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CONSULTANTS, CLIENTS AND CONTRACTORS

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(Presented at a joint meeting of the Boston Society of Civil Engineers and Massachusetts Section, A.S.C.E., held on October 23, 1957.)

Synopsis

In this paper the writer describes some of his experiences as a consultant to engineering organizations on five continents, on projects involving large earthwork operations. Special emphasis is given to the factors which may lead to partial or complete failure of a project in spite of sound advice rendered by the consultant.

Introduction

A consultant is a person who is supposed to know more about a subject under consideration than his client. Once an engineer has acquired a reputation for superior knowledge and discovers that there is a demand for his services, his future career depends upon what he expects to get out of life. If he longs for financial success and social prestige, he will find that his aims can hardly be satisfied without the assistance of an engineering organization. Once the organization exists he becomes a slave to it. His income increases, but so do his worries. Sometimes he has sleepless nights because he does not know how to handle all the orders which have rained into his lap, and at other times, because, his overhead charges begin to exceed his income. In any event, the Tax Collector sees to it that his income does not assume staggering proportions. He may still believe that he is a consultant, but in reality he has turned into a business man and executive, equipped with all the prerequisites for stomach ulcers.

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On the other hand, if he derives his principal satisfaction from practicing the art of engineering, he will desist from establishing an organization and concentrate all his efforts on broadening his knowledge in the field of his choice. In order to be successful in this pursuit he must be not only willing but eager to spend at least half of his time on unprofitable occupations such as research or the digest of his observational data. Therefore, his money-making capacity remains limited, but in exchange he has fewer worries and retains his freedom of action. That is the type of occupation which turned out to agree with my disposition.

Initiation Into the Consulting Fraternity

I never felt tempted to make a blueprint for my professional career, except inasmuch as I always considered the performance of work for the mere sake of earning money a waste of time and acted accordingly, quite often on the spur of the moment. Therefore, I did not join the consulting fraternity deliberately. I was dragged into it by accident and discovered afterwards that it was amazingly congenial. This happened about thirty-five years ago.

I was then forty years old and I was teaching applied mechanics and related subjects at the American Robert College in Istanbul. However, I spent most of my time on research concerning the physical properties of sedimentary deposits such as sand and clay. My interest in this field had been acquired in earlier years, while I was still engaged in the practice of earthwork engineering. During those years I became more and more impressed by our incapacity to predict the performance of soils under field conditions, and my affiliation with Robert College gave me a welcome opportunity to search for a remedy.

My research activities had no relation to my duties as a teacher, and they yielded no financial compensation. Yet I felt perfectly happy because I earned enough to live on and my venture into the unknown was so exhilarating that I felt no desire to exchange it for a more profitable occupation.

At the time referred to, I was engaged in digesting the results of my investigations concerning the consolidation of clay strata. In connection with this occupation I visited an industrial plant located at the head of the drowned portion of a valley in the proximity of Istanbul, because I was told that an open excavation was being made at the site of the plant. From the general geology of the region I knew that the subsoil of the plant consists of a deposit of soft silt and clay, with a maximum depth of several hundred feet, and I wanted to collect some specimens to be tested in my make-shift laboratory.

When I arrived at the site, I found, in addition to the excavation, a heap of precast reinforced concrete piles and the setup for a pile loading test. This fact aroused my interest, because I knew that the predecessors to the new structure rested on mat foundations. Therefore, I called on the general manager of the plant, whom I had met socially, and asked him to explain the project to me.

According to the construction drawings which I was shown, one-half of the proposed structure would have rested on point-bearing piles and the other half on friction piles embedded in soft clay. I was shocked and explained to the manager that the piles would do more harm than good. The portion resting on point-bearing piles is rigidly supported, whereas the portion on friction piles would settle at least several inches, whereupon the pile-supported foundation would fail like an overloaded cantilever beam by bending. Therefore, I suggested that the Company should sell the piles or throw them into the Bosporus.

After lengthy discussions, the manager began to realize the weight of my arguments but, he added, he would never succeed in inducing the design department of his organization, with headquarters in France, to accept my unconventional proposal. Therefore, he invited me to make a trip to France and try it myself. At the headquarters of the organization I, an obscure teacher, faced engineers with a well-established reputation, full of confidence in the soundness of their judgment. My arguments were received with utmost skepticism. The pile loading test had already shown that the settlement of the friction piles under the design load was negligible and, as a consequence, my pessimistic settlement forecast was considered to be wrong. Nevertheless, the mere existence of arguments in favor of the gloomy prediction made the designers of the foundation somewhat uneasy. Therefore, a compromise solution was proposed and accepted. The piles were retained as part of the foundation, but the site of the building was shifted away from the slope, whereupon all the piles assumed the function of friction piles.

I left France with the conviction that the structure would settle as if the piles had not been driven, whereas my clients believed that the results of the settlement observations would demonstrate the absurdity of my settlement estimate. The preceding controversies were very instructive and suited my tastes to perfection. Thus I had discovered an interesting field for the practical application of the results of my research activities and I wished to get more assignments of a similar kind.

I did not have to wait very long, because as soon as the structure under consideration was completed, it started to settle approximately at the rate predicted by me, whereupon confidence in my judgment was established and the usefulness of my professional services was recognized.

A few months after I had returned to Istanbul, the manager of the plant showed me the settlement record of the older portion of the plant. The structures were at that time about twenty years old and all of them rested on reinforced concrete mats. According to the settlement forecast of the designers, based on the results of surface loading tests, the foundations should have settled by amounts not in excess of half an inch. In reality the settlement of the structures had reached a value of 16 inches. That was the reason why it was intended to establish the new building on a pile foundation. Yet the performance of the new structure showed that piles had practically no influence on the settlement of structures resting on the subsoil of the plant.

At the time when the new building was started the rate of settlement of the older ones had already become insignificant. However, while the new plant was under construction the rate of settlement of the existing structures increased again to several inches per year, and I was asked to investigate the causes of this surprising development. Upon inquiry, I found out that the rate of settlement of the old structures had started to increase at almost exactly the time when the sinking of a nearby caisson well was completed and pumping operations were started. The water was drawn from a gravel stratum located between the clay stratum and the surface of the underlying bedrock. At the time of the inquiry I already had a clear conception of the mechanics of the consolidation of clay strata and there was no doubt in my mind that the increase of the rate of settlement was due to the reduction of the porewater pressure in the gravel stratum, produced by the pumping operations. Therefore, I had no difficulty in persuading the management to plug the well. As soon as this was done, the rate of settlement again became inconsequential.

Immediately after the well was plugged, the foundation of the crane rail of a revolving derrick located at the waterfront of the plant site started to settle unequally, at an alarming rate, although the settlement of the rail had previously been too small to be noticed. The crane rail was mounted on a semi-circular platform, resting on untreated timber piles acting as friction piles embedded in soft clay. The space between the original ground surface, a few feet below low tide level, and the base of the platform, about seven feet above this level, consisted of an uncompacted cinder fill.

At the time the settlement started, the manager of the plant was already fully aware of the undesirable properties of the clay deposit underlying his plant. Therefore, he blamed the performance of the derrick foundation on the clay and asked me to remedy the situation by underpinning, or some other suitable procedure. However, the history of the settlement of the derrick foundation appeared to me to be incompatible with the consolidation characteristics of the clay. Therefore, I started my investigations at the platform. A single test pit excavated at the edge of the platform through the fill to the original ground surface sufficed to show that the clay was innocent, and the culprit was the teredo. Above low tide level, portions of the piles, with a diameter of 14 to 16 in., were almost completely destroyed by the marine borers. After the fill was removed, some of the piles could even be knocked over by the laborers. Yet below the teredo-infested top section the piles were perfectly sound. Therefore, the reconstruction of the derrick foundation was performed by cutting the piles below low tide level and establishing the reinforced concrete rail support on the intact portion of the piles.

Important Consequences of Casual Observations

The assignments described under the preceding heading are typical of many others I had to handle during the following decades in various parts of Europe, the United States and North Africa. However, quite often the most essential services I rendered to my clients had no relationship to the original assignment. They grew out of casual observations I made while inspecting the site. The observations at the site of a multiple-arch dam are an example.

One of the buttresses of the dam had cracked and I was asked to make proposals for protecting it against further damage. At the time of my arrival at the site the reservoir was still empty. I found that the outer parts of the base of the buttress rested on sandstone and the middle part on shale. The cracks were produced by the compression of the sale and further damage could be prevented by a simple underpinning operation.

As a by-product of my visit to the site I noticed the following facts which had previously not received any attention. The shale bed responsible for the unequal settlement of the buttress formed part of a stratified formation composed of practically impervious layers of shale and intensely jointed beds of hard sandstone. The strike of the bedding planes intersected the direction of the crest of the dam at approximately right angles and the dip was approximately equal to that of the dip slope of the valley at the site of the dam.

A few hundred feet upstream from the left abutment, the left hand slope of the valley cut across the sandstone strata, providing free communication between the water in the reservoir and the joint system in the sandstone, whereas downstream from the dam the uppermost sandstone stratum was covered by a shalebed. Hence, while the reservoir was being filled the hydrostatic pressure on the base of the sloping shalebed would increase, and before the reservoir was full, the shalebed would be lifted off its seat and the dam would fail.

As a result of this discovery, the settlement of the buttress became a minor issue and the center of gravity of the problem shifted to the hydrostatic pressure conditions prevailing in the joint system of the rock strata underlying the site. By similar casual observations during construction, which had no direct connection with my assignments, I also prevented the failure of three major dams of the earth and rock-fill type.

Design Assumptions and Field Conditions

The assignments referred to under the preceding headings have one essential feature in common. In each case an engineering organization was in serious trouble and therefore willing to accept the consultant's recommendations. If the consultant is invited to cooperate on a project before unanticipated difficulties have been encountered, conditions may be radically different. This is due to the fact that some engineering organizations are subdivided into three independent compartments—the survey, design, and construction departments—or else they assign the supervision of construction to inspectors who have neither the duty nor the qualifications to judge whether or not the design assumptions are compatible with the field conditions.

The survey department is in charge of the topographic survey and the subsoil exploration by borings. The results of their labors are represented in a set of drawings which are turned over to the design department. The engineer in charge of the design may have visited the site of the proposed structure a couple of times, but the principal source of his information concerning the subsoil conditions is the boring records. These are accepted at face value, sometimes even without any inquiries concerning the qualifications of the personnel engaged in the boring operations. The draftsmen who prepare the construction drawings have not even seen the site. After the drawings are completed, "checked" and approved, they are transmitted, together with a set of specifications, to the construction department, whereupon the association of the design department with the project is practically terminated. The construction department receives orders to erect the structure in accordance with drawings and specifications, and has no obligation to make any inquiries concerning the design. Similar conditions prevail if the functions of the construction department are assigned to a group of inspectors who have not been connected with the design of the project.

In connection with structural engineering this administrative setup is perfectly satisfactory, provided the engineers in charge of design are reasonably familiar with the methods of construction. On the other hand, in the realm of earthwork and foundation engineering the absence of continuous and well organized contacts between the design department and the men in charge of the supervision of the construction operations is always objectionable and can even be disastrous. This is due to the fact that boring records always leave a wide margin for interpretation. If the site for a proposed structure is located on a deposit with an erratic pattern of stratification, such as a marginal glacial deposit, the boring records may not disclose a single one of the vital subsoil characteristics, and the real subsoil conditions may be radically different from what the designer believed them to be. Therefore, the design assumptions may be utterly at variance with reality.

The consequences of these conditions depend on the qualifications of the personnel engaged in the supervision of the construction operations. If the supervision is in the hands of a construction department it also depends to a large extent on whether or not design and construction departments are on friendly terms with each other. More often than not the two departments despise each other sincerely, because their members have different backgrounds and different mentalities. The construction men blame the design personnel for paying no attention to the construction angle of their projects, but they are blissfully unaware of their own shortcomings. The design engineers claim that the construction men have no conception of the reasoning behind their design, but they forget that the same end in design can be achieved by various means, some of which can be easily realized in the field, whereas others may be almost impracticable. If none of the men in charge of design has previously been engaged in construction, the design may be unnecessarily awkward from a construction point of view. In any event, the construction men have no incentive to find out whether or not the design assumptions are in accordance with what they experience in the field during construction, and serious discrepancies may pass unnoticed. If conditions are encountered which require local modifications of the original design, the construction engineer may make these changes in accordance with his own judgment, which he believes is sound, although it may be very poor. Important changes of this kind have even been made on the job without indicating the change on the field set of construction drawings.

Furthermore, the layout of temporary installations is commonly left to the discretion of the superintendent of construction. The drainage provisions for unwatering the site for an earthdam and those for the disposal of the water coming out of a wet tunnel belong to this category.

The drainage provisions for unwatering the site for an earthdam prior to the beginning of the filling operations may introduce an element of serious weakness into the structure without the superintendent of construction suspecting it. In one instance the box drains leading to the sumps at the site for an earthdam were laid out in such a manner that the completed structure would have failed by piping through the drains. When I arrived at the site the drains were already buried beneath fill material and no record was kept of the location of the drains. Fortunately, the thickness of the layer of fill located above the drains was still moderate. After I reconstructed the layout of the drains on the basis of the results of cross-examination, it was not yet too late to eliminate the sources of weakness represented by the drains.

On another project the excavation for a powerhouse was being made at the foot of a forest-covered talus slope. The talus consisted of a mixture of rubble and the sandy and silty products of rock weathering. The slope rose at the angle of repose of the talus material to the exit of a wet tunnel at an elevation of about one thousand feet. The water coming out of the tunnel was allowed to flow into the uppermost portion of the accumulation of talus, where most of it disappeared into the voids of the material. When the quantity of discharge reached a value of about 3 cfs a talus slide occurred. The slide removed the forest cover of the slope, killed two men who were working in the excavation and filled the excavation with a mixture of rocks and trees. Subsequent investigation showed that neither the resident engineer representing the construction department, nor the contractor's superintendent of construction had suspected that the flow of water into the uppermost part of the talus slope could have disastrous consequences.

Such can be the qualification of the men who are placed in responsible charge of erecting a structure "in accordance with drawings and specifications." If the field conditions are radically different from the design assumptions they may not even notice it. The following example illustrates the possible consequences of the failure of a field inspector to pay any attention to the design assumptions. The project involved the construction of a tall reinforced-concrete structure. The site was located above a steep rock slope which was subsequently buried in succession under a blanket of gravel, a layer of soft clay, a peat deposit and artificial fill. The site was explored by borings spaced 50 feet both ways. According to the soil profile which was constructed on the basis of the boring records, the surface of contact between the gravel structure and the overlying soft and highly compressible sediments was well defined and fairly even. Therefore, the design department decided to establish the structure on spread footings supported to pointbearing piles to be driven through the soft sediments to refusal in the gravel blanket.

When the piles were driven, the total depth of penetration varied within each cluster by amounts up to 16 feet. Yet the superintendent did not notice that this fact is incompatible with the design assumption. After all the piles were driven and the structure almost completed, the structure started to settle unequally by amounts up to one inch per month. It was not until then that the abnormal performance of the piles was brought to the attention of the design department. Subsequent investigation showed that the thickness of the gravel stratum was very much greater than the original borings indicated and that it contained thick lenses of soft clay. The bearing capacity of the individual piles was much greater than the design load, and the settlement was exclusively due to the consolidation of clay lenses. Some of the piles had met refusal in the gravel above a clay lens and others went through several clay lenses into the lower portion of the gravel stratum. This was the reason for the erratic variation in the total depth of penetration of the piles. If this variation had been brought to the attention of the design department as soon as it was observed, the causes would have been investigated and the pile driving procedure modified in such a manner that all the piles could be driven to bedrock.

Performance by the Contractor

If a job is carried out on a contract basis, one more element of uncertainty enters into the operation. It is the attitude of the contractor towards his work. The contractor cannot be expected to be interested, or even aware of, the reasoning behind the design. His sole aim is to perform the work covered by the contract at a minimum expense. (Occasional discrete departures from the specifications reduce the cost quite considerably.) The inspectors, too, may be inclined to consider uncomfortable items in the specifications as superfluous refinements, conceived in the hothouse atmosphere of the design department. Such an attitude is not conducive to rigorous inspection. Therefore, a consultant can never be sure how a structure was built unless he maintains continuous contact with the construction operations. To illustrate this statement the writer adds an account of some observations he made during the construction of a dam resting on decomposed rock.

The dam was a clay dam with internal drainage supplemented by a row of filter wells which were drilled through the decomposed rock into sound, jointed rock at a depth ranging between 40 and 90 feet. In order to coordinate the construction operations with the time schedule, the upstream portion of the embankment was constructed before the filling operations on the downstream side of the base of the dam were started.

The dam was built by a contractor with considerable experience in the field of earth dam construction. Yet every one of the memoranda I wrote describing my findings at the site after returning from my inspection trips contained passages like the following:

"At the site of the dam, the cutoff trench was already backfilled. Along the west slope of the first installment of the fill, the new fill was placed against older, dried out and uncompacted material. The gradient of the surface of the new fill was such that the next rainstorm will produce a pool in the northeast corner of the new fill. The pumping equipment is inadequate. Although the job calls for a large amount of hand tamping, the contractor has made no provisions for tamping equipment. On the upper level, in the upstream portion of the dam, filling operations should be discontinued because the water content of the borrowpit material is at present too low and the contractor has made no provisions for sprinkling.

