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THE CIVIL ENGINEERING MIND — NATURE AND NURTURE

by
Ralph B. Peck*

(First Sam J. Mathis Memorial Lecture, Massachusetts Institute of Technology, November 15, 1973)

Civil engineering is not an intellectual exercise although it has intellectual content. The civil engineer is action-oriented. He creates bridges to cross rivers; he collects, transports, purifies, and delivers water to drink and renders our waste products harmless. He creates facilities for moving people and goods — highways, subways, railways, airports and pipelines. He makes society as we know it possible. Increasingly he is recognized as the man to save the environment, not by stopping civilization but by accommodating it to nature.

Sam Mathis was such a man, a man of action. He created facilities to bring us energy from the oil fields of Venezuela and the gas fields of Libya. It is most fitting to name this lectureship in his memory and I am honored indeed to be chosen to bring you this first Sam Mathis Lecture. I hope my comments would have met with his approval.

Increasingly, we hear that the civil engineer should have more to say about government policy at all levels, that he should be heard, that he should not passively carry out technical assignments but should actively promote and support specific causes and courses of action. He should be advocate, politician, humanitarian. He should be in the forefront, fighting battles for the consumer, for conservation, for environment, and against pollution. He should, in short, make his influence felt.

I should like now to look at the engineer, man or woman, consider how he can best be trained, how he can best make his contributions, what responsibilities he should or can take, and how he can best serve society.

Ralph H. Burke

To set the stage, I should like to look at the accomplishments of one engineer with whom it was my good fortune to work for many of my younger years. This man quite possibly contributed more to the physical well-being of Chicagoans than any other engineer and perhaps as much as any person regardless of his profession. I doubt if you would know his name; it was not widely recognized by most Chicagoans even in his life-

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time. Ralph H. Burke died in 1956. I bring him to you as an example of what the civil engineer can accomplish in public service, both as a public employee and in private practice. He was a man of great technical ability, innovative, never satisfied with the past, a skilled negotiator, not a politician but one who had remarkable influence among politicians, a man to whom, indeed, politicians turned to solve many of their city's most pressing problems.*

Accomplishments

In 1934 he became Chief Engineer of the Chicago Park District, concerned with recreational facilities for Chicago's many neighborhoods. At the urgent request of Mayor Kelly in 1938 he was made available, while still holding that position, to serve as the Chief Engineer of Subways and Traction for the City of Chicago during the period of construction of the Initial System of Chicago Subways. The idea of subways for Chicago was not new; there had been many tentative plans for at least a generation, but no plan ever came to action. Suddenly the Public Works Administration, created to reduce unemployment during the Great Depression, approved an application by the City of Chicago for a grant to build its subways, but with the provision that construction be started within an incredibly short period of a few weeks. There were no plans, no specifications, not even an agreed-upon route. Little wonder that Mayor Kelly felt the urgency. Ralph Burke accepted the assignment, persuaded many disparate parties to agree on the subway routes, assembled a staff for design, surveying and construction, issued contract drawings and specifications for the first section of the subway, and had the work under contract within the deadline.

As we shall see, he was no stranger to tunneling, and he recognized the dangers and dislocations associated with tunneling in the soft clays beneath downtown Chicago. When he attended a meeting of the Western Society of Engineers at which Karl Terzaghi spoke on that subject, he engaged Terzaghi forthwith and agreed to Terzaghi's requirements of a laboratory staffed by suitable personnel under the direction of a man of Terzaghi's choosing. I was fortunate enough to be that choice.

Within a week of Mr. Burke's engaging Terzaghi, I arrived in Mr. Burke's office where he was impatiently awaiting me. As I had been told to come to the office immediately on reaching town, Mrs. Peck and I turned up on Friday morning directly from the railroad station. Mr. Burke graciously introduced himself to both of us and promptly dispatched me to the site of the first construction contract, where test borings were being made by a special crew under the supervision of a foreman imported from Boston

*I am indebted to J. L. Donoghue, President, Ralph H. Burke, Inc. for furnishing a number of the details about Mr. Burke's career.

because he knew how to take 2-in. Shelby tube samples. The problem seemed to be that Chicago clays did not want to stay in the Shelby tubes if treated by the procedures that were successful in Boston. Mrs. Peck waited while I dashed out to see the foreman, who certainly knew more about making Shelby tube borings than I did. Happily, he and I were able to work out some tricks that increased the recovery, and Mrs. Peck and I were granted the remainder of the weekend to rent an apartment and settle down.

The subway organization was still being created when I arrived. Mindful of his promise to Terzaghi, Mr. Burke enthusiastically assigned to the soil laboratory every inexperienced job applicant who had just received a Master's degree in civil engineering. We were truly an inexperienced group — all of us — but the nucleus of eight young men whom Mr. Burke selected worked together with great effectiveness. They went on, I am happy to say, in almost every instance to distinguished careers.

Although Mr. Burke liked to joke about his collection of inexperienced MS's, he was nevertheless entirely serious about the benefits to be obtained from soil mechanics as it began to develop before his eyes under Terzaghi's direction. When Terzaghi suggested a test section on one of the liner plate tunnel contracts, Mr. Burke instructed us to prepare a design on the basis of which he could negotiate with the contractor. Those of you who are familiar with Terzaghi's paper on Liner-Plate Tunnels on the Chicago Subway may recall the S-6 test section, the end result of Terzaghi's suggestion. In this day of expensive instrumentation, it is almost inconceivable that the added contract cost for the construction of the test section, not counting our own efforts, was about $6,000. Even at these bargain rates, the administrator of the federal funds, who was the overseer for the U. S. government on the project, refused to approve such an unprofitable expenditure. Nothing daunted, Mr. Burke persuaded the Chicago City Council that the test section would be a good investment, and the Council voted to finance it from City funds. That the investment paid off was demonstrated only a few years later when the lining of the Congress Street extension was designed to take advantage of the findings, at a substantial saving in cost.

After the first two years of subway construction, when the major problems had been faced and solved, Mr. Burke returned to the Chicago Park District in 1940. He stayed there as Chief Engineer until 1946, when at the age of 64 he left the Park District to establish a consulting engineering firm. One of his first assignments was the development of a major airport for the City of Chicago, the airport that today you all know as O'Hare. For many years it has been the busiest airport in the world and one of the most efficient. Mr. Burke had never had anything to do with airports before, but now he was responsible for what he felt would need to be the world's largest. He did not believe in thinking small. He considered a number of sites, including the possibility of one in Lake Michigan. After many comparisons and much discussion with all parties concerned, including the airlines, he settled on what was then known as the Chicago Orchard Site (did you ever wonder where the symbol ORD originated?), and developed a master plan that required the purchase of some 6,700 acres of property. At the time, jets had not yet entered civilian aviation in the United States. The jet age was on the horizon but nobody had any notion of its enormous impact on our ways of travel. Mr. Burke foresaw that the operation of great numbers of large jet-powered planes was inevitable within the lifetime of the proposed airport, and made his plans accordingly. Each time I visited him, for I was responsible for the initial soil studies of the area, I would see dozens of new master plans being laid out and evaluated. Many were unlike any yet in operation, but his imaginative mind led him to investigate all possibilities. The master plan of O'Hare field today is not Mr. Burke's, because he died in the early stages of its construction, but the preeminence and continued effectiveness of O'Hare can be attributed largely to his insistence on sound planning and the acquisition of enough property to permit the necessary development.

Mr. Burke was not only capable technically, but was also a patient and successful negotiator. At O'Hare, two major railroads crossed the site and a third skirted the edge. The two railroads on the site had, of course, to be relocated around the future airfield. Relocation of this sort is usually compensated by determining the increased length of trackage, by estimating future traffic, by determining the cost of the additional ton-miles associated with the relocation, and by capitalizing this cost. Such computation for the two railroads at O'Hare indicated a cost of several million dollars. Furthermore, the railroads were not overly cooperative in assisting the development of a competing form of transportation. Mr. Burke called in representatives of the two railroads and pointed out that air freight would become a major way of moving goods, that there would necessarily be an interface between air freight and rail freight, and that O'Hare would certainly become an air freight center. He foresaw additional business generated for the two railroads because of their favorable location and he declared that the city was willing to grant them an exclusive right for this traffic if they would be willing to be relocated without compensation. He went on to say that if they were unwilling to enter into such an agreement, the City would be inclined to give exclusive rights to the third railroad which merely skirted the property. The railroads wasted little time in accepting the proposal.

The relocated railroads had to cross a peat bog with a depth up to about twenty-five feet for a distance of about one-half mile. The agreement with the railroads required all unstable material beneath the right-of-way to be removed and replaced. Unfortunately, removal of the peat for the two-hundred-foot right-of-way would have been very expensive, and extremely flat slopes would have been required adjacent to the excavation for stability. The flat slopes would have encroached on a major highway and a high pressure gas line. We concluded that the peat could be stabilized by a combination of sand drains and surcharging, a procedure that up to that time had been used only to a very limited extent. The railroads, who had anticipated that the peat would be removed, raised objections. Mr. Burke blandly pointed out that upon completion of the stabilization there would be no unstable material beneath the track and, therefore, there would be nothing according to the agreement that needed to be removed. Although the consolidation due to the surcharging took about a year longer than planned, the operation was nevertheless successful. I smile every time I land at O'Hare and look out at the parallel railroad tracks on the peat bog.

Somewhat later he undertook the Grant Park Underground Garage, on its completion the largest parking garage in the world. Quite typically, Mr. Burke had never before designed a parking structure. Anxious as always to keep costs to a minimum, Mr. Burke inquired whether the two-level garage could be floated without any deep foundation although its base would rest on the softest part of the Chicago clays. The entire structure, a single floating raft three hundred by twelve hundred feet in plan, has served its purpose admirably. It was so successful that the Park District at a later time built a similar underground structure with three levels beneath Grant Park and Michigan Avenue several blocks to the south. It, too is supported by a floating foundation.

I have told you only some of Ralph Burke's major accomplishments, but I think they will demonstrate to you that this one engineer left his mark on many aspects of the City of Chicago. He was indeed an indispensable man. Notwithstanding that he was a lifelong Republican and a resident of Evanston, while Chicago was controlled by the Democratic party and while one of the requirements for employment by the City of Chicago was residency within the city itself, three successive Democratic mayors turned to him as the man to do the hard jobs.

Training

What were his background and training? For fourteen years he was employed by the Sanitary District of Chicago, where he advanced to principal construction engineer. The Sanitary District had an enviable reputation as a progressive engineering organization and, among other activities, pioneered tunnel construction in the soft Chicago clays. For another fourteen years he was chief engineer and general superintendent of a major contractor doing heavy construction in the Chicago area. Many of us might think that twenty-eight years constitute a full career, but for Mr. Burke it was a training period after which he embarked on and carried out his major engineering works. His education? He studied liberal arts at Northwestern University to please his father, although he wanted to be an engineer.

Thereafter, he attended MIT, from which he was graduated in 1906. Thus, Ralph Burke is one of your own most distinguished graduates, and it is partly for this reason that I have chosen to tell his story on this occasion. After MIT, he continued part-time studies at the Kent College of Law in Chicago, again to please his father. Although he did not receive a law degree, his legal education gave him a valuable background that had much to do with his ability to deal with contractors and to negotiate. Thus, we can say that he not only had the best technical education possible in his day, but an unusually broad, general education as well.

Experience

I have mentioned Mr. Burke's long period of practice as an engineer and a contractor while he learned the details of his profession from all sides. There is a feeling today that the young engineer who wishes to leave his mark should and can vault immediately into a position of authority where he can make major decisions and influence development without thorough broad education, technical training and apprenticeship. A few engineers have succeeded in doing so, but most who have tried to advance too rapidly, who have been too impatient, who have felt that the years of learning as apprentices are too costly, have experienced the frustration of failing to meet their goals and ambitions. They have been forced to make decisions without adequate knowledge. They have been forced to accept the opinions of too many others without being able to apply the test of their own experience. They have found themselves listening to equally persuasive but contradictory arguments and unable to reach a rational decision of their own. In short, they have lacked full preparation. I would not argue that 28 years are needed. The time could be much shorter, but the experience is invaluable.

Engineering Education

A successful and influential civil engineer like Ralph Burke has many backgrounds to draw upon but, in my view, he is fundamentally a technical man. Without technical competence, no matter what his other attributes, he is not an engineer. There is no substitute for decisions based on dispassionate, rational thinking grounded in experience. Hence, I am quite convinced that the engineer's education must, therefore, be primarily technical. It should not be narrow; it should not overlook communication with others; it should not avoid studying social values; but it must above all be technical. It should not be divorced from practice but pointed toward practice.

Much has been said about the need to include socio-humanistic subjects in the engineering curriculum. I would not disagree in principle, and Ralph Burke's broad background illustrates their value, but I submit that every person learns best what he is most interested in, at the time when his interest is greatest. A prospective engineer as a student is most interested in the practice of engineering. His courses, then, should be slanted in this direction. Even the basic courses should contain practical examples from which the student can get glimpses of the essential nature of his subject.

