# SYMPOSIUM PROCEEDINGS

# USING GEOMECHANICS IN FOUNDATION ENGINEERING

WANGANUI

SEPTEMBER 1972

NEW ZEALAND GEOMECHANICS SOCIETY

# PROCEEDINGS OF A SYMPOSIUM

ON

# USING GEOMECHANICS

IN

# FOUNDATION ENGINEERING

Held at the War Memorial Lecture Theatre, Wanganui

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# GEOMECHANICS IN FOUNDATION ENGINEERING

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As first speaker in this Symposium it is my task to give you some idea of its purpose and to explain in broad terms the manner in which it is arranged. I leave it to the experts who come after me to carry this purpose into effect. Each will be speaking as an individual from his own experience and though all are working within a broad framework laid down by the Society there will undoubtedly be points on which individual speakers disagree. There will also, I hope, be areas in which disagreement will be expressed from the floor. If we all agree about everything there is little purpose in running such a meeting as this. It is only where points of view differ that discussion can develop, and it is from discussion that we mutually learn.

The first thing I would like to do is to define some of the terms we will be using in this Symposium. The first is "Geomechanics".

Geomechanics arrived on the New Zealand scene with the formation, earlier this year, of the N.Z. Geomechanics Society. This Society developed from and replaces the former N.Z. National Society for Soil Mechanics and Foundation Engineering. In the same way Geomechanics has its roots in Soil Mechanics but this is only one of its three main sources of inspiration. The other two are Rock Mechanics and Engineering Geology but all of the mechanical sciences which can be applied to the surface of the earth - geology, geophysics, seismology etc. - contribute. I hope the meaning of Geomechanics will become clear to you during the course of the Symposium, but if a formal definition is necessary, and I believe it is, the best I am able to provide is "Geomechanics is the practical application of the mechanical sciences to the development of the surface of the earth".

It is not appropriate here to define all of the mechanical sciences which are included in geomechanics but the three main ones do need to be mentioned.

Engineering geology is concerned with the interpretation of the science of geology in engineering terms. For example in the geological time scale the whole of man's evolution has occupied an infinitesimal interval and what, in geological terms is a rapid deterioration may provide, in engineering terms, a useful life of tens or hundreds of years.

Both Rock and Soil Mechanics are concerned with the behaviour under load of the materials of the surface of the earth - what forces they can safely bear, how these forces are transmitted from particle to particle, what are the mechanisms of failure, the effects of pore fluid pressure and change in moisture content, what natural stresses exist, how past history has affected their properties etc.

With these definitions in mind it is easy to see how fundamental Geomechanics is to man's economic development. Obviously all mining and earthworks are included and many other developments such as roads, airports, docks and reservoirs. With the exception of air, and, infrequently, water, the development of every mineral resource depends heavily on Geomechanics. Even that favourite material of the structural engineer, reinforced concrete, involves Geomechanics to a surprising degree. In the winning of the raw materials this connection is obvious, but it extends right through to the mixed concrete. The desirable properties of economy, workability, and lack of bleeding in the plastic concrete are achieved by the application of such soil mechanics concepts as particle interaction and packing and pore fluid pressures.

From this vast range of activities we will be concentrating on one small sector only, the application of geomechanics principles to foundation engineering. But that only a limited area is being considered does not detract from its importance - nor its fascination!

It is also necessary to consider the meaning of the terms "Foundation" and "Foundation Engineering" as they will be used in this Symposium. "Foundation" is a word which is used in different ways by different people. To the architect or structural engineer it tends to be thought of as the bottommost part of the structure - the piles, footings, beams and slabs - which sit upon the earth and take the forces imposed by the building. To the practitioner of geomechanics (or geomechanicist) "foundation" tends to mean the material of the earth which underlies the structure and reacts to the forces imposed by it. Both are looking at the same problem from different sides but using the same word to describe two completely different concepts. To a large extent the structural engineer has his "foundation" problems created and defined by himself or others and thus can quickly decide which of the available methods of solution are appropriate. The geomechanicist on the other hand spends a great deal of his effort in trying to determine which problems Mother Nature, or sometimes other men, have hidden within the "foundation". Once he has solved this riddle he usually has little difficulty in choosing an appropriate method of solution. In the structural engineering context complex and precise solutions are appropriate, and warranted, but in the context of geomechanics the identification of the problem is usually so imprecise at the design stage that rather elementary and approximate solutions are more appropriate. On the one hand the structural engineer does not always understand why the geomechanicist cannot give him a "safe bearing pressure" or some such definite value to be used in his precise solutions. On the other hand the geomechanicist does not always appreciate that the structural engineer is working in a relatively precise context where "factors of safety" have some meaning. In geomechanics "factors of ignorance" is a much more appropriate term.

It is this gulf of misunderstanding that the foundation engineer must try to bridge and with which this symposium attempts to deal.

In sponsoring this symposium the Geomechanics Society had a very specific objective in mind. This was to follow on from the Site Investigation Symposium held in Christchurch in 1969 and propound and discuss the methods we believe are essential to the proper application of geomechanics in real life foundation engineering. Frankly we are here to teach. But also we hope to learn, both from each other and from the contributions of participants. It is a fundamental fact of foundation engineering that at all times one must keep ones mind receptive to new information, assess it and fit it into its place in the overall pattern. While the Christchurch symposium dealt with the problems, practices and philosophy of the site investigation process this symposium deals with the succeeding steps in the development of the site up to the successful completion of the building. It will be assumed that the site investigation has been completed to the extent that the feasibility of erecting a building of the proposed general type has been established and a general idea formed of the likely foundation conditions to be expected. But it will also be made clear that this is by no means the end point to the site investigation process. It continues for much longer, and it is not till the structure is built and has proved itself in service that the foundation engineering can be said to be finished. And in New Zealand it may be many years before the full service conditions - for example design earthquake - are encountered.

To serve this purpose the symposium has been divided into three sessions each dealing with a certain aspect of the process of foundation engineering. The first session is concerned with the "input" to the problem - the field data provided by the site investigation, the clients brief and the processes of reasoning which are used to assess this data and shape it into plausible conceptual models of the real situation. The second session is devoted to the tools available to the designer to reconcile the clients brief with the facts of the situation - the types of structural arrangement, the methods of obtaining solutions, the special tests that will be necessary to fill in the details of the site as the conception of possible solutions crystalise in the designers mind - and how the relative usefulness of these tools can be assessed in the actual circumstances as they are brought to light. The third session is concerned with the building phase; how the adopted solution can ease or aggravate the contractors problems; how the design can be modified to mitigate these problems or overcome the awful truth revealed when the foundation excavation gets under way; what predictions can be made about the performance of the structure and how these predictions can be checked against actual behaviour; in what respects, and by how much, performance can be permitted to deviate from predictions; and what to do about it all!

These three groupings are very broad, arbitrary, and with ill defined boundaries. There is good reason for this. The whole process of foundation engineering is cyclic. One starts with an armoury of thought processes, relevant and irrelevant standard solutions, client requirements, even preconceived ideas on the one hand and an array of facts, approximate data, folklore and downright false information about what exists at the site on the other. From these two "carpet bags" one has to construct a plausible model of the real situation and "try it for size". Inevitably there will be inconsistencies. After all the initial site investigation will only have brought to the surface something less than one part in one hundred thousand of the total material; much of that will be butchered in the process; so one has to extrapolate perhaps half a million fold in constructing a plausible model. Other models will fit almost as well overall, and better in some respects; facts and folklore will be hard to distinguish; the relevance of standard solutions will be obscure. So further information has to be obtained, the plausibility of the various models tested against it and a new set of "better" models postulated. To decide between these "better" models further data is required and so the process goes on, round and round till an adequate degree of certainty has been reached. And what may be an adequate degree of certainty for a reservoir may be excessive for a commercial building and totally insufficient for a nuclear reactor.

Nor should the cyclic process stop once the model to be used for design has been selected. As the site is excavated, and the structure built and put into service, the model can be used to predict performance. These predictions can then be compared with actual behaviour and any difference between prediction and behaviour used to further refine the model to allow further predictions to be made and the continuing adequacy of the design to be assessed.

This repeated process of postulating a model of the problem, predicing from the model, checking prediction against new (and old) data, modifying the model in the light of differences from prediction, making new predictions, and obtaining new data to check these is the essence of foundation engineering. It serves two very important functions. The first is to ensure the safety, adequacy and success of a job. Here the number of cycles will depend on the importance of the job, the risks involved and how close the clients demands come to the ultimate capacity of the site. The second is to ensure that the foundation engineer learns something new each time he works round the cycle - and the basic lesson he must learn, and never forget, is that Mother Nature is capable of infinite complexity when so minded. If he ceases to learn from his experience he falls into the error of believing that "one years experience repeated ten times is the same as ten years experience". Terzaghi had some very hard things to say about that belief!

One other aspect that was very much in the Society's mind when sponsoring this symposium was the great dearth of guidance in literature, and in most academic courses, to the real core of foundation engineering - how to recognise and describe the problems that in fact exist at a site. Most academic courses concentrate on formal aspects such as fundamental behaviour theory and ways of solving pre-set problems. In a qualification-by-exam context this is quite understandable as it makes the setting and marking of exam papers to a uniform standard a little easier. Similarly text books need publishers, and publishers have to make a living. Thus it is the texts that can secure a steady sale that get published, and one of the most likely routes to steady sales is to become accepted as a good "teaching" text. Thus teachers and publishers mutually tend to reinforce this concentration on the formal. Undeniably an understanding of fundamental behaviour theories and a good knowledge of methods of solving problems are basic skills and should be taught in all branches of engineering. In many areas where one is dealing with manufactured materials whose properties can be defined by standard specifications and whose critical dimensions can be calculated and made within small tolerances the formal studies are fairly adequate. But in foundation engineering many properties and dimensions cannot be specified or decided beforehand. They have to be determined for each case by methods which are rough and ready and which, having been devised to suit one particular set of circumstances, cannot be regarded as universally applicable.

Thus this Symposium is not intended to be the ultimate recipe book of foundation engineering. Were such a recipe book ever written it would contain so great a variety of specifics that it would be impossibly bulky and virtually unusable. Instead it concentrates on the thought patterns that underlie the whole process - the need to consider a wide range of possibilities in every case, to assess <u>all</u> the available information, to continually test this information against new data as it comes to hand, to be prepared to abandon a cherished theory when it no longer fits the facts and even when a course of action has been embarked upon to continually test prediction against performance and hold in reserve alternative courses of action which can be adopted when the gap between prediction and performance becomes too large to accept.

Finally I would like to add a personal warning on the cardinal sin of pride. It is very easy to be carried away with pride in the elegance or sophistication of a particular solution that one has devised and to try the same thing next time without first critically considering its relevance. Mother Nature lies in wait for such people! A solution that starts off being too-clever-by-half almost always gets into difficulties. Many times I have seen a solution originally simple in conception stretched to its limits by the complexities that lay hidden in the site; but at least it could be stretched to meet the case, whereas a more complex conception would have run out of "give" long before. The elegance of a solution may give its deviser a great deal of personal satisfaction, but the client may have other criteria of success - such as

"Does it do the job I wanted done?"

"Did it cost more than it should?"

"Was it ready on time?"

"Were unwarranted risks run or unnecessary precautions taken?"

These may be prosaic questions, but it is by prosaic standards that most engineering jobs are finally judged.

# THE LOGIC OF FOUNDATION ENGINEERING

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## 1. INTRODUCTION

The application of geomechanics to real-life foundation engineering tasks automatically brings one face to face with the complexities of actual subsurface conditions. Frequently there are sites where the materials and conditions are difficult to quantify before construction proceeds. All too often the complexities, difficulties or the limitations of available procedures are given as justification for the notion that the "engineering of foundations" must be regarded as an art in which competence is only acquired in some ill-defined way after long experience. This is a fallacious attitude and disguises the really important need which is for clear and logical thinking exercised within a planned, systematic process of deduction and decision making. This paper defines and discusses the necessary "thinking framework" and the key facets of which it is comprised.

## 2. EXPERIENCE AND JUDGEMENT

The umbrella terms "engineering experience" or "judgement" are frequently used, albeit unconsciously, to dignify a decision based on hunches, a hope-it-will-work feeling, or simply an unwillingness or inability within the available resources to get to grips with a given situation. These terms are wide open to the vagaries of individual opinion and while there will almost always be an element of engineering judgement in any engineering solution, it is important to strive to keep this to a minimum. Moreover and paradoxically, where a task is ill-defined or difficult to put into a systematic form, there is an even greater need for a critical appraisal of the facts, assumptions, and bases leading to the resulting engineering decisions.

Despite these somewhat derogatory comments and lest one gets the wrong impression, it is not intended to totally discredit the value of experience. When correctly interpreted, experience can be an important and sometimes the only real source of information. However this value will only be realised if a particular experience is analysed to determine which parts are relevant to a given new situation.

## 3. REQUIRED GENERAL APPROACH

### 3.1 Objective

The primary objective in any foundation engineering task is the provision of an adequate foundation for a specific structure. The degree of success in achieving this objective will be judged in terms of economy, acceptable level of risk, and general fulfillment of the client's brief. It is important to define clearly the endpoint target at the outset, in terms which express the client's needs, and to constantly keep this to the fore.

### 3.2 Recognise the Problem

Within the boundaries set by the client's brief, etc., the key issue in a geomechanics task is to recognise the importance of correctly identifying the technical problem one is trying to solve. For example there would be little point in obtaining laboratory compressibility and strength data for a soft clay overlying dense gravel if bore log descriptions clearly showed that the clay would be an inadequate foundation for the structure envisaged. This kind of comment may seem obvious and axiomatic, but all too often a failure to correctly identify the problem is the root cause of inadequate or wrong solutions, unforeseen problems, wrong interpretations of information obtained and waste of money. Success here will of course depend on the data, expertise, etc. available but one should continually be on the lookout for unjustified assumptions.

## 3.3 The Approach

The approach to any task should follow concurrent and parallel paths which in principle are:

(a) Proceed from the "broad" to the "particular" by a deliberate process of deduction.

(b) Recognise the known facts and boundaries, define the key elements of the problem, and thence direct the investigations, inquiries, analyses etc. towards an appropriate solution. Broad scale information such as the terms of reference, main boundary constraints, the broad aspects of materials knowledge (e.g. geological information) should be examined first because this will usually set boundaries around the task and limit the area and extent of further enquiry. It is often surprising how closely site conditions can be defined from the broad aspects (e.g. geology, landscape, etc.), on-site inspection of materials and conditions and the application of the principles of soil mechanics together with an understanding of the form and consequences of various natural soil forming processes. Frequently, the resulting understanding will indicate feasible alternative treatments or strategies, and the additional investigations, analyses, etc. that are warranted. For example the geology may quite clearly limit the range of materials and conditions which can be expected and show that a conventional borehole investigation would be unproductive. Alternatively, the broad scale information may show that while further quantification was desirable, it would be an unacceptable strategy within the resources available. Hence some other approach would be required, e.g. allow for revision of the foundation design after the job is started.

Throughout the examination of a foundation task it is important to distinguish between the "known" and the "unknown", i.e. to clearly identify established, irrefutable facts and discriminate between these and non-proven aspects such as assumptions, hypotheses, opinions, interpretations, engineering judgement, hunches, etc. This requires very clear and logical thinking together with an almost ruthless attitude of mind otherwise there are many pitfalls and a real danger of unsubstantiated conclusions. However it is reiterated that it is not intended to dismiss the value of experience or engineering judgement; both are often very valuable sources of information and guidance but must first be critically analysed to determine which aspects are relevant to the new situation.

### 3.4 Progressive Refinement

The recommended approach is a step-by-step process of narrowing down the degree of uncertainty such that the form of a task and appropriate solutions become progressively more clearly defined. Throughout there must be a willingness to repeatedly review hypotheses, deductions and conclusions as more information is obtained. At any stage of refinement, risks and consequences must be assessed but again these are conditioned by the reliability of the data on which the assessments are based. Invariably there will be alternative solutions which can be considered i.e. alternative strategies must be examined. Obviously the relative merits of different strategies will depend on the nature of the task, materials, etc. and the degree to which these can be quantified at the various stages of the design.

## 4. KEY ASPECTS

The primary purpose of this paper is to set down the complete logic sequence for dealing with foundation engineering tasks. However first let's examine briefly some of the key operations, and areas of thinking and decision making which are inherent in the process.

### 4.1 Start Data

This establishes the boundaries and known facts of a situation and includes:

(a) The client's brief which should specify the size, shape, form, purpose, location, etc. of the structure together with economic restraints.

(b) Existing information about site and subsurface conditions. This includes previous site investigation data and reports; geological and other broad scale data; and information that can be gleaned from observation at or adjacent to the site.

### 4.2 Principles of Material Behaviour

Foundation engineering requires a sound grasp of what is best described as the <u>principles</u> of material behaviour. This includes:

(a) The various geological processes by which materials are formed and particularly the nature of the resulting products; this gives an ability to predict materials and conditions most likely to be encountered e.g. weathering products; transportation by water, wind or gravity and hence various degrees of sorting; volcanically produced or derived materials; and so on.

(b) Soil mechanics <u>principles</u> (as distinct from detailed test procedures or methods of analysis) and how these relate to various materials and configurations. Knowledge of the limitations of available sampling, testing and site investigation techniques is also necessary but these aspects will be dealt with in other papers of this symposium.

### 4.3 Configuration of Materials and Conditions

A sound understanding of the particular "configuration of the materials and conditions" is a fundamental requirement in any geomechanics task. It must be deduced with an open mind which is free from preconceived ideas of what the situation or solution to the task might be. Similarly those aspects which are not known or understood must be freely admitted. Without this frank and honest approach a situation can be easily, albeit unintentionally, prejudged or misunderstood to the extent that subsequent decisions would in fact be based largely on unsubstantiated assumptions or assertions, and not the true facts of the case. The configuration of the materials and conditions is a basic requirement of the following:

(a) Establishing a hypothesis to fit the known facts for the particular materials and conditions. This important step must always be regarded as subject to modification as new facts come to hand but should give the best 'picture' of a given situation at any particular time.

(b) Determining the investigation(s) and/or testing desirable or warranted for further clarification of the problem, or verification or otherwise of the postulated behaviour.

(c) Determining a simplified model to represent the real life complex conditions. This modelling step is nearly always needed before analyses can proceed (see 4.6 below).

(d) Deciding whether meaningful analyses are feasible.

### 4.4 Assumptions and Hypotheses

Geomechanics situations will rarely be completely quantified so assumptions are inevitable throughout the whole design process. They are especially pertinent when deciding either the configuration of materials, or the suitability of tests, methods of analysis and investigation techniques. It is important to examine the sensitivity of decisions to feasible changes or errors in the assumptions.

In a similar way, there is often a need to postulate a framework which fits the available facts before a task can be put into a workable form. A definite framework, even if it is only a hypothesis, is preferable to a vaguely defined situation because it gives direction to the thinking, identifies indefinite aspects, and highlights where further enquiries should be directed. However it is important to recognise when hypotheses are made and repeatedly review them as more data becomes available. Again, solutions or decisions consequent upon a given hypothesis should be tested for sensitivity to feasible deviations from it.

### 4.5 Resources

Available resources will place definite limitations on the form that investigations, design and construction can take. There are five broad categories:

People

Skills (incl. particular expertise, methods of analysis)

Equipment

Time

Money

For a given task, one or more resources will exercise critical control over what can be achieved.

Time and money will often control but other resources, through their inherent limitations, can lead to important restrictions on the value of what is achieved. Thus it is essential to critically examine the relevance and suitability of investigation procedures and data, tests and methods of analysis, and the structural form of foundations in any given situation. There is often more than one approach to a problem and the deciding criteria will involve balancing value for money against risks and consequences.

### 4.6 Simplification or Modelling

Natural materials frequently give rather complex conditions. On the other hand, most methods of analysis are based on assumptions which apply to relatively simple, idealised soil profiles. Therefore even if a real-life subsurface situation can be precisely quantified, a simplified 'model' of the actual conditions will be a necessary pre-requisite before analyses can proceed. The reliability of this model and the corresponding analyses will be governed primarily by how representative it is. Therefore the configuration of materials (complete with all the inherent assumptions) must be determined before deciding whether a method of analysis is appropriate or applicable, rather than the reverse as is probably the more common approach. Otherwise there is the very real danger of assuming that a method of analysis is applicable whereas in reality it would totally misrepresent the actual conditions.

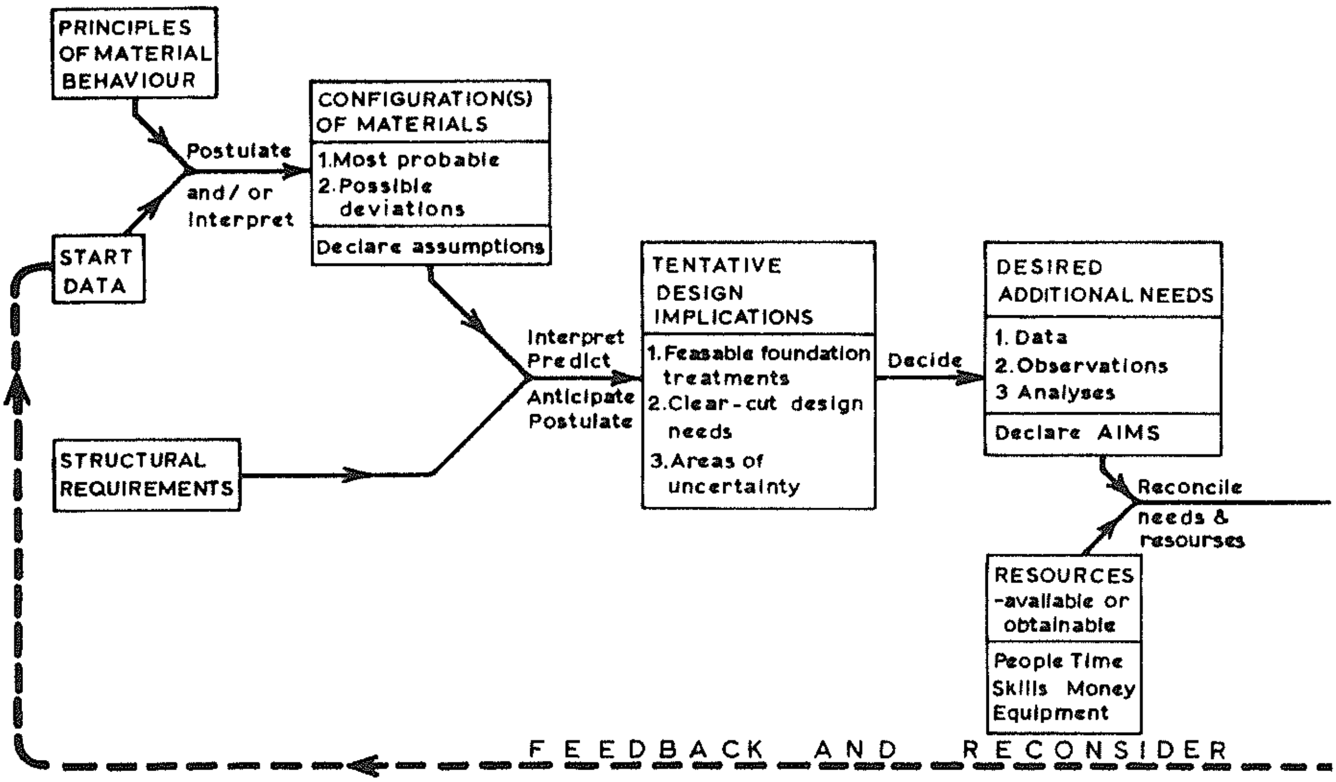
### 4.7 Reliability, Risks and Consequences

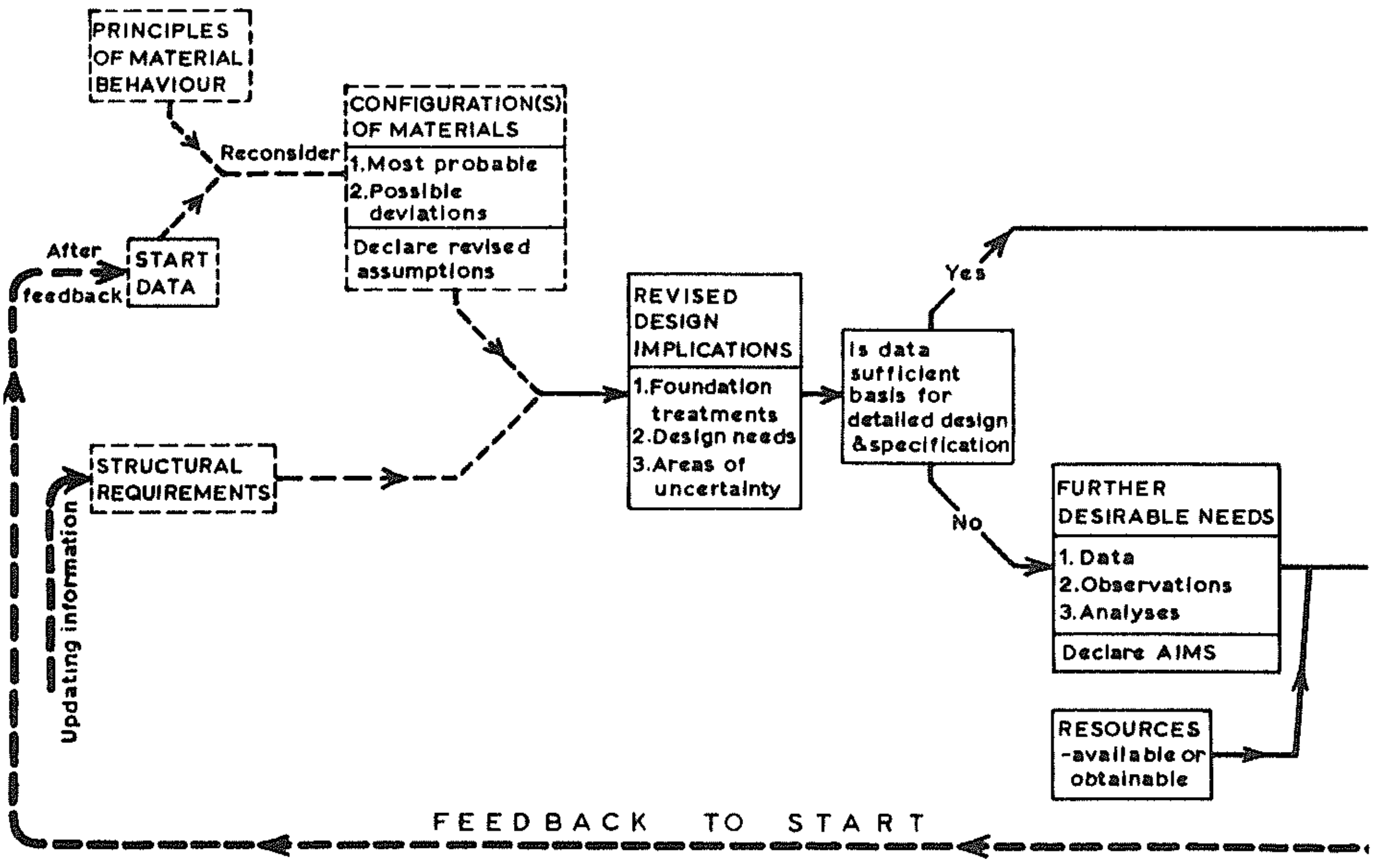
Any foundation engineering situation will contain an element of the unknown, regardless of how thoroughly it is studied. The particular nature of the sub-surface materials will impose limits on the extent to which they can be quantified and the procedures used will give data of varying reliability. Therefore it is virtually impossible to state categorically that a predicted performance will be precisely some single numerical value; in fact one should be very suspicious if such a prediction is encountered. At best, predicted performance should be given as a range, e.g. for settlement, with the values reflecting the reliability of the prediction. Obviously this must take into account the reliability of the basic data and the extent to which the task has been quantified. Again when dealing with a specific real-life task, reliability has wider implications than simply the technical accuracy of a result. A given foundation will be associated with a specific structure which will serve a definite purpose. Therefore, in the decision making process, reliability cannot be treated in isolation but must be related to the risks and consequences to life and property.

