SYMPOSIUM PROCEEDINGS

NEW ZEALAND PRACTICES
in
SITE INVESTIGATIONS
for
BUILDING FOUNDATIONS

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NEW ZEALAND NATIONAL SOCIETY FOR SOIL MECHANICS AND FOUNDATION ENGINEERING

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ON

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IN

SITE INVESTIGATIONS

FOR

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BASIC OBJECTIVES OF SITE INVESTIGATION

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DEFINITION AND OBJECTIVES

Site investigations are made to determine the natural conditions which affect the design, cost and performance of engineering works embracing in the broadest sense topographical, atmospheric, aesthetic, social, geographic and economic factors as well as subsurface considerations. The term is conventionally restricted in engineering practice to apply to the first and the last of these; the topographical and subsurface characteristics of the ground relative to structures and, even more specifically in this symposium, relative to buildings. Not restricting the scope any further we can consider buildings as space enclosing structures which are generally made of manufactured or natural materials (like stone) of relatively low bulk in relation to the space they enclose.

All buildings depend ultimately upon the earth to support them. The purpose of a "site investigation" is to determine the cheapest satisfactory means of using the ground to support the weight of the building and its contents with due regard to the requirements of existing adjacent buildings. It is worth observing at this point that no site is impossible to build upon but some involve much more expensive buildings and foundations than others. What then do we expect the earth to do for a building?

We look to it to provide support without complete collapse either immediate or progressively in the long term; without deformation either unsightly, leading to progressive disintegration, impairment of weather proofing or malfunction of enclosed mechanisms. Often we look to the earth to provide material which can be used in the construction, such as filling to modify the elevations of the surfaces on which parts of the building are erected.

The building moves with the ground on which it rests. The mechanism of ground deformation is examined in current practice from the points of view of shear strain; settlement either by consolidation (volume change due to expulsion of included water) or by compaction (closer packing of the soil particles without water expulsion) and overall stability of both existing or created slopes of ground surface. The subdivision between strains due to shear and consolidation is rather an artificial one but is still used for practical purposes.

BASIC FACTORS IN THE APPROACH TO A FOUNDATION INVESTIGATION

It is surprising just how often it is necessary, even today, to remind engineers and architects of the factors which are fundamental to the planning of foundations of a building.

1. Early Investigation:

   The sooner investigation of the site is commenced, the more scope there remains to modify the form of the building or to treat the ground to obtain the most economical building. To some extent this contradicts what is said later about tailoring the investigation to the problem and for best results a balance in time should be kept between planning the building and conducting the investigation.

2. Tailoring the Investigation:

The complete investigation of all the properties of the subsoil can involve a lot of very expensive work. Many investigations still are mounted in this way.

The maximum economy in investigation cost is obtained by directing it specifically to the particular building and adequate briefing of the investigator is necessary. It also requires the investigation to be closely controlled and directed by an engineer with sound appreciation of both the requirements of the building and the properties of soils.

3. Total Load:

We are still asked the nonsense question "what is the bearing capacity of the ground at this site". Out of the context of the magnitude, extent and disposition of the loads this question cannot be answered unless the ground is uniform and so strong that no significant deformations can occur. Such sites are rare.

A mental picture of the "bulb" of pressure is vital. Think first of the total mass of the building.

4. Distribution of Loads:

The total weight of the building and its contents is subdivided and distributed to the ground through various loading bearing elements, not the least of which is the ground floor. In fact the ground floor in many industrial buildings is the most important functional element and the superstructure is quite subsidiary in weight and importance. The floor is an exercise in pavement design.

Settlements of the various parts of the foundations are, generally speaking, proportional to the total load on each part rather than the unit intensity of load, unless of course the deformation is due to excessive shear stress. The interaction of the various footings or various buildings upon each other has to be considered. Damage to the building results from differential movement between the elements and it is therefore the differences between total loads on the various elements which is important.

5. Time of Load Application:

Great problems arise in soft ground where further building or other loads are placed adjacent to an existing structure to subject its foundation soils to further increases of stress. A common situation is the subsequent construction of contiguous additions to buildings. Addition of further floors is not so difficult to cope with provided provision is made in the foundations at the outset, and the additions extend over the whole plan area.

In either case the possibility of additional loads should be part of the brief.

Another aspect of the time or rate of loading is the phasing of placing heavy infill masonry, installation of heavy machinery or placing of underfloor filling relative to completion of the building frame and its cladding or glazing.

6. Tolerance to Deformation:

At the outset of the investigation, but more importantly in the "report and recommendation" phase, an appreciation should be made of the amount of settlement which the particular building can stand. Stiff "boxey" shear wall type buildings, and buildings on continuous raft foundations will settle as a whole and can accept much greater total movement than a framed structure on isolated footings. Further down the scale of tolerance still, industrial shed type buildings with sheet cladding are undamaged by much greater total and differential deformations

There are various quantitative criteria quoted as guides in the literature, in this respect.

Settlement tolerances for machinery in industrial buildings must be considered too but one wonders about the reality of the limits set by machinery manufacturers. The productive efficiency of machines are of course the most vital factor in many industrial processes but some unnecessarily elaborate and expensive foundations have been provided where their justification is emotive rather than rational.

So far I have had a lot to say about the building with little regard to the earth. This is deliberate of course, because one is so often asked to give a carte blanche to the structural engineer or architect who considers he has enough variables on his plate without any "soil mechanic" throwing in a few more.

Assuming that we have now obtained a reasonable briefing about what has to be held up by the ground, let us "break dirt" before finally getting back to the business of accommodation between mother earth and the hand of man - man being at a disadvantage in this.

7. Visual Examination of the Soil:

Whatever is subsequently done by way of quantitative measurement of soil properties by laboratory or in-situ testing, the first essential is for the soil to be seen either as core, as near continuous as possible, or in a natural or artificial exposure (an open pit). If a continuous sequence of soil specimen with all the grains present, albeit compressed or stretched, can be examined and handled by an experienced engineer, or described in standard terms by an experienced technician for the engineer's appraisal, then the investigation is more than half done. Without this initial appreciation any number of test results will be of much less value.

The early effort on investigation should be directed primarily to that end.

If the drilling method yields, at the same time, samples suitable for testing so much the better, but it is often cheaper to drill further holes to retrieve critical samples rather than to combine the two processes.

In-situ testing or probing is a valuable supplement but no substitute for seeing the soil, and by itself it can be dangerously misleading.

8. Comprehensive Investigation:

By this I mean that an investigation should be started with a mind open to all the possible means of founding the building. How often have we been called upon to re-examine sites where initial drilling has washed through overburden to the "solid" without any regard to foundations other than piled? The extra cost of recovering some sort of core on the way down usually is not great in competent hands.

9. Cut and Fill:

Changes of stress due to modification of the ground profile can be more important than those due to the weight of the building. For example an extra foot depth of clay filling is equivalent in weight to another storey of a concrete frame building. Three feet of fill to bring a floor up to cart dock height, or six feet of filling for a house patio can have a dramatic effect upon the associated building.

Even if the ground does not move significantly under this added load, the filling itself has to be compacted and its properties in this respect need consideration during the investigation.

Excavation of existing ground has two consequences. Slopes generated must be at stable angles or retained by additional structure. Secondly a reduction of in-situ pressures compensates for the subsequently added load of the building and the pattern and extent of such relief can be either an advantage or a disadvantage depending upon the final balance of net loads.

10. Slope Stability:

It is a large building which is heavy enough to affect greatly the overall stability of a ground slope. However the consequences of a slope failure immediately become more serious and one has to adopt a more suspicious and cautious attitude even if the slope has no record of collapse.

The majority of sites are not underlain by homogeneous isotropic ground and the calculation of slope stability requires testing and analysis much more expensive than for foundation loads, for anything like the same degree of conviction.

There is a continual pressure from rising real estate values in closely built up areas, to get buildings closer and closer to slopes and the attraction of unimpeded outlook often sways otherwise hard-headed purchasers.

Well, the soils engineer has to take a stand somewhere although he will be pushed hard - towards the edge!

11. Earthquake:

In the present state of the art, design practice is fairly sophisticated in assigning greater forces to the superstructure during an earthquake, but makes only crude assumptions about the ground supporting the building. Much more needs to be known about dynamic characteristics of the soil and compliance between the soil and the building.

Loose sands, loose saturated silts and very sensitive and soft grained soils must be recognized as potentially more susceptible to dynamic stresses. Procedures are available for measuring shear strengths under vibrating load and should be used for buildings of any size in these circumstances.

There is a current quickening of activity in studies of regional seismic characteristics and risk in foundation strata.

ACCOMMODATION BETWEEN THE BUILDING AND THE GROUND

Good engineering requires a spirit of give and take between building and ground. Both can be modified if the necessity is admitted in time.

Foundations can be isolated spread footings, continuous strip footings, continuous rafts or piles, all with their own characteristics of rigidity. Consolidation settlement can be reduced by compensating excavation but not all buildings have a use for basement space and a compensated basement raft (or "floating" foundation) is then an expensive way out.

If significant strains in the foundation soils are expected then the answer could well be in modifying the superstructure to provide either greater rigidity to resist differential movement (in the shape of shear walls or deeper spandrells) or to increase flexibility to permit movement without damage. One of the simplest concepts is to strive for a regular symmetrical column layout resulting in substantially equal column loads and consequently balanced settlements; this however often requires columns set back from the building perimeter.

The possibility of these sort of adjustments needs to be recognized before the superstructure planning and design has progressed too far. Considerable amounts of money can be saved on poor sites.

So much for changing the building - what about the ground?

Preconsolidation beyond the pressures to be exerted by building is a dramatic and often economic answer provided certain essentials are present.

These are:-

1. A cheap supply of soil to form the bulk load, either from site excavation or temporary use of material imported for other purposes.

2. Space to apply the bulky load clear of other structures.

3. Time for the load to do its work and be handled. Three months is probably a minimum total time but much more may be needed.

Dynamic compaction of granular soils can be effected by driving closely spaced piles or by vibroflotation. Cement or chemical grout injection can increase soil rigidity but these are expensive processes appropriate to larger or more valuable buildings and sites.

The simplest expedient often is to excavate the offending soil and either replace it with something better or to reinstate it in a more compact state.

If all else fails we can always shift to a better site.

The foregoing discussion ranges fairly wide. The point I am trying to ram home is that a good site investigation does not consist of providing a list of soil descriptions and test results. It is an integrated process with the superstructure design. It should be conducted to provide information about those of the above listed factors which are relevant to the particular building - not less and not too much more.

PLANNING THE INVESTIGATION

Some of the confusion and inefficiency which often attends site investigations can be avoided if an initial distinction is made between approaches which are either Exploration or Investigation. Figure 1 is modified from one by Fookes (Ref.2) who presents a very useful discussion on the subject. Whether all the phases are separated in execution depends upon the size of the project, but even if they are compressed into one operation, or some phases apparently omitted on small jobs, the same principles can apply, and can do so whatever the breadth of the term "site investigation".

Subsurface exploration is the first phase which determines the general characteristics and distribution of the soils over the site or sites as a whole. It may comprise merely an appraisal of pre-existing geological information or data from earlier investigations. It may be an almost subconscious drawing upon the investigator's experience of the area. On an extensive or strange site, it is a separate operation of drilling "reconnaissance" holes on a pattern designed to cover the site without too much reference to location of particular buildings, pointing up the feasibility of the site for the purpose. As such it is sensible and economic, but it should not go too long in this fashion.

