength in the levee embankment apparently results from weathering effects and strains induced by seasonal shrinking and swelling. During dry periods, shrinkage cracks open to a depth of 5 to 7 ft. These cracks expose the interior of the mass allowing deeper desiccation to occur and fissures to form due to irregular shrinking. Subsequently, water percolates through these cracks and fissures causing the material to swell and slake. Laboratory tests have shown that this slaking results in a permanent increase in volume which must be accompanied by increase in stress. These stress increases are concentrated along discontinuities and local over stressing occurs forming segmented slickensides in zones experiencing the largest strains. The discontinuous slip surfaces begin to interconnect at the toe of the slide and advance into the slope as the slide develops. It has been noted that the maximum depth of the slide coincides with the depth of desiccation in most cases.

The slough slides appear to be triggered by heavy rainfall after an extended period of drying. The extensive network of cracks and fissures developed by years of weathering increases the mass permeability of the embankment. When these cracks fill with water, the exposed surfaces along the cracks and fissures soften, reducing the shear strength along these discontinuities. Piezometric data obtained from this study indicate that a perched water table forms above the intact clay zone located below the weathered zone. The increase in driving weight and accompanying softening of the exposed clay combined with the progressive loss of shear strength due to long-term seasonal shrinking-swelling effects result in a slough failure.

ADDITIONAL FIELD INVESTIGATION

Sills (1981) recommended the Vicksburg District obtain additional riverside blanket borings with the intent to delineate reaches most susceptible to slough slides. It was felt that by determining the Atterberg limits of the levee material a delineation could be made since these indices are indications of the shrink swell characteristics of a soil. In an effort to identify problem areas and to search for more potentially cost effective methods of repair, an extensive riverside blanket boring program was conducted along the east bank Mississippi River levees in 1982. Over 919 borings were made to assess the extent of highly plastic clays existing on the riverside slopes. Samples were recovered for classification and Atterberg limit determinations. Borings were taken at a maximum spacing of 1,000 ft. The reach investigated began just north of Vicksburg, Mississippi, and extended up river to the limit of the Vicksburg District, which is just south of Memphis, Tennessee. The spacing was reduced to 500 ft in reaches where bulging or known sliding had occurred on the riverside slope. Borings were made with a spiral auger to a minimum depth of 5 ft. Stratum changes were noted and general samples were collected at depths of 3 and 5 ft.

It was found that approximately 50 percent of the area studied consisted of highly plastic clay (CH). From these data it appeared that a limiting value of PI may be associated with materials susceptible to slough slides on these 1V on 4H slopes. These data indicated that no slides have occurred in areas where the PI is less than 27 and very few where the PI is between 27 and 40. It should be further noted that these PI values do not necessarily correspond to the material involved in the slough slides since the data were obtained after many of these slides had been repaired and, in some cases, were extrapolated from borings adjacent to the slide area. Figure 2 is a plot of Atterberg limit indices of materials recovered directly from slides which indicates that materials susceptible to slough slides may be characterized as having a liquid limit greater than 60 and a PI greater than 40. Sills et al. (1983) reported these findings in a technical report prepared by the Vicksburg District dated 1983.

ADDITIONAL STUDIES

Additional research into this slide problem has revealed that in areas protected from the weathering process, no slide will occur. The best example of this is on a rock protected dam slope where the bedding gravel and rock tend to protect the soil from weathering on the lake side. Very few slides occurred on the lake side where as numerous slides occurred on the flatter land side. The protection riprap and bedding gravel offer from the weathering process was clearly demonstrated in the Lake Chicot Pumping Plant outlet channel near Lake Village, Arkansas. The Lake Chicot Pumping Plant was built in the Main Line Mississippi River Levee across the river from Greenville, Mississippi. For underseepage control, a soil bentonite slurry trench cutoff was constructed. The structure is tied to the slurry trench cutoff with a neoprene coated nylon fabric that is connected to the structure and anchored into the top of the slurry trench.

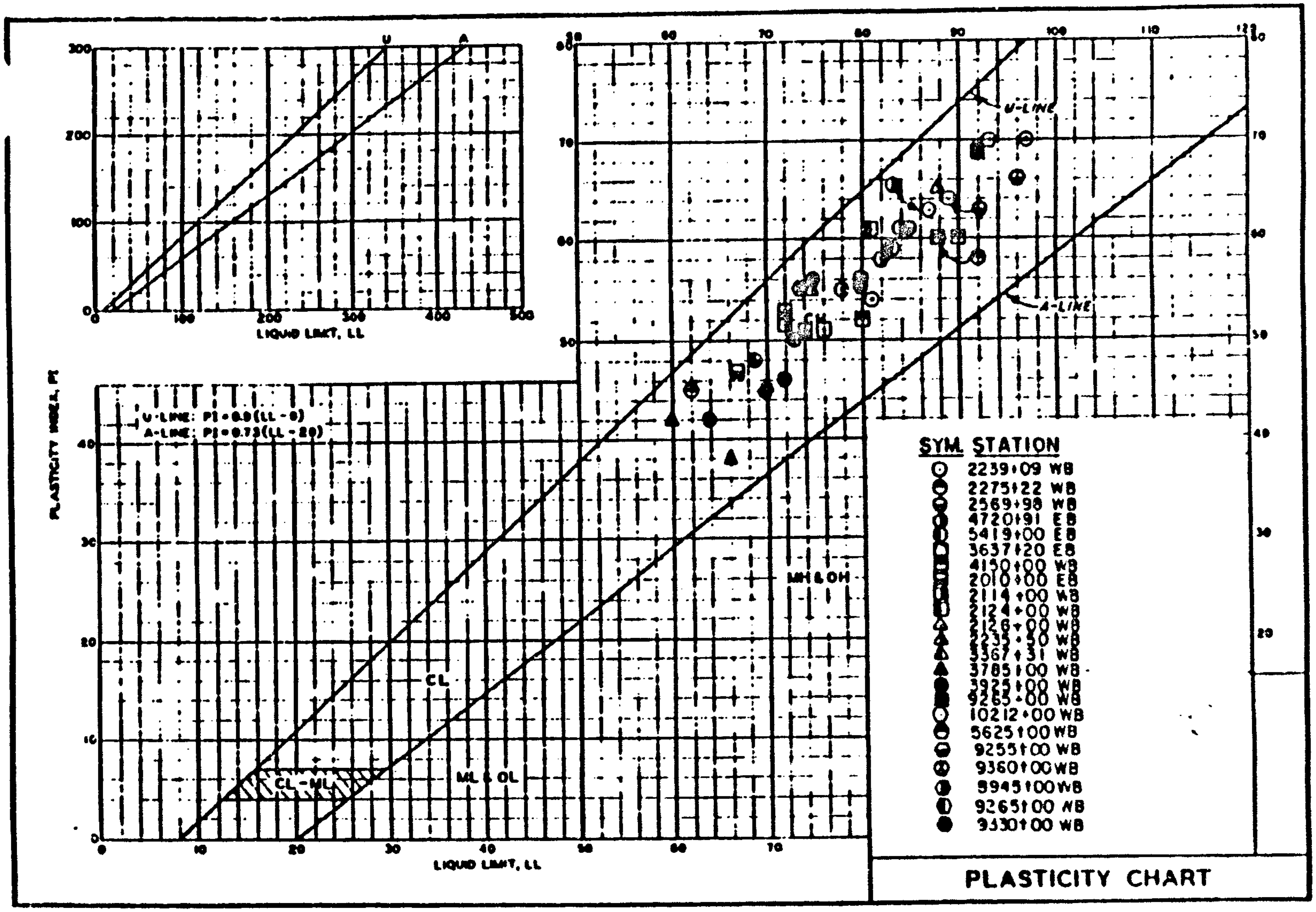

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Figure 2. - Limit data from slough slides.

The impervious fabric is covered with a 5-ft-thick clay (CH) blanket. In the riverside outlet channel, the fabric and clay blanket were placed on a 1V on 3H slope. The channel slopes nearest the structure were covered with bedding gravel and riprap. There was a portion of the channel slope capped by the impervious fabric and clay blanket that was not covered with bedding gravel and riprap. In 1986, less than one year after completion of the project, a slide occurred in that portion of the clay blanket that was not covered with gravel and riprap. The slip plane was at the interface of the fabric and clay material. The slide was repaired by replacing the failed material with similar type material. Bedding gravel and riprap were then placed on the remainder of the channel that had the clay blanket covering the impervious fabric. Since that time, we have experienced no further problems. This has been attributed to the belief that the bedding gravel and riprap greatly reduced the weathering process by maintaining a fairly constant moisture content in the clay blanket.

SLIDE REPAIR USING LIME

Since 1985 the Vicksburg District has been using lime where possible to repair slough slides because material having a P.I. of less than 40 is usually not available within a reasonable haul distance of slide repairs. So far, lime has been used in the repair of approximately 142 slides of various sizes.

The first step in the repair process is to obtain representative bag samples from each slide. A suitable area is chosen in the slide itself and all topsoil is removed. A sample of the underlying soil is placed in a bag and transported to the laboratory for soil tests which will be used to determine the amount of lime to be used in the repair.

In the laboratory, Atterberg limits are determined for the soil in its natural state and with varying amounts of lime added. The results of these tests are plotted and the optimum amount of lime is chosen. The optimum percentage of lime is the minimum amount that will lower the plasticity index below 40 and produce a Ph of approximately 12. Once the required lime percentage is determined the remaining portion of the bag sample is prepared for a compaction test to determine the optimum density and water content with the lime added. The results of the compaction tests are plotted and used for quality control in the field.

CONSTRUCTION PROCEDURES

The first order of work in the field is to remove all topsoil from the slide. The topsoil is stockpiled near the slide and will be replaced when the repair has been completed.

The slide material is then excavated to a point "about 2 ft" below the failure plane. The slip plane is usually very easy to find in the field. When the size of the slide being repaired permits, the lime-soil mixture can be processed in lifts in the embankment and compacted; therefore, the excavated material can be stockpiled at the toe the slide. If the slide being repaired is small, a soil processing area located adjacent to the slide may be required. The topsoil in the slide area should be removed from the soil processing area and stockpiled for reuse. Excavated material from the slide should be spread evenly over the soil processing area in a lift 6 to 8 inches in thickness.

The specified amount of lime should be spread evenly over the surface of the soil to be mixed. After application of the lime, a light application of water should be used to prevent dusting and achieve a good distribution of the lime when mixed. Several passes of a rotary pulverizer is then used to mix the soil and lime. In clays with a very high plasticity index, a double application of lime must be used to thoroughly incorporate the lime with the soil. If a double application of lime is used, the first application should be one-half the total amount of lime to be applied. It should be mixed by at least one pass of a rotary pulverizer and then sealed with a steel roller and allowed to cure for 48 hours. After curing, the remainder of the lime should be added and mixed.

After final mixing of the lime with the soil, the mixture is placed in the embankment in 6 to 8 inch lifts and compacted by at least three passes of a dozer. After the required number of passes, the water content and density are checked. If adequate density is not obtained, the water content is adjusted or additional compaction is applied. When placement of the lime-soil mixture is completed, the surface is dressed to final grade.

Topsoil is then replaced and fertilizer is uniformly distributed over the surface and worked into the soil by light disking. After fertilizing, grass seed is spread over all disturbed areas.

The surface is then sealed with a steel roller to retain moisture for better seed germination. The use of lime in the repair of slough slides has thus far proven to be much more economical and effective than flattening the slopes with berms. The slides are reconstructed to the original grade using material from the slide itself thereby reducing the right-of-way requirements and the need for additional borrow.

Based on the experiences described above that indicate riprap and bedding gravel placed on a clay slope tend to greatly reduce the weathering process, the Vicksburg District in 1990 began to use a modified procedure using lime to repair these slides. To date, a total of 14 slides have been repaired using a capping process. The slide area and failure plane are removed in the same manner as described above. The major difference is that only the outer 3 ft of soil on the reconstructed slope are lime treated instead of treating the entire mass. We are presently treating approximately 10 percent of the slides repaired each year in this manner to test the effectiveness of this capping process. It is our opinion that this will provide a protective cap of treated soil that will be resistant to the weathering process that cause the slides to occur.

CONCLUSIONS

The use of lime has proved to be a cost effective method of repairing slough slides on the Main Line Mississippi River Levees. Costs associated with obtaining rights-of-way and extra borrow material has been eliminated since all work can be carried on in the existing rights-of-way limits. The average cost of a slide repair using lime has been reduced to approximately $20,000 to $25,000. If the process of capping proves to be successful, these costs can be reduced even further.

ACKNOWLEDGEMENTS

The authors would like to express their thanks to the U.S. Army Corps of Engineers, Vicksburg District, for supporting these efforts. The authors would also like to thank Mr. Larry Cooley and Mr. Alexis E. Templeton for their efforts as a part of this study, Mr. McKinley Harris for preparing the figures, and Mrs. Sue L. Richardson for typing this paper.

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CONSTRUCTION CONTROL OF RCC AND SOIL-CEMENT USING THE HEAT OF NEUTRALIZATION TEST, NUCLEAR MOISTURE DENSITY GAUGE, VIBRATING COMPACTION HAMMER, AND VEBE

By E. Kunzer[1] and A. Benavidez[2]

Abstract: Soil-cement and roller-compacted concrete (RCC) require a combination of soil and concrete construction control methods. These methods need to be simple and quick because of field conditions, rapid placing rates, and need for fast feedback to the inspectors and the contractor. The Bureau of Reclamation (Reclamation) uses a modified heat of neutralization test to monitor cement content, vibrating table (Vebe) tests to check consistency and wet density of RCC, and a combination of sand cone density tests and nuclear moisture-density gauge measurements to monitor compaction. A vibrating compaction hammer is being evaluated to determine maximum dry unit weight and to prepare compressive strength test cylinders. Data are presented on the advantages and disadvantages of the various methods and their applicability to soil-cement and RCC.

INTRODUCTION

As construction materials, soil-cement and roller-compacted concrete (RCC) are a transition between soil and concrete. They combine the workability of soil when freshly placed with the increasing strength of conventional concrete as they cure. Soil-cement and RCC are used for such things as road subbases, backfill, pipe bedding, channel and reservoir linings, protective blankets, and slope protection. Also, RCC has been used for gravity concrete dam construction. Mixture proportions differ between soil-cement and RCC primarily in aggregate grading, consistency, and cement plus pozzolan content. The mixture proportions range from a stabilized soil mix with no plus 4.75-mm (No. 4) sieve material to a lean concrete mix with up to 55 percent plus 4.75-mm (No. 4) size material that may include water-reducing admixtures and pozzolan. Conventional soil and concrete placing methods and construction control must be adapted to deal with the special characteristics resulting from this range of materials and mixture proportions.

Soil-cement and RCC can contain 0 to 30 percent fine, nonplastic soil; enough water to wet the mixture to within ±1 percent of optimum water content; 3 to 16 percent portland cement; and may contain up to 55 percent minus 51-mm (2-inch) to plus 4.75-mm (No. 4) aggregate. All proportions are based on dry mass of soil and/or aggregate. RCC tends to have less fines and more coarse aggregate than soil-cement. After placing, the mixture is usually compacted to 98 to 100 percent of maximum dry unit weight (soil-cement) or air-free wet density (RCC) using modified earth fill placement techniques. Soil-cement is moist-cured for 7 days. When used, soil-cement and RCC are chosen because of workability, adaptability to a wide range of applications, rapid construction capability, and generally lower cost than alternatives. Reclamation uses soil-cement and RCC for slope protection when a source of high quality riprap is not economically available, or for overtopping protection. RCC is also used as mass concrete in dams and other large placements.

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Soil-cement and RCC placement is very rapid, field conditions can be demanding, and trained personnel are usually scarce. Because of these circumstances, inspectors and contractors need quick and accurate construction control tests requiring a minimum number of personnel using equipment that remains accurate under field conditions. To meet these testing needs, Reclamation uses a modified heat of neutralization test to monitor cement content of freshly mixed soil-cement, a vibrating table (Vebe) test to check consistency and wet density of RCC, and sand cone density tests and/or nuclear moisture-density gauge measurements to monitor compaction of both soil-cement and RCC. Reclamation is also evaluating the use of a vibrating compaction hammer to replace compaction machines presently used to determine maximum dry unit weight of soil-cement, and to prepare compressive strength test cylinders for soil-cement and RCC. This paper discusses the use of these tests (table 1) and presents some preliminary data from the vibrating hammer evaluations.

Table 1. - Mixtures/tests comparison.

Mixtures/tests	RCC	Soil-cement
Proportions	Specific grading usually has a high percentage of coarse aggregate and may include pozzolan and/or nonplastic fine soil	Gradation which includes 3-30 % pit run, nonplastic soil and may include up to 55 percent coarse aggregate
Vebe apparatus	Used to check consistency, determine wet density, and prepare compressive strength specimens	Not used - soil-cement often too stiff
Nuclear moisture-density gauge	Calibrated with RCC block from site	Calibrated with density block and checked by sand cone test
Cement content	ASTM titration method used	ASTM titration method or Heat of Neutralization test used
Compressive strength specimen preparation	Vebe test used, Vibrating hammer may have application	Impact compaction or rodding used depending on gradation; vibratory hammer may have application

HEAT OF NEUTRALIZATION TEST

Reclamation's revised heat of neutralization method (USBR test procedure) can be used to accurately and quickly determine the cement content of freshly mixed soil-cement containing from 3 to 15 percent cement by dry mass of soil and up to 55 percent plus 4.75-mm (No. 4) material. If the mix contains more than 13 to 15 percent cement or a cement plus fly ash combination, the specimen may gel during testing. Cement content can be determined within 15 to 20 minutes rather than 30 to 40 minutes for titration. As shown in tables 2 through 5, the heat of neutralization test is accurate to within ±1 percent of actual cement content based on dry mass of total material. Single operator precision of 8.9 kg/m^3 (15 lb/yd^3) for soil-cement compares favorably with 9.2 kg/m^3 (15.5 lb/yd^3) for the titration method (ASTM C 1078). Unlike the titration method, heat of neutralization does not require separation of the plus 4.75-mm (No. 4) material or the time-consuming task of wet sieving the soil-cement mix to obtain test specimens of the appropriate size and grading characteristics, nor does it rely on a somewhat subjective color change to determine the end of the procedure.

Table 2. - Summary of interlaboratory test program for calibration specimens with 5 percent plus 4.75-mm (No. 4) material[1]

Cement content[2] (%)	Temperature rise (°C)			Standard deviation (°C)	
	Average	Maximum	Minimum	Within-lab	Between-lab
5	10.4	11.8	8.4	0.5	0.7
7	14.3	15.7	13.4	0.3	0.6
9	18.2	19.7	17.2	0.3	0.7

[1] Based on 90 tests with a correlation coefficient of 0.981 (Scavuzzo, 1991)
[2] Cement content known by tester

Table 3. - Summary of interlaboratory test program for calibration specimens with 50 percent plus 4.75-mm (No. 4) material[1]

Cement content[2] (%)	Temperature rise (°C)			Standard deviation (°C)	
	Average	Maximum	Minimum	Within-lab	Between-lab
5	11.2	12.4	8.9	0.6	0.9
7	15.7	17.0	14.8	0.4	0.6
9	19.5	21.9	18.3	0.6	0.9

[1] Based on 72 tests with a correlation coefficient of 0.974 (Scavuzzo, 1991)
[2] Cement content known by tester

Table 4. - Summary of interlaboratory test program for specimens with 5 percent plus 4.75-mm (No. 4) material[1]

Cement content[2] (%)	Cement content (%)			Standard deviation (%)	
	Average	Maximum	Minimum	Within-lab	Between-lab
6	6.0	6.4	5.7	0.1	0.2
7	7.1	7.5	6.4	0.1	0.3
8	8.2	8.6	7.7	0.1	0.2

[1] Based on 90 tests with a correlation coefficient of 0.981 (Scavuzzo, 1991)
[2] Cement content unknown to tester

Table 5. - Summary of interlaboratory test program for specimens with 50 percent plus 4.75-mm (No. 4) material[1]

Cement content[2] (%)	Cement content (%)			Standard deviation (%)	
	Average	Maximum	Minimum	Within-lab	between-lab
6	5.9	6.5	5.3	0.1	0.3
7	7.1	7.4	6.5	0.2	0.3
8	8.0	8.5	7.8	0.1	0.2

[1] Based on 72 tests with a correlation coefficient of 0.974 (Scavuzzo, 1991).
[2] Cement content unknown to tester.

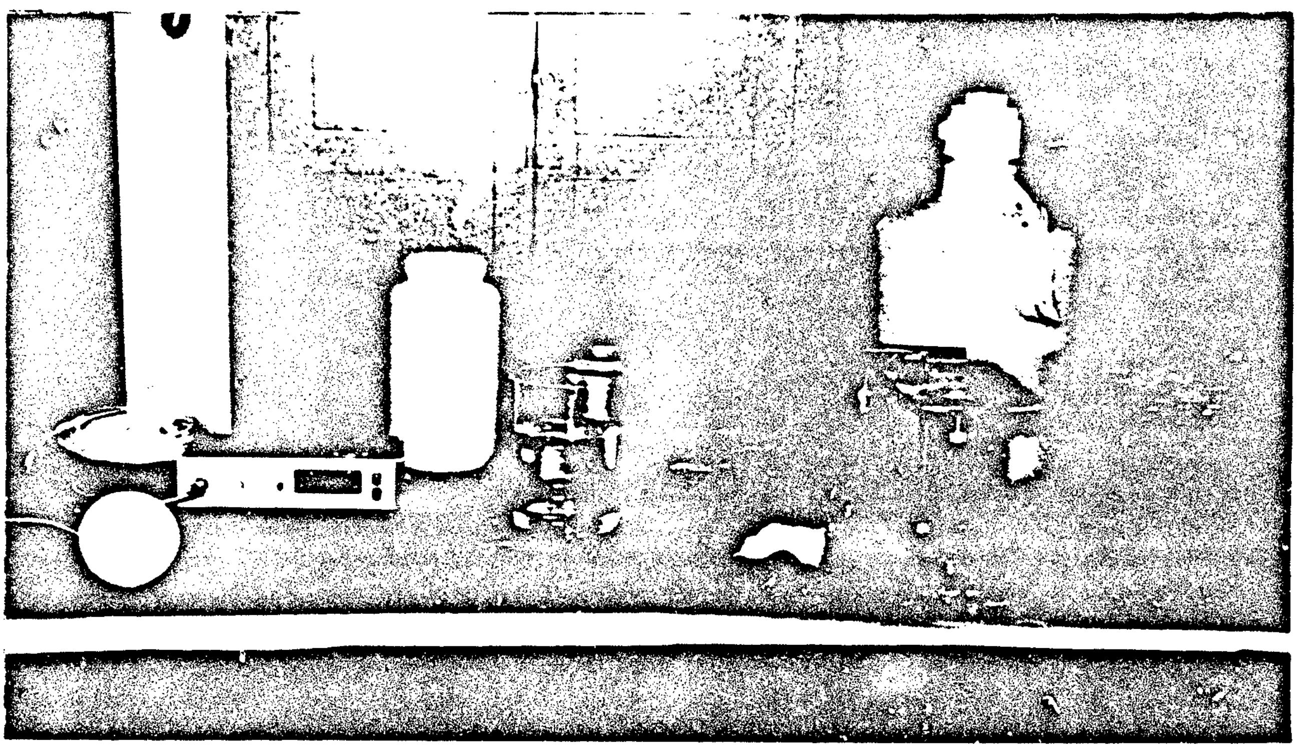

Figure 1. - Heat of neutralization apparatus.

Equipment for the test is shown on figure 1. A 3.8-L (1.0-gal) leakproof, screw-top plastic container is used for mixing a fresh soil-cement specimen with a glacial acetic acid-sodium acetate buffer solution. One lid for the container is pierced by a digital thermometer accurate to 0.1 °C. A funnel and scoop are used to transfer the soil-cement from the covered collection pail (not shown) to the mixing container. The plastic carboy with spigot is used to dispense the glacial acetic acid and a second carboy may be used to hold distilled water for making the buffer solution. The buffer solution is mixed in the 3000 ml beaker. Glacial acetic acid is highly corrosive and butyl gloves should be used to handle the acetic acid-sodium acetate buffer solution. A stopwatch accurate to 1 second is needed to time the temperature readings and the mixing time of the soil-cement and buffer solution. Angle iron stands are used to securely hold the inverted plastic container while the temperature of the mixture is being measured. Not shown are the balance accurate to 0.01 kg or 0.1 lbm for obtaining the mass of the soil-cement specimen and the balance accurate to the nearest 0.1 g for obtaining the mass of sodium acetate and glacial acetic acid for the buffer solution.

The basic procedure for the heat of neutralization test is as follows:

- Obtain a 1.5-kg (3.3-lbm) representative sample of freshuy mixed soil-cement and place it in a leakproof plastic container.

- Screw the lid which is pierced by a digital thermometer probe on the container and invert the container on the holder so the soil-cement mixture covers the thermometer probe. After 1 minute, measure and record the temperature of the soil-cement specimen.

- Determine and record the temperature of the glacial acetic acid-sodium acetate buffer solution separately.

- Pour the glacial acetic acid-sodium acetate buffer solution into the container with the soil-cement, screw the lid on and vigorously mix the soil-cement/buffer solution by repeatedly upending the sample through 180° for 4 minutes.

- After mixing, invert the container of soil-cement/buffer mix to cover the thermometer probe. One minute after inversion, measure and record the temperature of the soil-cement/buffer solution mixture.

- Subtract the average of the initial temperatures of the soil-cement and acetic acid-sodium acetate buffer solution from the final temperature of the mixture to determine the heat of neutralization.

- Obtain the cement content from the calibration curve for the site. The calibration curve is initially established using the same procedure given above and with the specified grading of site materials but with three known cement contents: one at the specified cement content mixture proportion and one each at cement contents plus and minus 2 percentage points of specified (fig. 2). By using site specific materials and gradings, possible high amounts of calcium in the soil or aggregate are accounted for in the calibration curve.

- Check and possibly repeat the calibration curve if there is a significant change in gradation or stockpile material.

- Make up the buffer solution for one test as follows: dissolve 225 g of sodium acetate in 500 ml of distilled water. When the sodium acetate has dissolved, add 360 g of glacial acetic acid to the acetate-water solution and enough distilled water to bring the final volume to 1.5 L.

Buffer solution can be prepared for more than one test at a time, but because of changes in ion concentrations, the buffer solution should not be used if it is more than 24 hours old. When freshly mixed, the buffer solution has a pH of approximately 2 and should be handled with protective gloves. However, after the acetic acid reacts with the calcium in the cement, the pH is approximately 6.5 and the mixture can be safely disposed of with waste soil-cement from the batch plant. Any excess buffer solution should be mixed with soil-cement prior to disposal.

CEMENT CONTENT - HEAT OF NEUTRALIZATION
CALIBRATION DATA   Designation USBR 5840-Draft

Sample or Test No 1	Project EXAMPLE	Feature CALIBRATION
Location LABORATORY MIX	Test Specimen Mass (3.30 lbm) 1.5 Kg	Buffer (3.64 lbm) 1.7 Kg
Tested by Date	Computed by Date	Checked by Date

Specimen No	1	2	3	4	5	6	7	8	9
Cement %	7	7	7	9	9	9	11	11	11
Water %	6	6	6	6	6	6	6	6	6
Gravel %	50.3	50.3	50.3	50.3	50.3	50.3	50.3	50.3	50.3
(1) Buffer Temperature °C	19.6	19.6	19.7	20.2	20.1	20.2	20.7	20.8	20.9
(2) Soil-Cement Temperature °C	20.2	20.5	20.6	20.3	20.6	20.8	21.1	21.1	21.9
(3) Average Temperature °C [(1)+(2)]/2	19.9	20.[illegible]	20.2	20.3	20.4	20.5	20.9	21.0	21.4
(4) Mixture Temperature °C	35.7	35.3	35.2	39.7	40.2	39.7	43.4	44.0	44.2
(5) Temperature Difference °C (4)-(3)	15.8	15.2	15.0	19.4	19.8	19.2	22.5	23.0	22.8
(6) Average Temperature Difference °C	15.3			19.5			22.8		

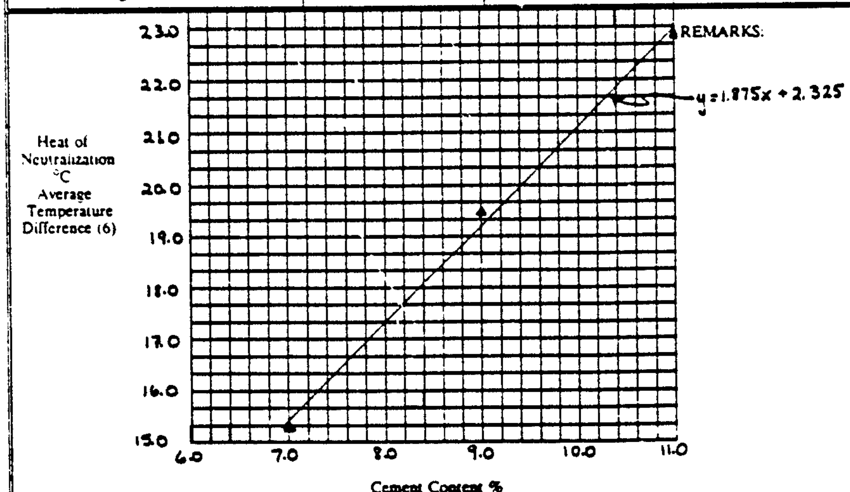

Figure 2. - Heat of neutralization calibration curve.

This test has several advantages over titration testing. It can be performed at the batch plant or in the field lab within 15 to 20 minutes with a minimum of equipment. It is simple to perform and accurate within ±1 percent of actual cement content based on total dry mass of the test specimen. It requires no washing or sieving of the soil-cement and the end of the test is a set time rather than a color change. Calibration for the method allows for local variation in calcium content of the soil by using the same material that will be used in construction. Waste from the test can be safely discarded with waste material from the batch plant.

NUCLEAR MOISTURE-DENSITY GAUGE

The nuclear moisture-density gauge can be used for rapid, nondestructive determination of in-place density and moisture content after compaction of both RCC and soil-cement. A nuclear moisture-density gauge measures the average backscatter surface moisture content and in-place or backscatter density of an RCC or soil-cement lift using electrons emitted from a radioactive source sealed inside the lower end of the probe (fig. 3). The probe is inserted into a pre-formed hole in an RCC or soil-cement lift and the photons are counted by a detector in the gauge housing (Troxler Laboratories, 1984). The density determination is heavily weighted toward the upper 3/4 of the lift so 98 to 99 percent compaction is usually specified to ensure sufficient compaction along the lift line (Dolen et al., 1988). Possible interferences with an accurate reading include voids on the surface under the base plate of the instrument, chemical composition of the materials, presence of reinforcing steel, gauge being placed too close to the wall of a trench or pit, and insufficient count time. Because of these possible interferences and because of the radiation source, the use of a nuclear moisture-density gauge requires trained, licensed personnel to use the equipment and the requisite radiation safety program.

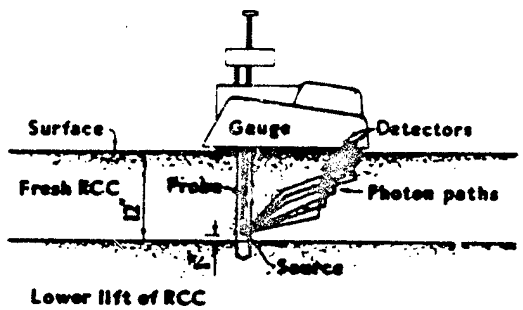

Figure 3. - Schematic of nuclear moisture-density gauge operat'

When using the nuclear moisture-density gauge with soil-cement, the gauge calibration is checked daily on the reference calibration block included with the gauge (USBR 7230, 1990). Site specific correction factors may be calculated by comparing the calibrated nuclear gauge density with an adjacent sand cone density determination. Figure 4 shows a comparison between nuclear moisture-density gauge and sand cone density determinations taken at the same time from the same lift during construction of Deer Flat Dams in Idaho.

When using a nuclear moisture-density gauge for RCC, the gauge is checked against an RCC standard of known density and a correction factor is entered into the gauge (ASTM C 1040, 1991). Sand cone density tests are generally not used with RCC because the sides of a density hole tend to slump during testing which results in a higher apparent density.

The nuclear moisture-density gauge test takes only 10 to 15 minutes to complete compared to 20 to 40 minutes for the sand cone and requires no lab facilities. It is usually preferable to establish the nuclear moisture-density gauge correction factor and then check it against a sand cone density determination a few times a day. This check should include a moisture content determination since the nuclear moisture-

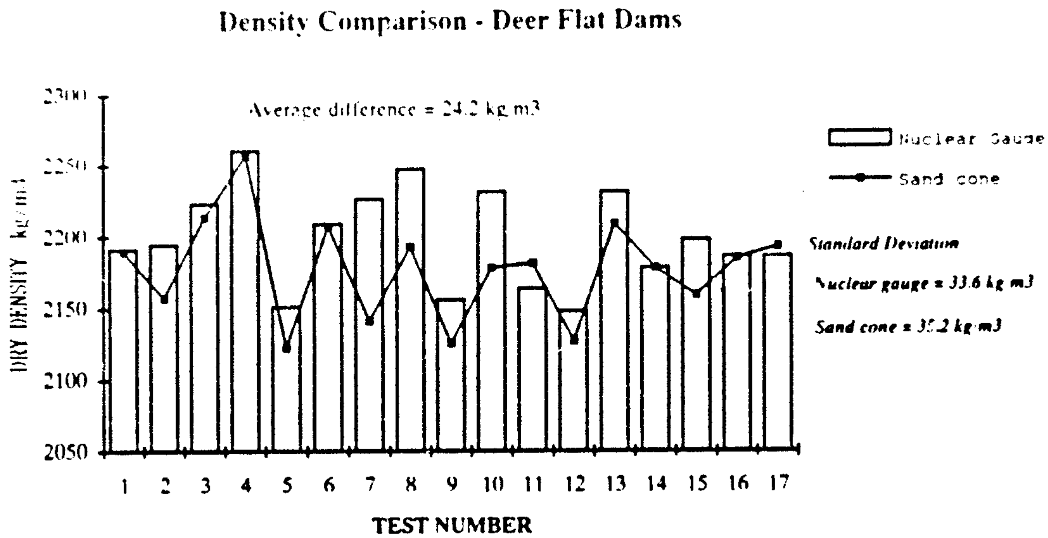

Figure 4. - Nuclear moisture-density gauge and sand cone density comparisons.

density gauge is measuring only surface moisture and the reading may be further skewed by the presence of free hydrogen or hydroxyl ions in the compacted mixture.

VEBE TESTS

The vibrating table (Vebe apparatus) used for RCC testing produces a sinusoidal motion with a frequency of 60 ± 1.67 Hz (3600 ± 100 vibrations per minute) and a double amplitude of 0.43 ± 0.08 mm (0.017 ± 0.003 in) when calibrated with a 27.2-kg (60.0-lbm) mass bolted to the center of the table. The Vebe can be used in construction control of RCC to determine the consistency and wet density of freshly mixed RCC where rodding or internal vibration cannot properly consolidate concrete of this consistency (ASTM C 1170, 1991). It can also be used to prepare RCC compressive strength test specimens (ASTM C 1176, 1991). The Vebe test is not used for soil-cement mixtures which are too stiff to consolidate fully using this method.

The Vebe consistency test provides a useful indication of the workability of RCC; much like the slump test provides an indication of the workability of conventional concrete (Dolen, 1991). To perform the consistency and density tests, place a known mass of RCC in a 0.0094-m^3 (1/3-ft^3) cylindrical mold of known mass. Attach the mold to the vibrating table, place a 23-kg (50-lbm) surcharge over the uncompacted RCC and vibrate the mold (fig. 5). Vibrate the mold until the specimen is fully consolidated as indicated by formation of a ring of mortar around the perimeter of the surcharge (fig. 6). The time from start of vibration until a complete ring of mortar appears is the "Vebe time." A Vebe time of approximately 5 seconds is similar to zero or "no-slump" concrete. Tests by Reclamation and others have shown that RCC with Vebe times of 10 to 30 seconds consolidates with approximately six roller passes using a dual drum vibrating roller and segregates less than mixtures with higher Vebe times (Dolen, 1991). Vebe time is affected by water content, maximum size aggregate (MSA), sand content, and percent of minus 75-µm (No.200) material in the mix.

After the Vebe time is recorded, determine the maximum wet density by removing the surcharge and continuing to vibrate the specimen for 10 seconds. Determine and record the mass of the sample, mold, and a plastic cover plate. Gently pour water onto the specimen until it forms a meniscus over the top of the mold. Determine the temperature of the water for density calculations and then use the plate to carefully strike off any excess water and to cover the mold. Determine the mass of the mold, specimen, plate, and water and calculate the density of the specimen.

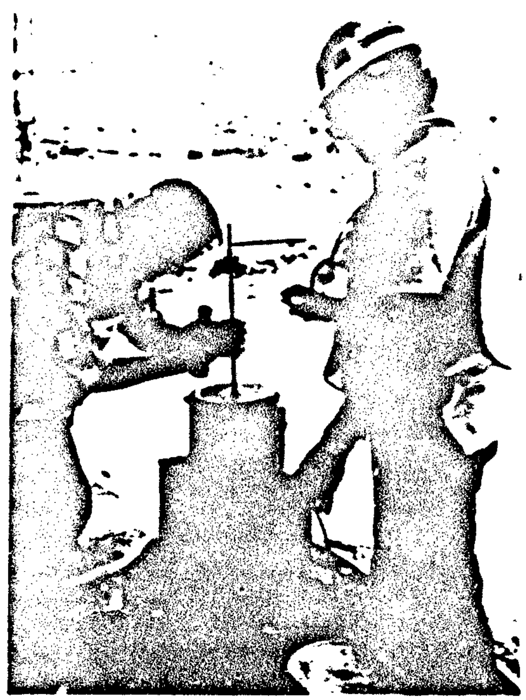

Figure 5. - Determining Vebe-time with 23-kg (50-lbm) surcharge.

The Vebe is also used to make strength test specimens. These specimens are made using a 150- by 300-mm (6- by 12-inch) metal or plastic mold which is attached to the Vebe apparatus. Place RCC in three separate lifts by vibrating each lift under a 9.1 ± 0.25 kg (20 ± 0.5 lbm) surcharge until a mortar ring forms around the perimeter of the surcharge

The Vebe must be securely anchored and should be calibrated regularly to ensure proper frequency and amplitude of the platform. When this is done, the Vebe allows rapid determination of RCC consistency and density and can fully consolidate RCC specimens for compressive strength testing. Testing and preparation time is similar to that of construction control for conventional concrete. The Vebe is not used for testing soil-cement

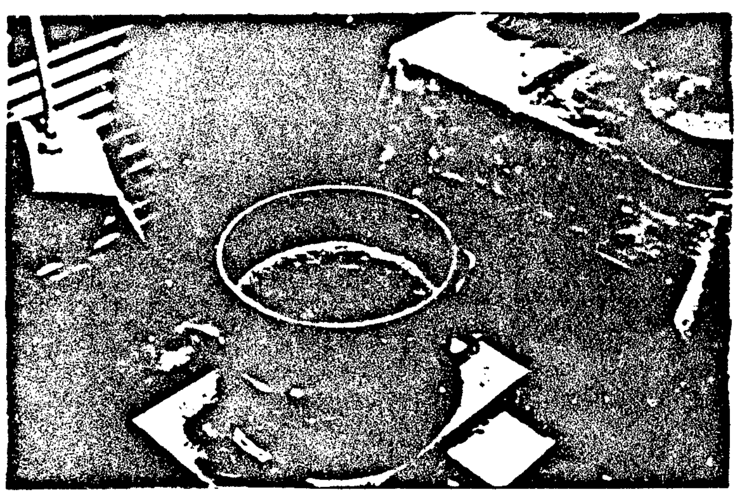

Figure 6. - Mortar paste ring in Vebe mold after vibration.

mixtures with a high pit run fines content because of incomplete consolidation and problems with formation of the mortar ring.

VIBRATING HAMMER

The vibratory hammer is a rugged, simple field instrument which has possibilities for determining the maximum dry density of cohesionless materials and preparing test specimens for RCC and soil-cement. The core of this apparatus is of the same type currently used for drilling and/or chipping concrete and other structural material.

Reclamation constantly evaluates new testing equipment and procedures to obtain optimum efficiency for field and laboratory operations. A possible replacement method for vibratory tables used to determine maximum dry density of cohesionless materials (relative density test) is needed because of problems with premature failure of table parts and sensitivity to line voltage and amperage fluctuations (Selig and Ladd, 1973). The British Standards Institute (BSI) has adopted vibrating hammers for densification testing of cohesionless soils and graded aggregates (BS 1924, BS 5835, and BS 1377).