"At the southeast corner the contractor has blocked the exit for the accumulating rainwater by a pile of waste material. Originally the lowest point of the saddle southeast of the site was 505. Now it is already 508.5 and the diversion of the rainwater towards the southeast will require a substantial amount of excavation which could have been avoided by intelligent planning.

"In my last memorandum I requested that the north end of the cutoff trench should be excavated down to decomposed rock. The inspector assured me that he has passed this request on to the contractors. Nevertheless, I found that the fill was placed against the pocket of very permeable alluvial materials.

"If the contractors continue to disregard the elementary rules for the construction of earth dams and to ignore the instructions of the inspector wherever they can, the resulting structure will be unsafe in spite of conservative design."

Consultants or Scapegoats

Conditions like those described under the preceding heading prevailed on many of the projects with which I was associated in the course of my professional career. In some instances they were considerably worse. Hence it is evident that the success of large-scale earthwork operations depends on many factors other than the adequacy of the original design. This fact introduces serious complications into the relationship between the client and a consultant who is retained in an advisory capacity in the design stage of a project.

The incentive for retaining a consultant commonly grows out of the fact that the functions of most engineering organizations cover a very broad field, including earthwork, structural, hydraulic, mechanical, and electrical engineering. Few, if any, of the members of such an organization have the time and the opportunity to specialize.

Hence, if a new project assigned to such an organization involves design problems of an unusual character, a consultant is retained who is expected to cooperate in the solution of the problems.

In engineering organizations with a watertight partition between designers and the personnel engaged in construction, the consultant is quite obviously placed at the disposal of the design department. After the design is completed his service period on the project, like that of the design department is considered terminated. He has no control over what the inspectors and the contractor chose to make out of the drawings and specifications, and he cannot even know whether or not the men on the job are competent enough to notice significant differences between design assumptions and field conditions. If the engineering firm does not maintain a construction department, or if the owner reserves the right to supervise construction, conditions may be even worse. The consequences depend on the type of service the consultant was asked to render, as shown by the following examples:

(a) The client requests the consultant to participate in the design of a structure and in the drafting of the specifications. He has the sincere intention of acting in accordance with the consultant's recommendations, but the service period of the consultant ends as soon as construction starts. The consultant's advice cannot be sounder than his knowledge of the subsoil conditions at the time when the advice was rendered. If these conditions are radically different from what the boring records indicated—which is by no means uncommon—the structure may fail in spite of conscientious adherence to the consultant's advice.

(b) The client invites the consultant to make proposals concerning design and construction, but he reserves—or assumes—the right to deviate from the recommendations as he deems fit, without informing the consultant about the final decision concerning the design. If this decision is the result of misjudgment or ignorance, the consultant is unable to prevent its consequences.

(c) The consultant gets the assignment of participating in the design of a small portion of a large unit, e.g. the design of the core for an earthdam which has been designed by others. If the structure fails on account of conditions which have no bearing on the performance of the portion investigated by the consultant, this portion goes with it, and after failure it may be impracticable to find out which part failed first.

(d) The consultant is asked to express an opinion concerning the design of a structure without being given an opportunity to make a thorough investigation of all those field conditions which determine the performance and safety of the structure. The consultant's opinion may be sound or unwarranted, depending on circumstances unknown to all the parties involved.

(e) An engineering firm requests a consultant to participate in the preliminary stage of a large project merely for the purpose of using his name as window dressing. If and when the firm gets the job, the consultant is shelved and remains in his state of retirement unless something goes wrong. After the shortcomings of the design have become noticeable it may be too late to correct the mistakes.

In each one of these five cases the name of the consultant remains permanently associated with the project. Proceeding from case (a) to case (e) the hazards to the good reputation of the consultant increase. In any event, if the project ends in disaster the consultant will find himself in the front row of scapegoats, because he was introduced to the owner as the foremost authority among the persons who participated in the design.

Conclusions

On account of the hazards involved in the lack of contact between design and construction departments on jobs involving large earthwork operations, progressive and competent engineering organizations maintain a soil mechanics department. During the design period this department supervises the boring operations and performs the soil tests. During the subsequent construction period it has the function of testing intermittently the materials derived from the borrow pits, supervising the compaction procedure and adapting it to changes in the character of the borrow pit materials. It has the additional function of comparing the design assumption concerning subsoil conditions with the conditions encountered in the field and, if necessary, modifying the design in accordance with the findings, requesting the design department to make the required changes. The importance of the services of the soil mechanics department is particularly notable on projects involving the design and construction of earth dams, because most of the favorable dam sites have already been utilized and soil conditions at the remaining ones may be so complex that the design assumptions based on the results of the subsoil exploration preceding the design stage are utterly at variance with those encountered during construction.

If a consultant is retained by an engineering organization in which the soil mechanics department maintains a continuous and intimate contact between design department and the job during the construction period, the cooperation between consultant and client is commonly frictionless and satisfactory, provided the members of the soil mechanics department are well trained and competent. Furthermore, the consultant can render a maximum of service in a minimum amount of time, because the soil mechanics department keeps him informed on whatever differences between design assumptions and field conditions are detected during the construction operations, and the department can be expected to take care that his instructions will be carried out on the job.

However, in most engineering organizations, design and supervision of construction are still divorced, though this fact may be camouflaged by a small soil mechanics department with no function other than providing the design department with the basic data for design. If a consultant is invited by an engineering organization with such an administrative setup to cooperate on a project in the design stage, he should watch his step. First of all, he should turn down the assignment unless it involves the duty to remain in active contact with the project until the end of the period of construction, and to inspect the job whenever he considers it necessary. In order to be able to perform his duty he must get detailed weekly reports informing him of all those observational facts which have a significant bearing on the validity of the design assumptions. Such a report can be prepared only by a competent soils engineer, who stays on the job permanently. Second, if the consultant accepts the assignment, he should find out as soon as possible whether or not the inspection of the construction operations on the job is satisfactory. If he arrives at the conclusion that the inspection is inadequate and his efforts to ameliorate the condition are unsuccessful, he should submit his resignation, leaving no doubt concerning the reasons which compelled him to do so.

The subject of this paper is of vital interest to consultants as well as to their clients and to the persons who furnish the capital for realizing their projects. The need for expert advice on difficult projects is universally recognized. However, the cooperation of consultants of high standing on such projects creates an unwarranted feeling of security, unless full advantage is taken of the services they are able to render. A satisfactory formula for accomplishing this purpose has not yet been established.

The preceding suggestions are based on my personal experiences and observations, the scope of which is inevitably limited. Therefore, other consultants and engineering firms employing consultants could render a valuable service to the engineering profession by presenting in the discussions to this paper some of their experiences and opinions concerning the relationship between consultants and clients.

Discussion

By Arthur Casagrande,* *Member*

A frank discussion of the use and abuse of consultants in earthwork and foundation engineering by engineers with broad experience was long overdue. It is, indeed, fortunate that Professor Terzaghi has undertaken the task to initiate and encourage such a discussion because, more than anyone else, he combines all the qualifications needed for pointing out clearly where the troubles lie and how they can be overcome.

The problem is complicated not only because of the great variety of technical questions on which the advice of a consultant is sought, but because many aspects of "human engineering" are involved. The most effective approach would be by a frank discussion of a number of typical case records. Unfortunately, that cannot be done without stepping hard on somebody's toes or kicking some other delicate part of the human anatomy. I regret to admit that I am responsible for having counseled Professor Terzaghi to delete some exceedingly instructive case histories from his original manuscript.

The situation is similar to the publication of settlement records and other observations on structures which have not performed satisfactorily. Progress in earthwork and foundation engineering would have been much more rapid if publication of such data would be the rule rather than the exception.

Professor Terzaghi has made it quite clear that principal causes of trouble are (1) the fact that the actual subsoil conditions cannot be known exactly during the design stage and that appropriate changes

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in design must be made as construction proceeds, and (2) the changes that the construction department will make or authorize without notifying the designers, or the "discrete departures from the specifications" by contractors. Professor Terzaghi points out that the principal requirement for solving such difficulties is a competent soil mechanics department that creates the liaison between the design and the construction departments, and that is empowered with sufficient authority. Most of the large construction organizations in this country which are engaged in earthwork and foundation engineering maintain such soil mechanics departments. They include the Corps of Engineers, the Bureau of Reclamation and many State Highway Departments, to name just a few of those doing excellent work in applied soil mechanics: California, Illinois, Indiana, New York and Texas. I regret that the Department of Public Works of Massachusetts is an example that lacks such a liaison between the design and construction departments. The result of such lack of cooperation culminated last year in a case that has made the newspaper headlines, and which is a good example of a consultant being made the scapegoat for mistakes by a contractor which are allowed to pass because of the lack of soils engineers who form the necessary link betwen design and construction.

It would seem that if a consultant values his reputation higher than public service, he cannot afford to work for those State and Municipal engineering organizations which are subject to excessive political pressure. But if no competent professional men were willing to risk being made the scapegoat, we would certainly not find such men in public service, and politicians would run everything.

The instance referred to above was made worse by the fact that field inspection was not part of the design contract. Therefore, the firm that prepared the design had no way of checking whether construction was carried out in accordance with the design. In connection with this question, attention is called to an excellent editorial on page 128 of the November 21, 1957 issue of *Engineering News-Record* that criticises the practice of "design without designer inspection."

When discussing the relationship between the engineering firms and special consultants they employ, there are certainly instances when the firm may have a right to make the final decision. The senior partner of a firm once made this comment: "After all, we are the ones who carry the responsibility, not our consultants. Besides, whenever we employ more than one consultant they usually disagree among themselves and then it is up to us, the designers, to make up our minds."

It would certainly seem logical that the division of responsibility should be clearly defined when consultants are employed. But even that is simpler said than done, because in many instances the responsibility of the consultant grows and changes as the job progresses.

Strictly speaking, every consultant, or board of consultants, is limited in the scope of their duties. However, a consultant should be free to question any aspects of a project, even outside the defined scope, if he believes that it may affect the safety of the project. E.g., on a number of dam projects I have questioned the freeboard because I did not trust the hydrologic data. On several projects, as a result of my insistance, a differential of several feet in freeboard was established between the main dam and one or several saddle dams, so that the saddle dams would be overtopped first, which would result in relatively minor damage. In other words, these saddle dams may become emergency spillways, although this may not be officially recorded in order to prevent protests from those who would suffer from the failure of such a saddle dam.

In conclusion, I should like to repeat my belief that there are too many variables involved in the relationships between Consultants, Clients and Contractors to permit hard and fast rules in order to assure that consultants will be used to the best interest of a project. But a careful study and re-reading of Professor Terzaghi's paper by all concerned will do much to develop a better awareness and judgment of the ramifications of the relationships between these three C's, of the conflicts that may arise from a lack of clarity in the definition of the duties of a consultant when he is employed, and of the paramount importance of continuous inspection of earthwork and foundation projects by competent soils engineers who form the liaison between the construction job, the design office and the consultant.

Discussion

By M. H. Cutler*

As Dr. Terzaghi spoke of his experiences at the "Annual Student's Night", the good fortune of the members of the audience, both younger and older, in being able to listen to the voice of international experience came to mind. Now that others will be able to benefit by publication of this talk, there are comments on certain phases which occur to me, based on my past 35 years of experience with a fully integrated engineering and construction organization which has been in existence more than twice this period.

This paper contains sound advice for the Engineer in any status, for the Client and for the Contractor. His recommendations are in line with organization procedures within a properly organized and integrated engineering and construction firm. Experiences similar to some of those described are the reason our organization by long established policy declines to function simply as design engineers without supervision of the construction.

The responsibility of an engineer to see that his design is executed within the framework of his assumptions and in accordance with his intents can not be avoided. Yet all too often inspection is cursory or regarded as something which can be left in the hands of a boy just out of school as a kind of "on the job training". The deficiencies in school and other construction revealed by earthquake disasters have led to the widespread revision in inspection of public construction in those areas and the development of a corps of capable inspectors with wide construction experience and sound judgment. And yet there are all too many other areas where this inspection is considered only as a political plum.

The comments regarding design assumptions and field conditions are particularly pertinent. Certainly complete cooperation among the departments involved in the design and execution of a project is essential. We have found it particularly advantageous to include one or more of the supervisory construction personnel in the early exploration and planning of a project in the office, during the time the basic design is initiated. In any case, continuous contact is maintained between the engineer and the field work by personal visits, telephone and teletype. There is, of course, no substitute for qualified and experi-

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enced personnel in all phases of the work. Earthworks and foundations particularly demand special care and cooperation between field and office, since compatibility between assumptions and actual conditions must be continuously checked with full appreciation that the earth's crust is not a uniform and quality controlled product, such as most of the elements making up a building superstructure. There is more than a germ of truth in the generality that the greatest consistency in foundation conditions lies in the variability.

The remarks on work carried out on a contract basis struck a responsive note. If the interest of the Owner, the Engineer and the Contractor are opposed, "the Contractor can not be expected to be interested or even aware of the reasoning behind the design." It is our philosophy that the best solution to this dilemma lies in a contract under which the interests of these parties are common.

There was a circumstance where we were constructing a plant from designs by others. Our superintendent's experience yardstick told him there was a discrepancy between the size of footings at the allowable soil bearing and the load to be carried. His comment to this effect was brushed off by the Engineer rather peremptorily but, being a persistent individual, the superintendent referred the matter to our engineering department which quickly verified that an arithmetic error had resulted in footings ¼ the proper size.

In another instance, a manufacturer had placed the responsibility for a project design of his process engineer, who, with the assistance of a contractor, had selected a site, cut and driven more than 3,000 spruce piles 50 ft long and poured some of the foundations, without benefit of suitable subsoil investigation. Some of the piles did not "fetch up" and another 50 ft length of pile was spliced weakly on the lower section. When three 50 ft lengths of piles, one on the other, still did not "fetch up", it was decided to obtain consulting advice, and we were called in. Subsoil investigations developed information that a surface stratum of sand was underlain by a substantial and variable depth of very soft plastic clay. Below the clay and above bedrock was another sand layer containing appreciable artesian pressure. Further investigation indicated that most of the piles as driven had their tips in the soft clay and that prohibitive differential settlements must be expected. During the investigation, evidence of suspected bank instability was proved and it appeared prudent to move the entire plant construction to another portion of the site where rock foundations were readily accessible. This procedure, of course, involved the abandonment of a substantial sum already spent on the plant construction and required prolonged and detailed discussions of the reasons for abandonment, particularly since "a similar plant had been built on another site in another section of the country on 50 ft wooden piles and had proved to be satisfactory." We are indebted to Dr. Terzaghi for his assistance as our consultant in this case and the added weight of his experience and confirming testimony which resulted in the mill being moved to a safe location.

In closing this discussion, it is a pleasure to pay tribute to Dr. Karl Terzaghi for his invaluable assistance as consultant to our organization on many complex foundation problems over the past 30 years.

Discussion

By D. J. Bleifuss*

I have read Mr. Terzaghi's paper with considerable interest; the subject is one which should be given a good deal of attention. I must start my discussion by disagreeing with him when he says his personal experiences and observations are limited; the scope of his experience is about as unlimited as it is possible for any one man's to be. There are few consultants as well qualified to discuss this subject.

He is quite right in saying that consultants are often not used to the best advantage. A client may not employ a consultant at all, when he really needs one badly. A client may select the wrong consultant. A client may make the wrong arrangement with the right consultant.

It is a curious fact that many laymen consider themselves qualified to criticise an engineer, or to do their own engineering. Time and education will take care of this, as the public comes more and more to realize that this civilization of ours is based on the work of the engineer. The roads we travel on; the cars we ride in; the machinery we use; the energy to drive our machinery; our communication systems; our water systems; they are all based on the work of the engineer.

A client may select the wrong consultant. To many people, an engineer is an engineer; they make no distinction between bridge,

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hydroelectric, sanitary, and other engineers. The same people would not dream of employing an obstetrician when they really need a skin specialist. Reputable consultants will not accept employment in a field where they feel they cannot do the best work. A consultant may be hopelessly incompetent; fortunately, there are very few of this class and they usually do not last long. A great name and reputation are no good guide in selection, which should be based on only one consideration, i.e., what the consultant has actually done in the field where his advice is being sought.

A client may make the wrong arrangement with a consultant. Wishing to save money, he may limit the consultant's employment to one particular phase of the work, such as preliminary layout and gathering data, design, or the supervision of construction. In the first two cases, there may be no "follow through," in the last case the consultant may be called on to supervise the construction of something he knows could be improved or is radically wrong. If limited to preliminary layout and gathering data, the consultant has no control over detailed design or the field changes inevitably necessary as construction develops new information. If limited to detail design, he may find his data insufficient (very common), that it has been misinterpreted, or that the preliminary layout is wrong. He will have no assurance that his careful design will be carried out, and, again, no control over field changes.

In such cases, if trouble develops, all the engineering on the job gets tarred with the same brush, regardless as to where the fault specifically lies. It is difficult to see what can be done about this, since a consultant cannot very well refuse employment on the grounds he is not being asked to do enough.

I wish to cite a few illustrations:

A. A dam site had been chosen and investigated. On being called upon to make a preliminary design and estimate, I found that a much better site close by had been disregarded.

B. An earthfill dam project—it had been reported that pervious material was plentiful, and impervious material scarce. When called upon for detailed design, we wished to check these data in the field, but the client insisted this was unnecessary and that he placed implicit reliance on his own engineers. The dam was designed accordingly. Upon personal investigation later, I found the data wrong; impervious material was plentiful, and pervious material scarce. The contractor had already started work, but the dam had to be redesigned. Then the client protested the extra expense.