What follows is the subsurface investigation which is now carefully planned to consider particular loads and problems. Additional bores are made at proposed building locations and sampling is directed more at specific strata which are significant to the superstructures. Closer briefing of the investigator by the designer and feed-back of preliminary test results from the laboratory, contribute at this stage to greater economy in the investigation. Better overall economy in the buildings will result from maintenance of an open mind on both sides as to possible accommodation between building and ground in this phase.

On larger civil engineering projects mainly, but also to a lesser extent in the case of buildings, what Fookes terms "Foundation Investigation" obtains further information during construction of the project. This may be just follow up and confirmation of the investigation phase, and as such should be part of the investigations brief. More than that however some soil sequences cannot be closely evaluated by small diameter bores, and exposure by excavation or test pits may necessarily be left until a larger construction force of men and materials is available. The preliminary assumptions leading the project to this stage of commitment must not be too far out however.

The final phase is the subject time and again of addresses by leading foundation engineers in papers and closures to symposia. Foundation engineering is an art which combines experience of the behaviour of buildings with the science of soil mechanics. The justification for many of the simplifying assumptions made in soil testing and analysis is simply that buildings designed on these assumptions function satisfactorily. Gross malfunction we are bound to hear about, lesser ones may be blamed upon us without justification or our knowledge. In the other direction our designs may be too conservative particularly in respect of tolerable settlement and we are not going to "sharpen up" in this until we have a lot more precise measurement and documentation of practical cases which are <u>not</u> subject to obvious failure. It is not easy to persuade clients to pay for these measurements.

The stages of investigation can be summarised as -

Briefing
Reconnaissance
Feasibility assessment
Briefing
Detailed Investigation
Report
Specification and design
Performance observation.

ORGANISATION

The other papers in this symposium will deal with methods and techniques in detail, but there are some considerations of the way in which a site investigation is organised which I consider fundamental to the Basic Objectives.

If a site investigation is to be done most efficiently the processes of drilling, logging, sampling, testing, reporting and analysis must be co-ordinated and closely controlled by an engineer with specialist experience in these things and capable of appreciating the real requirements of the building superstructure. Ideally perhaps that engineer should be beside the drill rig all the time but a perfectly satisfactory job results from employing specially trained technicians working in close touch with an engineer. This is an integrated process in which the soils or materials engineer is recognised as a specialist in the total engineering design team, a concept followed more in American practice than in Britain.

Arranging the investigation on the basis that a driller bores at fixed positions, samples at fixed intervals, sends a vast number of samples to a laboratory where they are all tested for every conceivable property regardless of relevance to the particular structure to be built, and then subjecting the whole accumulation to a post mortem by an engineer with a bunch of formulae in one hand and a slide rule in the other, is to arrange for the greatest waste of money, the greatest boredom and maximum chance of misinterpretation.

I am not exaggerating as much as you may think. Although it now occurs less than say ten years ago, too many investigations are still mounted from this view point and some of them have lead to expensive failure, or required for a repetition of the investigation.

British practice combines the logging, sampling and testing processes with drilling by contracting companies thus splitting the "factual" and the "opinion" aspects of the investigation at the wrong level leading to an undesirable disintegration, in my view. Elaborate specifications and quantity schedules seek (quite understandably) to obtain the advantage of competitive tendering but sacrifice flexibility and overall economy.

Investigation drilling is done because we do not know what we will find in the ground. How can a drilling contractor be expected to quote firm footage or sampling rates without covering himself for all eventualities?

There is a school of thought strongly against paying for drilling or engineering work on a "time and expenses basis" but in my experience it is cheaper and more reliable to investigate a site in this way, provided of course that experienced and competent control is provided.

CONCLUSION

What better than to quote Ralph B. Peck (Reference 3).

"To be sure, the high incidence of failures or of unexpectedly costly jobs is a consequence partly of the accelerated pace of construction, and partly of the fact that soil mechanics has opened the door to more complex and more daring jobs than would have been considered feasible a few years ago. Nevertheless, a disturbingly large residue of costly and unfortunate incidents remains to be explained, even in circles where soil mechanics is by no means unknown. This situation may have arisen out of our failure to discriminate between art and science. In an age of scientific marvels, civil engineers have lost sight of the accomplishments of the artist in his profession. We would do well to recall and examine the attributes necessary for the successful practice of subsurface engineering. These are at least three: knowledge of precedents, familiarity with soil mechanics, and a working knowledge of geology."

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Reference 1: C.P. 2001 (1957) "Site Investigations", British Standards Institution.

Reference 2: Fookes, P.G. (1967) "Planning and Stages of Site Investigation". Engineering Geology Vol. 2, No.2.

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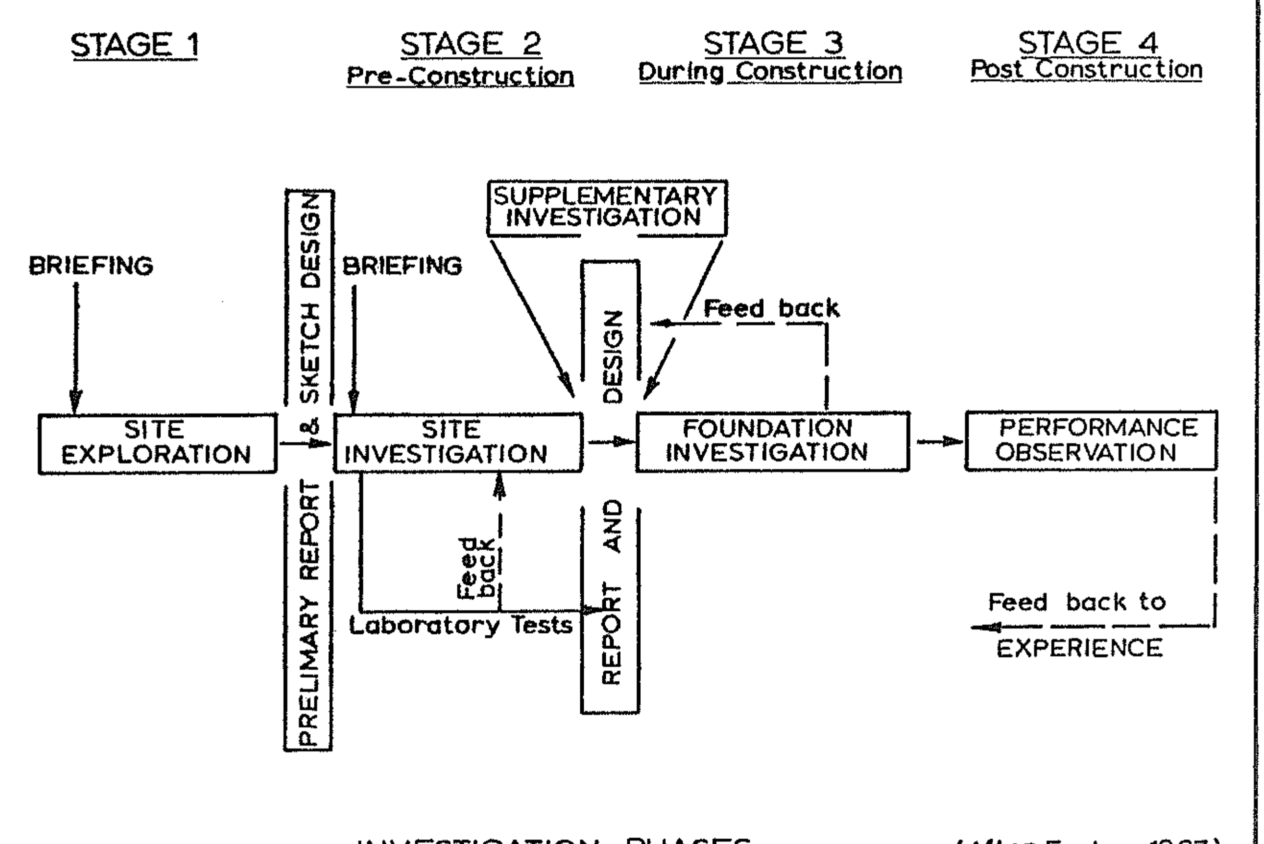

Fig. 1 INVESTIGATION PHASES (After Fookes. 1967)

OVERSEAS SOIL SAMPLING PRACTICE

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INTRODUCTION

It is not intended, even if it were possible, to make this paper a comprehensive examination of international soil sampling procedures. Rather, it is proposed to present a few recent developments, arising from the activities of the International Group on Soil Sampling (IGOSS) and its affiliated national bodies. In particular, the new draft German Standard DIN 4021 on drilling for soil investigation purposes and soil sampling, deserves wider attention. This standard brings out clearly the modern concepts of soil sample quality. With the permission of the German Committee for the improvement of soil sampling, it has been abstracted freely in this paper to make the information more readily available in New Zealand for discussion. A comprehensive review of soil mechanics aspects of soil sampling is at present being prepared by IGOSS but unfortunately copies are unlikely to be available here in time for this symposium.

QUALITY OF A SOIL SAMPLE

Until the activities of the Swedish Geotechnical Commission (1914-1922) little thought seems to have been given to the question of the quality of a soil sample. The recognition that soils change some of their properties when disturbed led to the concept of an "undisturbed" sample but this term has been so misused as to now have little meaning. One can only say that it is related to the way a sample has been taken and its intended purpose. "Why are soil samples taken?" This question seems to have two broad answers, roughly correlated with the early concepts of "disturbed" and "undisturbed". The first answer is clearly for identifying the ground. Initially this was limited to the type of soil, its texture and perhaps its colour. Later some indication of its consistency and perhaps structure was also expected. The second answer is for laboratory testing, especially where a knowledge of strength and deformation characteristics is of importance.

Even for disturbed samples a certain minimum quality is necessary. It is fortunate that there is now reasonably general recognition that thoroughly disturbed and incomplete samples are quite unsuitable for any soil mechanics purpose. It was once thought that information concerning the nature of soil strata could be obtained from the examination of the sediment from the return water of wash borings, but as Terzaghi (1953) has expressed so well, "similar techniques were used by some of the soothsayers in ancient Greece for the purpose of predicting future events and the results were equally reliable."

Of recent years there has been a growing recognition of the need to express more clearly soil sample quality, the degree of disturbance and the classification of sampling according to the intended use of the samples. Kallstenius (1958) suggests that undisturbed means "sufficiently little disturbed for the actual strength tests" and divides undisturbed sampling into three main classes as follows:-

"Research class - Highest possible quality of samples with little regard to costs. (Research, important buildings, expensive foundations).

Routine class - A fairly good quality of samples, with some attention to costs. (Routine cases for specialists in soil mechanics).

Simple class - The samples must not be seriously disturbed, but most consideration is given to simplicity of operation and low sampling costs. (Sampling by non specialists, often in accordance with standard instruction.)"

He considers there is an "optimum" sample disturbance which is largely an economic consideration. Sampling costs generally rise with the quality required, but the more consistent results of higher quality samples may well allow the use of smaller factors of safety and cheaper foundations. Discussing these classes Tomlinson (1963) suggests that the "Research class" would include sampling with elaborate apparatus such as the Swedish foil sampler (Kjellman et al., 1950) or carefully cut hand samples from trial pits; "Routine class" would require a good design of piston or thin-walled sampler which is pushed or pulled down into the soil and not driven with a hammer; while the "Simple class" might include open drive samplers hammered into the soil.

Richards and Parker (1967), largely with emphasis on ocean-floor sampling, put forward four classes of sample or sampler which show various degrees of sample disturbance. With minor changes Table 1 shows their classification and identifying characteristics of each sample class. It will be noted that the first three classes could be considered equivalent to the classes of Kallstenius (1958). Stephenson (1967) distinguishes between "deficient samples" as from overflowing wash water and "complete samples" of three categories

(i) Disturbed - as by an auger
(ii) Distorted - as from a thick walled tube sampler
(iii) Undisturbed (or Undistorted) - in several grades,

as an attempt to include all soil mechanics sampling in a coherent system.