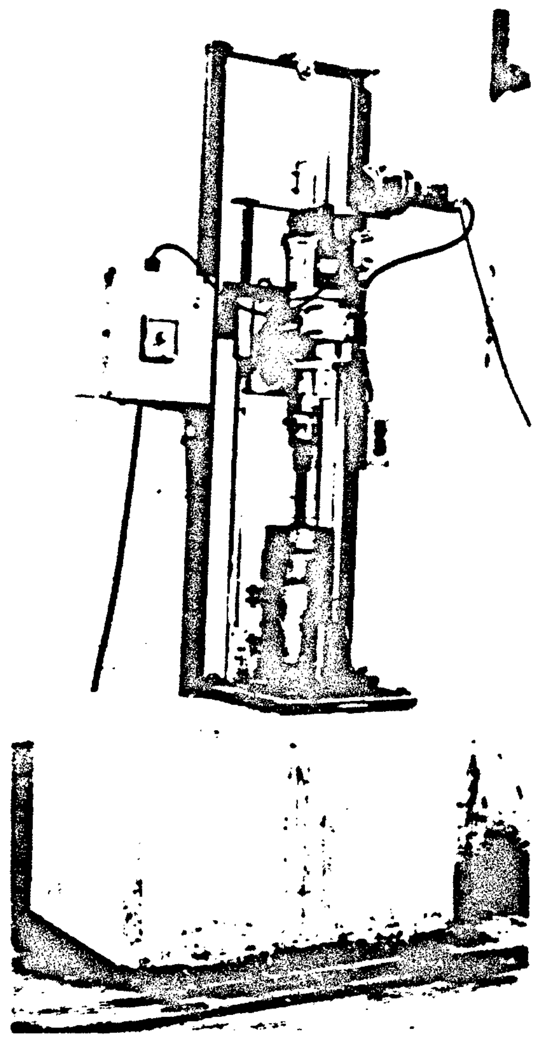

Figure 7. - Vibrating hammer compaction apparatus.

Reclamation is currently investigating the use of vibrating hammers for determining the maximum dry unit weight of cohesionless soils, preparing soil-cement and RCC test specimens, and determining wet density of soil-cement and RCC. Two sizes of Kango™ vibrating hammer are being studied. One is model 638 rated at 750 watts and delivering 2800 blows per minute, and the other is model 950X rated at 1020 watts and delivering 2000 blows per minute. The hammers are mounted in a compaction rig attached to a concrete block (fig. 7). The original design was adapted by addition of an electric winch to aid in raising and lowering the compaction hammer and an automatic timer to increase accuracy.

Initial results using the Kango™ hammer model 638 for maximum density determinations (table 6), indicate good correlation with results using an accepted standard (USBR 5330, 1990). The complete test results are forthcoming (Benavidez and Young in publication).

Table 6. - Comparison of relative density table and vibratory hammer compaction*

Test procedure	Test ID	Average dry unit weight kg/m³ (lbf/ft³)	Standard deviation kg/m³ (lbf/ft³)	Coefficient of variation (%)
USBR 5530[1]	A	1862.9 (116.3)	14.4 (0.9)	0.8
USBR 5530	B	1965.4 (122.7)	19.2 (1.2)	0.5
USBR 5530	C	1814.8 (113.3)	9.6 (0.6)	0.5
USBR 5530	D	2050.3 (128.0)	14.4 (0.9)	0.7
BS 1924[2]	A	1870.9 (116.8)	9.6 (0.6)	0.5
BS 1924	B	1967.0 (122.8)	16.0 (1.0)	0.8
BS 1924	C	1885.3 (117.7)	3.2 (0.2)	0.2
BS 1924	D	2144.8 (133.9)	14.4 (0.9)	0.7

* Based on summary table (Benavidez and Young in publication)
[1] *U.S. Bureau of Reclamation Earth Manual, Part 2,* Third Edition 1990 "Procedure for Determining the Maximum Index Unit Weight of Cohesionless Soils"
[2] British Standards Institute, BS 1924, 1975 *Methods of Test for Stabilized Soils*

At Upper Deer Flat Dam in Idaho, the Kango™ vibrating hammer model 950X and standard impact hammer compaction equipment were used for preparing soil-cement compressive strength specimens. The impact hammer equipment (fig. 8) uses a 4.5-kg (10-lbm) rammer with an 18-inch drop on a total of 6 lifts at 50 blows per lift in a 150- by 300-mm (6- by 12-inch) mold. At Deer Flat Dams, two vibrating hammer specimens could usually be prepared in the time it took to prepare one impact hammer specimen. As shown on figures 9 and 10, specimens compacted with the vibrating hammer had a sightly higher average dry unit weight and higher average 28-day compressive strength than those compacted with the standard impact compaction device 2223 kg/m³ (139 lb/ft³) versus 2196 kg/m³ (137 lb/ft³) for dry unit weight and 228 kg/cm² (3244 lb/in²) versus 206 kg/cm² (2928 lb/in²) for compressive strength).

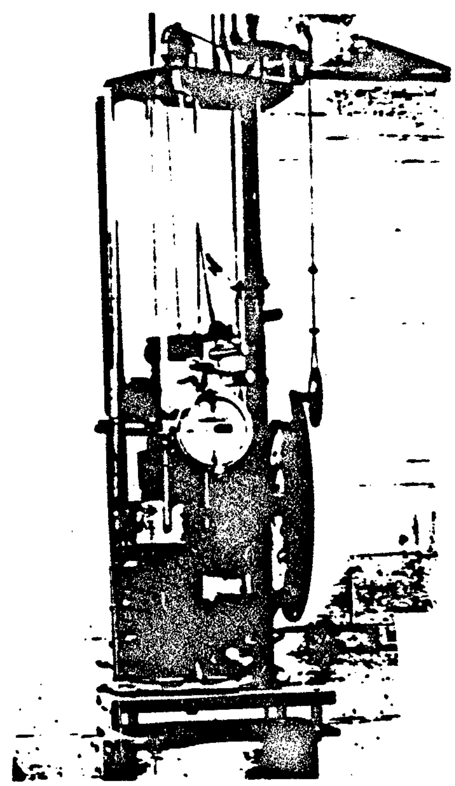
Figure 8. - Impact compaction apparatus.

Reclamation's Denver laboratory is also comparing the Vebe table, a standard impact compaction device, with the two models of Kango™ vibrating hammer for use in preparing RCC and soil-cement compressive strength specimens. Tests were conducted using a grading of 0 percent fines with 40 percent sand and 60 percent gravel, and a grading of 10 percent silty fines (liquid limit of 25 and plasticity index of 4), with 30 percent sand and 60 percent gravel. Both mixtures had a cementitious content of 50/50 portland cement/class F pozzolan with moisture contents ranging from 4.0 to 6.5 percent by dry mass of materials. Initial results are given in tables 7 and 8. The model 638 vibrating hammer was not considered for further testing because of the low dry densities and compressive strengths compared to the other types of apparatus tested.

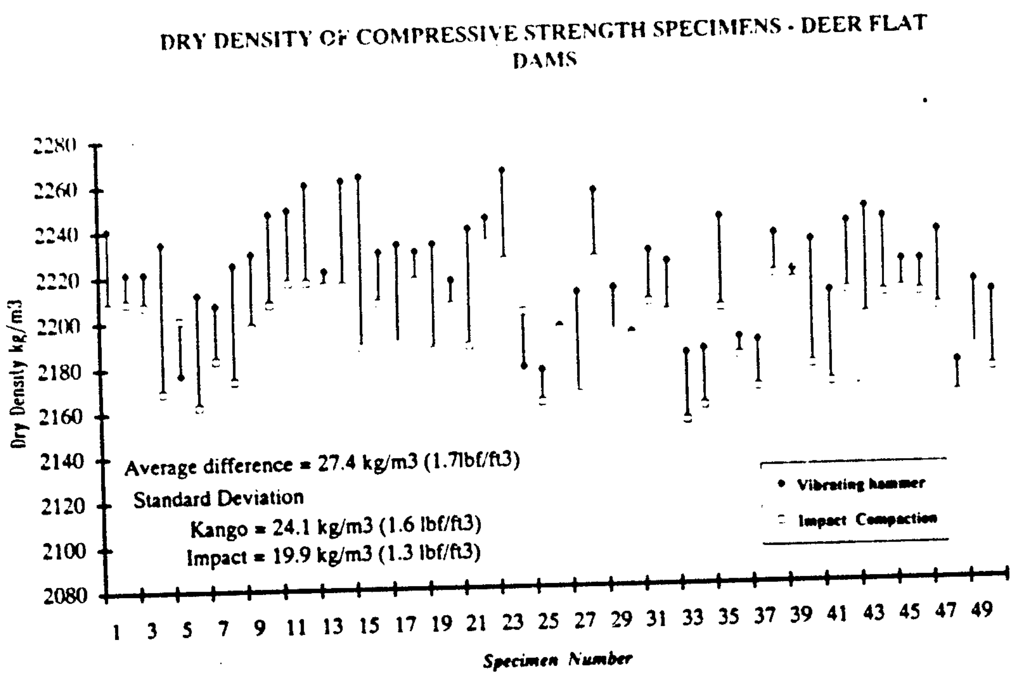

Figure 9. - Comparison of dry densities.

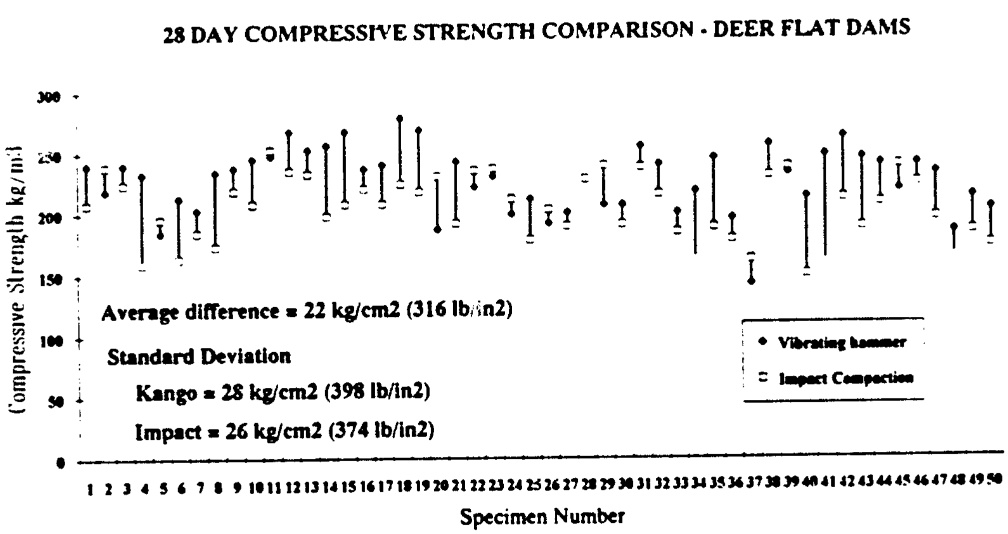

Figure 10. - 28-day compressive strength comparisons.

Table 7. - Compaction test comparison research, 0% fines
Type of compaction testing

Vibrating hammer Kango™ Model 638 750 watts			Vibrating hammer Kango™ Model 950X 1020 watts		
Water content (%)	Dry density (kg/m^3)	Unconfined compressive strength (kg/m^3) 90 days	Water content (%)	Dry density (kg/m^3)	Unconfined compressive strength (kg/m^3) 90 days
4.0	2220	73	4.0	2302	110
5.0	2351	138	5.0	2375	155
5.4	2316	148	5.4	2365	154
6.0	2327	150	6.0	2343	156
6.5	2319	145	6.5	2339	153

Impact compaction (22 960 ft-lb/ft³)			Vebe apparatus		
Water content (%)	Dry density (kg/m^3)	Unconfined compressive strength (kg/m^3) 90 days	Water content (%)	Dry density (kg/m^3)	Unconfined compressive strength (kg/m^3) 90 days
4.0	2335	96	4.0	2302	88
5.0	2387	148	5.0	2348	140
5.4	2378	165	5.4	2381	153
6.0	2370	171	6.0	2339	150
6.5	2346	161	6.5	2331	151

Table 8. - Compaction test comparison research, 10% fines
Type of compaction testing

Vibrating hammer Kango™ Model 638 750 watts			Vibrating hammer Kango™ Model 950X 1020 watts		
Water content (%)	Dry density (kg/m^3)	Unconfined compressive strength (kg/m^3) 90 days	Water content (%)	Dry density (kg/m^3)	Unconfined compressive strength (kg/m^3) 90 days
4.0	2203	61	4.0	2337	126
5.0	2334	185	5.0	2392	209
5.4	2336	199	5.4	2348	191
6.0	2327	224	6.0	2335	212
6.5	2284	207	6.5	2301	207

Impact compaction (22 960 ft-lb/ft³)			Vebe apparatus		
Water content (%)	Dry density (kg/m^3)	Unconfined compressive strength (kg/m^3) 90 days	Water content (%)	Dry density (kg/m^3)	Unconfined compressive strength (kg/m^3) 90 days
4.0	2304	93	4.0	2137	55
5.0	2348	155	5.0	2356	221
5.4	2359	187	5.4	2364	214
6.0	2337	190	6.0	2319	229
6.5	2315	193	6.5	2287	221

SUMMARY

The heat of neutralization test, the Vebe consistency test, and the nuclear moisture-density gauge offer rapid, relatively simple, non-labor-intensive methods for RCC and soil-cement construction control. Because the wide range of mixture proportions and consistency in soil-cement and RCC affect which construction control methods can or should be used, standards dependent on mixture proportions and consistency should be developed. A single method cannot be used for all situations. As mentioned previously, table 1 summarizes some of the current uses for these tests for RCC and soil-cement.

In the heat of neutralization test, very high calcium content in the soil or use of fly ash in RCC can interfere with the test by causing the mixture-buffer material to gel. Further work is needed to determine soil-cement or RCC/buffer solution proportions which would not gel. The Vebe apparatus may not completely consolidate soil-cement mixtures with a high content of pit run, nonplastic fines. Whether this is a problem should be determined during mixture proportioning. Sand cone tests may not be as accurate as nuclear moisture-density gauge checks on fresh RCC because of caving or squeezing of the sides of the hole resulting in high density values. The vibrating hammer offers promising possibilities as replacement equipment for that currently used, but needs further verification. Table 9 lists some possible suppliers for the newer equipment mentioned.

Table 9. - Equipment suppliers[1]

Equipment	Supplier	Address
Vebe table and mold	Soiltest	86 Albrecht Drive P.O. Box 8004 Lake Bluff, IL 60044-8004 PH: (708) 295-9400
Nuclear density gauges	Campbell Pacific Nuclear Company	2830 Howe Road Martinez, CA 94553 PH: (415) 228-9770
	Troxler Electronic Lab, Inc.	3008 Cornwallis Road P.O. Box 12057 Research Triangle Park, NC 27709 PH: (919) 549-8661
Kango™ vibrating hammer	U.K. Mfg. Kango Ltd.	Shrewsbury Avenue Woodston Peterborough Cambridgeshire PB 2 OBX England PH: 711-011 44 733 371 707
	U.S. (Wholesale) Kango, Inc.	63 North Planes Ind. Road Wallingford, CT 06492 PH: (203) 284-1185
	U.S. (Dist.) Hobelmann & Co.	1160 Chess Drive No. 3 Foster City, CA 94404 PH: (415) 571-5886

[1] In mentioning these companies, Reclamation is not implying that they are the sole or best suppliers of this equipment.

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THE USE OF POSTTENSIONED TENDONS AT STEWART MOUNTAIN DAM, ARIZONA: A CASE STUDY INVOLVING PRECISION DRILLING

By R. H. Bianchi[1] and D. A. Bruce[2]

Abstract: Stewart Mountain Dam near Phoenix, Arizona, was modified to resist potential seismic loadings by using posttensioned tendons. This is thought to be the first use of posttensioned tendons to rehabilitate a thin-arch concrete dam.

Stewart Mountain Dam, built during 1928-1930, is a double-curvature, thin-arch dam with thrust blocks and gravity sections. The arch section is 583 ft long, has a crest width of 8 ft, has a maximum structural height of 212 ft, and is up to 34 ft thick at the base. At the time of construction, the importance of good cleanup on the horizontal construction joints between lifts was not recognized. These joints were left untreated, resulting in a series of unbonded horizontal planes across the arch section of the dam on 5-ft vertical intervals. The left thrust block required stabilization due to foundation problems, and posttensioning was also chosen.

To provide the stabilization, 84 tendons were designed. Sixty-two tendons within the arch were installed at about 9-ft centers with free lengths ranging up to 216 ft and bond lengths from 35 to 45 ft. Tendon inclinations were required to vary from vertical to 8°40' off vertical. Twenty-two tendons were placed, inclined 30° off vertical, through the left thrust block. All tendons were placed in 10-inch-diameter holes with a specified 9-inch-per-100-ft drilling tolerance. Special considerations were necessary to complete the drilled holes for the 62 tendons within the thin-arch dam. A gyroscopic survey tool was the primary device used to ensure proper alignment.

This paper discusses the background, design, and construction considerations, and the problems encountered, lessons learned, and techniques used in completing the holes. The paper also details the precision drilling, primarily with the down-the-hole hammer, surveying, water testing criteria and techniques, and grouting techniques used for the installation of the 62 tendons within the arch portion.

INTRODUCTION

As part of its Safety of Dams Program, the Bureau of Reclamation (Reclamation) in the 1980's evaluated possible structural and hydraulic deficiencies at Stewart Mountain Dam, near Phoenix, Arizona. Structural and hydraulic studies performed on the dam indicated that a number of modifications would be required. Given the unique nature of the modifications, Reclamation decided to accomplish the work through two separate contracts, Stage I and Stage II.

Stage I was awarded to Kiewit Western Company, Phoenix, for $18.2 million. This contract was a conventional competitive bid solicitation. The work was completed in 1989. Stage I work included: adding an auxiliary spillway on the right abutment, removing and replacing of the old penstock and bypass outlet works, modifying the crest of dam, placing concrete overlays on the gravity sections and thrust blocks, installing a drainage system, and other miscellaneous items.

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[1] Bureau of Reclamation, Denver, Colorado.

[2] Nicholson Construction of America, Bridgeville, Pennsylvania.

In 1990, Stage II was awarded to Nicholson Construction Inc., Atlanta (Nicholson) for $6.5 million. Unlike the first contract, the Stage II contract was a negotiated procurement, because the work primarily involved the highly specialized installation of multistrand tendons through a thin-arch dam. It is the aim of this paper to describe design assumptions and the conditions leading toward this second modification contract. This paper discusses design and construction considerations, problems encountered, lessons learned, and techniques used in completing and sealing the drill holes for the tendons.

BACKGROUND

Stewart Mountain Dam is located on the Salt River approximately 30 miles east of Phoenix, Arizona. The dam was constructed from 1928 to 1930 by the Salt River Valley Water Users Association at a cost of $2.3 million. The Salt River Project (SRP) operates the dam as part of a water-storage and power-generation system on the Salt and Verde rivers. The reservoir, Saguaro Lake, is one of the principal water sources for the Phoenix metro area.

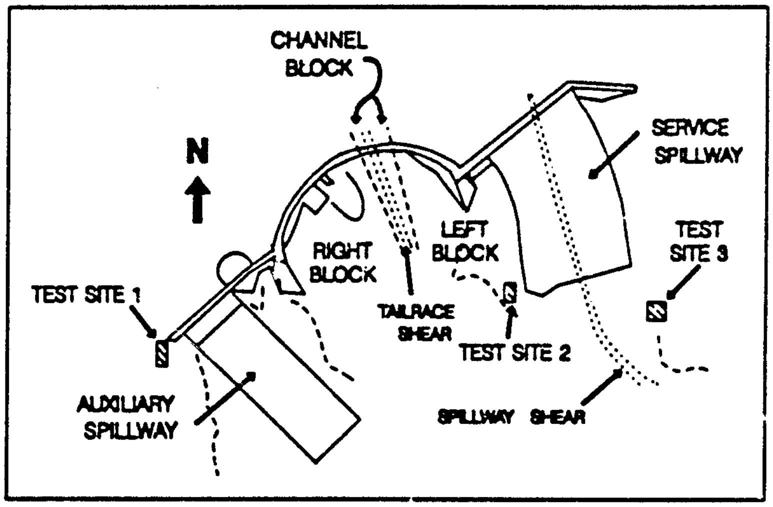

Figure 1. - Location of main features and anchor test sites.

The original dam was a composite concrete structure that included a double-curvature, thin-arch dam, two concrete thrust blocks, three concrete gravity sections, and a service spillway. The arch has a 212-ft structural height, an 8-ft crest thickness, a 34-ft base thickness and a 583-ft crest length. Through the arch portion, there is a 13.5-ft-diameter steel penstock connected to a 13 MW powerplant and a 7-ft-diameter opening serving as bypass outlet works. On the left side is a service spillway having a capacity of about 90,000 cubic ft/second. The modified dam (Figure 1) has an auxiliary spillway added under Stage I.

Construction practices in the late 1920's did not recognize the importance of cleaning horizontal construction joints before subsequent placements. Concrete placement therefore proceeded without cleanup at the top of the 5-ft-high lifts. This practice produced weak layers of laitance at every lift elevation resulting in planes with little or no bond. Vertical contraction and construction joints extending the height of the dam divide the arch into nine sections. The combination of weak horizontal planes and vertical joints created discrete concrete blocks kept in place largely by mechanical arch action.

Another problem impacting concrete, which was not recognized in the 1920s, was the incompatibility of certain aggregates with cement. By the early 1940's, extensive pattern cracking was noted on the concrete surfaces, and monitored. In addition, the arch portion of the dam separated from the powerplant, necessitating new support for the powerplant roof. By 1943 the problems were determined to have been caused by alkali-aggregate reaction, a chemical reaction between aggregates within the concrete, and the cement, which produces a swelling gel. This expansion resulted in significant displacement and cracking across the dam.

Erosion of the bedrock downstream of the left thrust occurred during high flows through the service spillway in 1966, 1978, and 1980 which overtopped the right training wall. The depth of erosion extended below the downstream toe of the left thrust block.

SITE GEOLOGY

The dam was built on Precambrian intrusive rocks, principally quartz diorite cut by irregular dikes of granite (GR) and smaller diabase (DI) and silicic dikes. From the right to the left abutments, the rock becomes increasingly more fractured and sheared. Two major shears are in the foundation area (Figure 1). The original river channel follows one of these shear zones, and the other is beneath the service spillway. Rock weathering is typically deep except within the active scour areas. Intense rock weathering often extends to a depth of 100 ft except in areas of active scouring. The shear zones are weathered and intensely fractured with clay infilling.

DAM SAFETY INVESTIGATIONS

Investigations to determine the conditions of concrete and rock began in 1984 (Wellendorf, 1985). Records consisting of photographs, designs and "as-built" drawings and documents were reviewed. Preconstruction investigations within the arch dam and thrust block area were evaluated. The investigation involved extensive subsurface and surface investigations for the dam and surrounding area. Sixteen drill holes were placed within the arch dam and thrust block to determine the condition of foundation rock and concrete, as well as to evaluate the uplift pressure and seepage through the foundation. Detailed surveys were conducted across the dam.

The excavation "as-built" drawings were found to be accurate to within a foot, confirmed by the exploratory drilling. The foundations were found to be excavated to moderately to slightly weathered rock except in the area of the shear zone near the center of the arch and in limited areas elsewhere the foundations. The bedrock foundation of the left thrust block was found to be intensely fractured. Embedded steel was found to be present within the thrust blocks as the design drawings had indicated. This block not only required installation of a drainage system to reduce the uplift pressures, but also required further stabilization in the form of an anchorage system to prevent sliding.

Unbonded lift lines were evident in core recovery. Core breaks nearly always occurred along the lift lines. Seepage had been noted along the lift lines soon after the filling of the reservoir, and also during drilling, when new seepage was noted across the concrete faces along the lift lines. Within the arch section, core drilling with water often required grouting and redrilling. Loss of drill fluids along the lift lines was a common occurrence particularly within the upper 50 ft of the arch. During the drilling of hole DH-503-SM, the generator was shut down when drill fluid from the borehole travelled along a lift line and daylighted along the downstream face within the powerplant (Figure 2).

The original seepage control for the gravity section and thrust blocks consisted of square redwood drains and a single line grouting program. Grouting was performed principally in areas of gravity sections and the thrust blocks. Grout records failed to indicate mix design(s) or the pressure applied during the operation. Grouting apparently did little more than backfill the holes, as the takes were small.

The concrete strength varies considerably from base to crest as well as within the structure. Compressive strength values average 4,950 lb/in^2 in the arch, 4,840 lb/in^2 in the left thrust block, and 3,220 lb/in^2 in the right thrust block. There was no steel reinforcement within the arch except for around the penstock and outlet works. However, within the thrust block, steel reinforcement was placed in mats at 5-ft centers.

A review of measurement points at the top of the dam indicates that there was a rapid volumetric increase from 1937 through 1968, but that the dam had been relatively stable from 1969 to present. The alkali-aggregate reaction within the concrete was found to have permanently displaced the dam crest 6 inches upstream and 3 inches upward (Nuss, 1987).

Water testing confirmed that the foundation and concrete appeared to have high local permeabilities. This testing consisted of mostly falling head tests within the concrete portion of the investigations. Pumps having a capacity of approximately 35 gal/min for most of the boreholes were unable to maintain a constant head within the concrete dam. Packer tests performed within the bedrock indicated that, generally, the bedrock was tight, ranging from 0 to 2.0 gal/min (0 to 23 lugeons). In a few intervals, like within hole DH-404-SM, higher takes of up to 53 gal/min, at 50 lb/in^2 pressure, (>100 lugeons) occurred. No packer tests were performed across the concrete/foundation contact, although drill fluid losses were often reported in the vicinity of this interface.

Vibrating wire piezometers were installed within the bedrock beneath the dam. High uplift pressures were found within the foundation indicating that the grouting operations during the original construction were not effective. Of particular concern were the areas under the wing dams and the thrust blocks.

FINAL DESIGN AND PRECONTRACT ACTIVITIES

Most of the dam required stabilization based on the revised MCE (Maximum Credible Earthquake) calculation and the foundation information obtained during the investigations. Seepage along the lift lines was determined to be unsightly but was not detrimental to the structure's stability (Nuss, 1987). It was determined that stabilization would be best achieved for most of the structures by the installation of drains within the structures and/or adding overlays. The principal exceptions were in the areas of the arch and left thrust block. Stabilization of the arch and left thrust block required the installation of posttensioned tendons.

Designs were significantly impacted by the open lift lines combined with earthquake potential, foundation conditions around the left thrust block, and high uplift pressures. Technical memoranda SM-220-01-87 (Nuss, 1987) and SMC-3110-01-90 (Nuss, in progress) contain the details of the analysis and the recommended improvements to the dam. One of the most significant improvements to the dynamic stability was the installation of tendons across the arch dam.

Since the installation of tendons through a thin-arch dam was believed to be a unique event, concerns were raised as to how the anchor loads on the dam would impact the structure. It was determined that stressing of the tendons would require a specific tensioning sequence and an evaluation period was recommended to determine if there were any adverse deflections or other unusual responses to these loads (Nuss, in progress). During the evaluation period, the strands required corrosion protection. Conventional tendon corrosion protections were inappropriate for the conditions at this site. Encasement of the strands or tendon with sheathing or grease, providing a corrosion-inhibiting environment such as calcium hydroxide, and use of galvanized strands were all options studied and rejected. Epoxy-coated strand impregnated with grit was chosen as the corrosion protection system (Nuss, 1992).

A major construction concern was the ability to drill the tendon borehole within a confined area (Figure 2). The drilling of large diameter boreholes has been performed on a variety of dams (Bruce, 1987). Drilling tolerances as small as 6 inches/100 ft have been specified. However, surveys at most of the sites were performed with magnetic survey tools with uncertainties between 0.1° on vertical holes (2 in/100 ft), to 0.2° on 10° off vertical holes (4 inches/100 ft), and to 0.8° on 30° off vertical

(18 inches/100 ft) (Wolf and deWardt, 1981). The uncertainty of the magnetic tools was often equal to the tolerance specified. At Stewart Mountain, drilling tolerances required the drill holes to fall within a 12-inch radius at a 100-ft length for an ideal alignment. Electrical interference from the powerplant could impact the bearing reading and within the thrust block, steel reinforcement was pervasive throughout the concrete and would significantly impact the magnetic readings. Technical memorandum 3610-89-28 (Bianchi 1989), based on published information and conversations with contractors, determined that the drilling could be performed within a 1-ft radius at a depth of 100 ft and that surveying tools were available with a maximum uncertainty of 2.5 in/100 ft for any of the arch tendon holes.

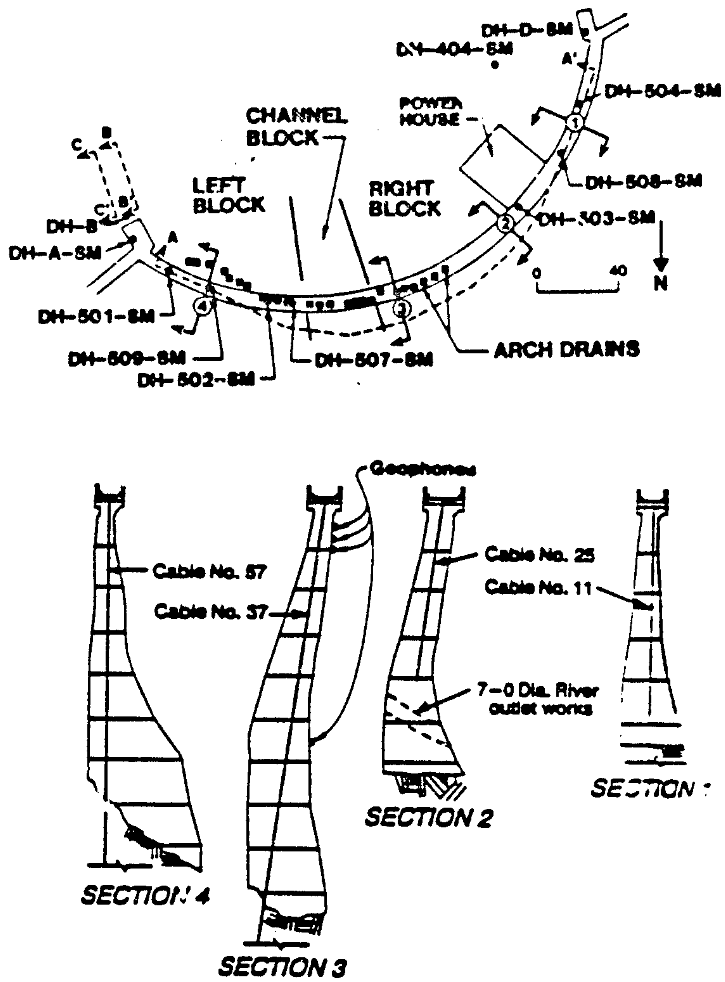

Figure 2. - Location of exploration holes, arch drains and typical sections.

Because of the relatively unique nature of the tendon installation, it was determined that a specialty contractor would be required. Much of the other work required more standard repairs or construction activities, such as adding the auxiliary spillway on the right abutment, adding overlays, removing and replacing a crest structure, and drilling a drainage system.

The specialized nature of tendon installation and the uniqueness of this job made it essential to obtain an experienced specialty contractor. Specific concerns were the drilling, surveying, installing, and tensioning along the crest of the arch. Evaluation criteria were developed to determine parameters for the type of equipment and methods used in conformance with the specification, and what contingencies would be used if specific situations arose, with particular emphasis on drilling a hole within tolerance and on minimizing induced vibrations. The negotiated procurement process was determined to be the best method for evaluating contractors and their construction methods. Once the decision was made, the installation of tendons was removed from the Stage I contract, as was the related work such as the backfilling area around the left thrust block and gravity section.

The negotiated procurement process required that offerors submit both a technical and a cost proposal which were evaluated separately. The technical proposal addressed specific concerns, qualifications of key personnel listed within the specification, and detailed the approach to the work. These were evaluated by a technical proposal evaluation committee. After completion of the technical evaluation, a cost/price evaluation was made by a cost/price evaluation committee on the cost proposals for all offers classified as technically acceptable. The evaluation factors were provided within the specification paragraphs. Award was determined by evaluating the combined technical and cost/price scores for all offerors.

STAGE I MODIFICATION

The main intent of the Stage I contract was to provide additional flood protection as well as to improve existing operational features of the dam. The work included the construction of a new auxiliary spillway, removing, modifying, and replacing the crest of the dam, adding 6- to 8-ft-thick overlays on the thrust blocks and gravity sections, replacing one river outlet works and power penstock, plugging two existing outlet works, providing additional drainage features, and grouting selected portions of the dam foundations, as well as various other items requested by SRP for ease of maintenance.

The primary improvement which affected the Stage II contract was the modification which added 3 ft to the top of dam across the entire arch section of the dam. Posttensioned anchor work required a bearing surface through which the high concentrated loads could be transferred and distributed to the nonreinforced arch structure. As a result, adding an additional 3-ft reinforced cover with blockouts provided an adequate area capable of supporting the 40-million-pound load exerted on the arch by tensioned tendons as well as providing a wider work area, easier access, and recesses for anchor heads.

Other improvements for Stage II, made under Stage I, were construction of blockouts within the left thrust block overlay and reduction of uplift pressures within the foundation. The grouting in the left thrust block helped by controlling seepage, and potentially reducing borehole caving and the amount of pressure grouting required in the thrust block bond lengths during the tendon installation. The installation of drainage within the thrust block and within the bedrock reduced the uplift pressure and seepage.

The only other item impacting the Stage II modification was the fact that the system of drainage holes' drill reports and core provided additional subsurface information. This assisted in determining the three rock mass categories used during the pullout tests. The only potential negative impact of the Stage I work was that by adding all the drains within the thrust block and arch, more potential obstructions were created if the tendon holes were not drilled within a tolerance of 9-inches/100 ft (Figure 2).

STAGE II MODIFICATION

The work within the arch portion of the dam consisted primarily of:

1. drilling, surveying, water testing, and, as necessary, pressure grouting 62 tendon holes ranging in inclination from vertical to 8° 40' off vertical;

2. furnishing and installing 62 tendons each consisting of 22 epoxy-coated strands, varying in free length from approximately 40 to 213 ft;

3. determining the recommended bond length, and grout mix design for the bond and free lengths;

4. grouting the bond length and stressing the tendon;

5. evaluating the tendon during the 100 day evaluation period;

6. when required, retensioning the tendons;

7. and grouting the free length, cutting the excess strands and backfilling the blockouts with concrete.

The left thrust block work consisted primarily of:

1. drilling, surveying, water testing, and, as necessary, pressure grouting 22 tendon holes inclined at 30° below vertical;

2. furnishing and installing 22 tendons, each consisting of 28 epoxy-coated strands;

3. grouting the bond length and stressing the tendons;

4. grouting the free length;

5. cutting the excess strands and furnishing and placing reinforced concrete within the blockout;

6. and placing and compacting backfill at the toe of the left thrust block and left gravity section.

Some of the key equipment mobilized to site included a Casagrande Model C-12 diesel hydraulic track drill, equipped with a down-the-hole hammer; two Model 950/350 air compressors; a 12 ft^3 colloidal grout plant, a backup grout plant; and an Eastman-Christensen (EC) Seeker-1 rate-gyro survey system.

Bond length determination test. - Prior to any production drilling of tendon holes, a full scale anchor test was required (Scott and Bruce, 1992). Three sites were selected to reflect major rock mass categories (Figures 1 and 2). The intent of the test program was to determine the anchoring requirements within different bedrock conditions. Other benefits from the test program included providing an opportunity to identify and correct any unforeseen construction difficulties through a full-scale trial run and to provide the field and design staff with an opportunity to be become familiar with the procedures and capabilities of the equipment.

The specifications required that two tests be conducted at each of the three sites, that each site be cored (N-size) prior to drilling the tendon hole and that other procedures be similar to the production tendon installation. This involved using the Casagrande C-12 rig to drill the hole, performing water tests to determine the water tightness of the boreholes, developing and using the proposed grout mix design, and using a maximum tendon pullout load of 150 percent of design load. Nicholson satisfied the requirements in the proposal, performed a more rigorous water testing procedure recommended by Houlsby (1976), added a test load based on the highest actual loads expected, performed actual surveys using the Seeker, and used tendons consisting of 28 epoxy-coated seven-wire strands.

From the bond length tests, the recommended bond length for the production tendons was determined to be 35 ft within the quartz diorite and granite as results from site 1 (quartz diorite) and site 2 (combination of quartz diorite and granite) were similar. Test site 3 was within a very intensely to intensely fractured mylonitized granite with fractures dipping subvertically. The 10-ft bond zone hole failed at 98 percent of design load (985 kips). From site 3 results, all holes falling within the shear bedrock (holes 40 through 47 and areas of holes 53 and 54) were designated to have a bond length of 45 ft.

Survey testing. - The drilled tendon holes for the arch and thrust block required precision drilling because of the proximity of free faces on the arch dam and the drainage systems installed under the Stage I contract (Figures 2 and 3). No offsetting of the tendon hole was allowed. The specification required a drilling tolerance within a 9-inch deviation at a length of 100 ft with an uncertainty allowance of 3 inches for the survey equipment (thus maximum 12-in/100-ft apparent deviation). The specification permitted advancement of tendon holes 10 ft ahead of the survey. This allowed surveying of the hole without withdrawing the down-the-hole hammer.

In addition, within the arch section, a curvature tolerance was required because no friction loss on the tendon sidewall could be tolerated in the upper 50 ft of a hole: the use of centralizers on the tendons in the upper 50 ft was prohibited. The curvature was measured relative to a line extending from the center point at the collar to the actual center point location at the 50-ft depth. The perpendicular distance between this line and the actual centerline of the hole at any point within the 50-ft length had to be less than 0.5 inch in order to ensure a 1-inch minimum clearance between the outside strands of the tendon and the sidewall of the hole.

Nicholson performed a test to confirm the sensitivity of the Seeker-1 instrument by suspending a metal pipe on the face of the auxiliary spillway. The pipe was independently surveyed by conventional means and the two sets of results compared. The biggest problem with the test was that on the day of the test the wind was significant and the pipe tended to sway in the wind: the pipe was only secured at both ends. The readings fluctuated accordingly, but did prove the acceptability of the method.

Drilling and survey testing on the arch. - The final acceptance of the drilling and survey procedure was performed on tendon hole 37 within the arch. This hole required an inclination of 7° 10' upstream, on a bearing of 344°. Based on the as-built top of excavation the anticipated length in concrete was 187 ft. The specified free length was therefore 197 ft (187 ft plus 10 additional ft below the concrete/rock contact) plus 35 ft for the bond length. The actual total length would be determined by the location of rock encountered during drilling.

The specifications required the tendon hole be surveyed at 10 ft intervals for the first 50 ft and at 20 ft intervals along the remaining length. Data from the survey readings were processed within the time it took to withdraw the survey tool from the borehole and reconnect the drill rods to the down-the-hole hammer. Initially, although the readings from the Seeker were within the specified tolerances, the data required conversion into formats that an inspector could appraise quickly (e.g. see Figure 4).

To promote hole straightness, the C-12 rig utilized a 10-inch button carbide bit on a down-the-hole hammer with a spiral overhammer stabilizer. The 6-ft-long hammer was followed by an over-the-barrel stabilizer on the first 6-inch-diameter drill rod. The 6-inch-diameter drill rods were in 10-ft lengths to a depth of 50 ft, then 20-ft lengths thereafter. The inclination of the mast was confirmed by a digital level; bearing was checked by reference to the radial scribe marks on the dam crest.