C. We designed an earthfill dam, but the client insisted on supervising construction with his own forces; we were to have nothing to do with it. On casually visiting the dam during construction, I found to my horror, that where we had specified sand and gravel, silt was being placed, and very wet silt at that. It was merely the client's and our own good luck that we caught this in time.

D. We designed a concrete dam, but were to have nothing to do with supervision of construction. However, when placing of concrete was started, I looked at the first test reports, and found the concrete was not up to specifications. Although it was none of my business, I protested vigorously and the condition was corrected.

A consultant's relationship with a contractor may be of two kinds, the contractor may be his client, or the consultant may be the owner client's representative. Many contractors consider the engineer as an unmitigated nuisance and evil and think they could very well get along without him; some contractors have progressed to the point where they admit the engineer is a necessary evil; the best contractors cooperate whole-heartedly with the engineer. Quite often, the contractor's employees, regardless of his attitude, have a mistaken idea of loyalty, and think that by cutting corners, they are serving the contractor's best interest. Or they have not the slightest notion as to the reason why certain things must be done in a certain fashion, and regard any requirement which may interfere with speed and production as quite unnecessary. A foreman who two years ago was a laborer, will argue with an engineer of thirty years' experience. I may cite one case: a lift of concrete had been placed, with dowel steel projecting upward from its surface to tie in the next lift. As soon as the concrete had attained its set and could be walked on without footprints being left, the contractor's men swarmed over it, erecting forms for the next lift. The dowel steel was pushed around, with the result that each rod was soon standing in a hole, with no bond at all for perhaps twelve inches below the surface. It was a rush job, and my protest was regarded as unreasonable interference with progress. Another case: transmission tower foundations had been placed as much as three inches out of line and guide; steel towers erected on them were in consequence very much distorted, and it was necessary to take down the towers, dig up the foundations and start over. A totally unwarranted interference with progress.

Our trouble with an engineer's performance is this: if his job is well done, the work goes smoothly, and client and contractor alike are apt to consider that the money spent on engineering has been wasted; if the work does not go smoothly, they are apt to place the blame on incompetent engineering.

A consultant may better his relations with a contractor by adequate explanations; most men really like to know why they must do thus and so. A very fundamental thing: a consultant must design with an eye on what construction methods are to be used, and materials available.

A client employs a consultant because he thinks the consultant knows more than he does, and he wishes the benefit of superior knowledge. It is only common sense to make an arrangement which will insure he does get such benefit. He wishes to be assured the project is safe, that it will function properly, and be economically designed both as to first and annual costs.

Many consultants are specialists in rather narrow fields, and it would be quite useless to employ them in broader fields, and expect them to perform well. Others are more general in their knowledge, and they should be employed to coordinate the work of the specialists. I can best illustrate this in the hydroelectric field, with which I am familiar. A hydroelectric project should have an engineer of broad experience in this field in over-all charge. He does not need to be an expert in all the detailed phases of his work, but he must be able to know when he needs a specialist's help; he must be willing to ask for it; and be able to use it when he gets it. (The same may be said of an engineer in charge of supervision of construction.) The client's own forces may be able to do this coordination; if they are not, a general consultant should be employed to do it.

Mr. Terzaghi is unquestionably correct in stating that the cooperation of consultants in high standing creates an unwarranted feeling of security, unless full advantage is taken of the services they are able to render. While a satisfactory formula for accomplishing this purpose has not yet been evolved, it must lie in the directions of coordinating their activities, and giving them all a chance to "follow through."

Discussion

By Robert F. Ogilvy*

In April 1957, about half a year before Dr. Terzaghi read his paper "Consultants, Clients, and Contractors" at a meeting of the Boston Society of Civil Engineers, I was asked to talk at a luncheon meeting of the Montreal Soils Group on the topic "What the Structural Engineer Expects from the Soils Consultant". This talk was repeated in Ottawa in early October. My remarks on this subject supplement in many respects the statements contained in Dr. Terzaghi's paper. Therefore, Dr. Terzaghi has suggested that I submit the following abstract of my talk to the Montreal Soils Group as a contribution to the discussion of his paper. Although my talk dealt essentially with the relationship between the soils consultant and the structural engineer, in many instances the structural engineer may represent the owners, or act as an intermediary between the soils engineer and the owners, and as such may assume the position of client as far as the soils consultant is concerned. The soils consultant and the structural engineer should be mutually and equally concerned with the contractor's performance, and determined that the work shall be carried out to conform to the basis of the design specifications.

The work of the soils consultant and of the structural engineer lies quite largely within the field of general construction work. Possibly a brief review of design and construction practices in North America will provide some background for more detailed observations later. On one of the first warm days of the spring of 1927, several engineers, engaged on the construction of the Gatineau mill for Canadian International Paper Company, were sitting in a quiet alcove on the sunny side of the grinder building during noon hour discussing the construction work which was just drawing to a close. An Australian engineer expressed great surprise in comparing his previous experience with what he considered the strange practices which had been involved in the construction of Gatineau Works. He described the procedure in Australia based on practice in Great Britain, which required that all drawings for the entire project should be completed and checked and approved before any construction work was started. This was in very direct contrast to the procedure followed at Gatineau

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and which is being followed throughout North America to a very large extent today. I had joined the C. I. P. staff at Gatineau, 1st February 1926, on the morning on which the concrete spread foundations for the digester building had been blasted out to make way for pile foundations. The entire plant had been designed to rest on spread footings, but investigation during the previous three months of the loaded spread footings and loaded concrete and timber piles had shown that the very fine grey silty clay on the east bank of the Ottawa River required the use of pile foundations. As a result, all the concrete foundations designed already as spread footings had to be redesigned to rest on Raymond cast-in-place concrete piles. Although this did add considerable unexpected burden on the paper company's designing staff in New York, it did not altogether explain the fact that drawings came into Gatineau on the morning mail and, in some instances, were being used for actual construction a few hours later. This is only an extreme example of the accepted practice in North America which encourages construction work to be started in the very minimum time after the decision to proceed has been made at the highest level, with the result that design drawings are used for construction purposes as soon as they become available for part of the project. As design progresses for other sections of the undertaking, it may become necessary to revise or adjust the parts for which design was completed first, and this may even involve changes in the field if construction has progressed far enough to require this. It is not uncommon to have buildings under construction before certified prints are received from manufacturers of equipment to be housed in the buildings, and the final information contained on these prints of equipment may show that adjustments have to be made to drawings of buildings and foundations, which in turn may or may not require adjustments in the field to structures already completed.

The owners represented by the company directors are anxious to bring into production as soon as possible any enterprise which has received approval. Management is eager to further construction with the least time delay, and engineers themselves are ambitious to accomplish the maximum in the shortest time. Everyone involved directly or indirectly in a construction undertaking must be aware of, and recognize honestly, the universal desire on this continent to complete the job as soon as possible. This may be universal all round the world, but there are varying interpretations of what is "possible", and correspondingly varying degrees of pressure and differences in method; with the North American viewpoint being one extreme.

When any structure is to be built today, the first question certainly is "What are foundation conditions? On what will this structure rest?" To an increasing extent in the past twenty years, this consideration has gained recognition as the first essential for structural design, and certainly is the first proper question today. The correct answer to this question should avoid the observed damage to some structures built many years ago without such consideration. The structural engineer must start by roughing out a very general picture of the structure itself. Before he can proceed to proper design, he must know what the ground conditions are and what bearing pressure the soil will take. Variations of this requirement apply to dams, to wharves, and to practically every other type of structure in addition to the more common demand for information needed for building sites.

Although organizations or companies may be involved in structural design and soil investigation, it will be easier to consider these groups as represented by individuals in the person of the structural engineer and the soils consultant. The structural engineer normally is responsible for layout and design of a structure, and for final design decisions on matters on which the soil consultant's advice has been obtained. The soil consultant normally is responsible for investigating the site and providing advice and recommendations concerning:

1. Suitability of soil conditions for safe support of the proposed structure.
2. Specific Limitations which soil conditions will impose upon design, construction, and operation of the proposed structure. These are qualifications which may govern not only the type of foundation but also the type of structure which can be used with normal safety.
3. Prediction of physical behaviour of the foundation soil under the given applied conditions, and the effects of this behaviour on the structure and its intended use. Specifically, (a) what settlement is expected—what magnitude and during what time period; (b) what is the sheer strength of the soil in relation to stresses applied; and (c) for dams and other hydraulic structures, what is the estimated seepage and hydrostatic pressure.

The proper relation of these two persons involves almost complete integration. The structural engineer and the soil consultant must work together—not in parallel with a distance between them, but in full cooperation with close interchange of thoughts to provide continuity and consistency in the work, and avoid either a gap in thinking or a misinterpretation of intent. This comes in several stages:

(a) Preliminary. The structural engineer briefs the soil consultant on the preliminary planning and layout for the project and provides some basic data essential to a preliminary analysis of the foundation problem such as the type and use of the structure contemplated. The soil consultant should be prepared to make preliminary recommendations as to the type and extent of the investigation program.

(b) Work in the Field. The soil consultant plans and directs field investigation and laboratory analyses and tests. That work should be carried out competently and expeditiously.

(c) Report Submitted. The soil consultant submits an interim report or a final report, which normally includes an outline and appraisal of the general soil conditions with general recommendations pertinent to the design of the foundation structure. This work allows the final choice of a site if there are several alternatives, or confirms the practical suitability of a specific site. This provides the opportunity for an exchange of thought, and for a detailed discussion of individual design problems. The structural engineer may want more detailed interpretation of some item in the report, or further explanation of soil conditions. At this time the soil consultant should be given information as to the final site selected and details of the type and loading of individual structures, and he should review these and submit specific recommendations concerning foundations for individual structures or special cases.

(d) Review of Final Design. In order to be sure that exchange of information has been thorough and without misunderstanding, the soil consultant should review the final foundation design. For instance, at Kitimat, we were prepared at one time to provide more cover against frost protection than was necessary for foundations on that particular type of thoroughly consolidated sand and gravel backfill. Our soil consultant was informed of our thinking, and furnished further information on that one point.

(e) Review of Site Conditions During Construction. The soil consultant's report is an interpretation of the probable soil conditions over the entire site, based on soil borings which actually represent only an infinitesimal cross-section of the total site. In many cases it is advisable to check the validity of the interpretation when field conditions are revealed in volume during construction, so that any variations from the original interpretation may be brought to light and corrections applied to the design made by the structural engineer.

The work involved in a complete foundation analysis by a soil consultant breaks down into five natural steps. All or part of these may apply to an individual project depending upon its size, the degree of complexity of the soil problem, and the terms of reference of the consultant.

(a) Subsoil Investigation. Reconnaissance of site, supervision of drilling, sampling, and special field testing. This may or may not include contract drilling by the consultant's force. There may be some advantage in having the consultants carry out all work, although some of the consultants prefer to devote their energies to engineering supervision of the gathering of field data rather than being involved as both soils consultant and drilling contractor. This relationship can be debated at considerable length as each arrangement has its advantages and disadvantages. We have had satisfactory experiences with both arrangements.

(b) Laboratory Testing. A laboratory testing program has to be planned and executed with the specific purpose of investigating the properties of the soil pertinent to the individual problem. The program is carried out in two phases. The first consists of visual examination of the samples, together with elementary classification tests, to obtain an indication of the soil types present and of the degree of variation or uniformity of the soil profile. The second consists of physical tests to determine settlement, shear strength, or permeability characteristics, which can be used in a quantitative analysis of the foundation behaviour. The first tests are routine whereas the second tests are specifically aimed at particular soil conditions indicated by the first tests.

Care should be taken to give particular attention to the second group of tests as these results should be tailored to the intended use of the soil and the soil type itself. In certain instances, the standardized procedure has not yielded information pertinent to the problem, because the problem itself had not been sufficiently explained to the soil consultant, whereas the person using the data for design purposes may not understand the limitations of its use. The soil consultant should exercise great care to establish definitely that the data is understood and is applied properly to the problem under consideration. This further emphasizes the absolute necessity of close contact between the soils engineer and the structural engineer. It is essential that the soil consultant is absolutely sure that the test data obtained from laboratory work does apply specifically to the structural engineer's problem.

(c) Compilation of Data. This phase consists of assembly and compilation of field and laboratory data covering the drill logs and test results, and the presentation of this data in a concise and probably graphical form, in which it can be studied and digested. This is drafting room work. It is essential that the vast mass of data obtained on a large project should be summarized concisely in order that it may be very much to the point and easily understandable by an engineer not fully coversant with soil problems.

(d) Engineering Analysis and Report. This phase consists of study and analysis of the data in relation to the engineering problem by fully qualified professional personnel. This is the step in which the closest liaison with the structural engineer is required.

(e) Inspection and Supervision of Construction. This phase includes review of actual field conditions in comparison to the assumed conditions, field control testing to assure compliance with specifications in the case of earth construction, and provision of field soils engineers as advisors to the construction engineering staff, when such assistance is warranted. This is the final follow-up which is particularly valuable in generating confidence in the owners as represented by the structural engineering staff.

At the risk of repetition, it may be wise to run over in further brief detail the factors which affect the close co-operation between the structural engineer and the soil consultant. The basic problem of the soil consultant is:

(a) To determine from the structural engineer what conditions, present and future, will be imposed upon the foundation soil by the proposed structure.

(b) To determine the soil conditions at the site by subsoil investigation.

(c) To analyze the behaviour of the particular foundation soil under the imposed conditions and to interpret the behaviour in terms of the effects it may have on design and construction aspects.

To enable the soil consultant to resolve this problem, the structural engineer must provide the following information:

(a) Plan area and average net loading applied over the plan area of the structure, because the magnitude of settlement is related directly to these. This includes building loads both live and dead, and anticipated fill loads.

(b) Sensitivity of the structure and its enclosed plant to settlement effects, in order to assess the safe settlement tolerances of the structure. The rigidity of the building frame, the type of interior finish, and the sensitivity of machinery are pertinent points of interest.

(c) Special features of the structure which might affect or be affected by the foundation. Such items as vibrating loads, deep excavations, adjacent fill or storage loads, probability of future extensions to the structure, nature of foundations of existing adjacent buildings, etc., may have a critical bearing on the foundation behaviour.

Particular aspects of the general problem considered by the soil consultant include the following details applied to specific types of structures:

A. Building Foundations, Bridges, Wharf Structures, Retaining Walls, Bulkheads, etc.

1. Aspects Related to Design Considerations

   i. Review of types of foundation support suited to soil conditions.
   ii. Analysis of magnitudes of total and differential settlements and of time-rate of settlement.
   iii. Analysis of shear strength of foundation soil.
   iv. Analysis of effects of vibrations and earthquakes on foundation soil.
   v. Consideration of effects of future expansion of structure on foundation of existing and future buildings.
   vi. Analysis of lateral earth pressure to be resisted by retaining structures.

2. Aspects Related to Construction Considerations
   i. Effects of weather and season of year on behaviour of soil relative to working conditions and access on site during construction. Protective measures which might be necessary at various periods of year to prevent damage to soil foundation such as shrinkage due to drying, swelling due to wetting, expansion due to freezing.
   ii. Unwatering—extent of problem and whether or not special methods required.
   iii. Pile Foundations—analysis of problems of driving piles, recommendation of criteria for establishing adequate penetration resistance to meet design requirements, review of suitable types of driving equipment, review of special precautions to be taken to prevent interference with driven piles during driving of subsequent piles.

B. Embankments and Fills for Building Foundations, Roadways, Dykes, also Dams and their Foundations

1. Aspects Related to Design Considerations
   i. Analysis of settlement of foundation and fill.
   ii. Analysis of shear strength of foundation soil and stability of slopes of fill section.
   iii. Analysis of permeability, seepage and hydrostatic pressure conditions within fill and foundation, for dykes and dams.
   iv. Recommendations concerning dimensions, slopes and cross-section of fill dictated by fill and foundation soils.
   v. Evaluation of borrow materials.
   vi. Specifications for selection of borrow materials and placement of fill materials.
2. Aspects Related to Construction Considerations
   i. Review of most suitable types of equipment and construction procedures in relation to soil conditions, prevailing weather conditions and specification requirements.

A variety of construction jobs comes to mind, in which the demands on the soil consultant differed greatly in application. The basic requirement on all jobs, however, is to earn the customer's confidence. The structural engineer must be confident that the advice and guidance he is receiving is well founded. The second requirement on practically every job in North America today is speed—test results and the consequent recommendations are required urgently in no time at all!! Nothing shakes the client's confidence as much as being given a series of unfulfilled promises on delivery of data and advice.

The third factor is the extent of responsibility the soil consultant displays. Does he make a quick trip to the site, submit a report, and then vanish? Or does he continue to appear at intervals during construction and keep his recommendations up-to-date and refreshed periodically? Perhaps the fee for the work won't permit this further attention, but in that case the soil consultant should do a better selling job for his services. Get confirmation of the advice given, and stay on the job until it is confirmed.