I do not mean that all courses, or even many courses, should be case-history studies. There is far too much to learn to permit the luxury of many such cases or courses. Yet, if the instructor has the desire and the background, practice can be brought into the most basic course. For a good many years I have taught two courses, the beginning course in foundation engineering for undergraduates, and the finishing course in foundation engineering for graduate students. The latter is strictly a case history course, and I find it challenging. Nevertheless, the beginning course for undergraduates is even more challenging. The young men and women meeting this subject are just beginning to glimpse what their profession is like. They have been injected with large doses of mathematics, physics, chemistry and mechanics, but they have seen very little of what engineering does with these fundamentals. We try to assign our best and most experienced teachers to that first undergraduate foundations course, because they can bring into it the brief first-hand glimpses of the profession that make the difference between action-oriented engineering and academia.

Moreover, even the most practical-minded student, the one most technically oriented, may be highly receptive to instruction in public speaking and in good writing if he realizes that his professional advancement depends on his ability to communicate his ideas. He may be receptive to psychology if he realizes that he will become a member of a team consisting of people of many disciplines and walks of life — architects, planners, politicians, financiers, interested citizens — and that if he is to make his contribution he must be persuasive, understanding, and perceptive. What better place to develop and insist on good English than in every engineering report? The English and rhetoric departments cannot go it alone; without the follow-through in the technical classroom, the future civil engineer will all too readily conclude that real engineers don't need such extravagances. But give him the incentive and he may learn the pleasure of good craftsmanship in writing and speaking.

I hasten to add that a surprisingly large number of civil engineering students have broad interests and take pains to cultivate them. I hope our curricula allow such students to follow their interests without unnecessary academic restrictions.

Research and Teaching

Research is essential in the development of civil engineering and of civil engineering teachers. Yet research in civil engineering is not civil engineering, and the two should not be confused. Unless civil engineering research arises out of practice, as it should, it is likely to lead to a false conception of what civil engineering is actually about. On the other hand, research closely and clearly related to practice is invaluable to a student participant at all levels, and essential at the highest graduate levels. It may be complex and sophisticated, but if the student (and his research supervisor) have a clear view of how it may fit into the advancement of practice, it can serve as an effective vehicle for education. If it does not arise out of practice, if the researcher must attempt to find some practical area in which his esoteric results might possibly find an application, it is a supremely poor educational experience.

Who teaches our civil engineers? Often their professors are not really civil engineers at all. They are researchers, theoreticians, analysts. They are working in areas essential to civil engineering, they often make contributions to civil engineering, but they are not in fact civil engineers and they cannot give undergraduate students a realistic picture of civil engineering. Worse still, they may attract graduate students to follow in their own footsteps, to become still more refined and still further removed from the profession. Although there are some happy exceptions among both professors and institutions, I fear that civil engineering education and civil engineering are drifting apart, and that someday our most highly rated schools may find themselves playing a secondary role in the education of civil engineers. I fear, too, that the accreditation of our engineering schools, to which we in academia pay so much attention, is akin to the self-ratings of members of an exclusive club.

How can we avoid this apparent divergence of civil engineering teaching from civil engineering practice? Not, as might seem obvious, by converting retired practitioners into professors because, with a few shining exceptions, these men have been separated from the academic world for so long that they fail to realize the legitimate demands on the time of students for the engineering fundamentals that must be taught. I believe it can be accomplished at least partly by three major steps, which I shall enumerate rather than attempt to develop in detail.

Practice

The first is to inject young staff members into practice. This can be done in many ways, and it reflects quickly in their teaching. Secondly, there should be administrative enthusiasm rather than administrative disdain for practice. When promotions are considered, significant practice should be recognized not only as being equal to the preparation and publication of research, but as being more desirable than publication or research. Undoubtedly, I am bucking the present trend with this statement, but I feel it strongly. I can't help asking what we are trying to do with our emphasis on publication and research: advertise our universities, or train engineers? Thirdly, we can use innovative techniques in teaching. I am convinced that case histories, properly used, can teach engineering judgment to a substantial degree. I was greatly pleased and favorably impressed by the report written on the Mangla Dam project by Professor Lambe's students a few years ago. This project was innovative to a high degree, and I am sure it left its mark on every student as well as every staff member who participated. I was intrigued not only by the interest of the students in Mangla Dam, but also by their great interest and considerable enthusiasm for the Tarbela Project, which they visited somewhat incidentally to Mangla. The reason for the enthusiasm and for the difference in attitude was that Tarbela was under construction, a going concern, whereas Mangla was essentially completed. Here, I think, is the key to good civil engineering education. The students are interested in the action of their profession. They should see their profession in action, and they will derive from these insights the motivation to learn whatever is necessary to engage actively in their profession. They can even be persuaded that economics, English, or law are worth their attention.

Engineering Judgment

Should the engineer be a partisan or advocate of one side in matters of consumer interest, of the environment, of all the issues that we encounter so frequently today? Only, I feel, if he has considered and fairly evaluated all the evidence that he can obtain. Partisanship based on suppressing part of the evidence, based on emphasizing the desirable aspects associated with one point of view and the undesirable aspects associated with the other, this partisanship tarnishes the public's view of the engineer who, in reality, has nothing to offer if it is not a balanced judgment derived from a dispassionate study. Biased partisanship, which makes the point but suppresses part of the facts, is not for the civil engineer. The civil engineer need not and should not be disinterested, but he should be fair. That his views may not be spectacular enough to sway the public at the height of a controversy is an unfortunate reflection on public opinion and how it is formed, but it preserves the integrity of the profession.

There is no room, in my opinion, for Ralph Naders in civil engineering. Such people undoubtedly have effectively and dramatically called our attention to faults and shortcomings in engineering practice and have brought to pass many desirable changes. Yet, their tactics have been to highlight one side of a picture and to underplay the valid arguments that almost always exist on the other. The tactics are those of advocates, of lawyers, but not of engineers.

In our part of the country a highly controversial reservoir project is being designed by the Corps of Engineers. The project has its advantages and disadvantages, its positive and negative impacts. One of the leaders of the opposition is an engineer and an academician. He advances sound technical reasons for his objections, but never admits the possibility that there may be compelling arguments in favor of the project. He is an engineer-hero in the eyes of the opponents and is often quoted in the press. If the press reports are accurate, he characterizes the Corps of Engineers as an incompetent, biased, self-serving organization. To be sure, the Corps has made its share of mistakes, but it is by no means incompetent and it is composed of many dedicated people including engineers. Partisanship by engineers leads to exaggerated statements that discredit engineers and engineering, and in the long run can only be destructive.

There is room, of course, for honest differences of opinion among engineers. There is every reason why engineers should debate the differences, even in public. In the now prevalent violent swings of public opinion and public reaction, a partisan proponent may see his view rise and fall, to be displaced by another. Even the public recognizes that the extreme views cannot be the ultimate and best solution, and eventually, if not as soon as might be desired, comes to a more reasoned middle-ground solution toward which it can be helped by the activities of the public-minded, unbiased, experienced engineer. After violent swings of public opinion, there is often a return to sanity. It would be my hope that the engineer will be found already occupying that position of sanity, and will have had a hand in bringing the public to that ultimate point of view.

In one sense, all civil engineers are in the public service, because their works are either used directly by the public or make available something the public needs or wants. For a quarter of a century, Sam Mathis was associated with one company, a private enterprise. The odds are high that the gasoline you and I have used in our cars at one time or another has passed through some facility he constructed. From the subsiding swamps and shorelines of Lake Maracaibo to the deserts of Libya, he built his share of the physical facilities that make possible society as we know it, and without which our enormous mass of humanity could no longer survive. We may debate, with good reason, how to order our priorities in growth of population and use of resources, but whatever the decisions, there will be need for the constructive action of civil engineers like Sam Mathis and for nurturing the civil engineering mind.

REMOVAL OF CATIONS FROM LEACHATE BY INTERACTION WITH SUBSURFACE SOILS

By Ira W. Leighton[1] and Frederic C. Blanc[2]

Abstract

This investigation was conducted in order to determine the magnitude of cation removal from leachate provided by soils under a landfill. The interaction of sodium, potassium, calcium, iron, and magnesium cations in leachate with a soil was examined for eight Massachusetts soils in laboratory soil column experiments. Removal capacities for the various soil types were developed based on the soil column experiments. The removal values ranged from 3.8 milliequivalents to 31.1 milliequivalents per 100 grams of dry soil. Shaker tests were used to verify the soil column experiments.

Introduction

In recent years the expansion of suburban areas combined with increased refuse production has resulted in greater public awareness of solid waste disposal problems. One of the problems associated with solid waste disposal sites is the deterioration of groundwater quality in the vicinity of the site resulting from the entrance of highly contaminated drainage i.e. leachate, into the underlying aquifers. This problem can be controlled by: not permitting refuse to be deposited below the maximum elevation of the groundwater table, minimizing the amount of infiltration into the fill, and by collection and treatment of the leachate. Where the leachate is allowed to enter the soil below the fill, the attenuating capacity of the soil and dilution must be relied upon to protect groundwater supplies.

Data on the actual leachate treatment provided by the subsoils are scarce. This investigation (4) was conducted to collect such data and examine the interaction between the leachate and the soils underlying the fill. Leachate can add a variety of inorganic contaminants to the local groundwater in the area of a landfill. Elements existing as ions, such as chloride, calcium, magnesium, potassium, and sodium, are usually present in high concentrations. Fortunately ion exchange and adsorption in the sub-soils result in the removal of some ions from the leachate-groundwater mixture. This removal can be considerable, particularly in the case of some cations. Nitrogen can be present as the ammonium cation, as undissociated ammonia at high pH,

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or as the nitrate. None of these is well retained by soils. Phosphorus as the phosphate anion is well retained by most soils.

Specifically, this research focused on the ability of eight different Massachusetts soil types to remove five major cations from leachate. The five cations studied were calcium, magnesium, potassium, sodium, and iron. All of the cations investigated may have a detrimental effect on groundwater uses. Calcium and magnesium cause hardness problems. Iron can cause staining problems and problems in industrial processes. Sodium is detrimental in drinking water to persons suffering from cardiac and circulatory diseases. The removal of such cations by soils is therefore of considerable interest. In addition to evaluating the removal capacity of the soils, the order of preference in removal and other factors were observed. The experiments were carried out to determine the removal which might be expected in the unsaturated subsoil zone between the refuse and the groundwater table, or an unsaturated subsoil which may be used for subsurface disposal of leachate.

Cation Removal in Soils

The capacity of a soil to remove leachate cations is due to any or a combination of the following mechanisms: ion exchange, adsorption, complexing, fixation, biological uptake, precipitation, and filtration. The relative contribution of any one mechanism to the total observed removal capacity is difficult to establish in such a complex system; however the literature suggests that in terms of orders of magnitude cation retention in soils is primarily a function of ion exchange and adsorption. Adsorption and ion exchange are surface chemical phenomena which are most pronounced in the finer sized particles within a soil mass. The importance of the colloidal-sized particles stems from the fact that although larger particles can develop the surface charges required to attract and hold ions, the larger particles do not have the tremendous surface area per unit weight which the smaller particles have. It is for this reason that fine-grained soils containing clay and silt particles have greater cation removal capacity than the coarse grained sandy soils. In addition to soil particle size, there are many other factors which determine the nature of cation removal by adsorption or ion exchange. Some of the other factors are: the composition of the exchangeable ions present in the soil, the amount of organic matter in the soil, the particular ions in solution and the concentration of the ions in solution.

It should be emphasized at this point that the term "removal capacity" as used to describe the results of this investigation, does not mean the total ion removal capacity nor is it the total cation removal capacity. The term as used here refers to the total removal of the five cations considered in the research.

Experimental Methods

Leachate Sampling and Testing

Leachate samples for this investigation were obtained from the dump fill refuse disposal site in the town of Scituate, Massachusetts. Four samples were taken by excavating a sump into the botton of an existing cut in the fill. The Scituate disposal site is situated in an area where the groundwater table is near the surface of the fill and the refuse is in contact with the groundwater. All of the samples collected were highly colored and possessed a strong odor which indicates that the fill was undergoing active decomposition.

Table 1 lists a reasonably thorough analysis of the Scituate Leachate. Comparison of this analysis with leachate characteristics observed in other studies (2) indicates that the Scituate Leachate can be considered as a typical strong leachate which would be produced in the years immediately after filling.

All the analyses conducted on the leachate were conducted in accordance with the procedures prescribed in Standard Methods (6), with modifications in sample pretreatment. The analyses for sodium, potassium, calcium, magnesium, iron, copper, nickel, zinc and lead were performed using an atomic absorption spectrophotometer.

Soil Sampling and Testing

The eight soil samples selected for this investigation were collected at various locations in the Boston Metropolitan area. Sieve analysis and hydrometer testing revealed that the soil types ranged from sand to silty clay. Table 2 indicates the classification of the samples based on the textural triangle nomenclature developed by the U.S. Soil Conservation Service (5). The soil samples were collected from both the A Horizon (surface soil) and the B Horizon (subsurface soil). In this case the B Horizon material was taken from the depths of 6 to 8 feet below the surface.