During the drill evaluation test, Reclamation personnel monitored the behavior of the dam with eight Geokon crackmeters and four geophones located on the downstream face of the dam (Figure 2). The crackmeters were variable resistance vibrating wire, continuous reading instruments, sampling every 30 seconds during drilling and recorded on an SR10 Data Recorder. The crackmeters were positioned vertically across alternate lift lines in the upper 83 ft of the dam. The geophones were bolted to the dam face and three were placed at intermediate locations in the upper 20 ft of the dam, while the fourth one was located approximately 110 ft below the dam crest. In monitoring the crackmeters, any movement in excess of 0.05 inch between any two readings would require further evaluation. Results during the drilling

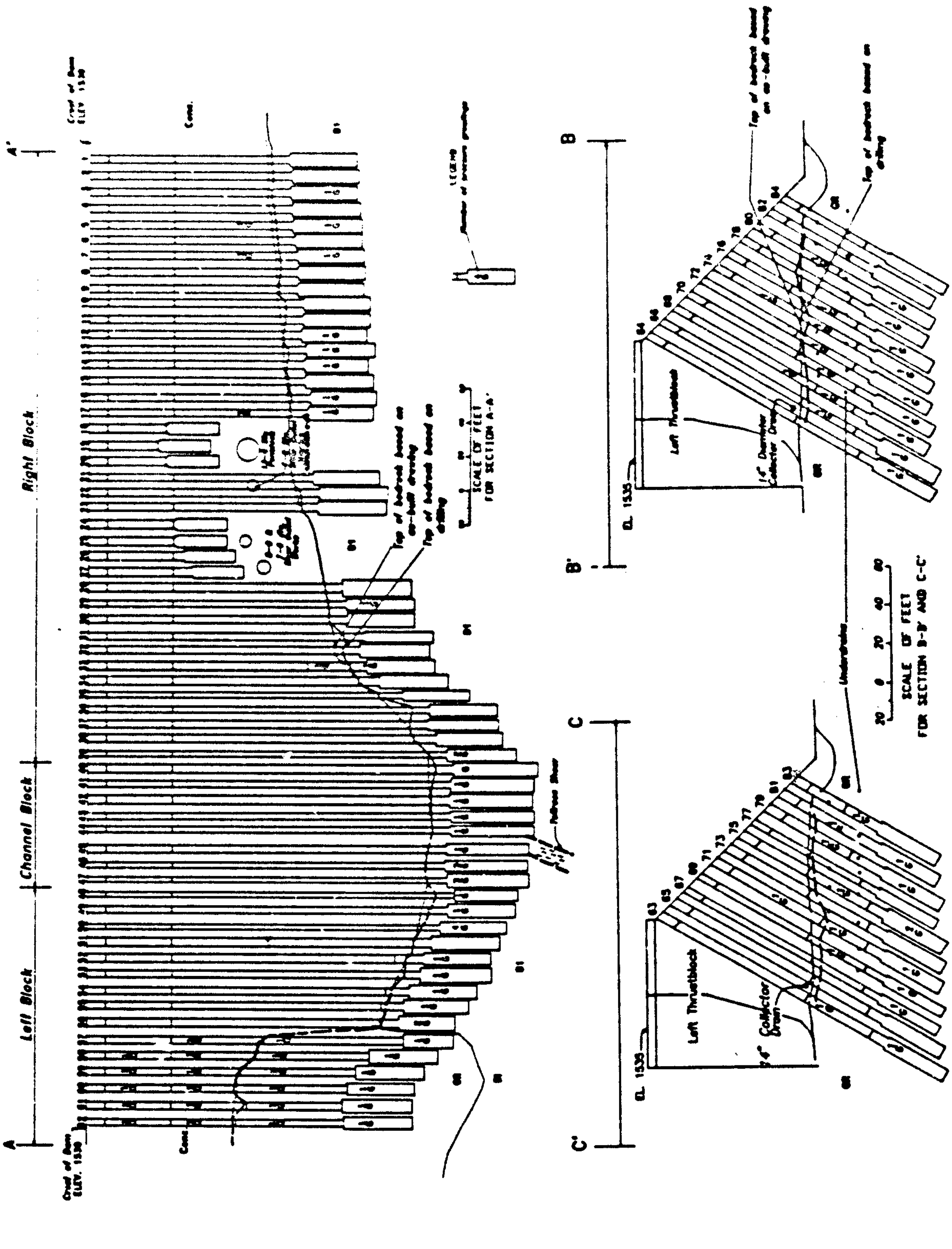

Figure 3. - Vertical sections through tendons showing grout intervals, and number of tests.

showed maximum movement varied from 0.00218 to 0.00510 inch overall (Table 1). For each of the monitored locations, the maximum crack movement occurred at precisely the time the drill bit was at the elevation of the lift line. Except for the moment when the bit passed across the lift line, the effect of drilling could not be differentiated from normal crack movement caused by temperature variations.

The four geophones measured peak particle velocity (ppv) on a continuous basis. In general, the maximum peak vector sum was greatest when the drill bit was closest to the measurement location (Table 2). The maximum peak vector sum of 0.147 inch/sec was well within the allowable criterion of 2 inch/sec for blasting vibration normally (Figure 5) accepted for structures by Reclamation (Reclamation, 1980) and Corps of Engineers (Department of Army, 1972).

The entire length of the hole was drilled in the dry, and progress was slowed only momentarily when steel was encountered at depths of 12 and 44 ft. The Seeker's reading indicated a deviation in the total length of hole of 5.48 inches in 230 ft or 2.38 inches in 100 ft. Curvature yielded a maximum deviation of 0.5 inch within the upper 50 ft of the hole. The borehole survey using the Seeker was checked by Reclamation's optical device, with both in close agreement and well within the required tolerances (Table 3).

Initial core drilling. - Overcoring of the upper 4 ft of each hole was performed by a subcontractor, Concrete Coring Company of Phoenix. To ensure the coring operation attained the correct alignment, each hole was surveyed and checked by both Nicholson and Reclamation. An alignment and centering guide was positioned within each concrete blockout. The drilling equipment consisted of two hydraulic-driven drill motors mounted on a 4-inch by 5-ft drill mast located exactly over the centering guide. However, at 11 sites, a 6-ft steel reinforcement was used as a

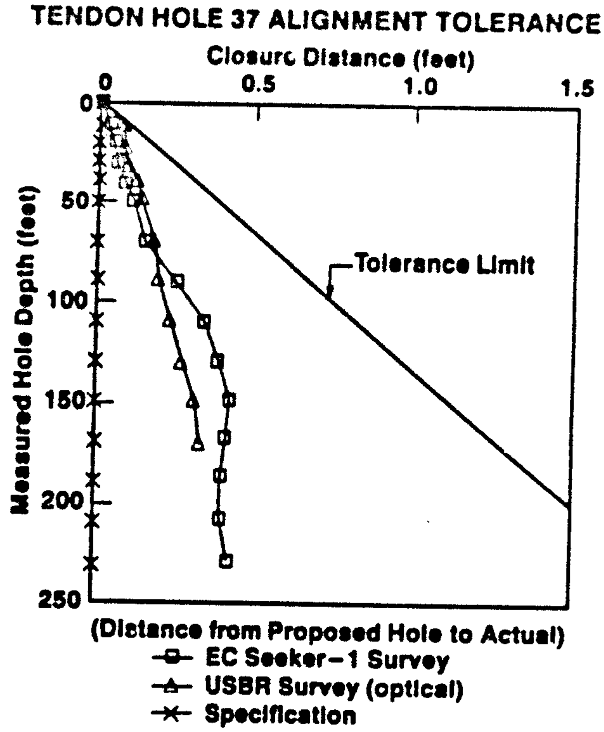

Figure 4. - Tendon hole 37 alignment tolerance.

Table 1. - Summary of crackmeter data.

Elevation of Crackmeter (feet)	Maximum Recorded Movement (in)	Typical Daily Movement due to Temperature Effect only (in)	Apprcx. Distance from Meter to Hole (ft)
1520.39	0.00239	0.00284	5.0
1510.45	0.00218	0.00284	5.5
1500.34	0.00409	0.00432	5.8
1490.43	0.00510	0.00348	5.8
1480.56	[1]	0.00353	5.5
1470.17	[1]	0.00376	5.5
1460.21	[1]	0.00459	5.5
1450.23	[1]	0.00440	5.5

[1] No discernable movement was detected during the drilling operation

centering guide rather than a wooden dowel. At these sites, concrete coring had to be extended to a depth of 8 ft to remove the steel guide.

After each hole was cored, 5-ft-long, 10-3/16-inch internal diameter steel cylinders were installed. The alignment was set by a tangential stringline, while radial inclination was set with a digital level. It was then grouted in place with a nonshrink grout. Within 4 days of setting each tube in place, the C-12 was positioned over the hole and the drilling began.

Production drilling on the arch. - Drilling on the arch tendon holes began in mid-March, 1991 and was completed by early May 1991, except for tendon hole 11 which had a broken bit that required grouting and coring to remove. Drill cuttings were removed by compressed air from tandem 350 lb/in^2, 900 cfm, Sullair compressors delivering through the center of the drill string. Minimizing high instantaneous pressures was addressed in the technical proposal. At test site 3, clay infilling within the bedrock plugged the stabilizer groves and a resulting temporary pressure buildup was noted. Nicholson modifications to the discharge ports of the stabilizer prevented such buildups from occurring, so that no lift lines on the arch or the left thrust block were threatened. One incident occurred during drilling hole 62 when large air bubbles were seen surfacing upstream of the dam. However, once the hole was advanced, the crack was apparently sealed by the cuttings. Another unusual observation occurred during drilling hole 40: seepage was observed coming out at the base of the arch drain collar in the vicinity of the hole. This condition disappeared after 2 days.

Vibration Monitoring of Hole 37

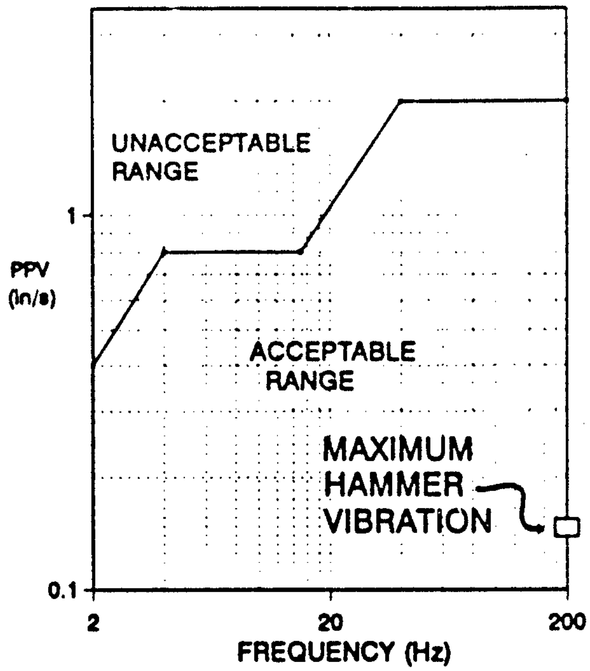

Figure 5. - Vibration monitoring of Hole 37.

Table 2. - Summary of geophone data.

Elevation of Geophone (ft)	Approx. Drill Bit Elevation at Max. Peak Vector Sum (ft)	Maximum Peak Vector Sum (in/sec)	Approx. Distance from Geophone to Hole (ft)
1521.12	1521	0.039	5.0
1517.14	1454	0.017	5.0
1512.94	1581	0.066	5.0
1423.15	1433	0.147	8.0

A total of four bits were lost down various holes; three were recovered quickly with a bit retrieval tool, while the fourth was recovered in a core barrel, after being encased in grout. Steel was regularly encountered at the 12-ft depth in many of the arch holes. Steel cuttings were usually lifted from the hole by air. Larger cuttings required magnets to remove them.

Surveying required inserting the torpedo-shaped Seeker into the rods by wireline. These surveys indicated no doglegs within any of the holes. Reclamation periodically surveyed the holes to determine if the Seeker was operating within the specified tolerance. The principal limitation of the optical survey check system developed by Reclamation was that it required a line of sight to determine the alignment. Table 3 is a select summary of holes surveyed by both methods and the difference between the two readings. Only one, Hole 4, was surveyed initially to be apparently outside of the required limit of 9 inches within a hundred ft. The Reclamation optical system confirmed the hole had deviated only 6 inches within 100 ft. As a result, realigning the hole was not required as the hole was truly within tolerance.

Another change made by Nicholson was to drill an additional 5 ft beyond that required. This additional length served as a sump to collect cuttings which were either too large or too heavy, such as steel, and that may have fallen into the hole and were not lifted from the hole as cuttings.

Table 3. - Survey deviation summary.

Tendon Hole	USBR Final Length (ft)	DEVIANCE (in) EC v/s Spec	USBR v/s Spec	E/C v/s USBR[1]
4	110	0.85	0.75	0.21
22	70	8.19	8.14	0.08
46	114	0.05	0.19	0.16
47	107	0.36	0.37	0.30
48	104	0.57	0.61	0.17
49	107	0.15	0.04	0.11
50	83	0.26	0.08	0.19
51	90	0.25	0.22	0.09
52	90	0.09	0.13	0.03
53	68	0.10	0.07	0.05
54	87	0.16	0.15	0.01
55	68	0.14	0.13	0.06
56	76	0.21	0.18	0.06
57	70	0.07	0.09	0.03
59	50	0.04	0.03	0.07

[1] Deviations are based actual location differences

Thrust block drilling. - A drilling evaluation test on tendon hole 82 was performed to determine the impact of drilling through the steel reinforcement and maintaining an alignment along a 30° incline from vertical. Upon successful completion of the performance test, production drilling commenced in early July 1991. All 22 holes were completely drilled to full depth in late July 1991. None of the holes required grouting to correct misalignment or caving conditions. None of the drains installed under the Stage I contract were encountered during the drilling.

Inclined drilling produced a definite curvature in an downward direction. Four tendon holes (77, 78, 79, and 80) were outside the drilling tolerance with deviations of 10, 10, 16, and 15 inches in 100 ft, respectively. However, these holes were not required to be redrilled since they were not in danger of intercepting any other tendon holes or drains in the area. This deflection was attributed to the weight of the drill rods combined with the thrust force tending to level the drill string. To counter the natural deviation, the drill mast was set at an initial inclination of 30°30' for each hole, rather than the 30° specified.

Water testing. - Water testing of arch tendon holes for the bond lengths was in accordance with the Recommendations of the Post-tensioning Institute (PTI, 1986). The maximum allowed water loss within the bond length was specified for a 10-minute interval as being 0.001 gallon per minute, per inch diameter, per foot of hole with a 5-lb/in^2 pressure at the collar. The free length requirements were relaxed to 0.002 gallon per minute, per inch diameter, per foot of hole with a 5-lb/in^2 pressure at the collar. The specified water testing performed during the drilling investigation had indicated that most of the intervals would fail even the free length criterion of water testing. The specification required water testing be performed on a minimum of two stages for holes 18 through 27: the upper 50 ft, and the remainder of the hole.

Arch holes 57 through 62 were to be have at least four stages (two stages within concrete and 2 stages within foundation). All the remaining holes were water tested at least in three stages (two stages within concrete and the bond zone). Figure 3 shows the test intervals and shows the number of times water retesting was required.

Water testing was typically conducted in groups of five or six holes. Problems occurred if the packer test was performed too soon after drilling. Holes which had been drilled on the shift prior to water testing tended to give higher water loss readings than holes which had achieved static water level. Figure 3 displays numbers of water tests required to fulfill the requirements of the specifications.

Pressure grouting. - Where stage water tests exceeded the criterion, pressure grouting was performed as specified. If the upper 50-ft interval was found to be permeable and the remainder of hole was found to be tight, the entire hole would require grouting and redrilling. If only an intervening interval was found permeable and the rest of hole was found tight, the permeable interval to the bottom of the hole required grouting. In nearly every case where there was an isolated interval showing a water take, the interval was retested after pressure grouting and was found tight. Figure 3 indicates the intervals requiring pressure grouting (G) and number of time pressure grouting was performed within an interval.

The specification required that after pressure grouting, the hole was to be redrilled and resurveyed. Based on the mix design, the grout was drilled out within a period of no less than 12 hours, and no longer than 48 hours after completion of grouting. If redrilling occurred after the 48-hour period, the borehole would have required resurveying. Pressure grouting and regrouting were generally only required in the bond length intervals. The tested zones in the concrete dam were very tight and met the specification criterion for water tightness. In Hole 50 the bond length required four pressure grouting operations prior to acceptance, but this was atypical.

Completion of tendon installation. - Fabrication of the tendons began shortly after determination of the bond lengths from the pull out test. These tendons were fabricated and wound onto reels at the Dywidag plant in Illinois, and delivered to site. Fabrication consisted of cutting strands to the required lengths and installing centralizers, stabilizers, and two 1-1/2 inch grout lines within the bundle of strands. One grout line was intended to grout the bond length; the second was for grouting the free length.

Prior to insertion of the tendons, the bearing plate was installed, leveled, and grouted. Installation of the first tendon began in June 1991.

Upon the satisfactory completion of each hole, the tendon was carefully unrolled from the reel into the hole to ensure that the epoxy-coated strands and grout pipe were not damaged during the installation. The tendon was suspended within the hole and the bond length was grouted. Specification required that the mix be capable of achieving a minimum compressive strength of 3,500 lb/in^2 in 7 days and that the mix be capable of flowing and fully encapsulating the tendons. By the end of August , 1991, all the stressing of the tendons on the arch was completed. This began the 100-day evaluation period. By mid September, 1991, all the thrust block tendons were stressed.

Upon completion of the stressing of thrust block tendons, grouting of the free length was conducted. The strand "tails" were then cut and the blockouts were backfilled with reinforced concrete to protect the head assembly from physical damage and corrosion.

During the 100-day evaluation period, Reclamation monitored the movements of the arch after the application of the load. If during the period any adverse defections or long-term trends were observed, Nicholson had the equipment onsite to detension the tendons and relieve the load.

The 100-day evaluation period for the stressed tendons within the arch was completed in early December 1991. Instrumentation and survey data indicated that there was no significant movement of the dam as a result of tendon loads. It had been decided that all tendons with final lift-off readings of less than 108.5 percent of design load (DL) would be restressed. Any tendons which had relaxed below 108.5 percent of DL would be restressed to at least 108.5 percent DL. None of the tendons required restressing.

The free length was grouted without significant problems using the same mix design of 0.45 water/cement ratio and with 0.5 percent of superplasticizer. The grout tube on tendon 32 was initially plugged but was eventually cleared and grouted. Due to the possibility of hydrofracturing the dam concrete, the grouting procedure for the free lengths in Holes 37 through 47 was modified. Following placing and setting of the first stage of free length grout, the tremie was perforated, allowing the rest of the hole to be filled. Upon satisfactory completion of grouting, the strands were cut and blockouts backfilled with reinforced concrete.

LESSONS LEARNED

Negotiated contract. - The negotiated type of contract was particularly beneficial in this situation since the work involves a highly specialized, rapidly developing and complex operation. With a negotiated contract, the Contractor provided from his experiences a technical proposal that provided the assurance and solutions that the equipment and personnel were capable of performing the work and that there were sufficient contingencies to correct potential problems should any occur. Also, a negotiated contract allowed for written and/or oral dialogue with the contractor, and ensured that only the more qualified contractors remained eligible during the best and final offer process.

Pullout test. - The test program helped determine the actual bond lengths and verified the design assumptions, but also served as a training opportunity for designers, inspectors and contractor. The equipment and techniques were adjusted to the conditions at the site eliminating the majority of the procedural problems prior to beginning actual production drilling. Most of the inspection staff and designers were provided an opportunity to observe the installation and tensioning of tendons. Also fundamental data on rock anchor performance were provided for the international community (Scott and Bruce, 1992).

Borehole survey tools. - Seeker or similar gyroscopic survey tools are best suited for inclined boreholes where tight drilling tolerances are required. The uncertainty of the survey needs to be considered in the selection of the survey tool. For vertical holes, magnetic survey tools maintain comparable tolerances and uncertainty, but bearing readings for magnetic devices on holes inclined from vertical at 30° have uncertainties of approximately 0.9° (18 in/100 ft) [Wolf and deWardt, 1981]. For conditions where magnetic influences can impact readings, the gyroscopic tool is ideally suited. The principal limitation of the device is that it is expensive to run and maintain, and as are all bore hole survey instruments, they are sensitive to vibrations. The best advantage of the gyroscopic tool is that it can be used within the drill rods, and so minimizes delays.

The use of the optical theodolite by Reclamation is considerably less expensive and apparently more accurate. The principal drawback to the system is that it is a line of sight instrument and therefore cannot be used in curved holes to any great depth and cannot be used with water or other fluid in the hole.

Down-the-hole hammer. - The use of the down-the-hole hammer when operated by qualified personnel and proper equipment can maintain a 10-inch borehole within a 9-in/100-ft alignment to depths of over 250 ft for subvertical holes to 30° off vertical. This hammer, considered in some circles as a potential source of vibration and cause of cracking and widening cracks, was not a problem at Stewart Mountain Dam. There were no indications of doglegs within the drilled holes. The hammer was capable of advancing through steel without significant problems on drilling rates or deflections. However, it should be pointed out that even with qualified personnel and the best equipment, four tendon holes (77, 78, 79, and 80), failed to fall within the tolerance specified until adjustments were made to the rig's setup.

Water testing of tendon holes. - The recommended water testing requirement of 0.001 gallons per minute, per inch diameter of hole, and per foot of interval is a very tight requirement. Packer tests within the foundation indicated takes in the range from 0 to 23 lugeons. Regrouts within the bedrock were required up to four times in one hole before acceptance. Water testing recommended by PTI does not take into account the surrounding sidewall conditions, and multipressure water testing criteria should be considered. Although not tested except during the pullout testings, the best criterion would be 0.05 gal/min/lb/in^2 over 10-ft length or 10 lugeons (Littlejohn, 1975). The only other problem was that water testing should not be performed prior to the hole reaching static equilibrium in order to prevent unnecessary grouting.

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TECHNICAL SESSION 6

Waste Management

ACCOUNTING FOR BOUNDARY LAYER EFFECTS IN THE MODELLING OF LEACHING FROM MONOLITHIC WASTE FORMS

By G. D. Allen[1] and W. W. Pitt, Jr.[2]

Abstract: Standard leach tests for the determination of the rate of release of a contaminant from a monolithic waste form show disappointingly high release rates. Partly, this is due to what is commonly called wash-off. The purpose of this paper is to apply physical considerations to develop a mathematical model for the leaching process that accommodates both the short-term rapid wash-off phenomena together with the long-term slower leach rates. The idea is to introduce a boundary layer which has initial concentrations of the contaminant and diffusivity higher than that in the interior. With appropriately chosen parameters, excellent agreement to existing leach data is achieved.

1. INTRODUCTION

The ultimate purpose of proper waste disposal is to discard unwanted materials in a manner which protects the public health and the environment from undue harm. One potential for harm from disposed waste lies in the migration of harmful constituents in the waste to the environment via water. The total migration of waste from the storage facility to the groundwater supply is an enormously complex process that has been studied for at least 25 years and is still a *hot* topic today. It involves several independent and/or coupled processes, each of which is complex itself. Among these are water penetration into the facility (Ahn and Suzuki, 1990), leaching from the waste form (Ahn and Suzuki, 1990), deterioration of the container and deterioration of the solid phase (Matsuzuru and Suzuki, 1989), migration through the backfill materials (Ahn, 1990), structural integrity of the facility, and migration external to the facility to the groundwater supply (Lever et al., 1983).

We focus here on the transport process of leaching. The rate of transport can be dramatically reduced by the stabilization/solidification of the wastes in a large monolith. In such a monolith, for example a cementitious form, there are numerous transport mechanisms including diffusion, advection, absorption and convection. In particular, we assume here that the solid phase is immersed in water; this eliminates all but diffusion as the dominant transport mechanism.

In this paper we continue our study of the mathematical modelling of the short-term leaching of a radionuclide from a cement paste waste form immersed in water (Pitt and Allen, 1992). Our principal goal is to determine long-term behavior of the Cumulative Fraction Leached (CFL) with extreme short-term leach data; "extreme short-term" means days. We assume a given geometric structure of the waste form and the knowledge of an effective long-term diffusion coefficient for the leachate.

We have determined that a number of factors are significant for achieving a mathematical model that gives good approximations to actual data. Before describing them, let us note that these same factors have not been considered by other investigators (Ahn, 1990; Godbee, 1980; Matsuzuru, 1978). Rather, they have concentrated on general theoretical solutions (usually of series type) of the diffusion equation based on a constant effective diffusion coefficient together with an empirical (mathematical) model which is "tuned" to obtain good agreement with experimental results (Cheng and Bishop, 1990; Spence and Godbee et al., 1991). See also Crank (1975) or Carslaw and Jaeger (1948) for general treatments of diffusion. While

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this approach may well yield good agreement with experimental data, it lacks a fundamental underpinning that gives credibility to long-term predictions.

In this paper we have attacked the problem of long-term prediction from short-term data using what we consider a fundamental physical basis. Our approach uses the presence of the inert filler material as actually present and a barrier to the leaching process, rather than as a factor contributing to the effective diffusion coefficient D_e. In addition, we have modelled the generally unstudied phenomenon of short-term "wash-off," a well observed phenomenon of very rapid diffusion of the leachate that occurs before the long-term effects become dominant. This short-term duration is usually less than 10 days.

The benchmark data with which we will compare our numerical results are the CEN-Saclay Long-Term Leach Data for the very mobile species cesium (Nomine et al., 1989). Various sized cylinders were used. All indicated very low long-term leach rates of 10^{-11} to 10^{-12} cm^2/sec. Unfortunately, as can be seen in Figure 1, these low leach rates were not observed until 30 to 60 days into the experiments. These low rates, which have persisted for more than 700 days, would not have been so measured in standard short-term leach tests, such as ANSI 16.1 (ANSI/ANS 16.1, 1986).

In the following sections we describe, in detail, exactly what factors our model includes. In particular, in Section 2, we state in general terms the mathematical model we employ. In Section 3, we detail the model for the diffusion coefficient and for the initial concentration. Finally, in Section 4 we discuss our numerical results. It is important to emphasize here that even with the physical (as opposed to empirical) approach developed here, there are significant empirical considerations included. These involve the numerical aspects of an introduced "boundary layer." While most investigators may agree that there is in fact a boundary layer effect as part of the short-term leaching process, there seems to be a complete lack of literature about it.

2. THE MATHEMATICAL MODEL

The model approach taken in this paper is, roughly, to numerically solve a one-dimensional model of a diffusive process, wherein the spatial variable, x, denotes the distance from the surface (of the three dimensional solid form). However, this seems to imply that the spatial requirement of the model should involve a half-line partial differential equation. Such problems, especially of the type we need, are difficult to solve numerically and impossible to solve analytically. Therefore, we take x as a finitely limited spatial variable, with x in $[0,1]$, generating a kind of slab geometry in which the diffusion of the leachate from both ends must be summed. With respect to the three-dimensional waste form, we interpret these values as the *per* surface area/volume diffusion. Thus, numerical output from the one-dimensional model must be corrected by the surface area/volume (S/V) ratio. Other normalization factors are also necessary and will be discussed below. Note that in this approach, one-dimensional results interpreted for a three dimensional problem are common. See for example (Godbee et al., 1980; Godbee, 1975; Pitt and Allen, 1992).

As indicated in the introduction, the diffusion coefficient, $D_e(x)$, to be used here is *not* constant. Application of Fick's law yields the appropriate model for the one-dimensional diffusive process. It should be

$$u_t = (D_e(x)u_x)_x, \quad 0 \le x \le 1, \quad t > 0, \tag{1}$$

where $u(x,t)$ denotes the concentration of the leachant at depth x (alternatively, at $1 - x$) and at time t. The initial condition is

$$
f_1(z) = \begin{cases} \dfrac{\pi}{2\sqrt{3}\,a^2}\,(r^2 - z^2), & \text{if } 0 \le z \le r, \\ 0, & \text{if } r \le z \le 2\sqrt{3}\,a, \end{cases}
$$

$$
f_2(z) = \begin{cases} \dfrac{\pi}{2\sqrt{3}\,a^2}\,(r^2 - (z - \sqrt{3}\,a)^2), & \text{if } \sqrt{3}\,a - r \le z \le \sqrt{3}\,a + r, \\ 0, & 0 \le z < \sqrt{3}\,a - r \text{ or } \sqrt{3}\,a + r < z \le \sqrt{2}\,a, \end{cases} \tag{5}
$$

$$
f_3(z) = \begin{cases} \dfrac{\pi}{2\sqrt{3}\,a^2}\,(r^2 - (z - 2\sqrt{3}\,a)^2), & \text{if } 2\sqrt{3}\,a - r \le z \le 2\sqrt{3}\,a, \\ 0, & 0 \le z < 2\sqrt{3}\,a - r. \end{cases}
$$

Then, in the interval $0 \le z \le 2\sqrt{3}\,a$, the effective diffusion coefficient $D_e(z)$ is given by

$$
D_e(z) = 1 - (f_1(z) + f_2(z) + f_3(z)). \tag{6}
$$

For x in the interval $[0,1]$, it is necessary to scale and translate the variable z so that there are as many periods of this function as desired. Taking the whole spatial interval, $[0,1]$ to represent one centimeter and taking a particle of inert filler to have a radius of about 0.01 centimeter, it follows that we should use about 40 periods. Note that the numerical results are relatively insensitive to higher numbers of periods, even up to 80. Next, it is necessary to consider the fact that there is a boundary layer on the surface of the form defined partly by the depth to the first layer (plane of close packed sphere centers) of filler material. We take this width to be a minimum of a units. With respect to the close-packed model, taking the minimum a implies that the form has filler spheres just touching (tangent to) the geometric boundary. In actuality, it is not known what the width of this boundary layer is.

An obvious exception to this model for $D_e(x)$ is that filler particles are not in general spheres of uniform radius, but they are polyhedra of various diameters and shapes. We conclude, therefore, that the model only serves to yield intuition about the functional nature of cross sectional occlusion by filler material.

Still, another boundary aspect is significant. In the boundary layer, it can be expected that since some of the pores are open to the surrounding environment, the diffusion coefficient may well be greater that in interior pure and closed pore cement. With the boundary layer of width w, we use the symbol D_b to denote this value. This factor is incorporated into our numerical model. For the CEN-Saclay experiments, with the boundary layer $w = 0.01$ cm, the percentage of boundary layer to total volume ranges from 0.8 percent to 2 percent.

Figure 2 illustrates the form of the diffusion coefficient for a specimen having a very thin boundary layer for various values of r. The coordinates are normalized to the interval $[0,1]$. Figure 3 illustrates, again with coordinates normalizes to $[0,1]$, the effective diffusion coefficient for a very thin slab, complete with a boundary layer of width $w = 0.1$ and with $D_b = 2$.

$$
f_1(z) = \begin{cases} \dfrac{\pi}{2\sqrt{3}a^2}(r^2 - z^2), & \text{if } 0 \le z \le r, \\ 0, & \text{if } r \le z \le 2\sqrt{3}a, \end{cases}
$$

$$
f_2(z) = \begin{cases} \dfrac{\pi}{2\sqrt{3}a^2}(r^2 - (z - \sqrt{3}a)^2), & \text{if } \sqrt{3}a - r \le z \le \sqrt{3}a + r, \\ 0, & 0 \le z < \sqrt{3}a - r \text{ or } \sqrt{3}a + r < z \le \sqrt{2}a, \end{cases} \tag{5}
$$

$$
f_3(z) = \begin{cases} \dfrac{\pi}{2\sqrt{3}a^2}(r^2 - (z - 2\sqrt{3}a)^2), & \text{if } 2\sqrt{3}a - r \le z \le 2\sqrt{3}a, \\ 0, & 0 \le z < 2\sqrt{3}a - r. \end{cases}
$$

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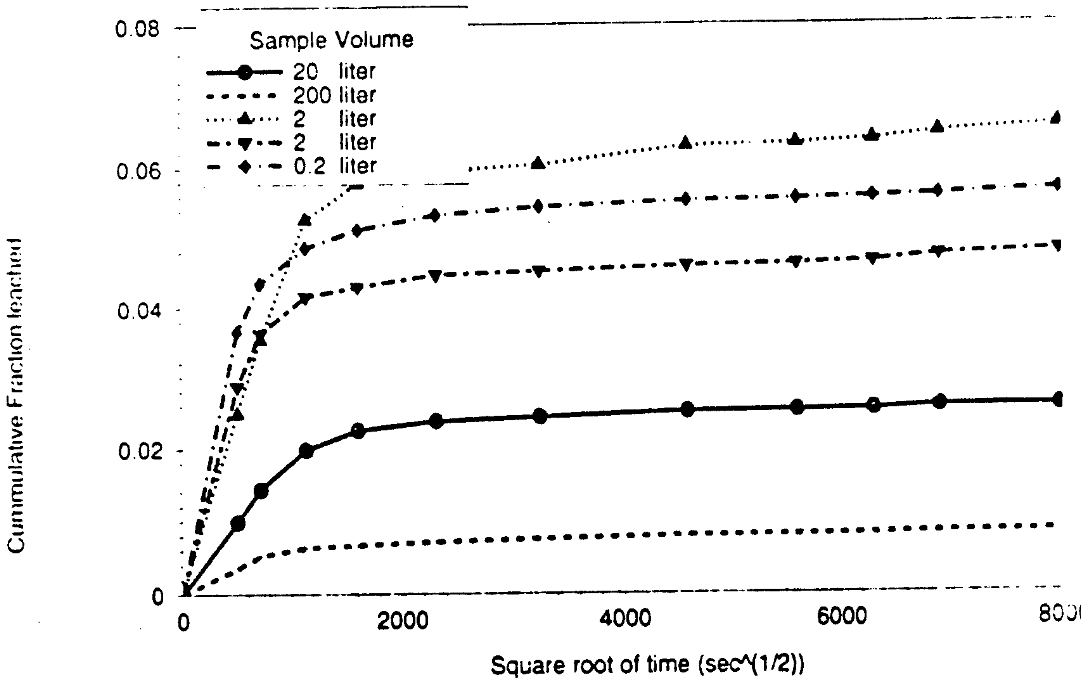

Figure 1. - Long-term leach data (CEN-Saclay).

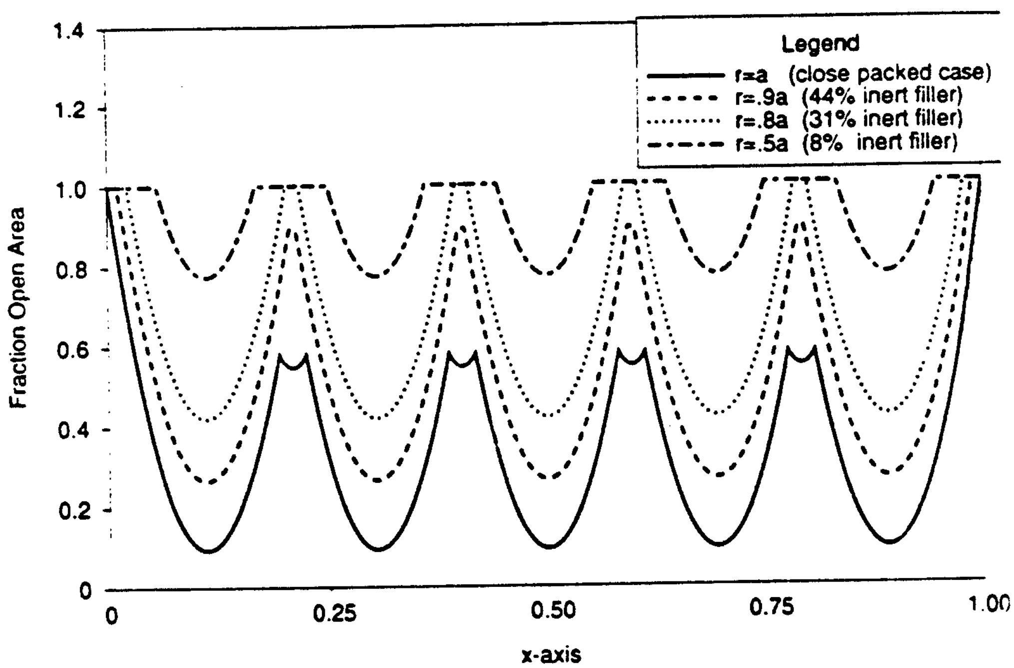

Figure 2. - Examples of the diffusion coefficient for various radii.

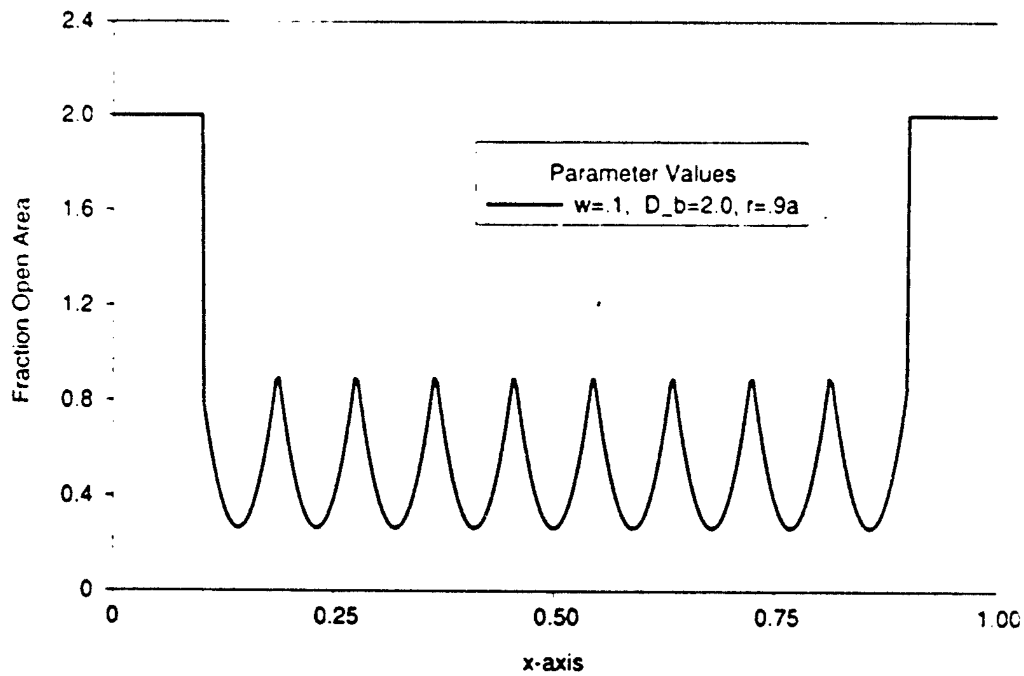

Figure 3. - Diffusion coefficient for r = .9a with boundary layer.

Let us note that we have observed experimentally, but we have not established analytically, that

$$\lim_{t \to 0} \frac{U(t)}{\sqrt{t}} = \kappa \frac{(D(0) + D(1))}{2},$$

where κ is a constant independent of $D_e(x)$. It is apparent that the very steep initial slope in the CFL curve depends greatly upon the diffusion coefficient in the boundary layer.

In conjunction with the boundary layer, we need the definition of the normalized leachate concentration there. With w as the boundary layer width, we define

$$C(x) = \begin{cases} C_b, & \text{if } 0 \le x \le w, \\ 1, & \text{if } w < x < 1 - w, \\ C_b, & \text{if } 1 - w \le x \le 1. \end{cases} \tag{7}$$

The value C_b, the boundary layer concentration, is generally larger than 1. The magnitude of this value has a considerable significance on the output of the numerical model in the extreme short-term, but not in the long term.

An exception to the defined constant values D_b and C_b defined above is that perhaps they should not be constant. Though this is probably true, its effect on the short-term or even long-term results obtained is small, at least numerically. At this stage of our investigation, little insight can be gained by such a *fine*-tuning of a model without specially constructed concrete forms to test it.

4. RESULTS

The principal result of this paper is to illustrate that with the introduction of the boundary layer paradigm, it becomes possible to obtain numerical results in good agreement with the benchmark CEN-Saclay data. It is necessary to assign the three new and important parameters introduced in §3. These parameters, the

- Boundary layer width, w,
- Diffusion coefficient in the boundary layer, D_b, and
- Concentration profile in the boundary layer, C_b,

have little or no treatment in the literature, and what their true values may be in a three-dimensional solid form is unknown. Therefore, we have selected values that are both physically reasonable and yield good results.

The output from one sample run is shown in Figure 4, using the parameter settings: $w = 0.002$, $C_b = 14$, and $D_b = 6$. Keep in mind that these parameters are relative. For example, $C_b = 14$ means that the concentration of the leachate in the boundary layer is 14 times what it is in the interior. Of course, the numerical output can be modified to generate other shapes. For example, with $w = 0$, there is little evidence of the initial steep slope observed in the CEN-Saclay data. With $C_b = 1$, we cannot achieve the relatively large *wash-off* effect. Certainly, fine, virtually microscopic experiments are needed to determine the nature of these values.