Some of the suggestions which have been outlined are based on very practical experience. On the Peribonka River, north of Lake St. John, the site for a concrete dam and powerhouse was established and the construction work started. As the scope of the site investigation was expanded, it became apparent that a hillside covered by overburden on the east bank of the river did not consist of country rock covered to the usual fairly shallow depth by earth and gravel but actually was an earth and gravel hillside with the country rock at some considerable depth. Investigation was required very promptly to determine just what design of concrete abutment would be necessary to fit into that hillside, and whether a core wall was necessary in the hillside. In order to give the designers some indication of site conditions in the very shortest time, a geophysical investigation was carried out from the surface to determine rapidly the depth of bedrock, and the results of this investigation were confirmed by diamond drilling which required several weeks' more time. This is one example of the work of the soils consultants being adapted to the time requirements of the particular job, which fairly definitely dictated how the investigation should be carried out. On this particular location, observation wells were placed in this hillside to permit seepage measurements to be taken at regular intervals to determine just how effective the structural engineer's design had been, and these seepage readings were discontinued only last April after a record of five years had shown that the conditions are entirely satisfactory.

At Kitimat the smelter plant is located in the Kitimat River valley and is built upon material laid down by the Kitimat River. Soft top soil has been stripped off to a depth varying from 1 foot to a maximum of somewhat more than 20 feet, and the stripped material has been replaced to the required grade by sand and gravel backfill. Compaction of this backfill material has been very good and practically no settlement takes place in this replaced layer. However, at a depth of approximately 40 feet, varying somewhat with the location, there is a horizontal strata of compressible gray silt which compacts under pressure and very appreciable settlement takes place as buildings loads are applied. Our soils consultants estimated these settlements five years ago, and estimated variations in the settlement according to the location. These predictions have been proven to be quite satisfactory although there have been some minor variations. However, actual settlements have been observed and measured very closely during the intervening years with the result that this experience has been sufficient to enable the soils consultants to prepare predictions with increasing accuracy, so that we believe that the predictions of settlement for Potline 8 will prove to be almost exactly what will be experienced during the next 5 years at that location.

The soils consultant must be persistent and must have self-confidence. On one job, we took exceptional care to consolidate the rock on which a concrete storage dam was being constructed. Every means was taken to ensure that the rock was sound and an extensive grout curtain was drilled and placed under the upstream face of the dam. There was the rewarding satisfaction derived from the comment by an experienced consultant in dam construction that he had never witnessed a dam built from which leakage had been cut to such an absolute minimum. However, our satisfaction with this accomplishment was tempered somewhat by the question from one of our own engineers as to whether such care was really justified or whether it would be better to take less pains and permit certain leakage. The soils engineer may be faced with a similar question as to whether a full investigation of soils conditions is actually necessary, or whether construction work can be carried out to somewhat less exacting standards. The soils consultant must have the answers to justify the work he is doing.

Of course, a very definite example of that justification was the experience with a timber pile wharf designed for the west coast. When this design was well under way and the type of construction definitely established, a soils consultant was asked to confirm the fact that this design was satisfactory for this particular location. This investigation very promptly showed that normal loading of this wharf would stress the piles at least 100% and any combination of added wind loads or wave action would very seriously overstress the piles. In this particular instance, the soils consultant had to convince the owners that his conclusions based on expert investigation were more reliable than the owner's own previous experience with timber wharf construction on the west coast but under somewhat different conditions.

The growth of soils engineering during the past 20 years has led to an appreciation of the value of this work. However, it is still necessary to increase this appreciation in some quarters and that is the responsibility of the soils engineer. The structural engineer expects from the soils consultant definite engineering advice presented with sufficient confidence that it will be approved by the owners.

For material for this discussion, I am very much indebted to personal contacts with the soils consultants in the Montreal district. The engineering staff of the Aluminum Company of Canada, Limited, has very kindly commented on specific details. Our soils consultant on the west coast has provided invaluable information through discussion on what may be expected from the soils consultants.

Discussion

By Ralph B. Peck*

Probably every consultant acquires a professional personality that reflects his own background and the fortunes of his professional life. No two consultants would have identical views about the relations among consultants, clients and contractors. Yet the publication and discussion of opinions and experiences concerning this subject may serve a most useful purpose, and we are fortunate that Dr. Terzaghi has ventured to open for debate a field with many controversial aspects.

The writer has by necessity given considerable attention to the special opportunities and problems of the professor-consultant. Soil mechanics experienced much of its early growth in academic surroundings and it is not surprising that many teachers and research workers developed consulting practices. Nevertheless, there are all shades of opinion regarding whether or how the professor-consultant

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should operate. On the one hand, we find some universities—even state-supported ones—formally serving as the vehicle for the consulting practices of their staffs. On the other hand, a senior partner of a well established firm specializing in soil mechanics recently suggested** that professors should abandon independent consulting activities forthwith, although such firms might upon occasion engage professors to study selected problems.

The writer takes a middle ground. He believes there are valid, even compelling reasons why professors should maintain a consulting practice. First, the experience cannot but improve the breadth and vigor of instruction. Secondly, certain phases of the work afford an opportunity for students to participate directly in the solution of current engineering problems. Thirdly, the work provides an impetus for research and develops judgment regarding the significance of problems worthy of research. Finally, the experiences provide the professor with raw material for digests of case histories, professional papers, and even textbooks. The bare fact that the majority of modern texts have been written by teacher-consultants rather than by members of consulting firms is strong justification for participation in consulting activity on the part of the professor.

Nevertheless, the writer feels that there should be sharp limitations on the nature and extent of consulting services by academic people. The professor should not allow himself to be in competition with practicing engineers. Particularly if he is an employee of a tax-supported institution, he should meticulously avoid the use of public funds or the uncompensated use of public facilities for private gain. Over a period of some 15 years the writer has gradually recognized a few criteria that he considers useful for judging whether a consulting assignment is appropriate. Of foremost importance, the assignment should involve his personal services on a personal basis. It should not involve participation in an organization or company having an existence separate from the University. Secondly, the assignment should involve the special skills of the consultant preferably in a novel way, that is, the work should not be routine or of a character within the ordinary scope of activities of the practicing engineer. For the most part, in fact, his work should be done for practicing engineers or engineering firms. Finally, the fee, which should be on a per diem basis, should be not less than that which would be charged

**Too Much In-Fighting, Consulting Engineer, Dec. 1956, p. 6.

by an independent consultant; it should, in fact, be somewhat greater to encourage the employment of the independent consultant if a competent one is available.

From time to time, assignments may have to be accepted which do not suit all these criteria. One cannot always tell in advance whether a project will be routine. For some problems the type of soil testing must be decided on a day-to-day basis as the project unfolds, and at the end of the program one may realize that only routine tests have been made. Occasional routine projects, moreover, have educational value, especially for graduate students. But, with the criteria to serve as a guide, the professor can usually pursue a consistent policy. He will not, of course, advertise his services or solicit work.

As a matter of fact, the professor-consultant need not feel on the defensive on account of his consulting practice. He has the opportunity, perhaps even the obligation, to provide a standard of professional competence not yet universal among consulting firms in the field of soil mechanics. Many such firms fall into a routine of boring, sampling, testing, and making recommendations for design that hardly constitutes professional service. All too few firms pay more than lip service to research or professional papers; they are too occupied by the urgencies of the moment. Fortunately there are notable exceptions.

Dr. Terzaghi has mentioned several types of service performed by the individual consultant before or during execution of a project. Often, in addition to these, the consultant is called upon in connection with a controversy even after the project is completed. Here there are both rewards and dangers. One of the rewards is the large amount of factual data, often concerning construction difficulties, that become available to the consultant in a brief time. Some of the writer's most interesting records of experiences have come to his attention in this way. In some instances, by assembling the facts from a disinterested point of view, the consultant may help to end the controversy without litigation; this is a rewarding situation indeed.

In controversial situations much depends on the attitude of the client. Some clients truly seek out the facts and a fair estimate of the situation. Others would like to direct the opinion of the consultant to their own purposes on interests. Occasionally one can do no more than withdraw from an assignment, if the client withholds or colors pertinent data or exerts pressure toward a favorable opinion. In controversies the true reputation of a consultant emerges. The writer has been dismayed on occasion to discover that the "opposition" had retained a particular consultant because he knew the consultant's opinion would reflect the desires of the opposing party. On other occasions, the writer has been delighted to learn that a different consultant was on the opposite side of a controversy, even though that consultant was a highly competent individual, because he knew that the facts would be fairly and dispassionately used. A consultant can hardly have a more enviable reputation than to be desired as an opponent in spite of his great professional competence.

Finally, the writer would suggest that the consultant should be wary of making non-technical judgments. He is not a lawyer. He is often not called into a controversy until the battle lines are drawn. If he ventures out of his technical specialty, he may become unwillingly a pawn in the struggle.

Discussion

By Frank A. Marston,* *Member*

I recall seeing a bas relief copied from an Italian church, of some centuries old, which depicted the head and shoulders of the architect, the contractor and the owner, in that order. The contractor was thumbing his nose at the other two. Apparently some of the problems of today in the field of construction are not new, particularly as regards human relations and responsibilities. Dr. Terzaghi's paper is worthy of thoughtful consideration. His understanding of the problems and experiences of a consultant in civil engineering can be appreciated sympathetically by any engineer who has practised in that field over a period of years.

There is no question but that the consulting engineer who can carry on his professional practise with a small office and only a few associates may have fewer worries than another who practises as part of a large organization. On the other hand, the former may not have the thrill and the satisfaction of accomplishment that comes to the latter, who not only consults with others, but is closely identified with all stages of the design and construction of important projects.

Much of the paper could have been titled, "The Importance of

*Partner of Metcalf & Eddy, Engineers.

Soil Mechanics in Foundation Engineering", but the experiences cited are effectively used to demonstrate the value of competent advice from a consultant, and his relations to the project.

Inexperienced officials are sometimes led to select an engineer for a project because of a proposed low fee, rather than on the basis of qualifications. Such an engineer may prepare designs which are unsuitable for the soil conditions actually encountered. Such officials may object to expending funds for adequate subsurface investigations by means of borings, test pits and geological studies. To employ a consultant to advise during the design and construction stages may be considered too expensive. Then again the contractor may conduct his operations in a manner contrary to the specifications or at variance with the assumptions of design. Thus, conditions result which seriously affect the interests of the engineer, the client and the contractor.

In building a sewer system, where sewer pipes are to be laid in deep cuts, if the plans and specifications require that only narrow trenches be excavated and the contractor is allowed to dig wide, V-shaped trenches, as with a power shovel and without sheeting or bracing, the loads coming on the pipe may be greatly increased and result in the destruction of the pipe. Such a situation may bring criticism by the client of both the engineer and the contractor.

Another example concerned the building of a large reinforced concrete sewer in a deep trench. The excavation was made in open cut with steeply sloping sides. The concrete was poured during cold weather. Before the concrete had attained its strength the contractor backfilled the trench by pushing excavated material over the edge of the trench and by dropping material into the trench from a clam shell bucket attached to a crane boom. The heavy eccentric loading which resulted caused the sewer to crack. Furthermore, an attempt was made during the backfilling to consolidate the backfill material between the sides of the sewer and the sides of the trench by dropping the bucket on the fill as it was placed. Cold weather, rain and clayey soil helped to aggravate conditions. Both the engineer and the contractor were criticized.

A consulting engineer can be of material service to a city engineer, or other local official, by carrying the responsibility for a project, thereby relieving the local official of that burden.

The method of making a boring to determine the character of subsurface conditions may have an important bearing on the samples obtained and conclusions reached. Information can be highly misleading if an unsuitable method is used. Here again the engineer may be subject to criticism.

The paper points to the importance of close cooperation that is needed between those making preliminary field surveys, the design group, and the construction supervision group. The project engineer who has charge of the design should be thoroughly familiar with the work of the other two groups and in close contact with them. One prominent consulting engineer made a practise of declining to design work unless he was given general supervision of construction of it, including an opportunity to visit the construction whenever he deemed it essential to do so. There is merit in such a position, particularly as regards foundations and structural features.

The paper refers to the case where a consultant is hired for preliminary studies only so as to get the value of his name as "window dressing". One way to discourage that practise is to require that a stated fee be paid for a specific time regardless of how much service is requested, or a percentage fee might be appropriate.

The services of a competent consulting engineer will be valuable in the event of a structural failure even though he may have had no contact with the project prior to the failure. In such a case it is essential to determine the facts, insofar as possible. On the basis of his report it may be possible to settle the controversy without court action. However, it would be better to have had the benefit of the consultant's advice throughout the several stages of the project.

Competent inspection of construction work is of benefit to all concerned even though there may be no legal obligation for the client to provide it. The resident engineer and inspectors should not be paid by the contractor and should not be obligated to him. When the client is not a public authority and the contractor can be selected for his experience, ability and integrity, it should be practicable to reduce the cost of inspection. However, the engineer responsible for the design should have frequent contacts with the progress of construction to be sure that no unforeseen conditions develop that might conflict with the design.

A consultant should so conduct himself as to have the respect of his fellow engineers. He will do well to conform to the "Canons of Ethics" formulated by the Engineers' Council for Professional Development. If he is a member of the American Society of Civil Engineers he should be guided by its "Code of Ethics", as well. Similarly, members of the Boston Society of Civil Engineers are obligated to conform to its "Code of Ethics".[1]

I commend Dr. Terzaghi's paper to all engineers engaged in private practise and to those who employ their services.

Discussion

By Carlton S. Proctor*

Dr. Terzaghi's highly interesting paper, reminiscent of his brilliant professional and pedagogical career, points out many of the pitfalls that beset the client of the "package deal" engineering-construction procedure. His paper makes clear the fact that such pitfalls are inherent to "package deals", whether practiced by the engineering-construction departments of the client organization or by an independent contractor.

While his paper primarily depicts a situation where the engineering-construction work is performed by the Owners' engineering-construction departments, the same arguments apply with equal or greater force to the situation where the work is performed under a package contract by contractors whose services include both design and construction. In the latter case, the situation is additionally weakened by top management pressures to produce designs fitting into requirements for the use of the contractor's own equipment and limited to the contractor's experience.

The writer's experience over the past 39 years, as a member of a firm of Consulting Engineers specializing in substructure, marine, dam and other "heavy" engineering, completely endorses such conclusions.

Dr. Terzaghi's paper presents the case for the independent consultant, unencumbered by an engineering organization; but it ignores the obvious fact that few independent consultants have acquired his pre-eminence as expert in his field. Hence this paper cites examples of potential and actual failures and engineering mistakes which would not normally be discovered in time by a lone consultant but which would have been precluded under standard professional provisions by

[1] Jour. BSCE, Vol. 38, July 1951, p. 331.

*Partner, Moran, Proctor, Mueser & Rutledge, Consulting Engineers, New York City.

an integrated, experienced consulting engineering firm, whose practice was limited to purely professional service.

With the steady increase in complexity of engineering design in this engineering age; with the mounting demands for a practical working mastery of modern theory of engineering science coupled with the all-important value of broad experienced engineering judgement; in an era of diminishing supply versus increasing demand for thoroughly trained and experienced engineers, only the well-integrated engineering firm can meet today's demands.

In the long established practice of the writer's firm, full professional responsibility for design, specifications and supervision is accepted only where our contract provisions permit full latitude as to the acquisition of all pertinent data; our own laboratory soil investigations, comparative design studies to produce maximum stability, economy and utility; detailed supervision and inspection. Where the agreement for engineering services does not encompass these integrated services, disclaimers as to professional responsibilities are clearly established. Designers should carry through on supervision and inspection and each element of the project, from site selection to completion, should receive the benefit of specific experienced judgment in all fields of specialized complexity.

The antagonisms, the mutual exclusiveness, and the lack of incentive between design and supervision forces, as depicted in Dr. Terzaghi's paper, are inconceivable within the organization of a reputable consulting engineering firm; they can obtain only where the "package" type of engineering services are utilized. And "the layout of temporary installations is commonly left to the discretion of the superintendent of construction" only when the responsible engineering services are departmentalized within a parent organization or where engineering is organizationally interwoven with construction interests. Because as the paper so aptly puts it, "the contractor cannot be expected to be interested in the reasoning behind the design. His sole aim is . . . minimum expense".

DRAFT FOR A REVISION OF PART 29 OF THE BOSTON BUILDING CODE

PREFACE BY A. CASAGRANDE

If someone would want to write a book on HOW NOT TO MAKE FRIENDS he might well suggest, as one of the most successful methods, serving on a committee for the drafting of the Part on FOUNDATIONS for a Building Code. Should the reader desire further proof for this statement he is invited to attend the meeting of this Society on February 19, 1958, when this new Draft will be open for general discussion. But even if he has no special desire to watch the trouncing of those who have written this new Draft for Part 29 of the Boston Building Code, but is merely anxious to learn the answer to questions that occur to him when reading this draft, he is welcome to attend the meeting and to state his questions. Readers who are not able to attend that meeting are invited to submit their comments in writing. Comments, either oral or written, should be concise.

In June, 1955, Mayor Hines appointed H. A. Mohr as chairman of an advisory committee to the Building Department to draft a long overdue revision of Part 29 — FOUNDATIONS — of the Boston Building Code. The other members appointed to that Committee were Henri D. A. Ganteaume, Harry J. Keefe, Maurice A. Reidy, Jr., and the writer.

Mr. Ganteaume served faithfully and exceedingly well until his death on October 15, 1956.

Mr. Mathoff, of the Boston Building Department, served as secretary of this committee and his comments, based on his extensive experience with the administration of the present Code, were most helpful.

In June 1957, the first draft of the revision was completed. In mimeographed form it was distributed to numerous engineers in Boston, as well as to a selected group of well-known foundation engineers throughout the country. From the comments received during the summer of 1957, it was realized that much more work was needed on this draft and a group of Boston engineers who had expressed willingness to serve as an informal review committee, met with the members of the original committee once a week in the period from September to the end of December 1957. This group included: Henry Brask, F. E. Brown, Edwin W. Colby, James F. Haley, O. G. Julian, Wm. J. LeMessurier, Frank L. Lincoln, Mark Linenthal, Paul W. Norton, Waldo Pike, Richard C. Tousley, and Othar Zaldastani.