### 4.8 Observation and Monitoring

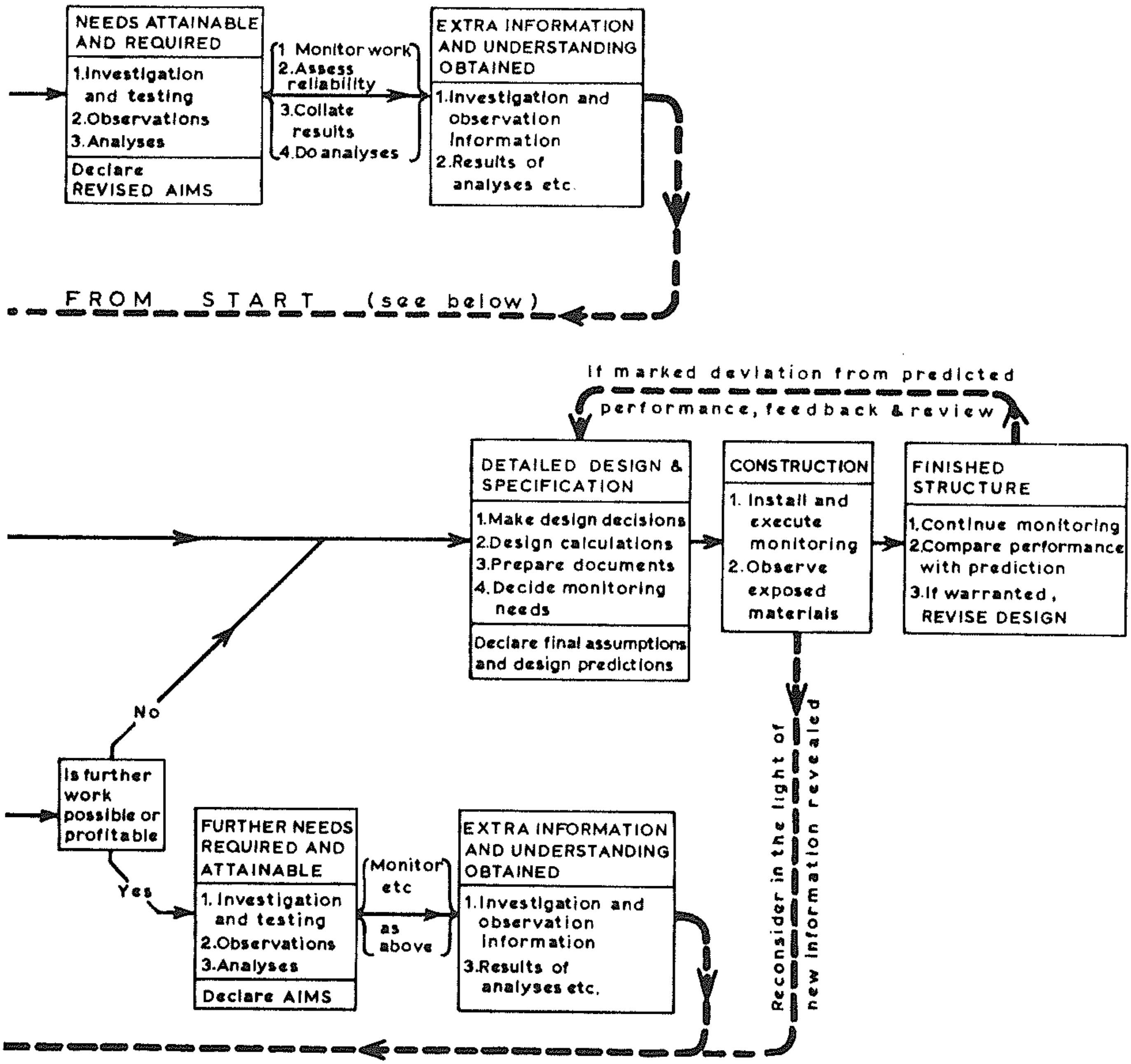
The designer's responsibility and concern for a foundation does not end with the production of the plans and specifications but continues throughout the construction and subsequent performance phases.

Sometimes, because of time or other limitations, it may not be possible to satisfactorily quantify conditions or predict performance before construction starts. Also there will always be limitations on the reliability of performance predictions. Therefore observations


PRINCIPLES OF MATERIAL BEHAVIOUR
START DATA
Postulate
and / or
interpret
CONFIGURATION(S) OF MATERIALS
1.Most probable
2.Possible deviations
Declare assumptions
STRUCTURAL REQUIREMENTS
Interpret
Predict
Anticipate
Postulate
TENTATIVE DESIGN IMPLICATIONS
1. Feasable foundation treatments
2.Clear-cut design needs
3.Areas of uncertainty
Decide
DESIRED ADDITIONAL NEEDS
1. Data
2.Observations
3 Analyses
Declare AIMS
Reconcile
needs &
resourses
RESOURCES -available or obtainable
People Time
Skills Money
Equipment
FEEDBACK AND RECONSIDER


PRINCIPLES OF MATERIAL BEHAVIOUR
Reconsider
CONFIGURATION(S) OF MATERIALS
1.Most probable
2.Possible deviations
Declare revised assumptions
After feedback
START DATA
STRUCTURAL REQUIREMENTS
Updating information
REVISED DESIGN IMPLICATIONS
1.Foundation treatments
2.Design needs
3.Areas of uncertainty
Is data sufficient basis for detailed design &specification
Yes
No
FURTHER DESIRABLE NEEDS
1. Data
2.Observations
3.Analyses
Declare AIMS
RESOURCES -available or obtainable
FEEDBACK TO START

# FIG. 1. LOGIC CHART FOR FOUNDATION ENGINEERING



during construction as the foundation site is opened up, together with monitoring of the performance of the completed structure, is an integral part of the total foundation engineering process. The purpose of any observation and monitoring should be one or more of the following:

(a) To confirm that the sub-surface materials and conditions are as previously interpreted, postulated, predicted, etc., or otherwise.

(b) To confirm the validity of a given design and predicted performance and give confidence in it.

(c) To give early warning of a need to modify or adjust the foundation or structure.

With the last it is important to recognise that alternative action(s) would probably only be feasible and practical, if anticipated before the design was committed. Hence a designer must consider the treatments needed to deal with the likely deviations from the predicted probable behaviour.

## 5. <u>LOGIC SEQUENCE</u>

With the above points in mind, the logic sequence for rational application of geomechanics to foundation engineering is as set out in Fig. 1. It is largely self-explanatory and gives a "thinking and decision making" framework which can be applied to any task. In sequence, the stages through the logic are as follows:

(a) Combine the available understanding of the 'principles of material behaviour' with existing 'start data' and postulate or interpret one or more configurations of materials and conditions which will fit the available <u>facts</u>. Inevitably the facts will present an incomplete picture so to get a plausible configuration will involve making assumptions; these must be clearly identified and recorded. Again, because of the incompleteness of data, one should postulate, interpret or even speculate on the possible deviations from the selected configuration, i.e. there is a need to consider the most probable and the most unfavourable configurations.

(b) Examine the structural aspects of the task and identify the key requirements, limitations, boundary conditions, etc. i.e. establish the broad form of the structural requirements. For example, establish if differential settlement will be critical to the proposed structural form, or alternatively, determine if adjacent properties will impose limits on the forms of foundation which can be considered.

(c) Consider jointly the structural requirements and the configuration of materials to determine:

  (i) feasible foundation treatments,

  (ii) the aspects where evaluation and design needs are clear-cut, and;

  (iii) areas where the problems cannot be defined. This step should highlight where assumptions are of most significance.

(d) Decide what extra data, observations, analyses, etc. would be needed to satisfactorily quantify the task. This will automatically invoke consideration of data reliability, situation complexity and the risks and consequences involved. It is important to declare the aims of the extra needs, firstly as a self-discipline on the thinking involved, but also so that the primary purpose of any extra work is clearly recognised and less likely to be forgotten.

(e) In parallel with the desirable needs, consider the resources which are available at the given time. These will set very definite limits on what needs can be satisfied.

(f) Next the designer must reconcile the desirable needs with the available resources and decide what additional data etc. can be pursued. Compromise will be inevitable and as well as the availability of resources, one must also assess the value of those that are available. The value of geomechanics data and analyses is very dependent on the skills of operators and technicians, techniques used, simplifications and approximations and a designer should assess all these factors before deciding what to do. In adapting to the available resources, some modification of the previous aims will be inevitable but as stated before, these revised aims should be declared. Note also that if analyses form part of the extra work, a simplification or 'modelling' step may be required.

(g) Next proceed to do the required additional work. Throughout there is a need to monitor the work, assess its value and the reliability of the data collected. The resulting information won't necessarily correspond to the kind of result which was expected but if the work is productive, the extra data should either confirm or add to the facts of the situation, i.e. it gives additional 'start data'. Hence the logic sequence involves feedback as shown in Fig. 1.

(h) Once the new facts are obtained, review the entire thinking sequence as indicated along the lower part of Fig. 1. Review previous assumptions and hypotheses and reconsider the configuration of materials; it will be either enlarged upon or modified. The improved understanding of site and sub-surface conditions, together with a corresponding better idea of the significance of various structural or construction limitations, will automatically lead to a reappraisal of the foundation treatments, areas of uncertainty and detailed design needs.

(i) Following the review, alternative situations* arise as follows:

EITHER

(i) Quantification of the task is regarded as sufficient to allow the design decisions to be made with an acceptably low risk. Therefore proceed with detailed design, specification and setting of monitoring needs.

OR

(ii) Available data and general quantification is still regarded as insufficient. Therefore decide what extra data, etc. is still desired but consider also whether useful information can be obtained. For example, will the devices or procedures available give worthwhile new information? Is there time and money available to do the work desired?

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* These alternatives should have been considered in the first cycle of the logic sequence but were omitted for clarity and ease of presentation.

If worthwhile additional work is possible then proceed through the decision and action logic as before If not, then the designer must proceed within the limitations of the available data and, if necessary, modify the design approach accordingly. For example he may adopt a more conservative attitude on some aspects or set very specific decision making stages as part of the construction monitoring.

(j) Monitor the construction, particularly in the early stages when a site is opened up, to determine if the designer's expectations (e.g deductions regarding sub-surface conditions) and the intent of the design will be realised The likelihood of getting feedback and its value will depend largely on how well the designer's thinking has been declared and conveyed to site personnel. Hence the reason why the declaration of aims, assumptions, impressions, conclusions, etc. is so important. If necessary, and/or possible, modify the design but the extent achievable will depend on how well the deviations have been anticipated

(k) The ultimate measure of the success of a foundation is the performance of the completed structure, especially as it meets the client's requirements Therefore to complete the logic sequence, measure the performance of the completed structure and compare with the design predictions. If performance does not measure up to that expected or required, review the situation and decide further action

## 6. CONCLUSION

Traditionally foundation engineering is regarded largely as an art in which competence is attained only after long and varied experience While experience may be a useful source of guidance, mere exposure to a variety of situations does not automatically guarantee success In fact unqualified references to experience should be regarded as a warning of possible undisciplined thinking and ill-founded judgements and opinions.

Foundation engineering is essentially a decision making process aimed at fulfilling a specific client's brief Sub-surface conditions can be complex and each site is usually different. Available geomechanics tools and procedures provide valuable assistance but here again serious limitations can arise Consequently, the application of geomechanics to foundation engineering demands clear, logical thinking exercised within a planned, systematic framework of deduction and decision making. Only then will the true value of experience be realised.

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# EVALUATION AND INTERPRETATION OF SITE INFORMATION

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## 1. INTRODUCTION

Site investigation for foundation engineering purposes is a recognised part of civil engineering construction works. The aim is to acquire enough information to allow a reasonably complete geotechnical assessment of the site to be made. Such site information is required for the assessment of the general suitability of the site for the planning, design and construction of new work, or for the examination of defects in existing works caused by site conditions. No two sites are quite the same so that each must be considered individually in relation to planned works. Well executed site investigation will reduce the waste of money and resources caused by unnecessarily conservative design and should lead to adequate, safe, and economic design.

The approach to site investigation should be that of any scientific investigation - a clear aim of the information required to be obtained, selection of methods of obtaining such information with greatest economy of effort, a knowledge of the ultimate use of the results of the experiments and a prior assessment of related work already carried out by others. However, unlike many scientific research projects, site investigations are always closely controlled by how much money and time can be allocated to their completion. The aim should be to establish the pattern of site conditions in sufficient detail so that all factors relevant to the project can be assessed with adequate confidence. Absolute certainty is rarely possible in a natural situation and the required degree of confidence is a matter of judgment. The limitations set by available time are too often overlooked by client and investigator alike. Inevitably in each investigation there comes a point in time after which the designer or the contractor can no longer make effective use of new information. It becomes valuable only as post mortem material or against future investigations of a similar kind.

## 2. GENERAL PRINCIPLES OF SITE INVESTIGATION

Before assessing the specific factors relevant to the project, a site investigation should be aimed at understanding the site together with the natural or imposed processes acting upon it. The site should always be considered in the context of the surrounding landscape, its topography, geology, vegetation, drainage and past use. The study should commence with a review of past records, maps and available literature followed by an initial walk-over survey of the site. At this stage a preliminary hypothesis of the subsurface conditions can usually be set up against which all future evidence can be considered. Bjerrum (1960), describing Terzaghi's method of working expresses the now generally accepted principles of site investigation very clearly.

"Terzaghi's work on a difficult dam project always starts with a study of the topography and geology of the entire area surrounding the site. During this part of the work he uses the library intensively. If he cannot obtain all the information which he requires in order to obtain a fairly clear picture of the geology of the area, he visits the area himself to secure the missing information. The intimate knowledge of the geology of the whole area is as necessary for his work as the subsoil exploration which follows.

On the basis of observations made during his visits to the job, the results of a few preliminary borings, and his study of the general geology of the region, he formulates a working hypothesis regarding subsoil conditions at the site, serving as a guide in later exploratory work. He then sets up a program for additional borings and for field observations. Some of the borings are intended to provide detailed information required for design purposes, but some might be included in the program only in order to test the working hypothesis. Borings may thus be made far from the actual site at places where, at a first glance, it seems unjustified to carry out investigations; but more than once the results of such borings have revealed important and unexpected features of the situation.

In addition to the borings, bulldozer cuts, shafts, and observation of groundwater levels play an important role in this phase of the investigation. The subsoil exploration is combined with an attempt to get as much information as possible concerning the natural groundwater conditions in the area. Therefore the measurements of gradients and discharges are started at an early stage of the field investigations. It is surprising to see how many conclusions concerning the permeability of the subsoil of the dam site can be drawn from such field data.

It is characteristic that he never prepares a detailed program for his subsoil explorations in advance. On most jobs, he starts with only a few borings. Every bit of information furnished by these borings is carefully studied and evaluated as it is received, and the program for further exploration is developed step by step as the geological information accumulates. Consequently, every one of the exploratory operations furnishes significant results. No efforts are wasted.

In his study, he collects all the information and prepares simple sketches and small-scale diagrams which show only the essential features of the findings. He continuously re-examines and, if necessary, revises his opinions of the geology of the site; and he is always prepared to change his mind even if he has previously expressed his opinion in written reports.

As a consequence of the careful study of the results of the exploratory operations, supplemented by his observations in the field during his frequent visits to the job, he acquires a profound knowledge of the site. No essential detail escapes his attention and he knows by heart the configuration of the terrain, the results of the borings, and all the data concerning the groundwater conditions."

While such a profound knowledge of the site is invaluable, a site investigation is not an end in itself. It can be an expensive process and the details must always be relevant. Generally a site is being investigated for a particular structure and the investigation must suit that structure as well as the site. For heavy and extensive structures there is little value in detailed measurements on soft surficial materials, except insofar as such materials may affect construction procedures, and likewise, expensive investigation at depth below a thick dense stratum has no place for small or light construction.

# 3. PROCEDURES AND LIMITATIONS

## 3.1 Collection of Existing Information

All available information on the site and surrounding area should be studied before undertaking field work. Air photographs of essentially the whole of New Zealand are available and provide a rapid and valuable means of obtaining basic information. Landforms, drainage patterns and other features can be interpreted from the photographs which can then aid identification of critical features on site. The N.Z.I.E. Borehole Index could become a most valuable resource. Published topographical, geological and soil maps provide some basic data but scales are rarely sufficiently detailed for other than extensive sites. Information on past use of a site can help to prevent surprises and ease planning the investigation. It is suspected that generally in New Zealand insufficient use is made of much existing information.

## 3.2 Site Inspection

After the desk study is completed a thorough inspection of the site should be made with careful and systematic recording of natural and man made features. Some knowledge of field geology is imperative and in other than simple sites there is considerable advantage in bringing an engineering geologist into the investigating team at this stage as outlined by Oborn (1969). The general environs of the site should also be studied in detail in order to understand the regional context. Since this inspection will form the initial basis of the planned investigation clear written records preferably with photographs should be prepared to guard against overlooking important details later. As at all stages of the site investigation, opinion, speculations, or conclusions should be clearly expressed as such and not confused with recorded facts.

## 3.3 Geophysical Survey

Where ground conditions are favourable, geophysical methods can provide a means of covering large areas quickly in broad detail though they will require correlation borings for correct interpretation. They can provide information on the structure and physical properties of the underlying geological features but only where sufficient contrast in particular physical characteristics exists. The information required and the strata physical properties will determine the geophysical method used. Ingham (1969) suggests that seismic refraction is probably the most useful though direct current resistivity and also a study of surface waves have important roles. It should always be remembered that the interpretation of geophysical data involves assumptions of certain idealised subsurface physical conditions and that there will be sites where the natural complexity can make the techniques ineffective.

## 3.4 Direct Subsurface Exploration and Sampling

The direct subsurface exploration of a site is ideally a two stage operation before that allowed by the main construction. Initially there is an exploratory phase aimed at determining the nature and distribution of the materials present, and whether a problem exists. The prime interest is in soil types, descriptions and perhaps classification tests. Sample disturbance is not of great importance provided a minimum quality complete sample can be examined. Preliminary assessment of ground water conditions are undertaken at this stage.

If the initial stage indicates that a problem exists, a more careful and detailed investigation must be undertaken with the obtaining of undisturbed samples of sufficient quality to allow measurement of relevant engineering properties. Laboratory studies may be augmented by suitable field tests to assess strata strength, deformation characteristics, and permeability as required.

Access to subsurface materials is generally by means of drilled boreholes using hand augers or well and rock drilling machine techniques, but open excavation methods should not be neglected. Hand excavation is expensive but mechanical excavation with back-hoe, trencher or bulldozer can often provide a cheap and rapid means of shallow access, especially on sites overlain by gravels or where an assessment of lateral variation is important. Available drilling and sampling methods from boreholes were outlined at the Christchurch Symposium, (Cornwell, Northey and Thomas, Northey, 1969) and need not be repeated here.

"Because of the great variation in physical properties of soils, it is unlikely that a single sampling method or type of sampler, which will produce satisfactory samples under all conditions, will ever be developed. On the contrary best results will be obtained at least cost when several types of samplers are on hand and are used in accordance with the character of the soil and the purpose of exploration, and when operators constantly watch for minor changes in soil conditions and make corresponding adjustments of the equipment and sampling procedure". This statement by Hvorslev (1949) is just as true today.

There is frequently a need for continuous sampling of the soil profile and for care in the interpretation of the sampling record. The positions <u>not</u> sampled should be emphasised quite as much as the actual samples taken. The weak layers, the fine sand partings, the defects in weathered rock are far more likely to occur in these gaps in the sampling record and must not be overlooked since they commonly control the overall behaviour of the material. Similarly weaker material found near the top and bottom of undisturbed samples should not be completely disregarded as "disturbed during sampling" but interpreted in terms of the overall pattern of materials found and suspected. Nor should it be forgotten that the material seen is but a very small part of that inferred. Even continuous 75 mm cores from bores at 30 m spacing reveals only 5 parts per million of the site conditions.

The field log must be factual, accurate, clear and complete. It is a written record of all the information observed and all the data obtained concerning materials and conditions encountered in individual test bores. Cornwell (1969) and Bullen (1969) describe the separate needs of the driller's daily record and the supervisor's systematic description of materials and conditions but I feel that ultimately we should aim at the greater recorded detail suggested by the Geological Society (of London) Engineering Group Working Party (1970). The field log provides the fundamental facts on which so many future conclusions are based that gaps in the record are inexcusable though we are all at fault.

While labelling, handling and transportation of samples to a laboratory for testing are not directly sources of field data it should be remembered that lack of care at this stage is just as efficient in producing erroneous information as lack of care in the taking of samples.

## 3.5 Field Testing

There is increasing attention being given to measurement of engineering properties of soils in situ. We are all aware of the limitations of tests on small laboratory samples and the concept of field testing of soils in bulk is attractive. There should be no controversy between field and laboratory testing, each is complementary to the other in providing subsurface information. The more expensive field tests should not be attempted until a reasonable understanding of the site has been obtained, but some field tests may be carried out in boreholes and consideration should be given to including these in the same bores as used for sampling, especially when strata are reached in which the obtaining of other than simple samples is extremely difficult.

It is natural to assume that some measure of the strength of the ground can be inferred from the resistance offered to penetration and many simple and complex penetrometers and theories have been used to this end. (Bondarik 1967, Sanglerat 1965). Consideration of in situ testing is largely concerned with such penetrometers varying in size from ¼ sq. cm hand penetrometers through the 10 sq. cm of the Dutch Cone to 5000 sq. cm of plate loading tests. A few of these will be discussed together with other field testing at exploratory and design stage.

### 3.5.1 Standard penetration test

This is a dynamic penetration test made at intervals in a borehole which has been advanced by other means. It is an old and very widely used test. The broad details of the test with its heavy walled sampler of diameter 50 mm outside 35 mm inside driven by blows of a 63.5 kg (140 lb) weight free falling 76 cm (30 in.) are well known but many papers have been written pointing up the lack of specification of many details which influence test results. Thus it has become a badly misused and abused test with some strong advocates and many dissenters. At best it is crude and at worst quite misleading. With a consistent set of equipment and procedure it can provide comparative values of penetration resistance on a given site to aid lateral correlation of strata, while providing a sample suitable for broad classification purposes. However, if the N values are to be interpreted later in terms of engineering properties of soils or used directly in design calculations, a minimum requirement is strict compliance with a published standard procedure. Procedures are included in ASTM Standards and also BS 1377 but I doubt if sufficient details have yet been standardised to reduce the controversy (Ireland, Moretto and Vargas 1970). In dense or coarse grained materials the sampler may be replaced by a solid cone of similar dimensions which may be used not only in a borehole but also direct from the surface without predrilling. While valuable for interpreting variability and aiding lateral correlation between boreholes such penetration resistance N values should be interpreted with even more caution than the standard penetration test.

### 3.5.2 Dutch Cone Test

This is probably the most widely used static penetrometer deep sounding device. The thrust required to push down a cone of apex angle $60^{o}$ and base area 10 sq. cm at 2 cm/sec. is recorded mechanically or electrically. Either continuous penetration or intermittent advance of cone and casing is employed. The use of the Begemann friction sleeve behind the cone proper allows a measure of the skin friction to be separately recorded, normally at 20 cm intervals. The device is made in a range of capacities from about 1 - 10 t depending on desired depth of penetration and soils to be penetrated. The total available reaction is also controlled by the mass of the equipment or the efficiency of its earth anchors and the maximum penetration depth may be limited by either high point resistance as in gravels or high skin friction as in firm to stiff plastic clays. It is a relatively cheap means of assessing depth to known resistant or weak strata, to check variability of a site or to fill in between boreholes. As such it is an extremely valuable test method which can save considerable time and expense in site investigation.

Empirical methods have been developed to assess soil texture and engineering properties from the recorded values and the relationship between point resistance and friction sleeve resistance, but such relationships should be interpreted with caution even on a single site. Schmertmann (1970a) has submitted a suggested standard procedure to ASTM.

### 3.5.3 Groundwater Studies

The securing of adequate information on groundwater elevations, including the depth of permanent, perched or artesian water tables is an essential part of any soil exploration. Rowe (1968) describes some spectacular mishaps following failure to recognise groundwater conditions during site investigation. While it should be normal practice to record groundwater levels found in drillholes especially after overnight standing, such holes are inefficient piezometers except in high permeability ground. If sufficient time has not been left for complete equilibrium to be reached the report of groundwater conditions must be suitably qualified. Installation of piezometers at various depths as well as pressure and pumping tests to determine permeability of particular strata should all be considered depending on geologic conditions and the proposed construction.

### 3.5.4 Vane Test

In its simplest form the field vane consists of a four bladed cruciform section vane which is thrust into the ground and then rotated to fail the soil on the circumscribing cylinder. From the torque required and the vane dimensions a measure of the soil shear strength can be obtained. The test may be carried out at the bottom of a borehole or by pushing the vane directly from the ground surface to the desired depth. It has been widely used to measure the undrained shear strength of clays and many comparisons have been made with strengths measured on samples or calculated from slope and foundation failures. Although the average strengths measured are frequently comparable, the scatter of results has been sufficient to generate some controversy concerning the test.

The insertion of a vane into the soil must cause some disturbance, a degree of remoulding and certainly stress changes. In an attempt to minimise disturbance it should be normal practice to use a clean, smooth, well sharpened, thin bladed vane of area ratio less than 12% (BS 1377) inserted at least three borehole diameters below the bottom of a well prepared borehole or 50 cm beyond its own protective casing. Stones or shells pushed ahead or caught by rotation can lead to completely unreliable results as also can a damaged vane. Adhesion of particularly sticky clays to the vane faces gives an effective increase in area ratio with consequent disturbance. Although it has been shown that immediate contact pressures are up to 10 times the undrained shear strength

(Kallstenius 1963) little drainage should occur in saturated, normally-consolidated, non-fissured clays provided the test is not unduly delayed, so that undrained strength of such materials should not be affected. With other soils an unknown drainage condition can exist though Blight (1968) has suggested that drained strength of some silts can be determined in suitably slow tests. Despite many attempts to extend the theoretical analysis of the vane test to other soils it is now fairly generally accepted that vane tests should only be used in "contractant" soils (Osterman 1964).

The strength measured can be influenced by rate of strain. Although there has been a lot of discussion on this variable, little work has been published since Cadling and Odenstad (1950) where 0.1 - 0.2 degrees per second was recommended. This is sufficiently slow that the vane head must have substantial gearing and simple torque wrenches on the vane rod do not provide an adequate control or measure of torque. Friction in the equipment must be reduced to insignificant levels since it all appears in the torque measured. Provision should be made in the method of operation (Newland and Allely 1956) or design (Gibbs, et al 1960) to measure the friction component in each test result. When calculating strength from torque it is common practice to assume a failure surface coincidental with the circumscribing cylinder of the vane blades with uniform mobilisation of shear strength on upper and lower faces as well as the cylindrical surface. This neglects progressive failure and also must assume that shear strength is independent of failure plane. Anisotropy could be investigated with vanes of different sizes and shapes at the same depth but would need considerable research before such results could be used in design.