The German Standard DIN 4021 (1969) gives five quality classes (Table 2) based on the information obtainable from a soil sample in each category. The lowest class (5) is an incomplete soil sample which would allow the determination of the broad geological stratum only, whereas the other four classes allow in turn the determination of the particle size distribution (4), the moisture content (3), the porosity, void ratio or density (2) and finally the compressibility and shear strength (1). P.W. Taylor (pers. comm.) has postulated the need of a higher class of soil sample in this system on which shear and compression elastic moduli could be measured. Such moduli are not commonly used in normal practice but are necessary for more sophisticated settlement analysės and response to vibrations. Taylor's concept is an interesting one since there appears little doubt that such moduli are more sensitive to sample disturbance than peak shear strength or compressibility and as such have been used in assessment of sample quality (Kallstenius 1958, 1963). However, in the further description of their quality classes the German Committee make it quite clear that class 1 would represent a "completely undisturbed sample". Beyond this of course is the "perfect sample", the in-place soil unobtainable by any sampling technique. They also point out that the desirable sampling procedures depend on the kind of soil to be investigated and on the specific purpose of the investigation. It would be uneconomical and quite unnecessary to require the same care for each investigation of the subsoil. In each case the quality required is determined by the information to be obtained. Even for similar soil conditions the required quality class can be high for one investigation and low for another. The required sample quality should be a deciding factor in the selection of acceptable drilling and sampling techniques.

DRILLING METHODS

The German Standard goes on to consider drilling methods and the effect each method can have on the quality of the sample obtainable. The information is summarised in Table 3 where all drilling methods known to the German Committee are classified into four main groups:

a - f with continuous extraction of cored soil samples
g - j with continuous extraction of non cored soil samples
k - o with extraction of incomplete soil samples
p - t with light equipment and small soil samples

The methods in each group are then further classified by the method of sample extraction and whether or not sampling is assisted by circulating fluid, using the terms rotating, ramming, pressing, percussive, and gripping. Most of the terms are quite clear but even with the explanatory note at the bottom of the table there is still a chance of misunderstanding the precise meanings attached to "ramming" and "percussive". Clarification is being sought, especially concerning method "i".

In the past it has been conventional and convenient to consider drilling as an operation distinct from sampling. Drilling was restricted to the means of advancing the hole to a particular depth at which a sample was required. Any sample taken during the drilling process would originally have been termed "disturbed" whereas an "undisturbed" sample was one taken in a coring tube of some kind as a deliberate separate operation. However, several modern drilling techniques allow a reasonably high quality of sample to be obtained during the drilling process so that the traditional cleavage between drilling and sampling is no longer completely appropriate. The German Standard has recognised this by including as drilling techniques those sampling methods which also advance the hole. This unity of concept simplifies the tabulation and thus the selection of a means of obtaining a sample of a given quality. Sampling methods that do not advance the hole, such as the use of open-drive samplers of smaller size than the hole, are not specifically listed.

If one accepts fully the validity of the German concepts and classification then, for an acceptable sample quality, the selection of a suitable drilling method in a given type of soil is easily made from the table. Where only the texture of the different strata is required, a rather primitive drilling method may be permissible while a very good method is clearly necessary if soil strength is to be investigated. It is not suggested that the classification precludes the use of a suitably combined technique whereby the hole is advanced by a technically limited drilling method, but good sampling technique and equipment is used to take the samples of the quality desired. The only significant omissions from Table 3 seem to be the drilling methods with continuous extraction of cores based on displacement and on auger core barrels. In the terms of the table the Swedish foil sampler (Kjellman <u>et al</u>., 1950) would be described as "pressing without flushing assistance", while the auger coring sampler described by Aitchison and Lang (1963) would appear as "rotating without flushing assistance". Both this latter technique and the modified use of hollow-stem augers described by Thomas and Barker (in prep.) would fall in line "a" of the table but the quality of sample obtainable is much higher than from rotational dry core drilling with a single core tube. Table 3 is a very useful summary of possible sample quality related to drilling method but perhaps its most important lesson is the reminder of how few drilling techniques can lead directly to even reasonable quality samples.

GENERAL CONSIDERATIONS ON UNDISTURBED SAMPLING

It is generally accepted that different soils require different samplers and sampling specifications and that no equipment as such can ensure good samples. However, by restricting discussion to "those soils which lend themselves to relatively undisturbed sampling by punching a thin-walled tube into the soil" IGOSS (1965) have offered some guide lines to good sampling:

(a) Preparation for sampling:

The drilling equipment and technique should be such as to cause minimum disturbance of the soils to be sampled. If casing is used it should not be driven long distances before cleaning out and the water level inside the casing should be controlled at or above that in the surrounding ground. Unless a piston sampler is being used the bottom of the hole should be adequately cleaned of disturbed material before sampling.

(b) Smooth clean samplers:

The walls of the sampler should be smooth and clean, preferably non-corrodible and with a low coefficient of friction between soil and sampler.

(c) Inside clearance:

The lower end of the sampler should be of slightly smaller inside diameter than the upper end with no other diameter less than that at the cutting edge. The ratio of diameter difference to the larger diameter is referred to as the "inside clearance ratio". Desirable values for this ratio vary slightly with the design of sampler and the type of soil being sampled, but as an example, a smooth clean sampler should have a clearance of ½ - 1% when sampling non-swelling soils to say 20 m (60 ft).

(d) Wall thickness and edge taper angle:

Clearly a thick walled sampler displaces more soil than a thin walled sampler and is thus more likely to disturb the soil being sampled. However, the greater wall thickness is acceptable provided the edge taper is such that it occurs some distance back from the cutting edge. A common criterion of acceptable wall thickness is the area ratio defined as $(Dw^2 - De^2)/De^2$ where Dw is the external diameter of the sampler and De the internal diameter at the cutting edge. Table 4 gives IGOSS (1965) suggested combinations as suitable for undisturbed sampling at the present state of knowledge.

(e) Sample length:

For high-quality samples of clay, the material within two diameters of the sample ends is suspect and, further, there is a maximum safe length-to-diameter ratio. IGOSS (1965) suggestions for optimum sample lengths are given in Table 5.

(f) Sample diameter:

Desirable sample diameter is dictated by grain diameter and sample quality. It should not be less than 50 mm (2 inch) and preferably 75 mm (3 inch), or more.

(g) Escape of air, water or soil:

Samplers should have pistons, or ample vents to allow escape of air, water or soil during sampling. Punching speed is controlled by vent size and IGOSS (1965) suggest a uniform velocity of about 2 m/min (6 ft/min) as optimum.

(h) Operations after sampling:

The sealing, handling, transport, and storage procedures after sampling should be so designed as to maintain the quality of sample from the drill head to the test machine in the laboratory.

In conclusion there is an interesting commentary from the German Committee for the improvement of soil sampling. While they hope that the adoption of the new German Standard DIN 4021 will lead to improved performance of drilling and sampling for soil investigations they feel that it is fundamentally necessary "to improve the knowledge of the responsible man working directly at the drilling machine. This man must know not only the details of the equipment but must also possess sufficient knowledge of all those soil mechanics problems being important in soil sampling". Thus they recommend that drillers should have to acquire a special licence to carry out borings for soil investigation purposes. "The licence shall be issued by the German National Committee of the International Society for Soil Mechanics and Foundation Engineering after half a year theoretical training, a week course on soil sampling problems and following a successful examination (1 day) on these problems."

The New Zealand National Society might care to discuss the implications of a similar requirement in New Zealand.

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TABLE 1 DEGREE OF SAMPLE DISTURBANCE

(After Richards & Parker, 1967)

Class of sample or sampler	Examples of sampler	Method of Sampling	Condition of sample	Axial core shortening	Distortion of stratification	Sensitivity of sample to in-place sensitivity	Sample Quality for shear strength testing
1	Swedish foil, hand block sample	Best research sampler using refined techniques of sampling	Minimum deformation	None	Unable to detect	Closely corresponds to in-place, except near ends of core	Superior
2	Thin-wall fixed-piston corer	Sampler incorporating features for minimising disturbance; smooth even drive stroke; sharp cutting edge, angle of 5^{o} or less; $C_s \approx 20$*	Slight deformation	Detectable with difficulty or not at all	Slightly bent near walls of core tube	Slightly less than sensitivity in-place	Good to very good
3	Long open-barrel and long removable-piston corer ($C_s > 20$)	Sampler incorporating few features to minimise disturbance; sample may be jarred during sampler penetration	Noticeable deformation	Moderate; probably noticeable with ease	Moderately bent near walls of core tube, but still undisturbed at centre	Conspicuously less than sensitivity in-place	Fair
4	Standard penetration; or very long sampler w/ or w/o piston ($C_s \geqslant 20$)	Samplers designed with little or no consideration for minimising disturbance	Marked deformation	Considerable	Severe or destroyed	Only a fraction of sensitivity in-place	Inferior

* C_s is length to diameter ratio of sample.

TABLE 2 QUALITY CLASSES FOR SOIL SAMPLES (DIN 4021)

Quality-class	Soil sample unchanged in	Primarily determinable
1	Z, w, γ, τ, E_s	boundaries of fine-stratification grain-size distribution consistency limits maximum and minimum density specific gravity organic matter moisture content dry density porosity, void ratio compression index shear strength
2	Z, w, γ,	boundaries of fine-stratification grain-size distribution consistency limits maximum and minimum density specific gravity organic matter moisture content dry density porosity, void ratio
3	Z, w	boundaries of fine-stratification grain-size distribution consistency limits maximum and minimum density specific gravity organic matter moisture content
4	Z	layer boundaries without fine-stratification grain-size distribution consistency limits maximum and minimum density specific gravity organic matter
5	also Z changed; incomplete soil sample	sequence of strata

Z = grain-size distribution and/or Atterberg Limits
w = moisture content
γ = dry density
τ = shear strength
E_s = compression index

TABLE 4

ACCEPTABLE WALL THICKNESS AND CUTTING EDGE TAPER FOR UNDISTURBED SOIL SAMPLES (IGOSS, 1965)

Area Ratio in per cent	Edge taper Angle in degrees
5	15
10	12
20	9
40	5
80	4

TABLE 5

SAFE LENGTH FOR UNDISTURBED SOIL SAMPLES (IGOSS, 1965)

Type of Soil	Greatest length to diameter ratio
Clay ($S_t > 30$)*	20
Clay (s_t 5-30)	12
Clay ($S_t < 5$)	10
Loose frictional soil	12
Moderately loose frictional soil	6