Soil Column Tests

In order to establish the cation removal capacities of the various soil types, a series of soil column tests were set up. The soil columns consisted of ½ in. (1.27 cm) ID acrylic tubing, approximately 18 inches (46cm) long. Each tube was threaded and a ¼ x ¼ in. nylon tube insert placed into the end of the column. The soils were supported on a small piece of 50 mesh stainless steel screen which had been taped into the nylon insert. In addition to the screen, one gram of double-washed coarse sand was added to each column to minimize the loss of fines in the effluent.

TABLE 1

COMPOSITION OF SCITUATE LEACHATE

Contaminant	Concentration*	Contaminant	Concentration*
pH	5.5	Nickel	0.16
Hardness, as $CaCO_3$	1950.0	Zinc	0.81
Alkalinity, as $CaCO_3$	729.0	Copper	0.02
Calcium	632.0	Lead	0.00
Magnesium	94.8	Total Solids	6770
Sodium	298.0	Suspended Solids	460
Potassium	440.0	Conductivity	2940 mhos
Iron (Total)	280.0	BOD_5	11700
Chloride	502	BOD_L	13820
Sulfate	298	K (BOD rate constant)	0.163 1/day
		COD	15000

*All results in this table with the exception of pH are reported in mg/1 unless otherwise noted.

TABLE 2

CLASSIFICATION OF SOILS INVESTIGATED

Soil Sample	Soil Classification	Source of Soil Sample
1	Sand	B Horizon - Scituate, Ma.
2	Loamy Sand	A Horizon - Sudbury, Ma.
3	Loamy Sand	B Horizon - Chelmsford, Ma.
4	Sandy Loam	B Horizon - Chelmsford, Ma.
5	Sandy Loam	B Horizon - Sudbury, Ma.
6	Silt Loam	B Horizon - Sudbury, Ma.
7	Silt Loam	A Horizon - Chelmsford, Ma.
8	Silty Clay	B Horizon - Sudbury, Ma.

The soil samples were weighed and then placed in columns in small increments, with each successive increment being tamped in place with a ½ in. (1.27 cm) glass rod. The depth of soil in the column was measured and the density of the soil computed. A blank column was prepared for the purpose of establishing how much of the observed removal was due to the column surface and supporting base i.e. the sand and 50 mesh stainless steel screen.

Leachate was added in ten milliliter increments to each column and the flow rate through the column recorded. The addition of each leachate increment was spaced so as to maintain an unsaturated condition in the column. The columns were allowed to leach freely until the ten milliliters of leachate had passed through the column or until the flow of leachate from the effluent stopped for an excessive period of time. The leaching was continued in this manner i.e. adding successive ten milliliter increments until the cationic concentrations of calcium, magnesium, sodium, potassium, and iron in the effluent were equivalent to that of the raw leachate. At this point the removal capacity of the soil was considered to have been exhausted and the leaching was discontinued.

Shaker Test Experiments

In order to determine the effects of varying contact times on the cation removal processes, it was necessary to utilize shaker tests to supplement the soil column data. The procedure utilized involved placing twenty grams of soil in a 50 ml plastic centrifuge tube and adding a 20 ml volume of leachate to the tube. A series of tubes was prepared in this manner for each of the eight soil samples. After sealing, the tubes containing the leachate and soil were placed in a variable speed shaker. The soils and the leachate were contacted for time periods of ½ hr, 1hr, 2hrs, 3hrs, 5hrs and 7hrs. At the end of the appropriate contact time a tube containing each soil type was removed from the shaker table and centrifuged for 15 minutes at 2000 rpm to separate the soil from the leachate.

After centrifuging the leachate was siphoned off and then analyzed for calcium, magnesium, sodium, potassium, and iron.

The removal was calculated based on flow volume and concentration relationships between the samples containing the soil and leachate and the leachate blanks which had been carried through the same procedures.

Discussion of Results

Soil Column Experiments

The selection of specific soil types utilized in the soil columns was based upon the desirability of using a fine-grained soil which would possess good removal potential and also be sufficiently permeable to allow the leachate to pass through the column. Figures 1-4 are typical plots of decrease in cation concentration, in milliequivalents per milliliter versus the cummulative volume of leachate passed through the column. Noted on each figure is the total removal capacity in milliequivalents per 100 grams of soil. As one can observe from the figures, the ability of each soil type to retain the cations in question decreases as each additional increment of leachate is passed through the columns until the removal capacity is exhausted. It is interesting to note from the figures that the soils do not approach exhaustion of removal capacity in the same manner. For the eight soil types tested the removal values ranged from a low of 3.8 milliequivalents per 100 grams of soil to a high of 31.1 milliequivalents per 100 grams of soil. Table 3 lists the removal capacities for the various soils. The removal capacities in this investigation are in the same range reported by Hill (3) for cation removal from domestic wastewater effluent for six major soils formations extensive in New England.

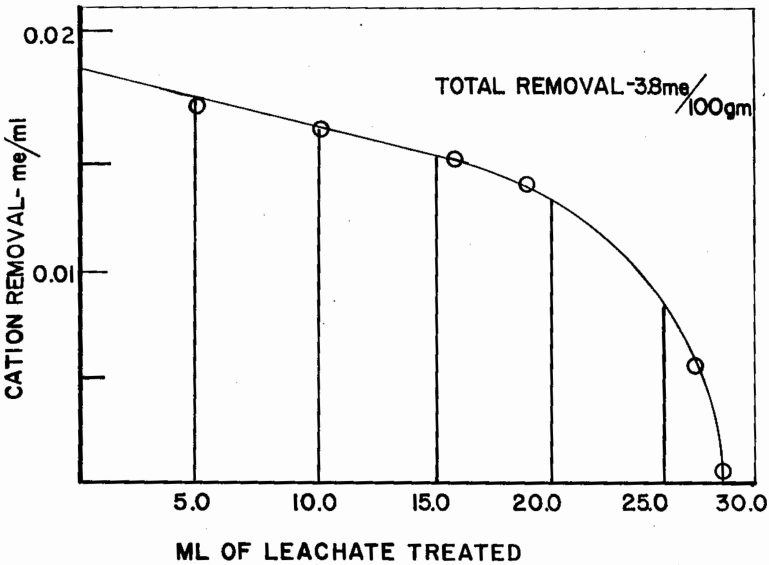

Figure 1 — Removal capacity of soil sample 1

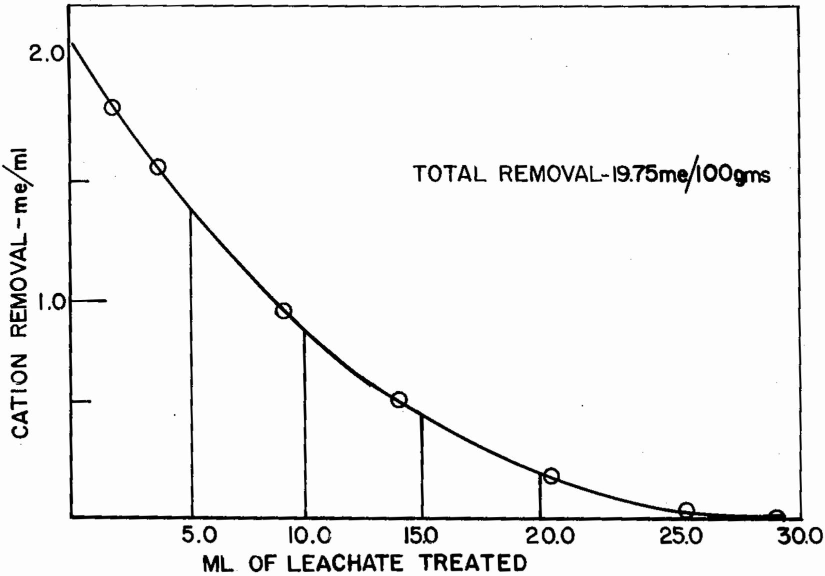

Figure 2 — Removal capacity of soil sample 3

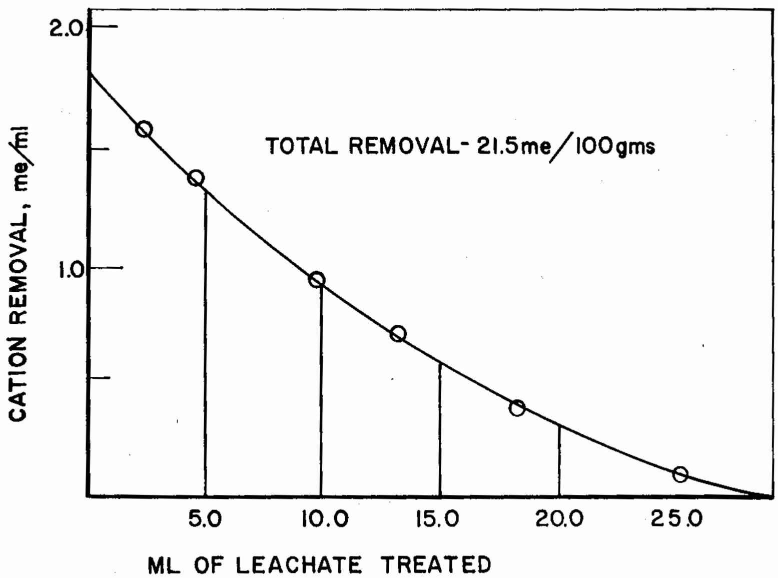

Figure 3 — Removal capacity of soil sample 6

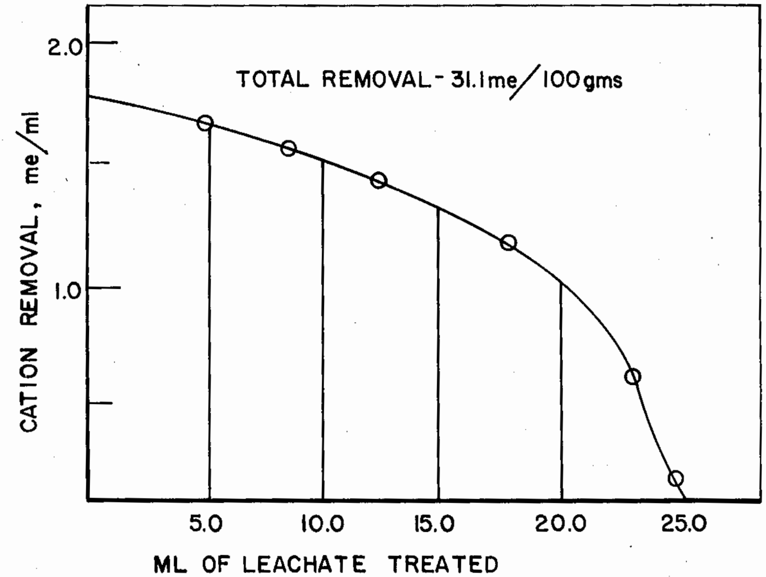

Figure 4 — Removal capacity of soil sample 8

TABLE 3

CATION REMOVAL CAPACITIES OF SOILS TESTED

Soil Sample	Soil Classification	% Weight Passing #200 Sieve	Removal Capacity Milliequivalents Per 100 Grams
1	Sand	2.44%	3.8
2	Loamy Sand	12.28%	17.1
3	Loamy Sand	18.66%	19.7
4	Sandy Loam	30.84%	18.1
5	Sandy Loam	32.54%	23.1
6	Silty Loam	55.0 %	21.5
7	Silt Loam	65.11%	29.8
8	Silty Clay	87.84%	31.1

The effect of particle size on the cation removal capacity is illustrated in part by Table 3. Comparing the percentage of the soil which passes a number 200 sieve (.075 mm in diameter) with the removal capacity of the sample shows the increasing removal capacity with decreasing particle size. It should be noted that there are other factors such as the nature and origin of the particles and the humus content which may also affect the removal capacity.

The selectivity ion exchange and adsorptive reactions taking place in the soil is well documented. Certain cations are adsorbed to a greater or lesser extent as a result of the following factors: 1. pH of the leachate, 2. Valence of the cations in solution. 3. Degree of hydration of the ions present, 4. The relative concentrations of the ions, 5. The nature of the anions associated with the cations.

Bear (1) points out that there is no one order of cation removal which consistently applies to all soil types, instead each soil develops an order of removal in response to the above-mentioned factors. There is, however, a definitive pattern of removal expressed with reference to each specific factor.

The relative quantities of calcium, magnesium, sodium, potassium and iron removed by the different soils tested are a function of any or all of the above-mentioned parameters. The relative order in which each ion was removed also varied with respect to each soil type. It becomes quite apparent that the mechanisms involved in this removal process are extremely complex.

An indication of how the removal order varies for the various soil types may be obtained from an examination of Table 4. This table shows results for two leachate samples of normal and low cation concentration. Leachate Sample A represents the normal concentrated leachate used in this investigation, while Leachate Sample B is a more dilute sample selected in an effort to ascertain the effects of concentration on the removal order. Table 5 lists the concentrations of the five cations tested in samples A and B. From the analyses it is apparent that, except for iron content, Sample A was approximately four times as concentrated as Sample B. This can be considered to be representative of the wide variation in leachate concentration which may be experienced by the soils under landfills. It should be noted that the results reported for leachate Sample A in Table 5 represent the percent of each cation contributing to the ultimate "removal capacity" i.e. leaching of the soil column to the point of exhaustion, whereas the percent of "removal" values for Leachate Sample B represent the "removal" from the first five milliliter slug of leachate passing through each soil column. The soil samples handling the dilute leachate could not pass the volume of leachate required to exhaust the cation removal capacity of the columns because solids caused a clogging of the columns before the exhaustion point was attained.