Flaate (1966) feels that the vane test should be calibrated to the particular problem. He points out the need to conduct the test in the same way every time - careful installation procedures, the same delay between insertion and measurement preferably not more than say 5 minutes, the same and constant rate of strain, and the same vane dimensions. However, there is growing evidence that the vane test can be used with confidence for measuring the undrained shear strength of soft to firm saturated non-fissured clays provided parallel samples are taken from the deposit to identify the materials tested. In this regard the square sampling tube recommended by Wilson (1970) is even more valuable. It allows a determination of in situ shear strength during rotation to shear off, as well as both a field identification of material sampled and an undisturbed sample of laboratory use. This is a major innovation in sub-surface exploration.

### 3.5.5 Pressuremeter

The pressuremeter is a relatively recent addition to the list of equipment available for testing soil properties. First suggested by Ménard (1957) it is now in fairly common use in many countries. Basically it is a kind of load test carried out at a specific depth within a borehole by means of an inflatable, cylindrical probe. The probe contains three cells, a central measuring cell and guard cells top and bottom to maintain a reasonably uniform distribution of pressure. A record of applied pressure against radial deformation may be interpreted in terms of various strength and compressibility parameters. (Gibson and Anderson (1961), Ladanyi (1963) ).

The applied pressure is increased at intervals and deformation recorded, though the rate of load application has not yet been standardised. Increments from 2 - 10% of failure load have been used with rest periods from 2 minutes to several hours causing substantial differences to estimated parameters.

The normal cell is about 1 metre length so that readings can not be taken at closer intervals than about 1½ metres. This might seem a disadvantage compared with equipment providing an essentially continuous record but it has been claimed to provide information on the macroscopic behaviour of soils (Komornik, Wiseman and Frydman 1970). By its nature it takes account of layer discontinuities such as weak lenses or fissuring which might not be noticed in testing of cores, if they can be extracted. Clearly for sensible interpretation of results the probe and borehole should have matching dimensions and the uncased walls of the borehole should be reasonably true. Disturbance during drilling operations is just as much a problem as for undisturbed sampling.

### 3.5.6 Other Field Tests

There are a large number of other field tests which have been employed to assess variability of a site or to determine specific engineering design parameters, many restricted by cost to major undertakings only. The reaction requirements of ½ sq. metre plate loading tests or pile loading tests put them outside the scope of this paper and for those who advocate the use of various pile driving formulae I would refer them to Whitaker (1970).

Plate loading tests of 1/10 sq. metre are carried out in shafts at proposed foundation depths as also CBR tests at a range of depths in boreholes. Both are essentially small strain measurements and are thus more likely to be relevant to foundation design than conventional strength measurements. However, the volume of soil tested is still quite small and the same criticisms can be made concerning soil disturbance as for conventional sampling procedures. Schmertmann (1970b) has suggested that screw plate loading tests are rather more satisfactory. Using a rotary drilling rig or the Dutch cone "hydraulic spanner" a single turn flat pitch earth auger can be screwed to a range of depths and loaded as for a conventional plate loading test. By controlling the rate of turning and rate of penetration, disturbance can be minimised.

An interesting strength measurement carried out within an uncased borehole is that suggested by Handy and Fox (1967). Two curved plates are expanded inside the hole to give a pressure normal to its sides. A shearing stress is applied to move the device axially along the hole, essentially a direct shear test. The obvious criticism, as for the Ménard Pressuremeter, is the need for true undisturbed walls to the borehole.

Monitoring, or post construction field tests, while an important part of a complete investigation, are not carried out in New Zealand as often as they should be. Failures or large deformation case histories, while spectacular are perhaps not as valuable to foundation design as successful case histories recording small deformations. Bozozuk (1971) discusses the role of such field instrumentation, especially settlement and heave gauges, installation of bench marks, and piezometers as well as earth pressure and horizontal movement gauges.

## 4. CONCLUSION

This has been a very swift sweep through a philosophy of site investigation with some reminders of the methods which might be used and some of their limitations. Probably the greatest lack in site investigation is a set of standard, completely reliable methods of assessing the ground against which each method used can be correlated and proved. At present the literature is still full of correlations of uncertainties - standard penetration test against Dutch cone against Ménard pressuremeter against a range of other similar measurements. Progress cannot be achieved by such means. Reliability is not improved by statistical correlations between crude measurements. We need to continue the practice of site investigation right through from exploration, design, construction, to performance of structure with many relevant careful observations at each stage before substantial progress will be made in the applications of geomechanics to foundation engineering.

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## OPENING

Mr W.J.H. Duckworth, the convenor of the symposium sub-committee of the N.Z.I.E. in Wanganui chaired a short formal opening. Delegates were welcomed to Wanganui by the Mayor, Mr R.P. Andrews and the symposium was opened by Mr K. Christie, the president of the N.Z.I.E.

## PRESENTATION and DISCUSSION. SESSION 1.

Chairman: J.P. Blakeley
(Beca, Carter Hollings and Ferner)

Mr J.H.H. Galloway, president of the N.Z. Geomechanics Society presented his keynote address "Geomechanics in Foundation Engineering". In the absence overseas of Mr Bullen the second paper was presented by Mr N. Major (M.O.W. Wellington). In his presentation Mr Major spoke about some possibilities of applying probability concepts in the interpretation of site information data. He said that the true nature and disposition of site materials was known only after exposure, and he put the proposition that nearly all site investigation, whether records search, opinion seeking, surface inspection or hole digging, was a collection of information of variable but estimable reliability. From this information the true nature of the materials was inferred. All such inferences had associated likelihoods or probabilities of being right, - dependent both on the quality of the data and the quality of the logic leading to the inferences. He suggested that predictions from site investigations should be thought of as alternative possible configurations of materials, each with its own "betting odds" of being true.
Dr Northey, in presenting his paper took up this probability idea and spoke of the need to include in any such computations not only the data and the logic but some personal characteristics of the individuals responsible for the measurement and collection of the data.

Mr T. Belshaw (M.O.W. Napier) opened the discussion by commenting that Mr Bullen's "clear logical thinking" and "planned systematic approach" appeared to lack flexibility.
Mr Major (replying on behalf of Mr Bullen) said that he thought Mr Bullen was advocating that the engineer should follow a pattern but at the same time think carefully about each step. This was so much better than taking steps and not realising that they had been taken. One purpose of Mr Bullen's paper he said, was to worry people and to make them think.

Mr G.L. Evans (Canterbury University) commenting on Mr Bullen's phrase that "assumptions are inevitable" stressed that the validity of these assumptions must be questioned at every stage because it was they which became in effect the basis of a design.

Mr N.S. Luxford (Tonkin and Taylor, Auckland) said that a logic sequence must be kept within a time scale. He referred to a recent job in which his firm were requested to do a foundation investigation for a 3 storey building with partially excavated basement. The time between being given authority to proceed with the investigation and the submission of the draft report to the designer was approximately six weeks. Two or three weeks later all the footings, the basement walls, and the first floor had been constructed.
Mr Major replied that the imposed six week deadline merely dictated that any foundation type considered must not need sophisticated testing or

analysis - of a sort that would take more than (say) three weeks. He suggested that the whole of the logic sequence was still applicable and was a most useful check. He emphasised that the logic sequence was a thinking sequence and that "as fast as you can assemble your thoughts you can do it". He said that he would reject any suggestion that time scale could reduce the need for clear logical thinking.

Mr B.C. Hadfield (Gilberd Hadfield Pile Co., Auckland) said that monitoring of the foundation construction by the designer sometimes lead to an attempt to justify and at times to enforce the design or method, in spite of the fact that sub-stratum conditions were substantially different from those envisaged. He suggested that an independent person should monitor, report, and if suitably qualified advise on changes of method or even of design.
Mr Major agreed but with the qualification that the original designer should be included in the new design team, - because he may be aware of other problems and subtle details which a newcomer could not be expected to catch up with.

Mr Hadfield commented that, although Mr Bullen had suggested that undue reliance on experience was unwise the previous, and where possible recent, experience of an engineer or contractor in similar materials could be invaluable. The advice which could be got relevant to methods, costs and difficulties could be of assistance in the economics of the design, he said.
Mr Major replied that he thought that information from related sites should be treated as such and not as prime data. Information from adjacent boreholes and excavations could be of good use, particularly if the man who conducted the investigations could be spoken to; fact founded opinion was valuable data.

Prof. P.W. Taylor (University of Auckland) agreed with the opinion expressed by Mr Major in presenting Mr Bullen's paper, namely that concepts of probability should be included in site investigation philosophy. He thought that such concepts would be used increasingly. He felt that insufficient emphasis had been given in the paper to the fact that a site investigation programme had to be related to the size and type of structure envisaged. A very detailed investigation which was economically justifiable for a major structure might be quite unjustifiable for a minor, inexpensive, structure on the same site. He also emphasised that problems should be approached with an open mind. The question to be answered should be "What is the most suitable foundation system for this job?" and not "How long should we make the piles?"
Mr G.A. Pickens (Tonkin and Taylor, Auckland) underlined the need to approach with an open mind not only the foundation design but the decision as to the type of site investigation which was appropriate. He observed that in the U.K. and Australia site investigations were let out to contract with consequent wastage and reduction in quality.
Mr E.R. Chave (M.O.W., Dunedin) said that in California minimum testing requirements for civil works were set by legislation.

Mr D.K. Taylor (Tonkin and Taylor, Auckland) commenting on Dr Northey's paper said that what he looked for from geophysics and geology was a clarification of the likely continuity and form of the soil between bores, pits and exposures. He felt that these disciplines did not have a primary responsibility for defining the engineering properties of the ground at all, and that geophysics was appropriate only for large scale investigations, much larger than would be appropriate for a single building, or even for a complex such as a university.
Dr Northey replied that he felt that the involvement of the engineering geologist should be broader than this; that he should be brought in before

the first borehole was sunk, and that he was often better able to define strata from borelog information than the engineer was. Dr Northey agreed that geophysical methods were generally appropriate only on very extensive sites.
Mr Evans said that a technique had been developed at Canterbury University, using reflection and refraction of waves, which was useful on small sites. Shear and elastic moduli (G and E) could be measured. Hard and soft layers had been detected in what had appeared to be a uniform layer of loess.

Mr D.K. Taylor said that the physical property whose changes were detected by a geophysical method was not always a physical property of engineering interest.
Dr Northey agreed but added that in many cases it could be hoped that boundaries detected by the geophysical method might be coincident with, or at least related to, boundaries between soil bodies with significantly different engineering properties.

Mr Belshaw said that Terrascout and other small geophysical machines were in his experience useless.
Dr Northey replied that he found few advocates for such instruments and added that Soil Bureau, D.S.I.R., had had the opportunity to buy one but after trying it out decided not to do so.

Mr Belshaw said that he had not found the Cornwell (1969) or Bullen (1969) drillers daily logs satisfactory. He also asked Dr Northey to define a "contractant" soil.
Dr Northey referred Mr Belshaw to the Geological Society Engineering Group Working Party plans for drillers logs (see his references). He defined a contractant soil as one which tended to diminish in volume when subjected to shear.

Mr Belshaw said that he felt that the Dutch Cone test had been praised too faintly and referred to case histories described in the I.C.E. Conference "The Behaviour of Piles" in which Dutch Cone tests had given vastly better predictions of ultimate bearing load per pile than had been obtained by any one of three firms using five and more boreholes each.
Dr Northey replied that there were many piling formulae which could be used - none of them totally reliable. He agreed that when ultimate pile load was what was required and where the geology of the site allowed it to be used- then the Dutch Cone penetrometer would be the most appropriate instrument.

Mr Evans said that it was sometimes not possible to acquire Dr Northey's "clear aim of the information required to be obtained" because 'you didn't know what you should look for until you had begun looking' He also asked whether it was conceivable that there would ever be such a thing as standardised testing. Each type of test could be standardised he said, but different types often gave different results. The designer was then left with a range of figures from which to select design values.
Dr Northey replied that one could always begin with a clear ultimate goal which could be made progressively more specific as the investigation proceeded. He agreed that getting different results from different types of test was something which would probably always have to be lived with.
Mr J.M. Booth (N.Z.R., Wanganui) initiated a discussion on the N.Z.I.E. Bore Hole Index. Dr Northey said that it should be a valuable resource but that engineers appeared to be neither contributing to it nor using it.
Mr J.P. Blakeley (Beca Carter et al., Auckland) said that the scheme now needed one or two enthusiastic members in each centre.

Mr J.H.H. Galloway (M.O.W. Central Labs.) raised the question of copyright, "who owns the data?" He said that it had been suggested that local bodies or the local branch of the N.Z.I.E. should hold the data.
Mr Belshaw said that in Napier the M.O.W. held the cards - several hundred of them.
Mr K.H. Gillespie (Brickell Moss et al., Lower Hutt) said that while he thought that Government departments, such as Geological Survey, should have access to the data, it would be difficult to guard against the type of engineer who might use it to avoid the cost of drilling on his own site. He suggested that the engineer who developed a foundation report should remain in possession of and be responsible for it.
Mr Chave said that in Dunedin the Index was held by the City Engineer and that in using it one had to refer back to the authority initiating the investigation and to the firm who carried it out.
Mr I.D. MacGregor (M.O.W., Hamilton) said that at the M.O.W. Hamilton office positions of bores are logged on 1" to 1 mile maps and job numbers are used to relate plotted positions to storage files.
Mr M.T. Mitchell (Waikato Technical Institute) said that all foundation investigation logs and reports were the property of the client. He said that it was not reasonable to call up an opposition firm and ask for information. He felt that each foundation investigation should be complete in itself and not rely upon the work of others.

Mr Chave said that in a recent job a Dutch Cone test had given reasonably constant soil properties with depth while the Raymond penetrometer had given properties improving with depth. The difference was due apparently to the mass of the drill rods in the latter method. A Hiley analysis had shown that the rod effect could increase N values by a factor of 2 or 3 at depths of 50 ft with rods weighing 10 lbs/ft. run.
Dr Northey replied that the effect of drill rod length was one of several variables in the SPT which was not taken into account as it ought to be. The British Standards stated that the rods should be at least 'AW' but they gave no upper limit. The mass, rigidity and length of the rods all had a considerable effect and factors of the order of 2 or 3 might be quite usual, he said.

Mr Blakeley asked for comment on the effect of confinement of the ground on SPT values.
Dr Northey replied that the original American practice was to use a hole no more than 4 inches in diameter, but that British practice had grown up to use penetrometers in much larger holes and even in open pits. There was no doubt that the degree of confinement had a tremendous effect on the values obtained.

In the discussion on the SPT which followed, Dr Northey and Mr Mitchell referred to a paper by de Mello (1971) which, it was agreed, should be consulted by those interested in this test.

Mr Blakeley asked for information on the Menard pressuremeter. He said that he was under the impression that the instrument measured deformation of the soil rather than shear strength.
Dr Northey replied that the instrument measured the deformation properties of the soil, and that usually (because of safety factor considerations) it was the deformation modulus rather than the ultimate shear strength which was of interest.

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Reference

DE MELLO V. 1971 "Standard Penetration Tests" Proc. 4th Pan American Conference on Soil Mechanics and Foundation Engineering Vol.1 pp 1-86.

# REVIEW OF THE INTERDEPENDENCE OF STRUCTURE AND FOUNDATION

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The responsibility of the Structural Designer is primarily to provide a structure of economic cost and of satisfactory performance for any given project. In respect of the foundation, satisfactory performance could be broadly defined as the absence of damage arising from differential settlement, lateral earth pressures, ground water problems or site instability. Similarly there should be no damage to existing adjacent construction arising from either the construction processes adopted or from settlement associated with the weight of the new structure.

Undoubtedly the prime objective is the satisfactory support of vertical loads and the major part of the paper will be confined to this one aspect. This paper, therefore, aims to review the principles of the relationships between design requirements for the superstructure and subsurface conditions for any particular project. It is hoped to show that such relationships are based primarily on differential settlement considerations leading initially to a decision on founding depth. If the Designer then has a clear understanding of the interdependence and influence of these two criteria on any particular project, he will be in a position to make the appropriate decisions on type of foundation, and on the use of the design soil stresses and coefficients recommended in the Site Investigation Report.

All structures experience settlement and hence differential settlement to some degree. Differential settlement will only show itself when the magnitude of the movement exceeds the capacity of the various structural and nonstructural components of the building to withstand the imposed relative deflections. The design approach for any particular project, therefore, is to determine what factors influence the absence or presence of significant differential settlement.

These factors can be regarded as falling in one of two categories, namely those associated with the potential for absolute settlement in the soils underlying the site, and those associated with the sensitivity of the building to differential settlement. A consideration of the first category requires that the Designer reviews firstly the probable settlement behaviour of all the underlying soil strata and identifies those zones of soil which represent zones of potential settlement for the project under consideration, and secondly the order of magnitude and variability across the site of potential settlements in these zones. Where zones of potential settlement exist, the chances of significant differential settlement increase both with the magnitude of the absolute settlement, and with the variation, either known or only suspected, of the nature or thickness of such zones across the site. This general information and assessment should be readily available from the Site Investigation Report, but it is incumbent upon the Designer to ensure that he has a full understanding of the range of possible behaviour of the underlying soil.

In the second category, the general settlement sensitivity characteristics of building structures, range from the relatively sensitive sheathing, finishes, and structures of most multistory commercial and public buildings and other masonry buildings to the less sensitive sheathing and structures of industrial type buildings, It should be noted, however, that other components of such industrial type buildings, such as crane beams, plant foundations or even the ground floor slab may be more sensitive to differential settlement than the actual building fabric itself.

The initial design process therefore must consist of evaluating the interaction between possible founding depths and the particular form of building required in a given project.

It would be reasonable to assert that most designers would not be prepared to accept the possibility of significant differential settlement for the first group of buildings mentioned above, on the grounds that impairment to weather-proofness and disfigurement from damage to non structural components may result. Equally it would require very special circumstances before most designers would consider detailing all the appropriate non structural components of such buildings to tolerate significant settlement.

On the other hand, most designers would be prepared to accept if need be, the possibility of significant differential settlement in the second group of buildings on the grounds that there are cladding materials and systems available which can tolerate appreciable movement without damage or loss of weatherproofing. Conversely, if circumstances of site and building layout are such that significant differential settlement cannot be avoided, then the Designer should adopt these types of cladding materials. The cladding materials in question include various proprietary metal and asbestos cement sheeting products. They are generally lightweight which is necessary if the magnitude of differential settlement is to be kept to a minimum, but they offer little or no fire rating and are therefore either excluded or are insufficient by themselves in situations where the Building Bylaws require fire rating on external walls. In a sense therefore, the Type of Construction required by Chapter 5 of N.Z.S.S. 1900 influences both the settlement sensitivity of any building, and, through the weight of the necessary construction materials, the magnitude of the settlement.

A consideration of the possible foundation systems for the first group of buildings, where significant differential settlements cannot be accepted, leads to three basic alternatives which are:

(a) to adopt a founding depth below those strata which represent zones of potentially significant settlement

(b) to found above potentially compressible zones by adopting a "load balance" system where the applied building loads do not exceed the weight of overburden soil removed from the site. This alternative usually but not always implies one or more basements and is usually associated with alternative (c) below

(c) to found above a potentially compressible zone by adopting a structural foundation system of sufficient stiffness to ensure the necessary redistribution of ground contact pressure and hence to control the pressures causing differential settlement. Examples are raft type foundations in multistory constructions, and continuous interconnected strip footings in domestic constructions on filled sites.

The settlements associated with the first alternative will generally be negligible and generally likewise for alternative (b) but significant settlements could be associated with the last alternative (c).

In this case the effect of this settlement on adjacent structures would have to be considered before making a decision to adopt alternative (c).

It should be noted that the "stiff foundation system" is not always an appropriate system when a marked difference in settlement will occur across a building, where, for example, potentially compressible soil occurs only under one edge or corner of a building. Rather the "stiff foundation system" is appropriate where reasonably uniform settlement is anticipated across the building, but where local variations in thickness and nature of compressible material could lead to independent footings experiencing excessive differential settlement.

The three foundation systems described above are of course equally applicable for the second group of buildings, where significant differential settlement could be accepted.

The fourth basic alternative foundation system applicable to the "less sensitive" group of buildings is to found above potentially compressible zones and rely on the settlement analysis to ensure that differential settlements while significant, will not be excessive for the particular structure. Where the anticipated differential settlements are judged to be excessive, it is often possible for buildings in this group to make provision for subsequent jacking to re-level the structure if this proves necessary.

Finally in this review of interaction between possible founding depths and probable building form, the influence of the four alternative foundation systems on the main structural system is reviewed. The first alternative, founding below all zones of potential settlement, does not have any direct influence or restriction on the structural system where the founding level is close to the ground surface. Where the founding level is at considerable depth however, there is an incentive to adopt relatively wide column spacing or equivalent structural system to reduce the number of deep foundations.

As noted previously, the second or "load balance" alternative usually implies a basement structure and associated stiff foundation structures as in the third alternative. In order to achieve effective redistribution of loads with these alternatives without an excessively lavish foundation structure, the use of bearing walls and/or close column spacing might be preferred.

The fourth alternative foundation system, with differential settlement of a possibly significant order, could have more notable influence on the associated main structure. Statically determinate type structures are preferred to the settlement sensitive indeterminate type and a low weight structure is preferred to minimise the absolute settlement. Conversely, the adoption of wider column spacing might justify the expense of deeper foundations to found on more competent soil and hence to reduce the magnitude of the potential differential settlement.

This above discussion, then has attempted to cover very briefly some of the principles of the interdependence of the type of building and founding depth. The paper has stressed the concepts of founding depth and differential settlement, and not, it should be noted, the less fundamental principles of type of foundation construction such as pile types or allowable bearing pressure. The type of pile or other foundation should be merely regarded as a means of achieving the desired founding depth. In practice for any given project, of course, there may be more than one possible founding depth, and it may be necessary to further evaluate the types of foundation construction and construction process before a final choice of founding depth is made.

# RELATING TESTS TO ANALYSIS - CURRENT PRACTICE

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## INDEX

## 1. INTRODUCTION

It is not intended in this paper to describe in detail the wide variety of soil mechanics tests which can be done, nor the many methods of analysis available. For typical foundation engineering considerations (such as bearing capacity and settlement) some of the more common methods of analysis are listed, together with relevant tests which may provide the information necessary for this analysis. While for some problems there are almost universally accepted tests and analytical procedures, for others a variety of approaches is possible, and a constant search goes on for better methods.

Tests can be subdivided into those done in the laboratory and those done in the field. Alternatively, they may be placed in three broad categories:

(a) classification and other simple physical tests

(b) determination of strength parameters

(c) determination of compressibility, or other deformation properties.

Some tests (for example C.B.R.) include elements of both strength and deformation measurement.

## 2. IN-SITU TESTS

Tests made upon the ground in-situ avoid the disadvantage of disturbance during extraction, transportation and extrusion of the sample for laboratory tests. Many of them do not measure fundamental soil properties. For valid interpretation they must all be preceded by positive visual identification of the soils concerned.

### 2.1 Large Scale Load Tests

Soil or industrial raw materials conveniently available, can be stacked on the ground to produce large scale load tests if close control or variation of load intensity is not required. Such tests are normally conducted to measure settlements, but can be contrived to produce general shear failure if the site is not to be used subsequently.

Test loads are usually applied to piles via jacks equipped with some form of load gauge so that loading rates can be controlled and load cycles varied. Definition of "failure" in such tests is not always clear and many tests are mounted with reactions inadequate to produce shear failure in the soil, in which case they may function only as proof tests to some particular load.

### 2.2 Plate Bearing Tests

At a smaller scale of test, loads are applied at controlled rates via jacks to rigid plates bedded on the ground at the level of interest. The commonest standard test of this type (Ref. 15) uses a 30" diameter plate to determine a modulus of subgrade reaction (k). A mobile load (eg. a loaded transporter) totalling about 30 tons is required. The tests are therefore expensive to conduct. Also the depth of their influence is only in the order of 1½ to 2 times the plate diameter and their use is really limited to pavement design, including floors. Smaller plates can be used, less accurately, if a correction factor is applied.

## 2.3 CBR Test

This is an even smaller load test using a 3 square inch plunger to obtain empirical values used for pavement and floor design. Its zone of influence is even smaller than for plate load tests, but is cheaper to perform in quantity, requiring only a truck or a landrover as a reaction load. (Ref. 15).

CBR tests provide empirical numbers not directly related to more fundamental soil strength properties.

## 2.4 Penetrometer Tests

The most sophisticated penetrometer in current use is the Dutch cone penetrometer (Ref. 26) which measures separately the static end resistance to penetration of a conical point, and adhesion resistance on a sleeve just above the conical point. Enthusiasts claim very wide capabilities of this instrument. Its real use has been in cohensionless soils, with less use in clays, or as a model-prototype device (Ref. 6) and it has been proved principally as a means of designing pile foundations. In New Zealand practice, its principal use is seen as a means of filling in information of continuity of strata between boreholes, but in some localities an accumulated experience in particular soil conditions has lead to reliance upon the test for solution of a variety of problems.

The Raymond penetrometer or standard penetrometer test (SPT) is a dynamic test measuring resistance to driving, with standard effort, a standard tool fixed to the end of drill rods (Ref. 27). The test is quite empirical but has been related to performance observations of buildings over many years, to build up a conservative design approach principally in non-cohesive soils.

Properly-conducted and corrected for overburden pressure, and some soil grain size effects, the SPT test has been the only economical means in common use for assessing bearing capacity of sands and gravels below the water table (Ref. 13). It also provides comparative information between boreholes in which more sophisticated tests are conducted.

Apart from the standard Raymond sampler, other kinds of sampler can be driven with standard effort to provide approximate assessments of variations of ground consistency, at least in bores where high quality samples are not being taken.

## 2.5 Pressuremeter Tests

Menard (Ref. 11,12) has developed a pressuremeter which basically measures the deformation of a limited stretch of borehole by subjecting it to controlled internal pressures, which simulate the behaviour of elastoplastic expansions of cylindrical or spherical cavities. Successive tests along the length of the bore provide the modulus (E) and the limit pressures ($P_L$) representing general failure of the soil. Boreholes must be made to a close tolerance on diameter to suit the probe and soil identification must be made. The metre long probe has the advantage over laboratory tests of testing a larger mass of soil at each stage.

Appropriate formulae use the E and $P_L$ values to compute bearing capacity, settlement, pile capacity and wall pressures.

## 3. OBTAINING LABORATORY TEST SPECIMENS

### 3.1 Types of Sample

Soil sampling methods will not be discussed in detail, but they must be mentioned in relation to their effect upon the results obtained in some tests. A full discussion by Northey & Thomas can be found in Ref. 25.

Liquid and plastic limits and particle size distribution may be determined from disturbed samples, and the water content is unimportant. Clays should not be allowed to become air dry however, as this may alter liquid and plastic limits. Density is usually measured on undisturbed samples, though for saturated clays neither density nor water content is altered by some sample disturbance.