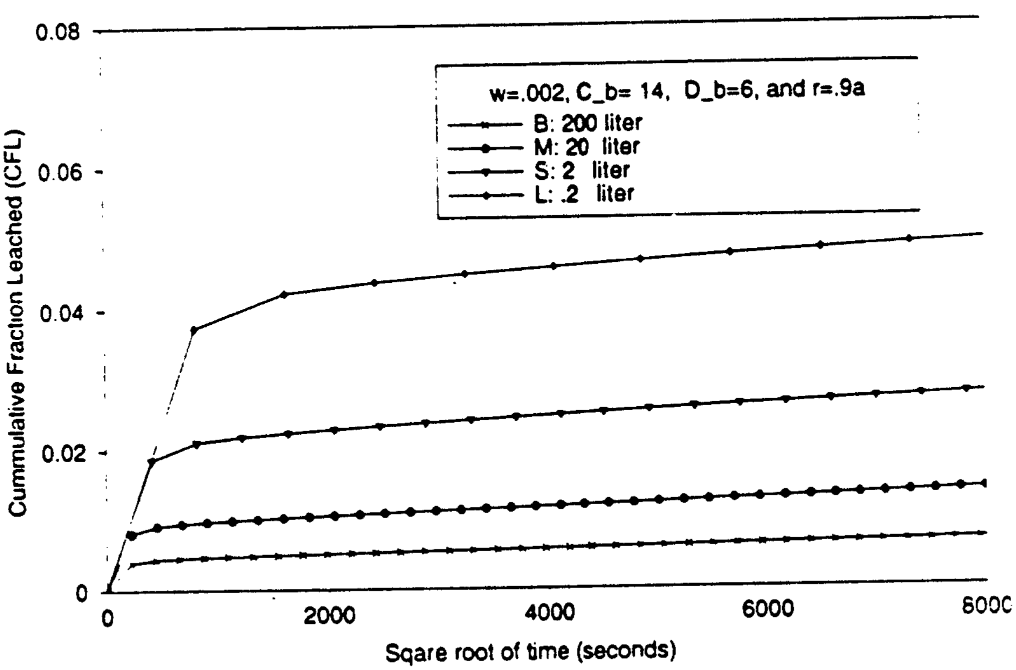

Figure 4. - Boundary layer model.

Numerical Procedure Details

The main PDE solver can be found in the IMSL Library. Typical computer runs used 1000 subdivisions of the spatial interval [0, 1]. These values are taken as the Chebyshev points (Conte and de Boor, 1975) to give not only a finer mesh in and near the boundary layer but also to avoid discontinuities of the derivative of $D_e(x)$. Recall Figure 3. The boundary layer for the parameters above was about 27 mesh points deep. We note in passing that this permits implicitly the assumption that $D_e(x)$ does not have corners (see Figures 2 and 3) but is continuously differentiable. Time steps were selected nonlinearly, but in fact linearly in square root time, so as to employ very small time values at the onset of the solution. The solid circles, etc..., in Figure 4 indicate actual output-value times. The relatively different placement of the indicators on the separate graphs is accounted for by the fact that the different sized test waste forms have different long-term effective diffusion coefficients, and this was incorporated into the output though a change of time scale. The computer runs were made at the Texas A&M University Supercomputer Center on the Cray Y-MP2/216.

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RETROFITTING PERIMETER LEACHATE COLLECTION SYSTEMS FOR EXISTING SANITARY LANDFILLS: A CHALLENGING PROBLEM

by T. R. West[1]

INTRODUCTION

During the 1950s and 1960s in the United States, solid waste was typically disposed of in open dumps. Fires were common and in some cases were set intentionally to reduce the fill volume. Rodent and bird activity was prolific. In many areas, only small equipment such as farm tractors with push blades were used to grade the trash. Fires were difficult to extinguish because burial with thick enough soil layers to smother the fire could not be accomplished with this small equipment.

In the 1970s as design considerations progressed, the concept of sanitary landfills developed. The trash was covered with 6 inches of soil each day which would prevent fires and discourage rodents and birds. Large earth-moving equipment, typically scrapers, was now required to supply cover soil for the trash. Dozers and compactors were necessary to level the trash and reduce its volume to minimize the exposed surface and the volume to be covered. Individuals with earth-moving knowledge and maintenance capabilities soon gravitated to the landfill business.

It was also realized that infiltration of rainfall into the landfill could be greatly reduced if daily cover consisted of clay or silty clay rather than sand or similarly permeable material. Prior to this time, landfills were placed in abandoned sand and gravel pits and the available, sandy soils were used as the cover material. Both infiltration through the cover and migration of leachate away from the landfill were problems for these early landfills (West, 1990).

Eventually two types of design concepts developed in the 1970s: natural attenuation landfills (NA) and containment type landfills (Bagchi, 1983). In natural attenuation landfills, leachate is allowed to percolate through the surrounding soil with the hope that leachate quality will be sufficiently improved by the soil action before reaching an aquifer or stream. Thickness of the clay interface below the trash and the cation exchange capacity (CEC) of the clay indicate the ability of the soil to attenuate leachate constituents (Indiana Department of Environmental Management, 1988). Experience has shown that some degradation of water quality is inevitable as soils are unable to remove all the constituents from landfill leachate (for example, chloride). Only dilution can accomplish a reduction in concentration for these constituents (Bagchi, 1990).

In containment-type landfills, as the name implies, the objective is to contain the leachate at the site rather than allow it to percolate away through the soil. A low permeability liner (natural or man-made) is used to prevent leachate migration, and a piping system collects the generated leachate. Gravity flow to a sump pit accumulates the leachate which can be pumped out on a regular basis. Disposal of leachate at a municipal water treatment plant (POTW) is typically accomplished.

Two basic categories of landfills are considered in the United States; hazardous and non-hazardous. Hazardous waste typically involves industrial wastes from chemical and manufacturing processes that

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include materials that are corrosive, toxic, ignitable or reactive. Depending on the concentration level of these materials and whether solid or liquid, they must be disposed of in a hazardous waste landfill or be incinerated (liquids). The procedures for siting, design, and construction of hazardous waste landfills are similar to those involving municipal solid waste landfills, except that for hazardous sites the requirements are considerably more stringent. Because of the nature of the materials involved, hazardous waste sites are designed as containment-type landfills. These are termed engineered or secured landfills.

Therefore, today, the two major landfill types are (1) conventional solid waste landfills and (2) hazardous waste landfills. Municipal solid waste is placed in the conventional solid waste landfill. Materials consist of garbage, trash, and other household products. Due to the high cost of construction today, hazardous waste landfills receive only those materials classified as hazardous. This typically involves industrial wastes from chemical and manufacturing processes that include materials that are corrosive, toxic, ignitable or reactive. No free liquids are allowed so that materials are solidified before placement. As indicated above, hazardous waste landfills are constructed as containment-type landfills.

Prior to late 1981 when RCRA went into effect, industrial chemicals were commonly disposed of in conventional landfills along with municipal solid waste. In some States of the United States, industrial wastes were termed special wastes and only a specific volume was allowed in addition to the greater amounts of municipal waste. This procedure was termed "co-disposal." Consequently, many of the older landfills in operation before 1981 contain significant amounts of industrial waste because of the overall composition of materials dumped there. Others contain significant volumes of drummed hazardous waste from co-disposal operations. Also, as these landfills were constructed during the early years of landfill development, they were designed as attenuation type landfills which lack leachate collection systems.

Hence, we find that landfills constructed in the 1960s and 1970s were not only attenuation type landfills, but many contained significant amounts of industrial wastes which today would be classified as hazardous materials. Leachate from many of these so-called conventional landfills contain significant amounts of toxic constituents. It has been concluded that old landfills both conventional and the so-called hazardous types contain toxic constituents in their leachate. Therefore, it is only a matter of degree for the extent of contamination between them. Typically these leachates contain elevated amounts of the common inorganic cations and anions, heavy metals, pesticides, and various organics, including volatile organics, PCBs, dioxins, and chlorinated hydrocarbons (West, 1990).

An interesting question can be raised. Will leachate from modern day, conventional waste landfills also show significant accumulations of toxic constituents? With stricter limitations on the materials placed in these landfills, it may develop that this leachate will be much less toxic than that from common landfills of the 1960s and 1970s. Also today, all new, conventional landfills tend to be containment landfills. Many are constructed using manmade liners (geomembranes) but properly designed and constructed clay barriers can provide proper containment as well (Daniels, 1987).

RETROFITTING LANDFILLS WITH LEACHATE COLLECTION SYSTEMS

Containment landfills are designed and constructed to provide (1) a low permeability liner that restricts leachate from moving away from the landfill and (2) a piping system that collects the leachate along the bottom of the trash fill. Natural attenuation landfills lack both a constructed liner at the landfill base and a leachate collection system to intercept leachate from the trash.

Natural attenuation landfills and containment landfills are illustrated in Figures 1 and 2 respectively. Leachate collection systems developed chronologically as two basic types. The first is a perimeter drain system which extends around the boundary of the fill area, and the second is a piping system located below the base of the landfill as mentioned above and illustrated in Figure 2. This type is known as a basal leachate collection system.

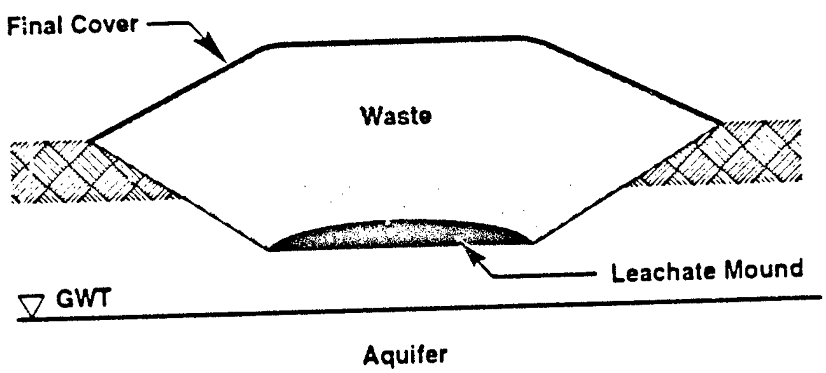

Figure 1. - Natural attenuation landfill.

Conventional landfills, those taking MSW (municipal solid waste), constructed in the 1970s and much of the 1980s, were natural attenuation types. When groundwater contamination was detected in adjacent monitoring wells, a common method for improving conditions was to install a perimeter leachate collection system around the existing landfill. In other instances, some conventional landfills were upgraded by adding perimeter collection systems even though no contamination was detected in the monitoring wells.

A perimeter drain leachate collection system is illustrated in Figure 3, shown in a cross section view. A cutoff wall, typically constructed using a slurry wall trench, is also shown. In some of the early retrofitted landfills, the cutoff wall was not included, particularly if the landfill base was a clay or silty clay soil. The purpose of a cutoff wall is to prevent adjacent groundwater from moving toward the landfill, which must be collected by the perimeter leachate system. This additional, unnecessary volume adds to the cost of treatment. In sandy or other highly permeable soils, a cutoff wall is necessary to prevent the increased inflow of water from the adjacent area.

Because many of the early retrofitted perimeter drains were constructed on a clay base, only small quantities of leachate were collected. Several thousand gallons of leachate collection per month for landfills 50 to 100 acres in size would be typical. This is considerably less than the estimated amount of infiltration through the landfill cover which should eventually appear as leachate. The example below is provided to illustrate this point. Assume a 50-acre landfill, 36 inches of rainfall per year and 10 percent infiltration through the landfill cover.

> 50 acres x 43, 560 ft^2/acre x 3 ft x 0.1 = 6.53 x 106 ft^3 of infiltrated water/yr.
>
> 6.53 x 106 x 7.48 gal/ft^3 = 4.89 x 107 gal/yr
>
> 4.89 x 107 x 1/12 = 4.07 x 106 gal/month ~ 4 million gal/month. This is more than 1000 times the leachate volume mentioned above.

From this example, it is clear that leachate escapes via a number of possible pathways. Included are (1) leakage through the landfill base, (2) leakage through the landfill sides forming leachate seeps, and (3) evapotransportation of infiltrated water.

For new conventional (MSW) landfills, the leachate collection system extends fully across the landfill base and is designed to accommodate the total volume of infiltration. This could range as high as 20 percent of the annual rainfall during the active years of landfill construction and 5 to 10 percent after closure. A sand layer below the trash, encapsulating the drainage pipes, encourages the migration and removal of leachate.

Retrofitting partially completed landfills can be accomplished as shown in Figure 4. The existing landfill is to be extended both laterally and vertically. A compacted sand layer is placed over the existing side slope of the landfill, and a barrier layer is constructed above the existing slope and extended over the base of the landfill addition. Piping is distributed over the base, including the former landfill slope, and an appropriate basal leachate collection system is constructed. Trash is placed over this base to the prescribed height of the new landfill. Leachate is collected by gravity flow in a sump pit, and subsequently pumped out for disposal.

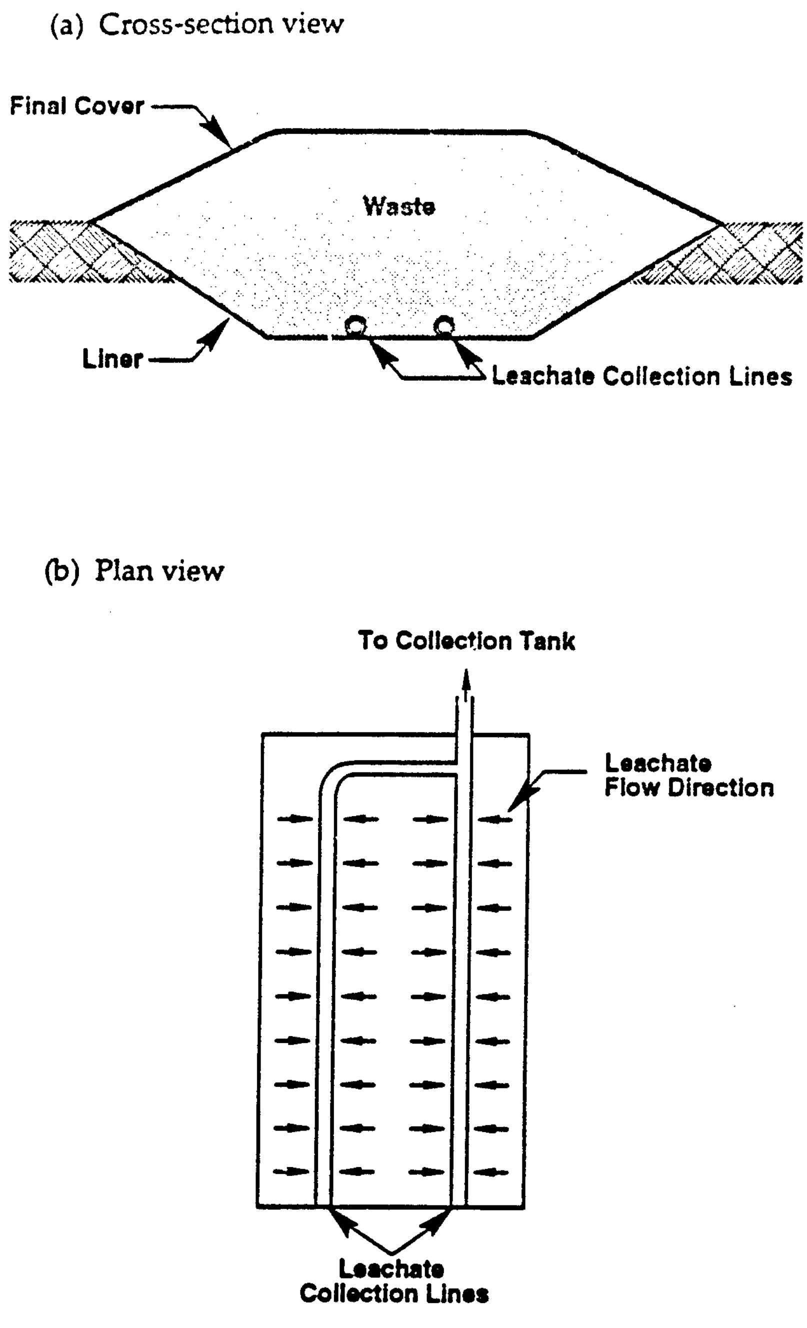

Figure 2. - Containment landfill.

A perimeter leachate collection system can also be added along the boundary of the existing landfill where no additional trash is to be added. This would be on the left side of Figure 4.

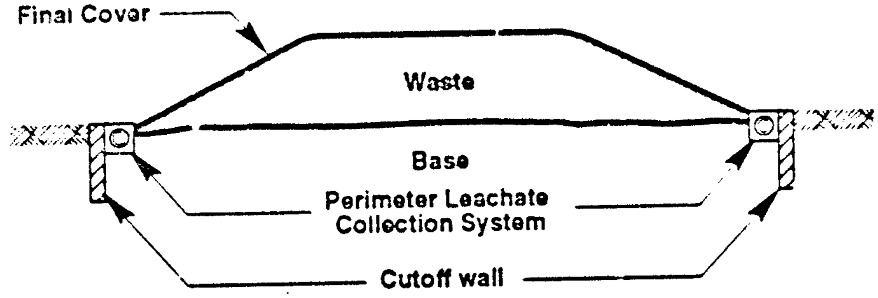

Figure 3. - Retrofitting an existing landfill with a perimeter leachate collection system.

CASE HISTORY EXAMPLES

Several examples of conventional and co-disposal landfills which have been retrofitted with leachate collection systems are discussed in the oral presentation. Several related design concepts proposed for other landfills will also be reviewed. These include typical examples of landfills in soils of the upper Midwest where glacial deposits prevail (West, 1989) and landfills in residual soils beyond the glacial boundary (West and Loughnane, 1988). The limited ability of clean sandy soils to attenuate chemical constituents from waste water effluent is also considered (LeBlanc, 1982).

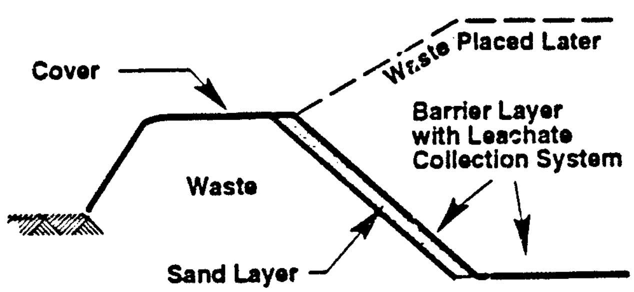

Figure 4. - Retrofitting a partially operating landfill with a basal leachate collection system.

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BASIC RESEARCH ON S/S MECHANISMS: IMPLICATIONS FOR PRACTICE

By Frank K. Cartledge[1] and Marty E. Tittlebaum[2]

Abstract: This paper presents several examples in which basic research into S/S mechanisms has led to conclusions with significant consequences for the practice of S/S. A preformed hydroxide sludge solidified with portland cement has been shown to be primarily a physical mixture of sludge and hydrated binder. Such cases can be expected to show leachability determined by the solubility properties of the sludge and the water transport properties of the matrix. The latter can in principle be controlled by the cement formulation, which alters density and porosity. Cr(III), As(III) and As(V) salts have been shown to have major effects on cement matrices, and sometimes the effects only become apparent after cure times of months, not days. These results mean that monitoring of the S/S products, particularly leach testing, needs to be carried out over much longer periods of time than current practice dictates. The interaction of nonpolar organics with organoclay additives follows a partitioning mechanism. As a consequence, basic properties of organic materials, such as octanol-water partition coefficients, can be used to give a good quantitative estimate of the concentration of the organic in TCLP leachates when S/S is carried out with cement and organoclays. These examples show the importance of research for S/S practice and give promise of many more conclusions that can result in conversion of the technology from an art to a science.

INTRODUCTION

Solidification/stabilization has been the subject of a great deal of research, much of it applied and often proprietary. Most of this work has been in the form of treatability studies, looking first for combinations of waste, binders and additives that result in solidification, then measuring properties of the solidified material, most often some combination of strength, permeability and leachability measurements. This kind of work has given rise to a large body of information, mostly not organized, about what combinations of wastes and solidification reagents work in practice.

There has been much less work of a fundamental nature, but what has been done falls into two broad categories: (1) exploration of the chemical and physical nature of the binding between waste and S/S matrix, and (2) modeling leaching behavior of solidified systems. Of course, the latter can make use of the former, and it is mainly research of the former type that will be treated in the present paper. Of course, ultimate goals of the mechanistic research are to understand the immobilization process and to use that understanding to predict and improve performance. The present paper will deal with research into cementitious and pozzolanic processes. Such systems are sufficiently complex that complete understanding of immobilization mechanisms is not possible. Nevertheless, we hope to show that research can lead to significant conclusions that have important consequences for practice. There are two broad mechanisms that restrict the leaching potential of wastes treated by S/S: (1) encapsulation that limits water transport and (2) chemical/physical binding that limits the partitioning of the waste into the aqueous phase. Just determining which of these two mechanisms predominates has practical implications. If encapsulation is the primary mechanism, then formulation of the waste/binder system in order to limit permeability is the major concern. If one then knows the rate of water transport in the binder and the solubilities of the

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waste-containing species, one can predict leaching behavior. If, on the other hand, real chemical/physical binding takes place, then there are almost unlimited possibilities for optimizing the interaction in order to maximize immobilization. However, intelligent optimization requires clear information about the chemistry of the system, and that is hard to come by. It is still possible to make predictions about leaching behavior, but the predictions will have to be based upon empirical data generated for the specific waste/binder system being treated.

ENCAPSULATION VS. CHEMICAL BINDING

Encapsulation on a microscopic scale is widely believed to contribute to immobilization. We have found one clearly documented case in which physical encapsulation occurs (Roy, 1992). The study utilized a synthetic electroplating sludge solidified in Type I Portland cement (OPC). The sludge contained Ni and Cr (mainly Cr^{3+}) at concentrations of about 80 mg/g, Cd at about 20 mg/g, and Hg at 0.1 mg/g. It was prepared by mixing metal nitrate salts in water, then precipitating with lime and dewatering. For solidification, the cement to sludge ratio was 1.2 and the water to cement ratio was 1.43. In this case the mechanistic conclusions were based on electron microscopy, energy dispersive X-ray microanalysis (EDX) and X-ray powder diffraction (XRD). Examination of the sludge alone and of cured OPC alone provided two baselines against which to compare the sludge/cement mixtures.

Only two phases could be identified unambiguously from the XRD diffractogram of the sludge: calcium hydroxide and calcium carbonate. There were other crystalline materials present, but an extensive computer search failed to clearly match the remaining peaks with any well-characterized phases in the powder diffraction files (Powder, 1986), including hydroxides of the heavy metals. Among the possible matches, only $Ni_3O_2(OH)_4$ gives any close matches, but missing peaks and intensity differences preclude making positive identification. Actually, it is not surprising that the XRD studies of the sludge did not allow identification of pure crystalline hydroxide salts. The sludge was prepared from lime addition to a solution of nitrate salts, and thus contains major amounts of nitrate, some carbonate from atmospheric CO_2, and probably other anions in smaller amounts. Precipitates formed from the addition of base to aqueous metal salts are notorious for producing many things other than binary, easily characterizable, crystalline metal hydroxides (Feitknecht, 1953). Salts containing halide anions in addition to hydroxide are particularly well known, but many examples of hydroxy nitrate salts are also known. The dewatered sludge is a very complex mixture, and metals can be present as noncrystalline hydroxide gels, as nonstoichiometric salts with more than one metal ion in the same lattice, as complex salts with more than one anion, or as solid solutions. All of these effects make XRD identification difficult.

Diffractograms of sludge/OPC mixtures were nearly simple sums of the diffractograms of sludge alone and cured OPC alone. There were minor differences in relative intensities of unidentified peaks, and there were larger proportions of unhydrated cement clinker minerals than present in cured OPC. But the implication of a physical mixture is strong.

Not surprisingly, EDX analysis of the sludge showed mainly Ca, Cr and Ni, with small amounts of Cd. Some variation was seen in Cr and Ni concentrations from one particle to another, but the approximate metal composition was 80 percent Ca and 20 percent (Cr + Ni). SEM examination of sludge/OPC mixtures showed considerable variation, but some of the samples contained abundant ellipsoidal particles that were tens to a few hundred µm's across. Usually these particles were separated from the surrounding matrix by an obvious void space. We have never seen such features in OPC alone. When EDX analysis is carried out on the ellipsoidal particles, the usual sludge metal composition is seen (80 percent Ca and 20 percent Cr+Ni). In the surrounding matrix, much smaller proportions of Cr and Ni are measured.

The investigation described above indicates that entrapment was mainly physical, the waste being confined to large particles, tens to hundreds of micrometers in diameter, embedded in the OPC matrix. The waste was also present in the matrix at low levels, but whether they were different chemical species than those in the sludge particles could not be determined using the methods of that study.

It appears that cementitious S/S can involve either chemical transformations of wastes or microscopic encapsulation. A number of studies to be referred to below have shown that the chemical forms of metal ions within a cement matrix are different from the form present in the aqueous waste. The results just described are the clearest evidence that microscopic encapsulation is important. It may be that the latter is the usual case when a preformed "hydroxide" sludge is solidified, but it is not yet clear whether the microencapsulation phenomenon is common. The two varieties of S/S should give different problems to modelers who attempt to predict long-term leachability of wastes from cement matrices. The ultimate goal of modeling would be to predict release rates based on thermodynamic properties of the specific chemical species involved. Those, of course, will be different depending upon whether the waste/matrix interactions involve chemical change or only encapsulation. The latter may be most successfully modeled based upon the leachability of the sludge, with a minor correction for transport through the cement matrix. Whatever form the modeling ultimately takes, it is important to know whether chemical transformations of the waste take place during S/S or whether they do not.

There are fairly well-defined procedures that can be employed during cement paste preparation that result in a denser, less permeable matrix. For instance, limiting water availability, often through inclusion of a superplasticizer, and filling pore spaces with glassy spheres from fly ash or silica fume has the desired effect. It is not yet clear, however, even in the cases where physical entrapment is the likely immobilization mechanism, that densifying techniques actually limit leachability in S/S products. A pure cement paste is a quite different system from one containing 50 percent or more foreign material (waste). Even the morphology on a microscopic scale is different for S/S products than for pure cement pastes, and the results above showed the presence of more unreacted clinker minerals when waste sludge was present. Nevertheless, effects of permeability alterations on leachability is a research topic that needs to be seriously explored in experiments where the number of variables is controlled.

CHEMICAL CHANGES

Most of the studies that have concentrated on attempting to understand the chemistry of wastes and matrix under S/S conditions have used systems that simulate treatment of aqueous waste streams. That is, heavy metal species in aqueous solution are mixed directly with cement or pozzolans with the wastewater serving as the water of hydration. For some metals there may be little difference between the results of pretreatment with $Ca(OH)_2$, then addition of cement, and the alternative in which an aqueous solution of a metal salt is added to solid cement. Clearly, one expects hydroxide precipitates to be formed in the latter case as well as the former.

Nevertheless, there is at least one case that is very different. Cement treatment of the pre-formed sludge described above results in most of the Cr(III) remaining in the sludge, presumably in the same chemical form. The results are quite different when aqueous Cr(III) is treated with cement. Cr(III) shows qualitatively different behavior from any other metal studied under S/S conditions. For instance, Pb is concentrated at surfaces in cured cement matrices, whereas Cr is not (Cocke et al. 1989), and it has even been proposed that Cr is actually incorporated into the developing calcium silicate hydrate (C-S-H) gel phase (Ivey et al., 1990).

We have investigated metal, including Cr(III), effects on cement matrices using solid-state NMR spectroscopy, and Figure 1 shows Si-29 NMR spectra after 28 days and after 1 year of cure for samples prepared from soluble metal salts and OPC, in each case containing 10 percent by weight of metal. In most cases the spectra show rather well-defined resonances at -71, -79 and -84 PPM (relative to tetramethylsilane) corresponding to silicon atoms in Q^0 ($\underline{Si}O_4^{4-}$), Q^1 (-$SiO\underline{Si}O_3^{3-}$), and Q^2 (-SiO-$\underline{Si}O_2^{2-}$-OSi) silicate units. Cr(III) has the most dramatic effects among the metals shown. It greatly retards silicate polymerization that is typical of cement curing reactions, since the proportion of Q^0 units is quite high even after one year of cure. Secondly, the polymerization that does occur shows up as a broad resonance without clearly defined peaks for Q^1 and Q^2 units. Clearly, the structure of the silicate matrix is quite different from OPC alone and from the other metal-containing cements shown in the figure. The broad resonance in the silicate region when Cr(III) is present is a reasonable effect to see if Cr is actually substituting within the silicate matrix, as suggested elsewhere.

Wherever the Cr is located in the matrix, it is clearly not readily available for leaching. Table 1 shows TCLP leachate concentrations for several metals solidified in OPC at 10 percent by weight metal. The numbers in parentheses in the table give the concentrations of metal observed in the leachate when the metal was solidified alone in OPC. For instance, the value of 0.4 mg/L represents the concentration of Cr in the leachate when $Cr(NO_3)_3 \cdot 9H_2O$ is solidified in proportions to give a Cr/OPC weight ratio of 1/10. Very little Cr is available for leaching. However, the fact that Cr(III) causes major differences in the cement matrix suggested to us the necessity to determine whether those differences result in a change in leachability for other metals that might be solidified along with Cr(III). The numbers not in parentheses in Table 1 show some data on binary mixtures of Cr(III) and Cr(VI) with As(III) and Pb(II). The results show that both Cr(III) and Cr(VI) improve the performance with regard to immobilization of Pb, but both degrade performance with As(III). Cr(III) and Cr(VI) are both hydration inhibitors through 28 days, so their effect on As immobilization is not too surprising. The effect of Cr(VI) on Pb immobilization is also not surprising because of the formation of insoluble $PbCrO_4$. However, the improved performance of Pb in the presence of Cr(III) *is* unexpected, and we only have speculative explanations at present. Perhaps Cr alters the C-S-H structure in a way which allows it to incorporate some Pb, decreasing the Pb concentration on surfaces.

The research cited above is still at an early stage of development. For instance, the concentration dependence of the Cr(III) effects is not known. But the results strongly suggest that when a metal has a major effect on the cement matrix, particularly Cr(III), prudent practice will dictate careful observation of leachability over a significant period of time in order to be confident about long-term performance of the S/S preparation.

Table 1. - TCLP Leachate Concentrations from Binary Mixtures of Solidified Wastes after 28 Days of Cure; The Binder Is OPC and Each Metal Is Present in the Sample at 100,000 ppm. The Value In Parentheses Is the Leachate Concentration of the Metal Mixed Individually at 100,000 ppm with OPC.

	Leachate Conc., mg/L			
Waste	Pb(II)[a]	As(III)	Cr(III)	Cr(VI)
As(III) + Cr(III)	---	15 (2.1)	0.4 (0.4)	---
Pb(II) + Cr(III)	5.2 (48)	---	0.3 (0.4)	---
As(III) + Cr(VI)	---	10 (2.1)	---	1100 (1400)
Pb(II) + Cr(VI)	0.6 (48)		---	2400 (1400)

[a] The salts used are $Pb(NO_3)_2$, $NaAsO_2$, $Cr(NO_3)_3 \cdot 9H_2O$, and $Na_2CrO_4 \cdot 4H_2O$.

LONG-TERM MATRIX CHANGES

There are clear matrix changes that take place over long periods of time, as shown in the NMR data in Figure 1. The substantial increase in percent hydration is a result of the considerable decrease in proportion of orthosilicate (Q^0) remaining after 1 year. Other effects that began to be seen at 28 days are more pronounced at 1 year. Pb is a promoter of silicate polymerization, and this is clearer after 1 year of cure. As and Cr salts are retarders of silicate polymerization, and this effect also continues very clearly through a year of curing time. Cr(III) is a serious inhibitor of both dimerization and polymerization reactions and leads to a matrix still containing mainly Q^0 after 1 year.

The matrix changes that take place over long periods of time have led us to investigate leachability after long cure times. Table 2 presents TCLP leaching data for solidified samples prepared from a single metal salt and various binding agents, and compares leachability after 28 days of cure (values in parentheses) with that after 1 year. The extra 11 months of cure clearly do not improve performance, except in a few isolated instances for binders that show little promise overall, such as Pb containment by Lumnite. Actually, quite the opposite occurs, most notably for Pb, and to a lesser extent for As(V) and Cr(VI). The effect for Pb-containing samples is quite consistent. Those samples which showed 28-day leachability in the range below 10 mg/L all show unsatisfactory performance in 1-year leachability; the single exception being OPC-fly ash. Immobilization of Cr(VI), already poor at 28 days, is consistently worse at 1 year. Major changes have occurred in the matrices between 28 days and 1 year, as described above. Clearly the changes affect leachability. It should be noted that these results have major implications for the prediction of long-term leachability. Ordinarily batch leaching tests over short periods of time are used to derive an effective diffusivity for the waste/binder combination. That effective diffusivity is then used to extrapolate leaching potential over long periods of time. *It does not take into account long-term changes in the matrix that affect diffusivity and leachability.*

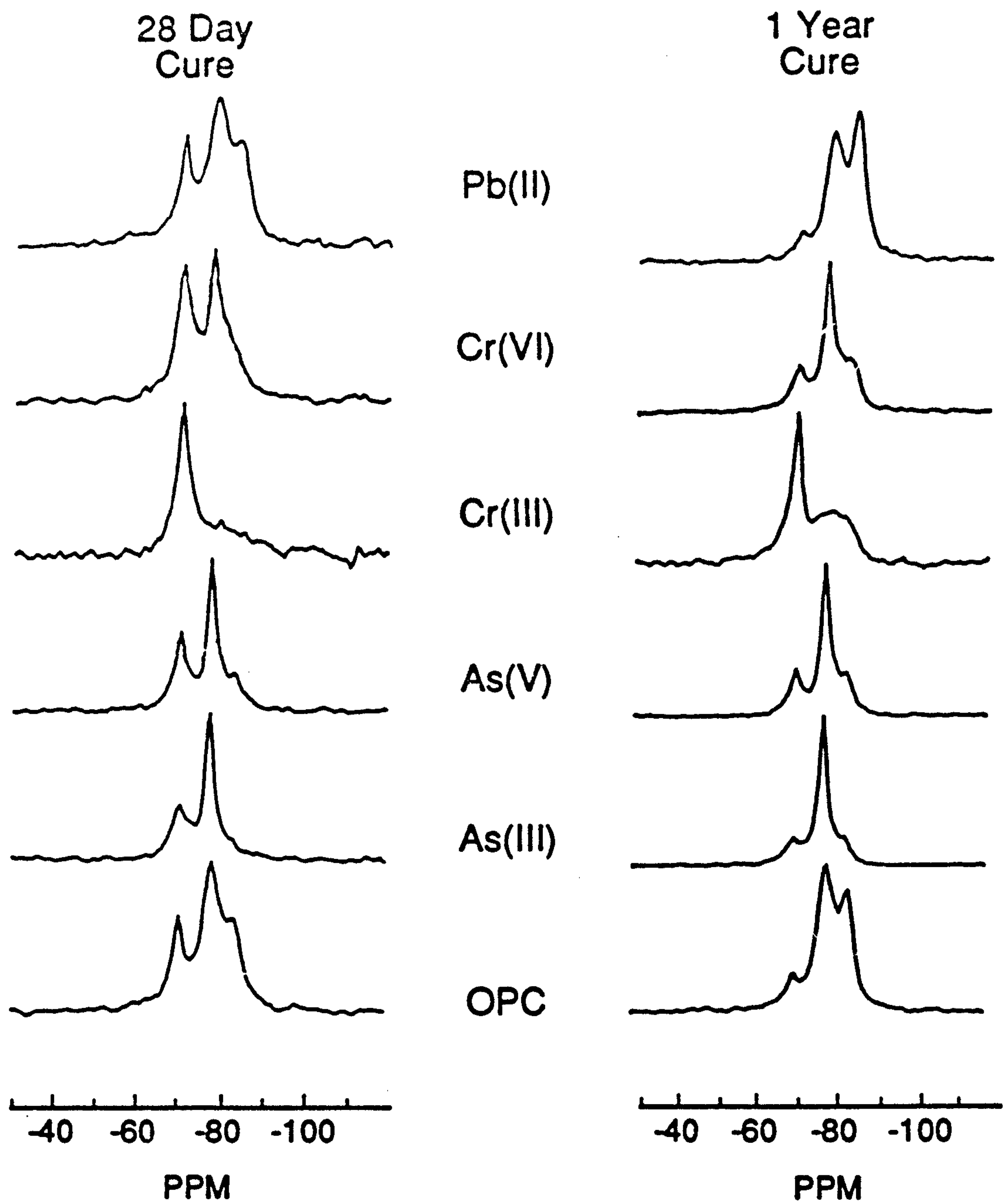

Figure 1. - ^{29}Si solid-state MAS NMR spectra obtained at 4.7 T and a spin rate of 5 KHz for Type I Portland cement samples containing 10 percent by weight of As (III), As (V), Cr (III), Cr (VI) and Pb (II) and cured for 28 days (left) and 1 year (right).

Table 2. - TCLP Leachate Concentrations from Solidified Wastes after 1 Year and after (28 Days) of Cure; Each Waste and Binder Mixed Separately at 100,000 ppm of Metal Ion in Binder

Binder[a]	Leachate Conc., mg/L				
	Pb(II)[b]	AS(III)	As(V)	Cr(III)	Cr(VI)
OPC	35 (48)	1.7 (2.1)	1.4 (1.7)	0.4 (0.4)	2500 (1400)
OPC, no gypsum	43 (2.2)	3.1 (2.7)			3200 (2400)
1:1 OPC:Type F FA	0.9 (0.3)	430 (540)	94 (7.9)	0.7 (0.2)	2900 (2600)
20:1 OPC:SiO_2	25 (24)	2.3 (2.4)			2500 (1600)
20:1 OPC:Na_2SiO_3	28 (1.8)	2.0 (3.5)	0.4 (0.1)	0.5 (0.2)	2500 (1800)
10:1 OPC:Bentonite	46 (1.4)	2.4 (30)			2400 (1700)
10:1 OPC:Organoclay	40 (16)	1.1 (30)			2800 (2400)
20:1 Type IA:SiO_2	41 (5.2)	2.5 (3.1)	0.4 (0.1)	0.2 (0.2)	2800 (1900)
20:1 Type IA:Na_2SiO_3	35 (44)	1.6 (1.2)			2700 (3300)
White	54 (1.0)	0.9 (1.7)	10 (3.6)	0.4 (0.2)	2900 (2000)
20:1 White:Na_2SiO_3	38 (6.0)	1.0 (2.5)	15 (8.0)	0.8 (0.2)	2800 (2300)
Lumnite	4 (400)	240 (140)			3700 (2300)
Refcon	190 (680)	190 (150)			3600 (1900)
Pyrament	1.7	47	4.4	1.4	2800

[a] OPC = Type I Portland, FA = fly ash, SiO_2 = silica fume, Na_2SiO_3 = Type N soluble sodium silicate, White = Portland with low iron content. Lumnite and Refcon are specialty cements that are for refractory applications and high in alumina content.
[b] The salts used are $Pb(NO_3)_2$, $NaAsO_2$, $Na_2HAsO_4 \cdot 7H_2O$, $Cr(NO_3)_3 \cdot 9H_2O$, and $Na_2CrO_4 \cdot 4H_2O$.

The most dramatic long-term matrix changes that we have observed have been with OPC/fly ash mixtures. While following the hydration reactions of As(III) samples solidified in cement/fly ash, we noted by ^{27}Al NMR an initial normal, albeit slow, conversion of the aluminate phase from tetrahedral coordination at Al to octahedral coordination. After 28 days of cure, however, that conversion had begun to reverse itself, as shown in Fig. 2, and after 14 months almost all the octahedral Al had reverted to tetrahedral Al. Furthermore, there is some indication that the silicate phase is undergoing depolymerization. The spectral changes leading to the latter conclusion are still within the range of experimental error (±4 percent), but they clearly indicate the necessity to monitor these changes over even longer periods of time. After seeing the effects noted above in the As(III)/OPC/FA mixture, we ran some OPC/FA control samples containing no waste. We see similar effects in both the aluminum and silicon spectra, although the extent of reversion is lower at 1 year in the absence of As. Table 3 shows that both As(III) and As(V) appear to catalyze the aluminate phase changes. Once again, such long-term effects that alter the matrix have serious consequences for the application of leach modelling to long-term predictions of release rates for contaminants. In the particular cases of the mixtures shown in Table 3, only the As(V) samples show clearly enhanced leachability after

1 year of cure compared to 28 days (Table 2); however, we will continue to monitor the leaching behavior as well as the NMR spectra over even longer periods of time.

Table 3. - Proportions of Tetrahedral Al and Percent Silicate Hydration in Portland Cement (OPC) and OPC/Class F Fly Ash Samples, Including Ones Containing 10 Wt-Percent Arsenic

Sample	Percent Tetrahedral Al		Percent Silicate Hydration	
	28 day	1 year	28 day	1 year
OPC	3	3	72	85
OPC/FA	10	26	95	87
OPC/As(III)	7	7	64	82
OPC/FA/As(III)	62	86	75	72
OPC/As(V)	8	8	62	76
OPC/FA/As(V)	40	62	76	85

[a] The binders are Type I Portland and Class F Fly Ash, and mixtures of the two are always 1:1 by weight. The salts used are NaAsO2 and $Na_2HAsO_4 7H_2O$, and the mixtures contain 10 wt-percent As.