The results indicate a variation in removal order with respect to at least two factors: 1. A variation in removal with respect to soil type and 2. A variation with respect to the concentration of the leachate. The results indicated that cation concentration and valence appear to be at least equally important in establishing the removal pattern, or order of removal for the different cations. The concentration of magnesium in the effluent from the columns increased above that of the raw leachate with Sample B. This indicated that magnesium was being released from the soil into the leachate. Magnesium is one of the most common exchangeable cations found in the soils of this region; therefore, the increase in magnesium concentration is evidence that exchange reactions are taking place in the soils.

Shaker Tests

In order to determine the influence of contact time on the cation removal process and removal capacity as determined by the soil column experimental procedures, a series of shaker test experiments were performed.

TABLE 4
CATION REMOVAL PATTERNS FOR CONCENTRATED AND DILUTE LEACHATE SAMPLES

	Concentrated Leachate Sample A					Dilute Leachate Sample B				
Raw Leachate Constituents Expressed as % of Total Milliequiv.	Fe	Ca	Mg	Na	K	Fe	Ca	Mg	Na	K
	5.0	38.0	6.0	21.0	30.0	27.0	41.0	7.0	10.0	15.0
Soil Sample	Removal of Cations as expressed as a % of "Total Removal"*									
1	4	25	6	23	42	67	6	0	10	17
2	8	39	0	13	40	82	17	0	0	1
3	4	67	6	6	17	65	28	0	0	7
4	11	50	10	9	20	80	13	0	2	5
5	10	60	9	11	10	78	17	0	0	5
6	12	13	0	21	54	88	8	0	1	3
7	4	38	6	16	36	67	26	0	3	4
8	9	42	6	13	30	75	18	0	3	4
Average	8	42	5	14	31	75	16	0	3	6

*Total Removal = total milliequivalents of the Fe, Ca, Mg, Na, K cations removed by the soil column in this experiment.

Initial shaker tests performed on two soil types produced an interesting result, namely an increase in removal with contact time up to a maximum followed by a decrease in removal. The results of this experiment are presented in Figures 5 and 6. The plot of removal versus contact time gives the impression that material which is adsorbed during a specific time interval is later placed back into solution. The soil's inability to reach an equilibrium with respect to the quantity of cations removed is of particular importance in that it has a direct bearing upon the soil's ultimate removal capacity.

TABLE 5

CATION CONCENTRATION ANALYSES OF SAMPLES A AND B

Cation	Sample A	Sample B
Calcium	110.0 Me/1	28.0 Me/1
Magnesium	17.8 Me/1	5.0 Me/1
Sodium	62.2 Me/1	7.8 Me/1
Potassium	90.0 Me/1	10.0 Me/1
Iron	14.2 Me/1	18.8 Me/1
Totals	294.2 Me/1	69.6 Me/1

In the interest of determining whether this phenomenon of an increase to a maximum removal value followed by a decrease was a function of the testing procedure or a function of a soil-leachate interation, the experiment was repeated. The second experiment consisted of contacting the remaining six soil types with the leachate as well as a synthetic ion exchange resin.

The second experiment produced the same results as the original test in the case of the six soil samples, as typified by Figures 7 and 8. However, the synthetic ion exchange resin (Rexyn H, a hydrogen regenerated cationic resin marketed by Fischer Scientific) did not exhibit a decrease in removal, as is illustrated in Figure 9. The removal pattern of the resin with respect to time resulted in a gradual increase in removal up to a maximum value which was near the exchange capacity of the resin, followed by a leveling off at this point. This result indicated that the inability of the soil samples to retain the cations after initial removal probably was the result of a soil-leachate interaction and not a function of the testing procedure. A specific contact time requirement for a soil to reach its maximum removal capacity is an understandable phenomenon. However, the apparent release of cations in the shaker experiments with longer contact time is a phenomenon which was not expected. Establishment of the exact mechanisms responsible for such behavior would have required an effort beyond the time limits of this investigation.

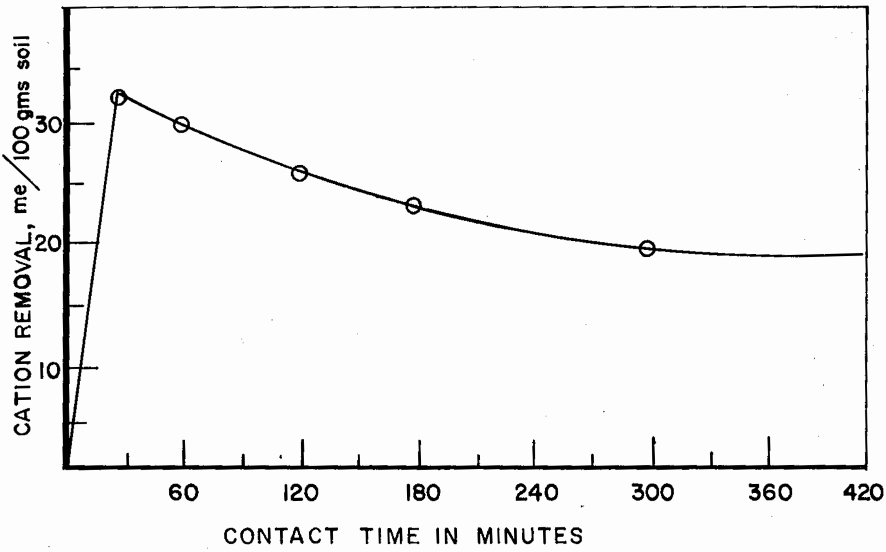

Figure 5 — Shaker test results on soil sample 5

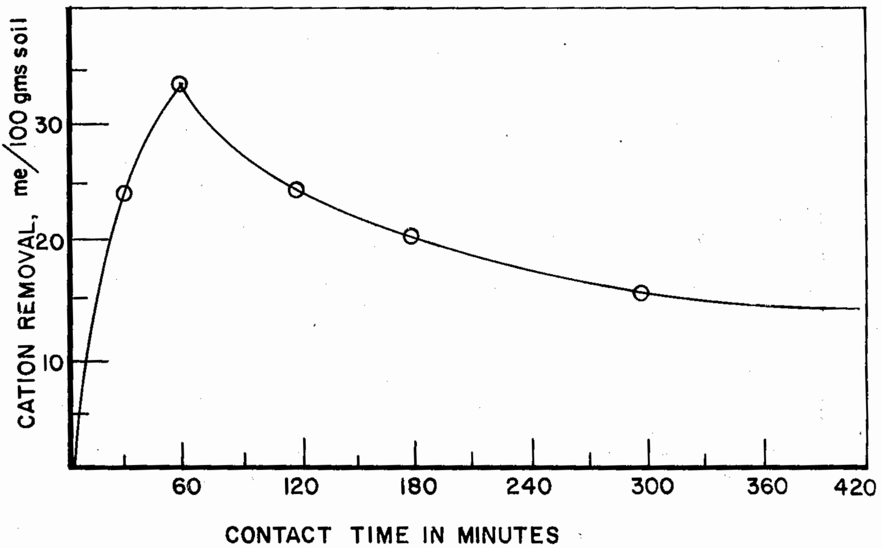

Figure 6 — Shaker test results on soil sample 4

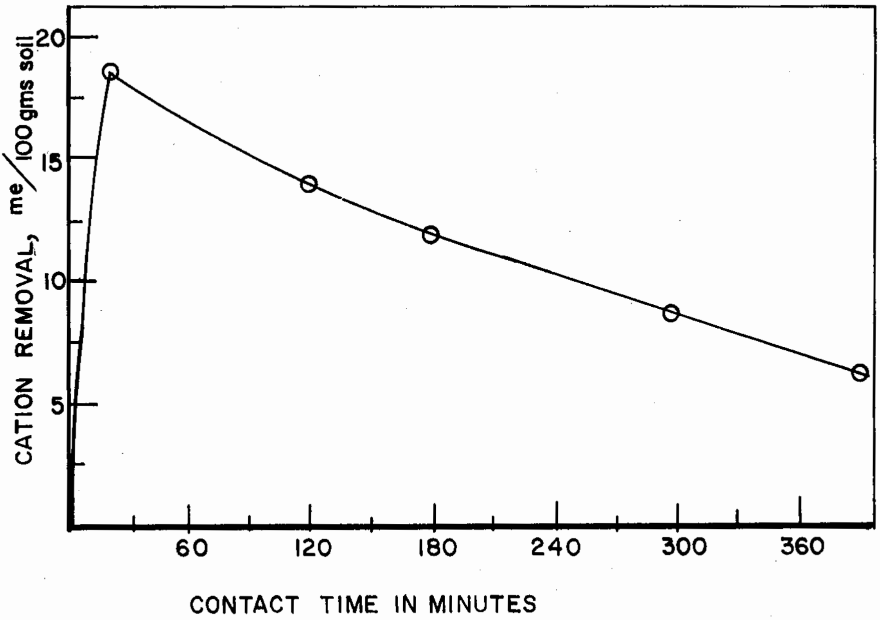

Figure 7 — Shaker test results on soil sample 3

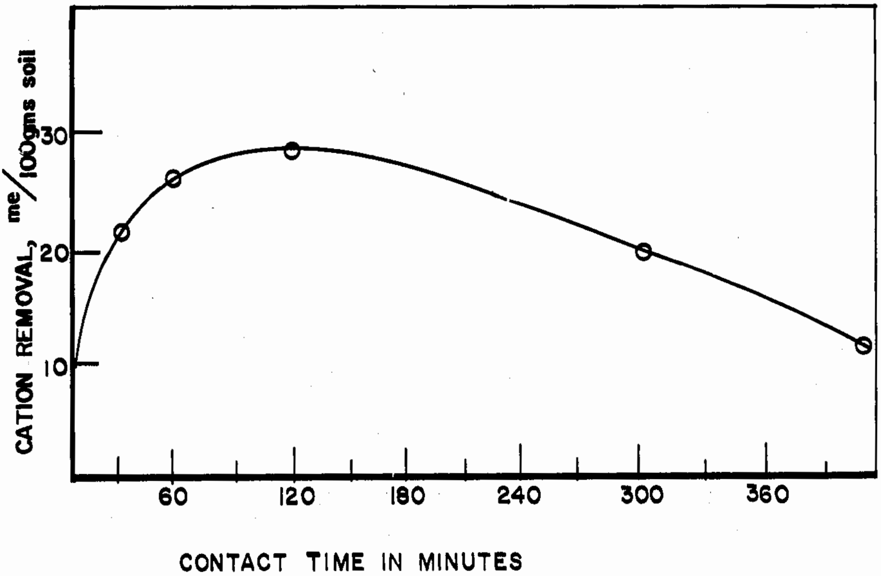

Figure 8 — Shaker test results on soil sample 8

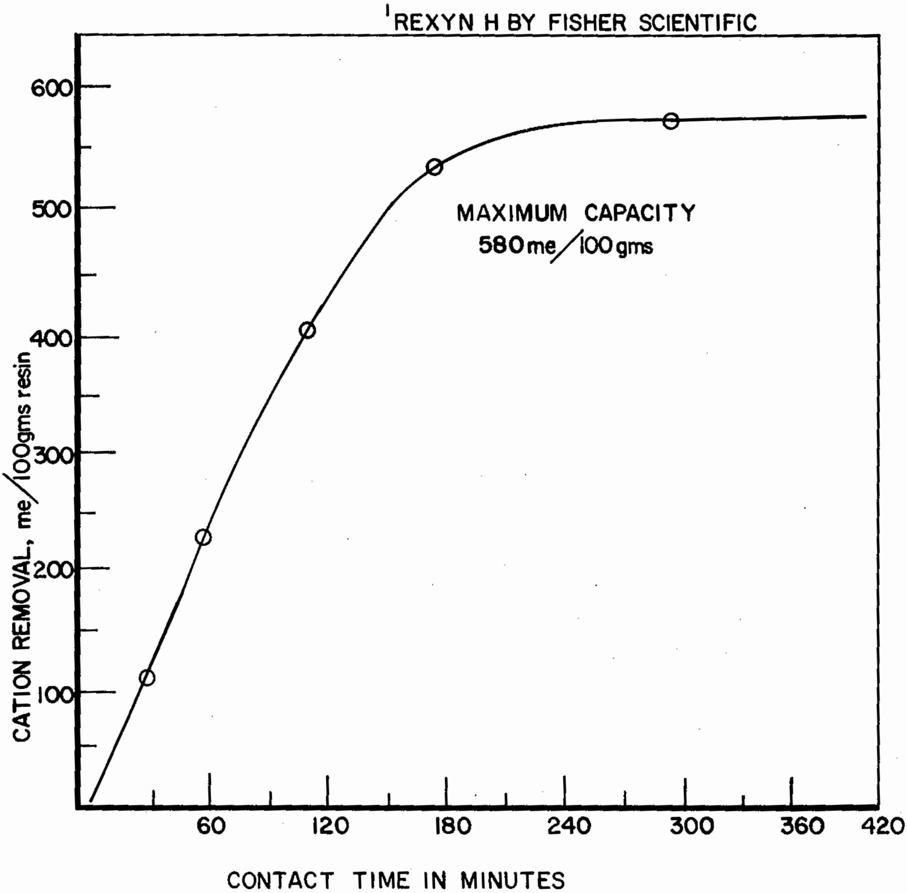

Figure 9 — Shaker test results on the synthetic ion exchange resin[1]

The results of the soil column experiments cannot be directly compared with those in the shaker test experiments due to the difference in the testing methods. However, the shaker tests indicate that a potential may exist for the soil to release previously adsorbed material. The fact that the quantity of material adsorbed by the soils was lower in the shaker tests after prolonged contact times than that observed in the soil column experiments at the time of breakthrough also suggests that the soil may reach equilibrium with respect to permanently adsorbed ions. It appears that this equilibrium may be lower than the total quantity of material initially removed by the soil.