* S_t is sensitivity

For Table 3 see pages 1-20, 1-21

TABLE 3

DRILLING METHODS FOR SOIL INVESTIGATIONS ACCORDING TO THE QUALITY OF SAMPLES OBTAINABLE (DIN 4021)

drilling methods					equipment		suitability of the drilling method		samples ****		
item	sample extraction **	flushing assis-tance	method of extraction	description	drilling tool	convent. hole Ø D(mm)	practicable for type of soil	unpracticable for type of soil	available quality class corres-ponding to table 1 referring to column 7	undisturbed in	remarks
column	1	2	3	4	5	6	7	8	9	10	11
1. Methods with continuous extraction of cored soil samples											
a	1. rotating	no	by drilling tool	rotational dry core drilling	single core tube	65–150	clay, silt, fine sand cohesive	noncohesive gravel, boulders	4, (3-2)	Z,(w, γ)	good in the centre, dried in the outer region
b		yes	by drilling tool	rotational core drilling	double core tube	65–150	clay, clayey, also cemented mixture of grained soils, boulders	noncohesive soils, silt	4, (3-2)	Z,(w, γ)	
c		yes	by drilling tool	rotational core drilling	double core tube with front attachment	65–150	clay, silt, plastic to solid, sand clayey	gravel-sand-silt mixture with little fines and clean gravel, boulders	2,(1)	Z, w, γ, (E_s, τ)	
d	2. ramming	no	by drilling tool	ramming core drilling	ramming core tube with inner cutting edge	60–300	soils with grain size to max. D/5	soils with grain size larger than D/2 high density	in cohesive soils 3, (2 – 1) in noncohesive soils 4,(3)	Z, w, (γ E_s, τ) Z,(w)	
e		no	by drilling tool	ramming core drilling	ramming core tube with outer cutting edge	60–300	soils with grain size to max. D/5	soils with grain size lager than D/2 high density	in cohesive soils 4, (3) in noncohesive soils 4	Z,(w) Z	ramming diagramm to count blows
f	3. pressing	yes	by drilling tool	pressing core drilling	core tube with inner cutting edge	50–150	clay, silt, fine sand	sand-silt-mixtures with little fines and sand with grain size larger than 0,2 mm, gravel, medium solid and solid clays	in cohesive soils 2(1) in noncohesive soils 4, (3)	Z, w, γ, (E_s, τ) Z,(w)	if used as extraction tool, also higher quality-classes
2. Methods with extraction of continuos noncored soils samples											
g	1. rotating	no	by drilling tool	manual auger drilling	rod string with auger, worm, spiral	80–400	above water level all soils below water level all cohesive soils	compact soils stones and boulders larger than D/3,	4,(3)	Z,(w) below water level only from muck of boring with large diameter of augers	
h		no	by drilling tool	mechanical auger drilling	rod string with auger, worm, spiral	100–2000	above water level all soils below water level all cohesive soils	boulders larger than D/3 high density	4,(3)	Z,(w) below water level only from muck of boring with large diameter of augers	
i	2. percussive	no	by drilling tool	percussive drilling	rope with percussive auger	150–400	above water level clay, silt below water level clay	above water level gravel, below water level silt, sand, gravel	4,(3)	Z,(w)	
j	3. gripping	no	by drilling tool	grab drilling	rope winch or rod string with borehole grab	400–2500	coarse grained soils, gravel, sand, boulders smaller than D/2	solid cohesive soils, boulders larger than D/2	above water level 3 below water level 5(4)	Z, w (Z)	

TABLE 3 (Continued)

drilling methods					equipment		suitability of the drilling method		samples ***		
item	sample extraction **)	flushing assistance	method of extraction	description	drilling tool	convent. hole Ø D (mm)	practicable for type of soil	unpracticable for type of soil	available quality class corresponding to table 1 referring to column 7	undisturbed in	remarks
column:	1	2	3	4	5	6	7	8	9	10	11
3. methods with extraction of incomplete soils samples											
k	1. rotating	yes	by direct flushing	rotary drilling (flushing)	roll chisel, jet chisel, step chisel etc.	60–500	all soils	boulders	(5)	unsuitable for soil mechanics	only for boring through the upper not interesting strata
l		yes	by reverse flushing	rotary flushing drilling	as above, but with hollow chisel	60–1000	all soils	boulders	5,(4)	(Z) if single pieces of core, if sample, then Z	
m	2. percussive	no	by drilling tool	percussive drilling	rope with valve drill, shell with valve	75–500	gravel and sand wet	above water level	5,(4)	(Z)	also possible in clayey soils with addition of water, but then unsuitable for soil mechanics
n		yes	by flushing	chisel flushing drilling	rope or rod string with chisel	75–300	on rope: clay, silt sand on rod string: clay, silt, sand, gravel	on rope: gravel, stones, boulders on rod string: stones, boulders	5	unsuitable for soil mechanics	
o	3. pressing	yes	by flushing	wash boring	jetting rod	30–80	silt, sand also cohesive	very cohesive soils, gravel, large stones, boulders	5	unsuitable for soil mechanics	only to find changing of stratification
4. methods with small equipment and extraction of few quantity of soil (driving tests)											
p	1. rotating	no	by drilling tool	manual auger drilling	auger, worm, spiral	30–80	above water level clay to medium gravel, below water level cohesive soils	coarse gravel larger than D/3 and compact soils	above water level 4,(3) below water level 4	Z,(w) Z	only for low depths
q	2. ramming	no	by drilling tool	penetration test with core extraction	driving sound with core tube front attachment	30–50	soils with grain size to max. D/5	soils with grain size larger than D/2	in cohesive soils 3(2) in noncohesive soils 4,(3)	Z,w,(γ) Z,(w)	determinable only with
r		no	by drilling tool	driving test with groove-and pipe-drill	rod with spline or pipe with longitudinal slot	20–40	soils with grain size to max. D/5	difficulties by extraction of samples in noncohesive soils below water level	in cohesive soils 4,(3) in noncohesive soils above water level 4,(3) in noncohesive soils below water level 5,(4)	Z,(w) below water level (w) only with longitudinal slot Z,(w) if sample, then Z	restrictions in chapter 5, 2, 1, 3
s	3. pressing	no	by drilling tool	manual drilling	pipe with longitunal slot	20–40	soft cohesive soils	semi solid to solid cohesive soils non cohesive soils	4,(3)	Z(w)	mentioned
t		no	by drilling tool	pressing test with core extraction	pressing sound with core tube front attachment	30–40	clay, silt, fine sand	solid and coarse grained soils	3(2)	Z,w,(γ)	

*) All numerial data of column 6 to 11 are only guidances

**) "ramming" means drilling tool used as ramming weight "percussing" means drilling tool percussed by a piston or separate weight.

***) data within brackets means quality class available only with good conditions, equipment and properties of soil.

INTRODUCTION OF PAPERS, SESSION 1

CHAIRMAN:
Mr J.H.H. Galloway, Ministry of Works, Wellington

BASIC OBJECTIVES OF SITE INVESTIGATION by D.K. Taylor

Introducing the paper, Mr Taylor said it was his hope that he was not just "preaching to the converted". He referred to the poor briefing which a soils engineer often receives and suggested that structural engineers and architects should understand at least a little of the field of soil mechanics. He said that the symposium should be considered as an opportunity for the critical review of techniques and standards.

Mr Taylor stressed the need for adequate briefing and outlined the range of basic information which a soils engineer requires. This includes existing ground levels and buildings, proposed ground levels, building layout and loads, style and rigidity of the proposed building and the proximity and sensitivity of adjacent buildings. He discussed tolerable settlements in buildings and quoted the following examples :-

Type of Structure	Total Settlement	Differential Settlement
Tall buildings (14 to 20 storeys, concrete framed)	2 to 3 inches	1/480 of span
Sheds and warehouses without masonry walls	1 to 2 feet	
Reinforced concrete silos on mat foundation (dry contents)	10 to 15 inches	
Rolling mills	9 inches	6 inches
Cylindrical steel tanks (at perimeter)	12 to 14 inches	

Mr Taylor described his preference for a two-stage investigation comprising an initial reconnaissance phase and a secondary sampling phase. He also referred to the use of computers for analytical work such as settlement calculations. He then went on to discuss the need for compatibility between a structure and the soil on which it is founded.

In conclusion he gave the following orders of cost for ground treatment:-

Treatment	Cost per cubic yard treated
1. Excavate and replace with imported hardfill (Auckland City)	\$4 to \$5
2. Vibroflotation (15 feet or deeper)	\$2 to \$4
3. Preload (with 18 feet deep sand drains) using cheap imported filling re-used for site development	\$1

OVERSEAS SOIL SAMPLING PRACTICE by Dr R.D. Northey

Note: In Dr Northey's absence overseas, the paper was presented by his colleague, Mr R.F. Thomas.

Mr Thomas said that the aim of the paper had been to introduce a wider cross-section of practising engineers to the International Group on Soil Sampling (IGOSS) and also to a very recent contribution to the Group by the German National Society. He discussed the development of undisturbed sampling procedures over the last 50 to 55 years and categorised the types of laboratory tests which could be reliably performed on specimens obtained by various sampl-ing procedures.

Mr Thomas provided a newly received extension of the German publication (Table 4) and discussed its application. He pointed out that in the footnote to Table 3 of Dr Northey's paper the words "ramming" and "percussing" had been inadvertently transposed during translation.

TABLE 4

Samplers for special samples from boreholes

Column	1	2	3	4	5	6	7	8
	kind of sampler	recommended dimensions of samples diameter mm	 length mm	drilling method	practicable for type of soil	unpracticable for type of soil	attainable quality class (corresp. to table 1, referring to column 5)	remarks
	open sampler with valve							
1	thin-walled	114	≈ 250	ramming or pressing	cohesive and organic soils of stiff consistency; same, of semi-solid consistency; sands above water level	gravel, sand below water level; pulpy, semi-solid to solid and solid cohesive soils; soils with coarse embedments	2 (1) 3 (2) 3	inside clearance C_i 0 required
2	thin-walled	75	≈ 200	as above	as above	as above	as above	wall thickness ≤ 2 mm
3	thick-walled	114	≈ 250	ramming	as above, furthermore: cohesive and organic soils of semi-solid consistency, with coarse grain as well; sands above water level with coarse grain to 20mm	gravel, sand below water level; pulpy and solid cohesive and organic soils; soils with coarse embedments	3 3	wall thickness 5 to 10 mm with liner as well

... Continued

TABLE 4 - Continued

Column	1	2	3	4	5	6	7	8
	kind of sampler	recommended dimensions of samples diameter mm	length mm	drilling method	practicable for type of soil	unpracticable for type of soil	attainable quality class (corresp. to table 1, ref-érring to co-lumn 5)	remarks
	<u>piston sampler</u>							
4	thin-walled	75	600 to 800	pressing	cohesive and organic soils of pulpy to stiff consistency; sensitive soils as well; sands of low and medium density	gravel; sands of very low or high density; semi-solid and solid cohesive and organic soils; soils with coarse grain	1 3 (2)	The inside clearance C_i has to be 0,5 to 1%
5	thick-walled	75	600 to 1000	pressing	cohesive and organic soils of soft to stiff consis-tency; sensitive soils as well	gravel; sand; pulpy, semi-solid and solid cohesive and organic soils; soils with coarse grain	2 (1)	with liner only; the inside clearance C_i has to be 0.5 to 1%

For thick-walled samplers with gripping - or closing device, an extraction of soils which fall out of other samplers - e.g. fine gravel, loose sand or soft silt - is possible under reduction of the quality class.

DISCUSSION, SESSION 1

Mr Galloway opened the discussion by inviting Mr Taylor's opinion on the value of microzoning in the solution of earthquake problems and whether this technique could be extended to other aspects and thus provide a starting point for site investigation work. He also commented on the cost of sampling and drilling as outlined in Dr Northey's paper. Mr Galloway said that minor damage to a soil sample only influences its rigidity and not strength or consolidation characteristics; he said that costly sampling techniques were therefore not warranted except for such things as sophisticated earthquake analyses. He followed by inviting comment, particularly from drillers' representatives, on the proposals made in Dr Northey's paper for education of drillers.

Mr Taylor was not sure what has been achieved with microzoning but stated that if it provided information on soil rigidity then it would be of some use as a background guide.

Mr W.L. Cornwell (M.O.W. Auckland) spoke as a drilling manager to Mr Galloway's question on education of drillers. He suggested that engineers should be educated before drillers because often in site investigation, engineers do not know what they want in the field.