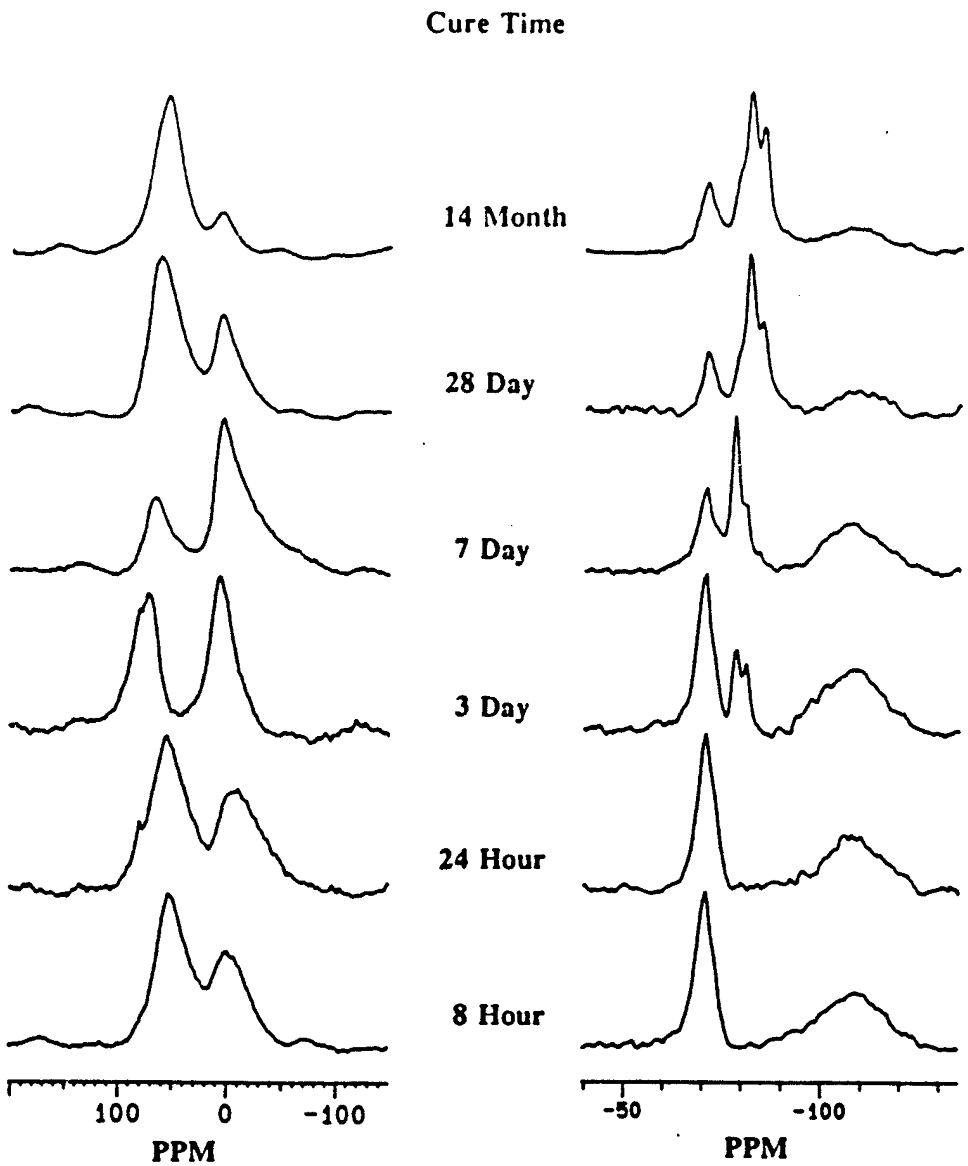

Figure 2. - ^{27}Al (left) and ^{29}Si (right) NMR spectra as a function of time for solidified samples containing 10 wt-percent $NaAsO_2$ in 1:1 Type I Portland and Class F fly ash.

ORGANOCLAY ADDITIVES

Organophilic clays have been widely touted as effective additives to S/S processes for wastes containing organics. Nevertheless, very little information is available in the literature that allows any judgement about the effectiveness of these additives (Gibbons and Soundararajan 1988, 1989; Montgomery et al., 1988; Sheriff et al., 1987). The usual composition of the organoclays is bentonite in which the alkali metal cations have been exchanged for alkylammonium cations, but the nature of the alkylammonium cation and the degree of coverage of the bentonite surface are major variables that can affect the process of adsorption of organics. If all exchange sites in the clay undergo replacement with alkylammonium, then the capacity for adsorption of organics should be highest; if some sites are not exchanged, the organoclay should have sites available for exchange with heavy metals as well as organics. Sheriff et al. (1987) have prepared organoclays under well-defined conditions, used them for adsorption of phenol and chloro-substituted phenols, then stabilized the organoclay with cement and tested the products with respect to batch leaching and compressive strength development. Gibbons and Soundararajan (1988, 1989) have examined a variety of organics adsorbed on proprietary and synthesized organoclays and examined infrared and thermal properties of the organoclays containing organic wastes. They see changes in infrared (IR) frequencies for the simulated wastes when they are adsorbed on the organoclays, and differential scanning calorimetry (DSC) shows that the energy required to drive organics off of the organoclays is greater than the heat of vaporization of the waste without binder. The IR and DSC methods have been applied only to organic-organoclay mixtures or to organic-organoclay-cementitious binder mixtures, the latter not having been allowed to undergo cementitious setting reactions (i.e. hydration) prior to analysis. Furthermore, rather high ratios of organic-to-organoclay, or organic-to-organoclay+binder are reported in the published work. The work was carried out in this way in order to avoid interferences from water or from high concentrations of compounds other than the one(s) being investigated.

We have initiated a systematic investigation of organoclay usage in S/S by starting with specific organics and determining adsorption isotherms for their adsorption to organoclays. The organic-organoclay product is then solidified and leaching tests applied in order to obtain a quantitative correlation between adsorption capacity of a particular organoclay for a particular organic and leaching performance after cementitious solidification (Faschan, 1992).

Correlations reported by Hasset et al. (1981) and Chiou et al. (1979) indicate that adsorption of organics by clays with high organic contents is a partitioning process. Chiou et al. (1979) showed adsorption isotherms of seven organic compounds with no curvature even at concentrations approaching saturation, which is consistent with the idea of partitioning instead of physical adsorption. Chiou et al. (1983) also discovered that the extent of organic partitioning was primarily controlled by the solubility of the organic compound in water.

Since organoclays are purposely altered to have high organic contents, it is expected that their adsorption of organics should also be controlled by a partitioning process. Indeed, the attachment of large organic cations like hexadecyltrimethylammonium to smectite leads to the formation of a medium similar to a bulk phase organic solvent like hexane (Boyd et al., 1988; Lee et al., 1989). The sorption of organic solutes from water by organoclays shows characteristics of solute partitioning, including linear isotherms, inverse dependence of the sorption coefficient on the water solubility of the solute, and correlation between the organic matter-normalized sorption coefficients (K_{om}) and the octanol-water partition coefficients (Jaynes and Boyd, 1990).

In our work, the organics chosen represent a range of polarity, 1,2-dichlorobenzene (DCB), nitrobenzene (NB) and phenol (PhOH), and 5 different commercially available organoclays were used: Claytone APA

(APA), Claytone GF (GF), Suspendtone (Susp), Bentec PC-1 (PC-1) and Bentec PT-1 (PT-1). All of the organoclay products had complete exchange of clay alkali metals by tetraalkylammonium, and all had at least one alkyl chain of 18 carbon atoms or longer. The organoclays were exchanged bentonites except for Susp, which was an exchanged attapulgite.

Adsorption isotherms were determined for each organic on each organoclay using distilled water and also a simulated TCLP leachate solution with a pH of 12. The data for each organic and clay combination were plotted using linear, Langmuir, Freundlich, and BET isotherm models, with the linear model always forced through the origin. DCB and NB showed excellent correlation with both linear and Langmuir isotherms, but given prior work which indicates that nonionic materials are transferred from water to soil organic matter by a partitioning effect, we believe the linear representation to be more appropriate. On the other hand, phenol was poorly modeled by a linear correlation, and much better by the Langmuir model, which might have been expected due to the higher water solubility of phenol. The high pH solution used to simulate TCLP leachate from a cement-solidified material had a major effect of decreasing phenol adsorption, but did not have a significant effect on DCB or NB adsorption.

Cement-solidified samples were then prepared for each organic and each clay using cement:clay ratios ranging from 3:2 to 13:2. The NB and PhOH samples were mixed (by hand) and stored in plastic specimen cups sealed with a lid and parafilm; the DCB samples used glass containers. The amount of water added to each sample depended on the amount of cement and clay in the sample and the liquid limit of the clay; according to the following equation:

$$\text{water (mL)} = 0.4(\text{cement in g}) + (1.2\ LL - wc)(\text{clay in g})$$

where LL = liquid limit of the clay
wc = natural water content of the clay.

The amount of organic added to each sample was the maximum observed to be adsorbed by each clay in the adsorption study.

After 28 days, the samples were subjected to the TCLP, and leachate concentrations of the organics were determined by gas chromatographic analysis. In addition, a mass balance study was carried out by including ^{14}C-labeled DCB and NB in several samples and measuring mass loss due to volatilization during mixing, due to volatilization during grinding, and mass remaining in the sample. The mixing was carried out in a glove bag whose outlet was fitted with adsorbent traps (silica gel for NB, activated carbon for DCB). The samples were cured for 7 days, then ground for the TCLP in the same glovebag arrangement. Adsorbent trap extractions were analyzed by both gc and liquid scintillation methods, and the ground samples were prepared for liquid scintillation by mixing 0.08 g of cellulose powder as a combustion aid for each 0.3 g of sample, followed by combustion in a Packard Oxidizer Model 306 and adsorption of CO_2 using an amine-base "Carbsorb" carbon trap.

Total recovery in the mass balance ranged from 89 to 101 percent for DCB and NB adsorbed on APA clay. Recovery was lower, down to 63 percent for the Susp samples, and as low as 24 percent for cement without organoclay. The low percent recovery with the cement samples was apparently due to the lack of homogeneity of the samples. After mixing, an oily organic layer was present on top of the cement. This had volatilized after 7 days of cure. The mass balance study did not attempt to account for organic lost during curing in the sealed specimen cup. Due to the lower percent recoveries and a somewhat poorer isotherm correlation, the attapulgite-based organoclay (Susp) was not included in the correlations to be described later.

A model was developed to predict TCLP leachate concentrations to be expected from samples prepared by solidification of nonpolar organics (based on DCB and NB in this study) with various proportions of OPC. The model utilizes the well-known relationship between octanol-water partition coefficient, K_{ow}, and the partitioning of organics on clay materials. The slope of the linear isotherm is predicted from:

$$K_p = 14.34(\%OM)(K_{ow}^{0.82}),$$

where %OM is the fraction organic matter of the clay.

Substituting the estimated K_p into the linear model equation gives the following expression for the expected TCLP leachate concentration:

$$C = (\text{organic})/[2.0 + 0.014(\%OM)(K_{ow}^{0.82})(\text{clay})],$$

where	C	=	concentration of organic in the leachate (mg/L) from a 100 g sample,
	(organic)	=	amount of organic in the sample (mg), and
	(clay)	=	the amount of clay in the sample (g).

Better predictions with this equation could be obtained by taking into account the fact that leachability decreased somewhat with increasing cement/clay ratios. That was taken into account with a correction factor:

$$C_{corr} = C(0.81)(\text{cement/clay}),$$

where cement/clay is the weight ratio of cement to clay in the mixture.

The model accurately accounted for the results for almost all of the samples to within ±10 percent for both NB and DCB and the bentonite-based organoclays. Curing time, water-to-solids ratios and order of mixing (i.e. premixing of organic with organoclay vs. mixing of organic, organoclay and cement simultaneously) had little effect on the results.

It remains to be seen how broadly applicable the model is. The only clear limitation seen thus far is that very water-soluble organics do not show a good correlation. However, the potential exists for application of the model to prediction of leachability on an a priori basis for a very wide range of organics solidified with organoclay/cement mixtures.

SUMMARY AND CONCLUSIONS

Several examples have been presented in which basic research into solidification/stabilization mechanisms has significant implications for practice.

- A heavy metal sludge prepared by lime precipitation of the metals in a synthetic metal plating solution has been treated with Type I portland cement, and the resulting stabilized system is basically a physical mixture of the sludge and hydrated cement. It is thus possible to distinguish this case of physical encapsulation on a microscopic scale from chemical fixation. Such a S/S system can be expected to show leachability determined by the solubility properties of the sludge and the water transport properties of the matrix. The latter can in principle be controlled by the cement formulation, which alters density and porosity.

- Cr(III) and As(III) and As (V) salts in aqueous solution, treated with OPC or OPC/fly ash, present clear cases of chemical fixation. This is evident in major changes in the fixation matrix. By affecting the matrix, these salts also influence the leachability of other components in the S/S mix. Hence, S/S when these salts are present needs to be investigated in greater detail than cases such as Cd or Pb, which merely form surface precipitates.

- Cr(III) has major long-term effects on OPC and OPC/fly ash matrices, retarding silicate hydration and polymerization and apparently becoming incorporated into the silicate structure. At long cure times, As salts catalyze matrix transformations in OPC/fly ash that only become apparent much after 28 days of cure. These results mean that there are cases where leachability and other forms of testing of S/S mixtures should be carried out at much longer cure times than current practice requires.

- The interaction of nonpolar organics with organoclay additives follows a partitioning mechanism. As a consequence, basic properties of organic materials, such as octanol-water partition coefficients, can be used to give a good quantitative estimate of the concentration of the organic in TCLP leachates when S/S is carried out with cement and organoclays.

These examples of the conclusions of basic research into S/S mechanisms show the importance of research for S/S practice and give promise of many more conclusions that can result in conversion of the technology from an art to a science.

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ENVIRONMENTAL AND GEOTECHNICAL TESTING IN SUPPORT OF WASTE STABILIZATION

Terry M. McKee[1]

Abstract: The various types of environmental and geotechnical testing required to manage stabilized waste materials are discussed in this paper. Testing is dependent upon the type and source of the waste materials, the stabilization processes and the disposal or final end uses.

INTRODUCTION

The type of environmental and geotechnical testing associated with stabilized waste materials may be dependent upon many factors, but most testing is in an effort to characterize the material, to determine its impact on the environment, and to predict how long its environmental impact can be measured.

ENVIRONMENTAL TESTS

One needs to determine if the untreated waste, the stabilization reagent, and/or the stabilized material are hazardous wastes as defined by local/State rules, and by the solid waste regulations promulgated by the U.S. Environmental Protection Agency (EPA) as a result of the Resource Conservation and Recovery Act (RCRA) of 1976. Title 40, Part 261 of the *Code of Federal Regulations* (CFR) define the various conditions under which a waste stream is determined to be a RCRA hazardous waste. Briefly, the regulations define a hazardous waste as a waste material that (1) exhibits one or more of four specified physical or chemical properties, (2) is a listed waste, (3) a combination of both, or (4) is a mixed or "derived from" waste. Certain waste streams are defined in 40 CFR 261 that are exempt from the solid waste regulations, or that are not regulated as hazardous solid wastes.

DISCUSSION

The four characteristic of a hazardous waste are ignitibility, corrosivity, reactivity, and toxicity, commonly termed "D wastes." In addition to characteristically hazardous waste, specified types of waste materials are listed hazardous wastes. These are wastes from nonspecific sources that are defined by constituents or components - the "F wastes," "K wastes," or those resulting from specific process, and "P or U wastes" defined as commercial chemical products, manufacturing chemical intermediates or off-spec commercial chemical products or manufacturing chemical intermediates.

Should a waste come in contact with a listed waste, the entire material is considered to be a hazardous waste, unless the listed waste is listed only for one or more of the characteristics, and the waste no longer exhibits the characteristic(s). This is commonly termed the "mixture rule." There are but a few wastes listed solely for one or more of the hazardous characteristics.

Another important rule to also consider in the stabilization of waste materials is called the "derived from" rule. This rule states that any waste material resulting from the treatment, storage or disposal of a listed waste retains that listing, and is itself a hazardous waste.

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Recently, the EPA has published proposed changes to the "mixture" and "derived from" rules. Called the "Hazardous Waste Identification Rule" (HWIR), the many options of the proposal attempt to replace the rules with either enhancements of the characteristic of toxicity, the implementation of concentration based exemption criteria, or combinations of both. Comments on the proposed rule were due on July 24, 1992, and the agency received in excess of five hundred, an all time high. As a result of court action, the "mixture" and "derived from" rules essentially evaporate in April 1993. Due to the many options presented by the Agency in the proposed HWIR, we expect to see a more focused proposal in a few months. Depending, of course, on the final version, the new rule could have a substantial impact on the way in which hazardous wastes are defined, and the volume of waste managed as RCRA hazardous.

Regulations require that the generator of a waste describe the material to the entity managing the waste for disposal. Information concerning the source of the waste, its composition, and rather or not it is characteristically hazardous or a listed hazardous waste should be obtained from the generator of the waste. The generator should describe the waste to the satisfaction of the treatment and disposal facility, and certify the information to be true and correct to the best of his/her knowledge. Most commercial waste treatment and disposal companies have preprinted forms that can be used by the generator for this purpose.

Environmental testing that may be performed on the waste material includes the four RCRA characteristics of a hazardous waste. For example, a solid waste that exhibits a flash point at less than 140 °F or catches fire at ambient conditions is an ignitable waste (D001). A waste that exhibits a pH of less than 2.0 or greater than 12.5, or that corrodes mild steel at greater than 0.25 inch per year, is considered a corrosive waste (D002). The defined characteristic of corrosivity are applicable to only liquid wastes. One should remember that the term "solid waste" in these rules refer to wastes regulated as solid wastes, not nonliquid wastes.

Waste materials that react in water or air, that release toxic concentrations of hydrogen cyanide or hydrogen sulfide gases within an environment of pH less that 2.5, or are unstable at ambient conditions are considered to be reactive wastes, D003. In order to determine if a waste material exhibits the Characteristic of Toxicity, it is first subjected to a mild acidic leaching procedure, or if a liquid, simply filtered. The leachate or filtrate is then analyzed for 8 metals, 8 herbicides and pesticides, 11 volatile organic compounds, and 13 semivolatile organic compounds. Should any one or more of these constituents be found at levels above the maximum concentration levels (MCL) set in the regulations, the waste material would then be considered a hazardous waste. It would be assigned a hazardous waste code of from D004 to D0043, depending upon the constituent(s) determined to be present in the leachate or filtrate above the MCL.

The methods for these tests are described by the Agency in *Test Methods for Evaluating Solid Waste, Physical/Chemical Methods*, November 1986, Third Edition, USEPA, SW-846 and additions thereto. Other tests methods to aid in the characterization of waste materials are available from various sources. For example, the American Society for Testing and Materials (ASTM) Committee D-34 on Waste Management has published several methods that are useful for determining physical or chemical properties of waste materials. Some of these methods not only assist in characterizing the waste, but also provide additional information often important when considering management of the waste material for treatment, disposal or for another end use such as recycling. For example, ASTM Committee D 34 has published tests dealing with single and sequential batch extraction of wastes, measurements of the sorption of volatile organic compounds by soils and sediments, screening wastes for ignition potential, sulfides, cyanides, pH, oxidizers, etc., and determining the stability and miscibility of a waste material.

GEOTECHNICAL TESTING

Waste stabilization often consists of mixing the waste material with a stabilization reagent in an effort to prevent the mobilization of the toxic constituents of the waste by chemical or physical means. The bonding of toxic heavy metals by an alkaline media, thus rendering the metals insoluble in aqueous media, is probably the most widely used type of chemical stabilization. Reagents commonly used for this purpose are lime, lime kiln dust, cement, cement kiln dust, and fly ash, most often a by-product from the burning of fossil fuels.

Other types of stabilization commonly used rely on the reagent's sorptive properties, or how well it will retain the toxic or hazardous constitutes of concern. Here, the spongelike characteristics and polar bonding properties of the reagent are the concerns. Reagents commonly used for sorbent stabilization are fly ash, clays, diatomaceous earth, and activated carbon.

Any by-product that has the appropriate physical and/or chemical properties allowing it to serve as a waste stabilization reagent should be investigated. Such products may be agriculture by-products such as rice hulls, and industrial by-products such as bag house dust or steel slag aggregate.

Tests commonly associated with determining if a material will act as a suitable waste stabilization reagent vary from the simple to the complex, depending upon the disposal technique of the material or its final end use. Laboratory testing usually begins with mixing the waste with the potential reagent(s) at various ratios of waste to reagent. One can almost immediately screen out proposed reagents that do not have the necessary properties simply by observing the initial mixing. Poor candidates will require excessive volumes, will be difficult to mix, and will leave visible remnants of the waste's liquid phase. Several ratios of each mix are then covered and left to set for a minimum of 24 hours to allow for a measurable cure time.

Each mix can then be examined for additional physical or chemical characteristics. Such tests at this time may be as simple as a unconfined compression test utilizing a pocket penetrometer, to as complex as a one-dimensional consolidation test, or a chemical leach test to determine the mobility of the toxic constituents of concern. Other geotechnical tests for stabilized wastes may involve particle-size analyses, liquid limits, plastic limit and plasticity index, water content and specific gravity.

A PROPOSED PROTOCOL FOR EVALUATION OF SOLIDIFIED WASTES*

By Julia A. Stegemann[1] and Pierre L. Côté[2]

Abstract: Solidification technologies are potentially useful for improving the chemical and physical properties of hazardous wastes to the extent that they are suitable for less expensive disposal or even utilization. Unfortunately, in most jurisdictions worldwide, there is no mechanism for reclassifying a treated, previously hazardous waste, as nonhazardous. In response to the need for such a mechanism, the Wastewater Technology Centre has proposed a protocol of test methods for cement-based solidified wastes.

The suggested test methods examine partitioning of contaminants as a result of their chemical speciation and their potential for slow release of contaminants. The methods are based on the mobility of the contaminants in the solidified waste matrix and the durability of the matrix. Most of the suggested tests are standards from the fields of hazardous and radioactive wastes, some of which have been evaluated in a cooperative study with vendors of solidification processes initiated by Environment Canada.

Based on the performance of a solidified product in the tests, it is considered for four utilization and disposal scenarios: unrestricted utilization, controlled utilization, segregated landfill and sanitary landfill. The protocol represents a first attempt to develop a management tool for solidified wastes that accounts for their physical and leaching characteristics in the context of different disposal scenarios.

INTRODUCTION

Solidification of hazardous wastes by mixing with hydraulic binders, such as portland cement or pozzolanic power plant fly ash, has great potential for reducing contaminant leachability by both chemical and physical means.

Unfortunately, although a great variety of test methods have been developed all over the world, there is no standard procedure for evaluating the efficacy of a solidification process and the risk to the environment which might ensue from disposal or utilization of the solidified product.

In response to this need, Environment Canada's Wastewater Technology Centre is proposing a protocol for evaluation of the environmental acceptability of solidified wastes, which takes into account the special properties of solidified wastes and the requirements of various disposal scenarios (Wastewater Technology Centre, 1991).

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OBJECTIVES

The main objective of the proposed protocol is to suggest a rational approach to regulating disposal and utilization of solidified wastes.

Secondary objectives of the document are (1) to identify the key properties of cement-based solidified wastes, which are related to the main leaching mechanisms, (2) to suggest a number of test methods which provide measurements of the key properties, and (3) to suggest performance criteria, which are compared to the test results in order to determine whether a solidified product can be safely disposed of or utilized.

APPROACH

Solidification with hydraulic binders usually results in a strong, durable matrix of low permeability. Most heavy metals are precipitated as hydroxides of low solubility in the resulting alkaline environment. Thus, solidification can immobilize contaminants in two ways: (1) by chemically binding them in an insoluble form, and (2) by physically trapping them in a rigid, impermeable and durable matrix.

The proposed protocol gives credit to both ways of reducing leaching, and it considers the characteristics of the final disposal (or utilization) scenario in assessing the short and long-term leachability of the solidified product.

The evaluation procedure requires initial testing (Level 0) to determine basic information about the solidified product and two levels of critical testing. Level 1 testing is directed at determining the efficacy of chemical containment, and Level 2 testing investigates physical containment.

Based on the results from testing, which are compared with set performance criteria, the solidified product is considered for one of four utilization and disposal scenarios:

> **Unrestricted Utilization.** - The solidified waste has a negligible leaching potential, and is environmentally acceptable as a construction material. Once a solidified product has been approved it becomes exempt from waste management regulations.
>
> **Controlled Utilization.** - The leaching potential of the solidified waste is assessed and determined to be acceptable for a specific scenario (e.g., road-base material, backfill, etc.).
>
> **Segregated Landfill.** - The solidified waste is not acceptable for utilization, based on its leachability or for practical reasons. The solidified waste is isolated in a monofill, which does not necessarily have engineered containment systems.
>
> **Sanitary Landfill.** - The characteristics of the waste do not permit utilization or disposal in a monofill, but disposal with municipal garbage in the presence of acidic leachate is permitted.

Since chemical and physical containment mechanisms are independent of one another, a solidified waste may fulfill either the Level 1 or the Level 2 performance criteria in order to qualify for a particular scenario. However, although a solidified waste which fulfills Level 1 performance criteria does not require Level 2 testing, Level 1 testing must always be performed.

In this way, the most economical disposal solution, which is also environmentally acceptable, is chosen. Solidified wastes that do not qualify for one of these scenarios would need to be disposed of in a secure landfill or treated by an alternative method.

Figure 1 shows a flow chart of the evaluation and decisionmaking procedure.

MEASUREMENT OF KEY PROPERTIES

The test methods suggested for investigating the properties of concern have been assembled from a number of sources, including standard methods of both the American Society for Testing and Materials and the American Nuclear Society. Many of the methods have been validated for use with solidified wastes in a cooperative study with industry (Stegemann and Côté, 1990a). The full list of test methods and key properties which they estimate are shown in Table 1.

Level 0 Testing

Before beginning critical evaluation of the solidified product, general information is required concerning the solidification process and the solidified matrix composition in order to ensure applicability of the protocol and aid interpretation of the data obtained from further testing. The most important characteristics of the solidification process are additive dosages, details of processing, and the resulting mass and volume increase factors (which have an important effect on economics). To examine the matrix composition, bulk density, water content, specific gravity (Stegemann and Côté, 1990b), contaminant concentrations, and organic content must be measured. Bulk density, water content, and specific gravity are used to calculate matrix porosity and degree of saturation, which may affect permeability and leachability.

Level 1 Testing

Contaminants in a solidified waste may be present as a variety of chemical species of varying solubility, or may be adsorbed by the solid matrix. Since a contaminant must be in solution for it to be released into the environment, the partitioning of contaminants between the liquid and solid phases and the reversibility of this partitioning, are critical factors in the success of solidification.

The initial concentration of contaminants dissolved in the pore water of the solidified product is approximated using a low liquid-to-solid ratio 7-day batch extraction (Stegemann and Côté, 1990b). This test provides an indication of short-term leachability. To investigate the leachability over the long-term, the total amount of contaminants available for leaching is measured using a pH 5 batch extraction (a modified version of the present Regulation 309 leaching test). The quantity measured in this test will not be available for leaching initially, but could theoretically become available, as a result of changes occurring over long periods of time. Contaminants which are not dissolved in this test are not likely to be dissolved by any environmental leachant, and these contaminants may be considered to be permanently immobilized. Because of a cement-based solidified product's alkaline character, its chemical resistance to acid addition is a particularly important parameter. The acid neutralization capacity may be determined using a series of batch extractions with varying amounts of nitric acid (Stegemann and Côté, 1990b).

To ensure that physical structure is not a variable, all Level 1 tests are conducted on finely ground material. A solidified product that meets the performance criteria for Level 1 testing does not have to be protected from physical degradation.

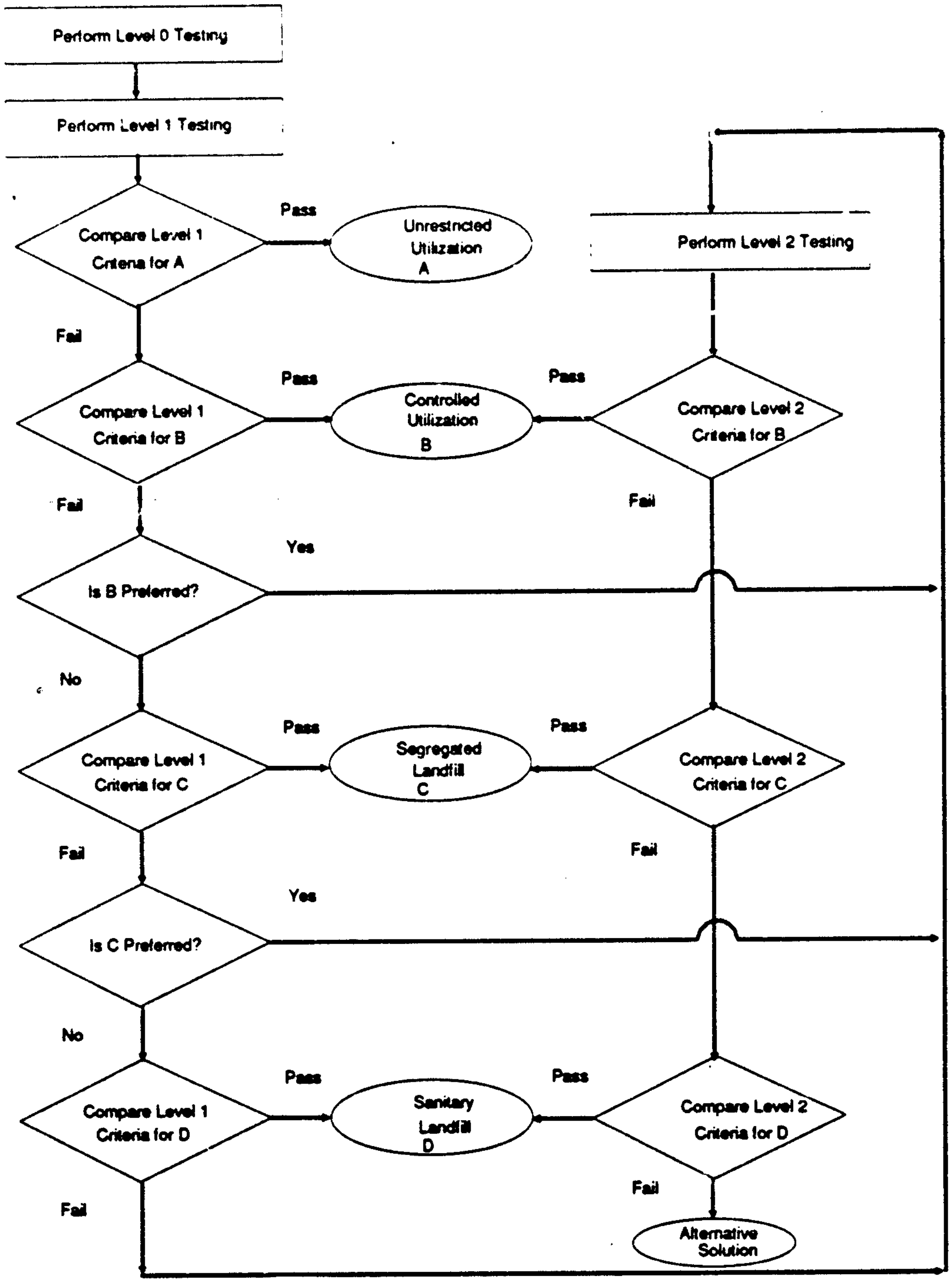

Figure 1. - Decision flow chart for the proposed evaluation protocol.

Level 2 Testing

Most cement-based solidified wastes are hard, monolithic materials with low permeability. Contaminants may be physically entrapped in a matrix of this kind to varying degrees. A solidified matrix which is completely impermeable provides no contact between the contaminants and the surrounding groundwater, thus, prevents leaching entirely. A matrix which has a very low permeability ($<10^{-9}$ m/s) limits significant flow of groundwater through the waste, and causes slow diffusion to be the main leaching mechanism; however, a matrix with a higher permeability (i.e., $>10^{-9}$ m/s) results in more rapid washout of contaminants by flow of groundwater through the body of the waste (advection). Therefore, hydraulic conductivity is seen to be one of the main physical factors controlling leaching. This property may be measured using a falling-head method on a specimen confined in a triaxial cell (Stegemann and Côté, 1990b).

If the hydraulic conductivity of a solidified waste is sufficiently low, the mobility of contaminants due to diffusion is of concern. The apparent diffusivities of the contaminants can be calculated using an immersion leaching test with a monolithic specimen and periodic leachant renewal. The contaminant concentration measured in each leaching interval is used to calculate a leachability index (American Nuclear Society, 1986), which may be interpreted as the negative logarithm of the apparent diffusion coefficient.

Although the physical properties of a solidified product may be initially satisfactory, the durability of the matrix must be examined to ensure long-term protection of the environment. While resistance to weathering can be measured by subjecting a specimen of waste to successive cycles of freezing and thawing, or wetting and drying (Stegemann and Côté, 1990b), unconfined compressive strength (before and after immersion) provides one measure of physical durability. Resistance of the solidified product to biological degradation may also be of importance for wastes with a high organic content. A number of laboratory methods which examine the ability of a medium to support microbial growth are available (e.g., ASTM G 22-76, Practice for Determining Resistance of Plastic to Bacteria).

PERFORMANCE CRITERIA

To be accepted for utilization or disposal, the results from testing must meet the appropriate performance criteria which are different for each of the scenarios described earlier.

The proposed performance criteria are based on existing regulatory criteria and a database of properties of state-of-the-art solidified wastes (Stegemann and Coté, 1990a). The criteria proposed for each of the four utilization and disposal scenarios are shown in Tables 2 and 3, and these are explained in the following paragraphs.

Level 0

The only performance criterion set for Level 0 ensures applicability of the test methods, rather than protection of the environment. Since the presence of high concentrations of organic compounds tends to interfere with cement-based solidification processes and may change the chemistry of the system, an organic content greater than 10 percent indicates that the solidified product is unsuitable for utilization and should be tested for resistance to biological degradation for the disposal scenarios.

Level 1

Performance criteria for Level 1 are set based on existing regulatory criteria for drinking water and classification of wastes.

The pore-water contaminant concentrations from the low liquid-to-solid ratio extraction may also be interpreted as initial undiluted leachate concentrations. Consequently, to account for dilution and attenuation between the point of leachate generation and the point of water use, performance criteria are set as multiples of drinking water quality criteria.

The total amounts available for leaching measured in the pH 5 extraction are compared with performance criteria which were calculated based on regulatory limits for concentrations in the USEPA TCLP (Federal Register, 1986). These leachate concentrations were converted to amounts per mass of waste. The selected criteria represent fractions of these amounts.

A solidified product with an acid neutralization capacity of 1 eq/kg, which is suggested as the performance criterion for the controlled utilization and segregated landfill scenarios, is capable of neutralizing the addition of 10,000 L/kg of a pH 4 mineral acid solution without reaching a pH below 9 where metals become more soluble. The acid neutralization capacity limit for the sanitary landfill scenario has been set at 3 eq/kg because a solidified product in a sanitary landfill would be exposed to a greater amount of acid from the decay of garbage.

Level 2

Since contaminants must be chemically immobilized for a solidified product to qualify for un-restricted utilization, no Level 2 performance criteria have been proposed for this scenario.

For the remaining three scenarios, performance criteria for hydraulic conductivity have been selected to prevent flow of groundwater through the body of solidified waste, thereby forcing transport of contaminants to take place by slow diffusion.

The performance criteria for the leachability index were based on modelling of diffusion leaching from a large monolith (surface-area to volume ratio < 6). For a leachability index greater than 9, as is proposed for the controlled utilization scenario, the cumulative fraction leached from a large monolith would be less than 10 percent in 100 years.

The post-immersion performance limit set for unconfined compressive strength for the controlled utilization and segregated landfill scenarios is in accordance with a 50 lb/in^2 (350 kPa) guideline suggested by the USEPA (1986). Since the solidified product is more likely to be subjected to physical stresses (from compaction and handling) in a landfill designed for municipal wastes, this value has been multiplied by a factor of 10 for the sanitary landfill scenario. As land-disposed solidified wastes are likely to become saturated with water, the unconfined compressive strength after immersion must be at least 80 percent of that of the dry solidified product.

Previous work (Stegemann and Côté, 1990a) has shown that solidified products which are subjected to freeze/thaw or wet/dry testing cycles tend to fall into two categories: (1) those which disintegrate completely after only a few cycles, and (2) those which survive 12 cycles with minimal weight loss. For this reason, the performance criterion for weathering resistance has been suggested as less than 10 percent weight loss in 12 freeze/thaw or wet/dry cycles.

Finally, if a waste has an organic content greater than 10 percent, it can not be utilized, and it must be demonstrated that the matrix can not support biological growth.

CONCLUSIONS

In light of the disposal scenario requirements, the approach recommended in the proposed protocol is a fundamental characterization of physical and chemical solidified waste properties. The introduction of this approach to regulatory testing is the main objective of the proposed protocol. It is recognized that test method development is an evolutionary process, and, indeed, continued development of test methods is encouraged.

The complete set of test methods for the proposed evaluation protocol can be run by an environmental or geotechnical laboratory in approximately a month. It may initially seem expensive to run a battery of tests rather than a single regulatory test, but the detail and quality of information obtained ensure that the final disposal solution is both economically and environmentally acceptable. Full testing need not be repeated for a permitted solidified product (i.e., a specific solidification process and waste stream combination) as long as process variables do not change. Ideally, quick tests will be developed for occasional quality control. Also, if the solidified product passes Level 1, it should be kept in mind that Level 2 testing is not necessary.

The proposed protocol outlines a framework in which field studies can be initiated to gather data on the impact and applicability of the proposed waste management approach. These data are urgently required to help set less arbitrary, more realistic performance criteria.

ACKNOWLEDGEMENTS

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Table 1. - Testing protocol.

Level	Testing Method	Property of Interest
0	No testing required	Process description
0	Bulk density	Mass/volume change
0	Moisture content Bulk density Solids specific gravity	Porosity/Saturation
0	Total organic content	Organic content
0	Digestion/extraction and analysis	Contaminant concentration
1	Low liquid-to-solid ratio extraction	Initial leachate concentration
1	Low pH extraction	Amount of contaminant available for leaching
1	Titration with acid	Acid neutralization capacity
2	Dynamic leaching test	Contaminant mobility in the matrix
2	Falling head permeability	Hydraulic conductivity
2	Unconfined compressive strength	Physical strength (before/after immersion)
2	Freeze/thaw or Wet/dry cycles	Weathering
2	Biological growth	Biodegradability

Table 2. - Performance criteria for utilization and disposal scenarios.

Level	Performance Indicator	Expression	Utilization Scenarios: Unrestricted Utilization	Utilization Scenarios: Controlled Utilization	Disposal Scenarios: Segregated Landfill	Disposal Scenarios: Sanitary Landfill
0	Additive dosage	[w/ww]	N/A[1]	N/A	N/A	N/A
0	Mass/volume change	ratio	N/A	N/A	N/A	N/A
0	Porosity/ Saturation	[v/v] [%]	N/A N/A	N/A N/A	N/A N/A	N/A N/A
0	Organic content	[%], of waste	<1%[3]	<10%[3]	No limit[3]	No limit[3]
0	Contaminant concentration (Use this data to select target contaminants for monitoring in leachate.)	[mg/kg], in waste	No organics[3]	No organics[3]	No limit[3]	No limit[3]
1	Initial leachate concentration[2]	[mg/L], of leachate	Water quality limit (WQL)[4]	5 x WQL[4]	10 x WQL[4]	100 x WQL[4]
1	Amount of contaminant available for leaching[2]	[mg/wet kg], from waste	See Table 4	See Table 4	See Table 4	See Table 4
1	Acid neutralization capacity	[eq/kg], in waste	N/A	>1 (to pH=9)	>1 (to pH=9)	>3 (to pH=9)
2	Contaminant mobility in the matrix[2]	Leachability index (diffusivity)	N/A	>9	>8	>8
2	Hydraulic conductivity	[m/s]	N/A	$<10^{\text{[illegible]}}$	$<10^{\text{[illegible]}}$	$<10^{\text{[illegible]}}$
2	Physical strength Before immersion After immersion	[kPa]	N/A	 440 350	 440 350	 4400 3500
2	Weathering	[% weight loss]	N/A	Survive 12 cycles <10% weight loss	Survive 12 cycles <10% weight loss	Survive 12 cycles <10% weight loss
2	Biodegradability	[pass/fail]	N/A	N/A	Must pass if organic content >10%.	Must pass if organic content >10%.