Significance of Results

In order to evaluate the significance of the soil "removal capacities" for the five cations determined in this investigation, the results must be related to a typical sanitary landfill operation. Comparing the shaker test results with the soil column experiments it seems reasonable to select removal capacities which are more conservative than the values obtained in the soil column experiments. For this comparison the following values were assumed based on the experimental data and typical soil densities:

	Five Cation Removal Values		Density of Soil	Gram Equivalents of Cations
Soil Type	Me/100gms	Me/lb	Lbs/ ft^3	Removed per ft^3
Sand	3.8	17.2	110	1.893
Sandy Loam	10.0	43.5	105	4.756
Silt Loam	20.0	90.6	105	9.513

An estimate of the quantity of the five leachable cations coming from a typical landfill can be made based on values reported in the literature. In this case the values in Table 6 were assumed as total quantities released from an acre-ft of compacted refuse during the critical leachate generation period when contaminant concentrations are at the highest levels. Based on the values in Table 6 and the conservative removal values selected for the soil types, the following soil to landfill refuse volume ratios may be calculated to provide total removal:

Soil Type	Acre-ft of soil/acre-ft of refuse
Sand	1.72
Sandy Loam	.69
Silt Loam	.34

TABLE 6
TOTAL QUANTITIES OF LEACHABLE CATIONS RELEASED FROM REFUSE

Ion	Gram Equivalents/acre-ft
Sodium	29,500
Potassium	27,500
Calcium	46,900
Magnesium	14,000
Iron	24,300
Total	142,200

Thus the results indicate that the unsaturated subsoil layer directly under a landfill can provide a considerable degree of leachate treatment with respect to cation retention. The possibility of using selected subsurface unsaturated soil deposits in the vicinity of a landfill for treating the collected leachate from the fill also appears to be promising based on these data. There is a definite economic advantage to this approach in many areas.

Conclusions

1. Results obtained from the soil column experiments confirm that the adsorptive capacity of a soil for the five cations tested is a function of the particle size distribution of the soil i.e. the percent weight passing a #200 sieve. The range of the removal values was as follows: 3.8 me/100 grams of sand to 31.1 Me/100 grams of silt loam (air dry weight of soil).

2. The shaker test experiments indicated that contact time is a parameter which should be investigated when determining ion removal capacities by soil column testing.

3. Comparison of the shaker test and flow test results for comparable contact times suggest that the equilibrium conditions observed in the shaker tests may be expected to occur in a flow-through system.

4. The order of removal of leachate cations is generally a function of at least two factors, namely, the relative concentrations of the various cations present in the leachate and the valence of the cations. Based on this investigation the concentration and valence appear to be equally important in establishing the removal pattern.

5. Application of the soil removal values to a typical landfill operation on a unit volume basis demonstrated the desirability of locating landfills in areas where the underlying soil is fine grained. The data suggest that collected leachate from landfills could be pumped to a separate site in the vicinity with suitable soil conditions for economical subsurface land disposal and treatment.

Acknowledgements

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THE FOLIATION SHEAR ZONE — AN ADVERSE ENGINEERING GEOLOGIC FEATURE OF METAMORPHIC ROCKS

by
Don U. Deere[1]

Introduction

A surprisingly large number of engineering projects constructed in recent years in metamorphic rocks have encountered poor rock conditions locally that have significantly affected the difficulty of construction. Both the cost and time of construction have been increased. In some cases remedial measures or redesign of project elements have been required.

An analysis of the problems shows that in nearly every case the cause of the difficulties was the local presence of thin sheared zones along the foliation of the metamorphic rocks. This shearing, inherited from the geologic past, had occurred inevitably along the weakest layer of the metamorphic rock, often in a mica schist zone with a high mica content (Patton and Deere, 1970, 1971). Such mica-rich zones occur in thick schist sequences but, more importantly, also in the massive metamorphic gneisses and quartzites as thin interbeds.

These sheared zones parallel to the foliation (layering of the platy minerals) may be termed "foliation shear zones" or "foliation shears". In the following sections the characteristics of these shear zones, their possible mode of origin, and the manner in which they can adversely affect an engineering project are discussed.

Characteristics of the Foliation Shears

The writer has examined foliation shears in more than a dozen construction projects where the number of foliation shears ranged from only 1 or 2 per site to 10 or more. While the variations in physical characteristics were noted to be considerable, certain generalizations may be made.

Thickness

The typical thickness of the foliation shear is in the range of 1 to 4 inches. Some zones have been seen as thin as ¼ inch to ½ inch although continuous over distances of 100 feet or more; only rarely will one be found as thick as 3 feet. Typically, the zones will thicken and thin somewhat. The rock adjacent to the zone will also be weaker than normal, as noted later.

Brekke and Howard (1972) use the term "seam", indicating "a minor, often clay-filled zone with a thickness of a few inches . . . seams may repre-

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sent very minor faults or altered zones along joints, dikes, beds or *foliation*" (emphasis added). The foliation shear of this paper, then, fits their definition of seam. In the field, they are often referred to as "slips" or "mud seams".

Continuity and Structural Attitude

The foliation shears may be traced for distances up to several hundred feet. However, at many construction projects the exposed rock is of limited area and the full length can not be seen. Experience would indicate that they are not strictly local occurrences of just a few tens of feet extent but of at least several hundred and, in some cases, probably a few thousand feet.

The shears trend parallel, or in some instances subparallel, to the strike of the foliation. They also follow the dip of the foliation but locally may cut across it where the foliation flattens or rolls. Because the foliation is typically steep in the meta-sedimentary rocks of the eastern United States, and in many other parts of the world as well, the foliation shear zones typically occur with dips of 50°-80°.

Shear Zone Materials

The weakest material in the shear zone is the ground-up rock material — typically a crushed mica schist gouge. Grain-size analyses of several samples of gouge from foliation shear zones in the mica schist of New York City Water Tunnel No. 3, currently under construction, indicated that the materials are essentially moist, plastic well-graded mixtures of clayey, silty sand and occasional rock fragments with average percentages as follows: 15 percent clay-size (less than 0.002 mm), 35 percent silt size, and 50 percent sand size. For the seven samples tested the clay-size percentage ranged from 7 percent to 30 percent (Fig. 1). Also shown in Figure 1 is the grain-size analysis of a gouge sample from a foliation shear in Washington, D.C., and the limits of 12 samples of fault gouge (the latter from Brekke and Howard, 1973).

The clay mineral analysis by X-ray diffraction of the New York samples (Wahl, 1973) indicated that the most plastic, slippery gouges contained montmorillonite or montmorillonite-chlorite (interlayered) as the main clay-size constituent. The less plastic ones had predominantly vermiculite or vermiculite-montmorillonite in the clay-size fraction. It is believed that the montmorillonites, vermiculites, and chlorites were formed by alteration of the primary micas and feldspars in the original rock by the heat, pressure, and fluid migration present during and immediately following the shearing. Surface weathering may also cause an increase in the clay content near the surface. Brekke and Howard (1972, 1973) emphasize the complexities of

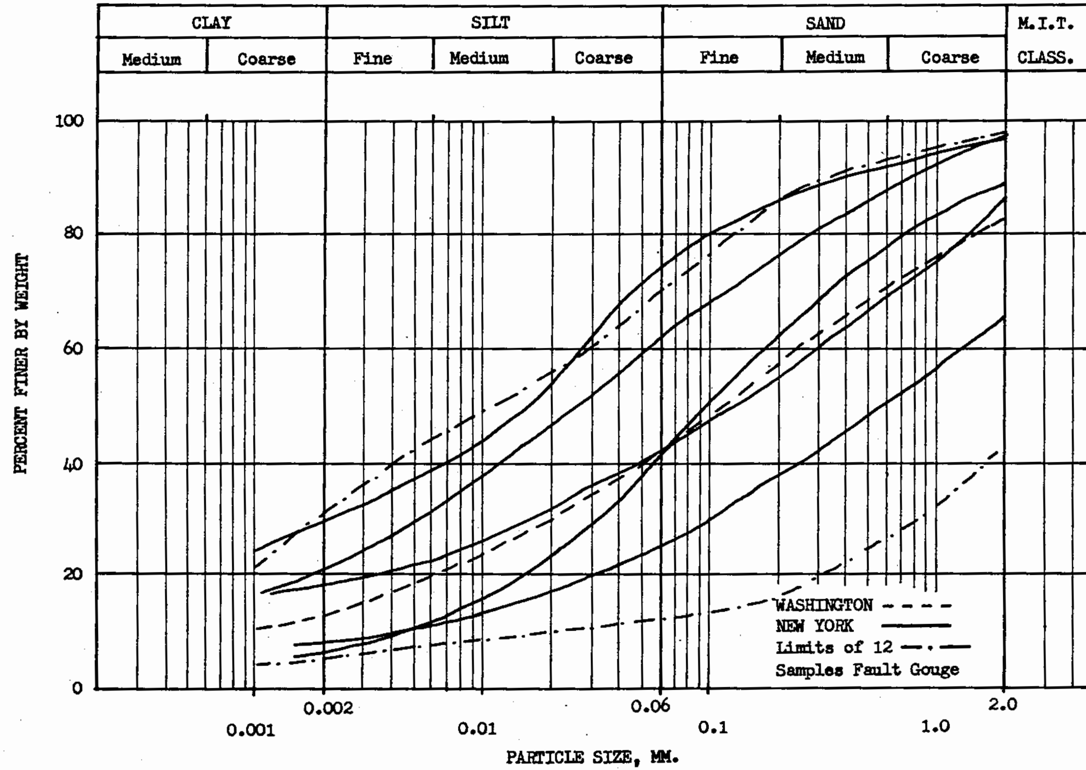

Figure 1 — Grain-size Distribution Curves of Foliation Shear Gouge from Washington, D.C., and New York City. (Also shown are the limits of 12 samples of fault gouge from Brekke and Howard, 1973).

fault gouge and the hydrothermal alteration of gouge materials which commonly occurs.

In addition to the plastic gouge which may be only a fraction of an inch to several inches wide, the remainder of the shear zone consists of partially crushed, sheared and slickensided rock. This zone often exhibits fracture cleavage. The gouge layer usually occurs on one (occasionally on both) sides of the shear zone where it is plastered against the adjacent hard rock. At times, the gouge layer, or thin branches of it, will occur in the middle of the shear zone.

The joints in the hard rock adjacent to the shear zone may be somewhat disturbed with some loosening, slickensiding, and chemical alteration (thin clay or chlorite coating on the joint surface). Thus, it is not just the few-inches thick shear zone itself that is weak; the slightly disturbed rock on either side may form a zone several feet wide that is substantially weaker and more compressible than the surrounding rock. Thinner foliation shears may occur parallel or sub-parallel to the main one at distances of several feet.

Spacing

The spacing of foliation shears is extremely variable, as one would imagine because of the great variety of metasedimentary rock types and the diverse stress conditions both regionally and locally to which they are subjected. On one project, the Churchill Falls Hydroelectric Project, Labrador, five foliation shear zones were found in about 6,000 feet of underground workings, giving a true spacing of from 500 feet to 1500 feet.

In the Washington, D.C., Metro, along Connecticut Avenue at the DuPont Circle Station currently under construction, the average spacing is about 25 feet. In some of the New York City tunnels the true spacing (calculated normal to the strike and dip) was found to be in the general range of a hundred to a thousand feet.

Permeability

The foliation shear zones, like many faults, are both dams and drains (Patton and Deere, 1971). Because of the fine-grained gouge, they have a low permeability in a direction crossing them. Therefore, water is often found perched above them and the shear zones act as dams. The zones adjacent to the foliation shear may be more permeable because of the disturbance and opening of the joints. Thus, ground water may travel more easily parallel and adjacent to the zone than in the main rock mass and the shear zones act as drains.

Offset Along Foliation Shears

Additional observations are needed in order to document much better the range of offsets along foliation shears. In many cases, it is probably just a matter of a few inches. In others it may be a few feet. A recent observation in Washington, D.C., showed 5 feet offset of a pegmatitic quartz dike.