Mr E.F. Richardson (Richardsons Drilling Co., Palmerston North) supported the idea of engineer ignorance. He thought it was a good plan to educate drillers, but doubted whether their classification was possible; he said that it had been tried before without success.

Mr J. Faulkner (Brown Bros Ltd, Hamilton) agreed with Mr Richardson on driller education and suggested that there was also a need for better communication between engineers and drillers.

Mr R. Gilmour (Auckland Regional Authority) said that current problems in site investigation drilling should not be blamed on drillers, but on engineers for not seeing that there was better equipment in the country to do the job. He stated that engineers should be able to give drillers more precise instructions but that this required close liaison with geologists to get a better overall picture before drilling commenced. He suggested that more lithological maps (1 in 25,000 series) should be published from information already available. He contended that site investigations should be extended beyond exploratory drilling to include more regular uses of seismic methods.
Mr Gilmour then asked Mr Taylor whether he had any rule of thumb method for pre-determining the number, spacing and depth of exploratory holes on any site once an initial reconnaissance had been completed.

Mr Taylor replied that the extent of investigation depended to a large extent upon the money available.

Mr Galloway asked Mr G.L. Evans to comment on microzoning.

Mr G.L. Evans (Royds, Sutherland, Evans & McLeay, Christchurch) said that the study of the dynamic characteristics of soils needs much more attention in New Zealand; structural engineers have long recognised the need for earthquake resistant design but have given little attention to earthquake response or the resisting capabilities of foundation materials. He said that there are practically no generally accepted standard tests which can be applied to determine the dynamic behaviour of a structure under a forced vibrational motion. Mr Evans pointed out that building and foundation vibrations were most commonly caused by machine or by earthquake. He said that the dynamic behaviour of a machine can be reasonably assessed and simulated in a dynamic laboratory test but in the case of an earthquake, the nature of the ground motion and its effect on the ground material and the structure must both be determined. He stated that overseas researchers, particularly in Japan, have tackled this problem by accepting a "design" earthquake of a given magnitude at a specified distance and some measure of dynamic ground properties and physical site data such as depth of strata and densities; from these properties a prediction of motion is possible. Mr Evans contended that what is missing is some form of standard test which can be applied to identify dynamic properties appropriate to any particular site; these include shear wave velocity and shear modulus and the compressional wave velocity and "elastic" modulus. He said that overseas investigators have established a close relationship of the strain condition of the soil to the shear and elastic modulii, both of which are variables.

Mr Evans went on to emphasise the need for definition of a "standard design" earthquake and for the development of a standard test for determining dynamic soil behaviour. He said that the earthquake risks in New Zealand often dictate structural requirements, but, because of lack of data, they are seldom considered in foundation design.

Dr B.R. Falconer (University of Auckland) referred to microzoning and pointed out that such work should be for a specific purpose. He said that in those countries where the territory has been zoned for the requirements of earthquake resistant design and construction, it is recognised that there is a need to subdivide the country further into specific requirements for the nature of the soil and buildings. He discussed the damage which an earthquake can do to a building and said this could be from shaking due to direct ground motion, from subsidence or slipping of land or from the ejection of groundwater. He said that microzoning means one thing to a structural engineer and something entirely different to a scientist studying the problems of soil response.

Mr L.E. Oborn (Geological Survey, Lower Hutt) said that there are many variables to be considered in the ground conditions relating to an earthquake before the intensity of the earthquake itself can be applied to the building; these include the nature of the earthquake, its depth, the material through which the wave vibrations have to come and the filtering effect of various formations. He said the DSIR recognises the need to undertake seismic microzoning and has formed a committee with inter-Divisional and engineering representation. The initial aim of the committee is to look into the feasibility and value of microzoning the Wellington City area, the relative worth of various technical apparatus, instrumentation and historic record. He said instruments had been installed to record earthquake microseisms and strong motion accelerations; geological structures have been mapped and studied, stratigraphic boreholes are at present being drilled and ground water data is being collated. He said an attempt is being made to relate the effect of past earthquakes and structures and land surface to geology and soils data.

Mr Oborn said the immediate, relatively simple aim is to determine the intensity of earthquake vibrations at the ground surface in places underlain by different types and thicknesses of rocks and soils, different heights of water table and different subsurface topography. He said that while it might eventually be possible to determine the parameters listed by Mr Evans, it would not be possible to achieve them all at this preliminary stage.

Mr P.G. McA. Imrie (Kingston, Reynolds, Thom & Allardice, Wellington) considered that while microzoning is being done the major faults must be located as these greatly influence earthquake effects on buildings. He also referred to the potential hazard of loose fine-grained sands and silts below the water table. He asked Mr Taylor to comment on whether enough investigational work was being done for deep fills and whether a site's past history should, if possible, be reviewed.

Mr Taylor replied that frequently there was insufficient investigation done for bridge approach and other fillings; generally anything that was done was quite crude. He added that knowledge of a site's past history would be useful but that in most cases this was difficult to determine.

To further answer Mr Gilmour's earlier question on the number of bores, Mr Taylor said a typical number for a building 60 by 100 feet was four plus one for high quality sampling. He quoted a not so typical example of an investigation comprising eighteen bores, one for each of the proposed building columns.

Mr I.C. Smith (D.A. Stock & Associates, Christchurch) referred to Mr Taylor's paper and commented that the design information required of the structural engineer before site investigation commenced actually represented a very advanced stage of building design. He suggested that information on foundation conditions should be available before the design has reached this point.

Mr Taylor agreed with Mr Smith but added that even at an early stage, the designer should have some preliminary idea of loads to be supported by the soil.

Mr Richardson commenting on a previous remark about drilling equipment by Mr Gilmour, claimed that there are two drilling companies in New Zealand with a complete range of modern testing equipment; he said these firms do not receive any greater payment than firms who have no equipment.

Mr Evans replied to Mr Oborn's comment about what could be determined from seismic investigations by stating that accelerations, whole displacements and velocities were important in finding out what a site will do to a building.

Dr Falconer commented on Mr Oborn's earlier contribution. He expressed surprise that the probable end results of the seismic study are not known or set at this time. He said that intensities were of little direct use to designers, that studies should be related to the end use and there were two types of information to be obtained:

(a) those of long-term scientific value, and

(b) those of direct use to the designer. He claimed there was a greater need for co-operation between designers, the D.S.I.R. and other similar organisations.

Mr R.J.P. Garden (E.R. Garden & Partners, Dunedin) said that microzoning will be a long and protracted study before answers are known in any detail, but that a start must be made now, using rough rules; this has been done in other countries. He said that ground motion under earthquake can be magnified four to eight times by soft layers. He questioned how long it will take the committees to produce useful data and said he had served on similar committees several years ago. He contended that all filled areas which are suspect of magnifying ground motion should be rated with higher earthquake co-efficients until the Local Authority can prove by investigation that this is unwarranted. He also stated that ground strata which have a half wave length in frequency with earthquake motion will magnify that motion; from existing information on response spectra it is possible to show which frequencies of an earthquake are most severe. This would allow definition of areas likely to cause magnification and again such areas should be rated with higher earthquake co-efficients. He claimed it was a waste of time putting strong motion seismographs around the country until the characteristics of the site are known.

Mr Oborn reassured Mr Garden that there is co-ordination within the DSIR at the present time. To Mr Evans he said the search for intensities or accelerations is only the end of the first stage in the study; the current committee is mindful of the information that engineers need but this can only be developed in stages. To Dr Falconer he said it would be a bold man who publicly predicted the outcome of scientific research before the work was completed. He said it was essential in our geologically and seismically complex country to first establish whether realistic microzoning is feasible.

Mr R.O. Bullen (M.O.W. Wellington) said he agreed with much of Mr Taylor's paper but considered that site investigations could not be set into stereotyped phases. He referred to a flow chart (Figure 1) and suggested that it should be the basis of all investigations.

Mr J.P. Blakeley (University of Canterbury) said that too often engineers put down borings without having a clear idea of what they are looking for; in this case the art of subsurface engineering is completely lost and the investigation becomes a blindfolded chore. He stressed that the engineer must have a clear idea of why the investigation is being carried out and should have a well-thought out but flexible programme. He commented that whereas we strive to be economical in most other areas of civil engineering, in site investigation there is sometimes a strange lack of desire for economy or efficiency. He stressed that for better economy engineers must be actively interested in the work at all stages and must maintain effective communication with the driller.

Mr A.C. Arneson (Lyttelton Harbour Board) commented that he could find no mention in Dr Northey's paper of classifications of sample quality required for laboratory permeability tests. He said such tests were needed for the solution of problems such as de-watering, seepage and sand drain spacing and asked where such tests would fit on the classification charts.

Mr Thomas said that category I sampling would be required.

Mr Dodd stressed the need for special care in removing soil samples from the boring and asked Mr Thomas to point out the techniques needed to ensure the production of a truly undisturbed sample.

Mr Thomas replied that the sampling tube valve must be in good order; the sampling tube should be withdrawn slowly, after a delay and without twisting. He urged the use of tubes sufficiently long that the end thirds of the specimen could be discarded.

Mr Garden referred to the high standard of "no disturbance" specified in Dr Northey's paper. He asked whether a soil suffers damage in seisms and suggested that, if this were the case, then the care and expense of high quality sampling may not be justified.

Mr Thomas replied that insufficient was known of the effects of earthquake on foundation materials and that much more investigational work in the field was required.

Mr Bullen then replied saying earthquake vibration does affect soil for a certain amount of time or else there would be no failures after earthquakes.

Mr Taylor asked how it was possible to recognise disturbed samples. He said that refined techniques such as the Bishop sampler put the drilling cost up by a factor on the order of five and yet there must always remain some doubt about whether the sample is truly undisturbed.

Mr Thomas said that the only way of checking granular soils is by measuring the recovery ratio. For fine grained soils, a reasonable guide was the shape of the stress-strain curves.

Mr Evans said the classification of sampling techniques given by Dr Northey was good and recommended it be promoted by the Society. He said the problem of how many holes and how many samples had only been raised briefly and asked if there is some way of finding this out. He referred to the standard of sample required for shear and compression modulii and shear strength tests and discussed the effect on test results of local variability in soil conditions. He said the idea of qualification for drillers who are proficient in drilling and taking soil samples was a good idea but could be promoted only to keep pace with availability of good equipment and trained people.