[1] Not applicable.
[2] Criterion is applicable to every target contaminant.
[3] Local regulations may limit bulk concentrations of organic content and organic and inorganic contaminants in wastes for utilization and land disposal
[4] e.g.: Ontario Drinking Water Objectives (MOE, 1983), but may vary according to the jurisdiction.

Table 3. - Proposed criteria for the amount available for leaching for some metals.

Contaminant	Regulatory limit [1] expressed as		Criteria for Amount Available for Leaching [mg/wet kg waste]			
	Leachate Concentration [mg/L]	Amount Leached [mg/kg wet waste]	Unrestricted Utilization	Controlled Utilization	Segregated Landfill	Sanitary Landfill
Arsenic	5.0	100	1	5	10	100
Barium	100.0	2000	20	100	200	2000
Cadmium	1.0	20	0.2	1	2	20
Chromium	5.0	100	1	5	10	100
Lead	5.0	100	1	5	10	100
Mercury	0.2	4	0.04	0.2	0.4	4

[1] Regulatory limits for the U.S. EPA TCLP (Federal Register, November 7, 1986). The factor of 20 between columns 2 and 3 corresponds to the liquid-to-solid ratio in the leachate test.

TECHNICAL SESSION 7

Waste By-Product Stabilization/Utilization

ASSIMILATION OF WASTES AND BY-PRODUCTS INTO THE HIGHWAY SYSTEM: STATUS REPORT AND REGULATORY INFLUENCES

By Robert J. Collins[1]

Abstract: As the volume of solid wastes generated in this country continues to grow, more and more landfills are closing, resulting in dramatic increases in the cost of waste disposal. Recovery and reuse of usable wastes and by-products is one of many options available for reducing the amount of material being placed in landfills. Because of the enormity of the highway market, efforts to recycle various waste materials are frequently aimed at assimilating such materials into the highway system.

The Transportation Research Board (TRB) has conducted an 18-month study to determine the extent and types of waste materials and by-products that have been used in highway construction. More than two dozen such materials have been used in some type of highway related application. Among the most frequently used are reclaimed asphalt and concrete pavements, coal fly ash and bottom ash, scrap tires, blast furnace and steel slags, and mining wastes. Every State department of transportation, as well as the District of Columbia, has experienced some usage of a waste material or by-product in their construction program.

In some cases, the evaluation or actual use of a particular waste material or by-product is initiated or stimulated by the passage of legislation or regulations at either the Federal or State level. The purpose of this paper is to provide a status report regarding legislative efforts to encourage recycling of solid wastes, particularly directives aimed at the recovery and reuse of specific waste materials by the highway construction industry.

BACKGROUND

One of the most challenging problems of modern society is how to effectively and economically handle, dispose of, or reuse growing volumes of solid wastes, while at the same time contending with a declining number of acceptable disposal sites. An increasing scarcity of available landfill space has resulted in a significant escalation in the cost of waste disposal in many sections of the country.

Sharply higher waste disposal costs adversely affect all segments of the American economy, particularly during a recessionary economic period. It has now become obvious that waste handling, treatment, and disposal costs account for a disproportionate share of the annual budget for business, industry, and virtually all levels of government. Huge sums of money are being spent to dispose of a wide variety of wastes, many of which are recyclable. Separation and processing of usable waste materials into products would be a significant economic benefit. Moreover, recovery and reuse of such waste materials and by-products would help to conserve finite supplies of natural resources.

Because huge volumes of materials are involved, the highway construction industry is frequently a target for the recycling of a wide variety of wastes and by-products. The Transportation Research Board (TRB) recently conducted an 18-month study on the use of waste materials and by-products in highway construction (Collins and Ciesielski, 1992). Among the findings from this study are that all 50 State transportation agencies, as well as the District of Columbia, have evaluated, experimented with, or are

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using one or more waste materials in their highway construction program. Over two dozen such materials have been utilized at one time or another by these agencies. The most frequently used wastes or by-products are reclaimed asphalt pavement (RAP), coal fly ash, scrap tires, blast furnace slag, steel slag, reclaimed concrete pavement (RCP), bottom ash, and mining wastes.

Increased public awareness concerning solid waste management is being reflected in a growing number of legislative initiatives and enacted laws targeting various aspects of solid waste recycling and reuse. A number of the wastes and by-products cited in the preceding paragraph are being utilized, at least in part, as a result of legislative or regulatory input. This paper presents and summarizes the extent and current status of Federal and State laws or regulations encouraging recycling and reuse of various components of solid waste, particularly as highway construction materials.

FEDERAL LEGISLATION

Resource Conservation and Recovery Act

The Resource Conservation and Recovery Act (RCRA) of 1976 (U.S. Congress, 1976) was probably the first statute to call attention to the seriousness of the solid waste disposal problem and the need to develop alternative solutions for the handling of solid waste. In promulgating RCRA, the U.S. Congress stated with respect to solid waste materials that:

- Millions of tons of recoverable materials which could be used are needlessly being buried each year.
- Methods are available to separate usable materials from the solid waste stream.
- The recovery and conservation of such materials can reduce the dependence of the United States on foreign resources and reduce the deficit in its balance of payments.

Among the stated objectives of RCRA are to conserve valuable material and energy resources by:

- Promoting a national research and development program for improved solid waste management and resource conservation techniques...and new and improved methods of collection, separation, recovery, and recycling of solid wastes.
- Promoting the demonstration, construction, and application of solid waste management, resource recovery, and resource conservation systems which preserve and enhance the quality of air, water, and land resources.
- Establishing a cooperative effort among Federal, State, and local governments and private enterprise in order to recover valuable materials and energy from solid waste.

Section 2004 of RCRA provided for grants equal to 5 percent of the purchase price of tire shredders (including portable shredders attached to tire collection trucks). Section 5003 authorized the Secretary of Commerce to take such actions as may be necessary to:

- Identify the geographical location of existing or potential markets for recovered materials.
- Identify the economic and technical barriers to the use of recovered materials.

- Encourage the development of new uses for recovered materials.

Section 6002 of RCRA requires that procuring agencies of the Federal Government, and certain entities receiving funds from the Federal Government, must procure items composed of the highest practical percentage of recovered or recycled materials, consistent with maintaining a satisfactory level of product quality, technical performance, and price competition. In addition, procuring agencies must undertake a review and revision of specifications to eliminate exclusion of recovered materials and to require recovered materials to the maximum extent practical, without jeopardizing the intended end use of the item.

Under Section 6002 of RCRA, the Administrator of the U.S. Environmental Protection Agency (EPA) was authorized to prepare, and from time to time to revise, guidelines for use of procuring agencies in complying with the requirements of this section. Such guidelines were to set forth recommended practices with respect to the procurement of recovered materials and items containing such materials, and to provide information as to the availability, sources of supply, and potential uses of such materials and items.

Procurement Guidelines for Recovered Materials

To date, EPA has promulgated five different procurement guidelines for the use or reuse of recovered materials in items or materials that are purchased with Federal funds in excess of $10,000 per year. These five guidelines cover:

- Coal fly ash in portland cement concrete
- Recycled paper
- Retreaded tires
- Building insulation
- Re-refined oil

The first guideline prepared by EPA, effective January 28, 1983, was entitled "Guideline for Federal Procurement of Cement and Concrete Containing Fly Ash" (U.S. Environmental Protection Agency, 1983). This guideline designated cement and concrete, including concrete products such as pipe and block, containing fly ash as a product area for which Government procuring agencies must exercise affirmative procurement. This guideline did not mandate the use of fly ash in concrete for Federal construction projects, but did require that cement or concrete containing fly ash be allowed to be bid as an alternate on such projects.

The guideline applies to all purchases of cement or ready-mix concrete in excess of $10,000 in which Federal funds are involved. This guideline was determined to be applicable to the Federal Aid highway construction program and has been fully implemented on the Federal and State level. All Federal and State agencies, including highway and transportation departments, have modified their specifications for portland cement concrete in order to permit the use of fly ash. Furthermore, the Federal Highway Administration (FHWA) sponsored a continuing series of regional conferences during the mid 1980's with State transportation agency personnel, engineers and contractors to discuss implementation of this guideline.

Surface Transportation Act of 1982

The Surface Transportation Assistance Act of 1982 (U.S. Congress, 1982), which authorized an additional five cents per gallon Federal tax on motor fuels, contained a Section 142, entitled "Innovative Technologies." This section of the act authorized a 5 percent increase in the share of Federal funds to a

State for construction of a highway project in which recycled materials or commercially available asphalt additives, such as ground rubber tires, were used in pavements. The intention of this section was to encourage and promote utilization of highway materials produced with a significant amount of recycled material, or which contained asphalt additives to strengthen and prolong the life of the pavement and lower maintenance costs.

On April 6, 1983, an FHWA notice was published on the subject of innovative technologies (Barnhart, 1987) Several definitions were included in this notice. Of most importance were the following:

- Asphalt Additive - a substance added to an asphalt paving mixture, such as rubber, natural, or chemical and/or natural products. However, materials such as mineral fillers, anti-stripping agents, and defoaming agents that are routinely being used are not considered eligible for additional funding under this program.

- Pavement - composite of all of the paving materials placed above the subgrade.

- Recycled Paving Materials - a paving material containing a significant amount of salvaged materials, such as ground rubber tires, and also contains the addition of a binding agent. Salvaged materials include suitable industry by-products, although it is not clear if power plant ash is included among these suitable industry by-products.

In its April 6, 1983, notice, FHWA stated that

> "On or before September 30, 1985, the Division Administration may approve an increase in the Federal Aid share by 5 percent for the entire project cost on highway and bridge surfacing, resurfacing, or restoration type projects, if the project:
>
> "a. incorporates bituminous paving materials which contain an asphalt additive in amounts consistent with standard practice and which has been proven to increase the strength of the materials and prolong the life of the pavement through a systematic evaluation program. This program shall include appropriate laboratory testing and successful in-service performance on at least three projects containing similar traffic, environmental factors, and pavement structure as the proposed project and/or
>
> "b. incorporates a significant amount of recycled paving materials."

The April 6, 1983, notice further stated that work or projects considered maintenance was applicable for Federal Aid participation. In addition, the total Federal aid share may not exceed 100 percent for any Federal Aid highway project.

The April 6, 1983, FHWA notice could be interpreted to mean an additional 5 percent share of Federal Aid highway funds would be available only if:

- Paving materials are used which contain asphalt additives such as rubber, chemical or natural products that have been proven to increase the strength of the paving material, prolong the life of the pavement, and reduce maintenance costs.

- The project incorporates a significant amount of recycled paving materials, which according to FHWA could be from 20 to 35 percent of the total mix, which would have to be declared by the contractor at the bid opening.

It is not known whether any State was able to obtain an additional 5 percent of its Federal Aid share of matching funds, as a result of this Innovative Technologies provision, but it is likely that very few, if any, States qualified for the additional funds.

Surface Transportation Act of 1987

A similar incentive provision was introduced into the Surface Transportation and Uniform Relocation Assistance Act of 1987, under Title I, Section 117, entitled Federal Share (U.S. Congress, 1987). Subsection 117(f) of Title I of this act was entitled "Incentive Program for the Use of Coal Ash." Under the provision in this subsection, the Federal Aid share of highway and bridge construction funds would be increased by 5 percent if materials produced from coal ash were used in significant amounts. This incentive program did not mandate the use of coal ash or restrict its use to certain applications or to certain portions of the transportation system, such as interstate or primary highways.

Although the term "significant amounts" was not defined in the legislation, the question of what constitutes such amounts was to be determined by the U.S. Department of Transportation (DOT), consistent with good engineering practices and the normal specifications and guidelines of the American Society for Testing and Materials (ASTM). The American Coal Ash Association (ACAA) suggested to FHWA that a minimum threshold limit of at least 1,000 tons of coal ash per project be used as a qualification for a State to be able to obtain an additional 5 percent share of Federal Aid funds (Tyson, 1987).

It is not known for certain whether any State was able to qualify for the additional 5 percent Federal Aid matching share. However, one of the problems that was later discovered regarding this incentive program was that FHWA interpreted the additional 5 percent to be available only to States which had not already received their full Federal Aid allocation. In other words, there were no additional matching funds available over and above the total amount of Federal Aid funds allocated to all the States each year. It was only in the event that one or more States were unable to provide the proper amount of State matching funds for their Federal Aid share that the unclaimed Federal Aid funds would be made available as matching funds for this incentive program. In practice, all States were able to match and claim their full Federal Aid funds, so that no additional or unclaimed funds were ever used for this coal ash incentive program (Pound).

Surface Transportation Act of 1991

Section 1038 of the recently enacted Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 addresses the use of recycled paving material, specifically the use of asphalt pavement containing recycled rubber (U.S. Congress, 1991). The Secretary of Transportation and the Administrator of the EPA shall coordinate and conduct, in cooperation with the States, a study to determine:

- The threat to human health and the environment associated with the production and use of asphalt pavement containing recycled rubber

- The degree to which asphalt pavement containing recycled rubber can be recycled

- The performance of the asphalt pavement containing recycled rubber under various climate and use conditions.

The Secretary of Transportation and the Administrator of EPA, in cooperation with the States, shall also jointly conduct a study to determine the economic savings, technical performance qualities, threats to human health and the environment, and environmental benefits of using recycled materials in highway projects, including asphalt containing over 80 percent reclaimed asphalt, asphalt containing recycled glass, and asphalt containing recycled plastic.

Not later than 18 months after the date of enactment of the Surface Transportation Act of 1991, a report shall be submitted to Congress on the results of the studies conducted under this section of the act. The report shall include a detailed analysis of the economic savings and technical performance qualities of using such recycled materials in highway projects in terms of reducing air emissions, conserving natural resources, and reducing the disposal of such materials in landfills.

The Surface Transportation Act of 1991 also has a provision for the required use of asphalt pavement containing recycled rubber. Beginning on January 1, 1995, and annually thereafter, each State shall certify to the Secretary of Transportation that such State has satisfied the minimum utilization requirement for asphalt pavement containing recycled rubber. The minimum utilization requirement for asphalt pavement containing recycled rubber as a percentage of the total tons of asphalt laid in each State shall be:

- 5 percent for the year 1994
- 10 percent for the year 1995
- 15 percent for the year 1996
- 20 percent for the year 1997, and for each year thereafter.

This minimum utilization requirement applies to all highway construction financed by Federal Aid highway funds. There is, however, a further stipulation in the Surface Transportation Act of 1991 that any recycled material or materials determined to be appropriate by the studies referred to earlier may be substituted for recycled rubber under the minimum utilization requirement, up to a maximum of 5 percent. Furthermore, the minimum utilization requirement for asphalt pavement containing recycled rubber may be increased by the Secretary of Transportation to the extent it is technologically and economically feasible to do so, and if an increase is appropriate to assure markets for the reuse and recycling of scrap tires.

STATE LEGISLATION

Pennsylvania Solid Waste Management Act Amendment

In December 1986, the Pennsylvania State Legislature amended the State's Solid Waste Management Act (P.L. 380, No. 97), in order to allow for the beneficial use, reuse or reclamation of coal ash without considering the ash as a solid waste if it is reused in certain well defined applications. Act No. 168 amended the Pennsylvania Solid Waste Management Act by adding Section 508 entitled Coal Combustion Ash and Boiler Slag (Commonwealth of Pennsylvania, 1986). According to this Section, beneficial use, reuse, or reclamation of coal ash shall include, but not be limited to, the following:

- Uses which are the subject of Federal Procurement Guidelines issued by the Environmental Protection Agency under Section 6002 of the Solid Waste Disposal Act
- Extraction or recovery of materials and compounds contained within coal ash
- Use of bottom ash as an anti-skid material

- Use as a raw material for another product
- Use for mine subsidence, mine fire control and mine sealing
- Use as structural fill, soil substitutes or soil additives

The Pennsylvania Department of Environmental Resources (DER) may, in its discretion, certify coal ash that is used as structural fill, soil substitutes, and soil additives. A person using coal ash for such purposes shall notify DER prior to such use. The DER may, in its discretion, establish operating standards governing the use of coal ash as structural fill, soil substitutes and soil additives. There are probably other States besides Pennsylvania that have enacted legislation encouraging beneficial reuse of recovered materials such as coal ash.

State Environmental Agency Questionnaire

In conjunction with the TRB Study, a questionnaire was distributed to all 50 State environmental agencies seeking information on the extent of State laws or mandates requiring State transportation agencies to investigate possible uses for waste materials. The questionnaire also requested information on beneficial use provisions in State laws, mandatory recycling laws, landfill space availability, and out of State waste reuse. A total of 45 States responded to the questionnaire. Table 1 presents the summary of responses from this questionnaire.

Out of 45 States responding, a total of 26 States (57.8 percent) indicated that legislation had been passed in their State requiring the Department of Transportation or other State agencies to investigate waste material use. At least 27 States (60.0 percent) have some form of beneficial reuse provision either in their State laws or in their waste regulations. Only 17 out of 45 States (37.8 percent) have enacted mandatory recycling laws.

Only 6 out of 45 States (13.3 percent) indicated in the questionnaire that they did not permit the reuse of out of State waste materials. These States were Hawaii, Montana, Nevada, North Dakota, Vermont, and West Virginia. Concerning the availability of landfill space, only 5 out of 45 States (11.1 percent) indicated that they did not have sufficient landfill space now. However, 18 out of 45 States (40.0 percent) indicated that they do not expect to have sufficient landfill space in the next 5 to 10 years.

State Recycling Laws

In 1990, the National Solid Waste Management Association (NSWMA) conducted a comprehensive study of State legislation related to recycling (National Solid Waste Management Association, 1991). The report from this study indicated that, as of 1990, a total of 33 States had passed some type of legislation concerning recycling. Of these 33 States, 17 States either had mandatory recycling goals or requirements that recyclable materials be separated from solid waste. The separation could either be at the source (home or business) or through the community (curbside collection or dropoff centers). Table 2 provides a summary of the status of recycling legislation, as indicated from the NSWMA study.

There are also nine States which have enacted beverage container deposit laws (so-called "bottle bills"). Michigan has a ten cent deposit on all beverage containers. The following eight States have a five cent deposit on all beverage containers:

- Connecticut
- Delaware
- Iowa
- Massachusetts
- New York
- Oregon

- Maine
- Vermont

The Scrap Tire Management Council periodically surveys the status of State legislation pertaining to the disposal and/or recycling of scrap tires within each State. Individual briefing sheets have been published for each State, indicating the status of current or pending legislation involving scrap tires. According to the most recent set of these briefing sheets, there are 34 States that have enacted some form of recycling or disposal legislation that either includes or specifically targets scrap tires. There are 7 States that prohibit landfilling of whole tires and 12 other States require that tires be cut, sliced, or chipped prior to being disposed of in landfills (Scrap Tire Management Council, 1991). Table 3 includes the findings from the Scrap Tire Management Council survey.

Disposal Bans

Disposal bans have become an increasingly common method of legislating the prevention of bulky or toxic products from entering landfills or being accepted at incinerators, thereby stimulating the potential for recycling of such products. As of 1990, according to an NSWMA report, at least 100 product disposal bans have been enacted by 29 States and the District of Columbia (11). Materials most frequently cited in these disposal bans include lead-acid batteries (27 States), unprocessed tires (14 States), yard waste (13 States), and used oil (11 States). Other waste materials have also been targets of disposal bans, as shown in Table 4.

SUMMARY

There is an increasing amount of Federal or State legislation aimed at stimulating or mandating the recycling and reuse of waste materials. In 1983, a Federal procurement guideline was issued for purchase of cement and concrete containing fly ash. Federal transportation bills in 1987 and 1992 included incentives for highway use of coal ash and scrap tires, respectively. The 1992 transportation bill mandates the use of ground rubber from scrap tires in asphalt paving, starting in 1994. Pennsylvania has amended its Solid Waste Management Act to allow for the beneficial reuse of coal ash. At least 33 States have passed solid waste recycling legislation and at least 29 States have passed laws banning one or more types of solid waste from being landfilled. At least 34 States have enacted some form of recycling or disposal legislation involving scrap tires.

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TABLE 1

SUMMARY OF RESPONSES TO STATE ENVIRONMENTAL AGENCY QUESTIONNAIRE

	Legislation for Use of Waste Materials	Beneficial Reuse Provisions in Law	Mandatory Recycling Provisions in Law	Lack of Available Landfill Space	
				Now	5 - 10 years
1. Alabama	X		X		X
2. Alaska*					
3. Arizona	X				
4. Arkansas*					
5. California			X		X
6. Colorado		X			
7. Connecticut	X				
8. Delaware		X			
9. Florida	X		X		
10. Georgia*					
11. Hawaii				X	X
12. Idaho	X	X	X		X
13. Illinois					
14. Indiana	X	X			
15. Iowa*					
16. Kansas			X		
17. Kentucky					
18. Louisiana	X		X		
19. Maine	X	X	X	X	X
20. Maryland			X		
21. Massachusetts		X			X
22. Michigan	X	X			X
23. Minnesota	X	X	X		X
24. Mississippi	X	X	X		X
25. Missouri	X	X			

*Did not respond to questionnaire.

TABLE 1 (Continued)

SUMMARY OF RESPONSES TO STATE ENVIRONMENTAL AGENCY QUESTIONNAIRE

	Legislation for Use of Waste Materials	Beneficial Reuse Provisions in Law	Mandatory Recycling Provisions in Law	Lack of Available Landfill Space	
				Now	5 - 10 years
26. Montana		X			
27. Nebraska		X			
28. Nevada	X		X		
29. New Hampshire	X	X			
30. New Jersey	X	X	X	X	X
31. New Mexico		X			
32. New York	X	X		X	X
33. North Carolina	X				X
34. North Dakota					
35. Ohio		X			
36. Oklahoma		X			
37. Oregon	X				X
38. Pennsylvania	X	X	X		
39. Rhode Island	X	X	X		X
40. South Carolina		X			X
41. South Dakota		X			
42. Tennessee*					
43. Texas	X	X			
44. Utah		X			
45. Vermont					X
46. Virginia	X	X	X		
47. Washington	X	X		X	X
48. West Virginia		X	X		X
49. Wisconsin	X		X		
50. Wyoming		X			

*Did not respond to questionnaire.

TABLE 2

SUMMARY OF STATE RECYCLING LEGISLATION

	States With Recycling Laws	Status of State Recycling Laws			
		Recycling Plans Only	Mandatory Goals	Source Separation	Community Separation
1. Alabama					
2. Alaska					
3. Arizona	X				
4. Arkansas	X	X			
5. California	X		X		
6. Colorado					
7. Connecticut	X			X	
8. Delaware	X				
9. Florida	X		X		X
10. Georgia	X				X
11. Hawaii	X				
12. Idaho					
13. Illirois	X				X
14. Indiana	X	X			
15. Iowa	X	X			
16. Kansas					
17. Kentucky					
18. Louisiana					
19. Maine	X			X	
20. Maryland	X		X		X
21. Massachusetts	X	X			
22. Michigan	X	X			
23. Minnesota	X		X		X
24. Mississippi					
25. Misscuri	X	X			

TABLE 2 (Continued)

SUMMARY OF STATE RECYCLING LEGISLATION

	States With Recycling Laws	Status of State Recycling Laws			
		Recycling Plans Only	Mandatory Goals	Source Separation	Community Separation
26. Montana					
27. Nebraska					
28. Nevada					
29. New Hampshire	X	X			
30. New Jersey	X		X		
31. New Mexico	X	X			
32. New York	X			X	
33. North Carolina	X				X
34. North Dakota					
35. Ohio	X		X		X
36. Oklahoma	X	X			
37. Oregon	X				
38. Pennsylvania	X			X	
39. Rhode Island	X		X	X	
40. South Carolina					
41. South Dakota					
42. Tennessee	X	X			
43. Texas					
44. Utah					
45. Vermont	X	X			
46. Virginia	X		X		X
47. Washington	X				
48. West Virginia	X	X			
49. Wisconsin	X				X
50. Wyoming					

TABLE 3

SUMMARY OF STATE SCRAP TIRE LEGISLATION

	Status of Scrap Tire Legislation			
	No Tire Laws	No Landfill Disposal	Cut or Shred for Landfill	Tire Tax or Fee
1. Alabama				
2. Alaska	X			
3. Arizona			X	X
4. Arkansas			X	X
5. California				X
6. Colorado	X			
7. Connecticut				
8. Delaware	X			
9. Florida			X	X
10. Georgia	X		X	
11. Hawaii	X			
12. Idaho				X
13. Illinois				X
14. Indiana				
15. Iowa		X		
16. Kansas		X		X
17. Kentucky			X	X
18. Louisiana			X	X
19. Maine				X
20. Maryland				X
21. Massachusetts				
22. Michigan				X
23. Minnesota		X		
24. Mississippi			X	X
25. Missouri		X		X

TABLE 3 (Continued)

SUMMARY OF STATE SCRAP TIRE LEGISLATION

	Status of Scrap Tire Legislation			
	No Tire Laws	No Landfill Disposal	Cut or Shred for Landfill	Tire Tax or Fee
26. Montana	X			
27. Nebraska	X			X
28. Nevada	X			
29. New Hampshire				
30. New Jersey	X			
31. New Mexico	X			
32. New York				
33. North Carolina			X	X
34. North Dakota	X			
35. Ohio			X	
36. Oklahoma				X
37. Oregon			X	X
38. Pennsylvania				
39. Rhode Island		X		X
40. South Carolina				X
41. South Dakota	X		X	
42. Tennessee		X		
43. Texas				
44. Utah	X			X
45. Vermont		X		
46. Virginia				X
47. Washington				X
48. West Virginia			X	
49. Wisconsin				X
50. Wyoming				

TABLE 4

SUMMARY OF STATE DISPOSAL BANS FOR VARIOUS WASTE MATERIALS

	States with Disposal Bans	Waste Materials Banned From Disposal (or Incineration)					
		Lead-Acid Batteries	Yard Waste	Whole Tires	Used Oil	C & D Debris	Beverage Containers
1. Alabama							
2. Alaska							
3. Arizona							
4. Arkansas							
5. California	X	X					
6. Colorado							
7. Connecticut	X	X	X[1]	X			
8. Delaware							
9. Florida	X	X	X[2]	X	X	X	
10. Georgia	X	X					
11. Hawaii	X	X					
12. Idaho							
13. Illinois	X	X	X	X			
14. Indiana							
15. Iowa	X	X	X	X	X		X
16. Kansas	X	X		X			
17. Kentucky	X	X					
18. Louisiana	X	X		X	X		
19. Maine	X	X					
20. Maryland							
21. Massachusetts	X	X	X	X	X		X[5]
22. Michigan	X	X	X		X		
23. Minnesota	X	X	X	X	X	X	
24. Mississippi							
25. Missouri	X	X	X	X	X		

TABLE 4 (Continued)
SUMMARY OF STATE DISPOSAL BANS FOR VARIOUS WASTE MATERIALS

	States with Disposal Bans	Waste Materials Banned From Disposal (or Incineration)					
		Lead-Acid Batteries	Yard Waste	Whole Tires	Used Oil	C & D Debris	Beverage Containers
26. Montana							
27. Nebraska							
28. Nevada							
29. New Hampshire	X	X					
30. New Jersey	X		X[1]				
31. New Mexico							
32. New York	X	X					
33. North Carolina	X	X	X[2]	X	X		
34. North Dakota							
35. Ohio	X	X	X	X			
6. Oklahoma							
37. Oregon	X	X		X			
3€. Pennsylvania	X	X	X[1]				
39. Rhode Island	X			X[3]			
40. South Carolina							
41.							
42. Tennessee	X	X					
43.							
44. Utah							
45. Vermont	X	X		X	X		
46. Virg!nia	X	X					
47. Washington	X	X					
48. West Virginia							
49. Wisconsin	X	X	X	X[4]	X[4]		X
50. Wyoming	X	X					

[1]Disposal ban applies only to leaves. [2]Applies only to lined landfills.
[3]Banned only from incinerators. [4]Can be burned at energy recovery incinerators only.
[5]Applies to aluminum, plastic, glass and metal containers.

RECYCLED RUBBER IN HIGHWAY CONSTRUCTION AND MAINTENANCE

By Jon A. Epps[1] and Amy L. Epps[2]

INTRODUCTION

Combinations of asphalt and cement and rubber have been used as a pavement construction, rehabilitation and/or maintenance material for over 50 years. Natural, synthetic, and recycled rubbers have been used in the United States with varying degrees of success. The use of scrap rubber and recycled tire rubber for paving purposes largely resulted from work conducted by U.S. Rubber Recycling in Vicksburg, Mississippi, in the 1960's and 1970's and by various producers in Arizona (Sahuaro Petroleum and Asphalt and Arizona Refining Company) stimulated by the work of Charles McDonald in the 1970's.

A number of Federal agencies including the Federal Highway Administration, Federal Aviation Administration, Department of Energy and the Environmental Protection Agency have conducted research which considers the use of scrap rubber and recycled tire rubber in pavements. Additionally, several state departments of transportation including Arizona, California, Oregon, Texas, Florida, New York, and Connecticut have conducted fairly extensive laboratory and/or field research projects. The countries of Australia, Canada, and South Africa have also performed research on asphalt cement-rubber binder systems.

Terminology used to define combinations of scrap rubber and recycled tire rubber in combination with asphalt cement for use as a pavement binder has been developing over the last 20 years. Unfortunately, universally accepted definitions are not available and need both development and acceptance. This problem is reflective of the fact that a fairly wide spectrum of materials have been used to formulate so called "asphalt-rubber binders." Some asphalt-rubber binders contain only scrap rubber, others contain whole tire rubber, or combinations of scrap rubber and selected segments of tires. In addition, asphalt-rubber formulations may contain petroleum diluents, aromatic oils and/or polymers of different types. The technology is developing as lessons are obtained from both laboratory and field research programs.

The major uses of scrap rubber and tire rubber in pavements have been as a binder and/or partial aggregate replacement for:

1. chip seals and interlayers,
2. dense-graded hot mix asphalt,
3. open-graded hot mix asphalt,
4. joint and crack sealers,

and other miscellaneous uses including membranes, slope protection, fills, fuel and is a raw material source for other products.

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[2] Graduate Student, University of California, Berkeley.

A brief discussion of each of the major uses in pavements follows.

CHIP SEALS AND INTERLAYERS

Spray application of asphalt-rubber binders include those for chip seals, interlayers and surface treatments. Asphalt-rubber binders for these uses are those obtained by reacting the rubber with an asphalt cement and is often referred to as the "reacted" or "wet" process. Most often special blending, reacting and spraying equipment is used to prepare and apply the binder.

Asphalt-rubber chip seals and interlayers can typically cost two to three times that of a chip seal constructed with a conventional binder. Performance information is not conclusive but to be cost effective the asphalt-rubber chip seal must have a performance life approximately 3 times that of a conventional chip seal (approximately 18 years versus 6 years) or be applied under traffic and/or environmental conditions that do not allow for the placement or result in a short life of a conventional chip seal.

Asphalt-rubber interlayers, as chip seals, have been used fairly extensively in the United States. Relatively good performance has been obtained with this type of construction in the hot climates. From an annualized cost analysis, overlays with asphalt-rubber binders interlayers must last approximately 50 percent longer than an overlay constructed without an interlayer.

In general, material specifications and design methods are available for this type of construction. Asphalt-rubber binder specifications are, however, in need of better test methods and limits. Most binder specifications are method types.

DENSE-GRADED HOT MIX ASPHALT

The use of asphalt-rubber binders in hot mix asphalt has accelerated in the late 1980's. Laboratory research suggests that improved fatigue performance and rutting resistance can be obtained with these mixtures. Field data are largely inconclusive at this time due to the limited time these materials have been subjected to traffic. Field performance in California does, however, suggest that improved performance and beneficial life cycle costs are obtained when these mixtures are used as overlays on pavements with high deflection.

Typically costs for asphalt-rubber hot mix asphalt are 25 to 50 percent greater than conventional mixtures on an equal thickness basis.

Asphalt-rubber binders for hot mix asphalt materials are prepared by either the "reacted or wet" process and the "nonreacted or dry" process. The "dry" process produced binder is often called a rubber-modified binder and in this process the rubber is placed in the aggregate prior to the introduction of the asphalt cement and mixing occurs. In the "dry" process the rubber is larger in size than that used in the "wet" process and typically 2 to 4 times the amount of rubber is used. Fewer items of specialized construction equipment are required with the "dry" process as compared to the "wet" process.

Material specification and mix design methods are underdeveloped for hot mix asphalt products. Research is largely inconclusive relative to the benefits in terms of snow and ice removal, friction number and noise reduction derived from using asphalt-rubber binders in hot mix asphalt. Some reports suggest that snow and ice removal is improved when the "dry" produced asphalt-rubber binder is used as compared to conventional mixtures. Other reports indicate no benefit in terms of snow and ice removal and wet friction values. Noise reduction appears possible with the use of asphalt-rubber modified mixtures.

Specialized construction equipment is required for blending, reacting and pumping asphalt-rubber binders produced by the "wet" process for hot mix asphalt. The "dry" process requires extra proportioning and delivery equipment only. Conventional hand and laydown equipment is typically used without major problems. Joint placement has been a problem on some projects.

OPEN-GRADED HOT MIX ASPHALT

The use of asphalt-rubber binders in open-graded hot mix asphalt has developed in parallel with that of dense-graded hot mix asphalt. Field performance data suggests that improved performance can be obtained as thicker films of binder are possible. Conclusive life cycle costs studies are largely not available due to the relatively limited time these materials have been subjected to traffic.

The State of Florida has determined that the most effective use of tire rubber will be in surface course mixtures.

As stated previously, both the "wet" and "dry" processes have been used to produce the asphalt-rubber binders for this form of hot mix asphalt. Mix design and construction concerns relative to the use of open-graded and dense-graded hot mix asphalt are similar.

JOINT AND CRACK SEALERS

The use of asphalt-rubber products as joint and crack sealers is widespread in the United States and probably the most accepted use of the binder system. These sealers are normally produced using the reacted system and often contain additives other than rubber and asphalt cement.

Field performance has largely been acceptable as compared to other products presently marketed. Cold temperature performance of joint and crack sealing products is questionable. These products appear to be cost effective.

Depending on the product produced, specialized heating and pumping units may have to be used for application. These extra equipment items are relatively low in cost.

SUMMARY

1. Asphalt-rubber research, development, implementation and marketing has been underway on an expended scale since the 1970's. A large number of different types of binder systems have been developed and marketed as "asphalt-rubber" and hence performance and life cycle cost information on the present generation of binders is somewhat inconclusive. The development effort has resulted from lessons learned from both laboratory and field research efforts.

2. Major uses of scrap rubber and tire rubber in pavements has been as a binder and/or partial aggregate replacement for:

   a. chip seals and interlayers,

   b. open-graded and dense-graded hot mix asphalt, and

   c. joint and crack sealers.

3. Field performance studies to date on these various uses of asphalt-rubber are often inconclusive as are less cycle cost studies. Some applications of materials containing asphalt-rubber are often inconclusive as are life cycle cost studies. Some applications of materials containing asphalt-rubber binders appear to be cost effective.

4. For equal thickness applications, asphalt-aggregate mixtures containing asphalt-rubber binders are more costly. Funding for this difference in first cost must be considered prior to widespread acceptance of this binder system as potential disposal systems for tires. Fees collected for the disposal for tires must be diverted to revenues for pavements if disposal is in the pavement. These funds could be used for research, development and construction costs.

5. Improved test methods and specifications for asphalt-rubber binders are needed.

6. Improved test methods and specifications for hot mix asphalt are needed.

7. Health, safety and environmental issues need to be more thoroughly researched. The question of fumes relative to health and safety when using asphalt-rubber binders has not been answered.

8. Recyclability of mixtures containing asphalt-rubber binders must be addressed from a construction equipment and health and safety viewpoint.

9. The life cycle costs of asphalt-rubber binder systems needs to be compared to other polymer-modified binders with adequate consideration given to environmental benefit of reducing the solid waste stream.

10. Public works and contractor personnel will need training to adequately use (design, produce and construct) asphalt-rubber binder systems.

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EPA FEDERAL PROCUREMENT GUIDELINES AND THE IMPACT OF RCRA REAUTHORIZATION

By Barbara M. Scharman[1]

OVERVIEW OF PROCUREMENT GUIDELINES

Identifying markets for recovered materials is crucial to efforts to reduce the quantities of solid waste currently requiring disposal in the United States. The Federal government purchases enormous quantities of certain items and, thus, can have a major impact by purchasing those items made with recovered materials. Federal purchases of items made with materials otherwise destined for disposal have a direct impact on decreasing the size of the solid waste stream. At the same time, the Federal government's commitment to buying these items sends a message to industry, both manufacturers and recyclers, that steady and significant markets exist for both the recovered materials and products manufactured using these materials as feedstocks. Under Section 6002 of the Resource Conservation and Recovery Act (RCRA), Federal procuring agencies are required to comply with requirements for purchasing items containing recovered materials. The Environmental Protection Agency (EPA) is responsible for issuing Federal procurement guidelines to assist Federal procuring agencies in complying with the RCRA requirements. Federal procurement guidelines designate procurement items containing recovered materials and set forth recommended practices with respect to their procurement.

RCRA requirements and EPA procurement guidelines apply to Federal, state, and local agencies and their contractors when more than $10,000 per year of Federal funds are used by an agency to purchase a guideline item. Procuring agencies are required to (1) revise or draft product specifications to allow for the purchase of items containing recovered materials, and (2) to establish an affirmative procurement program to implement procurement guidelines. Federal specifications that unnecessarily exclude items containing recovered materials must be reviewed and revised to allow for the purchase of items containing these materials. Likewise, Federal agencies should not exclude items containing recovered materials when drafting new specifications. Each procuring agency must also design an affirmative procurement program to implement procurement guidelines. Affirmative procurement programs must contain:

- **A preference program** specifying minimum content standards for recovered materials in procurement items, case-by-case procurement involving competition between products containing virgin materials and those containing recovered materials, or some similar approach;

- **A promotion program** whereby procuring agencies express their intention to buy items containing recovered materials in general, and more specifically in bid solicitations and requests for proposal;

- **Estimate and certification verification** by vendors that provide both estimates of recycled content of procurement items and certify the percentage of recycled content as part of the contract award;

- **An annual review and monitoring program** by procuring agencies to adjust the minimum content standards they are using, should market factors warrant a change.

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EPA has published five guidelines designating products containing recovered materials as procurement items when purchased by procuring agencies:

1. Procurement Guideline for Cement and Concrete Containing Fly Ash - January 28, 1983 (48 FR 4230)

2. Procurement Guideline for Paper and Paper Products - June 22, 1988 (53 FR 23546)

3. Procurement Guideline for Lubricating Oils Containing Refined Oil - June 30, 1988 (53 FR 24699)

4. Procurement Guideline for Retread Tires - November 17, 1988 (53 FR 46558)

5. Procurement Guideline for Building Insulation Products Containing Recovered Materials - February 17, 1989 (54 FR 7328).

EPA recommended minimum content standards for items containing recovered materials in paper products, lubricating oils, and building insulation products. EPA recommended that all construction contracts contain provisions allowing for the use of cement and concrete-containing fly ash. The EPA guideline for retreaded tires recommended that procuring agencies use retreading services for their used tires and choose retreaded tires when purchasing tires.

As of the spring of 1992, EPA is planning to propose a guideline for fiberboard containing recovered paper and is studying the feasibility of developing guidelines for other products. The products currently under consideration include geotextiles and plastic pipe made with recovered plastic, hydraulic mulch made with recovered newspapers and yard waste compost.

EPA has established four criteria for determining the feasibility of selecting products as designated procurement items. These criteria are used in evaluating candidates when proposing a guideline. They are:

- The recovered material used in making the product must be a significant solid waste management problem in the United States;
- Economic methods of separation and recovery for the waste material must exist;
- The waste must have technically proven uses in the designated item;
- The Federal government's ability to affect purchasing or use of the item containing recovered material must be substantial.

Procuring agencies are not always required to purchase items containing recovered materials, as specified in a guideline. The reasons for not purchasing a guideline item are:

- The price of the product containing recovered material is unreasonable;
- Purchasing the product containing the recovered material, as specified by the minimum content standard, would result in inadequate competition;

- The designated items are not available in a reasonable period of time;

- The designated items fail to meet reasonable performance specifications.