Both deformation properties and strength parameters can be significantly affected (usually adversely) by poor sampling. In this respect some of the more common sampling methods are classified below:

Table 1 - Sampling Methods

| | Result | Remarks |
|---|---|---|
| **From Pits or Shafts:** | | |
| Hand carved blocks | Good | Samples awkward to handle in the Laboratory |
| **From Boreholes:** | | |
| Thin walled samples, 4" dia. | Good | Large enough to obtain three 1½" dia. specimens from one level |
| B.R.S. sampler (4" dia.) | Fair | (ditto) |
| Ring-lined sampler | Fair | Liner rings used in consolidation and direct shear tests simplify laboratory specimen preparation |
| Thin-walled samplers, 2⅜" dia. | Good | Used in clean sands above water table |
| Bishop sand sampler, 2⅜" dia. | Good | Complicated equipment necessary in clean sands below water table. Expensive field work |
| Thin-walled samplers, 1½" dia. | Fair | Suitable for hand auger bores in cohesive soils, peripheral disturbance proportionately large |
| Piston samplers | Good | Adaptations of thin-walled samplers |

| | Result | Remarks |
|---|---|---|
| Double or triple tube core barrel sampler | Fair | Suitable for stiff clays and harder materials |
| Open barrel sampler | Poor | Sample suited to classification tests only |
| Raymond sampler, 1⅜" dia. | Poor | Disturbed; suitable for classification tests only |

A tremendous variety of types and sizes of sample is used. With metrication impending, some degree of standardisation - at least of sample diameters - is surely worth consideration.

There is some justification for the range of sample sizes however. Samples about 4" diameter have the advantage that consolidation specimens (conventionally 3" diameter) may be obtained from them, while the fact that three 1½" diameter specimens may be taken from the same level minimises sample variations in a set of three triaxial tests. For clean sands 2⅜" (60mm approx.) appears to be the optimum diameter. While the alteration of density by sampling is always unknown, it is obviously greater in a smaller diameter sample; with a larger tube the sample falls out!

Some organisations use small diameter continuous sampling for preliminary investigation and classification, followed by a few larger diameter boreholes from which 4" diameter undisturbed samples are taken for strength and consolidation tests. The required number and location of the 4" samples is optimised from the preliminary investigation.

During transport and handling the sample must be prevented from moving in its tube and sealed against moisture loss. If the sampling tube is to be fitted with end caps for transport, then any space should be filled, usually with soil, distinguished from the sample by a separating plastic sheet. Firmly tightened screwed caps on BRS tubes provide an adequate seal if the samples are to be tested within a few days.

### 3.2 Preparation of "Undisturbed" Specimens

Even if the sample reaches the laboratory in an almost undisturbed state, careless handling during preparation of the test specimen can radically alter its properties.

For clean sands, the sample dimensions should be measured in the sampling tube in the field and the whole sample bagged then dried and weighed in the laboratory to determine its density. Tests are done on samples recompacted to original density. Fortunately, many sands are sufficiently cemented to make it possible to transport and prepare undisturbed specimens.

Cohesive soils are formed into test specimens either by extrusion into suitable moulds, or by hand trimming. Careful extrusion, particularly of saturated soils, can result in very little disturbance to the specimen. The cutting edge of the specimen mould should be at the same level as the end of the tube from which the sample is being extruded. When extrusion is from a 4" diameter tube into three 1½" diameter specimen tubes, special precautions must be taken to avoid remoulding.

For consolidation tests the sample is extruded into a ring (usually 3" diameter and ¾" or 1" deep). It should project beyond the ring on both sides before being trimmed flush. Techniques for doing this depend on the nature of the soil. Usually cutting with a taut wire followed by finishing with a straight edge is satisfactory.

Firm to stiff soils and soft rock can be formed into cylindrical samples for triaxial tests by hand trimming in a so-called 'lathe'.

Drilled cores of soft or hard rock are often tested without modification of diameter. Cutting the ends of the specimen parallel and flat is of extreme importance. Any irregularity can cause stress concentrations which will cause premature failure in a compression test.

## 3.3 Effects of Sample Disturbance

If, in the sampling operation, or in the specimen preparation, remoulding or other disturbance occurs, the most careful test will not yield results which are truly representative of the soil in its natural state. In order of sensitivity to disturbance, tests may be grouped into those which determine:

(a) Deformation properties (compression modulus or shear modulus)

(b) Undrained shear strength

(c) Consolidation properties

(d) Strength parameters in terms of effective stress ($c'$,$\phi'$)

There have undoubtedly been instances where unnecessarily expensive foundation engineering works have been carried out because tests on disturbed samples led the designers to believe that conditions were much worse than was truly the case.

Perhaps the most valuable information to be gained from a consolidation test is the preconsolidation pressure. Partial remoulding may render the estimation of this pressure impossible.

## 3.4 General Standards of Testing Work

The quality of the testing is of vital importance. Before the designer can use test results in his analysis with confidence, he must be assured that the test equipment is suitable and well-maintained, and that the operator knows his job and will conscientiously record the results - not as he expects them to be, but as they really are - and will record also the occasions when "things go wrong" as they sometimes, inevitably, do in geomechanics testing. Because it is misleading, the result of a poorly conducted test may be worse than no test result at all. Quantity is no substitute for quality. A single accurate test result may be of more value, in some cases, than a dozen inaccurate results. On the other hand, because of the variability of natural soils, it is usually necessary to carry out a number of tests to obtain a single parameter. From this group of tests, not only can an average result be determined, but also the degree of variability (scatter) be found. In some cases it may be necessary to estimate possible maximum and minimum values so that upper and lower bounds can be found.

## 4. CLASSIFICATION AND OTHER SIMPLE PHYSICAL TESTS

The simplest and most valuable classification test is visual inspection either in the field or in the laboratory. Subjectivity of classification is largely overcome if a standard method of description is used by experienced personnel. The standard means of classification of fine-grained soils is by liquid and plastic limit tests which become much more meaningful if they are related to natural water content, and if they are actually plotted on a plasticity chart.

The bulk density of a soil is required in most calculations. For cohesive materials this is most simply obtained by measuring the mass of an undisturbed sample whose dimensions are known accurately; in granular soils dimensional distortion is much harder to prevent in sampling.

Particle size distribution can be obtained from tests which, although basically simple, can be time-consuming (and therefore expensive) if extended to include the silt and clay ranges. For soils with appreciable fine fractions the information obtained is of limited value. For sands and gravels on the other hand, such tests are more readily performed and give worthwhile information.

The tests listed above are of value for correlation purposes; that is, recognising similar soil strata in different boreholes and determining the extent of material which, for purposes of analysis, may be considered "uniform". For cohesive soils, not only liquid and plastic limits should be compared - water content or density may be just as valuable for correlation purposes.

One of the biggest difficulties in any geomechanical analysis lies in reducing the mass of data describing (incompletely) the actual subsurface conditions to a simpler model, which can reasonably be analysed. Gross simplifications must be made in order to make the problem analysable at all; yet the finally-selected model must still resemble the field situation in its essentials. There is no doubt that the judgement and experience mentioned so frequently are important in this preliminary phase of the analysis. Accurate soil descriptions and adequate classification tests are necessary before these can be put into action.

## 5. STRENGTH TESTS IN THE LABORATORY

The strength of a soil may be expressed in terms of effective stress parameters $c',\phi'$, or in terms of total stress parameters $c,\phi$.

The use of these parameters in practical problems is discussed later in this paper.

The most versatile method of determining soil strength is in the triaxial compression test. Drainage from the sample can be controlled, the rate of testing can be varied over a wide range, and in undrained tests, pore water pressures can be measured.

For saturated non-fissured clays, $\phi = 0$ and, in terms of total stress, only the cohesion $c$ is to be determined. Triaxial testing for such soils is unnecessary. A simple unconfined compression test will give the required result, but undrained tests on overconsolidated or brittle soils are better done with triaxial pressure. The cohesion c is half of the compressive strength. A shear vane test will also determine the cohesion. In the laboratory a miniature vane (usually ½" diameter) is used.

For soils of high permeability such as clean sands or gravels, the cohesion is zero and it is the angle of shearing resistance, sometimes termed the angle of internal friction $\phi'$ (an effective stress parameter) that is required. $\phi'$ can be found from drained triaxial (or shear box) tests on specimens recompacted to the original density. This is often inconvenient for gravels in large samples. A diameter at least 5 times, preferably 10 times, maximum particle size must be used. Frequently $\phi'$ is estimated from density and classification tests for these materials, and not measured directly in the laboratory.

In clays and other slow draining soils, effective shear parameters $c'$ and $\phi'$ are found from special tests. Drained triaxial tests will give these parameters but on low permeability soils may require to be run at inconveniently slow rates, possibly taking several days for a single test. Undrained tests with measurement of pore water pressure enable the result to be obtained more rapidly, but much more elaborate equipment is required. Such triaxial testing is not to be undertaken lightly. The equipment and techniques entailed are well described by Bishop and Henkel (Ref. 2) in a sizeable book devoted exclusively to triaxial testing.

While the basic principles have not changed, noteworthy developments in recent years include the use of "free ends" (that is almost frictionless conditions at the ends of the specimen); electrical transducers to measure pore water pressure conveniently and accurately; and "high air entry" porous end plates to ensure in unsaturated soils that it is pore <u>water</u> pressure, and not pore <u>air</u> pressure that is being measured.

## 6. <u>CONSOLIDATION TESTS</u>

Good sampling and specimen preparation techniques are especially important in consolidation testing. Samples which are appreciably disturbed will yield consolidation test results which may be misleading.

The apparatus commonly used accommodates a 3" diameter sample, ¾" or 1" thick, though smaller diameter samples are sometimes used. Loads are applied by adding weights to a lever, and the usual procedure is to double the load at regular intervals. Daily increments were once regarded as 'standard' but more frequent increments may be applied, provided that the intervals are equal and that 90% consolidation is reached at every load. For soft clays ¼ to 8 tons/sq.ft may be an adequate range, but for preconsolidated clays it may be necessary to go up to 16 or even 32 tons/sq.ft to define the 'virgin consolidation line' adequately. Dial gauge readings should be taken during unloading (usually in one or two steps) as this information is of considerable value.

Information on the amount of compression is given by a graph of void ratio (or sometimes strain) against the logarithm of the consolidation pressure. From this laboratory curve the preconsolidating pressure is estimated. Then the 'field consolidation line' must be deduced (Refs 17,22) before realistic settlement estimates can be made. Even then an accuracy of ±30% is all that can be expected.

The rate of settlement of the structure is found from the coefficient of consolidation determined during the test. Estimates of rate of settlement are even less accurate. Usually (because of horizontal drainage etc.) actual settlement occurs more rapidly than estimated.

Recent developments in consolidation testing include:

(a) loading by air or water pressure (as in Karol-Warner, and Rowe cells)

(b) provision for application of back-pressure, which results in better estimates of coefficient of consolidation, and

(c) continuous loading, as in the controlled gradient test, enabling preconsolidation pressures to be more accurately assessed.

## 7. METHODS OF ANALYSIS OF GROUND DEFORMATION UNDER APPLIED LOADS

The foregoing sections of this paper have discussed some of the available test methods, the results which they yield and the kind of samples necessary for those tests.

The following sections consider the ways of using the site and laboratory information to estimate the effect on the ground of the loads imposed by buildings, under the main headings of the kinds of reactions usually of concern. The methods of analysis discussed, in the main, are those in established use.

### 7.1 Bearing Capacity Against Shear Failure

7.1.1 The first concern is that the ground has sufficient shear strength to support the building loads communicated to it through a system of structural foundations, without failure of the soil and consequent large displacement of the foundations.

7.1.2 Data required:

The magnitude distribution and duration of all the dead and live loads of the building and any expected eccentricity of loads on foundations are required, together with arrangement of the load bearing columns and walls; there may be alternative arrangements of load bearing members to be considered. From this information, foundation configuration, depth and size have to be postulated for analysis.

Information about the ground must include its shear strength under appropriate conditions of drainage, in-situ density down to depths below the footing of 1.5 to 2 times the footing width, water table level and variations of water level for any reason.

7.1.3 Analysis

The commonly used bearing capacity theory of Terzaghi (Refs. 21 & 22) provides adequate formulae applicable to cohesive and non-cohesive ground, which has been elaborated by others to allow for embedment, two layer ground systems (Ref. 21.1), eccentric loading (Ref. 21.3) and sloping ground (Ref. 21.4). The theory assumes general failure around a complete surface of shearing at one time, that is a plastic condition. A factor of safety of 3 commonly is used to cover uncertainties of shear strength measurement as well as inadequacies of the theory, so that failures which might give checks on the theory are rare.

In ground which is not homogeneous, yield probably will commence locally in the weakest strata; an eventuality which should be checked by comparing soil shear strengths with imposed shear stresses estimated from elastic theory (Ref. 15).

These methods are easily applied in cohesive soils. In sands and gravels however direct measurement of angles of friction for use in the Terzaghi or other formulae, is difficult. Recourse is then made to indirect derivation from measurements of relative density or penetration resistance (Ref. 13). Standard penetrometer (Raymond) tests (or SPT), while crude in nature, are cheap tests to perform in quantity and results can be averaged. At shallow depths where SPT tests are unreliable, in-situ density can be measured and samples reconstituted for shear tests.

Ménard (Ref. 12) has developed design procedures which use results of in-situ pressuremeter tests, not directly related to the shear parameters of Terzaghi's formulae. These procedures require specialized equipment and experienced interpretation, but apparently yield results of practical reliability in all classes of soil. The test method is particularly valuable in cohesionless soils from which it is very difficult to extract undisturbed samples.

#### 7.1.4 Future Development

Further theoretical development of general bearing capacity theory and factors is not as important as are means of measuring shear strength parameters of the soil accurately and cheaply, so that sufficient numbers of results are available for averaging or for plotting.

The stress path methods of Lambe (Ref. 10) represent a better theoretical approach, but the triaxial tests entailed are very difficult to carry out and when soil properties vary in the stressed zone, as they often do, several tests are required.

### 7.2 Settlement

Whereas the ultimate condition of general shear failure is prevented by adoption of a factor of safety, settlement will always occur and can be measured.

7.2.1 The amount of tolerable differential settlement between parts of a building is commonly the limiting criterion in its foundation design, rather than shear failure.

Settlement is caused by the weight of the building, and its contents, filling beneath or adjacent to the building, and variations in water table level.

#### 7.2.2 Required Data

Data about the building must include the magnitude, rate of application, duration of the loads, the area over which they are distributed (which involves an initial assessment of footing sizes in terms of bearing capacity against shear failure), their founding depth, and their position in relation to each other. Loads include structure and contents, positive loads of new filling and negative loads of excavation.

With regard to the ground, it is essential first to identify the nature of the strata down to a depth equal to twice the width of the structure or fill load, unless incompressible strata are known to exist below a lesser depth. Values of in-situ bulk density and water table level are necessary to compute the existing in-situ vertical pressure $P_o$.

Compressibility characteristics of the strata must be determined in sufficient quantity to represent the significant strata. In the case of consolidation settlement of fine grained cohesive soils, values of in-situ void ratio ($e_o$) compression index ($C_c$), swelling index ($C_s$) and preconsolidation pressure ($P_c$) are required; together with time rates of consolidation in relevant pressure ranges. The immediate settlement component requires values of elastic modulus from triaxial or in-situ tests. Compression or elastic moduli of sands and coarser "non-cohesive" soils under static load can only be tested in the laboratory if samples can be obtained at, or reconstituted to, in-situ density which is very difficult to measure below the water table unless the soil possesses some degree of cohesion (eg. pumice ash relatively unweathered).

7.2.3 Analysis

7.2.3.1 Cohesive, slow draining soils

The conventional analysis (Ref. 21.5) uses initial and final vertical effective stresses obtained from published plots of contours of stress derived from three-dimensional elastic theory. These vertical pressures are then applied to elements of soil for which one-dimensional consolidation characteristics have been found in oedometer tests, in which lateral strains are excluded, and one-dimensional strains of the layers are summed. Reasonably accurate estimates of settlement are obtained where the depth of consolidating soil is less than the width of the footing. Consolidation occurring within depths of 1.5B for square or circular loads and 2B for strip loads (where B is the width of the loaded area) generally are computed. In soils of very high compressibility, like peats and recent muds, significant absolute amounts of settlement may occur at greater depths.

The soil is divided into horizontal layers of different consolidation characteristics and layers of relatively incompressible soil, like sand, are considered not to affect the assumed distribution of pressure.

The time rate of settlement also is computed from oedometer test results but this is of a much lower order of reliability than the estimate of amount of settlement, and is greatly affected by included minor, barely visible layers of faster-draining soil (Ref. 16).

The one-dimensional analysis gives reasonable estimates of settlement even though immediate settlement resulting from soil displacement is neglected. Some authors (Ref. 20) suggest that consolidation settlement in overconsolidated soils may be overestimated by conventional theory. They advocate a correction factor and the addition of immediate settlement. Thus it appears that conventional methods may give approximately the right result but for wrong reasons. This immediate movement is often not included in the timing of field observations.

Computed settlements are very sensitive to errors in estimating the preconsolidation pressure ($P_c$), beyond which the slope of the e/log p curve may increase by a factor of as much as 10; this slope itself is greatly affected by sample disturbance.

Secondary consolidation contributes a long-term addition to settlement, which is important in softer and organic soils.

7.2.3.2 Granular Soils with Little Cohesion

Settlement in these soils is due to rapid compaction and is estimated from penetrometer resistance, elastic moduli or load tests in-situ.

SPT tests have been related to post construction observations in published charts, recognised as being very conservative in recent reports which suggest modification toward realism (Ref. 19). The field SPT values must be corrected for overburden pressure (Ref. 7) and the effect of high pore pressures set up in fine silty sands (Ref. 19); this is not always made clear in texts.

In elastic analyses (Ref. 10) moduli are used which are difficult to measure as they are stress dependent and very sensitive to disturbance of the soil structure and density by sampling. Above the water table, in-situ density can be measured and reproduced in triaxial test samples. Below the water table this is almost impossible and in-situ measurement of moduli with the Ménard pressuremeter is more convincing (Ref. 11,12). The other problem in this type of analysis is the depth of influence which should be assumed, since the analysis is not incremental (See Ref. 10).

7.2.3.3 Other Methods

Ménard applies pressuremeter test results in theories of his own, which yield analyses of settlement in all types of soils (Ref. 11). The method has been tested in practice and avoids the difficulties of extracting and handling satisfactory samples. The method of analysis is sometimes complex if the effects of adjacent footings and compensating excavations are included.

Field load tests can also be used. Plate loads suffer from the drawback of small size and depth of influence (i.e. a scale effect). Large scale tests, with soil heaps as the load, are particularly useful in heterogeneous ground, such as rubbish tips or in soft peat soils, which cannot be sampled effectively.

7.2.3.4 Computer Programmes

Programmes are in use which permit calculation of pressures and settlements from complex interaction of adjacent loads and many subdivisions of soil strata. It must be borne in mind that the mathematical refinement of these calculations is currently way ahead of the quantity (and probably quality too) of data obtained from site investigation and laboratory test programmes. Considerably more money will have to be spent on drilling, sampling and testing to get the most out of computer calculation, and this is generally justified only for larger buildings and more critical situations.

7.2.4 Future Development

The stress path method (Ref. 10) may become more widely used to measure strain at selected points for a summation of strains, or to determine the modulus for use in the elastic method. The same limitation on numbers of tests applies as mentioned in Section 7.1.4.

## 7.3 Floors on the Ground

7.3.1 Large area loads such as stacked bulk materials are considered from the standpoint of bearing capacity and settlement, as discussed in Section 7.1 and 7.2; the floor itself has a small effect on distribution of the load.

In respect of wheel loads and static loads covering small areas, floors on the ground are designed, as are road and airport pavements, either rigid (concrete) or flexible (bituminous). (Ref. 24).

Special types of "swelling ground" cause problems of ground pressure against ground floors, but in all soils which exhibit marked cracking or drying, stability of moisture content must be considered (Refs. 9,10).

7.3.2 Data Required

Intensity, area and frequency of loading are required, and ground capacity is assessed from CBR, subgrade reaction modulus (plate bearing tests) or shear strength. For heavily loaded floors (industrial buildings, aircraft hangars, etc.) quality and grading of basecourse stones and reactions of soils to lime, cement or bitumen stabilisation may be relevant.

7.3.3 Analysis

Various methods of analysis are described by Yoder (Ref. 24) for example. Those building floors which are specifically designed are commonly dealt with by CBR or subgrade modulus methods which essentially relate in-situ test results to pavement thickness via charts built up from performance observations.

Fairly complicated analyses of swell pressures of ground wetting up below floors can be based upon soil suction tests. In most New Zealand situations it is probably sufficient to ensure that highly plastic soils are brought to near saturation at a density compatible with adequate shear strength and low compressibility, before the floor is placed upon them.

7.3.4 Future Development

Probably, attention to details of construction, particularly at joints of rigid floors, uniformity of subgrade and base preparation are more important than better design methods.

## 7.4 Raft Foundations

7.4.1 If the bearing capacity of the ground is so low that there is no room for individual footings of sufficient size, or if differential settlements are intolerable, then the whole building can be placed upon one footing slab covering its whole area, and this is commonly called a 'raft' or 'mat' footing. The rigidity of the raft slab may be varied widely by virtue of variation of thickness or by ribs, which may be walls in a basement space.

7.4.2 Data Required

Net "floor" load from the structure and contents, compensated by the weight of any excavated ground, and distribution of the load are required. All the data as for bearing capacity and settlement analysis are required from the soil, together with a modulus of subgrade reaction, if the flexibility of the raft is to be considered.

7.4.3 Analysis

Methods of analysis, such as described by Teng (Ref. 21.6), rely heavily upon assessment of the subgrade modulus or "spring constant", as it is sometimes referred to. This is a purely empirical test on a limited area of ground, suffering from the drawbacks of scale effect. A good deal of refined structural analysis therefore tends to be done upon rather tenuous data.

7.4.4 Future Development

Better methods of measuring soil rigidity in relation to the structure are likely to produce more economical structures.

7.5 Retaining Wall Pressures

7.5.1 Ground pressures on retaining walls depend primarily upon the deformation of the wall and hence upon the method of construction and structural fixity and rigidity of the wall. Large factors of safety in the structural design often cover wide uncertainties about earth pressure.

Surcharge loads of adjacent buildings should be considered and may be of importance in underpinning problems.

7.5.2 Data Required

The wall rigidity and installation methods, as they affect yield of the supported ground, are fundamental. Soil density, total and effective shear strength parameters both behind and under the wall, ground water level and fluctuations and the relationship between vertical and lateral pressures in the soil (K) are required. Conditions of water drainage behind the wall are important.

7.5.3 Analysis

Minor walls are designed using published methods in which lateral coefficients K are selected on very broad evaluations of wall deformation (Ref. 8). The distribution of assessed total pressure must then be adjusted to suit the assumed deformation (Ref. 8,14), a fact which is often forgotten in dealing with basement walls of buildings which are usually rigid and attract higher than "active state" pressures.

More sophisticated analyses by methods of Rowe and Brinch-Hansen are justified for major walls, in which case worthwhile economy may result.

7.5.4 Future Development

More work on Rowe and Brinch-Hansen theories, to reduce these to a form suitable for general use in design, appears to be the most profitable line of development.

For a review of pressures on propped walls see Ref. 12.

## 7.6 Slope Stability

7.6.1 Adequate analysis of the stability of ground slopes is a complex matter needing special care. It is only major buildings whose weight contributes significantly to the stability of a slope, but the consequences of slope failure become more serious with the presence of the building.

Major slopes in heterogeneous ground require very careful identification of all minor irregularities in strata, expensive testing to establish shear parameters and special studies to postulate mechanisms for analysis.

Only slopes of small height and apparent stability, in homogenous ground, are appropriately included in a symposium on building foundations.

### 7.6.2 Data Required

The nature of the soil, its density, total and effective stress, shear parameters, water table level and fluctuations, existing and proposed slope angles and surcharge loads are all required. Careful examination for superficial signs of instability in the area should be made.

### 7.6.3 Analysis

The first and simplest approach is to compare the case slope with existing slopes which are similar in height, water table regime, drainage and soil composition - if there are any. For minor slopes such precedents may be sufficient assurance.

The immediate stability (i.e. before pore pressure changes or weathering are effective) of slopes in homogenous soil can be checked by use of undrained shear strengths and Taylor's curves (Ref. 4).

Stability in terms of effective stress can be examined by use of charts published by Bishop and Morgenstern (Ref. 3). These assume homogeneous ground and an accurate assessment of future pore water pressures in the ground.

## 7.7 Heave of Excavation

In the case of major basement excavations producing considerable relief of existing ground pressure the elastic expansion of the soil may be of consequence, but in the writer's experience such movements have not had any practical effect upon buildings, although heave or swelling movements have been detected where bench marks have been located in quite minor excavations.

## 7.8 Piled Foundations

7.8.1 Previous sections dealing with bearing capacity have been related to relatively shallow foundations. Piles are used to carry building loads down to depths where ground strength improves sufficiently to justify much higher bearing pressures. If this capacity is sufficient for the piles to be considered end bearing then calculations are simple.

In softer ground available analytical methods of assessing bearing capacity of piles involve much greater simplification of mechanism and consequently are far less reliable than in the case of shallow footings. It is common practice to carry out full scale test loads on some of the constructed piles on a site to prove their capacity.

Estimates of load capacity are made either from dynamic formulae using data recorded during the driving of the pile, or from static formulae (Ref. 23); only the latter are considered here.

### 7.8.2 Static Formulae

These require strata identification density and shear strength to be determined to depths well below proposed founding level, also compressibility of soft soils which may exert a downward drag on the pile shafts, adding to the total load to be borne.

The main difficulty about static formulae is to take account of the yield necessary to mobilise the shear strength of the ground, and the resulting complexity when different strata contribute to the pile support, and where end bearing and shaft friction components are both considered.

It is sound practice to require a load test in those cases where a well established local precedent is absent and where estimates are made from static formulae - except perhaps where hard bed rock is reached.

Group action of a number of piles in a restricted area must also be considered as the total bearing capacity is not the sum of individual pile capacities.

## 8. GENERAL CONCLUSIONS

Computer facilities permit analysis of much more complex theories of soil behaviour, but this is an advantage in practical use only if real soil properties in-situ can be determined reliably, in the form required by the theory and in sufficient quantity to assess their inevitable variability.

Furthermore, all that matters on the final outcome is that the structure performs its function at the lowest cost. The function it is required to perform is not always adequately defined in realistic terms. Economy of cost has to include site investigation and design cost as well as construction cost.

In broader terms, performance of the structure may be obvious; for example complete collapse or gross deformation of the foundations. The margins of safety in foundation design methods currently in use are sufficient to take care of gross malfunction attributable to the soil, provided a site investigation is intensive enough to locate and identify soils of grossly inadequate or variable capacity.

If design methods are to be refined, and this really means in respect of ground deformation (i.e. immediate, consolidation and secondary settlement) then this can only be done as a result of precise and very carefully prescribed measurements of the behaviour of the structure, which can be compared with predictions made from the analysis.

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## PRESENTATION AND DISCUSSION. SESSION 2.

Chairman: L.E. Oborn
(Geological Survey, D.S.I.R.)

The authors in this session presented their papers in person. Mr Imrie amplified points made in his paper and illustrated these with case histories. Mr D.K. Taylor and Prof. P.W. Taylor, faced with the task of producing a "state of the art" resume in 4500 words confined themselves to listing procedures and considerations which they regarded as "current practice" in their own activities, making brief reference to new developments and amplifying points by quoting case histories. Shear vane tests had not (by an oversight) been mentioned in their paper. They quoted cases in which field vane tests had given consistently higher shear strengths than laboratory tests. In view of this they suggested that whereas it was common to employ a F.S. of 3 when working from laboratory shear strengths, lower factors of safety might often appropriately be used if strengths were derived from field vane tests.