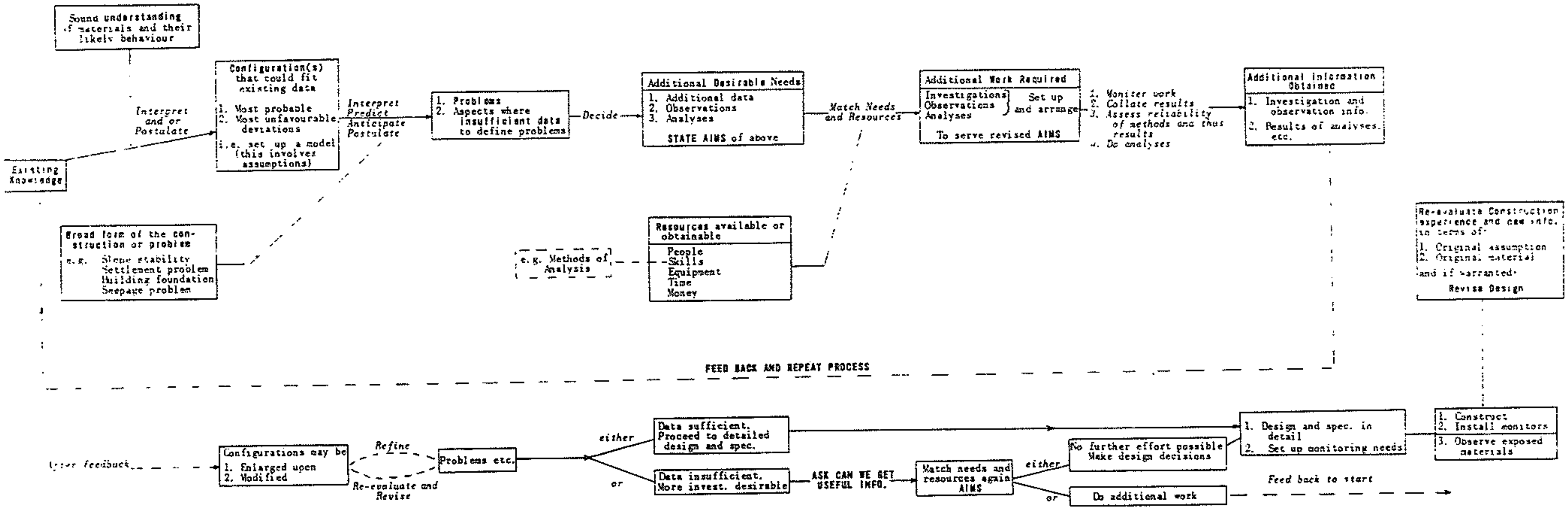
Mr Thomas said that soil rigidity would be expected to suffer much more from sample disturbance than would shear strength and therefore samples for shear strength analysis could be of a lower quality.

Mr P.J. Alley (Soil Mechanics and Foundations Ltd, Christchurch) commented on the term "disturbed" as used in Dr Northey's paper to describe a soil sample taken by an auger. He claimed that this term suggested too many unknowns and suggested it be replaced by "representative".

Dr Falconer wound up the session with a vote of thanks to the opening speakers, to Messrs Taylor and Thomas who presented the papers and to those who had contributed to the discussion.

MINISTRY OF WORKS
SITE INVESTIGATION

PREPARED 11 AUGUST 1955.
BY: R.D. BULLEN
Materials Engineer



GEOLOGY AS AN AID TO SITE INVESTIGATIONS

L.E. Oborn, Chief Engineering Geologist, Geological Survey
D.S.I.R., Lower Hutt

INTRODUCTION

Each building site is unique, geologically. The depths to the various geological formations, their thicknesses, attitudes, defects and compositions are unique. The problems they raise must also be unique and cannot, prudently anyway, be inferred from another nearby site. But no site is completely isolated, geologically, from all other sites. Every site must be regarded as another piece in a huge jigsaw that extends over a vast area of space and time.

FOUNDATION ENGINEERING AND ENGINEERING GEOLOGY

The main role of engineering geology in foundation engineering is to interpret the geology of a building site in terms of both the geology of the region in which it is situated, and the needs of the foundations of the building to be constructed.

Knowledge of New Zealand's regional geology is now sufficiently extensive to enable a generalised assessment to be made of the type of rocks that will be encountered in most parts of the country. Certainly enough is known to ensure that the correct questions are asked. This is an important first step. Knowing what the problems are likely to be, the next logical step is the planning and evaluation of a detailed site investigation to find the answers.

Geology certainly cannot answer all questions raised by foundation engineering; it does not provide numbers. But it does provide a philosophy, or an art if you prefer it, that gives an enhanced degree of confidence to the interpolation and extrapolation of numerical data from sampling points. It can also discuss, in the wider view, those geological hazards that must be allowed for in foundation and building design.

FOUNDATION ENGINEERS AND ENGINEERING GEOLOGISTS

For the foundation engineer to get the most out of the geologist he must consider him a full member of the team. Effective communication can only be established when there is both direct personal contact and interest, and mutual trust and confidence. Geologists are not foundation engineers, in fact they have much less engineering in their education than most engineers have geology. Where a geologist might see a major defect in a site, the engineer might see little of any significance. Conversely, a subsurface feature that the geologist feels is hardly worth commenting on or is of academic interest only, might be significant to an engineer. It is to avoid such misunderstandings that the geological background and significant features of the building should be freely discussed early in the planning stages.

Engineers and geologists cannot communicate effectively unless they speak the same language. Most geologists are aware of engineering terms, but through lack of continuous usage, are not always 'at home' with numerical values. It would be to the mutual advantage of both foundation engineering and engineering geology if there were to be a greater exchange and correlation of geological field and laboratory data with foundation engineering field and laboratory data. An ever growing body of knowledge of this sort would very soon enhance the value of the preliminary stage of investigations.

GEOLOGY IN PRELIMINARY SITE ASSESSMENT

An essential part of any investigation is to know what is being sought. An objective assessment of local geology can, in most places in New Zealand, reveal both the assemblages and sequences of rocks and soils that will be encountered, and the environments in which these were deposited, their post-depositional history, and from regional studies, the degree to which they have been tectonically disturbed. Armed with this knowledge, and a reconstructed geological history linking all events with time, it is possible to plan a realistic yet effective preliminary site investigation programme. The most appropriate drilling and geophysical equipment can be engaged to determine the depths to the various formations, their thicknesses and physical properties.

New Zealand has the doubtful honour of possessing an impressive number of natural foundation engineering hazards. Fortunately these do not all occur at any one site, but the country is so small that it would be imprudent to assume that any site exists that does not contain at least one of these. Geology can make an important contribution to the preliminary assessment stage of investigations by warning of the likelihood of encountering any of them, and of suggesting where and how test drilling and other forms of site investigation should be made.

The foundation problems that are met in New Zealand, and are directly attributable to the geological setting, are not necessarily unique to this country, but the assemblage of them into an area of about 100,000 sq. miles probably is. These geology-induced foundation problems can be grouped under four main headings:- materials, tectonism, subsurface topographic defects, and geological hazards.

Materials and Foundation Problems

New Zealand possesses nearly every known rock and soil type, from old deep-seated large-scale plutonic igneous rock (granites and the like) to historically deposited volcanic rocks, ranging from hard lava flows to weak troublesome volcanic ashes that have been deposited locally on irregular surfaces on a very small scale; and from fine sediments deposited in warm seas, to glacially deposited tills with 'boulders' up to 40 ft. Changing sea levels have repeatedly caused rivers to cut down their beds near the sea, only to have these drowned and back filled with soft weak sediments as sea levels once again rose. Estuaries have been formed in many places around our coasts, and these, being sensitive to minor sea-level fluctuations and climatic variations, have their history recorded in the laminated beds and organic sediments deposited. Lake shores, especially those of glacial origin, pose special foundation problems by their extreme and unpredictable variations of soil types over short lateral and vertical distances.

The rocks and soils in our geological record were deposited in times when depositional environments differed markedly from those of today. At any one site the changing temperatures and sea depths of past ages have had a significant effect on the materials into which building foundations are constructed.

New Zealand has for a very long time been a narrow island elongated north and south, spanning a wide range of latitude, and this has largely controlled the type and degree of weathering that has altered our foundation materials. In the north, chemical weathering is severe, as a result of subtropical environments, but in the south, it is minimal, especially the alpine regions, but mechanical weathering is especially severe. Widespread volcanism in the central and northern North Island has resulted in many sources of abundant supplies of hot mineralised solutions that have changed rocks and soils by hydrothermal alteration to materials possessing new and vastly different physical properties.

Tectonism and Foundation Problems

The rocks and soils that make up the New Zealand landmass have not been static since deposition. They have been continuously warped, folded and faulted, both in the short term by small-scale movements, and in the long term by massive large-scale deformation. These movements are still taking place. Each period or episode of movement or regime of applied stress has left some record in the rock mass. These effects are regional rather than local, and can be extrapolated from known outcrops to similar rocks of the same age occurring at building sites.

Regional folding and warping, even if this took place a hundred million years ago, can adversely affect the stability of a building site on sloping ground, or the slopes of large foundation excavations. Young unconsolidated soils are little affected by this very slow process of folding.

The folding not only re-orients pre-existing planes of weakness (e.g. bedding) but also, because of changed stress distribution, induces forces that produce new defects. The joints formed in this way, and the zones of shattered and crushed rock that result from intense folding of competent and incompetent beds, reduce the strength of the rock material and cohesion of the rock mass to a greater or lesser extent. To some extent the attitude of the planes of weakness in a rock affect the loading that can be safely applied to it.

Regional joint and defect patterns are very complex and variable in our 'basement' rocks, and usually it is unwise to extrapolate these, but where well-defined persistent patterns can be recognised, these can be used to determine preferred orientations of excavations that are likely to have reasonably stable walls. This information is of value only in those sites where a freedom of choice of site orientation exists.

Little imagination is required to picture the chaos that would accompany direct rupture of a building sited on a fault that moved. More is required to assess the likelihood of movement before it occurs. Few engineers would knowingly build an expensive or important structure on an active fault trace. But fault traces are sometimes very difficult to distinguish from small terraces. More often than not in built up areas a trace might not be visible at a site; it might, for example, have been removed in earlier days of land settlement. Possibly the site lies on the extension of a fault trace, and this might in fact be the section of the fault to move next. Geologists look at a building site, not only as the small area on which a building is to sit, but also in the wider view of how it fits into the regional geological setting.

The geologist's inability to put figures on his statements is a continual source of frustration to him as well as to engineers. He cannot tell when a fault will move again. He can, using a crude probability approach, suggest that a fault which has moved repeatedly in the recent past, for example, in the last 5000 years, can be expected to move again, possibly within the life of the structure. But he is always sobered by the knowledge that on the day before the Murchison earthquake he would not have described as active the White Creek Fault which moved about 14 feet.

Less dramatic, but of far greater significance to the country, is the effect that earthquakes and fault movement vibrations have upon natural materials on which buildings are constructed. Geology has much to contribute, along with many other related disciplines, to the study of the relation of local geology and seismic accelerations at foundations. At present a geologist can comment in a subjective way only, basing his opinions on his assessment of the surface and subsurface geology and topography.

Faults, whether recently active or not, have disturbed the rocks they cut. Hard rocks are usually shattered and crushed, and in some places wide zones of weak slightly plastic gouge are produced. The degree of disturbance produced by past fault movements, and the direction of strike and dip of the faults, can usually be judged by observing a fault zone along its extension. Geological mapping can usually predict whether a known fault passes through a building site. The site investigation programme should delineate fault lines and examine fault zone material if this promises to be significant to foundation design. Fault zone material can permit differential settlement if sufficiently crushed and plastic, and can cause slope stability problems in excavations. Ground water rising along shatter zones or draining higher ground can prove troublesome, especially if it is high in sulphate dissolved from sulphide vein minerals.

Subsurface Topographic Defects

Many present day land surfaces successfully obscure site defects by providing them with a covering blanket of young sediment. These defects include the buried river and stream channels that are roughly parallel to, but distant from present river beds, and those, submerged beneath harbours waters or coastal lowlands, that are back-filled with weak soft estuarine and marine sediment. These buried channels pose problems where depths to suitable foundations prove to be deeper than expected, and where the young back filling is compressible.

Foundation problems can be encountered where steeply dipping and well compacted strata are overlain by thickening weak material, and where the upper surface of the underlying resistant formation is irregular.

Inland, on the hillsides and rolling country, subcutaneous erosion, and solution and erosion cavities, provide additional natural foundation hazards. The possibility of mine subsidence is ever present in mining districts. These various site defects cannot be pin pointed by preliminary geological assessment, but because they seldom occur without some expression elsewhere in a region, geology can give a warning that they might occur, and can recommend appropriate site investigations.