It has been suggested that foliation shears should be termed foliation faults since there is evidence of some shearing displacement. However, since they are probably associated primarily with folding and stress relief, have no great depth of penetration, are of local extent of a few hundred to a few thousand feet, and have only inches to feet of offset, it would appear that the term foliation shear is more appropriate.

Origin of Foliation Shears

One may speculate on the origin of foliation shears. It is likely that severely different processes are responsible — one operating in one set of environmental conditions of stress, temperature, and lithology and others operating in other circumstances. Three modes of origin will be considered.

Differential Movement Associated with Folding

It is well known that in folding the adjacent layers must slide differentially to satisfy kinematics. The differential movement would tend to concentrate in the weaker layers which would be the mica schist layers (or, in other cases, chlorite schist, talcose or graphitic schists, or slates or argillites). The massive, thickly-bedded or foliated gneisses, quartzites, marbles, amphibolites, etc., tend to fracture upon folding with the shearing being concentrated along one or more of the weak schistose interbeds.

The dimensional extent of the shearing would depend on the scale of the fold. For foliation shears to extend from hundreds to a few thousands of feet the extent of the fold limb would have to be of equal or greater dimensions. Smaller drag folds of a few feet amplitude would be expected to produce strictly local shearing (as was recently noted at the foundation excavation for a dam in Brazil).

Under some conditions the weak bed might be sheared and thinned on the flanks of the fold and thickened, contorted, and sheared on the crest. For example, during construction of the Kariba arch dam across the Zambesi river a so-called "mica seam" of contorted, sheared, and pulverized biotite schist was encountered on the right bank in Rhodesia (Lane, 1963, 1964). The seam occurred in the middle of a thick quartzite sequence over one hundred feet thick. On the flank of a fold the sheared mica shist was only 5 feet thick. Where encountered near the trough of the syncline it was 30 feet thick. Since the arch dam was to abut directly across the seam, it had to be mined out by tunnels and shafts and replaced with concrete.

It would appear that the main mode of origin of foliation shears may well be that associated with folding. Igneous intrusions may also introduce stresses in adjacent metamorphic country rock which may induce local warping and differential slippage between beds causing foliation shears as has been postulated for the underground powerhouse foliation shear at Churchill Falls, Labrador (Merritt, 1972).

Stress Relief

Erosion of hundreds to thousands of feet of surface rock results in great stress changes in the underlying rocks. The various rock types will respond differently because of their differences in shear strength and modulus. Lateral movements will be associated with valley cutting and vertical rebound movement with general lowering of the ground surface by erosion. The response of the metamorphic rocks could well induce differential movements between adjacent beds, again with the movement being concentrated in the weak layers.

This mode of origin has been postulated for the shearing of thin shale interbeds in limestone and sandstone sequences forming "shale mylonites" along valley slopes. Some of the foliation shears no doubt have similar origins.

Thrust Faulting

It is possible that some major thrust faults in metamorphic terraines die out in a number of 'horsetail shears'. Some of these might be concentrated along the weak layers of the metamorphic rocks forming the foliation shear zones observed. Also, in isoclinal folding shearing often occurs along the axial planes producing structures similar to the described foliation shears.

Engineering Problems Associated with Foliation Shears

Significant Engineering Geology Features

Emphasis has been given in the past few years by the author in his classes at the University of Illinois (and as visiting professor at the University of Florida) to those few geologic features which have proven to be quite troublesome and costly on many projects. These were termed "significant engineering geology features" and were those that were not only troublesome but that were also very common. They were termed 'significant' because they affected adversely one of the mass properties of the in-situ rock — its *shear strength,* its *compressibility,* or its *permeability.* They included: (1) joints, bedding planes, or foliation planes; (2) shear zones and faults; (3) weathered rock; (4) groundwater conditions; and (5) rock type (or soil type and its pattern of distribution). It is clear that the foliation shear zone is a combination of (1) and (2). Some of the examples in the following sections should reinforce the idea that foliation shears are, indeed, significant engineering geology features.

Engineering Properties

Shear Strength — The foliation shear zone and the rock around it are low in strength, particularly in a direction parallel to the foliation shear. The shear strength in this direction is almost entirely that of the drained residual frictional strength of the gouge material. There is little to no cohesion or interlocking of rock surfaces and, consequently, no high peak strength. The frictional resistance is almost strictly a function of the grain size, shape, and alignment of the gouge and its mineralogy. This could be expected to be no more than 15 °-25°, in general, and less in some circumstances (Cording, Hendron, and Deere, 1971; Patton and Deere, 1971).

Compressibility — The compressibility of the shear zone and its effect on engineering projects would depend on the thickness of the gouge and heavily sheared rock plus the amount of adjacent loosening and fracturing. The gouge material itself might have a modulus of deformation as low as 25,000-50,000 psi and the entire affected zone several feet across of 100,000-250,000 psi, corresponding to very poor to poor rock with RQD values of 0 to 50. At two sites, seismic velocities across the zone were found to be in the 4,000-6,000 feet per second range.

Permeability — The permeability as mentioned previously would be highest parallel to the feature and could be expected to fall in the range of 10^{-3} to 10^{-5} cm per sec.

Concrete Dam Foundations

Arch dams are particularly affected adversely by the presence of foliation shears in the abutments. There may be a problem of potentially excessive deformation because of the low modulus requiring excavation and backfilling with concrete as at the 460-feet high Kariba arch dam in Rhodesia (Lane 1963, 1964), or requiring grouting and pre-stressing of an abutment slab by means of anchored tendons as for a dam recently constructed in Venezuela.

Sliding resistance of an arch dam abutment could also be a serious problem depending upon the orientation of the shear zone with respect to the dam thrust and the ground surface configuration.

Concrete gravity dams may also be adversely affected by the shear zones. The associated low modulus may require that the shear zone material be excavated and backfilled with concrete, as was done to a depth of 60 feet in a recent dam in western United States. Flat-lying foliation shears in the foundation rock of gravity dams would be very critical with respect to sliding stability.

Permeability considerations may also be important for any shear zone cutting across the abutment or beneath the dam foundation — not because of water loss so much as for the potential of piping. In addition to grouting, filtered drainage holes and perhaps weighted filters in the downstream outcroppings may be needed.

Stability of Cut Slopes

Where excavated cuts are made in rock so as to cause the day-lighting of a foliation shear zone, a critical stability condition would obviously be created (Deere and Patton, 1971; Patton and Deere 1970, 1971). Depending on the orientation, failure could occur on the shear zone alone or in combination with another fracture forming a wedge failure. The portal of the tailrace tunnels of Churchill Falls required heavy rock bolting to stabilize foliation slabs, some of which were sheared and all dipping directly down the dip slope of the hill into which the portal cuts had to be made.

Tunnels and Underground Chambers

Tunnels — Tunnels are susceptible to slabbing and slipouts along foliation shears. If the tunnel cuts across the zone at right angles or nearly so, the tunnel is only affected over a distance of a few feet and the weak zone may be handled by a few steel ribs or a nominal amount of rock bolts and shotcrete. This was the case at a few places in the Churchill Falls tailrace tunnels (Benson, Conlon, Merritt, Joli-Coeur, and Deere, 1971; Merritt, 1972).

If the tunnels are sub-parallel to the foliation shear, then the problem is much more extensive as stability problems will first be encountered on one wall of the tunnel, then the roof, and finally across the other wall of the tunnel. Depending on the angle between the strike of the foliation shear and the direction of the tunnel the poor rock condition may continue for several hundred feet and hundreds of steel sets or large quantities of shotcrete and bolts might be required, as in the current water tunnel being driven in New York, or the subway tunnels and stations in Washington, D.C. (For the latter case see Bawa and Bumanis, 1972; Cording and Deere, 1972; and Mahar, Gau, and Cording, 1972).

The effect of tunnel size is also important. It is well known that stability problems with the roof rock increase with size. However, a second geometric effect comes into play when a foliation shear is subparallel to the tunnel. For a larger diameter tunnel the length of tunnel adversely affected is much greater than for a smaller one. For instance, if a 15-feet diameter tunnel were to be affected over a distance of 300 feet, a 30-feet diameter one would be affected over a distance of around 600 feet.

Berkey (1933) in Guidebook No. 9, New York City and Vicinity, for the XVIth International Geological Congress writes as follows regarding City Tunnel No. 2 which was constructed in 1928-1935:

> The tunnel is large. Much of the ground is closely jointed. The tunnel runs nearly parallel with the structure or schistosity for long distances and the schistose structure dips to the side at an unfavorable angle. All these conditions are unfavorable but the chief cause of trouble in construction arose from the fact that the tunnel so nearly parallels the structural trend.
>
> Whereas the rock is somewhat weakened by decay, soft and slippery secondary minerals lubricate the joints so that blocks thus bonded tend to fall out of the roof endangering the workmen and requiring extensive protecting supports for long distances. These conditions were to be expected of course but the extent to which protective measures have had to be used could not be predicted in advance.

Berkey did not specifically call these features foliation shears but there is little doubt but that the described features conform to today's concept of the foliation shear. Inter-office memorandum in 1933 by the project geologist referred to the features encountered while tunneling in the Yonkers gneiss and Manhattan schist by such terms as "shearing parallel to the foliation", "slabby walls", "overbreak along foliation", "shearing in the foliation weaving in and out of the walls", "rock plates in the walls", and "rock dangerous because of large slips along the foliation planes". Again, these features describe the presently used term *foliation shears*.

Tunnels excavated by means of tunnel boring machines (TBM) may also encounter trouble with foliation shear zones. Where the shear zone passes over the tunnel, fall-out is a common problem and immediate roof support must be placed — light half-circle steel ribs or liner plates supported on pins, rock bolts with steel straps, or shotcrete. Where the shear zone goes into the floor the machine may slowly sink by several inches or more. Jack-thrusting may also be a problem where the shear zone intersects the tunnel walls. All these problems occurred to some extent in the few shear zones that were encountered in the recently completed Queen Lane water tunnel in the Wissahickon Schist in Philadelphia and the Interceptor Sewer Tunnel along Riverside Drive in the Manhattan Schist in New York. Both tunnels are 11-feet in diameter and were quite successfully driven with Jarva TBM's.

Underground Chambers — Several large underground powerhouses constructed in recent years in metamorphic rocks have encountered shear zones parallel to the foliation. These include the Morrow Point powerhouse in Colorado (Dodd, 1967), the Oroville powerhouse in California (Kruse, 1971), and the Churchill Falls powerhouse in Labrador (Benson, Conlon, Merritt, Joli-Coeur, and Deere, 1971; Merritt, 1972). In all cases construction was slowed to some extent and remedial measures were necessary, involving one or more of the following: heavy rock-bolting, concrete backfilling with mesh and rock bolts, or tensioned deeply-anchored cables.

The DuPont Metro Station under construction in Washington is as large as most underground powerhouses (approximately 75 feet wide x 700 feet long x 45 feet high) and it has only 35 feet of rock cover, overlain in turn by 30 to 40 feet of soil and fill. Numerous foliation shear zones were encountered in the pilot exploratory tunnel driven prior to bidding along the crown position of the future station. These shear zones cut across the axis of the station at angles of only 20°-30°. Special design and construction measures were taken and extensive instrumentation was carried out (Cording and Deere, 1972; Mahar, Gau, and Cording, 1972).

Figure 2 (after Fig. 8, Cording and Deere, 1972) indicates in cross section the relative position and orientation of the foliation shears and the DuPont Circle Station. Figure 3 (after Fig. 8, Mahar, Gau, and Cording, 1972) shows in plan view the orientation and position of four foliation shear zones in the double track tunnel just south of the station. These were mapped from exposures of three of the shears in the existing tunnel and shaft and from the results of 4 angle core holes from the surface. Later excavation confirmed their presence and relative positions. One of the shear zones led to major fallout of the roof in one area, and to unequal loading and severe distortion of several heavy steel sets.