With regard to consolidation testing they drew attention to the effects of sample disturbance on laboratory e vs p curves (and hence on estimated preconsolidation pressures) described by Schmertmann (1955).

Dr M.J. Pender (M.O.W. Central Laboratories) who was unable to be present submitted the following comment which was read at the opening of the discussion.

"Prof. P.W. Taylor and Mr D.K. Taylor emphasise the promise of the stress path methods, of Lambe and others, for settlement predictions. The crux of these methods is the determination of deformation moduli in laboratory tests such that the applied stress path is the same as that anticipated in the field. This alleviates the problem presented by the stress path dependency of soil deformation behaviour. However, there are problems associated with the prediction of stress paths and also, as the authors point out, the laboratory testing required is of a much higher standard than that for routine work, and consequently expensive.

For normally consolidated and lightly over consolidated clays the work of the Cambridge group offers an attractive alternative. This is based on a theory for an isotropic, work-hardening, elastic-plastic material, Schofield and Wroth (1968). Although the theoretical treatment is not perfect it constitutes a more realistic model for soft clay than does the assumption of elastic stress-strain relations. Burland (1971) develops a simplified approach to the use of this theory of plasticity and applies it to predict pore pressures beneath embankments and deformation in laboratory model tests. The results are encouraging. This approach is also better suited to predicting horizontal movements than are elastic methods.

A somewhat more rigorous approach to the application of the Cambridge work is the incorporation of the mathematical model into a finite element programme. An analysis of the deformation beneath a trial embankment by such methods is reported by Wroth and Simpson (1972). Emphasis was placed on using parameters from a routine site investigation for the computer analysis. The comparison between predicted and computed settlements for both drained and undrained deformation was good as was the prediction of horizontal deformation. The pore water pressures were also measured but the correspondence with the computed values was not so good; the authors were able to suggest reasons for this. The significant aspect of this work is the fact that one set of soil properties, obtained from routine laboratory tests, lead to a successful prediction of both drained and undrained deformation of the soil beneath the embankment.

Thus we have an interesting alternative for soft clay. One can use the stress path method which is based on concepts everyone knows about and requires very elementary calculations coupled with relatively complex and expensive laboratory work. On the other hand one can use the theory explained by Schofield and Wroth. This is based on concepts that are not quite so well known as those of elasticity but which are not difficult to grasp. Relatively simple and standard laboratory tests (the determination of $c_u$, $\phi'$, $C_c$) are all that is required. These are then used in calculations a little more involved than those for elastic material. I suggest that the combination of the slighly more involved theoretical approach coupled with simple laboratory testing is more economical than the simple theory and very complex laboratory work".

Prof. P.W. Taylor thought that Dr Pender might be right. He said that there had been many versions of the stress-path method (of Lambe and others) presented, and in all of them examples were quoted which illustrated the superiority of each particular author's approach. However, in trying to follow such new methods limitations became apparent - the laboratory tests were usually much more difficult to perform than they appeared to be from the papers and the theories were limited by the fact that they depended on elasticity (Boussinesq) to determine the stresses. Another serious difficulty in applying the stress-path method lay in the redistribution of stresses which can take place under a building and keep it standing whereas a sample of the soil from under the building, when tested in the laboratory, collapsed under the applied loads. The theory developed by Roscoe and his co-workers at Cambridge U.K., referred to by Dr Pender, may well prove to be a more useful approach.

Mr I.M. Parton (M.O.W., Central Laboratories) said that computers might prove most useful in performing repeated calculations on a given problem, covering the range of soil parameter values likely to be encountered, and so obtaining upper and lower limits to a problem.

Prof. P.W. Taylor agreed, saying that the theory of probability might have to be introduced. For one computer run soil parameter values such that 90% of the material on the site was better than they indicated might be used, and for another run values such that 90% of the material was worse.

Mr D.K. Taylor emphasised the necessity of remaining in conversational mode with a computer. "When you get an answer to a computation you need to be able to look critically at that answer, alter the programme or the input data and then rerun the computation immediately. It is just no good setting up a problem, sending it off to a computer and having to wait several days for an answer". He said that it was essential to use programmes which you could understand and alter in accordance with the needs and peculiarities of a particular job.

Mr G.L. Evans (University of Canterbury) raised the question of standard sample diameters and asked about the sealing of field samples in tubes using expanded O-ring seals.

Prof. P.W. Taylor suggested that a list of preferred sizes might be adopted and that the change to metric sizes provided an opportunity to do this.

Mr D.K. Taylor said that they (Tonkin and Taylor) normally sealed their samples with wax on the site.

Mr I.D. MacGregor (M.O.W. Hamilton) said that M.O.W. Hamilton were using the rubber O-rings with thin walled tubes.

Mr G.L. Evans observed that no mention had been made of the effect of earthquakes and other dynamic conditions. He mentioned the possible considerable increases in lateral pressure on basement walls, and the liquefaction of

saturated fine sands. He said that it should now be possible, for major site investigations, to predict the predominant period of a soil layer.
Mr P.G. Imrie said that he usually considered three seismic effects (1) the changes in vertical and horizontal loading (2) the changes in soil properties (such as liquefaction) and (3) whether the seismic effects were likely to be augmented or diminished by the immediate local geological conditions, i.e. by the foundation materials themselves.
Mr I.C. Armstrong (M.O.W., Wellington) said that the design of public buildings placed particular emphasis upon detailing for ductility together with other seismic performance criteria and that the soils engineer should report (where appropriate) on subsoil conditions which could affect the seismic performance of the soil - structure system. For example, a report should indicate whether the subsoil was in the flexible (soft) category as distinct from the intermediate and hard soils. Also, where piles were to be designed for ductility (i.e. prevention of brittle shear failure during plastic hinging at underside of pile cap) advice should be given on the likely range of soil strength resisting pile deformation (e.g. 9 $C_u$ for cohesive soils, see Broms, 1964).
He said that a significant change had recently taken place in the basic approach to the design of R.C. framed buildings, applicable to low buildings as well as to major structures. Some aspects of this change were likely to be included in revised N.Z. loading requirements (NZS 1900 Chap. 8). Design for ductility recognised that in order to survive severe earthquake attack the structure must develop plastic hinges and deflect well beyond yield while preventing any form of premature, brittle failure. In many structures this approach showed that higher column moments and axial loads could exist, and that these must in turn act on the soil.
An example concerned a multi-storey R.C. framed building which, if founded on individual footings might require ground anchors at exterior columns in order to resist column tensions resulting from hinging of beams.
Mr Imrie replied that he would be interested to know what one could actually do with information that the ground was (say) flexible. Was it a matter of putting a rigid short period building on a soft long period subsoil? He agreed that shear in piles was a possible serious weakness in many foundation designs. With regard to column tensions he thought that it hadn't in the past been as big a problem as it appeared to be in the initial design; buildings usually failed somewhere else before they got near to overturning.
Prof. Taylor referred to a paper by Duke (1958) in which information on earthquake foundation damage had been gathered together. He thought that very few foundation failures actually occurred in the ground. Failure was commonly in the structure at some weak point such as the pile-pile cap junction. He said that with regard to soil-structure interaction in the small-strain elastic range, the stiffness (load/deflection) of the soil against horizontal movement, vertical movement and rotation could be measured and perhaps this information should be given to the structural designer.

Mr Galloway said that M.O.W. had recently been examining the ambient vibration response of tall buildings in Wellington. The system used displacement transducers of sufficient sensitivity to measure the very small displacements induced by wind vibration or traffic noise. So far about four buildings had been examined and the records clearly showed components of foundation compliance in translation, rocking and torsion. At present the work was directed towards checking the elastic stiffness of the structure and no work had yet been done on the geomechanical implications of the results. He emphasised that the work related only

to the very small strain (elastic) range of deformation and expressed the hope that the geomechanical work would be prosecuted within the next year or two.
Mr G.E. Barnard said that theory appeared to be behind testing procedures, especially in regard to seismic analysis. He asked whether, in view of this, factors of safety as low as 1.2 should be continued to be used.
Prof. Taylor agreed that there were gaps in the theory - not only in seismic analysis but in almost every branch of the subject. With regard to the low factors of safety commonly employed in embankment design he said that these had been used now for some time and people had been getting away with using them so the theories and the testing procedures couldn't be too bad.
Mr Blakeley asked Mr Imrie to give an opinion on what figure for likely differential settlement should be used for design purposes on a variable site where the range of likely settlement at one point was 2 - 4 inches and at another ½ to 1 inch. Should it be quoted as the difference between the lower limits of the ranges (1½ inches), the difference between the upper limits (3 inches), the difference between the mid points of the ranges (2¼ inches) or the maximum conceivable differential (3½ inches).
Mr Imrie replied that the question must in any particular case be related to the type of building. In general he thought that the "probable maximum" differential settlement should be the aim of prediction - recognising that the actual differential would probably turn out to be less.
Mr D.K. Taylor said that until clients could be persuaded to spend more on measuring what actually happened to their structures then they would get conservative estimates from him. Measurement of actual performance was what was needed in this field before any substantial progress would be made. In general clients just didn't want to know.

Mr Blakeley recalled that Mr Imrie had emphasised that to the structural designer the primary consideration was by how much the structure would settle whereas to the foundation engineer the primary consideration was that the structure would not fail disastrously in bearing capacity (shear).
Having got the safe loads the engineer evaluated the settlement likely to occur. If it was too high he reduced the working pressure on the ground. A large amount of effort commonly went therefore into checking that the structure would stand up and only a small amount into predicing by how much it would settle. He asked whether foundation engineers spent too much laboratory effort on shear strength tests and too little on consolidation tests.
Mr Taylor replied that he didn't think that they (Tonkin and Taylor) did. He observed that undrained shear tests were cheap things to perform in comparison with consolidation tests. Effective stress tests were generally not justified for building foundations - unless there was some question of slope stability.
Mr Imrie said that shear strength tests did give a quick picture of the condition of the clay - in terms of whether it was slightly or heavily overconsolidated. This was often useful information to have early in an investigation.
Mr G.R.W. East (M.O.W., Auckland) said that shear strength tests were useful as a means of comparing material from small diameter boreholes with that obtained, for consolidation testing, in larger diameter holes. He referred to Skempton (1951) for comments on the relationship between settlements and factors of safety in bearing capacity failure.
Finally, he said that a rough guide to the preconsolidation presssure could be obtained from the undrained shear strength (of cohesive soils). Bjerrum (1954) had plotted $c/_p$ versus plasticity index for Norwegian marine clays ($c/_p$ = undrained shear strength/apparent overburden pressure), and other clays described by Skempton. Although this Bjerrum plot applied strictly only to lightly overconsolidated alluvial soils it had been found to describe (roughly)

more heavily overconsolidated residual soils such as the Waitemata clays. As an example a soil with a P.I. of 60 should, according to the Bjerrum curve, have a Cu/Pc ratio of about 1/3. If the undrained shear strength Cu was measured as (say) 1500 $lbs/ft^2$ then the value of Pc would be estimated to be about three times this, i.e. 2 $tons/ft.^2$. In this way a good guide could be obtained as to whether a settlement problem was eminent.

Mr T. Belshaw mentioned the 12" x 12" (Terzaghi and Peck) plate bearing test which he found a useful and simple supplementary test (to deep soundings or bores) for narrow strip footings such as those for housing.
Mr D.K. Taylor replied that if the area and magnitude of the test load were about the size of the load to be put there then they were "the proof of the pudding".

Mr Belshaw said that it was now possible to estimate immediate settlements (due to expulsion of air and lateral bulging) from Dutch cone deep soundings.
Mr D.K.Taylor replied that he usually found that if he were to add estimated elastic and immediate settlements to consolidation settlements he would be doubling predictions which were too big anyway.

Mr Belshaw made a plea that engineers should always arrange to have a man with some training in geology and soil mechanics to record the bore log. The operator had too many other problems to worry about. The extra cost would be money well spent.
Prof. Taylor strongly endorsed this view.

Mr G.D. Mansergh asked about the use of the Menard Pressuremeter in soils which displayed anisotropy of elastic modulus (E) and limit pressure (Pc)
Mr G.A. Pickens replied that the deformation measured in the horizontal plane could be related to deformations in other directions using an appropriate value of Poisson's ratio and other soils parameters obtained from full scale load testing. This aspect he said, was as yet imperfectly covered and was probably the main reason for the gap in theory linking pressuremeter results with real foundation behaviour. Use of the method was based on semi-empirical formulae which needed expert interpretation he said.

Mr M.T. Mitchell (Waikato Technical Institute) said that he would like to have seen something about field instrumentation - pore pressure measuring equipment, inclometers etc., in the Taylors' paper.
Mr D.K. Taylor replied that field instrumentation was normally justifiable only for embankments and cuttings - not for foundations.

Mr I.D. MacGregor (M.O.W. Hamilton) said that he was surprised to see in Table 1 of the Taylors' paper that thin walled tubes were rated as "good" and the triple tube as only "fair". He had recently recovered loose sands using a triple tube (4" core) and mud where thin-walled tubes had been unsuccessful.

Dr J.G. Hawley (Soil Bureau, D.S.I.R.) closed the discussion with a vote of thanks to the speakers.

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# SITE INVESTIGATION AND UNDERGROUND CONSTRUCTION

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## Part 1. INTRODUCTION

1.1 Geomechanics to me is "the study of the physical behaviour of all forms of earth, ranging from loess to rock, under changing stress environments", adapted from a definition of rock mechanics by D.U. Deere of the University of Illinois*. I am speaking in this paper for the two engineering disciplines of design and construction. We start from the point that most site information contained in contract documents in New Zealand is insufficient to enable the designer to produce a satisfactory design, and too scanty to permit the contractor to make a proper estimate of cost. Many N.Z. contracts therefore ignore the science of geomechanics entirely and the contractor accepts the risks of imposing structures upon an environment of which he knows little more than a spade could tell him. It is a scandal, moreover, that many contracts contain the legal reservation that the "site information is not guaranteed and the principal takes no responsibility for its accuracy". The site information and borings ought to be signed facts and given as much care as the legal drafting of the contract.

### 1.2 The Construction Man

No symposium touching on foundation engineering would be complete without expression of the point of view of the contractor or "construction man". The contractor is the forgotten man in the contract - the man who takes all the risks and the blame, but equally worthy of praise along with the designer who creates a work of art and the owner who invests money in an idea. Contractors are inarticulate before a learned society, though not at the bottom of a muddy trench, and they have little protection against the words of a contract except the right to arbitration, provided the contract grants that right. And a request for arbitration is often a declaration of war.

### 1.3 Risks and Warnings

The contractor takes the primary risks in any project, risks that range from the safety of his men and the public to the safety of adjacent buildings. Many of these risks can be insured against, and the contractor is required to indemnify the owner against all claims and to take out an "all risks" policy covering structure, materials and plant. The paramount factor in assessing risk is the quality of the site investigation. The first approach to any scheme by the contractor is by reading the contract documents, which will include drawings, specification, general conditions of contract, and perhaps some unwarranted site information such as bore log data or geological reports. If general conditions of contract NZSS 623 (1964) are used he will find the following clause 2.1.2:

*("Rock mechanics is the study of the physical behaviour of rock under
( changing stress environments," quoted by J.P. Blakeley (1967), The Scope
( of Rock Mechanics, N.Z. Engineering, 22, p.75.

"The tenderer shall inspect and examine the site and its surroundings and shall satisfy himself ----- as to the nature of the ground and sub-soil (so far as is practicable and from data supplied by the Principal) the form and nature of the Site -----". Fortunately successful tenderers under the mantle of NZSS 623 have some redress under clause 8.2: "If ----- the contractor shall encounter physical conditions ----- or obstructions ----- which could not have been reasonably foreseen by an experienced contractor -----" extra payment may be claimed by giving notice in writing, establishing the case, and satisfying the Engineer. The contract documents normally contain enough warnings to put the contractor on guard before he even sees the site.

1.4 Adequate Site Investigation

A geomechanical investigation of site and materials is necessary to promote good design and skilful construction. This is a development of one theme in a report by a working party for the Economic Development Committee for Civil Engineering entitled "Contracting in Civil Engineering since Banwell" (1968 U.K.),* section 2.2 at p.3: "We cannot therefore emphasise too strongly that adequate site and soil investigations are an essential prerequisite not only to good design, but also to the efficient and economical execution of the works". This leads me to the conclusion that all civil engineering and structural foundation work over a certain value should be based upon sufficient site investigation, warranted by an expert as to the facts. I dislike the present trend in contract documents of producing results of borings limited in number or position or depth and qualified by a complete disclaimer by the owner of any responsibility for the test results or for any opinions advanced. Professional men should take full responsibility for the extent and scope of the site investigations and for the facts and opinions expressed. If they are wrong why should the contractor have either to spend a large sum on site investigation before he gets the contract, or have to meet additional cost on foundations because he was misled by an "unwarranted" report? I am well aware that difficult legal problems arise when vital information in the contract documents is guaranteed, and that such problems are unloaded upon the contractor by saying that the test results are "not warranted". But the consequences of the present policy are unfair to the contractor and often expensive to the owner, who in the long run either pays more or gets a less satisfactory job or suffers unnecessary delays. To sum up, proper site investigation without disclaimer of responsibility would lead to fairer tendering, less risk-taking, lower prices, better work, and fewer delays.

1.5 Unexpected Conditions

No underground site investigation can ever be perfect, because that would mean exposing every square foot of foundation area before one could even guess at latent defects. The contract documents should recognise that unexpected conditions may be encountered and should provide ready and flexible means of correcting or overcoming such conditions. It may even be necessary for work to stop while the changes are documented, any alterations made to the design, and the owner and engineer notified. Claims should be made for extra work at once, or as soon as completed, rather than be left till the end of the contract when the facts are buried or forgotten. Prompt liaison is required between contractor and engineer for confirmation of unexpected conditions, agreement on fact, and settlement of appropriate extra payments. The more thorough the site investigation, the fewer the design changes and the fewer

*(Much reliance has been placed by the author upon this report, which he considers to be a landmark in the contracting field.

the arguments over extras or variations. Proper initial investigation puts owner, designer, and contractor all on the same basis, knowing the facts as far as they can reasonably be ascertained by sampling. It is right and proper that the owner should then pay for "unexpected conditions" because, if the true situation had been known and covered by the specifications, the bids would have reflected the cost of any additional work required to meet such conditions.

## Part 2. N.Z. FOUNDATION CONDITIONS

2.1 The New Zealand terrain ranges from hills to plain to pumice to swamp; the rivers are generally steep eroding mountain torrents; the soil types are numerous; and the land is exposed to earthquakes, volcanic activity, slips, and faulting. It is no wonder that the country produces most types of foundation hazard and it could be described as a foundation engineer's paradise or a contractor's nightmare. The designing engineer has been forced to diversify, to produce a wide range of designs in seeking economic answers to foundation problems. Land is scarce in settled areas, so owners have found it necessary to build larger and more specialised structures on difficult building sites. Increasing loads have sometimes demanded unusual or less conventional types of foundation.

### 2.2 Site Investigation

Changing conditions have forced site investigators to develop new techniques in drilling, soil sampling, and interpreting geological features to provide data required by the engineer, the designer, and ultimately by the tenderers and the contractor. The engineer's instructions to the site investigator will have regard to

(a) The owner's special requirements and the possible types of foundation;

(b) The structure loadings and their method of transmission;

(c) The probable construction methods and their consequences (e.g. vibration, dewatering, noise, adjoining structures);

(d) The end effects or consequences of completion of the contract upon the country or the structure or on adjoining structures;

(e) The need to obtain enough information to produce the most efficient design and the lowest satisfactory tender.

### 2.3 The Economics of Site Investigation

The first problem of the engineer, after the owner has decided to build, is to decide how much money is to be spent on site investigation and who should carry out this work. It is not easy to generalise, other than to say that each case depends on its merits. If a simple structure is to rest upon solid rock, or upon a shingle plain with no possibility of erosion, no site investigation is called for. If the foundation problems are apparent, as in permafrost or a peat swamp or a loess hillside, it may be necessary to spend 5% to 10% of the estimate upon investigation. This is the cost-benefit analysis situation, and only experience and precedent can guide the engineer. He must avoid the situation where the designer uses unnecessarily high safety factors because of uncertain foundation stresses, and also the risk of high tenders, expensive variations, or unwelcome construction difficulties and delays. Included in the cost of investigation is that of interpreting the results. This requires expertise which has to be paid for. Not everyone is capable of looking at an X-Ray photograph and deciding whether to operate.

### 2.31 Illustration

The importance of expert interpretation was highlighted recently during the construction of a N.Z. hydro-electric scheme. The site investigation was very thorough. Many thousands of dollars were spent on drilling, using expensive equipment and skilled drillers, but there was no expert available to inspect and interpret the bore log data which for once had been procured. The warnings inherent in the bore log data, so expensively obtained, were completely overlooked and went unheeded. The owner was ill prepared for the delays and for the increased capital costs of the extensive grouting programme required to salvage one part of the scheme. An expert interpretation of the site investigation information might have led to abandonment of one design in favour of another.

### 2.4 Other Aspects

Site investigation should be made as early as possible and have regard to seasonal and to weather conditions. The designer should follow up his design with site inspections during foundation construction. The engineer should make available to all tenderers the site information together with any expert interpretation of the results. I repeat that I find the disclaimer unwelcome in contract documents, and consider that if the site investigation is good enough to design on, it should be considered as fact which a tenderer could trust to tender on. The contractor has few foundations upon which he can rely: the elements and the site conditions conspire against him; he is at the mercy of labour and shipping and supplies; the specifications are stacked against him if difficulties arise; and on top of all these he is told that the site information is not guaranteed and to go and check it all for himself. For too long has the contractor been at the mercy of the engineer and of harsh specifications in foundation work, and it is time this was rectified.

## Part 3. FOUNDATION CONSTRUCTION - CHRISTCHURCH

### 3.1 Peculiar Conditions

Christchurch was built upon a swamp. It is not surprising, therefore, that foundation work in Christchurch has its peculiar problems, which are a legacy of the process of formation of the Canterbury Plain by out-wash from mountain torrents during and following successive glaciations. These problems are all associated with water, and can be classified under four headings:

(a) High ground water levels;

(b) Water-bearing aquifers of sandy gravels and running sand, with layers of impermeable material;

(c) Swampy and peaty areas;

(d) Underground streams.

Underlying the whole of the flat area of Christchurch is a vast artesian basin, which extends in a coastal strip north of Christchurch roughly to the Ashley River, but disappears where it meets the Port Hills to the south. The high ground water levels through Christchurch are indicative of poor drainage and lie in the range from 4 ft. to 8 ft. below ground level.

The swamp and peat areas were heavily vegetated in recent times, some right up to settlement during last century. The swamp areas contain layers or lenses of peat material, highly compressible, scattered throughout the sandy gravels. The underground streams flowing through Christchurch pose serious dewatering problems. The peculiar conditions throughout the urban and industrial area of the city lead to particular difficulties for the designer and for the foundation contractor. Due to recent experiences of settlement and subsidence, and of damage to adjoining buildings, some designers are now restricting excavation for foundations to a level above that of ground water. Up to the present time, all larger buildings in Christchurch have had basements below ground level. One large multi-storied structure has a double basement, but there are indications that future buildings will be founded at ground level.

### 3.2 Types of Foundation

Buildings in Christchurch rest on or are tied to piled, raft or strip footing foundations. Driven and bulbed piles are becoming more common, but most of the larger structures are founded upon driven precast piles. With such a range of types in use in the city, combined with all sorts of underground water problems for the actual constructors, all the principles of site investigation, foundation design, and evaluation of construction methods are called into use and question. One example will be discussed in section 3.3.

### 3.3 Car Park Basement

The site faced a busy city street, bordered on rear and one side by older masonry buildings, and by a new structure on the other side. These three adjoining buildings were on strip footings. The brief for a car park basement raised the problems of how to dewater the site and of how to support the structure. One test bore showed water level to be 8 ft. below ground, and shingle with fine sand to depth. There was no information known as to ground conditions beneath the adjoining buildings, but it was assumed that the sand and shingle continued under them. There was no guarantee that lenses* of peaty organic material did not exist under one or more of the adjoining buildings, and there was no reasonable way of finding this out.

Dewatering commenced using a 6" diameter wellpoint in the centre of the site, about 6 ft. below the bottom of the basement raft. After initial removal by pumping of a quantity of sand, an inverted filter was placed around the suction head of the pipe, further pumping leaving a perfectly dry site to depth. (Refer to Figure 1). Excavation and underpinning the adjoining structures commenced, and each end of the excavation was supported, or rather retained, by rail piles and timber sheeting. The pump worked continuously for over three months, but fortunately little or no settlement occurred beneath the adjoining buildings.

Both the designer and the contractor can consider themselves fortunate in this case. Had the dewatering drawdown curve intercepted an isolated lens of organic material next door, and thus caused settlement of the overlying building, the contractor would have been held responsible,

*(Lens used here refers to an isolated band or layer or pocket of material
(underground, not of continuous or extensive nature.

under the terms of his contract, for the subsequent structural damage to the adjoining building.

The point that arises is that if the designing engineer cannot guarantee that there are no lenses of organic material in adjoining areas, which might be dewatered and settle so as to cause structural damage to adjoining buildings, then his design should allow for or avoid this possibility. No responsible designer would ever give such a guarantee without first making a complete sub-surface investigation of all critical areas. The information so obtained, probably at great cost if within a built-up industrial area, should be warranted and included in the contract documents. The alternative in this case would be to specify some type of cutoff wall, such as steel sheathing or some proprietary design which might well constitute part of the final structure. Dewatering would then be confined to the protected area, with appropriate safeguards against heave and piping. Excavation and construction would then proceed in the normal way inside the protected area, as shown in Figure 2.

It is submitted that this alternative, though expensive, is preferable to specifying that the contractor must accept all risks of damage to adjoining buildings if he uses pumps or any other form of dewatering equipment during a contract.

## Part 4. SEWERAGE AND DRAINAGE CONSTRUCTION

### 4.1 Definitions

The present comments are confined to aspects of the underground construction of sewerage systems and of stormwater drainage in the city of Christchurch, though the underlying principles will have general application to wet areas or where ground water is close to the surface. Stormwater disposal is a branch of land drainage, and similar techniques are used for the construction of sewers and of stormwater drains underground. The distinction is made that sewage refers to the matter conveyed in sewers, and sewerage is the wider term covering the drainage system of pipes and equipment used to drain sewage away, by pumping or gravity, to collecting ponds or to an outfall.

### 4.2 Data Required

The basic requirements of a contractor tendering for or performing any drainage work are to find out:

(a) What depth is the water level below ground?

(b) What support will the trench require?

(c) What would be the effects of dewatering?

(d) What type of material exists in each stratum or layer down to 3 ft. below the invert level of the pipe?

The site information usually provided will consist of a series of bore logs, generally stopped off at invert level or when running sand is encountered. These bore logs will not be warranted, and the contractor will be asked to satisfy himself as to the subsurface conditions. In Christchurch the contractor will almost certainly be told, within the conditions of contract:

"A sheet showing details of the strata as shown by borings along the route is available for the information of tenderers, but this information is not guaranteed and the owner will take no responsibility for its accuracy. Tenderers are advised to make their own investigations along the route."