Geological Hazards

Foundation engineers working in New Zealand must always be mindful of the four major 'geological' hazards, and where appropriate, design for these. These are:

Volcanism, including hot water and steam, which at its best constitutes a nuisance, and at its worst, could with inappropriate precipitate remedial measures, trigger off a major disaster. Regions in which these problems can be expected are well known.

Tsunamis could inundate any of our low lying coastal lands, but regions where the risk is most serious and where the size of a wave is likely to be greatest are reasonably well known. The magnitude of the wave is not predictable.

Active fault movements and earthquakes constitute a serious hazard over a widespread area.

Geology can help define the regions in which these various hazards are likely to be serious, and can perhaps assess the order of seriousness, but it cannot quantify risks, probabilities of occurrence, frequency or magnitude.

GEOLOGY IN PLANNING SITE INVESTIGATIONS

The extent and pattern of site exploration must be dictated by the local geology, and no rules can be formulated or directives-to-geologists be suggested that can have universal application.

In site investigations geology is a tool for engineering. Only those geological problems need be solved that will help solve engineering foundation problems. No full scale geological investigation is needed, and engineers need not fear that geologists will ask for one. The paucity of geological data obtainable from a building site, however, usually necessitates that the geology of the countryside surrounding the site be looked at, although in many places this will be well known, and sometimes that large diameter or angled drill holes be sunk. On rare occasions it might be important to have a hole sunk, or geophysical work done, beyond the boundaries of the site to gain some critical geological information. These requests should be considered seriously. Geologists should give, and engineers can expect, good reasons why some different approach is suggested. A geologist is amenable to advice and welcomes constructive suggestions on how to achieve the results he is seeking.

Once the preliminary geological assessment of a site has been completed, and some, if not most of the likely defects have been indicated, it remains for detailed investigation to confirm geological interpretation, and to examine further those defects that the engineer considers relevant to his foundation design.

The broad geological interpretation can usually be verified with a few holes to check the rock sequences and their compositions, the strata thicknesses and attitudes, the position of critical formation boundaries, and the locations of the principal geological structural features. These holes will reveal the degree of uniformity or variability of the subsurface material, and will suggest the density of test holes needed to give adequate and reliable data. One or more of these holes should yield information on the depth to the water table, hydrostatic head, and the rate at which an excavation is expected to make or lose water. Test drill holes can be used as observation wells in which to observe fluctuations of water level.

Continuing geological oversight and flexibility of drilling contracts are essential if effective geological investigations are to be made at lowest costs. The number of holes drilled and the depth to which these are to be sunk, should be able to be changed as new data becomes available.

GEOLOGY IN EVALUATING SITE-INVESTIGATION DATA

Geology's contribution is mainly in those initial stages of an investigation that lead up to the selection of the best type of foundation for a particular site. Once the type of foundation has been decided upon, samples from drillholes at structurally important positions of a proposed building will give the physical properties needed.

The information required in the first phase is a three dimensional assessment of the distribution of physical properties of the rocks and soils at the site. It is doubtful if this is ever produced directly from borehole data, as the laboratory testing of samples provides no criteria, other than from statistical methods, of extrapolating beyond the samples tested. What is feasible is a three dimensional geological map, where the density of holes is adequate, showing the ranges of physical properties of the various formations encountered. Geology expects, from surface and subsurface exploration (down-hole inspection or viewing, open cuts and tests boreholes), to be able to identify each formation present, its position, thickness, attitude, composition and order of variation, and from physical tests, its physical properties. Geology expects, also to be able to indicate with some accuracy the locations and attitudes of major geological features, including folds, faults, joints and crush and shatter zones. Excavations and cores must be logged and photographed carefully by geologists familiar with the region, and observations must record in detail the rock type and sequence, geological structure and structural defects, and groundwater information (depth to water, water-level fluctuation, water loss). When necessary samples should be taken for petrographic examination as these will aid correlation within and beyond the site, and will detect deleterious minerals. Deleterious minerals likely to be present at a site vary from one geological setting to another, but possibly of greatest importance are the clay minerals in joints, crush zones, altered rock material, and moisture-sensitive soils. Zeolite in veins, and to a lesser extent minerals that are soluble in water at site ground-water temperatures, can also cause foundation problems. Excavation and drillhole logs should be prepared on suitable log forms and presented together with all other geological and hydrogeological maps and cross sections that show factual information only. Interpretive maps and sections, drawing data both from the factual maps and logs, and from regional mapping, should be presented on a different set of maps.

SUMMARY

Geology can enhance preliminary site assessment by drawing relevant data from geological mapping beyond the site as well as at the site, and from records of published geological work. From the geological record it is possible to deduce past geological environments, and infer which site defects are usually associated with these environments. Rock and soil sequences and geological structures can be postulated, but thicknesses and local variations and textural changes cannot be determined unless the geology is unusually simple and uniform, or unless the density of exploration holes is large. Geology cannot give quantitative answers, but it can provide, in most places anyway, an assessment that enables data obtained from materials testing to be extrapolated over the area of site. Site investigations can then be designed to test relevant geological hypotheses, and to obtain the numerical data required for foundation design.

The geologist should be able to recommend, and preferably initiate and change, drilling programmes, when this is necessary to ensure effective and lower cost drilling. Drilling contracts should be sufficiently flexible to make this possible.

Drill cores and all excavations should be logged and photographed, and factual and interpretive maps and sections produced. Rock and soil defects, especially clays, and water sensitive and deleterious minerals should be examined and their likely effect on foundations assessed.

Geological judgements can be only as good as the basic data on which they are based. Where this is scanty, or where the variability of stratigraphy and geological structure is large, orders of confidence in hypotheses are low.

The geologist should be a member of the investigation team if he and engineers are to communicate effectively. Geologists cannot do the impossible. Nor can they produce meaningful, instant, verbal, geological site reports. They can only give their best assessment based on the evidence available. Because the amount of evidence can only rarely be adequate, the geologist's judgement on when to ask for more investigations must be tempered by an appreciation of the engineer's needs. And the engineer, too, requires to appreciate the geology of the site well enough to help the geologist in deciding when the return from further work does not warrant the cost - and time - involved.

GEOPHYSICAL METHODS OF SITE INVESTIGATION

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INTRODUCTION

All of the main techniques of geophysics have been used in site investigations. In order of importance these techniques are:

Seismic refraction surveys;
Electrical resistivity surveys;
Magnetic surveys;
Electromagnetic surveys;
Gravity surveys.

Equipment for working with all of the above techniques is available in New Zealand where, with the exception of gravity methods, all have been applied to site investigations. The extent to which this is done is, unfortunately, limited by a shortage of experienced staff.

The few items of equipment designed specifically for site investigation are quite inadequate and it is fortunate that sophisticated equipment designed for other exploration purposes has proved suitable for work at the relatively shallow depths required in site investigations. Not only are the multi channel seismographs, gravity meters and magnetometers, marketed to serve the needs of the oil industry, giving satisfactory service but also much of the technical literature published in that connection is applicable to site investigation problems. Much the same can be said of electrical prospecting equipment and the metalliferous mining industry.

SPECIAL REQUIREMENTS OF GEOPHYSICS APPLIED TO SITE INVESTIGATION

The need for fine detail and for accuracy are the main differences between geophysics done for site investigation and geophysics done for other purposes. A third difference is that site investigations are precisely related to a planned engineering structure and information must be sought where the engineer needs it regardless of surface topography, cultural development or other complications. In most non-engineering geophysical surveys it is possible to plan the fieldwork in a way that avoids gross surface irregularities and built-up areas.

Lastly, site investigations are usually confined to shallow depths though this is by no means always the case.

CHOICE OF METHOD

Site investigation problems which can be solved by geophysical methods fall into two groups. Those in the first group are concerned with the measurement of physical properties of the surface rocks. Are the rocks strong enough to support a heavy structure? Are there zones of weakness? Will the rock be rippable without explosives?

Problems in the second group involve the mapping of sub-surface interfaces. These are usually the contact of alluvial deposits and bedrock but may also be the water table, or the contact between different rocks brought into juxtaposition by fault movement. Geophysical methods for solving these problems are chosen after consideration of the contrasts in physical properties on either side of the discontinuity being investigated. The seismic method is applicable only where there is a seismic wave velocity contrast at the surface to be mapped; electrical methods require an electrical resistivity contrast; magnetic methods a contrast in magnetisation and gravity methods a density contrast.

Where it can be used, the seismic refraction method is usually first choice for site investigations. It is the only method where ray theory is applicable and because of this it can be made to yield finer detail in geological structure than any of the other methods. All of the other methods which depend on potential field theory involve excessive computation if the geometery of the discontinuity is not very simple. Operational characteristics of the various methods can be taken into account but the final choice of a geophysical method is often dictated by which physical properties offer a working contrast at the discontinuity which is to be mappped, and a geophysical sampling followed by a re-definition of the problem is often necessary.

The literature provides more assistance in the interpretation of electrical resistivity surveys than it does for other methods depending on field theory, and for this reason resistivity surveys are usually the next choice when circumstances are unfavourable for a seismic survey. Electro-magnetic methods are less used than DC methods but in a limited number of cases they give results that are equivalent to those of resistivity surveys. Magnetic methods are not suitable for detailed surveys, but are often used in reconnaissance work.

Gravity surveys are little used in engineering work. Even in those applications where the emphasis on accuracy is not great the pre-requisites for a gravity survey are rather onerous and if some other method is possible it is usually preferred. Both magnetic and gravity surveys are sometimes used as additional information.

SEISMIC REFRACTION METHODS

All seismic techniques depend on the fact that seismic waves travel through different rocks with different velocities. The velocity of a seismic wave depends on the elastic constants and density of the rock in which it travels. A rock boundary can only be located if there is a velocity contrast between elastic waves travelling on either side of it. It can only be located by the refraction technique if the velocity on the lower side of successive interfaces is greater than that on the upper side.

When seismic waves generated by a hammer blow or an explosion in the ground travel to a remote geophone, the path followed by the first arriving energy is such that its travel time is a minimum. As the path is lengthened the rays will traverse the upper surface of successively deeper layers in the ground. There is a range of path length over which first arrivals at the surface will have traversed a particular layer.

If the layers are not parallel it will be necessary to interchange the position of the shot point or hammer blow and the geophone. The principle of depth determination can be easily seen from Figure 1. In Figure 1(a) G is a geophone and A and F points at which seismic waves are generated. If the travel time for the ray path ABEF is subtracted from the sum of the travel times for ABCG and FEDG we get the travel time for CG + DG or approximately the travel time for 2DG. Provided that an adequate range of source-detector distance has been used, the velocities V_1 and V_2 can be determined so that distances DG can be converted to depths; usually depths perpendicular to the refractor.

Extending the range of source distances enables the depth of successive layers to be determined. If figure 1(a) had been extended to include the more remote source points M and V then the ray paths would be as shown in figure 1(b) and the travel time for the parts of ray paths PG + SG = 2SG could be determined. This can be converted to a depth at G if the depth of the upper layer has been previously determined as in figure 1(a). By successive applications of this process it is possible to obtain the depth of the nth layer.

In practice a line of geophones called a spread is laid on the ground. Shots are fired or hammer blows made at each end of it and at suitably spaced intervals along it. In this way impact distances suitable for determining the depth to successive layers are provided in relation to each geophone. For every impact the ground motion at each geophone is recorded separately on paper; this record is called a seismogram. It is thus possible to read the travel times of initial wave fronts arriving at each geophone. These are commonly known as "first arrivals" and are normally the only information used in shallow refraction surveys.