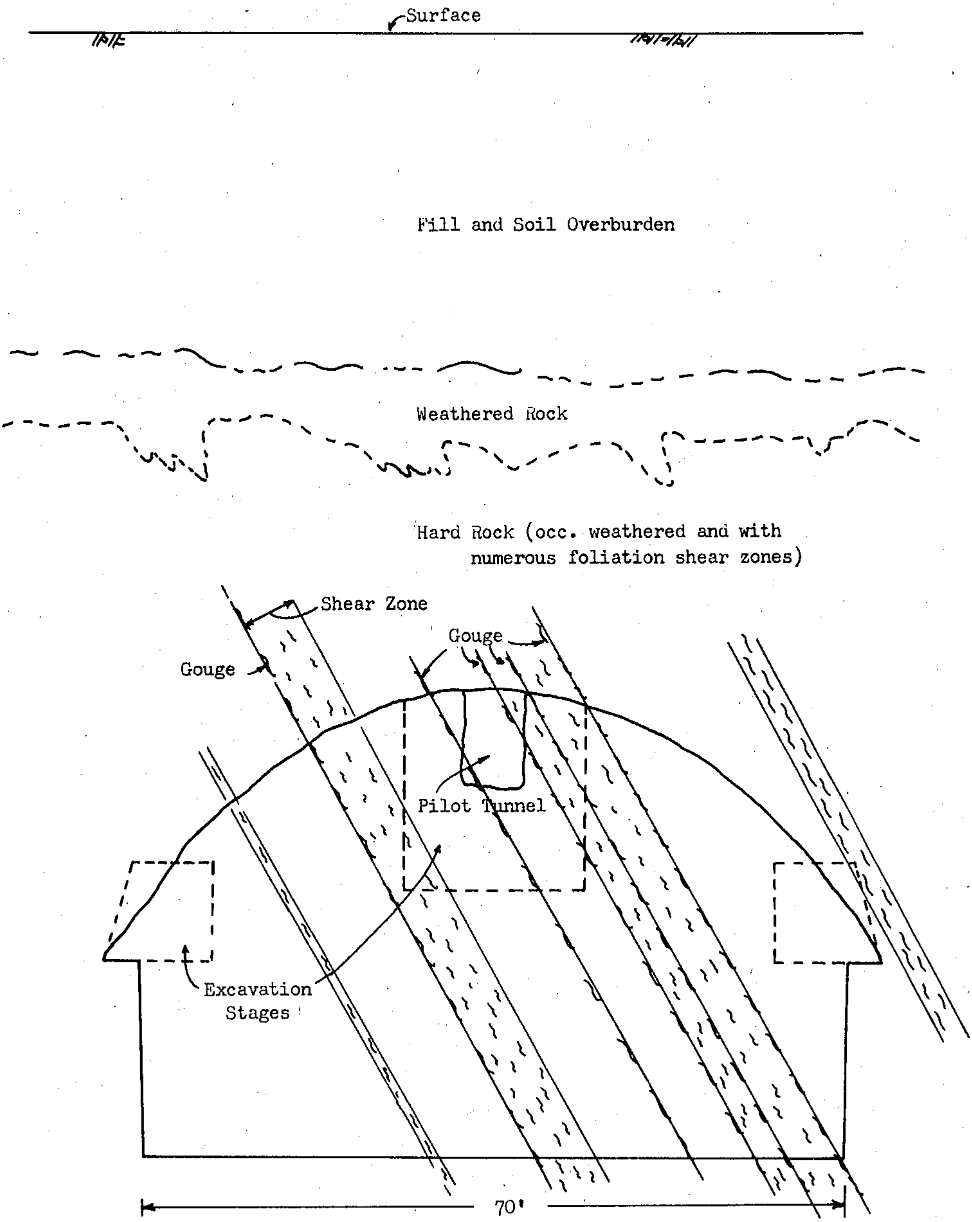

Figure 2 — Cross Section DuPont Circle Metro Station, Washington, D.C., Showing Typical Foliation Shear Zones (Schematic after Cording and Deere, 1972).

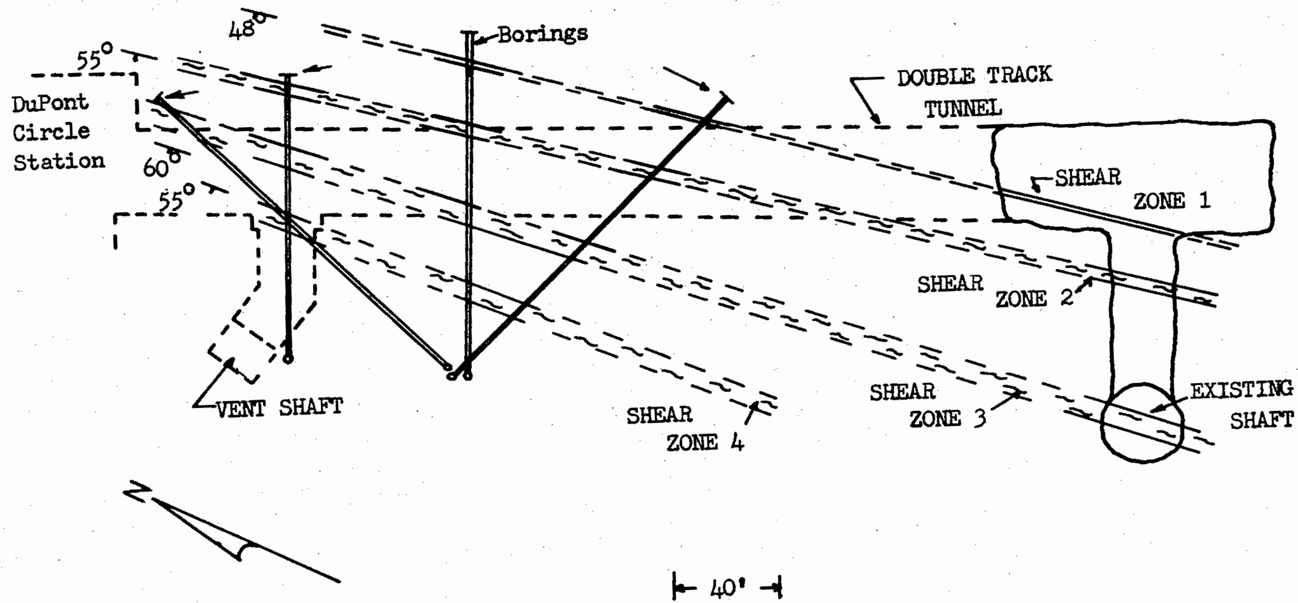

Figure 3 — Four Foliation Shear Zones Mapped from Exploratory Information, Washington, D.C., Metro DuPont Circle Station (after Mahar, Gau, and Cording, 1972).

Conclusions

Foliation shear zones are of common occurrence in metamorphic rocks. Although the gouge zone may be only a few inches wide, the overall affected zone with open or altered joints may be several feet wide and the extent of the zone along its trend may be from hundreds to a few thousands of feet. Differential slippage along weak micaceous interbeds during folding or stress relief probably accounts for the origin of most of them.

Many engineering projects have been adversely affected by the presence of these features because of their low shear strength and modulus. These projects include rock slope excavations, dams, tunnels, and underground chambers.

It behooves the engineering geologist and geotechnical engineer to suspect the presence of foliation shear zones at any site in metamorphic terraine and to devise an appropriate exploratory program to ferret them out or to disprove their presence. For large underground chambers and concrete dams exploratory adits, shafts, and trenches are almost a "must". In other situations, core borings with triple-tube core barrels, borehole photography, or integral sampling (Rocha, 1971; Rocha and Barroso, 1971) should be judiciously used. Down-hole geophysical logging would also be of value.

Once the presence, location, and orientation of a foliation shear is established then its possible affect on the design and construction must be evaluated. Rock mechanics testing, either-in-situ or in the laboratory on undisturbed samples, may be desirable to help evaluate the shearing strength, modulus, and/or permeability.

Special design features may have to be incorporated. However, it is much preferred to do the studies during the design stage than to have to do it during construction as redesign to cope with the sudden appearance of a foliation shear. The purpose of this paper is to direct attention toward the desirability of specifically exploring for the ubiquitous foliation shears and of evaluating in the design phase their possible affect on the project. Experience has shown that a single foliation shear zone can cause serious construction delays and may increase costs from a few hundred thousand to several million dollars.

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ENGINEERING FEES AND THEIR COMPONENTS*

by Charles A. Parthum, member**

It is extremely important for the engineer of today to keep up with technology after he has obtained his degree, not only for his own benefit but also for that of his employer and for his client.

Similarly a knowledge of the business and management aspects of an engineering firm is essential if the engineer is to move up in his work. The young engineers of today will be the leaders in our profession tomorrow and must have an opportunity to learn how an engineering fee is determined, what must be included in it, and the magnitude of costs involved.

By first educating our own engineering family, then through them our clients and the general public, a better understanding of the engineering profession should be achieved.

A fair fee, and one that all consulting engineers need and strive for, is a fee that returns to the consulting engineer the costs for doing the work plus a reasonable profit. The following are the components of a fee.

Basic Wages and Salary Burden = Salary Cost

One of the most important costs is the amount of money that an engineer receives each week in his paycheck; that is the basic wage that is paid the people that work for the consulting engineer. In addition to the basic wage, what is known as "salary burden" becomes a cost to the firm. The salary burden includes paying an engineer's salary when he is sick, on vacation and for holidays, plus unemployment insurance costs, excise and payroll taxes, contributions for social security, workmen's compensation insurance and such fringe benefits as retirement and pension plan coverage, and medical and insurance benefits. As a percentage of direct wages paid to engineers, salary burden can range up to 30 percent or more. In other words, for every $100 in salary an engineer receives, taxes, sick time, vacations, holiday pay, unemployment insurance, retirement benefits, pension plans, medical and life insurance benefits add up to an additional $30 or more to the cost of doing business. Basic wages plus salary burden equals what is commonly known as salary cost.

Other costs in addition to salary cost that apply to a consulting engineering firm include what are called "overhead" costs and consist of *Physical Plant* costs, *Support* costs, *Professional* costs, *New Business* costs and *Miscellaneous* costs.

*Excerpts from talks before NSPE seminar entitled "Do I Want to Manage", and before Mass. Section ASCE in May 1973.

**Senior Vice President Camp Dresser & McKee

Overhead Costs

Physical Plant Costs

The cost of maintaining an office, whether it be a rented office in the downtown part of a city or a building which the consulting engineering firm owns, is a physical cost. The cost of the office includes rent, heat, light, telephone, air conditioning, maintenance, and the fire and other insurance costs that go along with the plant.

Support Costs

Consulting engineering offices must have secretaries, typists, telephone operators, administrative and personnel people, a library and a librarian. Many now also have computers. All the above plus liability insurance, office and drafting supplies, printing, legal and audit work, furniture, laboratory, employment advertising, interest on money due, employee tuition, subscriptions, etc., can all be described as essential support costs.

Professional Costs

The cost of keeping up with the profession is also a cost which the consulting engineering firm must absorb if it is to grow. These costs vary with the consulting engineering firm, but could include such items as the payment of professional engineering dues and the cost of sending people to committee and society meetings including the cost of the man-days as well as the expenses of travel and attendance at such functions. The cost of the man days that the firm "gives up" by sending engineers to various meetings, plus the expenses, are costs that must be figured in the professional overhead of the firm if it is to have an accurate idea of how much money it takes to stay in business.

New Business Costs

The cost of travel, man-days, printing, consulting, meeting, investigating, etc., necessary to prepare proposals and contracts for new or prospective work is a real cost, whether or not the prospective job is obtained. Many times, small services to cities and towns who are long-time clients are done when needed regardless of whether funds are available for payment at that time because a consulting engineer has a professional obligation to keep his client advised of what or what not to do.

Miscellaneous Costs

Not all jobs, unfortunately, make money. Consulting engineers must have enough reserve to absorb losses and to pay for such items as Christmas parties, employee outings and a host of other costs not already mentioned.

Fee Determination

All of the above costs, obviously, must be paid by the consulting engineer regardless of whether all his people are working full time on jobs (in which case all the basic wages paid can be classed as billable wages), or whether some employees are "marking time" or tying up loose ends on jobs (whose wages are not billable). In order to relate costs to a base that brings in funds to cover these costs, they must be related to billable wages. Simply stated, all costs (and profit) must be applied and received from projects for which wages are paid for productive (billable or chargeable) time.

The overhead costs described above, including the physical plant costs, support costs, miscellaneous costs, the professional costs and new business costs, may add up to 120 percent or more of billable wages paid. In other words, the overhead costs may run more than the actual billable wages paid by consulting engineers. This is perhaps a bit difficult to understand, but it is so, and it is one of the facts that clients (cities and towns, smaller cities and towns especially) do not readily appreciate. If the city or town itself had the necessary qualified permanent engineering force, perhaps the consulting engineering profession as it relates to cities and towns would disappear. But the cost of maintaining an updated engineering division in a city or town, the cost of the rental of the space that they would occupy, the cost of equipment, employee benefits and other items, are usually too much for a municipality to support permanently. In other words, whether an engineer works for a consulting firm or private industry, or municipal, state or federal government, the costs are there. Sometimes, the costs may be "hidden" in some other budget, but they are still real costs.

It has been indicated that $30 per $100 on a salary may cover salary burden, and that perhaps $120 or more per $100 of billable wages may cover the overhead, and support and professional costs. This adds up, for every $100 of billable wages, to $250 of costs, but without any allowance for profit. If a consulting engineer hopes to make a 15 percent profit on his services before his own corporation taxes, then he should realize about 15 percent of this $250, or about $40 more. This $40 added on to the $250 totals $290. In other words, in this example, for every $100 of basic billable wages paid to an engineer, about 2.9 times that is needed in order to stay in business.

Smaller firms with less overhead may be able to make a reasonable profit at less than 2.9 markup. Many larger firms with more fixed costs (and the fact that a firm has more fixed costs doesn't mean it is more inefficient because the firm may be spending more to improve itself in the field) may require that the engineering fee be more than 2.9 times basic wages.

This is one reason why professional engineers do not competitively bid on the basis of cost for projects. A cost may be obtained from an engineering firm that is not as proficient in the work expected to be done mainly because it has not spent the money necessary to keep itself, its employees or its office abreast of the developments in the profession. What any client wants is the best work done, and therefore, should select a consulting engineer on the basis of experience, qualifications and reputation, and then negotiate a reasonable scope of work and fee. The ASCE Manual No. 45 indicates all of its bases of reasonable fees should be used as a guide, — and the work "guide" should be emphasized by the profession in dealing with prospective clients who have little knowledge of what is involved in engineering or in the costs of engineering.