EXECUTIVE ORDER ON FEDERAL AGENCY RECYCLING AND PROCUREMENT

On October 31, 1991, President Bush signed Executive Order 12780, entitled Federal Agency Recycling and the Council on Federal Recycling and Procurement Policy. This order requires all Federal agencies to establish recycling programs and increase the use of products containing recovered materials. The purpose of the executive order, in terms of the Federal procurement provisions, includes:

- The implementation of cost-effective Federal procurement programs favoring the purchase of designated items produced from recovered materials;

- The review of Federal agency specifications and standards and any necessary changes to these to enhance Federal procurement of products made from recovered materials;

- The collection and dissemination of market and price information for products made from recovered materials;

- The reporting of Federal agency efforts to develop and adopt Affirmative Procurement Programs.

CONGRESSIONAL LEGISLATION

At this time, several bills to amend RCRA are being considered before the Senate and House that include provisions concerning Federal procurement requirements. The potential impact of RCRA reauthorization includes requiring EPA to prepare additional guidelines for procurement items, to define "unreasonable price", and to include other steps, depending on the proposed bill. The draft Senate Bill S. 976 (March 27, 1992) requires the following of EPA and other Federal agencies:

- Procuring agencies must give preference to products containing the highest percentage of recovered materials practicable, regardless of whether guidelines exist for the products.

- An "unreasonable price" for products containing recovered materials is defined as a price that exceeds by more than 10 percent the price of similar products not meeting the guidelines.

- Procuring agencies must set aside funds to purchase products that satisfy the guidelines for contracts worth $1 million or more.

- The EPA Administrator will contract with the American Society for Testing and Materials to prepare guidelines for the development of performance standards for items containing recovered materials.

- EPA will prepare final guidelines for rubber products (including rubberized asphalt pavement), compost, plastic products (including containers), and lead-acid batteries within 12 months of enactment of the bill.

- EPA will prepare final guidelines for products made from glass, ferrous metals, and nonferrous metals within 24 months after enactment of the bill.

- The Office of Procurement Policy in the Executive Office of the President will enforce compliance with procurement requirements. Federal agencies will report to EPA and the Office of Procurement Policy on compliance with procurement requirements.

- The Secretary of Defense and the Administrator of EPA are required to review all specifications for items procured by the Department of Defense and eliminate any requirements that discriminate against the use or acquisition of items containing recovered materials.

The House bill, H.R. 3865 (March 26, 1992), requires the following:

- An "unreasonable price" for products containing recovered materials is defined as a price 10 percent higher than that for products made from virgin materials.

- Procuring agencies must procure items that comply with guidelines unless the items are not available within a reasonable time period, fail to meet applicable performance standards, and are available only at unreasonable prices.

- Federal agencies must develop a procurement program to carry out procurement requirements within 1 year of enactment of the bill.

- Federal agencies must review specifications and eliminate both exclusions of products made with recovered materials and requirements that items be manufactured from virgin materials.

- EPA must prepare guidelines for agencies to use in complying with RCRA procurement requirements.

- EPA will revise guidelines for paper and used oil fuel within 12 months of enactment of the bill.

- Procuring agencies must comply with new guidelines for paper by January 1, 1997.

- Procuring agencies must publish annual reports on their progress in implementing procurement requirements.

- EPA will prepare final guidelines for compost, glassphalt, lead-acid batteries, and rubberized asphalt within 18 months of enactment of the bill.

- EPA will prepare final guidelines for additional paper products, products made from pulp and paper mill sludge fiber, and rebuilt auto parts within 3 years of enactment of the bill.

- EPA will establish a procurement information clearinghouse.

The actual impact of RCRA reauthorization will depend on the final version of the bill that eventually is enacted. At that time, guideline development requirements, price preferences, funding allotments for guideline item purchases, requirements of EPA, and procuring agencies will be better understood.

FLORIDA RESOURCE RECOVERY AND MANAGEMENT ACT

By Lawrence L. Smith[1]

The Florida Department of Transportation (FDOT) began investigating the feasibility of utilizing certain waste materials for highway applications as mandated by the Florida Legislature (S.B. 1192) in August 1988. These wastes included flyash for soil stabilization, used motor oil, glass for asphalt (glassphalt), plastics, and ground tire rubber in asphalt. When deemed feasible, it was imperative to develop acceptable test protocols such that specifications could be developed to permit state purchases.

Plastic studies initially focused on fence line posts but now have expanded into sign substrates (specifications written), rebar support chairs and flexible delineator posts. Currently, under evaluation, are work zone barrier posts, signposts, guardrail offset blocks, guardrail posts and drainpipe. FDOT has recently purchased 5,000 recycled plastic fenceposts.

Waste motor oil in DOT related cement and asphalt hot mix industries was to be utilized as an alternative burner fuel. Both industries could use nearly 10 million gallons although only 5.1 million gallons annually are available. This would result in estimated fuel cost savings of 1 to 25 percent compared to bunker oil grades 2, 5, and 6.

Ground tire rubber in highway applications has been evaluated; and recommendations for use in open graded friction courses, dense graded friction courses and membrane interlayers have been developed. Specifications for FDOT review are complete and ready for submittal to FHWA. Construction of three "technology transfer" projects are planned within various areas of the State to familiarize FDOT and contractors with the process. Following construction of these projects, full implementation will begin throughout the State.

Glass as an aggregate replacement in asphalt mixtures has been evaluated in the laboratory. Based on the data generated, a "glassphalt" project will be constructed on a county road in West Palm Beach. FDOT will evaluate the performance of this material and development specifications for further use. Economically, it will be beneficial to use crushed glass where large amounts are generated and where local sands are not available for use in asphalt mixtures. In addition, other uses for crushed glass are being evaluated.

For soil stabilization, three sources of flyash from the western area of Florida were tested and used in the design of several test sections. A proposal for a demonstration project using flyash in soil stabilization and embankment construction has been submitted to the Department of Environmental Regulation (DER) Tallahassee Office. A positive response from DER will lead to the completion of the project.

[1] Florida Department of Transportation, Tallahassee, FL.

TECHNICAL SESSION 8

Soil Stabilization With Contaminated Soils

LANDFILL SOIL STABILIZATION - IMPLICATIONS OF LEACHATE, COMBUSTIBLE GASES, AND PHYSICAL CONSTRAINTS TO REVEGETATION

By T. W. Hilditch[1]

Abstract: The erosion of a landfill soil cover can create opportunities for the migration of contaminants. The erosion allows moisture penetration into the entombed refuse, resulting in an increase in the production of combustible gases and leachate.

A 3-year research program conducted in Ontario, Canada, resulted in a determination of the degree to which landfill erosion was a problem and the production of a series of recommendations to stabilize soils on landfills. A "how to" manual was produced to assist landfill owners and operators in managing soil covers.

Through a field trial program and literature review, it was determined that key factors affecting the stabilization of landfill soils via revegetation were: degree of slope, soil characteristics, plant material selection and planting techniques. More specific field trial results are presented along with information regarding the tolerance of various plant species to combustible gases and leachate.

INTRODUCTION

Landfills present some specific challenges to those charged with the responsibility of developing closure plans and maintaining closed facilities. In the past, refuse has been buried and covered with a relatively compacted layer of soil. The texture and depth has varied but in many cases, it has provided insufficient protection of the refuse from the environment. In some cases planned revegetation programs have been implemented to ensure the long term stability of that soil cap. More commonly, however, the compacted cap itself has been left exposed to environmental conditions (e.g., freeze-thaw cycles, precipitation and desiccation) and prone to erosion. It has typically been seeded with an inappropriate mix of grasses, or was abandoned in favor of colonization by aggressive, weedy grasses and herbaceous plants.

Inadequate protection of the landfill cap can lead to erosion problems. During our assessment of the Ontario situation, it was determined that fully 75 percent of landfills investigated reported some degree of erosion. On 40 percent of landfills, no formal revegetation program had been undertaken upon closure. Other studies have presented similar conclusions (e.g., Johnson, 1986a; Johnson, 1986b). Swope (1975) reported that several Pennsylvania landfill operators displayed a lack of knowledge about how to remediate landfill soil erosion problems. He reported that erosion was present on 18 of the 19 landfills that he examined.

The erosion of a landfill cap allows an opportunity for moisture penetration which can create contaminant migration problems. Increased moisture levels within the entombed refuse leads to increased leachate and combustible gas production and release.

The anaerobic decomposition of buried refuse produces relatively high concentrations of methane and carbon dioxide and smaller concentrations of ammonia. Under some conditions, methane can create

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explosive hazards. These gases act to displace oxygen from the zone in which vegetation roots (Shafer et al., 1984). This suffocation can lead to plant stress and mortality which can create opportunities for further cap erosion and contaminant migration. Leachate springs can result from ground water emergence, generally at the base of landfill slopes. These seepage areas possess certain chemical constituents, depending upon the nature of the refuse, that can cause a phytotoxic response in some vegetation.

Beyond the effects on vegetation, the migration of contaminants from these soils can result in increased maintenance costs, degraded aesthetics and the likelihood of noncompliance with the site license or regulatory requirements.

This paper summarizes a 3-year research program which investigated the degree and extent of erosion on landfill sites in Ontario, and developed recommendations to stabilize landfill surfaces. A "how to" manual for the erosion control aspects of landfill closure was published.

METHODS

This 3-year research study was commissioned by Ontario's Ministry of the Environment (MOE). During the early stages of the study, a sense for the magnitude of this specific erosion problem was obtained by circulating a questionnaire to 19 MOE District offices, and by completing followup field inspections at 24 representative landfill sites across the province. Locations of those sites are depicted on Figure 1. Detailed observations of cap and cover soil materials, compaction and the presence of combustible gases and leachate, were made. Combustible gases were measured in the upper 50 cm of soil using an M.S.A. explosimeter. Oxygen levels were measured using a Pyrite oxygen tester.

Building upon the questionnaire and reconnaissance inspections of 24 sites, the study moved into its second phase - an extensive literature review in conjunction with the establishment of a series of Test/Demonstration Plots.

From the 24 sites examined, 8 were selected for the establishment of plots. They are highlighted on Figure 1. The plots were intended to satisfy two objectives:

1. to test or compare a variety of preferred revegetation techniques, and

2. to provide demonstration plots that could serve as an educational and scientific resources to interested parties.

The plots were selected to provide maximum representation in the different MOE regions in the province. Specific sites were selected where landfill owners/operators were willing to participate in research and where the site size allowed an opportunity for optional plot placement. The plot designs were relatively complex and were used to test a variety of parameters and techniques. A brief summary of plot establishment follows.

Test plots were initially subject to a variety of tillage techniques to reduce soil compaction. Fertilization requirements were determined from the collection and analysis of composite soil samples from each site. The composition of and application rates of the fertilizer varied from site to site but were generally between 300 kg/ha and 500 kg/ha.

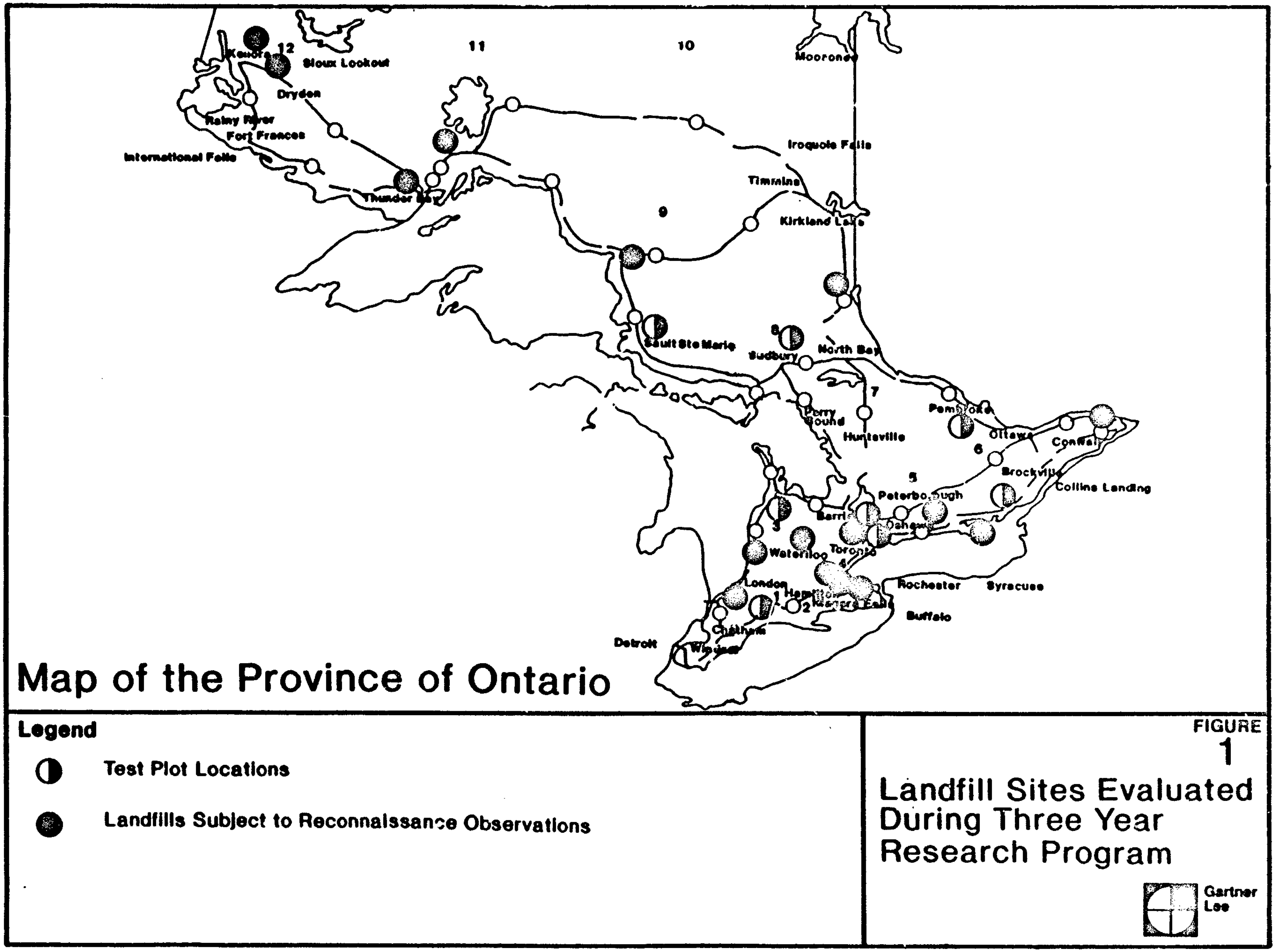

Figure 1. - Landfill sites evaluated during 3-year research program.

Lime (calcium carbonate) was required for two northern Ontario sites in order to increase pH levels in the soil. Sewage sludge was incorporated into the topsoil of one site and spread over the surface on a second landfill. In both cases, the material was obtained from municipal sewage treatment operations.

A seed mix (Table 1) was applied to the demonstration plots. Criteria for the selection of an appropriate mix are discussed later in this paper. Three seeding techniques were applied: hydroseeding, broadcast seeding and seed drilling. Seed application rates were 100 kg/ha on all southern Ontario sites and 120 kg/ha on the two more northern sites. Those two sites were characterized by relatively steep-sided slopes and more coarse infertile soils. Seeding for sites occurred in May and June 1988.

Table 1. - Plant species seeded on demonstration plots.

Agropyron dasystachyum	Northern Wheat Grass
Agrostis gigantea	Red top
Bromus inermis	Smooth Brome Grass
Koelia cristata	June Grass
Lolium multiflorum	Annual Rye
Lolium perenne	Perennial Rye
Phleum pratense	Timothy
Poa pratensis	Kentucky Bluegrass
Trifolium repens	White Clover

A variety of mulches were tested including: latex spray, fibre-mulch, straw with emulsified asphalt and erosion control blankets. Rates of mulch application varied, but were in the order of 1,500 kg/ha.

RESULTS

The following is a summary of results from the test plot program. Information was gathered regarding plant species effectiveness in establishing on the difficult soils and population trends during the first two growing seasons.

Generally half of the test plots displayed dense, vigorous vegetative growth (50 to 70 percent total cover). Table 2 indicates that the other half displayed a poor catch (0 to 20 percent total cover). In all cases, the vegetative cover consisted of a mix of planted and invasive species. Table 2 provides further information regarding the percentage of planted versus invasive species on each plot. It is apparent that planted species dominated the majority of test plots, as would be expected. Also provided is an indication of whether those planted populations were increasing or decreasing in percent cover from the first to second year of observation. Populations decreased on the two northern Ontario plots, probably in response to more drought conditions associated with the coarse, sandy soils used for the landfill cap and cover.

More specifically, an examination of species lists compiled for each demonstration plot suggested the following were the most frequently encountered species: Smooth Bromegrass (*Bromus inermis*), Creeping Red Fescue (*Festuca rubra*), Rye (*Lolium sp.*), Timothy (*Phleum pretense*), and Canada Bluegrass (*Poa pratensis*). Bromegrass and Timothy were often more dominant in terms of percent cover. Common Ragweed (*Ambrosia artemisiifolia*), Barnyard Grass (*Echinochloa crusgalli*), Witchgrass (*Panicum capillare*) and Green Foxtail (*Setaria viridis*) were the most frequent and dominant nonplanted, or invasive, species, recorded from the sites.

Table 2. - Demonstration/Test plot results - average vegetative cover (%) on study sites.

Landfill location / Degree of cover and year of observation	Toronto	Peterborough	Kingston	Pembroke	Sudbury	Sault Ste. Marie	Owen Sound	London
Average total cover (%) 1988	12.6	59.5	19.3	-	17.3	5.3	50.4	52.5
Average total cover (%) 1989	17.4	28.6	35.7	67.6	14	4.6	24.3	47.4
Percent change in average Total cover: 1988 to 1989	+38	-52	+85	-	-19	-13	-52	-5
Percent of average cover composed of planted species versus invasive								
1988 (%)	28	6	74	-	98	88	64	67
1989 (%)	47	21	89	89	87	64	90	75
Percent change in percent of planted species versus invasive species: 1988 to 1989	+68	+267	+20	-	-11	-27	+41	+12

Notes:

1. Average total cover was determined by summing the results of quantitative measurements on each plot at each landfill, then dividing by the number of plots. This provided an opportunity to examine gross trends in growth rage over the 2 years of monitoring.
2. Quantitative measurements were undertaken by using the randomly placed quadrants within each plot on a landfill. Each landfill had an average of four demonstration plots plus a control plot established.
3. Vegetative cover on control plots was excluded from calculations.

Similar positive results were achieved with Timothy and Annual Ryegrass (*Lolium multiflorum*) in another southern Ontario landfill situation (MTRCA, 1984). In that situation, Tall Fescue (*Festuca arundinacea*) and Creeping Red Fescue formed dense cover in some isolated patches. Duel et al. (1986) found that in their landfill revegetation trials, Red Top (*Agrostis alba*), Hard and Sheep Fescues (*Festuca longifolia and F. ovina*, respectively), consistently provided superior cover development.

Due to the difficulties in establishing vegetation on areas of gas emissions, the performance of certain plant species in areas of known combustible gas release was of special interest. Six invasive species were consistently observed in areas of measured, oxygen poor soils. These were Barnyard Grass, Green Foxtail, Common Ragweed, Witchgrass, Redroot Pigweed (*Amaranthus retroflexus*), and Lady's-thumb (*Polygonum persicaria*).

Random measurements on a number of sites suggested that symptoms of stress (grey brown coloration and mild curling of leaves) became pronounced when total combustible gas levels in the soil were between approximately 5 and 15 percent of the Upper Explosive Limit. More severe symptoms and some mortality were evident above 15 percent, while complete mortality occurred in most areas where combustible gas levels exceeded 30 percent of the Upper Explosive Limit.

DISCUSSION

The literature reviewed, sites visited and test plots evaluated, all contributed to an understanding of the physical constraints to soil stabilization and vegetation establishment in a landfill setting. Specific summaries are provided below, regarding the most influential factors: slope, soil characteristics, plant material selection and planting techniques. Recommendations to optimize success are also provided.

Slope

It was apparent from observations on the sites examined that steeper slopes were more prone to slope slippage and erosion. Vegetation establishment was more difficult owing to the more severe runoff effects. A slope of 50 percent (26 inches) is the maximum upon which vegetation can reasonably be established and maintained, assuming ideal soil with low erodibility and adequate moisture holding capacity. A slope of 33 percent (18 inches) is the maximum for establishing acceptable vegetation cover on less than ideal soils, and the maximum for safe maintenance. A slope of 25 percent (14 ft) is an optimal maximum for vegetative stability.

Soil Characteristics

Tillage techniques are influenced by the physical setting and in some cases by the availability of equipment. Harrows, disks and spike tooth cultivators are the most commonly used equipment. The tillage effectiveness was heavily influenced by the degree of soil compaction present prior to plot establishment. Better results were attained where original soils were not heavily compacted (i.e., less than 1.5 g/cc bulk density).

Gilman (1982) suggests that bulk density measurements should not exceed 1.3 g/cc in a coarser soil, in order that revegetation may succeed. Those recommended limits were exceeded on two of the test plot locations (Owen Sound, Toronto). In both cases bulk density values reached 1.7 g/cc. Those two sites were also noted to possess lower vegetative cover values in 1989 (24.3 percent and 17.4 percent, respectively). While compaction decreased in subsequent annual observations, it is likely that this factor influenced both the degree of tillage success and amount of initial vegetation catch.

Soil moisture also contributed to the success or failure of vegetative cover. While moisture levels vary across an individual site, landfill soils are generally drier than adjacent, nonlandfill soils (Gilman 1982). The driest soils encountered were those coarse grained landfill covers associated with the two northern sites, Sudbury and Sault Ste. Marie. Both performed poorly with vegetative cover values in 1989 of 14 and 4.6 percent, respectively. Within the Sudbury site, the test plot which received the erosion control blanket outperformed all other plots including the control. This would tend to support the importance of soil moisture and surface protection as influencing factors.

Plant Material Selection

Vegetation plays an important role in the stabilization of surface soils (Gray, 1974), especially on steep, erosion-prone side slopes. A dilemma is faced when establishing vegetation on landfills. Planting vegetation introduces the possibility of root penetration into the cap, opening up possible avenues for the addition of moisture, via infiltration. This can result in a subsequent increase in leachate and combustible gas production. Without a vegetative cover, however, the landfill cap soils would be exposed to the effects of heat and drying forces of erosion and the effects of the annual freeze-thaw cycle (Johnson 1986a). This would also pose a threat to the integrity of the cap.

In determining the best species mix for a landfill cap, the following characteristics were considered to be the most important. The species selected needed to be characterized by a relatively shallow root system, be tolerant to periodic drought and infertile conditions, and create a vegetative system that requires a minimal level of maintenance. Some consideration was also given to providing a mix of annual and perennial grasses and herbaceous species, tolerant of low oxygen conditions (such situations result from combustible gases produced from the decomposition of refuse in landfills).

Duell et al. (1986) studies these low oxygen "hotspots" at 15 locations in New Jersey, New York, New England, Washington, Oregon and Alabama and observed some effects of combustible gases on surface soils. Those effects included increased pH values; modified nutrient availability; and the presence of heavy metals in higher, although not toxic, concentrations.

Similarly MTRCA (1984) found that high levels of methane (CH_4) and carbon dioxide (CO_2) greatly reduced vegetative cover. Specifically, they found vegetation to be scattered, stunted and scorched in areas where CH_4 exceeded 40 percent and CO_2 exceeded 20 percent. That study also noted the variability of combustible gas levels over very short distances on the landfill cap. Other areas of sparse vegetative cover were determined to be caused by high soil temperatures (a situation in which plants require larger volumes of oxygen) and lower levels of combustible gases.

Other authors have examined the tolerance of tree and shrub species to combustible gases in landfill settings (Duell et al., 1986; Gilman et al., 1976; Gilman, 1982, Gilman et al., 1983; and MacKenzie, 1985). Findings have included the identification of some species with apparent tolerance of high levels of combustible gases (i.e., trees: Black Gum *Nyssa sylvatica*, Norway Spruce *Picea abies*, White Birch *Betula papyrifera*, Ginkgo *Ginkgo 2biloba*, Red Maple *Acer rubrum*; shrubs: Bayberry *Myrica pennsylvanica*, Japanese Yew *Taxus cuspidata*; and mosses: *Ceratodon purpureaus*. That moss species was observed during our field investigations in similar combustible gas zones.

Planting Techniques

Almost without exception, seed drilling and broadcast seeding produced greater vegetative cover than did hydroseeding. Others (Gilman et al., 1983; Johnson, 1986b) have also recommended that it is better to embed the seed in the soil rather than using hydroseeding, which applies the seed only to the soil surface. Our observations indicated that this approach will be more likely to result in germination and successful catch. Experience has demonstrated that the mulch protection afforded by hydroseeding with a seed and mulch mixture is frequently insufficient to protect seeds from washoff or disturbance by erosive forces of wind. The ability to drill or broadcast seed will be dictated by the degree of slope. On the most difficult slopes, our results revealed greater success with the more costly method of hydroseeding and the application of an erosion control blanket, than with hydroseeding with hydraulic mulch.

Drilling the seed at 90° angles at the flow of surface runoff creates a leaf and root line which helps reduce the erosive impact (Johnson, 1986b). Others have experimented successfully with corrugated roller-seeders to apply seed and composted sewage sludge.

CONCLUSIONS

The development of a cap that is effective in protecting the entombed refuse from moisture penetration relies upon the establishment of a vigorous vegetative community. The cap and vegetation when functioning properly will limit the production of leachate and combustible gases and will restrict their migration through the cap.

Our investigations have lead us to conclude that there are some key factors to be considered in optimizing vegetation establishment.

Foremost is the degree of slope. Without the creation of acceptable slopes (i.e., <33 percent), it is not practical to apply preferred revegetation techniques (i.e., seed drilling), which would optimize vegetation cover. Such situations are more prone to slope failure and to the displacement of planted seeds and seedlings by the erosive forces of wind and water.

Soil compaction also appeared to be a key constraint to planting success. Prior to planting, the bulk density of the soil should be less than approximately 1.3 g/cc and should be relatively homogeneous across the site. The use of a cover of topsoil over the impermeable cap will facilitate this. This creation of a suitable planting bed with only the low permeability cap material is not likely to be effective

To a lesser degree, soil fertility, pH, and the presence of combustible gases will influence revegetation success. On landfills typical of those observed during this study, fertility and pH were not limiting and gas and leachate problems were restricted to relatively minor pockets. In such a situation, those pockets could be addressed in a very specific manner, in consultation with the regulatory agencies.

The question of what to plant on the landfill soil surface is equally important. In very few situations are the cap and cover materials present at depths that could allow the growth of deeply rooted plant species without negatively affecting the low permeability nature of those materials. In most situations, a mix of species should be selected that will minimize the potential for cap interference. Shallow-rooted grasses should dominate the species mix to create a turf cover requiring relatively low maintenance. The precise mix of species will be dependent upon geographic region. Recommended species could include: Smooth Bromegrass, Creeping Red Fescue, Timothy, Ryegrasses, and Canada Bluegrass.

In parallel with this technical work program, a manual was produced (Gartner Lee Limited, 1990) which describes steps that should be undertaken in planning a complete revegetation program for landfills.

The reader is referred to this manual for further details regarding site characterization and planting program parameters that should be considered. In addition to planning a complete vegetation program, the manual also addresses how to remediate problem areas in an otherwise well established cover.

ACKNOWLEDGMENTS

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A PROTOCOL FOR EVALUATING PROPRIETARY SOLIDIFICATION/ STABILIZATION TECHNOLOGY USING LABORATORY LEACH DATA

By R. Mark Bricka and Norman R. Francingues[1]

Abstract: A protocol is described for comparing contaminant masses released during leaching of solidified/stabilized soils, sediments, and sludges. The protocol involves referenced leaching of untreated wastes and normalization of the leach data for solidified/stabilized waste to account for dilution of waste constituents by the solidification/stabilization (S/S) process additives. The protocol is applicable to a variety of laboratory leach tests including the Toxicity Characteristic Leach Procedure (TCLP), and other leach tests. The results of case studies are presented to show how reference leaching of untreated wastes and normalization of leach data for solidified/stabilized wastes can be used to document anticipated benefits of S/S processing. In several cases, S/S actually increases the mobility of specific contaminants.

INTRODUCTION

The U.S. Army Engineer Waterways Experiment Station (WES) has been involved with the research and development activities associated with the S/S of hazardous waste since the early 70's. During this time, WES has conducted studies or has been associated with programs which were funded by the U.S. Environmental Protection Agency (USEPA) which emphasized S/S technologies. These programs include, but are not limited to, the Superfund Innovative Technology Evaluation (SITE) program, treatability studies conducted for Comprehensive Environmental Response, Compensation and Liability Act (CERCLA) and Resource Conservation and Recovery Act (RCRA), the municipal ash incinerator program, and technical testing and support of the Best Demonstrated Available Technology (BDAT) treatability standards under the land disposal restrictions. A major short coming identified in conjunction with these efforts is that no standard protocol or guidelines exist for determining the treatability effectiveness of various S/S processes. This paper attempts to outline one such protocol developed and utilized by WES.

BACKGROUND

Solidification/Stabilization

S/S is a process that involves the mixing of a hazardous waste with a binder material to enhance the physical and chemical properties of the waste and to chemically bind any free liquids (Bricka et al., 1988a). Typically, the binder is a cement, pozzolan, or thermoplastic based process. Proprietary additives may also be added to enhance the inmobilization of various contaminants. Examples of such additives include sodium silicate, sulfides to render metals less soluble, and activated carbon and/or organophilic clays to adsorb organic contaminants.

In addition to differentiating S/S techniques by the binder and additives types, S/S techniques may also be differentiated by the processes which are used for its implementation. Generally S/S techniques can be grouped into three main categories by processes as listed in table 1. A more detailed discussion of binder/additives and S/S processes is provided in the USEPA "Handbook for Stabilization/Solidification of Hazardous Waste" (USEPA, 1986a).

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Table 1. - Types of solidification/stabilization processes.

Generic applications of nonproprietary binders/additives
Proprietary recipes using nonproprietary binders/additives
Proprietary combinations of nonproprietary and proprietary binders/additives

REGULATORY CRITERIA

Generally when S/S techniques are applied to wastes, the USEPA Paint Filter Test and TCLP is used to determine if the treated waste has measurable free liquids and exhibits toxicity characteristic, respectively (USEPA, 1986b). Free liquids are important to the potential waste disposer because waste containing free liquids has been banned from landfills (RCRA 3004 (m)) The TCLP results are important because this will determine if the waste should be classified as a hazardous waste due to toxicity and thus mandated disposal in a RCRA approved hazardous landfill. The only other regulatory testing procedure for S/S waste that the author is familiar with is the State of California's Waste Extraction Test (WET), which has a similar use as the TCLP but only used in California (California Administrative Code, 1985).

While these tests serve their intended purpose quite well (the classification of wastes for disposal purposes), they offer little in the way of determining the treatability effectiveness of the S/S process. In fact to date, there is little offered by the regulatory agencies in the way of guidelines, performance criteria, or standard engineering practice for the evaluating of S/S treatment effectiveness.

CRITICAL CRITERIA

As previously stated, S/S has two main purposes: to increase physical stability and chemically immobilize the contaminants. While physical stability is a concern for handling, transporting and placing the material in the landfill, little information is available regarding the long-term effects of physical stability and its interaction with contaminant mobility. Intuitively, it appears that a waste remaining in a monolith will contain the contaminants and shed water more effectively, but no long-term studies have been performed to verify this. While the author has strong opinions regarding physical property testing and evaluation, physical protocol testing will not be presented here. This paper will focus on the evaluation of chemical contaminant immobilization using leaching protocol.

AVAILABLE LEACHING PROTOCOL

There are over 30 different types of published leaching tests available for hazardous waste testing (Wastewater Technology Centre, 1990). Generally these leach tests can be classified into one of two main groups, batch type leach tests, and solid diffusion leach tests. Batch leach tests are generally performed for a shorter period of time than the solid diffusion tests and samples are usually ground or pulverized. In these tests, the solid waste is allowed to proceed towards conditions of equilibrium conditions with the extraction fluid. With solid diffusion tests, generally a monolithic sample of the waste is placed in the extraction fluid and the contaminants are leached from the intact monolithic block. In this type of extraction testing, kinetics are generally limited by the diffusion of the contaminant from the waste block to the surface of the block.

S/S PROTOCOL OUTLINE

The S/S protocol outlined by WES has five main steps as shown in table 2. In an effort to determine the effectiveness of various S/S processes, it is important the leach test results of the untreated waste material is compared with that of the S/S waste. With batch type leach tests, this is relatively a simple procedure, but with solid diffusion type tests, this may not be possible. The solid diffusion tests are performed on a monolithic block of waste, thus if the waste can not be compacted or molded into a monolith, it may not be possible to perform diffusion type of leach tests.

Table 2. - Solidification/stabilization protocol elements.

1. Set an objective for the leaching evaluation.
2. Select a leaching test for the evaluation which can be referenced if possible.
3. Perform the referenced leaching test.
4. Normalize the leach test resists.
5. Compare the untreated waste leaching results to the leaching results for the S/S waste and calculate the treatment results.

It is also important to note that if the proprietary S/S treatment process is being evaluated, then the leaching results must be compared to waste samples treated with generic processes. In this manner one is able to determine if the more expensive proprietary S/S treatment process offers advantages over the less expensive generic treatment processes.

Normalization of the leach data are also critical when evaluating the effectiveness of S/S processes. This is a problem for the TCLP as it is written. This is best illustrated in example 1. If the TCLP is normalized to the dry, raw waste concentration, the results can be compared and a treatment effectiveness can be calculated. If fact, the results for any leach test could be normalized and used to compute a treatment effectiveness. The equations for normalization are given in equations 1 to 4 below:

> **Example 1. - Hypothetical S/S Process Subjected to a TCLP Extraction.** - A waste is to be treated by a S/S process which utilizes a binder which is added to the waste 50% by weight. A TCLP test on the raw waste utilizes 100 g of raw waste. A TCLP test on the treated waste utilizes 100 g of treated waste but this 100 g contains 50 g of raw waste and 50 g binder. Thus, the treated material should leach less contaminant due to the fact it has only one-half the driving force of the raw untreated materials.

$$Cd_r = \frac{C_r}{W_r \times M_r} \qquad (1)$$

where:

Cd_r = TCLP contaminant mass/dry weight untreated waste, mg/g
C_r = untreated waste TCLP mass for the contaminant of interest, mg (calculated as: TCLP contaminant concentration, mg/λ x TCLP extraction solution volume, λ)
W_r = net weight waste extracted, g
M_r = solids content of the untreated waste used in the extraction expressed as a decimal

$$Cd_t = \frac{C_t}{W_t \times M_t \times B_t} \tag{2}$$

where

Cd_t = TCLP contaminant concentration/dry weight waste after S/S, mg/g
C_t = S/S waste TCLP mass for the contaminant of interest, mg (calculated as: TCLP contaminant concentration, mg/λ x TCLP extraction solution volume, λ)
W_t = weight of wet S/S waste, g
M_t = solids content of the S/S waste used in the extraction, expressed as a decimal
B_t = weight fraction of FBdI-Ash in stabilized/solidified waste calculated as follows:

$$B_t = \frac{\text{weight of waste}}{\text{weight of waste} + \text{weight of binder}} \tag{3}$$

$$PT = \frac{Cd_r - Cd_t}{Cd_r} \times 100 \tag{4}$$

where

PT = percent of contaminant not leached due to S/S

EXAMPLE OF DATA ANALYSIS ON VARIOUS CASE STUDIES

To illustrate the effectiveness of the protocol and normalization procedure, figures 1 to 3 are presented. Figure 1 presents the results of a serial-graded batch extraction procedure preformed on different raw and solidified/stabilized metal sludges (Bricka, 1988b). With the normalized data, it is easy to see that for the starch xanthate sludge, S/S actually increases the mobility of the chromium. Figure 2 presents normalized data which compare three S/S processes (Montgomery, 1988). With these normalized data, it is easy to see that for the waste containing high concentrations of phenol, the cement S/S process offers the most effective treatment. In figure 3 the normalized data are presented as percent treatment effectiveness for a KO48 and KO51 ash (Bricka, 1988a). With these data, it is clear that S/S increases the mobility of chrome but is quite effective in immobilizing the other metal contaminants.

CONCLUSIONS

The following conclusions can be stated regarding the proposed protocol:

1. Reference leaching and normalization are needed to document the anticipated benefits of S/S processing.

2. The protocol presented is versatile and can be used with any leach test.

3. Application of the protocol can minimize the risk of misapplication of S/S technology.

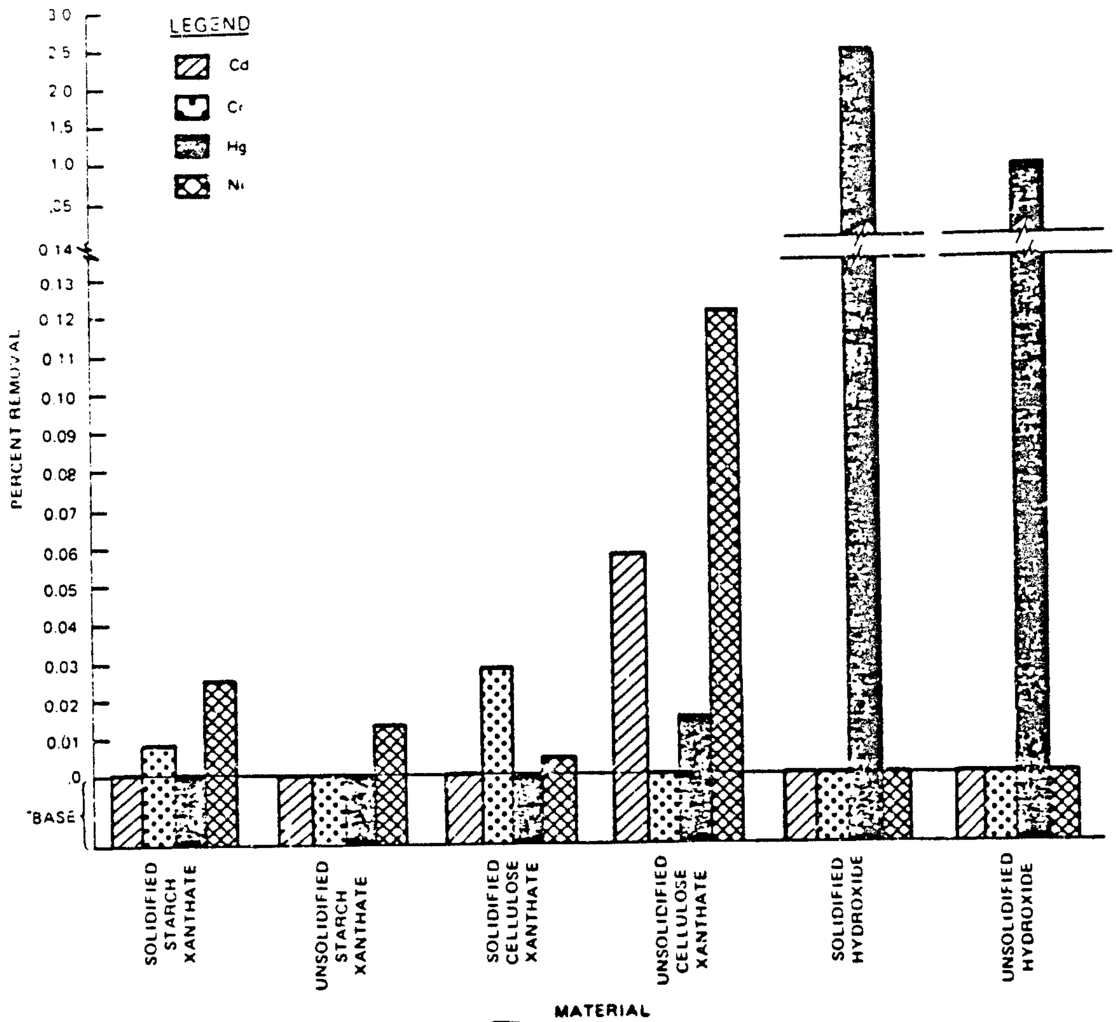

Figure 1. - Results of the serial graded batch extraction procedure at a L/S ratio of 20:1, presented as the percent of contaminant removed from the solid phase.