It was, of course, not possible to arrive at a unanimous agreement on all aspects of code requirements in this draft for Part 29. But the writer believes that on most of the vital points the final agreement was unanimous.

Based on the comments which will be received in the open meeting on February 19, and on those submitted in writing, a final revision of this draft will be prepared before it is submitted to the Mayor of Boston for further action.

At the final meeting on December 30, one of the senior members of the enlarged committee asked, half-jokingly, how much the City would have to pay if every participant in this effort would compute his time at his regular professional fee? Suffice to say that there are quite a number who have contributed hundreds of hours each, and that at a time when they could ill afford it because of their heavy regular working load.

PART 29.

Excavations and Foundations.

Section

Section 2901. — Excavations.

(a) Until provision for permanent support has been made, excavations shall be properly guarded and protected by the persons causing them to be made so as to prevent such excavation from becoming dangerous to life or limb. Where necessary, excavations shall be sheet-piled, braced or shored, and permanent excavations shall be protected by retaining walls or other permanent structures to prevent movement or caving of the adjoining soil.

(b) Structures near an excavation and owned by another than the person causing the excavation to be made shall be supported as follows:—

(1) Where an excavation is carried below the curb grade, at the common property line, or below the surface of the ground where there is no such curb grade, the person causing such excavation to be made shall, at all times, if accorded the necessary license to enter upon the adjoining land, and not otherwise, at his own expense, preserve and protect from injury any wall, building or structure, the safety of which may be affected by said excavation, and shall support it by proper foundations. If the necessary license is not accorded to the person making such excavation, then it shall be the duty of the owner refusing to grant such license to make such wall, building, or structure safe and to support it by proper foundations; and, when necessary for that purpose, such owner shall be permitted to enter upon the premises where such excavation is being made.

(2) Where a party wall is intended to be used by the person causing the excavation to be made, he shall, at his own expense, preserve such party wall from injury and shall support it so that the said party wall shall be safe for the purposes intended.

(c) If the person whose duty it shall be under the provisions of this section to guard and protect an excavation, or to prevent adjoining soil from moving or caving, or to preserve or protect any wall, building, or structure from injury, shall neglect or fail so to do, the Commissioner may enter upon the premises, and make safe such excavation, wall, building or other structure as provided in section one hundred and sixteen of Part 1.

Section 2902. — General Requirements for Foundations.

(a) The foundations of every permanent structure shall be supported by satisfactory bearing material which shall mean:

(1) Natural deposits of rock, gravel, sand, rock flour (inorganic silt), clay, or any combination of these which does not contain an objectionable amount of organic matter;

(2) Compacted fills which satisfy the provisions of section twenty-nine hundred and four (a)(4);

(3) Natural deposits or artificial fills which can be changed into satisfactory bearing materials by pre-consolidation with a temporary surcharge in accordance with the provisions of section twenty-nine hundred and four (a)(5);

(b) Where footings are supported at different levels, or at different levels from footings of adjacent structures, foundation plans shall include vertical sections showing to true scale all such variations in grade. The effect of such differences in footing levels on the bearing materials shall be considered in the design.

(c) The foundations and grade beams of permanent structures, except when founded on rock, and except as otherwise provided in paragraph (d) of this section, shall be carried down at least four feet below an adjoining surface exposed to natural freezing. No foundation shall be placed on frozen soil. Foundations shall not be placed in freezing weather unless adequately protected.

(d) Foundations of detached garages or similar accessory structures not exceeding eight hundred square feet in area and not over one story high, and grade beams of all structures need not be carried more than one foot below an adjoining surface exposed to natural freezing if the underlying soil to a depth of at least four feet beneath the surface, and extending at least six feet outside the building, is sand, gravel, cinders or other granular materials containing not more than five per cent by weight passing a No. 200 mesh sieve.

(e) Structures subject to artificial freezing shall have adequate provisions to prevent damaging upheaval of foundations and floors.

(f) Basements or cellars shall be waterproofed up to a grade at least two feet above the maximum probable ground water level. Under boilers, furnaces and other heat producing apparatus, the waterproofing shall be protected against damage from heat.

Section 2903. — Soil Information.

(a) Before issuing a permit for the erection or alteration that will affect the foundation of a permanent structure, the Commissioner shall require the applicant to furnish adequate soil data. Where borings or tests are required, they shall be made at a sufficient number of locations and to such depths as are necessary to provide a reasonably complete understanding of the soil conditions underlying the site of the proposed structure. It is desirable that the scope of the soil investigation be discussed with the Commissioner beforehand. When it is proposed to support the structure directly on bedrock, the Commissioner may require drill holes or core borings to be made into the rock to a sufficient depth to prove that sound bedrock has been reached.

(b) Duplicate copies of the results obtained from all completed and uncompleted borings, plotted to true relative elevation and to scale, and of all test results or other pertinent soil data shall be filed with the Commissioner.

Section 2904. — Classification of Bearing Materials and Allowable Bearing Values.

(a) The terms used in this section shall be interpreted in accordance with generally accepted geological and engineering nomenclature. In addition, the following more specific definitions are used for bearing materials in the Greater Boston area.

(1) *Rocks.*

Shale — A soft, fine-grained sedimentary rock.

Slate — A hard, fine-grained sedimentary rock.

Roxbury Puddingstone — A hard, well-cemented conglomerate.

(2) *Granular Materials.*

Gravel — A mixture of mineral grains at least 70% (by weight) of which are more than one-quarter inch in diameter and possessing no dry strength.

Sand — A mixture of mineral grains which passes a No. 4 mesh sieve and which contains not more than 15% (by weight) passing a No. 200 mesh sieve.

Coarse Sand — A sand at least 50% (by weight) of which is retained on a No. 20 mesh sieve.

Medium Sand — A sand at least 50% (by weight) of which passes a No. 20 mesh sieve and at least 50% (by weight) is retained on a No. 60 mesh sieve.

Fine Sand — A sand at least 50% (by weight) of which passes a No. 60 mesh sieve.

Well-graded Sand and Gravel — A mixture of mineral grains which contains between 25% and 70% (by weight) passing a No. 4 mesh sieve, between 10% and 40% (by weight) passing a No. 20 mesh sieve, and containing not more than 8% (by weight) passing a No. 200 mesh sieve.

(3) *Cohesive Materials.*

Hardpan — A glacial till that generally overlies directly bedrock and consists of a highly compacted, heterogeneous mixture ranging from very fine material to coarse gravel and boulders. It can be identified from geological evidence and from the very high penetration resistance encountered in earth boring and sampling operations.

Clay — A fine-grained, inorganic soil possessing sufficient dry strength to form hard lumps which cannot readily be pulverized by the fingers.

Hard Clay — An inorganic clay requiring picking for removal, a fresh sample of which cannot be molded in the fingers.

Medium Clay — An inorganic clay which can be removed by spading, a fresh sample of which can be molded by a substantial pressure of the fingers.

Soft Clay — An inorganic clay, a fresh sample of which can be molded with slight pressure of the fingers.

Rock Flour and Inorganic Silt — A fine-grained, inorganic soil consisting chiefly of grains which will pass a No. 200 mesh sieve, and possessing sufficient dry strength to form lumps which can readily be pulverized with the fingers.

(Note: Dry strength is determined by drying a wet pat of the soil and breaking it with the fingers.)

(4) *Compacted Granular Fill.*

A fill consisting of granular materials (gravel, sand-gravel mixtures, sand, crushed stone, slag, or cinders) containing not more than five per cent (by weight) passing a No. 200 mesh sieve, shall be considered satisfactory bearing material when compacted by one of the following methods:

I. In six-inch layers, each layer with at least four coverages with the treads of a crawler-type tractor with a total weight, including equipment, of not less than fifteen tons and operated at its top speed;

II. In twelve-inch layers, with at least three coverages with the wheels of a rubber-tired roller having four wheels abreast and weighted to a total load of not less than thirty-five tons;

III. Other types of materials and other compaction equipment and procedures may be approved by the Commissioner on the basis of sufficient evidence that they will achieve compacted fills having satisfactory properties.

Application of water is permitted, and for uniform sands may be required in order to achieve satisfactory trafficability and compaction.

The Commissioner shall require a competent inspector, qualified by experience and training and satisfactory to him, to be on the work at all times while fill is being placed and compacted. The inspector shall make an accurate record of the type of material used, including grain size curves, number of coverages and type of compaction equipment, the use of water and other pertinent data. Whenever the Commissioner or the inspector questions the suitability of a material, field density and laboratory compaction tests shall be made by a competent soils engineer in order to determine the degree of compaction achieved. A copy of all these records and test data shall be filed with the Commissioner.

(5) *Preloaded Unsatisfactory Materials.*

The Commissioner may allow the use of certain unsatisfactory natural soils and uncompacted fills for the support of one story structures, after these materials have been preloaded to not less than one-hundred and fifty per cent of the stresses which will be induced by the structure. The Commissioner shall require loading and unloading of a sufficiently large area, conducted under the direction of an approved, experienced soils engineer, who shall submit a report which demonstrates that the compressibility of the preloaded material will not cause objectionable settlements of the structure.

(b) The maximum pressure on soils under foundations shall not exceed the allowable bearing values set forth in the following table, except when determined in accordance with the provisions of sections twenty-nine hundred and fifteen and twenty-nine hundred and sixteen, and in any case subjected to the modifications of subsequent paragraphs of this section.

CLASS	MATERIAL	Allowable Bearing Value in Tons per Square Foot (*)
1	Massive igneous rocks and Roxbury Puddingstone, all in sound condition (sound condition allows minor cracks)	100
2	Slate in sound condition (minor cracks allowed)	35
3	Shale in sound condition (minor cracks allowed)	10
4	Residual deposits of shattered or broken bedrock of any kind except shale	10
5	Hardpan	10
6	Gravel, well-graded sand and gravel	5
7	Coarse sand	3
8	Medium sand	2
9	Fine sand	1
10	Hard clay	5
11	Medium clay	2
12	Soft clay	1
13	Rock flour, inorganic silt, shattered shale, or any natural deposit of unusual character not provided for herein	(**)
14	Compacted granular fill	2 to 5 (**)
15	Preloaded unsatisfactory materials	(**)

*The allowable bearing values given in this section, or when determined in accordance with the provisions of section twenty-nine hundred and fifteen, will assure that the soils will be stressed within limits that lie safely below their strength. However, such allowable bearing values do not assure that the settlements will not exceed the tolerable limits for a given structure.

**Value to be fixed by the Commissioner.

(c) The tabulated bearing values for rocks of Classes 1 to 3 inclusive shall apply where the loaded area is on the surface of sound rock. Where the loaded area is below such surface these values may be increased ten per cent for each foot of additional depth, but shall not exceed twice the tabulated values.

(d) The bottom surface of any footing resting on materials of Classes 4 to 15 inclusive shall be at least eighteen inches below the lowest ground surface immediately adjacent to the footing.

(e) The allowable bearing values of materials of Classes 4 to 9 inclusive may exceed the tabulated values by five per cent for each foot of depth of the loaded area below the minimum required in paragraph (d), but shall not exceed twice the tabulated values. For areas of foundations smaller than three feet in least lateral dimension, the allowable design bearing values shall be one-third of the allowable bearing values multiplied by the least lateral dimension in feet.

(f) Whenever there is any doubt about the settlements of a proposed structure or the effect on neighboring structures, the Commissioner shall require that the magnitude and distribution of the probable settlements be investigated as specified in section twenty-nine hundred and sixteen. Otherwise, the tabulated bearing values for Classes 10 to 12 inclusive shall apply only to pressures directly under individual footings, walls, and piers; and in case structures are founded on or are underlain by deposits of these classes, the total load over the area of any one bay or other major portion of the structure, minus the weight of excavated material, divided by the area, shall not exceed one-half the tabulated bearing values.

(g) Where the bearing materials directly under a foundation overlie a stratum having smaller allowable bearing values, these smaller values shall not be exceeded. Computation of the vertical pressure in the bearing materials at any depth below a foundation shall be made on the assumption that the load is spread uniformly at an angle of sixty degrees with the horizontal; but the area considered as supporting the load shall not extend beyond the intersection of sixty degree planes of adjacent foundations.

(h) Whenever, in an excavation, soil and ground water conditions are such that an inward or upward flow of seepage is produced in the bearing material, special excavating methods and control of ground water shall be employed to prevent disturbance to the bearing material. If there is evidence of disturbance of the bearing material, the extent of the disturbance shall be evaluated and appropriate remedial measures taken, satisfactory to the Commissioner.

Section 2905. — Foundation Design.

(a) Foundations shall be designed to distribute to the supporting materials all vertical, horizontal and inclined loads, as specified in section twenty-nine hundred and five, without exceeding the allowable stresses specified elsewhere in this Code for the materials of which the foundations are to be constructed. Concrete in all foundations shall be proportioned for an ultimate strength of at least two thousand pounds per square inch.

(b) The loads to be used in computing the maximum pressure upon bearing materials under foundations shall be the live and dead loads of the structure, as specified in Part 23, including the weight of the foundations, but excluding loads from overlying soil. Foundation mats or floors resting on the ground shall be designed to resist the maximum probable hydrostatic uplift.

(c) Eccentricity of loading in foundations shall be fully investigated and the maximum pressure on the basis of straight-line distribution shall not exceed the allowable bearing values.

(d) Where the pressure on the bearing material due to wind is less than one-third of that due to dead and live loads, it may be neglected in the foundation design. Where this ratio exceeds one-third, foundations shall be so proportioned that the pressure due to combined dead, live and wind loads shall not exceed the allowable bearing values by more than one-third.

(e) One story structures not exceeding eight-hundred square feet in area and having no masonry walls may be built on spread foundations founded on a crust not less than three feet thick of satisfactory bearing material which is underlain by unsatisfactory bearing material, provided, however, that the stresses induced in the unsatisfactory material by the to-be-added live and dead loads including that of new fill, if any, within or adjacent to the building area, will not exceed two-hundred and fifty pounds per square foot.

(f) The earth pressure against foundation walls and other types of retaining walls shall be determined in accordance with Soil Mechanics principles. Particular attention shall be paid to the type of backfill, drainage and the lateral support which may cause substantially larger earth pressures than the active earth pressure. In addition, such walls shall be designed for a hydrostatic pressure corresponding to the maximum probable ground water level.

Section 2906. (Now included in Section 2905).

Section 2907. — Footings and Foundation Piers.

(a) The footings of foundation walls or piers shall be of plain or reinforced concrete or other satisfactory masonry or steel grillages. Structural steel grillage foundations shall have at least six inches of net concrete cover below the bottom of the steel and shall have at least four inches of net concrete cover above the steel and between the sides of the steel and the adjacent soil. Footings of wood may be used under temporary structures.

(b) Foundation Piers are here defined as structural members, built in an excavation, or made by filling an excavated shaft with concrete, extending to a satisfactory bearing material.

(1) The manner of construction shall be by non-displacement methods and shall permit manual inspection of the bearing material in place.
(2) The bases of foundation piers may be enlarged by spread footings, pedestals or belled bottoms.
(3) Foundation piers built within excavations that are then backfilled by an approved method and those built by filling the excavated shafts with concrete, may be designed as continuously supported columns.
(4) Bell-shaped bases shall have a minimum edge thickness of four inches. The roof shall slope not less than sixty degrees with the horizontal unless the bases are designed in accordance with Part 26.
(5) When the center of cross section of a foundation pier at any level deviates from the center of the load more than one-sixtieth of its height, or more than one-tenth of its diameter, it shall be reinforced as provided in Part 26.
(6) With the approval of the Commissioner concrete may be placed through still water by means of a properly operated tremie or bottom-dump bucket.

Section 2908. — Driven Piles — General Requirements.

(a) Types of pile construction not specifically provided for in this part shall meet such additional requirements as may be prescribed by the Commissioner.

(b) A detached column supported by piles shall rest upon not less than three piles; except that for one story buildings a detached column may rest upon two piles when its axis is not more than one and one-half inches off the line connecting the centers of the two piles, or upon a single pile when other than wood or wood-composite piles are used, and its axis is not more than one and one-half inches off the center of the pile.

(c) A foundation wall, if properly restrained laterally both during and after construction, may be supported by a single row of piles.

(d) The method of driving shall be such as not to impair the strength of the pile and shall meet with the approval of the Commissioner. Shattered, broomed, crumpled or otherwise damaged pile heads shall be cut back to sound material before continuing the driving.

Followers — A follower shall be of steel, seasoned white oak or hickory, equipped on its lower end with a metal socket or hood suitable for encasing the pile head and to protect it from being damaged during driving.

Cushion Blocks — Except for wood piles, a cushion block consisting of a material equivalent in its elastic properties to that of seasoned white oak or hickory, enclosed in a metal housing to prevent its lateral deformation, shall be placed between the hammer plunger and the top of the pile.

(e) Jetted piles shall be driven to the required resistance after the flow of jet water has stopped, except as provided in section twenty-nine hundred and nine, paragraph (d)(5).

(f) Additional piles shall be driven to replace piles that have been driven in locations other than those indicated on the plans, damaged, or that have capacities less than required by the design, if such deficiency causes objectionable effects in the supported structure. In such cases the affected pile groups and pile caps shall be investigated and, if necessary, redesigned.