## 4.3 Bore Logs

In the past many bore logs went only to the point where water was encountered or where the material fell in faster than it could be removed. This usually meant that the contractor was deprived of the very information that mattered to him most - what was the material like at the bottom of the trench? "Informed" bore logs may now be obtained fairly readily to average trench depths, by hand augering in a bentonite solution. This gives an excellent indication of the nature of most material and of its depth. The spacing of bores along one trench should not exceed 300 ft. in uniform going, with closer spacing in variable country.

A series of bore logs taken down to depth provides the designer with design data, and also the contractor with construction information upon which he can tender without taking crippling risks. It may be apparent whether timbering will be necessary or whether open cut trenching is possible. An indication will be given by the bore logs of the feasibility of wellpointing, and perhaps of its effectiveness. To take a simple example, if an impervious pug layer near invert level has a seam of running sand above it, wellpointing would not stop sand movement, but the wellpoints could penetrate the pug seam and reduce water pressure underneath and so reduce heave and sand boils in the trench bottom. The bore logs may provide the designer with some indication of the likelihood of settlement if extensive wellpointing is adopted in construction. The adoption of efficient and extensive bore hole testing offers advantages to both designer and constructor over the trial and error method so often adopted in the past.

## 4.4 Wellpointing and Settlement

It was mentioned earlier that wellpointing may lead to drying out the country to the extent that settlement occurs. This consideration matters first of all to the designer, because it may control what he designs or how he designs it. In the case of a pipeline, dewatering is relatively short-term, and (provided sand is not being removed from the trench) the draw-down curve does not extend very far. As shown in Figure 3, the presence of a layer of peat might lead to settlement from dewatering, but the damage would be minor and confined to a narrow strip each side of the trench.

Take the case of the construction of a pumping station, using the caisson method. Here the process of dewatering is more localised than in the pipelaying situation, but it lasts for a much longer time. Once again, any information available from bore logs upon water levels and the nature of the country is useful in determining the design, e.g. the friction on the caisson. But the over-riding consideration now in Christchurch, when the use of dewatering equipment is contemplated is the effect of pumping upon buildings situated in the region of the drawdown curve. In the long term situation such a drawdown curve could extend out radially for a distance of 200 to 300 ft. from the caisson. In one instance, experience showed that the site investigation should have extended over the entire area contained within a circle of 100 yards radius centred upon the caisson. The case is illustrated in Figure 4. The house was within 300 ft. of the caisson and was built on semi-consolidated clay filling some 10 ft. deep. This filling overlay a former swamp which had turned into a lens of peat.

Dewatering dried out the peat, which sank along with the fill and the dwelling, and the dwelling broke its back. The contractor was held liable.

### 4.5 Responsibility

Now that these problems and the causes are better known and understood it seems to me quite wrong that the contract documents should place all responsibility for damage upon the contractor, who has in turn either to insure against damage or load his price to cover damage. Surely it is the responsibility of the principal, who speaks through his engineer-designer, to ensure that the construction of his pumping station should not damage any nearby dwellings. Adequate site investigation by the engineer would in most cases reveal the presence of compressible layers likely to sink where dried out.

It may be that the design of the structure can be changed so that dewatering is not necessary for sinking or construction, or that a cutoff wall can be added and paid for if this is required. Perhaps the contract documents themselves should draw attention to the risk of settlement or should guarantee that the country is stable. After the experts have done all they can to define the work and the limiting conditions, the contractor should be able to tender with minimal risk and maximum confidence, at the lowest price.

## Part 5. BRIDGE FOUNDATIONS

### 5.1 Historical

Site investigation techniques for pier foundations have always left much to be desired. My own experience with bridge foundations began as a construction engineer, but in recent years I have acted as consultant to contractors engaged on bridge work.

Before 1950 the normal bridge foundations consisted of driven or bored reinforced concrete piles, or else of footings or cylinders founded in solid material.

The standard method of investigation for bridge foundations (still used, and known with unconscious humour as "driving a test pile") was to drive a 70 lb. railway rail with a light rig and to instruct someone not otherwise occupied that day to borrow a carpenter's 3 ft. rule and to measure the set from each blow or series of 10 blows. From this accurate information and the approximate weight of the monkey the engineer performed a series of calculations applying higher mathematics and the Engineering News Formula* with its built-in factor of safety of six. For the larger bridges additional investigations were carried out: (a) examination of the Roman numerals carved with a chisel on the tops of existing timber piles; (b) inquiries into the lengths of previous piles driven with a saw; (c) analysis of the material in the country deposited from wash bores. From this wealth of information the engineer predicted with accuracy the length, size, and number of steel, timber, or concrete piles in the new bridge. In most cases the foundation piles proved exactly what was required to support the new bridge, but in rare instances the engineer was proved wrong, as we shall see in section 5.2.

* ( $Q = \frac{2WH}{S+C}$ Refer to Terzaghi and Peck, Soil Mechanics in Engineering Practice, 2nd ed., p.230, for explanation and critical comment of this formula.

## 5.2 The Tongariro River Bridge At Turangi.

I quote from Mr C.W.O. Turner's book "Contracts and Contract Administration" (1966) at p.101:

"In one contract, the Tongariro River Bridge at Turangi, the plans provided for concrete piles, and the contractor expended much time, energy, and money in attempting to drive them, using one technique after another. It was eventually agreed that steel piles could be substituted because of the "impossibility" of driving the concrete piles envisaged by the contract. The contractor claimed "impossibility of performance" and payment quantum meruit for his expenses. The dispute went to arbitration and the arbitrator, after taking advice on legal principles, denied the claim, stating that the "impossibility" of driving the concrete piles did not vitiate or frustrate the contract, and the contractor was not entitled to special consideration in respect of the costs that he had incurred in attempting to drive these piles, other than payment as provided in the schedule of rates."

Clearly here, the owner and the engineer were hiding behind the legal rule that the impossibility of driving the concrete piles did not frustrate the contract, with the result that the contractor carried all the loss. The steel piles were driven, but one might doubt whether the bridge was well and truly founded even if it did uphold the law. There are many instances of bridges on steel piles where floods and erosion have revealed that some piles did not reach the required penetration depth due to splitting, rolling up of the flanges, or deflection upwards in hockey stick fashion. "Out of sight, out of mind" is not a reliable dictum when driving piles.

## 5.3 The Alexandra Bridge over the Clutha

An interesting case arose with the driving of very heavy steel H piles for pier B of the bridge at Alexandra over the Clutha River. The specification called for the driving of a particular type of H pile to refusal, but did not limit or specify the drop or the weight of the hammer. The country was typical Central Otago alluvial schist gravel to depth, and appeared to offer little resistance to driving. A 3 ton hammer with a 5 ft. drop produced a regular set, and the only unusual factor was that one pile rotated through $90^{\circ}$ while being driven. After much discussion between contractor and engineer as to the soundness of the driven piles, a test hole alongside a pile revealed that after 15 ft. of penetration the flanges of the H pile had rolled up in sardine tin style. Complete excavation of the pier foundation showed that 60% of the forty piles driven in the pier had failed similarly.

## 5.4 Inspection and Control

Following these disturbing revelations, some improvements were made in the control of the driving of steel piles. The weight of the drop hammer, the maximum height of drop, and the maximum penetration, were all specified or limited. Sight tubes were built into steel piles as standard practice. These were significant advances, and far ahead of the site investigation techniques of the same period.

## 5.5 Prestressed Concrete Piles

The use of prestressed concrete piles is a recent development in piling in New Zealand. The inherent problem with these piles arises from the reversal of stress which occurs when they are driven through hard ground into softer ground. A homely illustration of this phenomenon is given in the removal of a cork from a bottle, by hitting the bottom of the bottle longitudinally while holding the bottle itself and preventing more than limited movement. Under certain driving conditions, normal prestressed piles may fail, as a result of reversal of stress, somewhere in the bottom third of the length. One means of overcoming this type of failure is to increase the prestress during casting to the order of 800 to 1000 lbs per sq. in. Another means is to limit the penetration to say ¼ in. per blow when driving through soft ground. It is obvious that a thorough and detailed borehole investigation is warranted, with penetrometer readings every foot or so, in order to recognise the danger spots when driving prestressed piles.

## 5.6 Crowding

Another problem that may occur while driving a group of piles, of any type, arises from the effect of "crowding". The crowding effect varies with the void ratio,* which is the ratio of that portion of the volume of the soil not occupied by mineral grains, to the total volume of the solid substance. Put more loosely, the crowding effect is a function of the compressibility of the country. The driving of the first piles in the group tightens up the country so much that lateral movement of the soil particles becomes hindered. The driving of successive piles continues until refusal, at the stage when lateral movement of the displaced particles is no longer possible. A site investigator taking note of the degree of consolidation in the country being examined should be able to advise whether "crowding" is likely to be a problem if groups of piles are to be driven there.

The solution to crowding is to "pre-dig" the holes and drive to final penetration and end bearing only. This can be achieved with proprietary rotary or grab equipment. Holes may be driven in the wet or the dry, sometimes using bentonitic drilling mud.

## 5.7 Illustration

A recent example with a "pre-dug" foundation is given of a railway bridge in the South Island. The tender documents contained the logs of two wash bores done several years previously. This unwarranted information showed the subgrade conditions to consist of 10 to 15 ft. of boulders 6 in. diameter or greater, overlying sand and small gravel, of less than 1 in. size, to depth. The penetrometer results were grouped as being in excess of a certain figure anywhere in the hole. During the tender period the tenderer who was later awarded the contract looked over the site and assured himself that the top boulders exceeded 6 in. diameter. He assumed that the data on the underlying material was correct. The design called for 24 in. diameter octagonal piles, up to 50 ft. long, grouped in a standard railway raked configuration. The piles were to be driven through pre-drilled holes.

The tenderer considered this problem and decided that, once the top layer was excavated to 15 ft. and backfilled with sand, the pile bores could be done with a jet assisted air lift, and he tendered accordingly. After being awarded the contract, he became suspicious of the subgrade conditions and drilled three holes. One reached the depth of 40 ft. and showed the underlying material to be coarse material and not fine gravel. The other two bores reached 10 ft. only. The contractor was forced to redesign the foundations, and he arrived at the following solution.

*(Terzaghi and Peck: Soil Mechanics in Engineering Practice, 2nd ed. p.24.

Holes 4 ft. in diameter were dug with a hammer grab to within 6 in. of depth. Precast piles 42 ins. in diameter weighing 35 tons were placed in these holes and driven the final 6 ins. using a 15 ton hammer. The changed design provided an excellent foundation. A thorough site investigation before original design might have enabled the owner to foresee the probable difficulties and to allow for them in the design concept.

## Part 6. TUNNELS - THE CASE OF MANAPOURI

### 6.1 The Contractual Issue

The Manapouri Tailrace Tunnel contract signed in 1963 provides an interesting example of insufficient site information. The following remarks have no bearing on the wider issue of Manapouri and the environment, but are confined to technical matters on tunnelling. The tunnel section of the Manapouri Power Project is no longer a matter of dispute, so that the site investigations and the construction problems can be discussed to provide lessons for both engineers and tenderers. The contractural issue in the Manapouri contract was the application of Article 9 dealing with "Latent Conditions" and the request for the granting of a "Change Order". The events leading up to the determination of this application provide the lessons to be learned.

### 6.2 Historical

In May 1961 a contract was let to sink an exploratory shaft 14 ft. 8 ins. diameter by 700 ft. deep in the proposed underground power house area at West Arm of Lake Manapouri, together with supplementary diamond core drilling. There had been a geological reconnaissance late in 1959, followed by geological mapping and topographical surveys during the first four months of 1960. Air photographs covering a strip one half mile wide were used for locating positions and plotting observations. Auxiliary surveys were also carried out by levelling and traverse parties. The shaft was completed in December 1961. There were no developments in the following year, but in March 1963 the shaft was dewatered and tenders were called for a low level tailrace tunnel discharging to Deep Cove, with the lowest point 136 ft. below sea level, and for an access road to Deep Cove and for the outfall work. One can only conclude from the calling of tenders, that with the sinking of the shaft at West Arm, the log of diamond drill cores in the same area, and the data from the geological surveys, all concerned with the project felt that there was sufficient information to plan and design in detail a fully lined tailrace tunnel 5-3/4 miles long with 90% of the tunnel invert below sea level. In other words, the investigational work and the reports by the geologists and the engineers, combined with estimates and cost-benefit analyses, must have satisfied the Executive that this scheme was both feasible and economic. The government's estimate is thought to have been about 9 million pounds. The completion dates specified meant that tenderers had of necessity to plan for an average daily advance of about 40 ft.

### 6.3 Performance

The original contract, signed on 16th July 1963, provided for 27,200 ft. of tunnel excavation in 31 months, say 775 working days, at about 35 ft. per day. The contractor ran rapidly into the following difficulties and unexpected conditions:

(a) Rock materially fractured and of different structures;

(b) Large quantities of water, under great pressure;

(c) The need for much more grouting;

(d) Much more steel support than anticipated.

On 20th February 1966 the date for completion of tunnel excavation was advanced from 1st September 1966 to 15th January 1968. Some indication of the difficulties experienced by the contractor is given by progress, from February to July 1966 inclusive, at an average advance of only 7.2 ft. per day. The contractor claimed latent conditions under Article 9.0 of the contract.

To sum up, there was enormous overrun of cost, and the contract period was exceeded by several years.

## 6.4 The Reason

The reason for the poor performance by the contractor was that insufficient site investigation had been carried out by the principal to disclose the hidden difficulties of the job. It has to be admitted that the terrain above the tunnel was mountainous and bush-clad. It was precipitous and extremely difficult of access. Even so, in a contract of such magnitude it would not have been impossible or unreasonable to perform a traverse along the centre line, using seismic equipment brought in by helicopter, to estimate the depth of certain strata and the degree of fracturing of bedrock. It would have been possible to put down one diamond drill hole at each predicted danger spot (Refer to Figure 5) to ascertain the jointing pattern of the rock and to gain some appreciation of water pressure and of inflow at tunnel level.

Reference was made in section 6.2 to the geological work done early in 1960. A report entitled "Geological Investigations, Lake Manapouri Power Project", dated 15th June 1960, was prepared for the principal by Dr Keith R. Miles a geological consultant from Australia, who headed a team of four geologists and seven (later eleven) field assistants in the field from 23rd January 1960 till some time in April 1960. This report was not released to tenderers on the tailrace contract, though some general geological notes based upon the field work were placed upon one of the specification drawings. As it so happened, some of the conclusions in the geological report were not borne out by the experiences of the contractor, particularly in relation to drilling conditions and to pressure water.

## 6.5 What Might Have Been

The lessons of the past need to be evaluated and appreciated for the sake of the future. It is our obligation as professional men to weigh up what were claimed to be the omissions in this contract and to consider what might have happened if a more comprehensive geophysical investigation had been performed along the line of the proposed tunnel, as suggested in section 6.4 above. The findings of major importance might have been:

(a) Owner, engineer, and tenderers would have been better informed of the difficulties to be overcome;

(b) A more realistic estimate of tunnel cost and progress would have been reached;

(c) The fundamental decision would have been to drive the tunnel at lake level to Deep Cove and to place the power station at the outfall end.

It is understood that the alternative design mentioned in (c) was considered, and there is no doubt now that this would have provided a much cheaper scheme.

### 6.6 Other Factors in the Contract

The tenderers had the short period of approximately three months in which to inform themselves of the physical conditions below the surface along the tunnel line. Surely it was unreasonable to expect tenderers to do this in three months when a strong competent team of geologists were unable to do so after spending several months on a detailed geological survey of the site. The contract documents contained the usual disclaimer that the owner does not guarantee the conditions portrayed by the documents, but fortunately for the contractor a latent conditions clause was incorporated in the contract, which paved the way for later adjustments to overcome conditions totally unknown to all parties at the date of signing the contract.

### 6.7 The Lesson

The lesson of the Manapouri Tunnel is that in large schemes (and this was in the ten to twenty million pound class) adequate site investigation should be carried out, on a scale comprehensive enough to determine the most appropriate design, and to reduce the doubts and risks of the work to an acceptable level. The design consideration is paramount, because the saving in cost will usually be many times the cost of the comprehensive site investigation. But fair play to all tenderers also justifies payment by the owner to ascertain the physical facts, and to guarantee that these facts are correct. It is adding insult to injury for the owner to disclaim any responsibility for the correctness of the tender information and then not to give a reasonable time for the tenderer to make his own investigations at his own cost. Finally, fact information used by the designer should not be withheld from the contractor when it might or could be of assistance to him in preparing the tender or in the actual performance of the contract. It would be proper for reservations to be made as to the reliability or scope or application of particular information, but this is not submitted as an argument supporting the usual disclaimer.

## Part 7. CONCLUSIONS

### 7.1 The Facts

One single thread has been woven into the fabric of this paper - the need to know the facts. The facts are the foundation material for the designer, for without them he cannot design properly. Without knowing the facts the contractor is not in a position to make a proper tender, so he is sometimes tempted to guess what the ground conditions will be like when he opens up the site. If he guesses right the contractor goes on to fame and fortune. If he guesses wrong his fate is bankruptcy.

### 7.2 Fault

The problems that have concerned me in recent years about engineering contracts are wider than the subject of this symposium, but they are nevertheless closely related to my subject of site investigation and underground construction. It will be helpful to set down these questions for discussion within the limits of this symposium.

(a) Are there any general faults in our system of carrying out engineering works?

(b) If so where do these faults lie?

(c) What are their consequences?

(d) What can be done to correct them?

## 7.3 Site Information

An array of evidence has been presented showing the inadequacy, (sometimes the non-existence), of site information for large and small contracts throughout the land. It has been shown that the consequences have been unfortunate for the contractor and often unwelcome to the owner. The results have invariably been unfair to one or more of the parties to the contract.

The naive contractor might assume on reading a contract document that the site information contained therein had been specially obtained to guide him in preparing a tender. Nothing could be further from the truth, as he will find out in due course after having signed a statement that he has made his own assessment of the ground and the work and the risks. The contractor will also discover in the fullness of time that the site information is not warranted, is not guaranteed, may well be wrong, is probably unfair, but that he has no redress.

This unwarranted site information has in fact been obtained by the engineer at the owner's cost, to enable the designer to produce what he considers to be the most suitable design for the work. That is, upon the basis of facts which the legal document states are not guaranteed, the owner's technical expert has prepared plans and specifications for what he thinks is economic, sound, efficient and appropriate work which will meet the owner's special requirements. How can the designer produce his best work if the facts about the site may not be relied upon? In practice the designer does rely upon his site information, because first of all he has not got anything better, and, more important, because he suffers no penalty if the information is wrong, or faulty, or misleading. But the contractor is in a different position. He has to make good, no matter how misleading the site information, or he will not be paid. The law says that the contractor enters the contract with his eyes open, takes all the risks, and must take all the consequences. If the site investigator had to take all the consequences of his work, the investigation would be done properly.

The present system is almost a benefit system in favour of the owner or principal and his advisers, because no one will accept any responsibility except the contractor, who is forced to tender on the owner's terms and to accept such crumbs of payment as those terms legally permit. The contractor's only certain right under all contract documents is the right he has not to tender.

## 7.4 The Need for Change

The case is submitted that all site investigations should be carried out competently, and in a thorough and extensive manner, and guaranteed, so that the appropriate scheme can be designed, tendered for, and constructed. The owner should pay for the site investigation. He started this operation, designed the project, and should lay a proper foundation for other people to estimate upon. The time is not far away when the engineer will have to advise the principal that before plans and specifications can be prepared an extensive site investigation must be made, and that it would be negligent not to carry out such investigations. Experience abroad has shown that owners are rapidly becoming more "claims conscious", and the production of sound reliable facts, upon which all parties can rely, is the only effective protection against claims of professional negligence.

## 7.5 Interpretation

Accurate underground or sub-soil data can be obtained only by the correct interpretation of a complete site investigation. It has been shown earlier how much the engineer needs both the site information and its professional interpretation to design the most economic and functional scheme. He has particularly to bear in mind how the foundations are going to be constructed, because this may determine the very nature, type and position of the structure. As an ancillary matter he should also consider the benefits to be obtained by employing construction companies with specific skills, techniques, or plant, and by taking advantage of local knowledge.

## 7.6 Summary

Foundation work shares a common denominator with all construction work - it depends for its beginning and its end upon facts and teamwork. To carry the analogy a little further, the commencement of all schemes is the desire of men to find out the facts, to interpret them, and to build from them. All parties, from the owner to the subcontractors, are members of the team. Some of you will be surprised to learn how many parties and skills contribute to an engineering work, be it a building or a bridge, and I list some of them here:

owner or principal
financier
engineer
site investigator
interpreter
designer
tenderer
contractor
many subcontractors.

The best results are obtained when all members of the team are fully informed, when the facts are discovered, explained, guaranteed, and made available to all parties. No one really profits from the unfairness of the present system which seems to concentrate more upon protection from legal responsibility than upon discovering and building upon a foundation of reliable fact. The fundamental plea in this paper is for proper investigation of the land and what is beneath it, of the soil and the ground-water and the obstacles underground, so that suitable foundations may be designed and constructed to harmonise with our environment and without wasting our limited resources.

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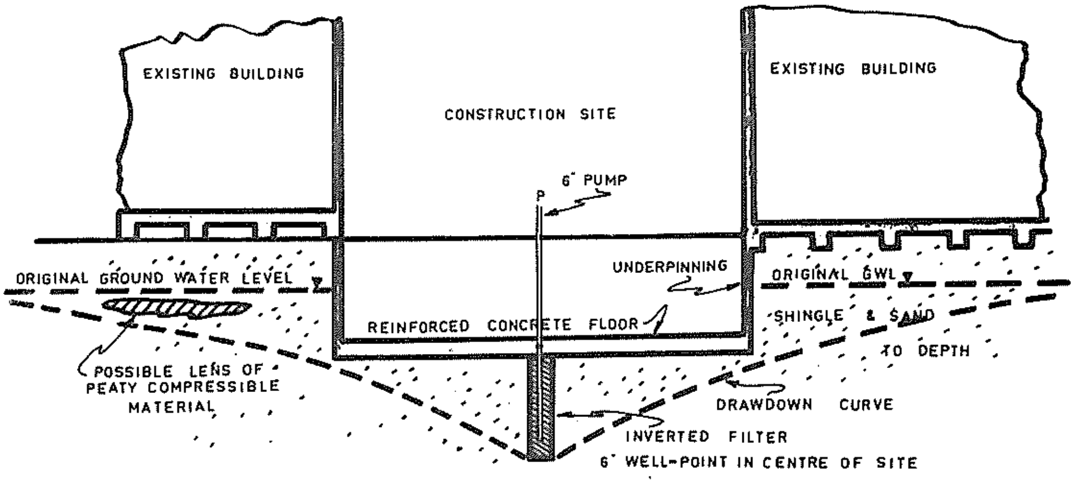