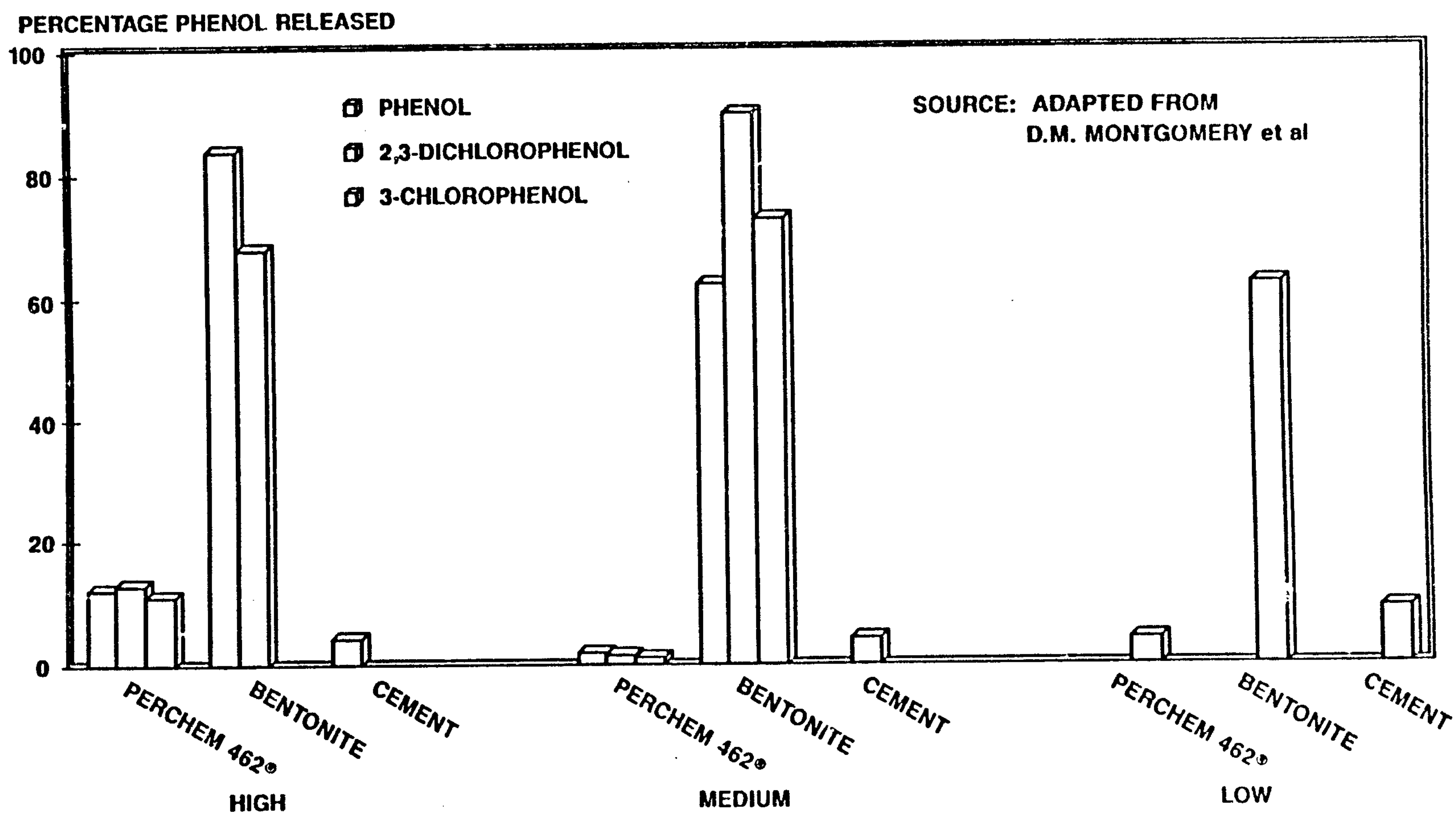

Figure 2. - Phenolic bentonite dynamic solid diffusion leach data

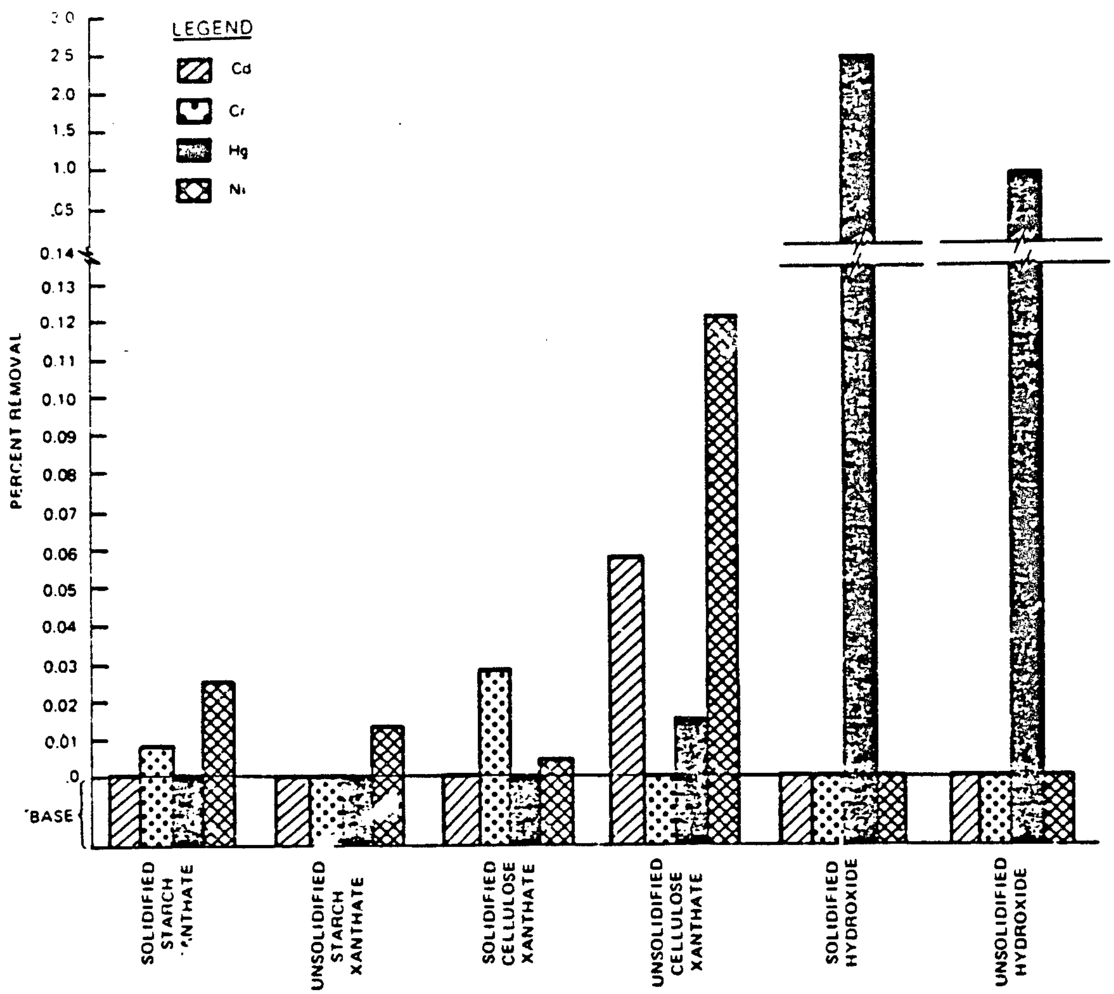

Figure 3. - Normalized TCLP data presented as the percent of contaminant immobilized in a single step TCLP extraction for A1, As, Cr, Mg, Se, Na, V, and Zn

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TREATABILITY OF METALS IN SOILS BY SOLIDIFICATION/STABILIZATION

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Abstract: Solidification and stabilization (S/S) techniques have been used for years to immobilize metals in contaminated soils. Whether applied *in situ* or to excavated material, binders and additives are often selected from the results of treatability tests. Such tests involve treating representative samples, curing the treated waste forms, and subjecting both raw and treated materials to one of several leaching tests to measure contaminant mobility. In a project sponsored by the U.S. Navy and the U.S. Environmental Protection Agency (U.S. EPA), Battelle Memorial Institute collected treatability test data in a data base listing contaminant concentration and matrix, binder type and ratio, and effects of S/S treatment on 18 metals. In this analysis, data for soils contaminated with RCRA-toxic elements and copper and zinc were examined to assess their treatability by S/S. Overall, cadmium and zinc were most susceptible to immobilization; arsenic and chromium were most problematic. Several generic and proprietary systems were successful in treating each metal. The degree of immobilization was associated with pH for several metals.

INTRODUCTION

Solidification/stabilization (S/S) technology has been used for several decades to immobilize contaminants in soils destined for land disposal (Conner, 1990). Solidification of contaminants in soils refers to the sequestering of contaminants in an impermeable and impervious matrix, thereby reducing contaminant mobility in the environment. In the context of this paper, stabilization refers to chemical conversion of the contaminant to a less soluble form, a less toxic form, or to a form bound to the treatment matrix. Most available technology involves both solidification and stabilization processes although the relative contribution of each is often unknown.

S/S technology is most often applied to hazardous and radioactive wastes dominated by metal contamination and to soils contaminated by metals. While the goal of S/S application in these cases is to prevent migration of toxic contaminants in the environment, the actual cleanup requirements for a given contaminated material are variable. Since S/S treatment does not reduce the amount of contaminant present, its efficacy is measured as the change in resistance to contaminant mobilization. Much less frequently, stabilization is judged on the degree of chemical conversion to a less toxic form (e.g., reduction of hexavalent chromium). Under the Resource Conservation and Recovery Act (RCRA) *et seq.*, the leachability of contaminants is measured by the Toxicity Characteristic Leaching Procedure (TCLP) (U.S. Code of Federal Regulations, 1990) to determine if the material exhibits the toxicity characteristic and to determine if treatment has succeeded in reducing leachable contaminant levels below the toxicity characteristic threshold. The percentage reduction in leachable metal required for any site depends on leachable metal concentration in the untreated material and the cleanup goal set for the site; in some cases, 50 percent or lower reductions are sufficient.

Before the TCLP was developed, the toxicity characteristic was measured by a leach test known as the Extraction Procedure (EP) (U.S. Environmental Protection Agency). States may require the use of an equally or more stringent test to determine characteristically toxic wastes, as California has done with their

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Waste Extraction Procedure (WET) (California Code). In addition to the TCLP test, other physical, chemical and leaching tests are sometimes applied to assess the short- and long-term effectiveness of S/S treatment processes (U.S. Environmental Protection Agency, 1989).

Many public domain and proprietary formulations are available for S/S treatment of contaminant metals (Conner, 1990). Generic binder materials include cement, lime/fly ash and other pozzolanic mixtures. Most binder systems probably immobilize metals by a combination of solidification (encapsulation; physical restriction of contaminant mobility) and stabilization (alteration of the contaminant's chemical form or binding). A treatability study is usually performed prior to applying the technology to any waste or contaminated soil (U.S. Environmental Protection Agency). The study generally includes testing of several treatment formulations at multiple waste-binder ratios and water addition rates. Degree of immobilization, compliance with regulatory or remedial targets, volume increase, and cost are typically evaluated to select a particular treatment.

Treatability tests can be fairly expensive and time-consuming. It would be preferable to have a method to predict the performance of binder systems on various wastes and contaminants, so that laboratory studies can be directed toward formulations having a high probability of success. Toward this end, the U.S. Navy and the U.S. EPA sponsored Battelle Memorial Institute to construct a data base of existing treatability data and analyze it for correlations across waste properties, binder parameters, and treatment performance (Battelle Memorial Institute, 1991).

SOLIDIFICATION/STABILIZATION TREATABILITY DATA BASE

The Battelle data base is comprised of treatability data on 18 metals of environmental interest in both soils and wastes. The data were selected from published and unpublished studies, and entered in a spreadsheet containing the fields listed in Table 1.

Only contaminated soils records were selected for this analysis. All elements regulated by the toxicity characteristic under RCRA (arsenic, barium, cadmium, chromium, lead, mercury, selenium, and silver) were selected, as well as copper and zinc. The latter two metals were included because they are frequently included in remediation plans at Superfund sites. Coverage of the selected elements in the data base was uneven, with little data available for barium, silver, selenium and mercury.

Data base records were omitted from this analysis if leachable metal concentration prior to treatment was not reported or if a nonstandard leaching test was used. Data were retained if leaching tests were based on the Extraction Procedure (EP), the Toxicity Characteristic Leaching Procedure (TCLP), the Multiple Extraction Procedure (MEP), the California Waste Extraction Test (WET), or the Monofilled Waste Extraction Procedure (MWEP). At present, only the TCLP protocol has regulatory meaning at the Federal level. However, the other tests are commonly used to evaluate S/S treatment processes.

For the purpose of this paper, a leachate produced by any protocol was considered to be characteristically hazardous if the leachate metal concentration exceeded the limit set for the TCLP protocol. This is not correct for protocols other than TCLP, which may be more or less rigorous in leaching of contaminant metals; however, it provides one way to compare data across the various common tests. Another reference point cited throughout the text is the potential of a soil to exceed the TCLP limit value for a given metal. For the TCLP test, conducted at a ratio of 20 Ml leachant to 1 g soil, the leachate limit value multiplied by 20 yields the maximum soil concentration in mg/kg that can not fail the TCLP test. That is, a regulatory level of 5 mg/L leachable metal can not be exceeded if a soil contains 100 mg/kg of the metal or less, *even if the metal completely dissolves under the leaching conditions.* Any soil containing the metal at a higher concentration has the potential to be hazardous by the toxicity characteristic.

Table 1. - Structure of records in S/S treatability data base.

FIELD NAME	DESCRIPTION
Reference Number	Code to track an individual record in data subsets.
Metal	Chemical symbol for the element.
Waste	Brief description of waste matrix (soil, sludge, ash).
Leach Test	Procedure used to determine leachable metal concentration.
Leachate pH, un.	Final pH of the leachate, untreated waste.
Leachate conc., un.	Metal concentration of the leachate, untreated waste.
Leachate pH, tr.	pH of the leachate, treated waste.
Leachate conc., tr.	Metal concentration of the leachate, treated waste.
% Reduction	% reduction in leachable metals after treatment.
Total conc., un.	Total metal concentration in waste, untreated.
Total conc., tr.	Total metal concentration in waste/binder, treated.
Leachable/Total Metal, un.	Ratio of leachable metal concentration to total metal concentration in the untreated waste.
Binder	Description of binders and additives used in treatment.
Binder:Waste Ratio	Relative amounts of treatment materials added to the waste.
Reference	Citation of data source.
Specific Waste	Specific description of waste type from source report.

Records were omitted if the leachable metal concentration in the untreated soil was below the drinking water maximum contaminant level (MCL) for that metal[2]. Values lower than the MCL occurred when the leachate was analyzed for all regulated metals even though the soil was contaminated with only a subset of those metals. Throughout this report, the MCL is cited as a reference value, although the MCL is only properly applied to water samples, and not to leachates produced in the laboratory from contaminated soils.

Because the data base contains data from many different treatability studies, the records are not suitable for statistical analysis. For example, one study may include only one treatment of each soil while another may include 10 or more. A numerical analysis would be biased toward those samples that are represented by multiple records. Therefore, simple statistics (e.g., range) and graphical analyses were used to interpret the data. This gross interpretation should serve as guidance for the design of future studies intended to fully evaluate the role of various factors in controlling the mobility of metals from S/S-treated soils.

RESULTS

Results are presented first for those elements represented by a large number of records in the data base. Generalizations drawn for these elements should be stronger than those made for metals represented by a small number of records.

Arsenic (As)

Arsenic data in the data base represents 10 soils, some at multiple contaminant levels. Total As concentrations ranged from 18 to 12 200 mg/kg in those untreated soils for which the parameter was measured; all but one contained less than 2000 mg/kg. Leachable As concentrations ranged from 0.16 to 575 mg/L, or 2.0 to 38 percent of the total As present in the soil. Leachable As tended to increase with increasing total As (Fig. 1). Of the 45 records reporting both total and leachable As for the raw soil, 78 percent contained enough As to fail the RCRA limit on complete dissolution and 69 percent actually produced leachate concentrations exceeding that level.

Leachable As concentrations in the treated waste ranged from 0.002 to 580 mg/L. After treatment, 14 percent of the leachates met the drinking water MCL (0.05 mg/L) and 54 percent met the RCRA toxicity limit of 5 mg/L. Reductions in leachable concentrations after treatment ranged from -85 percent to >98 percent. In 7 of 56 individual samples, treatment resulted in an increase (negative reduction) in leachable As. Three of these cases are not reliable because the leachate concentration from untreated soil was only 0.09 mg/L. Small analytical errors or sample variations could explain the apparent increase. In contrast, the other cases occurred when untreated soil produced leachates containing 16 and 420 mg/L As. Although data are not available to investigate the reasons for negative treatment effects, changes in pH and oxidation state that may be induced by treatment can increase as well as decrease soluble As concentrations. Treatment agents may also contribute leachable arsenic. At least 90 percent reduction in leachable As was achieved in only 29 percent of the treatability samples. This finding contrasts with several other elements, for which S/S treatment more readily achieved more than 90 percent reduction in leachability. Of 38 samples that exceeded the RCRA toxicity limit prior to treatment, 26 failed to meet the limit after treatment although the leachable levels typically decreased.

[2] The MCLs used are those cited in 40CFR141.11, which are no longer current for barium, cadmium, chromium, or selenium.

Except for some negative treatment effects mostly observed at low total As concentration, treatment performance did not appear to be related to either total As concentration or leachable As prior to treatment. A variety of binders, including both generic and proprietary formulations, achieved maximum reductions in leachable As of more than 90 percent. Four soils were not successfully treated by any of the S/S formulations tested.

Cadmium (Cd)

The data base contains 95 records for cadmium obtained from ten treatability studies. Reported total Cd concentrations ranged from 2.3 to 10 000 mg/kg; 9 records omitted total Cd. Eighty percent of the samples contained enough Cd to potentially fail the RCRA toxicity test. Prior to treatment, leachates produced by various protocols contained 0.01 to 338 mg/L Cd, with some samples yielding virtually 100 percent of the total Cd in leachable form. The leachates failed to meet the RCRA limit of 1.0 mg/L in 67 percent of the cases. As observed for arsenic (Fig. 1), leachable Cd concentrations tended to increase with increasing total Cd.

Cd immobilization was generally achieved by S/S treatment: 86 percent of the treated samples in the data base met the TCLP limit of 1.0 mg/L leachable Cd. Successful S/S treatment systems included cement, kiln dust, lime/fly ash and several proprietary formulations. Despite good overall performance, a few negative treatment effects were observed. In 6 percent of the samples, leachable Cd increased after treatment; however, all of these cases had no more than 0.1 mg/L leachable Cd prior to treatment. The negative treatment effect may reflect analytical error or sampling variability, factors that are more significant at low concentration.

Chromium (Cr)

Limited data were available for Cr immobilization. The data base includes 8 studies, but most of the records (42 percent) are from one study that only evaluated the leachability of Cr (VI) from a heavily contaminated soil. Further, four of the studies were conducted using raw soils that were not characteristically hazardous for Cr. For these reasons, the following summary should not be assumed to apply to Cr-contaminated soils in general.

Raw soils contained total Cr at concentrations of 510 to 72 000 mg/kg; only one study failed to report total Cr. Leachable Cr ranged from 0.06 to 441 mg/L in the raw soils, or 0.4 to 63 percent of the total Cr present.

S/S treatment reduced leachable Cr concentrations in most cases. Reductions ranged from -100 to >99 percent overall and from 34 to >99 for those samples that were characteristically hazardous prior to treatment. Again, the negative treatment effects were confined to raw soils containing low levels of leachable Cr.

Copper (Cu)

The data base contains 41 records on five studies involving Cu-contaminated soils. Total copper, reported for 29 of those records, ranged from 170 to 110 000 mg/kg. Leachable Cu concentrations for all records ranged from 1.1 to 140 mg/L, representing 1.3 to 68 percent of the total Cu for those cases where total Cu was reported. Cu is not a toxicity characteristic metal, so there is no maximum TCLP value for it. In contrast to some of the other metals, Cu leachability did not show a strong tendency to increase with increasing total Cu (Fig. 2).

Treatment generally immobilized Cu, with -925 to >99.9 reductions in leachable levels reported after treatment. The three negative treatment effects were all reported for a single sample that was initially low in leachable Cu, but was not near the typical detection limit (~10 ppb). Treatment materials (cement, fly ash, kiln dust) may have contributed leachable Cu, but data for specimens made only from the treatment materials were not reported. Forty-nine percent of the samples exhibited at least a 90 percent reduction in leachable Cu following treatment; 39 percent of the treated samples produced leachate Cu concentrations below the drinking water MCL (1 mg/L).

One study included in the data base reported leachate pH for raw and treated soils (Fig. 3). These data suggest that immobilization of leachable Cu was achieved by providing an alkaline environment for precipitation of copper hydroxide. Most of the contaminant metals discussed in this paper are susceptible to forming sparingly soluble hydroxides and oxides.

Lead (Pb)

Nineteen treatability studies comprised of 168 records for Pb are reported in the data base. Untreated soils contained 86 to 140 000 mg/kg Pb concentrations (82 percent of the records reported total Pb). Leachable Pb ranged from 0.056 to 2300 mg/L, or 0.8 to 100 percent of the total Pb in the sample. Pb in the leachate exceeded the RCRA limit of 5 mg/L in 77 percent of the untreated soils. Like As and Cd, Pb leachate concentrations tended to increase with increasing total metal concentration (Fig. 4). For lead, as for most other metals, the increase was not linear and the fraction of metal that leached declined with increasing total metal concentration (Fig. 5). Thus, the most heavily contaminated sample produced the highest leachate Pb concentration but the leached Pb in that sample represented only a small fraction of the total Pb.

S/S treatment was generally quite successful for Pb. Posttreatment leachate Pb concentrations ranged from <0.002 to 330 mg/L, with a maximum reduction in leachability exceeding 99.9 percent. Only 26 percent of the posttreatment samples exceeded the RCRA limit for Pb, in contrast to 77 percent of the pretreatment samples. Sixty-five percent of the treated samples exhibited leachability reductions of at least 90 percent. Successful treatment was achieved by a variety of generic and proprietary S/S formulations. Like most of the readily treatable metals, negative treatment effects were concentrated among the lower values of leachable Pb in the raw soils. High reductions in leachability were observed throughout the range of contaminant concentrations.

pH data were reported for raw and treated soils in four studies (Fig. 6). Like Cu, the leachability of Pb appeared to be controlled to a large extent by pH. Untreated soils, in general, showed increasing leachable Pb with decreasing pH. This behavior is typical of Pb^{2+} in solution and results from the formation of hydroxyl complexes that exhibit minimum solubility at alkaline pH. In contrast to the general trend, a few untreated soils showed low concentrations of leachable Pb at acidic pH. The two data points below pH 5 were from one site; they showed very low leachable Pb concentrations before treatment and no change after treatment. This suggests that the Pb contamination is in an insoluble form, perhaps as elemental Pb. Four of the data points between pH 5 and 6.2 that exhibited low leachate concentrations of Pb had low total Pb concentrations. Thus, virtually all the Pb in the sample was dissolved by the leaching procedure, without reaching the solubility limit.

Posttreatment Pb-pH data showed low solubility of Pb over the pH range of 8.7 to 13. This is a wider solubility minimum than expected for the $Pb-H_2O$ system. However, the highest leachate pH values were associated with treatment formulations that included silicates and sulfides, which may form Pb compounds that are more stable in alkaline conditions than oxides or hydroxides.

Zinc (Zn)

Eight studies in the data base included data for Zn. Total metal concentrations, reported for some or all samples in 6 of the studies ranged from 220 to 233 000 mg/kg. Leachable zinc concentrations were 6.3 to 12 500 mg/L in the raw soils, or up to 100 percent of the total zinc. As with most of the other metals, Zn leachability tended to increase with increasing total Zn concentration.

Following S/S treatment, leachable Zn decreased a minimum of 39.1 percent and a maximum higher than 99.9 percent. The excellent treatability of Zn contrasts with that of other elements, which always included cases where treatment had an apparent negative effect on leachability. For Zn, 85 percent of the records showed at least 90 percent reduction in leachability following treatment. The success of treatment may be related to the pH increase induced by treatment. The raw samples, showing high leachable Zn, all occurred at pH < 6.1; all the treated samples, showing low leachable Zn, produced leachates at pH > 8. There were no data available to confirm the expected amphoteric behavior that would lead to higher solubility at highly alkaline pH (~12).

Silver (Ag)

The data base contains records on five soils for which silver was measured. In three cases, the TCLP-leachable Ag before treatment was below the drinking water MCL of 0.05 mg/L and remained below the MCL after treatment. In the other two cases, where leachable Ag was only measured after treatment, the posttreatment leachates complied with the drinking water standard.

Barium (Ba)

Seven soil records are in the data base, covering soils at three Ba concentrations from two studies. Total Ba concentrations in the untreated soils were too low to be characteristically hazardous; however, 100 percent dissolution of Ba in each soil in a leaching test could exceed the drinking water MCL (1.0 mg/L). In fact, the TCLP-leachable Ba levels for untreated soils ranged from 1.0 to 1.6 mg/L.

After treatment, TCLP-leachable Ba concentrations ranged from 0.8 to 2 mg/L, reflecting an apparent increase in leachability in five of seven individual samples. This apparent increase could have resulted from analytical error, from a contribution by the treatment agents, or from an actual increase in leachability of Ba from the contaminated soils. Only the last factor represents a real failure of the S/S process, while the other two factors confound analysis of treatment effects. For most of the metals in the data base, apparent negative treatment effects are predominantly found at the low end of the contaminant concentration range, suggesting that sample and analytical variation are primarily responsible for observed increases in contaminant leachability. For some metals, such as barium, a contribution from treatment materials, such as fly ash, is possible.

Mercury (Hg)

The data base only contains one record for Hg in soil. The untreated leachate concentration was well below the drinking water MCL of 0.002 mg/L.

Selenium (Se)

The data base includes selenium data for one soil at two contaminant concentrations. Total Se concentrations in the untreated soil were 7.4 and 12 mg/kg, below the minimum concentration that could potentially be hazardous by the toxicity characteristic (20 mg/kg). Leachable Se, measured by the WET protocol, was 18 and 49 percent of total Se or 0.2 and 0.4 mg/L. These values are higher than the drinking water MCL of 0.01 mg/L.

The soils were treated with conventional binders: cement, kiln dust, and lime/fly ash. Posttreatment Se concentrations in the WET leachate ranged from 0.07 to 0.3 mg/L, indicating reductions in leachable Se of 5 to 81 percent. While these values are below the RCRA characteristic limit, they all exceed the drinking water MCL.

CONCLUSIONS

The data base on S/S treatability of metals in soils provides a basic description of the capability of the technology for immobilization of six metals: arsenic, cadmium, chromium, copper, lead and zinc. Each metal evaluated was treated successfully by at least one, and usually several, treatment formulations. Nonetheless, there were certain contaminated soils that could not be treated to meet the toxicity hazard limit for each of the four RCRA-toxic contaminants. These cases were usually highly contaminated with readily leachable forms of the element.

Most of the contaminants, when present at moderate concentrations, were highly amenable to immobilization by RCRA standards. Reductions in leachable concentrations met or exceeded 90 percent for at least 80 percent of the samples contaminated with cadmium or zinc. In contrast, reductions in leachable concentrations met or exceeded 90 percent for only 30 percent and 17 percent of the samples contaminated with arsenic and chromium, respectively. Treatability of copper and lead showed intermediate success, with 90 percent or better reduction in leachability in 49 and 64 percent of the samples, respectively. There is more uncertainty in these treatability comparisons than would be the case if the data were all obtained from a single study covering the same number of samples and concentration range for each contaminant.

The data base cannot be used for a fair comparison of the performance of different binder systems because of uneven representation of the various binders. However, the data clearly show that common generic binders - cement, lime/fly ash, and kiln dust - can achieve high reductions in leachable concentrations of most of the contaminant metals (Fig. 7). Chromium was less treatable than the other elements in this comparison.

Data on leachate pH is relatively sparse in the data base because the standard protocols do not require its measurement at the conclusion of leaching. However, solution pH should be a strong controlling factor for leachate concentrations of those metals present in the treated soil as hydroxides or free cations. Most of the available pH data illustrates minimum leachability at pH values between 8 and 11 for copper, lead and zinc. More data are needed to refine the concentration-pH curves and to evaluate pH effects for other contaminants.

Additions to the data base are planned as additional treatability studies become available. Continuing analysis should improve our ability to select appropriate treatments for contaminated soils from knowledge of contaminant levels, matrix and binder performance history.

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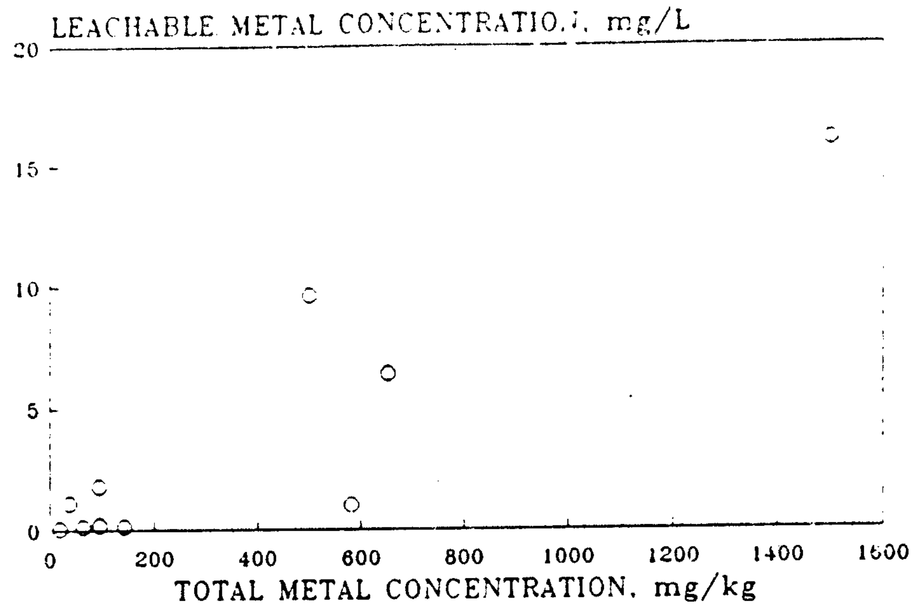

Figure 1. - Leachable arsenic concentration versus total arsenic concentration in untreated soils.

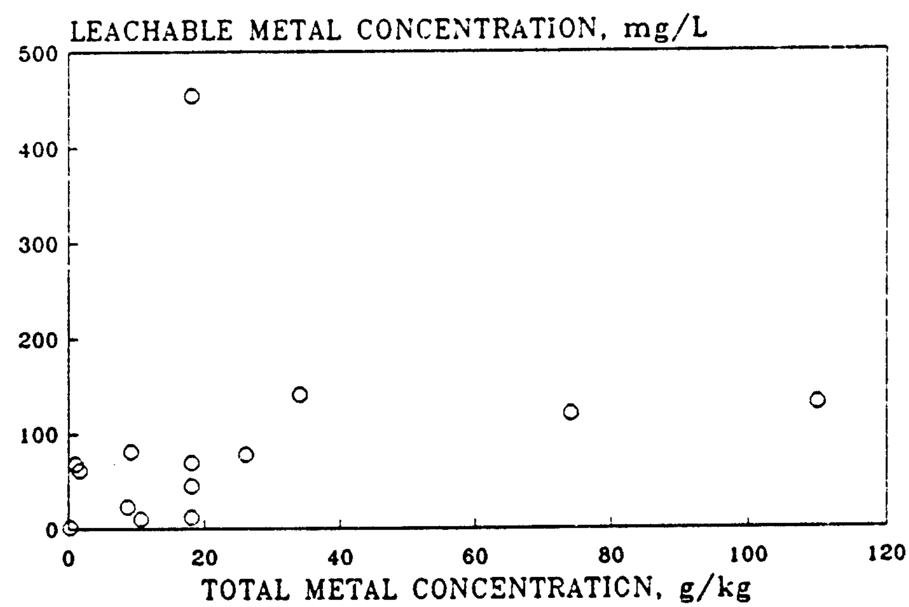

Figure 2. - Leachable copper concentration versus total copper concentration in untreated soils.

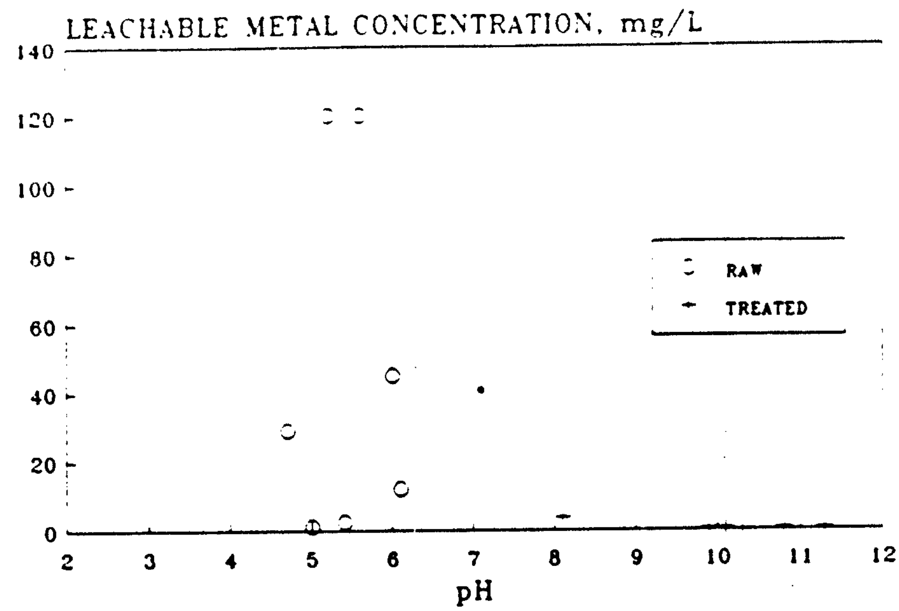

Figure 3. - Leachable copper concentration versus pH for untreated and treated soils.

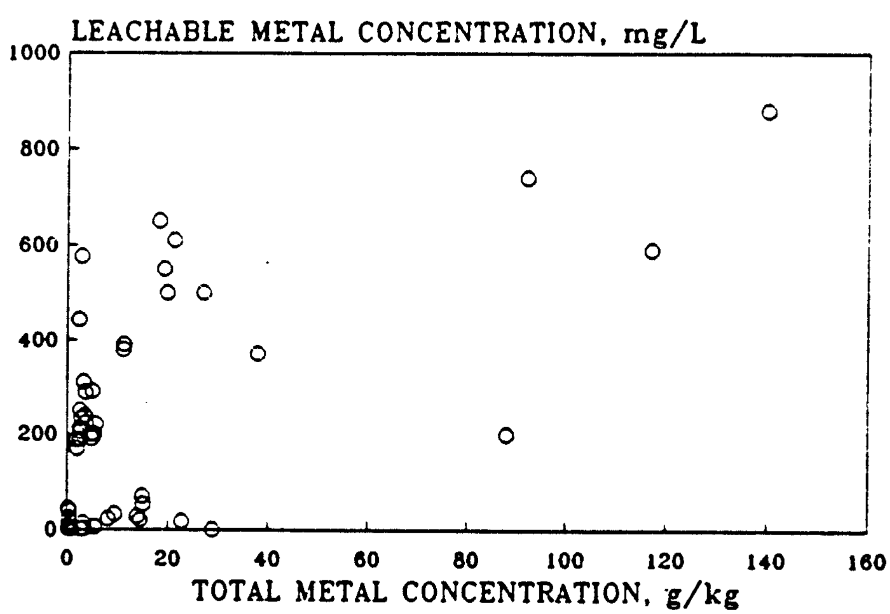

Figure 4. - Leachable lead concentration versus total lead concentration for untreated soils.

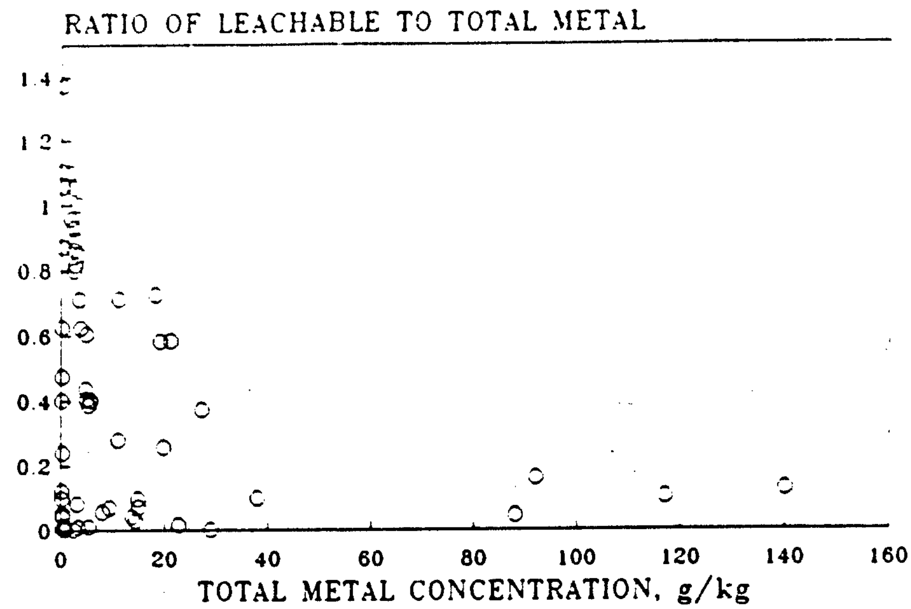

Figure 5. - Ratio of leachable lead to total lead versus total lead concentration for untreated soils.

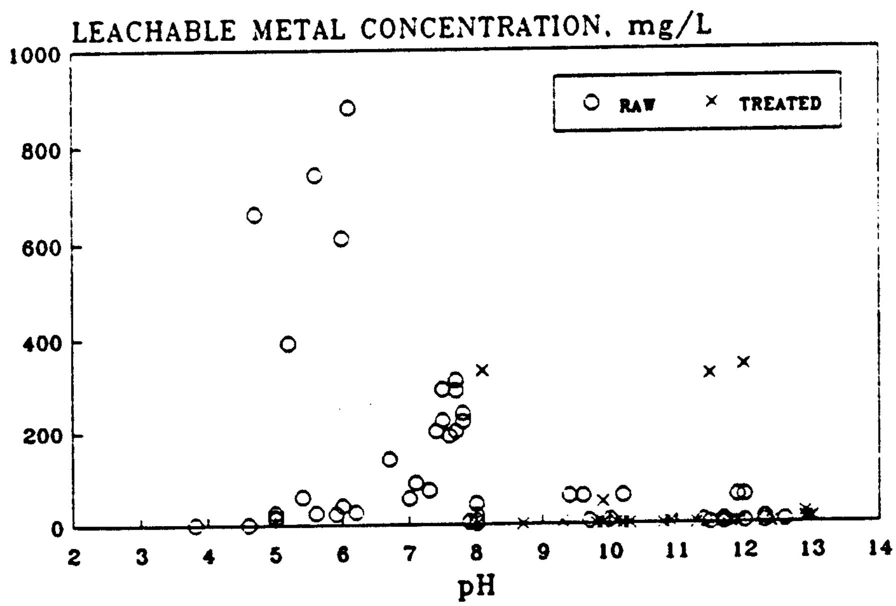

Figure 6. - Leachable lead concentration versus pH for untreated and treated soils.

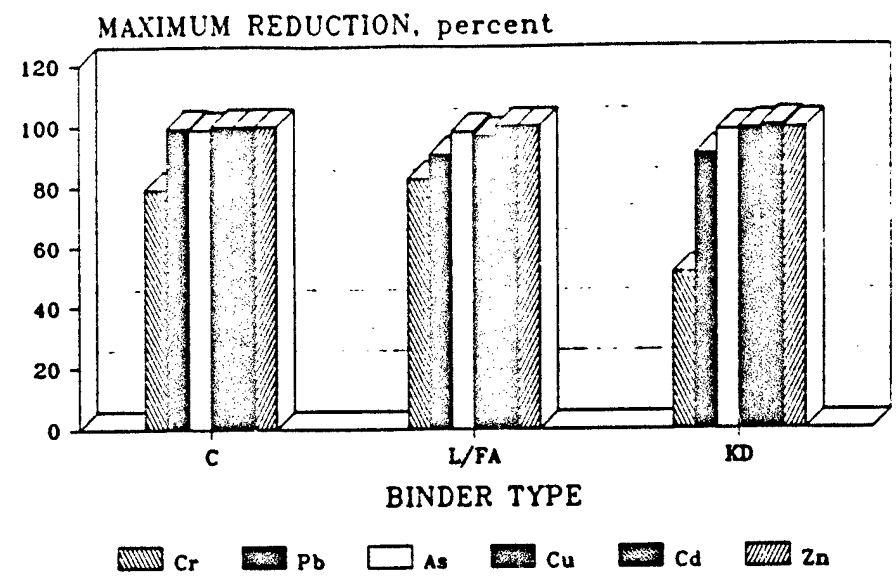

Figure 7. - Maximum reduction in leachable metal concentration for six metals and three generic binders. C = cement; L/FA = lime/fly ash; KD = kiln dust.

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