In summary, using the above example, no matter what type of fee arrangement is used on a job, a consulting engineer cannot afford to pay out in wages and direct job expenses more than about $34,500 on a project for which he will get $100,000 in fees (2.9 x 34,500 = 100,000) if he is to make a 15 percent profit before taxes.

Major Types of Consulting Work

There are four major areas of the consulting engineering business. Although there are many off-shoots and many other types of work done by consulting engineers, the major areas of work are:

1. Preliminary engineering investigations and reports

2. The preparation of design construction plans and specifications

3. The administration of the construction during the construction phase, and

4. A newer item that is becoming more important in municipal facility work, the start-up and operation including the preparation of operating manuals for the client.

The method of payment for these different parts of consulting work varies. It also may vary with the client, or it may vary because of federal or state grants in aid being offered to pay a portion of the work.

Different Fee Bases

The different types of fees and how to determine them, and the portions of the major areas of consulting engineering to which they usually apply are as follows:

Per Diem Fee

The per diem fee, familiar to all of us, is simply so much per day for the different grades of individuals working on the job. Per diem fees are normally figured in the same manner as discussed above. If an engineer gets $250 a week, the consulting engineer would have to charge about $725 (250 x 2.9) a week for his services. Divided by five, this means $145 a day for his services. That sounds like a great deal of money to the $10,000 a year municipal employee, and it takes a bit of discussion to make him realize that $145 a day isn't the engineer's salary. Per diem fees are used in many types of consulting engineering services where the scope of the work cannot be defined exactly or even anywhere near exactly. Most report work, investigative work, or preliminary engineering work is done on a per diem basis, many times with an upper limit. If an upper limit is required, a consulting engineer uses his experience and judgment and "track record" on previous similar investigations to determine what is a reasonable upper limit, keeping in mind all the time that whether or not the upper limit is high enough, he will have to do a complete job or run the risk of losing his reputation and his client. A per diem type of arrangement is flexible enough so if half way through an investigation other aspects should be investigated or more emphasis put on other parts of the study, the consulting engineer can more easily do this.

If a special project, such as expert testimony, will involve senior people in a firm, per diem fees are sometimes higher than standard because the people involved are "tied up", and their normal duties are interfered with.

Multiple of Salary Fee

In many ways, this type of fee is the same as a per diem fee except that instead of spelling out so much per day per grade of engineer, the multiplier of basic wages, or salary cost plus a multiplier of salary cost, is indicated. This method allows the consulting engineer a little flexibility because increases in wages are automatically passed on to the client (as long as the upper limit is not exceeded).

Cost Plus a Fixed Fee and Cost Plus a Percentage Fee

These two types of fees are very similar and are the sum of salary costs and normal overhead, plus a fixed fee for profit in the one instance or plus a percentage of cost for profit in the other. When a cost plus a fixed fee or a cost plus a percentage is used as the basis for doing work, reimbursable costs must be carefully defined in advance so that there is no misunderstanding about whether they are or they are not included in the fixed fee, the percentage, or whether these are additional costs for which the client should reimburse the consulting engineer with or without a separate markup.

The above four fee arrangements are commonly used in the preliminary engineering investigative phase, for pilot plant work, or other special investigations. Once in a while, if the scope of work for a preliminary report can be defined in detail and is specific enough so that there is no misunderstanding and everything is covered, the consulting engineer may agree to a lump sum payment for the work.

Percentage of Construction Cost

The preparation of detailed construction plans and specifications is one of the larger (fee wise) types of work in which the consulting engineer becomes involved. Usually the final design is the result of a preliminary report which he has prepared for the same client, and therefore the requirements of the work are known. Here again, the consulting engineer must set up a fee that will be reasonable to cover all costs plus a reasonable profit. The ASCE Manual No. 45 contains median curves for two different types of work: Curve A is a curve that will give a higher percentage of construction costs than Curve B, and the types of work applicable to each curve are also indicated. It may be difficult for people to understand why fees from a curve can be applicable to many, many jobs. They are not. However, these curves are offered to the general public by ASCE as a guide so that clients not familiar with consulting work or fees can determine whether the fee requested in a specific instance is reasonable or whether more definition or more explanation of it is needed. Naturally, on any project, a consulting engineer cannot determine a fee blindly from these curves-but should always check it by some other means.

Final design work is often done on a percentage of construction cost or on a lump sum arrangement.

During the construction phase, the contract administration portion of a consulting engineer's work, which includes attendance at bid openings, tabulating, checking, interviewing and recommending contract awards, the reviewing and checking of shop drawings, the general administration of the contract, the preparation of periodic payments and certifying that they are due the contractor, taking care of the many items during construction for the client, and the overseeing of the final testing are paid for in many instances as another percent of construction cost.

Other costs such as the cost of a resident engineer or inspector on the project and for start-up and operation are usually paid for on a per diem or a cost times a multiplier plus expenses because of the difficulty in determining how long these services are needed. The type of contractor, his efficiency, strikes, acts of God and other such items can delay construction and require the engineer or inspector to be on the job for many more weeks or months than what could be anticipated beforehand.

General Comments

Some firms allow the prospective project engineer for the job, wherever possible, to work up the proposed engineering agreement and fee. In this manner, he has more involvement with the fee with which he will have to work. It also gives him experience in determining where the costs are and makes him consider all of the things that might be involved.

Finally, the writing of an engineering agreement is just as important as the fee. The fee should cover what the agreement says will be done. The agreement must be clear about payment for items directly connected with engineering work but involving necessary work to be done by others such as surveying, borings, material testing, etc., and whether they are in the fee or whether they are extra items that will be paid for separately, or whether the client will furnish them itself. That's only logical. But, if this is forgotten, the least that will happen is that relations with the client will deteriorate, and many times money will be lost. Clients are looking to the consulting engineer to guide them, to advise them, and to do all that is necessary to get the job done. There is good faith involved, and if the agreement does not spell out and make it entirely clear as to who pays for what all the way through, everyone concerned will wish it had.

It is difficult to arrive at a fair fee that returns to the consulting engineer all the costs for doing the work plus a reasonable profit, but it is essential.

PROCEEDINGS OF THE SOCIETY

Minutes of Meetings

BOSTON SOCIETY OF CIVIL ENGINEERS

September 12, 1973 — A joint meeting of the American Society of Civil Engineers (Mass. Section) with the Boston Society of Civil Engineers was held at the Branding Iron Restaurant, 75 Blossom Court, Boston. Following a social hour and luncheon, the meeting was called to order at about 12:45 p.m. by President Hirshfeld of the Massachusetts Section, A.S.C.E. Following a few brief announcements, President Hirshfeld called upon the Secretary of B.S.C.E. to conduct any Society business.

The Secretary announced receipt of applications from the following:

For Member: Stephen Alsup, John Dewsnap, Roy Fedotoff, Joseph Kerrissey, Jr., Alden Jenkins, Donald McAllister, H. Gordon O'Reilly, Jr. and Robert Vallee.

For transfer to Member: Thomas Hurley, Leland Jenkins, Thomas Turton and Charles S. Young, III.

For Junior Member: James Flagg and Kevin McRae.

The members then observed a moment of silence for deceased member Lawrence S. Burke.

Mr. Cranston Rogers, National Director of American Society of Civil Engineers made a few brief introductory remarks on the subject matter of the program, which was a panel discussion of Retirement and Pension Systems.

The first speaker, Mr. Pat Brennan of Tippetts, Abbett, McCarthy and Stratton spoke for Mr. Daniel Reisner on the existing profit sharing (retirement) plan of his firm. This was followed by a description of typical retirement systems, tax implications, and future trends by Mr. James Moynihan of State Mutual Life Assurance Co. of America. The final speaker, Mr. Edmund Lang, Director of Professional Services from the American Society of Civil Engineers' National Headquarters gave a description of the A.S.C.E. interest and viewpoint on current and future policies of such systems.

The speakers answered various questions from the floor after their presentations.

Adjournment was at 2:00 p.m.

Approximately 50 members and guests attended the meeting.

Joseph F. Willard
Secretary

October 24, 1973 — The Annual Student Night Joint Meeting of the Boston Society of Civil Engineers and the Massachusetts Section of the American Society of Civil Engineers with Student Chapters was held at Northeastern University, Boston, following dinner. The meeting was called to order by Vice President Liu of the American Society of Civil Engineers, at 7:35 p.m. After introducing the head table, Dr. Liu turned the meeting over to President Sorota of the Boston Society of Civil Engineers. Following welcoming remarks, Mr. Sorota called upon the Secretary to conduct any Society business. The Secretary read announcements of forthcoming meetings and, then, read the names of new members elected by the Board of Government on October 17, 1973 as follows:

For Member: Stephen A. Alsup, Robert H. Fitzgerald, and Robert Peter Vallee.

For transfer from Junior to Member: David F. Doyle, Bruce O. Tobiasson, and Thomas H. Turton.

Applications for membership from the following were announced: Edward L. Byrne, Herbert H. Einstein, and David L. Freed.

This concluded the Boston Society of Civil Engineers' business meeting.

Dr. Liu, then, introduced Mr. Cranston Rogers, District Director of the American Society of Civil Engineers, who made a few timely comments on ethics. Mr. Thomas Flynn, President of the Northeastern University Student Chapter, also made a few remarks on the same subject.

The next order of business was the awarding of the 50th Anniversary Certificates to the Student Chapters from Massachusetts Institute of Technology and

Worcester Polytechnic Institute granted by the American Society of Civil Engineers' Board of Direction. This was followed by awards of the 1972 Certificates of Commendation to the Student Chapters from Merrimac College, Northeastern University, and the University of Massachusetts.

Mr. Lee Worth was called upon to moderate a debate on "Nuclear Power Plants Versus the Environment." Dr. James McKenzie, of the Audubon Society, gave the Environmental viewpoint, while Mr. Lawrence Minnick, of Yankee Atomic Electric Company, gave the power plant viewpoint. After the presentations and rebuttals, the meeting was opened to a general question and answer period with a number of excellent questions being raised from the floor.

Approximately 200 members, students, and guests attended the dinner and meeting, which was adjourned at about 9:35 p.m.

Joseph F. Willard
Secretary

CONSTRUCTION SECTION

October 17, 1973 — The Regular Construction Section luncheon meeting was held at the Red Coach Grill, 43 Stanhope Street, Boston. The meeting was called to order at 12:15 p.m.

Mr. Robert Norton of Metcalf & Eddy was the guest speaker. The theme was "Construction of Large Sewers in Urban Areas". A recently completed sewer construction project in Salem & Peabody was the subject under discussion.

A total of 37 members and guests were in attendance.

The meeting adjourned at 2:00 p.m.

Laimonis Rieksts
Clerk

ENVIRONMENTAL SECTION

October 30, 1973 — The Environmental Section of the Boston Society of Civil Engineers met at the Playboy Club in Boston. There were 36 in attendance.

The meeting commenced with cocktails from 5:30 p.m. to 6:30 p.m., followed by dinner served from 6:30 p.m. to 7:30 p.m. At approximately 7:30 p.m. Chairman Guertin introduced Dr. Otis Sproul of the University of Maine, who talked on the Inactivation of Viruses. Dr. Sproul's talk went on for approximately one hour, followed by a question and answer period, which extended up until approximately 9:00 p.m. when the meeting adjourned.

Paul D. Guertin
Chairman

STRUCTURAL SECTION

October 3, 1973 — The October 3rd meeting of the BSCE Structural Section, held in the auditorium of the Boston Public Library, was opened at 7:00 p.m. by R. Zallen, Chairman. After a number of announcements by Max Sarota, BSCE president, Mr. Joseph Jones of the AISC presented to W.J. Le Messurier a special citation recognizing his contributions to the design of steel frame structures.

In the technical portion of the meeting, Professor Lynn Beedle, Director of the Fritz Engineering Laboratory at Lehigh University, gave the T.R. Higgin Lecture on the results of recent research on beam-column and connection behavior.

The meeting, attended by approximately 100, was adjourned at 8:30.

Kenneth M. Leet

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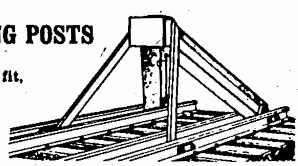

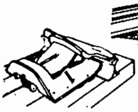

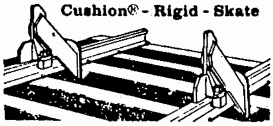

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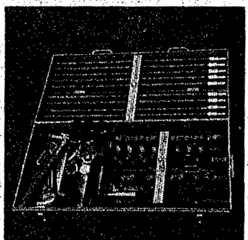

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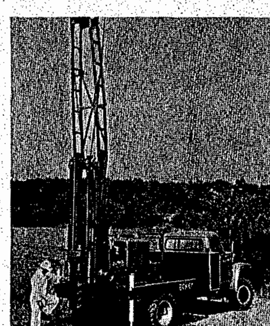

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