FIG 1: **CAR PARK BASEMENT, SHOWING METHOD OF DEWATERING**

REFER TO SECTION 3·3 not to scale

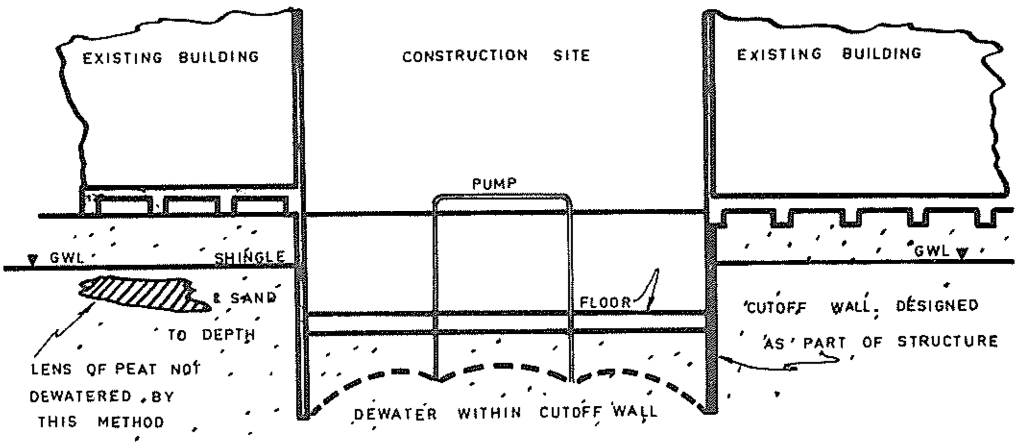


FIG 2: **CAR PARK BASEMENT, SHOWING ALTERNATIVE METHOD OF DEWATERING WITHIN CUT OFF WALL**

REFER TO SECTION 3·3 not to scale

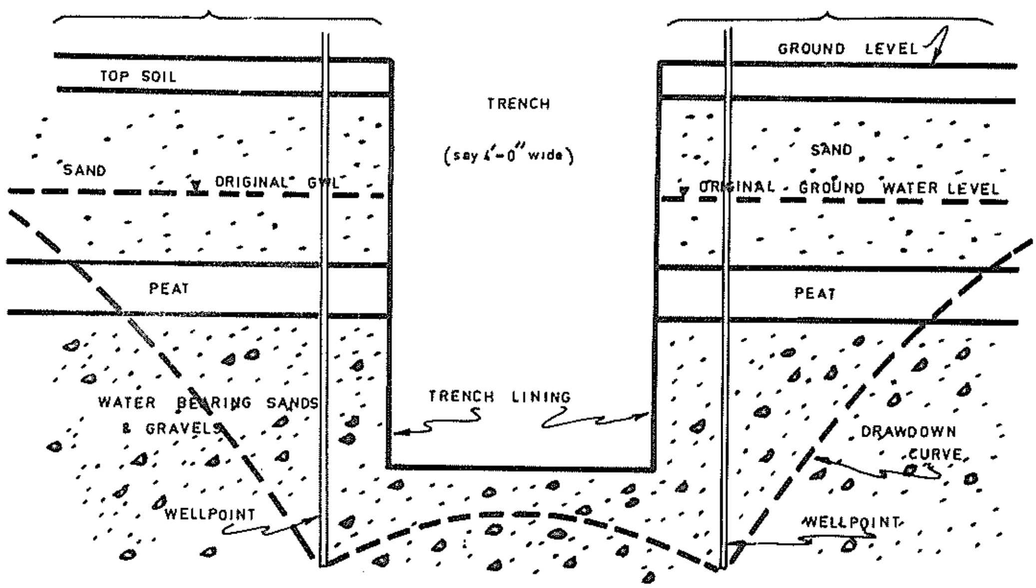


FIG 3: **DEWATERING SEWER TRENCH**

THE NARROW ADJOINING STRIPS ARE EXPOSED TO SETTLEMENT IF THE PEAT SINKS

REFER TO SECTION 4-4 not to scale

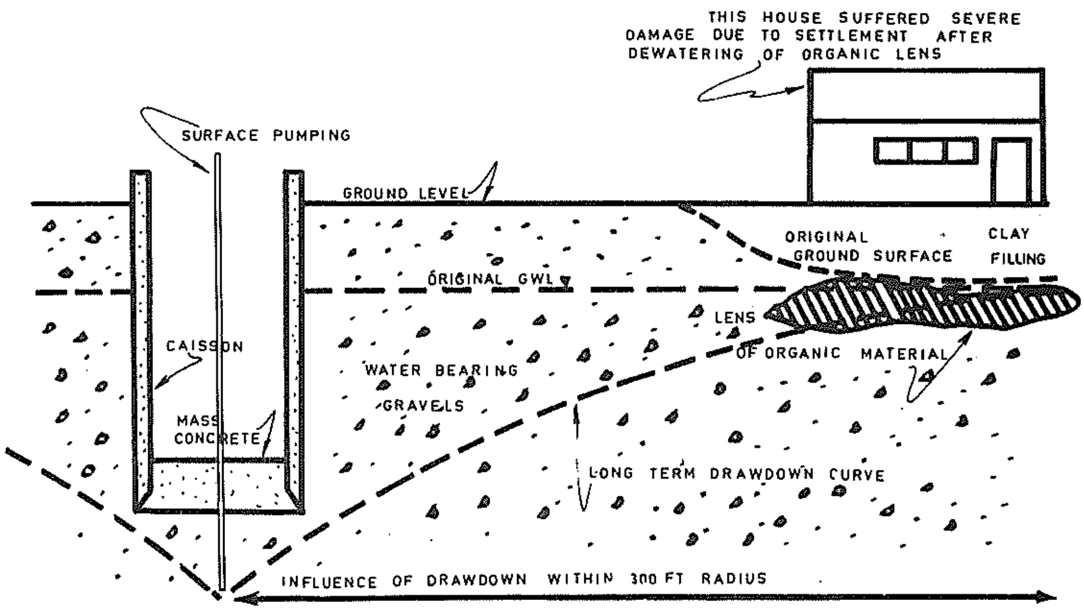


FIG 4: **DEWATERING PUMPING STATION**

CONTINUOUS PUMPING AT SITE MAY CAUSE SETTLEMENT WITHIN 300 FT

REFER TO SECTION 4-4 not to scale

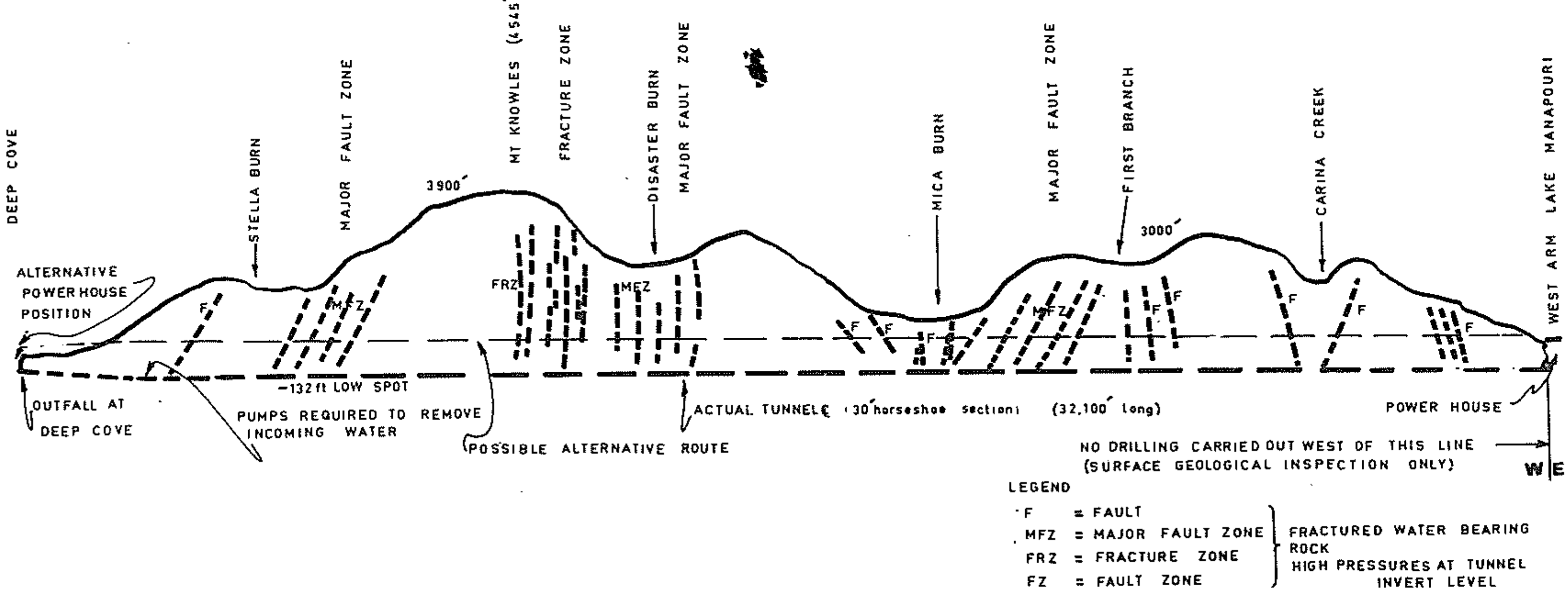


FIG 5: **ELEVATION ALONG LINE OF TUNNEL**

(90% OF TUNNEL INVERT IS BELOW SEA LEVEL)

**MANAPOURI POWER PROJECT**

**TAILRACE TUNNEL CONTRACT 4545-100**

AFTER TIPTON & KALMBACH, INC., DENVER
(original plan dated 22 July 1966)

not to scale

REFER TO SECTION 6

# TREATMENT OF VARIATIONS
# FROM ASSUMED CONDITIONS

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## INTRODUCTION

During his planning work, the design engineer develops conclusions on the probable subsurface materials upon or within which he has to obtain support for the building. Information from site inspections, from the design and performance of adjacent structures and from exploratory drilling is assessed to make finite decisions on the type of foundations to be used. The supporting capacity of the proposed bearing stratum is either assumed or derived from field and laboratory studies and consideration is given to the long term settlement performance of the building.

At best, the subsurface conditions are determined by a series of exploratory borings but, all too often, they are assumed by surface inspection or the results of some nearby investigation.

When the site is opened up for foundation construction, subsurface conditions are inevitably found to depart from the assumptions made during design. Adjustment or modification to the foundations must then be made for the real situation which has been encountered.

## SUBSURFACE INVESTIGATION

Information on subsurface conditions is estimated or determined by a range of methods.

1. In the most elementary form, no investigation is made at all. The classic example of this is normal residential housing where the designer almost invariably accepts that the site will be suitable for a house with conventional or standard foundations. Even during the construction phase, soil problems such as inadequate bearing capacity, differential settlement characteristics and ground stability are quite often not recognised or dealt with. Approximately one-sixth of the total number of soil mechanics jobs handled by the author's firm in the past 12 months relate to problems with residential dwellings either during or after construction.

2. As a slight improvement on the first case, the designer may determine the type of foundations used on some adjacent building and copy this.

3. Subsurface data of a form can be derived by engaging a drilling contractor to bore holes and provide a boring log. Such work commonly includes Raymond penetration tests and these results are then applied in various ways to determine foundation type and size. This form of study places undue responsibility on the drilling contractor; he is skilled and experienced to drill holes but generally does not have any formal training on the identification or classification of soils and rock.

4. Complete and proper subsurface investigation comprises identification of the soils and rock, including measurement of their relevant engineering properties, to the full depth of influence of the proposed building.

## CONSTRUCTION PROBLEMS

Construction problems generally arise from variations in the assumed soil and rock conditions, from excessive ground water flow or as some effect of the construction process. These can occur individually or together and usually in accordance with Flannigan's Law ("if anything can go wrong it will"). The treatment of each and every variation must be considered on the basis of sound engineering judgement combined with consideration of both the time and the additional costs involved.

### Variations in Assumed Soil and Rock Conditions

In preparing a finite design for tendering and construction, the engineer has to specify the depth which he requires foundations to extend. It is normal that he also specifies the nature of the proposed bearing stratum and requires some form of visual identification or field proving test.

The most common variations arise from discontinuity of the bearing stratum or from erratic changes in its surface level. Typical of this problem are alluvium infilled valley floors which have been built up by stream or river deposition. Although exploratory drilling may establish the existence of a suitable bearing stratum beneath the site, its full extent cannot be determined until the foundations are opened up. Unanticipated changes in the surface level of the stratum can generally be dealt with by deepening the foundations, be they shallow footings or piles, or by excavating the unsuitable overburden and replacing with compacted fill or mass concrete. Discontinuity of the stratum can be a severe problem, especially if it requires additional exploratory drilling. The difficulty is generally not recognised until after the building contractor has fabricated his formwork and reinforcing for the foundations; the cost of these items, together with delays to the contractor must weigh heavily in the decisions which are taken. Having performed additional exploratory work to find and prove an alternative bearing stratum, the solutions will comprise:-

1. excavate to new level and replace with compacted filling or mass concrete;

or 2. deepen foundations;

or 3. redesign foundations to a completely different solution.

The third alternative may be necessary where an initial design on shallow spread footings is found to be impracticable. However it may also be necessary for pile foundations where the specified construction method cannot penetrate deep enough to reach the alternative bearing stratum.

A further common source of variation occurs where the originally selected bearing stratum appears or is proven to be weaker than was assumed in design. Such a conclusion may be drawn from visual inspection by an experienced supervisor or from the results of some form of field proving test. Again the solutions comprise excavation and replacement, deepen foundations or redesign foundations.

Where structural settlement is a factor in redesigning foundations, the engineer must be careful in considering the apparently simple solution of only increasing the bearing area of foundations. Although a widened footing presents a lower intensity of applied pressure, it does have a deeper range of influence. Thus for a stratum of compressible material which has significant depth, it is normal to find in analyses that settlements will be of comparable order.

Natural hazards such as the crush zone of an earthquake fault present a special problem where discovered in construction. These zones have significant width and are usually steeply dipping. The communited rock is invariably weaker and more weathered than the surrounding parent material. Alternative solutions to this problem comprise:-

1. bridge over the fault zone with a specially designed foundation system;

2. extend pile foundations through the fault zone;

3. increase the bearing area of the foundation.

Man-made filling and unanticipated buried structures are often missed in exploratory work but encountered during construction. Obviously the occurrence of such features may be anticipated from local knowledge or from a review of historic records but their full extent, depth and form cannot be verified until the site is opened up. The reclaimed waterfront areas of Auckland and Wellington Cities are especially difficult in this regard. Both were developed over a long period of time by stage construction. Each new area of reclamation was bounded by a perimeter wall and/or a breastwork structure. With further stages of reclamation, these walls and breastworks have been left in place and filled over.

Almost invariably, old deposits of man-made filling comprise material with inconsistent quality and standard of compaction. Only under light and exceptional loading conditions could such fill be used to provide support for foundations. Normally, it is necessary to remove the fill and either replace it or deepen the foundations.

The treatment for buried structures will depend upon the influence they may have on the new building. Rarely it may be possible to re-use the foundations but more often they must be either cut back or completely removed. Where the latter solution is necessary the new foundations have to be designed to extend beyond the zone of disturbance which is caused.

<u>Groundwater</u>

During exploratory drilling, it is common practice to flush out the bore holes and allow a static water level to develop in the shaft. This data is then presented as the "ground water level". At best this practice is approximate and at worst it is exceedingly dangerous. It may be the result of a perched upper table, of a depressed lower table measured in a cased shaft or of a water table which is subject to artesian pressure. Aside from its use in analyses, accurate data on groundwater is essential to the contractor for the basement and foundation construction.

Where groundwater is encountered during construction at levels higher than anticipated it can commonly be handled by pumping. However unless dealt with promptly, the water can cause damage to proposed bearing surfaces, resulting in the need to deepen excavations and refill with mass concrete.

In the construction of bored piles or caisson foundations, groundwater inflow causes collapse of the shaft side walls. It then becomes necessary to case the shaft with steel or concrete liners. Where high capacity end bearing is required, the shaft and casing should, if possible, extend to soil conditions and depth sufficient that the casing is sealed into a near impermeable layer. The shaft can then be dewatered, cleaned out and inspected prior to concreting. The need for sealing off the groundwater with casing at times necessitates that piles be deepened beyond the tentatively selected founding level.

The building of structures which extend below ground level as depressed basements or tanks requires special consideration by both designer and builder. Any cellular type of construction must be checked to verify that it is not subject to the risk of flotation either during construction or after completion. Accurate details of groundwater levels and their maximum range of variation due to seasonal and tidal causes are essential to the designer and builder. However, in addition, they must give consideration to the introduction of water from an external source such as a burst or leaking pipe. Where the subsurface materials (soil or rock) have low permeability, the introduction of water from an external source can rapidly induce a flotation condition in a depressed structure.

Effects of Construction

Soil mechanics and foundation engineering continues to undergo increasing sophistication as an applied science. At all times however, the structural designer must be supplied with basic data on the soil and rock conditions as these will be during and after foundation construction. Thus although it is possible to obtain almost completely undisturbed samples and to carry out comprehensive and accurate laboratory testing, the derived parameters may be entirely unrealistic if the materials of the proposed bearing stratum are damaged during construction.

Materials derived from volcanic ash showers are typical of those which can be damaged during construction. The natural deposits of ash have medium strength and are almost invariably overconsolidated. They are however extremely sensitive (suffer a significant decrease in strength on remoulding) and do undergo volumetric change when subjected to minor variations in water content. Thus it is essential that the designer transmits this information to the builder and specifies suitable and adequate methods of construction. The most practicable method of dealing with this problem is to minimise the extent of excavation carried out ahead of foundation construction. Final trimming to design founding level must be made with equipment that does not impose load on the exposed subgrade and a protective layer such as polythene or blinding concrete placed immediately. Bearing materials which have undergone remoulding or volumetric change due to wetting or drying must be excavated out and replaced.

The installation of pile foundations by percussion methods can induce ground surface vibrations ranging from minor and inconsequential to severe and damaging on adjacent structures. In specifying a particular foundation type, the design engineer is morally if not legally bound to ensure that the system is buildable. Where surface vibration from pile driving does cause concern, it can be reduced by predrilling through the upper soils at each pile location.

Occasionally, problems arise in construction due to inexperience or lack of equipment on the part of the building contractor or his foundation subcontractor. Assuming these are not sufficient to warrant the ultimate fate of dismissal, the design engineer and his soils adviser must be able and prepared to offer guidance. The basic concept should be that the design group and the contractor form a team which provides the Principal with a finished product which is sound and satisfactory.

Current design concepts in New Zealand tend to leave the problem of support for excavations entirely in the hands of the contractor. Thus specifications and drawings commonly do little more than acknowledge the need for such support and demand the contractor to engage a Registered Engineer for design. Unfortunately, the amount of factual information available for this design is usually insufficient. The contractor must then perform additional exploratory work or, as is more often the case, make an arbitrary decision based upon previous experience. His support design could be more rational if the soils exploration work were to provide him with the necessary basic parameters of the materials which are to be retained.

## CONCLUSIONS

Wherever possible, exploration of subsurface conditions should determine all relevant engineering properties of the materials influenced by and adjacent to the proposed development. It must be performed not only to provide data for the foundations of the structure but also for the contractor to determine, in detail, how he is going to construct those foundations.

Variations in assumed conditions of soil, rock or groundwater which arise during construction must be dealt with by full consideration of the actual conditions encountered together with the time delays and additional costs involved.

# THE NEED FOR A CRYSTAL BALL AS AN AID TO FOUNDATION ENGINEERING

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## INTRODUCTION

The designer of any structure is usually confronted with a foundation design. He must consider what measurements and tests (if any) should be made, what predictions are possible from his measurements and what might happen to the structure as a result of the predicted foundation behaviour. Whether or not he has a foundation problem often depends on the size of his investigation budget and the availability of a crystal ball to sort out the meagre data that might be revealed in tests. A problem situation may be apparent only when the hole is excavated for a foundation and it is found to bear practically no resemblance to the picture inferred from the test data except at the few places where the bore holes were taken. A further series of tests is then needed and maybe a corrective design of the foundation. Both of these could involve increased cost which could have been a lot less or at least anticipated with better site investigation knowledge.

A wide range of tests is available from which the foundation engineer may select any or all, depending on the importance of the job or the size of the investigation budget. Simple tests for physical characteristics and density can be made, or inferred strength tests such as penetration, or CBR tests. More complex tests may be needed such as plate bearing tests, piling tests, dynamic tests, or laboratory tests for classification shear and consolidation etc. Each of these is used to determine some property of the soil from which some prediction of its later behaviour can be made under different conditions. Terzaghi, in 1957 said "we must assign to each type of material of the earth numerical values which make it impossible to mistake it for another one with significantly different engineering properties" - and this statement related to his work done in 1916. The predicted behaviour of the ground material (such as settlement under load, ultimate strength, permeability or response to vibration) is then used in the "safe" design of the structure, but there remains a big question - how near reality are the predictions?

For the structural design the specification defines the required test strength and quality of steel and concrete and generally check tests are taken to ensure compliance with the specification. The material under the structure is very much part of the structure because what this material does very greatly affects the structure above it. The approach to the soil component of the structure is often quite different. The foundation material often does not get the same analytical attention and checking as is given to the material forming other parts of the structure.

Testing techniques and our understanding of soil behaviour are improving as research advances, but frequently reliance is placed on what is often termed "judgement and experience". There is no doubt this helps greatly and the greater the experience the better is the crystal ball. The use of this crystal ball must continue until the time that foundation investigations can be made accurately and predictions made with consistent reliability. Techniques and tools are available but the limitation is often financial, particularly relating to any follow up work to check predictions. It is because of such a lack of feed-back information or check tests that foundations are generally designed conservatively and consequently expensively.

## Tests and Measurements

The basis of any engineering design is the measurement of the properties of the materials used in the structure. This is done to quantify variables and obtain relationships between variables. In foundation materials this process is intrinsically more difficult because of the extremely great range of variation that occurs with soil materials. However we must measure something that will reveal some useful properties of the material (e.g. strength) or its response to some future loading (e.g. settlement under load or behaviour under vibration). Although not completely exhaustive some of the things that can be measured are:-

- physical classification - grading - density and index properties
- strength as shear strength for immediate or long term conditions
- bearing capacity
- stress-strain relationships
- elastic moduli, which can be static or dynamic values of "elastic" modulus (E) or shear modulus(G)
- settlement parameters, $c_v$ coefficient of consolidation,<br>$C_c$ compression index
- settlement variables such as void ratio (e) and moisture content
- permeability coefficients
- indirect tests such as penetration tests calibrated against some other property which is required
- pressure of soil or pore water
- pile testing as a direct loading or as dynamic test

## Reliability of Measurements

Any tests or measurements must be made to the best possible accuracy under the prevailing conditions. The prevailing conditions may be the controlled situation in a laboratory or the broadly based situation in a field test.

Field tests are generally less precise than laboratory tests but may give a better basis for prediction because of the elimination of disturbance variables.

Laboratory tests using small samples (in which lack of disturbance cannot be guaranteed), can create accurately controlled conditions and will provide quite precise results from the samples under test. However, there is always the question of how well do the laboratory conditions simulate the real situation and how typical are the samples of the field strata?

The indirect type of test, such as a penetrometer test, has some merit in that it can detect uniformity or otherwise in the site materials and relate this to other parameters such as strength or bearing capacity.

A combination of field and laboratory tests would generally provide the best picture of existing conditions under a proposed foundation.

Six fairly well defined steps can be taken in a site investigation

1. Site inspection and geological reconnaissance.
2. Preliminary trial borings; possibly use geophysical methods, and build up a picture of the geological structure of the site.
3. Take samples and observe ground water conditions.
4. Decide what deep soundings or borings will be needed.
5. From samples and location decide which ones need laboratory testing.
6. From test results and requirements of the design use theoretical soil mechanics to predict the behaviour of the material.

In assessing the reliability of any test the limitations of the test or of the data indicated by it, should be fully understood and appreciated. When extending the test results into the realms of prediction several questions must be carefully considered:

(1) How representative is the sample for laboratory tests?

(2) Do laboratory conditions simulate the field conditions?

and in any field test:

(3) How accurate is the measurement of the property being sought?

(4) How consistent are the results in a series of tests?

(5) Does the test data reveal the properties of all the critical strata

<u>Why Test or Measure?</u>

This may seem a somewhat naive question, but it is only by trying to answer it that we can obtain some indication of the sort of tests to be done and measurements to be taken on any foundation material.

The first simple answer is the classification of the material, to help in understanding its properties. For example, sands, silts and clays have quite different behaviour under static or dynamic loads. The values of density, strength, void ratio, moisture content, and moduli indicate what the material is like in the "as is" situation.

The predictive part of the investigation is to find what the material will be like or how it will behave under some other future loading condition. In other words we wish to know the "as will be" condition of the foundation materials.

Tests are usually made at the investigation stage to provide information from which to predict the soil behaviour and so provide for a rational design of a foundation to the structure. There should be an intermediate stage of testing to check whether the predicted design assumptions are being realised, e.g. pore pressure or settlement measurements can be made. All too often this is not done and hence the initial design must be made over-conservative to cover the factor of ignorance.

Continuing to a much later stage check tests should be made, but all too frequently these are dismissed as of academic interest only and not carried out because no one is prepared to pay for them. Long term settlement measurements come into this category, but usually the only time these tests are done is when something has obviously gone wrong in the foundation design and ominous cracks are appearing in the structure. In most cases there is little feed-back information and the designer of a foundation never knows just how good or how much underdesigned the foundation really was. He is usually told very quickly if it proves to be inadequate.

## Predictions

The need for predictable results relates to the proper functioning of a structure (e.g. an earth dam must be watertight under the action of the water pressure on it), or to its general safe behaviour. For example a building will usually settle to some extent but this is of little consequence if there is no differential settlement causing cracking and sewers continue to flow from the building and not towards it.

The behaviour of a large foundation is based on a projection of the results from tests on a few very small samples. It should not therefore be surprising if the large hole that is dug doesn't always appear the way we thought it should. On the slender evidence usually available predictions should be very cautiously made; nevertheless we must still be bold enough to make them otherwise there is nothing on which to base a carefully calculated analytical foundation design.

From measured data, and particularly from laboratory test results, any predictions can have a high degree of calculated accuracy, but experience indicates that such predictions can still be very wide of the mark. Actual settlement and settlement time, for example, can be anywhere between 50% and 200% of the prediction. This is not a very good scoring rate although the calculations themselves may be very accurate. There must be a proper evaluation of the reliability of the predictions of theoretical soil mechanics.

Predictions sometimes agree with later measurements (if these are taken) but more often they do not agree. The later measurements may merely indicate that the foundation design was within safe limits, i.e. the predicted results have not been exceeded. Why this difference between predicted and actual behaviour?

Several factors are involved:

(1) The initial test data could be completely inadequate as a basis for the predictions

(2) The wrong data was measured initially or maybe the foundation design was changed to cope with unforeseen conditions.

(3) Foundation material is too variable to provide any average parameters for design purposes.

Because of the formidable gaps in the knowledge of foundation material behaviour and the inadequacy of most of our crystal balls, designs of foundations must be kept within very conservative limits. The material under the structure is as much a part of the structure as any other material used above the ground, but we know far less about it. The design of many structures is carefully and precisely calculated but an almost blind faith is placed in the adequacy of a practically unknown material underneath it.

In general troubles experienced in foundation design come into four categories:

(i) Engineering failures due to no proper site investigation

(ii) Boring sampling and laboratory testing without a full understanding of the site geology

(iii) With proper investigation and full understanding of the site, failure to apply the theoretical principles of soil mechanics in the correct way

(iv) With everything done and applied correctly there can still be cases of exceptional difficulty, where conditions encountered could not have been foreseen.

Self Correcting Feed Back

If a foundation designer can find out more about the later behaviour of a structure it would be possible to check and refine his assumptions and predictions. Check tests of strength under imposed loading and long term settlement under load could both be of very great value in creating better and more economical foundation designs.

To ensure that the correct initial data is available the selection of the things to be tested or measured must be carefully considered. This implies a three stage investigation, as originally proposed and used by Terzaghi but seldom followed completely:

(1) get available information about the site to find out what sort of problems there are likely to be

(2) take a few preliminary borings to formulate a working hypothesis and decide what additional data is needed

(3) Plan a programme for a complete step by step investigation and observation to check or modify the working hypothesis. The planned programme may have to be altered as work progresses so that no effort is wasted and the right information is obtained to solve the right problem.

For too long the foundation designer has been forced by economic considerations to use a crystal ball. Little is spent on a foundation investigation and consequently more must be spent on a conservative foundation design.

Two simple examples may help to illustrate the two extremes of foundation problems.

Firstly, an inadequately investigated site and the consequences, and secondly a carefully investigated site and the results from feed-back information.

RETAINING WALL FAILURE

The failure of a crib-log wall exemplifies the consequences of having no site investigation and of disregarding the principles of soil mechanics relating to the changing properties of soil with moisture and time. The investigation had to be done after the failure in order to find economical corrective measures.

The wall, which is about 2¼ chains long, had been constructed by a property subdivider on a sidling cut road on a loess hillside. It had been accepted by the local authority as satisfactory and in fact stayed this way for several years. Suddenly after rain, about a chain of the wall some 13 ft high collapsed by overturning just above the base. Preliminary inspection showed that the loess fill behind the wall was not adequately drained and gradual saturation had reduced the shear strength and increased the pressure on the wall until the soil load exceeded the ultimate strength of the wall.

It was found that the base of the wall was built on fill material (loess), but as the failure had been by overturning and the base had not moved it appeared reasonable to reinstate the wall with improved drainage conditions. However, as the base was known to be on fill material no real guarantee could be given for the complete and ultimate safety of the wall, without total excavation of this material. This would have meant a higher wall at much greater cost. The remedial steps taken were:

(1) a solid concrete base was poured to spread the load;

(2) all surface water on the roadside channels was prevented from passing near the fill behind the wall;

(3) free draining material was used for backfilling the wall.

This was a safe and economical reinstatement of the wall in its original position and size. Its cost was about $9,000 in 1962.

## Delayed Consequences

About a year after the reconstruction, following heavy rain, the whole wall moved and cracks appeared in the tar sealed road above the wall. This probably aggravated the condition by letting more water in. The displacements at various points were carefully measured over a period of several years. During the second year of measurements the movements gradually reduced as the ground dried up in the summer. Observations were taken at five points and the one plotted in Figure 1 is the point giving maximum movement. Very little movement occurred in 1964 but after heavy rain in the winter of 1965 further considerable movement developed and by this time the wall had dropped about 14 inches with a similar outward movement, but retained its proper shape and batter. The movements were outwards and downwards as shown in Figure 2.

## Further Investigations

It was then obvious that more detailed tests were needed to ascertain the cause of the movements. Properties of the base material and fill were determined from triaxial test samples. Test pits and borings were made to determine the exact depth of the old ground line. From these investigations it was concluded that the movement was occurring near the original ground line. Safety factors were calculated from the test data as follows:

On the slip plane SF = 0.9

On a deep circle failure line SF = 3.36

On a shallow circle line (C = 0) SF = 0.5

On a shallow circle line with test values found for C & Ø SF = 1.0

With these indicated safety factor values it is not surprising that the wall moved. Again the cause was the saturation of the loess resulting in reduction in shear strength of the material.

## Remedies

Corrective measures could have been difficult and expensive. Grouting was considered at a cost of about $3,000, but after further action on diverting surface water and keeping water off the footing of the wall and toe material, the movement gradually slowed.

Some planting was established on the toe material and in the course of time became effective in drying out the toe material and stabilising the ground. This was a very cheap solution.

## Costs

The cost of the investigational work was about $800 and the repair costs on the wall more than ten times this.

Although the completed wall as originally constructed was not a very big job it is apparent in hindsight that considerable saving of repair work and trouble could have been enjoyed if the ground investigation had been made first and not afterwards.