(g) Concrete capping for piles shall be proportioned for an ultimate strength of at least two thousand pounds per square inch. The concrete shall extend not less than twelve inches above the pile heads and shall fill the space between and around the piles for a depth of at least three inches. The minimum horizontal distance from the edge of the pile cap to the nearest pile surface shall be six inches and there shall be at least two inches of concrete between the top of a pile and steel reinforcement.

(h) Where piles are driven through soft soil to hard bearing material providing high point resistance, the grades of all piles or pile castings previously driven or redriven shall be measured to detect uplift; and if uplift of one-half inch or more occurs in any pile or pile casing, such pile or pile casing shall be redriven to its original point elevation and thereafter to the required final driving resistance.

(i) The Commissioner shall require the owner to engage a competent inspector, qualified by experience and training and satisfactory to the Commissioner, to be on the work at all times while piles are being driven. The inspector shall make an accurate record of the material and the principal dimensions of each pile, of the weight and fall of the ram, the type, size and make of hammer, the number of blows per minute, the energy per blow, the penetration of each pile for the last fifteen blows, together with the grades at point and cut-off. A copy of these records shall be filed in the office of the Commissioner.

Section 2909. — Allowable Load on Piles

(a) The supporting capacity of piles shall be obtained from bearing upon or embedment in bearing materials as defined in section twenty-nine hundred and four.

(b) The allowable pile load shall be limited by the provision that the vertical pressures in the bearing materials below the points of the piles produced by the loads on all piles in a foundation shall not exceed the allowable bearing values of such materials, as specified in sections twenty-nine hundred and four, twenty-nine hundred and fifteen, and twenty-nine hundred and sixteen. Piles or pile groups shall be assumed to transfer their loads to the bearing materials by spreading the load uniformly at an angle of sixty degrees with the horizontal, starting at a polygon circumscribing the piles at the top of the satisfactory bearing material in which they are embedded, but the area considered as supporting the load shall not extend beyond the intersection of the sixty degree planes of adjacent piles or pile groups.

(c) The allowable load on each pile shall be further limited by the requirement that such load shall not cause excessive movement of the pile relative to the soil. Satisfactory proof of this load for all soil conditions and all types of piles can be obtained from load tests conducted in accordance with section twenty-nine hundred and fifteen. In the absence of such proof of the supporting capacity, the load on a single pile shall not exceed the higher of the two values determined in accordance with paragraphs (d) and (e) of this section.

(d) The allowable load may be computed from the driving resistance as follows:

(1) Driving formula

$$R = \frac{k\ E}{s + c}$$

where

R = allowable pile load in pounds

E = energy per blow which for drop hammers and single-acting steam hammers is the product of the weight of the striking part of the hammer and the height of fall in feet, and which for other types of hammers must be verified in a reliable manner when determining the penetration s.

k = a constant which is 2.0 for wood piles, and 1.6 for all other types of piles.

s = average penetration per blow, for the last five blows, in inches.

c = a constant which is 1.0 for drop hammers, and 0.1 for all other types of hammers.

(2) For allowable pile loads of more than forty tons the energy E per blow delivered by the hammer shall be numerically not less than one-eighth of R.

(3) For double-acting and differential steam hammers and Diesel hammers, the value of "s" must be determined with the hammer operating at not less than 90% of the maximum number of blows per minute for which the hammer is designed.

(4) The data used in determining driving resistance shall be obtained during the driving and not upon re-driving when a pile has been allowed to stand more than one hour after having been driven.

(5) When any type of tapered pile is to be driven through a layer of material of Classes 6 to 10 inclusive and Class

14 exceeding five feet in thickness, and through an underlying soft stratum, the bearing capacity shall not be determined in accordance with the driving formula unless jetting is used through said layer during the entire driving of the pile.

(e) The allowable load on a pile stopped in inorganic clay as found in Greater Boston, may be based on a friction value of six-hundred pounds per square foot of embedded pile surface. The embedded length shall be the length of the pile below the surface of the inorganic clay, or below the surface of immediately overlying satisfactory bearing material. The embedded pile-surface-area shall be obtained by multiplying the embedded length with the perimeter of the smallest circle or polygon that can be circumscribed around the average section of the embedded length of the pile. The method of determining the allowable load described in this paragraph shall not be used for a pile in which the drive-pipe is withdrawn.

(f) When piles in clusters are driven under the provisions of paragraph (e), the allowable load shall be computed for the smaller of the following two areas: (1) the sum of the embedded pile-surface-areas; (2) the area obtained by multiplying the perimeter of the polygon circumscribing the cluster at the surface of the satisfactory bearing material with the average embedded length of piles.

(g) The allowable load on a single pile installed by jacking shall not exceed one-half the load applied to the pile at the completion of jacking, provided that the final load is kept constant for a period of four hours and that the settlement during that period does not exceed one-twentieth of an inch.

Section 2910.—Wood Piles.
General Requirements

(a) Every wood pile shall be in one piece, cut from a sound live tree, and free from defects which may materially impair its strength or durability. It shall be butt-cut above the ground swell, and shall have substantially uniform taper from butt to point. Wood piles shall measure at least six inches in smallest diameter at the point, at least ten inches in smallest diameter at the cut-off, these measurements being taken under the bark. The axis of a wood pile shall not deviate from a straight line more than one inch for each ten feet of length nor more than six inches for the entire length.

(b) The load on a wood pile shall not exceed the allowable load specified in section twenty-nine hundred and nine and, for a pile of the minimum dimensions specified in this section, shall not exceed twelve tons for Spruce, Norway Pine, and woods of similar strength which will be referred to as Type A, nor sixteen tons for Oak, Southern Yellow Pine, and woods of similar strength which will be referred to as Type B. These loads may be increased for each full inch by which both the cut-off and point diameters exceed the minima specified, by three tons for woods of Type A, but not to exceed a total load of twenty-four tons; and by four tons for woods of Type B, but not to exceed a total load of thirty tons.

(c) Piles shall be cut to sound wood before capping is placed.

(d) The center-to-center spacing of wood piles shall be not less than two and one-half times the cut-off diameter.

(e) The size of the hammer shall be such that the driving energy in foot-pounds per blow shall not exceed numerically the point diameter of the pile in inches multiplied by fifteen-hundred. The total driving energy in foot-pounds for six inches of penetration shall for all types of hammers be numerically no greater than the point diameter in inches times twenty-two-thousand for woods of Type A or times thirty-two-thousand for woods of Type B. For the last inch of penetration the energy in foot-pounds shall not exceed numerically the point diameter in inches multiplied by six thousand for woods of Type A and seven-thousand five-hundred for woods of Type B.

(f) The cut-off grade for untreated-wood piles shall be below the probable permanent ground-water level, and shall be subject to the Commissioners approval.

(g) *Additional Requirements for Treated Piles.*

(1) Timber piles pressure treated with creosote or creosote-coal-tar solutions, and conforming to the requirements of this section, may be cut off above permanent ground water level when used for the support of buildings of Type V or VI or for one-story buildings of other types.

(2) Before any treated piles are driven, the Commissioner shall be furnished three copies of a certificate of inspection, issued by an approved independent testing laboratory, certifying that the piles were free of decay, were properly peeled and otherwise prepared before treat-

ment; and that the method of treatment, the chemical composition and the amount of retention of the preservative conform to the requirements of this section.

(3) Treated piles shall be of Norway Pine, Southern Yellow Pine or Douglas Fir and shall be impregnated with preservative in accordance with standards C1-57 and C3-57 of the American Wood Preservers' Association, or as required by the Commissioner.

(4) Piles exposed to sea water shall be Southern Yellow or Norway Pine treated with Grade B creosote-coal tar solution, conforming to standard P2-57 of the American Wood Preservers' Association. Piles not so exposed shall be treated with creosote conforming to standard P1-54 of the American Wood Preservers' Association.

(5) The retention of preservative shall be not less than twenty pounds per cubic foot for piles exposed to sea water and not less than twelve pounds for other piles.

(6) After being cut to grade, the top surface of the pile shall be brush treated with not less than three heavy coatings of the treating material applied hot.

Section 2911. — Precast Concrete Piles

(a) Precast concrete piles shall be so proportioned, cast, cured, handled and driven as to resist without perceptible cracking the stresses induced by handling and driving as well as by loads. The minimum lateral dimension of a precast concrete pile shall be twelve inches at cut-off and eight inches at the point exclusive of the metal point, if used. Each pile shall be cast in one piece. The concrete shall have a minimum compressive strength of three thousand pounds per square inch and shall fulfill other requirements of Part 26. No pile shall be handled or driven until it has cured sufficiently to develop the necessary strength as shown by standard test specimens made from the same batches of concrete cured under similar conditions.

(b) Piles shall be proportioned so as to satisfy the requirements of Part 26. Additional requirements are as follows: For a length equal to at least three times the minimum lateral dimension at both ends of the pile, lateral ties shall be spaced not over three inches center-to-center or an equivalent spiral shall be provided. Reinforcing steel shall be embedded in concrete forming the body of the pile a net distance of at least one and one-half inches from any exposed surface and in piles exposed to sea water such coverage shall be at least three inches.

(c) The maximum water-cement ratio and the minimum cement content of the concrete for piles exposed to sea water shall be 4.5 gallons per sack, and 8 sacks per cubic yard, respectively.

(d) The minimum spacing center-to-center of precast concrete piles shall be two and one-half times the square root of the cross-sectional area at the butt.

(e) When precast concrete piles are driven to or into bearing materials of Classes 1 to 5 inclusive, or through materials containing boulders, they shall have metal tips of approved design.

(f) The load on a precast concrete pile shall not exceed the allowable load specified in section twenty-nine hundred and nine, and shall not exceed fifty tons for a pile of one square foot cross-sectional area. For piles of larger cross-section, this limit of load may be increased in proportion to increase in area, but not to exceed a total load of eighty tons.

Section 2912. — Cast-in-place Concrete Piles.

(a) In this section a distinction is made between poured-concrete piles and compacted-concrete piles. A poured-concrete pile is formed by pouring concrete into a driven casing or drive-pipe that is installed in the ground either permanently or temporarily. A compacted-concrete pile is formed by placing concrete having zero slump, in small batches, and compacting each batch.

(b) All cast-in-place concrete piles shall be so made and placed as to ensure the exclusion of all foreign matter and to secure a well-formed unit of full cross section. While placing the concrete the casing or drive-pipe shall be free of water.

(c) *Poured-Concrete Piles.*

(1) The diameters of metal-cased poured-concrete piles, when measured on the outside of a plain cylinder, or of horizontal, helical or vertical corrugations, shall be not less than eight inches one foot above the point, nor less than twelve inches sixteen feet above the point. The shape of the pile may be cylindrical, or conical, or a combination thereof, or it may be a succession of cylin-

ders, with the change in diameter of adjoining cylinders not exceeding one inch.

(2) For uncased poured-concrete piles (i.e: when no metal casing is left in the ground) the outside diameter of the drive-pipe shall be not less than fifteen inches.

(3) The load on poured-concrete piles shall not exceed the allowable load specified in section twenty-nine hundred and nine, nor twenty-two and one-half per cent of the twenty-eight day strength of the concrete, but not exceeding 900 pounds per square inch, when applied to the cross-sectional areas computed on the following basis:

for metal-cased piles driven to or into materials of classes 1 to 5 inclusive, using the diameter measured one foot above the point and as further specified in paragraph (1), minus one-half inch.

for metal-cased piles driven into materials of classes 6 to 14 inclusive, using the diameter at the surface of the bearing stratum in which the pile receives its support, and as further specified in paragraph (1), minus one-half inch.

for uncased piles driven to or into any bearing material, using the inside diameter of the drive pipe minus two inches.

In no case shall the maximum load on a poured-concrete pile exceed seventy-five tons.

(4) The spacing of poured-concrete piles shall be such as to ensure the preservation of the full cross-section. The spacing center-to-center shall be not less than two and one-half times the outside diameter of the casing or drive-pipe at cut-off. Where the center-to-center spacing is thirty-six inches or less, no casing or drive-pipe shall be filled with concrete until all casings or drive-pipes within a radius of five feet have been driven to the required resistance.

(d) *Compacted Concrete Piles*

The load on compacted concrete piles shall be limited by the provisions of sections twenty-nine hundred and eight and twenty-nine hundred and nine paragraph (b), except that the circumscribing polygon shall start at the junction of the shaft and the enlarged base, and the bearing area shall be taken at planes six feet or more below said junction; nor shall the allowable load on a compacted concrete pile exceed one hundred and twenty tons. The installation of such piles shall fulfill the following listed requirements:

(1) The drive-pipe used for installing the pile shall be not less than twenty-inches outside diameter.

(2) The enlarged base of the pile shall be formed on or in bearing materials of Classes 1 to 8 inclusive.

(3) The concrete shall have a twenty-eight day strength of at least 3750 pounds per square inch, shall have zero slump, and shall be placed in batches not to exceed five cubic feet in volume.

(4) The last batch of concrete shall be driven into the enlarged base with not less than twenty blows, each of not less than one hundred and thirty thousand foot-pounds.

(5) As the drive-pipe is being withdrawn, not less than two blows of not less than thirty thousand foot-pounds shall be applied to compact each batch of shaft concrete.

(6) An uncased shaft shall not be formed through organic or inorganic clay or silt unless an excavation at least equal to the inside diameter of the drive-pipe is first augured through such soil, or the individual piles are located more than nine feet apart.

(7) A permanent metal-cased shaft, not less than sixteen inches in diameter, shall be formed through organic or inorganic clay or silt if requirement (6) is not fulfilled. The permanent metal casing shall be fastened to the enlarged base in such a manner that the two will not separate. The concrete may be placed in the metal casing in the same manner as for poured-concrete piles. No metal casing shall be filled with concrete until after all piles within a radius of at least nine feet have been driven. The allowable load on the shaft shall be limited as specified for concrete-filled steel pipe piles in paragraph (b) of section twenty-nine hundred and thirteen.

(8) The center-to-center spacing of piles shall be not less than four feet and six inches.

Section 2913. — Steel and Steel-Concrete Piles.

(a) At locations where steel piles will be in contact with sea water, organic soils, cinders, slag or any fill containing material that might attack steel, the surface of the piles shall be effectively protected against contact with such materials.

(b) *Concrete-filled Pipe Piles.*

(1) Piles consisting of steel pipes and concrete-filled after driving shall have an outside diameter of not less than ten and three-quarters inches and a pipe wall-thickness of at least two-tenths of an inch. Splices shall be welded to one hundred per cent of the strength of the pipe. Pipes may be driven open-ended or closed-ended, and the provisions of this section apply to both types.

(2) After driving, the inside of the pipe shall be carefully cleaned to the bottom and its curvature and cross section verified to the satisfaction of the Commissioner. The diameters shall not vary more than twenty per cent from the original value. Pipes shall be filled with concrete having an ultimate strength at twenty-eight days of at least three thousand pounds per square inch and as further specified in Part 26. Concrete shall not be placed through water.

(3) The center-to-center spacing of concrete-filled pipe piles shall be not less than two and one-half times the outside diameter of the pipe.

(4) The load on concrete-filled pipe piles shall not exceed the allowable load determined in accordance with section twenty-nine hundred and nine, nor the load on the concrete at twenty-two and one-half per cent of the twenty-eight day strength, but not exceeding nine hundred pounds per square inch, plus the load on the steel at eight thousand pounds per square inch; nor shall the load carried by the steel exceed one-half the total load on the pile.

(c) *H Piles.*

(1) Rolled steel H or other approved sections having a minimum thickness of metal of 0.4 inch may be used as piles. They shall be spliced to one hundred per cent of the strength of the section.

(2) The center-to-center spacing of such piles shall be not less than two and one-half times the width of the flange.

(3) The load on such piles shall not exceed the allowable load determined in accordance with section twenty-nine hundred and nine, nor seven thousand pounds per square inch on the area of the cross section.

(d) *Concrete-filled Pipes with Steel Cores.*

(1) The pipe shall be so installed that its lower end is firmly seated in bedrock of Classes 1, 2 or 3. It shall be of sufficient diameter to permit manual inspection of the bedrock socket. Splices shall be welded to one hundred per cent of the strength of the pipe.

(2) A socket, approximately of the inside diameter of the pipe, shall be made in bedrock of Classes 1 or 2 to a depth that will assure load transfer when computed for a bearing on the bottom surface of the socket in accordance with paragraphs (b) and (c) of section twenty-nine hundred and four, acting together with a bond stress on the perimeter surface of the socket of one hunded pounds per square inch.

(3) The steel core shall consist of a structural steel member. The ends of the sections of the core shall be milled for bearing and the splices shall be so made as to safely withstand the stresses developed during installation. The steel core shall be centered in the steel pipe and shall rest on a layer of cement grout on the bottom of the socket.

(4) Concrete shall have a minimum compressive strength of four thousand pounds per square inch at twenty-eight days. It shall be so poured that its surface around the steel core will at all times be substantially level.

(5) The allowable load shall be computed on the basis of nine hundred pounds per square inch on the net area of

the concrete, eight thousand pounds per square inch on the steel area of the pipe and fifteen thousand pounds per square inch on the area of the steel core.

(6) The details of the installation, including inspection and cleaning of the socket, the placement of concrete under water and in the dry, the method of centering the steel core and all other phases of the work shall be submitted to the Commissioner for approval.

Section 2914. — Composite Piles.

(a) A composite pile shall consist of a combination of not more than two of any of the different types of piles provided for in this Part. The minimum dimensions and other provisions applying to each type shall be those specified herein. The connection between the two types of piles shall be constructed so as to prevent their separation, to maintain their alignment, to support the load and to be watertight where concrete must be placed subsequent to the driving. The design and the details of the connection shall be subject to the Commissioner's approval.

(b) The use of wood-composite piles shall be limited for support of buildings of Types V and VI, and for one-story buildings of other types.

(c) The center-to-center spacing shall be governed by the larger spacing, required in this Part, for the types composing the pile.

(d) The allowable load on composite piles shall be that allowed for the weaker of the two sections. For wood-composite piles the allowable load shall not exceed eighty per cent of that allowed for the wood section alone.

Section 2915. — Bearing Tests.

(a) Whenever the allowable bearing value on bearing materials or on piles is in doubt, the Commissioner may require bearing tests to be made.

(b) Before any bearing test is started, a sketch of the proposed test arrangement and an outline of the procedure to be followed shall be submitted to the Commissioner and shall have his written approval.

(c) Bearing tests shall be conducted in the presence of an inspector, qualified by experience and training, and who is satisfactory to the Commissioner. A copy of the test results obtained and a graph of the time-settlement curve for each increment of load and of the load-settlement and rebound curve for the entire test shall be submitted to the Commissioner at the completion of each test.

(d) The load shall be applied by direct weight or by means of a newly calibrated hydraulic jack. The application of the test load shall be in steps equal to not more than one-half the contemplated design load, to twice the contemplated design load, except as provided in paragraph (g) of this section. The unloading shall be in at least two steps, to the design load and then to zero load. The contemplated design load during the loading and unloading cycles shall be maintained constant for at least twenty-four hours and until settlement or rebound does not exceed two-hundredth of an inch in twenty-four consecutive hours. The load for all other load steps including the zero load at the end of the test shall be maintained constant for a period not less than four hours. Sufficient readings for each load step shall be made to define properly the time-settlement and rebound curve.

(e) Observation of vertical movement shall be made with dial extensometers with an accuracy of at least one-thousandth of an inch. The readings shall be sufficient in number to define the time-settlement and rebound curve and shall be referred to a beam the ends of which rest on or are fixed to reliable supports located at least six feet from the center of the test. In addition, the elevation of the supports shall be checked frequently with reference to a fixed benchmark. The entire measuring set-up shall be protected against direct sunlight, frost action and other disturbances that might affect its reliability. Temperature readings, both inside and outside the test enclosure, shall be made when the vertical movements are recorded.

(f) *Additional Requirements for Soil Bearing Tests.*

(1) Soil bearing tests shall be applied to the soil at the elevations of the proposed bearing surfaces of the structure.

(2) The excavation immediately surrounding an area to be tested shall be made no deeper than one foot above the plane of application of the test, except that for material of Class 14 the test load may be applied directly on the surface. The test plate shall be placed with uniform bearing. For the duration of the test the soil surrounding the test area shall be protected effectively against evaporation and frost action.

(3) For bearing materials of Classes 1 to 5 inclusive, the loaded area shall be not less than one square foot and for other classes not less than four square feet. For bearing materials of Classes 1 to 3 inclusive, the Commissioner may permit compression tests on rock cores to be substituted for bearing tests. Each test specimen shall have a height not less than twice its diameter.

(4) The proposed design load shall be allowed provided that the requirements of section twenty-nine hundred and four are fulfilled and the settlements under the design load and twice the design load do not exceed three-eighths of an inch and one inch, respectively.

(g) *Additional Requirements for Pile-Bearing Tests.*

(1) A single pile shall be load tested to not less than twice the design load. When two or more piles are to be tested as a group, the total load shall be not less than one and one-half times the design load for the group.

(2) The design load shall not exceed the load allowed in this Part for each type of pile nor one-half of the maximum applied load provided that the load-settlement curve shows no sign of failure and provided that the permanent settlement of the top of the pile, after removal of all load at the completion of the test, does not exceed one-half inch.

Section 2916. — Settlement Analysis.

(a) Whenever a structure is to be supported by medium or soft clay (materials of Classes 11 and 12), the settlements of the structure and of neighboring structures due to consolidation of the clay shall be given careful consideration. In case one or more of the following listed conditions, or other conditions prevail that might cause similar effects, the Commissioner may require a settlement analysis to be made by a competent engineer with specialized training and experience in soil mechanics:

(1) The structure induces a net increase in stress greater than one-quarter of a ton per square foot in underlying soft clay.

(2) The structure induces a net increase in stress greater than three-quarters of a ton per square foot in underlying medium clay.
(3) The structure has substantial variations in load at foundation grade.
(4) The structure is underlain by soft clay showing large variations in thickness.

(b) A settlement analysis will be usually based on a computation of the net increase in stress, after deducting the weight of excavated soil and other loads under which the clay was fully consolidated, that will be induced by the structure, combined with soil compressibility data derived by one or more of the following means:

(1) Consolidation tests on undisturbed specimens with a diameter of at least two and one-half inches. The report shall include a description of the method of sampling and a description of the quality of the samples including representative photographs of longitudinal sections through such samples when partially dried.
(2) Use of empirical relationships between compressibility of the clay and the natural water content, the liquid and the plastic limits.
(3) A review of the settlement records and behavior of other buildings having similar subsoil profiles.

(c) Should the analysis indicate that the total or differential settlements of the structure would exceed values which would cause excessive stresses in the structure or would impair its usefulness, the design of the foundation and/or superstructure shall be modified.*

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*This may be accomplished by one or a combination of the following measures:
(1) Use of a deeper basement under portions of, or over the entire building area.
(2) Increased stiffness, particularly of the foundations.
(3) Changes in the design of the building which will effect a more uniform load distribution.
(4) Changes in the design of the building which will effect a reduction in the total load and thereby in the stresses induced in the compressible soil strata.

Frequently, the use of substantially reinforced concrete girders beneath all walls are a desirable or necessary protection of those elements of a building that are most sensitive to differential settlements.

OZONE AS A DISINFECTANT FOR WATER AND SEWAGE

By Werner Stumm*

(Presented at a Meeting of the Sanitary Section, B.S.C.E., held on October 2, 1957.)

Introduction

Most of the conscious efforts in water supply and waste-water disposal have been, and continue to be, directed toward the removal and destruction of pathogenic microorganisms.

Chlorine and some of its compounds meet the general requirements for a disinfectant so well that they have been used almost exclusively. However, we have to reconsider periodically whether other disinfectants could be employed expediently for disinfection purposes.

Ozone has been used in municipal water supplies mainly in Europe. It has, however, never received widespread recognition as a disinfectant in this country. Although information on the properties of ozone is very limited and the few quantitative data available in the literature are scattered and controversial, all recent investigations agree that ozone is capable of efficiently destroying pathogenic organisms borne by water or sewage.

Production of Ozone and Its Transfer into Water

Ozone is relatively safe and easy to handle. It is unstable and must be produced at the point of application. The ozonator is generally made up of a number of elements consisting of flat hollow blocks between which are set pairs of flat glass plates. Each glass plate has an electrode on its exterior surface. A voltage of 8,000 - 20,000 volts is applied across the electrodes and A. C. of 500 cycles is commonly used. The air or oxygen passing to the ozonator must be dried, and normally the refrigerant type of dryer is used. The air or oxygen volume passing through the ozone generater is such that only about 1 - 5 mg ozone per liter of gas is created.

Ozone can conveniently be applied to the water by either one of the following methods: (1) ozonized air is intimately mixed with water in a type of injector; (2) ozone is dispersed under pressure into

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the water through porous media at the bottom of a deep tank; (3) the gas is introduced by means of vortical rotors (cavitators). However, the devices used today let a considerable proportion of the applied ozone escape into the atmosphere. It may therefore be expedient to recover and reuse this ozone.

The rate of ozone transfer into solution is dependent upon the ozone concentration in the gas stream and the applied pressure, according to the laws of Henry and Dalton. If ozone is produced from oxygen instead of air, the yield of ozone (for the same energy uptake) and, therefore, the transfer of ozone into water can be improved considerably. Additional research on mixing and gas transfer in solution is needed to evaluate the dependence of gas-solution transfer on parameters such as bubble size, turbulence, depth of the contact vessel, surface activity of the solution, etc. Such studies should lead to the design of more profitable and efficient ozone-water contacting vessels.

Cost of Ozone

It is a general belief that the use of ozone is limited because of cost. A few data on operational and maintenance cost given by European water supplies indicate, however, that the cost of disinfecting water supplies by ozone is comparable with that of chlorine.[1,2]

Table 1 gives a recent analysis of energy uptake for the produc-

TABLE 1
Energy Uptake for the Production of Ozone and Its Transfer into Solution
(*Scheller*[2])
(0.5 mg O_3 is applied per liter water)

	Energy Uptake in watts per gram Ozone
Drying of air	21
Ozone production	23
Ozone transfer into water	20
Auxiliary equipment	2
Total	66

tion of ozone and its transfer into the water by the ozone disinfection plant of the municipal water supply in Berne, Switzerland.[2] This plant disinfects 5.4 mgd of polluted spring water by the application of approximately 0.5 mg of ozone per liter of water. The disinfection of one mgd can be achieved by a total energy uptake of only 120 kilowatt hours.

Chemical and Physical-chemical Properties of Ozone Solutions

An appropriate knowledge of the chemical properties of a disinfectant is a necessary background for the understanding of its disinfecting behavior. The information on chlorination chemistry, for instance, has proved to be of major importance for the intelligent application of chlorine and its compounds for disinfecting purposes. The chemical and physical-chemical properties of ozone, however, are not yet adequately known. This lack of knowledge has obstructed the collection of consistent data on the germicidal properties of ozone.

Ozone Demand

Ozone is next to fluorine in the list of powerful oxidizing agents. It reacts with inorganic and organic reducing substances to produce an "ozone demand." The reaction of organic substances is generally more rapid and more extensive with ozone than with hypochlorous acid. This would indicate a rapid reaction rate of the disinfectant with the vital centers of the microorganisms. The higher oxidation potential of ozone as compared to chlorine is certainly of advantage with respect to removal of color and taste producing constituents in the water. The strong tendency of ozone to combine with reducing substances may, however, also be a disadvantage. Large dosages are required in solutions that contain oxidizable constituents to acquire "free ozone." The "ozone demand" of natural waters is generally larger than the corresponding chlorine demand. The "ozone demand" of a sewage may even be exhorbitant.[3,4]

Ozone Decomposition

Ozone in solution is unstable and decomposes into oxygen. The decomposition is temperature dependent and is strongly catalyzed by trace concentrations of many inorganic and organic constituents of the water. The decomposition rate is especially dependent on the hydroxyl ion concentration (Figure 1).[5] For this reason, ozone does not persist within the treated water. It provides no "residual protection" against recontamination. Attributable to this rapid decomposition, the water disinfected by ozone, on the other hand, is free of odor and the taste of the germicidal agent.

The Intermediates in Ozone Decomposition

The mechanism of ozone decomposition is not fully understood. The strong dependence of the rate of decomposition upon pH and the decomposition catalysis by trace quantities of water constituents can only be explained by assuming that the decomposition occurs in a

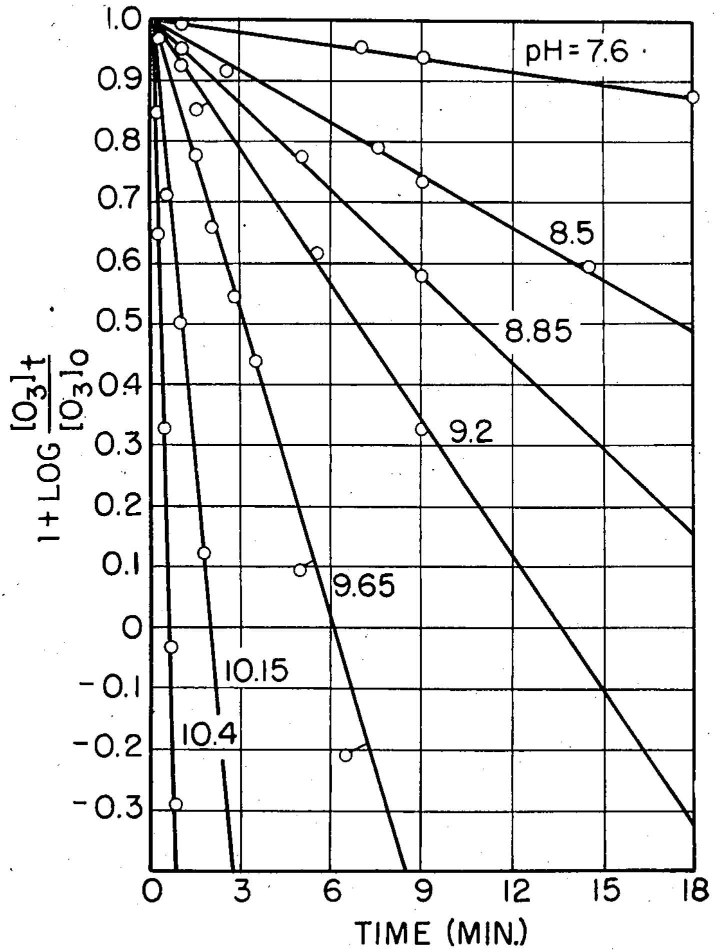

FIG. 1.—DECOMPOSITION OF OZONE IN AQUEOUS SOLUTIONS IN DEPENDENCE OF pH (Stumm[5])

Temperature: 14.6° C; Ionic Strength: $\mu = 0.05$;
Buffer system: $CO_2/NaHCO_3/Na_2CO_3$

stepwise fashion producing, in turn, short lived radicals. Alder and Hill,[6] for example, suggest the following mechanism:

$$
\begin{array}{l}
O_3 + H_2O \rightarrow HO_3^+ + OH^- \\
H_3^+ + OH^- \rightleftharpoons 2HO_2 \\
O_3 + HO_2 \rightarrow HO + 2O_2 \\
HO + HO_2 \rightarrow H_2O + O_2 \\
\hline
2O_3 \rightarrow 3O_2
\end{array}
$$

Despite the considerable controversy on the mechanism of the decomposition reaction, most investigators[5,6,7] agree that the radicals HO_2 and OH are intermediates. We do not know definitely whether these intermediates are of any significance for the interpretation of the chemical and germicidal properties of ozone solutions.

It has been confirmed that the same radicals (OH and HO_2) are produced by irradiation (e.g. x-rays, β-rays) of aerated water.[8,9] The chemical reactions in which these free radicals take part are known to some extent from radiation chemistry. Whether the radicals react with dissolved substances or combine with one another depends on their concentration and on the susceptibility to attack of the dissolved material. The radicals are capable of oxidizing efficiently inorganic[10] and organic[11] constituents and of inactivating enzyme systems.[8] Some investigators assume that OH and HO_2 contribute significantly to the killing of microorganisms by irradiation.[8,12]

These considerations sustain the possible importance of the decomposition intermediates in the reaction behavior of ozone.

Solubility of Ozone and Ozone Uptake by Different Types of Water

Ozone is about 10-12 times more soluble than oxygen, but, because it is at low partial pressure, the maximum concentration that can be obtained in solution is only a few mg/1. Table 2 gives the distribution coefficient in relation to temperature.[3]

Figure 2 represents schematically the ozone uptake by different types of water.[3] Ozonized air is introduced continuously and at a constant rate into different waters under the same mixing conditions. The concentration of "free ozone" is plotted as a function of time, which is representative of the amount of ozone added to the solution. Curve A is obtained with a pure "ozone demand free" water. It is a typical first order gas absorption curve. Curve B is obtained with a

TABLE 2

Solubility Coefficient for Ozone in Water (Stumm[3])

$$g = \frac{O_3\ (\text{Water})}{O_3\ (\text{Gas})}\ (\text{at 1 Atm.})\ \text{Ionic Strength: } \mu = 0.05$$

Temperature, °C	5	10	15	20	25
Solubility coefficient, g	0.45	0.41	0.37	0.34	0.30

The solubility is independent of pH up to pH = 8.5.

water that exerts an ozone demand. A substantially completed oxidation-reduction process takes place before attaining "residual ozone." Thereafter the ozone uptake is the same as with an "ozone demand free" water. Such curves are frequently obtained with the ozonization of ground waters. With some waters (e.g. surface water, sewage)

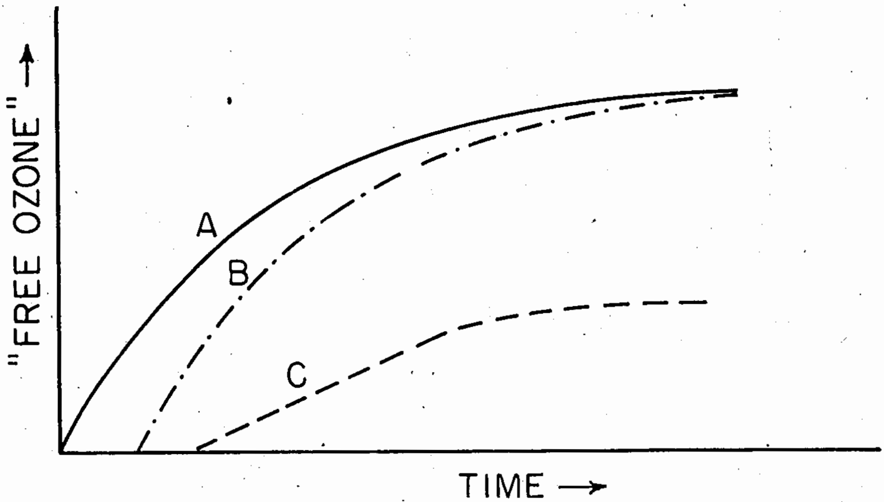

FIG. 2.—OZONE UPTAKE BY DIFFERENT TYPES OF WATER (Schematically, Stumm[3])

curves of type C are customary. In addition to an ozone demand, the rate of uptake of free ozone is decreased due to an additional ozone consumption caused by slow oxidation reactions and by increased catalysis of ozone decomposition.