Acknowledgement is made to Royds, Sutherland and McLeay, Consulting Engineers, (who were engaged to report on the initial failure) for permission to use this information.

# MEDICAL CENTRE FOUNDATION

The site for this nine storey building was investigated carefully by Tonkin and Taylor, Consulting Engineers, before a design was started. Five bore holes were made in four of which standard penetration tests were done. Laboratory test samples were taken where possible to establish changes in strata and properties.

The properties established by the investigation enabled the structural designer to provide a suitable foundation which was effectively a partially floating foundation at 13 ft below ground level.

The building contract specification required level points to be established in the foundation and that these should be checked after each floor was poured. This was required primarily to check any movements in relation to adjacent buildings so that these could be maintained at correct level by jacking when needed.

## Levelling Programme

Inside the basement were thirteen primary levelling points and others were established outside where accessible for levelling on jacking points.

The nine floors were poured at the rate of about one per month, but no settlement movements became apparent until three floors had been constructed. This amount of loading would be approximately equal to the weight of soil excavated.

The foundation slab was of cellular construction and as an investigational project more levelling points were established in the lower compartments of slab to see if it was possible to detect any change in level due to deflection of the slab, either between stiffening walls, or as an overall effect of tilting or bending. The positioning of all levelling points was greatly restricted by the partition walls in the basement. Lines of sight had to be carefully planned, before the levelling pins were inserted. These comprised three eighth inch terrier sockets with brass bolts screwed in.

The two series of levels were taken independently, but cross checked and related to two external bench marks. The level used in the investigation is a Zeiss Koni 007 Automatic precise level giving an accuracy of .02 mm, with precise staff graduated in 0.5 cms. The reading accuracy that could be achieved was $\pm$ .02 mm for over 90% of the readings.

The drawing in Figure 3 shows the foundation layout and the location of levelling points, and Figure 4 indicates the changes of level with time. This record is incomplete, but observations are continuing. The precise levelling as described above is being undertaken as a research project at Canterbury University.

Grateful acknowledgement is made to Frederick Sheppard and Partners, Consulting Engineers, Tonkin and Taylor, Consulting Engineers, and G.C. Osborne and Co., Surveyors, for the use of information contained in this report.

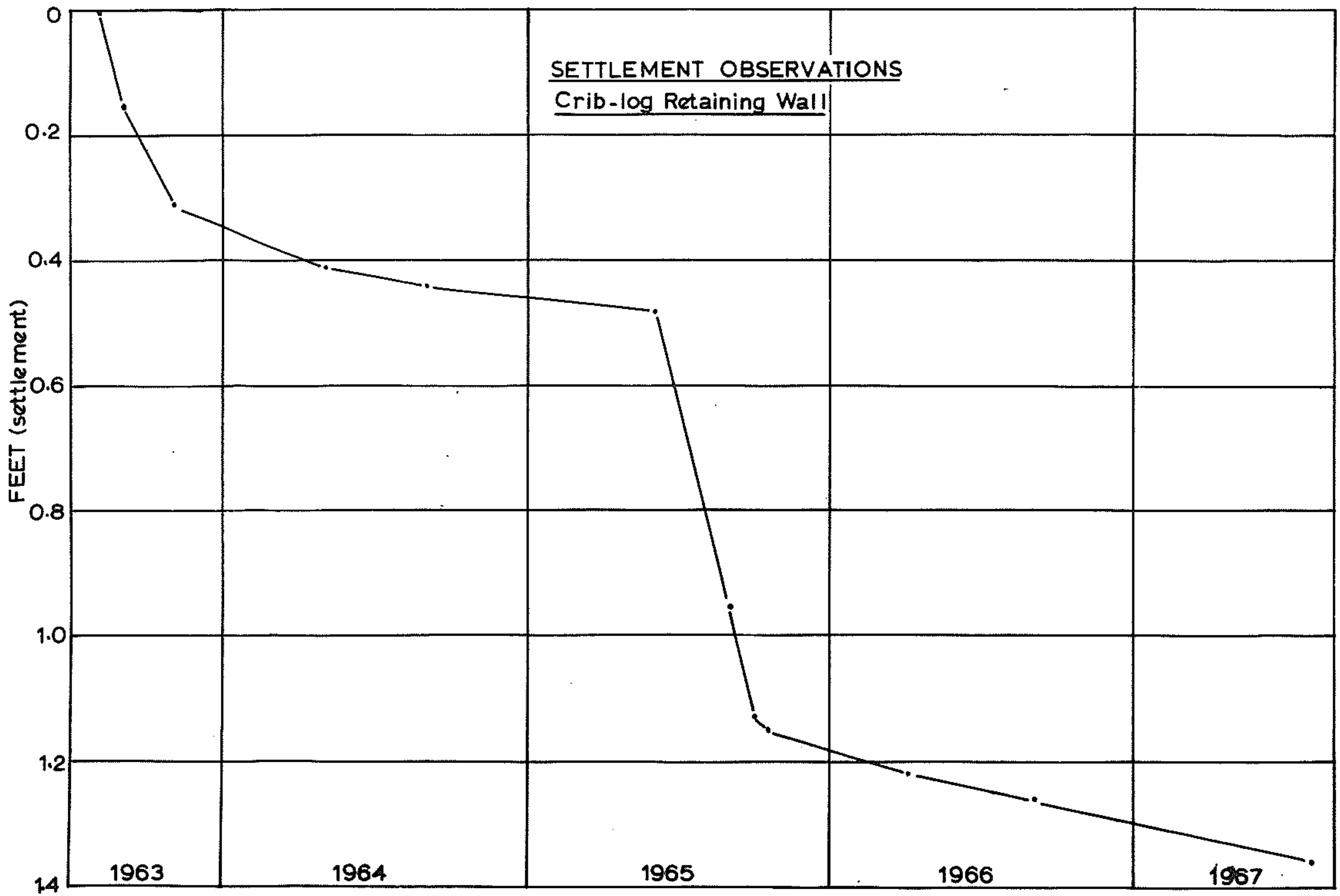
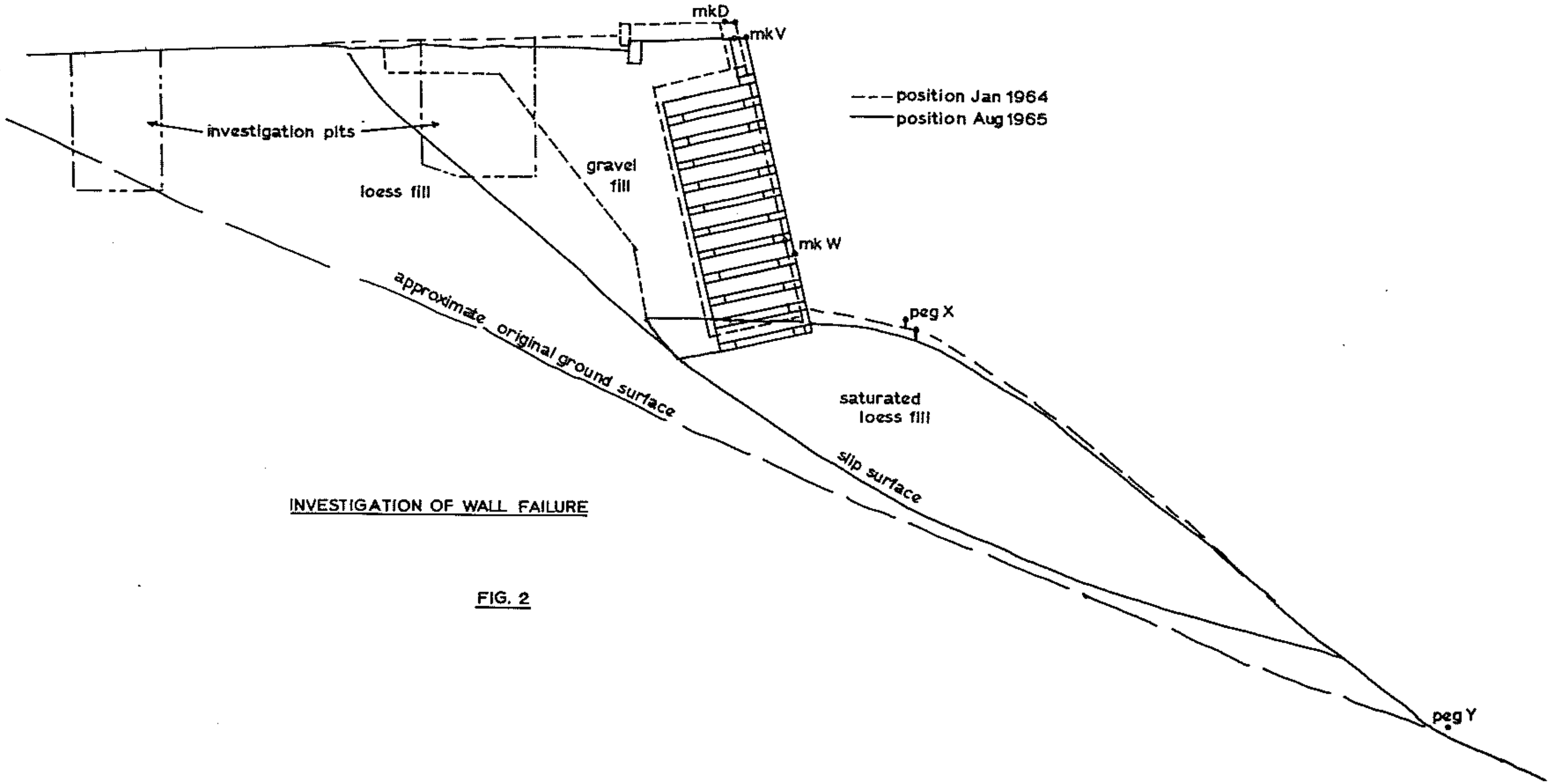
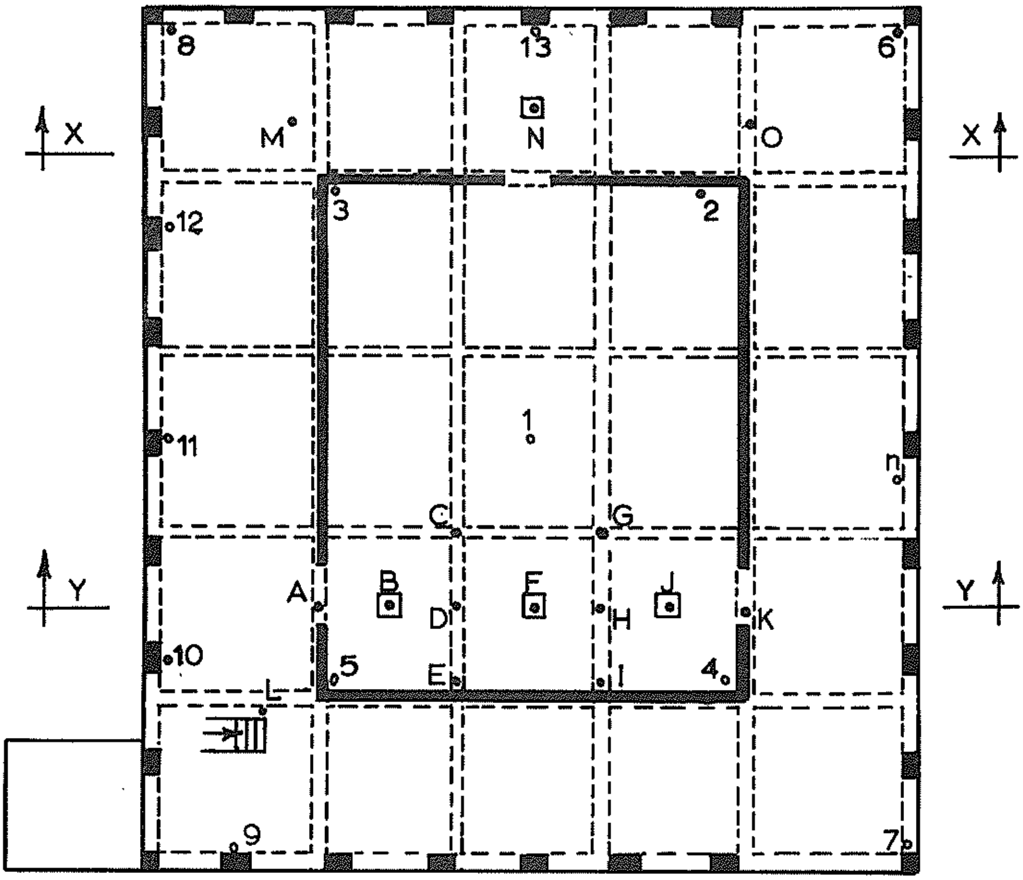

SETTLEMENT OBSERVATIONS
Crib-log Retaining Wall
FEET (settlement)
0
0·2
0·4
0·6
0·8
1·0
1·2
1·4
1963
1964
1965
1966
1967

FIG.1

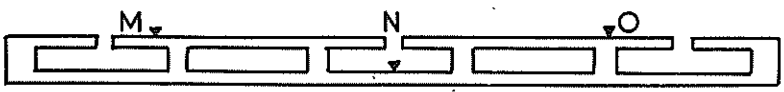



INVESTIGATION OF WALL FAILURE

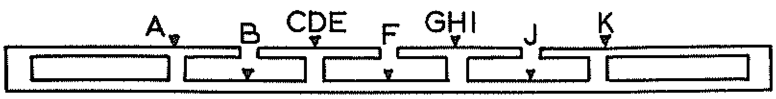
FIG. 2




PLAN




SECTION XX




SECTION YY

FOUNDATION

MEDICAL CENTRE: CHRISTCHURCH HOSPITAL

UNIVERSITY OF OTAGO

FIG N° 3

PRESENTATION and DISCUSSION. SESSION 3.

Chairman: B.C. Hadfield
(Gilberd Hadfield Pile Co.).

An extremely stimulating presentation by Mr Morris got the third session of the symposium off to a good start. Mr Gillespie and Mr Evans presented their papers and a lively discussion followed.

Mr S.R. Robson (Consulting Engineer) commenting on Mr Evans' paper said that there was not normally a sufficient number of soil samples taken to enable a valid coefficient of variation to be calculated. When enough results were available the scatter was likely to be high enough to call for large factors of safety. Experience with concrete testing indicated that sampling and testing techniques themselves could be a source of serious deviation from the mean - quite apart from deviations due to the actual nature of the material.
Mr Evans agreed with this and said that an alternative approach was to look at the extreme values and ask why these were extreme.

Mr Robson commenting on Mr Gillespie's paper said that on the Wellington waterfront, caisson foundations were commonly designed for belled out bases on substantial alluvial beds with N values of 60+. However, it was not uncommon for a few caissons to miss the hard beds. Was there any value, he asked, in forming bells in several reasonably firm layers in such cases and was there any hope of equalising deflections - with those experienced by the caissons based on hard beds.
Mr Gillespie said that it was rare in the Wellington area that at depths of 40 ft. or more the alluvium was weak. He said that, particularly when the caisson was already part way down, the cheapest solution would normally be to carry on deepening the pile. He said that in the Auckland area this might not be the right answer at all but in Wellington properly engineered foundations rarely went beyond 80 ft.
Mr Robson recalled an exceptional instance where 39 out of 40 caissons found good bearing between 20 and 30 ft but the 40th went down 90 ft without finding anything firm. They had had to reinforce the pile cap and span across the gap.

Mr N.S. Luxford (Tonkin and Taylor) said that Mr Morris' figure of 5 - 10% of estimated cost being spent on foundation investigation was very high. His own experience was that ½% of total cost or 1% of structural cost was common for buildings, and 3 - 4% for civil engineering works on difficult sites.
Mr Morris replied that he had sought to make the point that proper site investigations cost a lot of money. He said that he could think of several major engineering projects in which, if 5% of the then estimated cost had been spent on investigations everyone would have been wiser and richer. Some projects would not have been entered into but others that we did not now have would have been built.

Mr J.C. Rutledge (M.O.W. Wellington) asked Mr Morris whether seismic tests taken 3000 ft above the Manapouri tunnel could have been expected to show anything of interest. He said that in a test drive for the Wellington Terrace tunnels for the urban motorway a number of seismic tests had been carried out but these had not given reliable correlation with the geological logging..
Mr Morris replied that he felt that some boreholes should have been put down at Manapouri.

Mr D.K. Taylor (Tonkin and Taylor) agreed with Mr Morris that bore logs and test results should be made available to tenderers and that it was unrealistic to expect all tenderers to obtain this information for themselves. Whether the reasoning and opinions expressed in the site investigations report should be made available was, he said, another matter. The conclusions of such reports were available because they were expressed in a design. If the tenderers' assessment of the bore logs was such that he considered that the design could not be built, then he should not tender. There is, he said, a principle expressed by some contractors that "you first get the job, - then argue". An engineer acting for an owner had to keep up his guard against this approach.
As to guarantees Mr Taylor said that no engineer could be completely sure about everything - he had to make up his mind as to practical courses of action, present them with conviction, and then stick with them until he had good cause to change. Even the so-called 'factual' information in the bore logs and test results had been subjected to interpretation i.e. opinion. He said that in the final issue a guarantee must mean a source of funds to make good the consequences of error. He asked who should, or could, supply that money. The client wittingly supplied some in contingency items, - he may be able to afford more; the contractor had limited assets at risk, and the engineer tailed the field. Any further guarantee could come only from spreading the risk onto the public either via government funds or via private insurance policies. The question then was 'who could obtain the best cover most cheaply.'
There was a danger, he said, that reliance upon insurance might encourage irresponsibility at the personal level. Clients must be firmly told by engineers what investigation was necessary to reduce risks to a reasonable level - all risks could not be removed. Levels of competence varied amongst contractors as they did between design engineers but the contractor must be regarded as being more expert in methods of construction and should bear the main responsibility for that aspect; he must have good ground information for this purpose.

Mr Morris said that, particularly on very big jobs, it was at the earliest stage that the site investigation had to be top notch - before the job went to tender, i.e. before there was any commitment to go ahead with the project at all. (He said with regard to the Manapouri project that had all the facts been known and the degree of difficulty appreciated at an early stage then the scheme would probably never have been entered into). If after investigations had been made there remained a lot of unknowns and risks, but the decision was made nevertheless to go ahead, then the job was not a suitable one for letting out to tender. A better course was to bring owners, engineers and contractors together and form a group with the declared purpose of completing the job in the most sensible and economic manner. Nobody got satisfaction out of going to arbitration or the courts. Court decisions were liable to be based on earlier court decisions - he had had one case referred back to a precedent in 1820, when the whole business of engineering was entirely different.
Insurance Mr Morris maintained, was to cover disasters - things over which one could have no control. Recently insurance companies had started to refuse to give more cover than this. It was bad to rely on insurance he said. There always would be difficult situations where there were unknowns. Sometimes you didn't know the facts until the job had been started, or finished, or finished for some time. In these situations the team must be brought together and the thing negotiated step by step. In this way nobody was left out on a limb - everybody was taking a fair share of the risk.
He agreed with Mr Taylor that the contractor had the right not to tender; recently he had advised people on several big projects to do just this. But this, he said, was a negative approach. The person who made a good appreciation of the job and came to the decision that he would not tender was possibly

the most informed potential tenderer. He was probably the best man who could have been got to carry out the work. The danger was then that the owner would be left with the tenders of a few gamblers.
Very often when things went to arbitration a decision was made in the light of the wording of a particular clause in the Specification - a clause which very often turned out to be a badly drafted one (probably 'lifted' from documents relating to some other contract) i.e. one which did not clearly express the intent of the designers.
Mr Taylor agreed with Mr Morris' remark about eliminating the best contractors. On the 'trick' wording of Specifications he said that they (Tonkin and Taylor) had had Q.C.'s interpret clauses in ways which no engineer would think of.

Mr M.T. Mitchell (Waikato Technical Institute) said that in N. America it was customary to provide in the soils report, detailed descriptions and classifications of every soil sample returned to the laboratory.
Contractors could not then claim (in court) that relevant information had been withheld from the contract documents.
Mr Morris said that he hoped N.Z. would, nevertheless, not go the N. American way where everything was settled in the courts. He advised keeping away from courts and from arbitration, and advocated getting people to work as a team. In difficult jobs, he said, if the legalistic approach were adopted then the engineers, designers and owners could find themselves spending a lot of time and effort in drawing up Specifications so that the main job, that of building the structure did not get the attention it should.
Mr Gillespie endorsed this view, and mentioned a book published by the legal society of a west coast N. American city, entitled "How to sue your consulting engineer". On the subject of teamwork he spoke of two projects in which the design team comprised the design engineer, architect, foundation engineer and contractor. These teams grew as the design developed until they included the person who was going to do the cleaning. This team approach was a good way of getting the best job for the client.

Mr B.W. Buchanan (M.O.W. Wanganui) asked whose responsibility it was to decide whether the soil conditions encountered when the site was opened up were those envisaged by the foundation designer. In general, he said, owners failed to call in the foundation consultant during construction.

Mr Gillespie agreed that the foundation designer should be there when the site was opened up. Any tests at this stage had to be quick ones because holding machines on a site was so expensive. Such tests must take at most a few hours to perform. He thought that a soils report should describe clearly the stratum on which the foundation was designed to rest - in terms of the material type and quality assumed in design calculations.

Mr A.J. England (Murray-North, Hamilton) noted that in para 1.5 Mr Morris had stated that "It is right and proper that the owner should then pay for unexpected conditions". How did he educate his clients to accept this viewpoint?
Mr Morris replied that it was the duty of the engineer to make it plain to the client just how much was known and how much was not known about the site. He recalled an instance where his firm took over a foundation design job on the condition that site investigations could be carried out. These investigations showed 7 ft of peat underlying 5 ft of sediments. They had then told the owner that if he would not agree to meeting the cost of removing the peat then he would have to go to somebody else. Had a shallow footing design been put forward a permit would probably have been issued and the building would probably have stood satisfactorily - until somebody lowered the water table in the area.

Mr W.J.H. Duckworth (N.Z.R., Wanganui) took issue with Mr Morris' view that a designer should stand by his bore log information and assist the contractor with advice on what equipment to use. He said that N.Z.R. try to avoid doing this because if the contractor's equipment is poor, his 'know-how' limited, or his management unsatisfactory the job could fall down and he would shelter behind the excuse "you told me how to do the job".

Mr E.R. Chave (M.O.W., Dunedin) said that M.O.W. had in the past attempted to contractually absolve themselves from responsibility for the adequacy or accuracy of site investigation data. In recent years, however, they had generally accepted responsibility for the data but not for any interpretation of it made by the contractor.
Many contractors, he said, didn't bother to take advantage of what site information was available before tendering, and quoted three jobs in the 0.5 - 1 million dollar bracket in which only one or two out of a large field of tenderers had bothered to call and look at, or collect copies of, investigation data. Some were content with a verbal answer on the telephone to questions, such as "was it hard or easy driving" - others didn't even do that.

Mr Morris agreed that contractors were guilty in this respect. He said that the attitude of first getting the job and then worrying about the site was common.

Mr J.D. Shaw (Longyear N.Z. Ltd., Auckland) asked Mr Morris what he considered was the best type of contract for subsurface construction - lump sum, day rates or what.
Mr Morris replied that in a contract for foundations only, lump sum tenderers would probably put in high tenders because if they lost money on it they had no other part of the job in which they could hope to make up a loss. On such jobs contractors usually made a lot of money or lost a lot; they seldom came near to breaking even. The law, he said, took the attitude that a man signed a contract with his eyes open, knowing that site information was unwarranted and that there were risks involved. The position now was, he said, that many well-established contractors were not prepared to enter into the cut and dried lump sum contracts - so one got left with a few gamblers. He advocated the negotiated contract - cost plus perhaps. He mentioned NZ 623 with its latent condition clause which he said, if applied to the letter, was not a bad clause. Work proceeded according to the contract until the contractor considered that conditions had changed. He then advised the engineer of this and explained what he proposed to do, stating what it would cost. The difficulty lay in getting the owner and/or the engineer to agree that there was a latent condition. However, the specification stated that despite disputes work must proceed - and normally by the time the issue was settled, the job had been finished.

Mr J.C. Rutledge (M.O.W., Wellington) commenting on Mr Evans' paper asked whether, in addition to the factors of safety for slip surfaces for the crib wall, any bearing capacity calculations for the wall foundation had been done. He said that if formulae or methods, such as those associated with the names of B. Hansen or Meyerhof were used, with their appropriate corrections for individual load, eccentricity of load, and steep slope, then very low factors of safety often resulted.
Mr Evans replied that in this case the deformation readings had indicated a slip zone type of failure. This, and the limit on the time which could be spent analysing such a small job, had meant that bearing capacity analyses had not been made.

Mr Galloway (M.O.W., Central Labs) said that it was necessary in levelling work to run monitoring measurements three or four times before any loading changes were made, in order to establish the actual precision of levelling. Repeated surveys on a dam had shown that the actual precision was ± 0.02 ft. for 80 - 90% of the results: the other 10 - 20% showed variations of several tenths of a foot.

Mr Evans said that on the job described in his paper they were using four external benchmarks but they were interested primarily in the relative movement of points within the structure. He had two people taking readings alternately - each person taking three readings on each side of the staff. The fifth decimal place of a metre was estimated and he thought that the accuracy of the averaged reading could be taken as the fourth decimal of a metre (mm/10).

Mr G.A. Pickens (Tonkin and Taylor) pointed out the need for benchmarks to be founded below the zone of ground subject to seasonal fluctuations, i.e. deep and sleeved.

Mr J.P. Blakeley (Beca, Carter et al.) asked about the cost of installing sockets in the structure and Mr Evans replied that in their case it had proved negligible. Mr Blakeley thought that the precision of Mr Evans' levelling might discourage people from attempting to do this important follow-up work and that for practical purposes settlement measurements to ¼ inch accuracy was what was of interest.

Mr B.W. Buchanan (M.O.W., Wanganui) said that his experience with the recording of levels of buildings with ordinary equipment was that about 45% of them appeared to be going up a bit, 45% of them appeared to be going down a bit, and 10% were fairly clearly going down. He spoke of the difficulty of preserving benchmarks over a period of years, especially on construction sites.

Mr Evans said that on his job they had had the utmost cooperation from the contractor: their benchmarks were protected in cast iron boxes and the contractor assured him that these were not bumped. He said that if a contractor is instructed to engage a registered surveyor to take levels then he has an interest in preserving benchmarks.

Mr Morris described a situation in Christchurch - a deep foundation where the contract had called for precise levels to be taken by a registered engineer on adjacent buildings. He said that the scatter of results was amazing. He said that they now simply made visual examination and all appeared well.

Prof. P.W. Taylor (University of Auckland) endorsed Mr Morris' plea for a more reasonable treatment of the contractor in contract documents. Speaking to Mr Gillespie's paper, he agreed that it was part of the designer's job to devise a means of construction. The great engineers of the past, he said, considered this to be one of the most important parts of the job. Both Mr Morris and Mr Gillespie had quoted cases where temporary works - strutting or dewatering of excavations - were not considered at all by the designers: this was bad engineering.

Prof. Taylor said that several items had emerged from the symposium. These included the need for:

1. greater responsibility on the part of the engineer for the method of construction,

2. "follow-up" by soils engineers, during construction, to ascertain whether their assumptions were justified,

3. more enlightened and realistic financial and professional arrangements with contractors, and

4. more site investigations.

It was for the professional engineers themselves, he said, to see that these needs were fulfilled.

Prof. Taylor proposed a vote of thanks to the speakers which was carried with acclamation.