Ozone Analysis

No analytical technique has yet been developed that permits a sensitive and specific quantitative determination of residual ozone.

The most specific method, the spectrophotometric measurement of absorption of ultraviolet light,[4] is not very sensitive. More sentitive methods reported in the literature (oxidation of o-Tolidin,[4] Mn^{+2},[13] Fe^{+2},[14] Indigo,[15] Iodide[16]) are principally based on the determination of the total oxidizing capacity of the solution rather than the amount of residual ozone only and are not entirely specific. Oxidized inorganic (Iron, Manganese, etc.) and organic constituents and possibly the decomposition intermediates interfere.

Germicidal Properties of Ozone Solutions

The lack of knowledge on the chemistry and kinetics of ozone solutions and the uncertainty of analytical techniques are serious obstacles in all investigations on ozone disinfection. The usual process of bubbling ozone through a bacterial suspension and then measuring the total oxidizable residue may fail to give appropriate estimates of the concentration of bacteriacidal "free ozone." Other investigators have added definite amounts of ozone to a suspension of microorganisms and report values for "lethal ozone dosage." Such values are not necessarily representative, because each particular test solution, even bacterial suspensions in "ozone demand free" water, must be assumed to have a certain ozone consumption. (Minute amounts of nutrients attached to the bacteria exert an ozone demand and catalyze the ozone decomposition strongly.)

A very careful investigation on the kinetics of ozone disinfection has been made by Wuhrmann and Meyrath.[17] Some of their results are summarized in Figures 3 and 4. During each experiment the ozone concentration was kept constant by continuously bubbling air containing small amounts of ozone through the solutions. The results indicate that ozone disinfection is mainly a function of contact time, concentration of ozone, and temperature of the water. The disinfection mechanism, seemingly, is similar to HOCl disinfection. Their investigations reveal that the contact time necessary for 99% destruction of E. Coli is seven times smaller with ozone than with the same amount of hypochlorous acid, while the killing rate for spores (B. megatherium cereus) is about 300 times larger with ozone than with chlorine.

Fetner and Ingols,[18] however, postulate on the basis of their investigations with E. Coli that ozone has a different mode of action than chlorine. They claim that ozone shows an "all-or-none type of effect" within the contact time of one minute. They found no effect of the ozone below a certain critical concentration and above this concentration no detectable survivors.

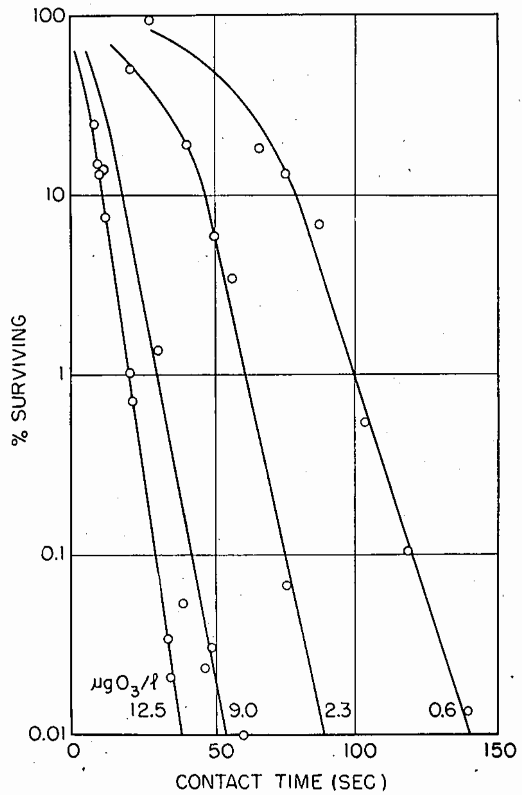

FIG. 3.—BACTERICIDAL ACTION OF OZONE AGAINST *E. Coli* (Wuhrmann and Meyrath[17])
Temperature: 12° C; pH = 7.0; Buffer: $CO_2/NaHCO_3$

Bringmann[19] reports that ozone destroys the following organisms more rapidly than chlorine: E. Coli, spores, different algae, and protozoa. His results indicate that ozone has a considerably higher lethal efficiency than chlorine especially for organisms which exhibit high resistance to disinfection by chlorine.

According to Newton and Jones[20] over 99% of cysts of E. histolytica were killed in water within 1 - 3 minutes after the application of 0.5 - 1 mg/1 of ozone. Hettche and Ehlbeck[21] claim that 0.15 mg/1 ozone inactivates polio virus.

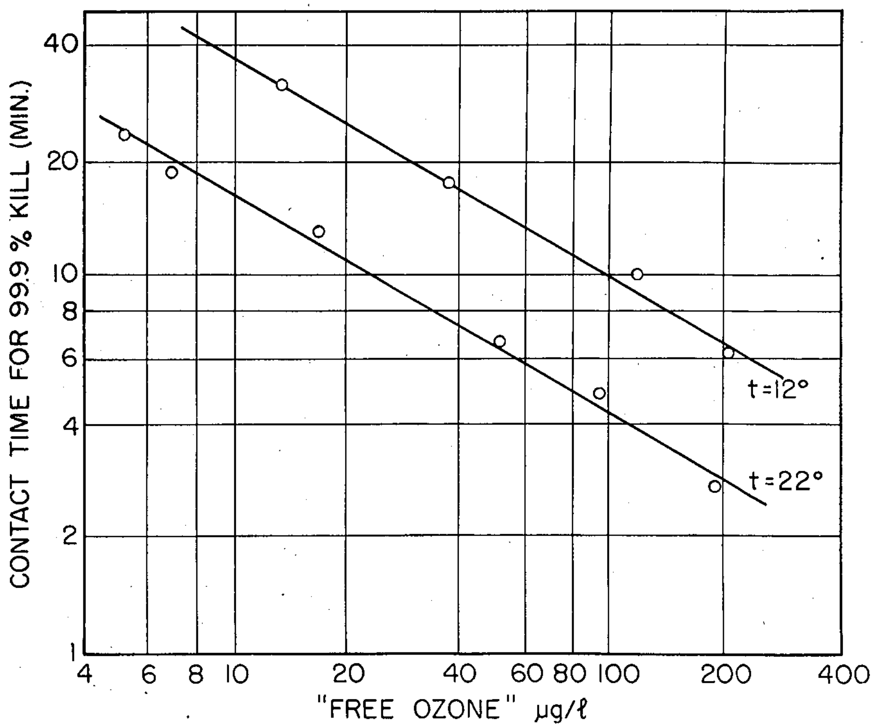

FIG. 4.—BACTERICIDAL ACTION OF OZONE AGAINST SPORES OF *B. megatherium/cereus* (Wuhrmann and Meyrath[17])
Age of the spores: 77-79 days; pH = 7.2; Buffer: $CO_2/NaHCO_3$; Temperature coefficient: θ = 2.3 per 10° C.

Sewage Disinfection

The constituents of a sewage cause a remarkable ozone consumption. Therefore one would assume that they will heavily interfere in the ozone disinfection and that suspended matter will efficiently shelter embedded organisms against ozone attack.

Some recent reports, however, claim the feasibility of sewage disinfection. A Research Group of the Armour Research Foundation, Chicago, Illinois, and of the Biological Warfare Laboratories, Fort Detrick, Maryland,[22] investigated the possibilities of disinfection and sterilization of sewage by ozone. Their investigations are mainly related to the problem of sterilization of liquid effluents from infectious disease laboratories. These effluents are sterilized today by prolonged heat treatment at 300°F. Their laboratory results indicate that ozone can be successfully used for sterilization of sewage containing Bacillus anthracis, influenza virus, and Bacillus subtilus morphotype globigii (Bg), and for inactivation of toxin of Clostridium botulinum. To obtain sterility the ozone consumption in most experiments was between 50 and 115 mg ozone per liter. The time required to obtain complete sterilization was, with only a few exceptions, below 30 minutes.

Mechanism of Ozone Disinfection

Most investigators agree that the mechanism of disinfection by ozone is similar to that of chlorine. It is assumed that both inactivate essential enzymes of the cells. An important step in the action of the disinfectant seems to be the penetration of the cell wall. Ozone apparently diffuses very well through the cell membrane.

In applying the criterion of free ozone it is assumed that the effectiveness of the bactericidal action is a function of the ozone remaining. In the disinfection of polluted waters or sewage it seems, nevertheless, possible that ozone molecules may achieve the killing of microorganisms while simultaneously satisfying the ozone demand. Leiguardia's[23] investigations support the assumption that the bactericidal action of ozone proceeds parallel with the oxidation of organic matter. It may be inferred that the particles that are produced by the ozonization of water or sewage are, to some extent, also effective in reducing the concentration of microorganisms. Oxidized organic constituents (for example, ozonides which are structurally identical with organic peroxides) may be toxic to microorganisms. (Indications exist that organic peroxides are more bactericidal than hydrogen peroxide.) The possibility that the decomposition intermediates of ozone, the perhydroxyl and hydroxyl radicals, are able to interact with the molecules of cell cytoplasm and contribute to effective disinfection cannot be excluded.

Discussion

The information presented here gives sufficient evidence that ozone has to be considered as a potential disinfectant for water and sewage. It is available at reasonable cost, easy to handle, and can conveniently be applied to the water. Ozone seems to be especially promising with respect to the destruction of those pathogenic organisms that are highly resistant toward the action of chlorine compounds: cysts, some viruses, etc.

Our knowledge, at the present time, of ozone as a disinfectant is not adequate, and more investigations are needed to evaluate the applicability of ozone as a practical tool for the sanitary engineer in water treatment and waste-water disposal.

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OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

Boston Society of Civil Engineers

NOVEMBER 20, 1957.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the United Community Service Building, 14 Somerset Street, Boston, Mass., and was called to order by President Arthur Casagrande, at 7:05 P.M.

President Casagrande stated that the minutes of the previous meeting October 23, 1957 would be published in a forthcoming issue of the JOURNAL and that the reading of those minutes would be waived unless there was objection.

President Casagrande announced the death of the following members:—

Louis A. Chase, who was elected a member November 17, 1926, and who died November 3, 1957.

Robert Gillespie, who was elected a member December 2, 1925, and who died October 5, 1957.

Arthur E. Harding, who was elected a member November 16, 1921, and who died October 4, 1957.

Edward F. Kelley, who was elected a member May 15, 1935, and who died October 29, 1957.

Arthur A. Shurcliff, who was elected a member November 19, 1924, and who died November 12, 1957.

The Secretary announced the names of applicants for membership in the BSCE and that the following had been elected to membership:—

October 23, 1957

Grade of Member—Kriker Ermonian, Royal C. Flanders, Donald H. Hastie, Leo G. Keefe, Anthony P. LaRosa, Roert S. Larsen, Harold K. McAfee, Andrew C. Patch, Leonard J. Peterson, Alexander Sibbald, John W. Towers, Frederick T. Webb.

Grade of Junior—John J. Chisholm, Ellis L. Chouinard, Nicholas R. D'Alessandro, Nicholas H. Fitzgerald, Jr., Richard L. Foster, Allan P. Giovannini, Arthur R. Giangrande, John P. Hickey, Jr., Richard D. Howard, Robert H. McDonnell, William Moy, George H. Power, Wallace G. Sanborn, Peter P. Saunders, Angelo B. Veneziano.

November 18, 1957

Grade of Member—Francis E. Carty, Charles E. Cannon, David F. Greenwood, Lewis H. Holzman, Robert E. McQuade, Max D. Sorota, James M. Symons.

Grade of Junior—Frank A. Marino.

President Casagrande introduced the speaker of the evening, Philip C. Rutledge, Moran, Proctor, Mueser & Rutledge, New York, who gave a most interesting talk on "Philosophy of Pile Foundations".

A brief discussion period followed after which a collation was served.

One hundred and five members and guests attended the meeting.

The meeting adjourned at 9:05 P.M.

ROBERT W. MOIR, *Secretary*

DECEMBER 18, 1957.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the United Community Service Building, 14 Somerset Street, Boston, Mass., and was called to order by President Arthur Casagrande, at 7:05 P.M.

President Casagrande announced the death of the following member:—

John R. Dyer, who was elected a member February 18, 1931, and who died December 15, 1957.

The Secretary announced the names of applicants for membership in the BSCE and that the following had been elected to membership December 16, 1957.

Grade of Member—Robert M. Lincoln, Paul F. Pierce, Lawrence J. Tierney.

Grade of Student—Harry L. Guzelimian.

President Casagrande introduced the speaker of the evening, Dr. Guilio Pizetti of Italy, visiting Professor at M.I.T., who gave a most interesting illustrated talk on "A Consideration of New Trends in Reinforced Concrete".

A brief discussion period followed after which a collation was served.

Sixty-six members and guests attended the meeting.

The meeting adjourned at 8:35 P.M.

ROBERT W. MOIR, *Secretary*

SANITARY SECTION

DECEMBER 4, 1957.—The meeting was called to order by Chairman Flaherty at 7:00 P.M. at the Society Rooms after an informal dinner at Pattens Restaurant. Twelve members and guests attended the dinner and forty-five members and guests attended the meeting.

Chairman Flaherty announced the coming meetings of the Section and then called for nominations for a Nominating Committee to submit a list of nominees for officers for the Executive Committee for the year 1957 to 1958. The following were nominated by Mr. Reece:

Professor Edward W. Moore
Mr. Ariel A. Thomas
Mr. Darrell A. Root

Mr. Dallas moved that the nominations be closed and the aforementioned be elected which motion was seconded and accepted by unanimous vote. The Chairman then introduced Doctor Leslie Silverman, Associate Professor of Industrial Hygiene at Harvard University, School of Public Health, who gave a talk on "Air Pollution Control Engineering". The talk included a resume of the air pollution problem as it exists today, the kinds of problems encountered and the nature of the technical training necessary; also the activities of Sanitary and Industrial Engineers in this field, a brief summary of air pollution control engineering research needed and the activities of the Harvard Air Cleaning Laboratory in this field. The talk was illustrated by slides, and a tape recording was made. At the conclusion of the paper, there was an interesting discussion between the speaker and the members and guests.

The meeting was adjourned at 9:00 P.M.

JOHN C. FLAHERTY, *Chairman, Clerk, pro tem*

HYDRAULICS SECTION

NOVEMBER 6, 1957.—A meeting of the Hydraulics Section was held at the new Society Rooms, 20 Pemberton Square, Boston, Mass., and was called to order by Clyde W. Hubbard, Chairman, at 7:30 P.M.

The Chairman announced that the next meeting would be held jointly with the Sanitary Section and the full Society with Professor Harold A. Thomas, Jr., of Harvard University as the speaker on the subject of "Ground Water Levels in New England.

The chairman called on the clerk to summarize the minutes of the meetings of May 11 and February 6 which had been published in the Society Journal.

Robert W. Moir, Secretary of the Society, moved that future section meetings be held at 7:00 P.M., and his motion was seconded and carried.

The chairman introduced the speaker of the evening, Robert Stevenson Kleinschmidt, of the Research Staff of Harvard University, whose subject was "Design of Hydraulic Laboratory for Instruction and Research".

Mr. Kleinschmidt's talk covered the genesis and evolution of the new hydraulic laboratory at Harvard which is now partially completed. Mr. Kleinschmidt, who has had a major role in the planning, design and construction of the new laboratory, outlined the history of engineering training at Harvard and explained that the Gordon McKay bequest for Mechanical Engineering was being tapped for the new lab which would be used for both instruction and research by students of civil and mechanical engineering. The speaker, illustrating his talk with excellent slides, explained how the old Pierce Hall was being adapted to the lab and described the facilities of the new lab which will center around two flumes: one, a large, glass-walled, flatbottom flume; the other, a tilting "water table" for shooting flow experiments. A question and discussion period followed the speaker's prepared talk.

Total attendance at the meeting was twenty-one.

The meeting adjourned at 8:50 P.M.

LEE M. G. WOLMAN, *Clerk*

ADDITIONS

Members

Charles E. Cannon, 50 Barrett St., Needham, Mass.

Francis E. Carty, 9 Alban St., Dorchester, Mass.

David Greenwood, 40 Ripley Road, Dorchester, Mass.

Lewis H. Holzman, 28 Larned St., Framingham, Mass.

Leo G. Keefe, 39 Nevada Road, Needham, Mass.

Robert M. Lincoln, 85 Homstead Avenue, Weymouth, Mass.

Harold K. McAfee, 92 Thurber Avenue, Brockton, Mass.

Paul F. Pierce, Gilbert Stuart Road, Saunderstown, R. I.

Max D. Sorota, 23 Canton St., Lowell, Mass.

James M. Symons, 1-083 Mass. Inst. Technology, Cambridge, Mass.

Robert G. Esterberg, 188 County Way, Scituate, Mass.

Juniors

Allan P. Giovannini, 276 West Elm St., Brockton, Mass.

Frank A. Marino, 146 Prospect St., Lawrence, Mass.

DEATHS

John R. Dyer, Dec. 12, 1957.

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