SURVEYS WITH SINGLE CHANNEL SEISMOGRAPHS

Instruments of this type and some shallow resistivity measuring equipment are among the few geophysical appliances designed and marketed largely for engineering purposes. The seismographs have only one geophone which is left stationary while the energy source, usually a 10 lb hammer, is moved out to successive points on the spread. In some instruments the travel time appears on a digital read-out, in others the source-detector distance is pre-set on a dial before each hammer blow and the data are presented as a plotted time-distance curve.

Inadequacy of the hammer as an energy source is the basic limitation of the simple seismic units. The adverse effect of the initially low seismic energy adds to the inability of such devices to time signals that are not well above ground noise level. The timing and presentation system used also introduces the possibility of timing something other than the first arrival.

Problems for investigation with the one channel seismograph must be chosen with due regard for both the required depth penetration and for the velocity structure. If the surface layer of the ground has inferior elastic properties, much of the hammer's energy will be lost in non-elastic deformation at the point of impact. If on the other hand the wave velocity in the surface layer is too high the velocity contrast with the lower layer will be reduced and the spread length for a given depth of penetration will be inordinately long. Experience has shown that even in good conditions, 600 ft is about the greatest distance over which the seismic arrivals from a hammer blow can be consistently timed. Even in the optimum case of two horizontal layers this corresponds to a depth penetration of 100 ft for a 5:7 velocity contrast at the interface or 140 ft for a 1:2 contrast. Makers claim achievable depths ranging between 75 ft and 200 ft or more. To achieve the latter without exceeding the 600 ft limit for the source-detector distance would require a velocity contrast of 1:2.5. In order to get good conversion from the kinetic energy of the hammer to seismic waves the surface layer might have to have a velocity of 6.0 ft/msec. If the limit of 600 ft for the source-detector is not to be exceeded this implies a velocity of 15 ft/msec for the bedrock and such conditions are seldom met in practice.

Part of a survey recently carried out for the Tongariro Power Development seemed to offer suitable conditions for a hammer seismograph survey. The problem was the determination of the gravel cross section at the point where the proposed Moawhango tunnel route passes beneath the Waipakihi River bed.

The water table was virtually at the surface, the gravel was compact, having a velocity of 6.9 ft/msec, the bedrock had a velocity of 14.0 ft/msec and was for the main part less than 100 ft deep. Even here an early survey using the hammer seismograph had failed. In this case it can only have been that the river noise was too great for the use of the hammer as an energy source.

Several of these single channel seismographs are available in New Zealand and a number of surveys have been made by Universities and by Ministry of Works. Of the last 23 surveys made by the N.Z. Geophysical Survey none could have succeeded had an instrument of this type been used. The fact that N.Z. Geophysical Survey has not done, or been asked to do any jobs where single channel equipment would suffice is probably an indication that in the range of problems where one could be used, engineers prefer more direct investigation methods such as test pits, auger holes and dozer cuts.

SEISMIC SURVEYS WITH MULTI-CHANNEL EQUIPMENT

This type of equipment usually provides for the simultaneous recording of ground motion at 24 geophone points. It is superior to the single channel equipment in every way though in engineering work most of its advantage is gained from using one ground impact and 24 detecting points instead of 24 impacts and one detecting point, thus making the use of explosives as an energy source economically reasonable. In this way waves can be generated which are capable of travelling much greater distances before their amplitudes are reduced to noise level.

In the case of explosives the seismic efficiency, usually quoted as the ratio of seismic wave energy to total energy released, has been given in the literature as 4% for sandstone, 9% for small explosions in clay and 10% to 18% in granite gneiss.

Using the lower figure of 4% and 1.6×10^{13} erg/lb as the total chemical energy of explosive we get about 6.4×10^{11} ergs of seismic energy per pound of explosive. In the case of a 10 pound hammer swung through an arc with 5 ft radius and reaching a maximum velocity of 5π radians/sec the total kinetic energy available is about 1×10^7 ergs. No figure comparable to the seismic efficiency for explosives can be found in the literature but even if this is taken as 100% for the hammer we still have a ratio of $1:10^4$ between the ideal case for the hammer and a conservative one for 1 lb of explosives.

Seismic waves with sharp onsets can be produced at distances of 2 or 3 miles with moderately sized explosive charges. This compares with 500 or 600 ft with the hammer.

The futility of using a hammer in poor conditions is illustrated by the seismogram shown in Figure 2(a). This seismogram was made with 50 ft geophone intervals and a charge of 50 lb of gelignite in an auger hole 12 ft deep.

First arrivals are marked, these are upward on the top twelve traces and downward on the lower twelve, the difference being due to electrical connections. For comparison the seismogram in figure 1(b) was made with 1/3 lb of gelignite in a hole 4 ft deep. The geophone intervals and amplifier gains were the same as in the first case.

The 1st shot which gave sharp arrivals for only 475 ft was fired in dry pumice (v_o = 1.2 ft/msec) overlying wet pumice(v_1 = 4.6 ft/msec) which was in turn underlain by a tight andesitic gravel (v_2 = 10.4 ft/msec) at about 300 ft. The second shot was fired in clay soil (v_o = 5.3 ft/msec) with a thickness of about 20 ft, overlying greywacke (v_1 = 12.5 ft/msec). In the latter case, although only 1/150th of the charge was used, sharp arrivals could be observed along the total spread (length 575 feet).

Recent Seismic Surveys

Work on the Tongariro Power Development has provided some interesting problems in seismic surveying.

Special problems are created by the dry pumice layer (see seismogram Fig. 2(a)), thicknesses in excess of 100 ft, which often exists above the water table. In many places, probably where it is driest, the velocity in this material is lower than sound wave velocity, and unless special precautions are taken to damp the sound wave the pumice velocity cannot be measured. In addition to its velocity being difficult to measure problems arise from the high losses that occur when seismic waves traverse this loose pumice. Very large shots must be used to compensate for the energy loss and this leads to very high ground amplitudes at the close in geophones. The close in phones have to be electrically damped to prevent them from generating voltages that are too high for the insulation in the geophone cable and plugs. The latter problem could be overcome by placing the charges beneath the dry pumice with a drill; but this has the disadvantage of giving very little information about the dry pumice. Because it has a very low velocity any wrong assumptions about its thickness lead to large time errors which are reflected as large depth errors in lower layers.

The presence of the pumice layer has led to high explosives expenditure; more than 6,500 lb of gelignite having been consumed since work on the T.P.D. started. Although charges in excess of 100 lb are frequently necessary, the economics are strongly in favour of surface shooting, even on the basis of drilling cost alone. If drilling were done, a good deal of roading would be necessary; this roading would load the drilling cost and would lead to more damage to the countryside than isolated shot-holes. For much of the time it has been possible to offset a good part of the explosives cost by using two 24 channel seismographs at the same time.

Another problem frequently encountered is connected with a high velocity andesitic gravel bed that frequently overlies the greywacke in the area. A typical velocity column in the parts that have been investigated is:

Material	Seismic Velocity ft/msec	Thickness ft
Dry pumice	0.6 - 1.2	0 - 150
Wet pumice	4.6	0 - 300
Andesitic gravel	9.0 - 10.5	0 - 100
Greywacke	15.0	-

The andesitic gravel has often been about 40 ft thick in which case it never provides first-arrivals on the seismograms when near surface shots are used. It can only be detected by the use of shots in drillholes or in the bottoms of river valleys where much of the surface material is missing. In other cases the gravels are thick enough to provide first-arrivals, but only over a very limited range of shot-geophone distance. The required distance varies quite irregularly because of the steep topography on both the ground surface and the greywacke surface and it is necessary to "feel" for this layer by the use of shots at varying distances.

The last point of interest concerns zones of weakness, probably crushed zones, along faults in the greywacke. One of these was discovered along the axis of Paradise Valley. This is a deep valley which is crossed by the Moawhango Tunnel route. It was known to be filled with gravel deposits overlying tertiary beds. The thickness of these beds was not known and the original purpose of the seismic survey was to check that the tunnel route did not pass through the gravels. A "low velocity" zone (11.0 ft/msec) about 600 ft wide was discovered in the greywacke (15.0 ft/msec). Useful information might be obtained by making a seismic search for similar features along those parts of the tunnel route where access is possible.

Rock Properties Using Surface Waves

A number of in situ determinations of Youngs modulus and rigidity modulus have been made by seismic methods in the past. This has entailed the determination of rock density, P wave velocity and S wave velocity. S wave velocity has always been difficult to determine. Explosions are not a satisfactory source of S waves and the non-explosive sources that are used are all weak. For these measurements to be made therefore it was usually required that the rocks in question be exposed either at the surface or in a tunnel. Ray paths long enough to reach rocks even at moderate depth could not generally be used.

The above difficulty has been overcome by Prof. F.F Evison of V.U.W. who has determined the S wave velocity from the dispersive characteristics of Rayleigh waves travelling in the surface layer. Rayleigh waves develop and travel in the surface layers by a process of multiple reflections.

Surface waves account for about 2/3 of the total energy radiated from a surface source. They have two further advantages in that being radiated within a cylindrical envelope with depth equal only to the thickness of the surface layer they give higher energy densities than the S wave which is radiated over a full hemisphere; having longer periods they suffer less absorption from scattering than do S waves. It is thus possible to measure large Rayleigh wave amplitudes at considerable distances from weak sources. Curves and tables giving the dispersive charatteristics of Rayleigh waves for a single overburden layer and substratum have been published. These curves relate period T, group velocity v and phase velocity c of the Rayleigh waves to shear velocity, compressional velocity, density and Poisson's ratio in the two media. If rock density, group velocity and period of the Rayleigh waves are measured it is possible to determine S velocity for both the surface layer and its substratum. A three layer problem could not be handled without difficulty.

ELECTRICAL METHODS

Electrical methods can be divided into those which use DC and those which use AC current sources.

Direct Current Methods

Current is impressed into the ground between two widely spaced electrodes and the resulting potential drop is measured at two intermediate electrodes. The configuration of the electrode array is maintained constant and the whole array moved along a traverse line if lateral changes in ground resistivity are being mapped; or the array is expanded about a fixed point if changes with depth are being measured. The plot of measured resistivity against electrode separation in the case of vertical sounding, or resistivity against distance along traverse in the case of horizontal profiling, is compared with families of theoretically produced curves which have been previously prepared for various geophysical models. Theoretical curves for vertical and dipping discontinuities, dykes, laminae, buried spheres and horizontal layering are available in the literature.

For the simplest possible case, that of determining the depth to a horizontal substratum covered by a single horizontal overburden layer, the accuracy that can be obtained from a resistivity survey is 10%.

One recent example of the use of a resistivity survey for site engineering was the investigation of the true and perched water tables beneath the planned Pukaki dam.

Alternating Current Methods

The electromagnetic methods are less used in site engineering than D.C. methods and the theory necessary for their quantitative interpretation is not as well documented.

As their frequency is increased electromagnetic waves travelling in the ground remain closer to the surface and because of this effect it is possible, using A.C. methods, to keep the source-detector separation constant and to obtain a depth sounding by changing only the frequency. This can in some circumstances lead to a simplification of the field work.

MAGNETIC AND GRAVITY METHODS

As applied to site engineering, both of these methods tend to give only background information.

The magnetic method can be used to estimate the depth and shape of buried bodies of magnetic rocks; but it is susceptible to errors from magnetic inhomogeneities near the surface and, in general, will give neither depth control nor resolution of fine detail that is sufficient for site investigation purposes. Because the measurements can be made quickly and cheaply magnetic surveys are often used to give a first indication of the presence of igneous or other magnetic rocks.

A gravity survey was recently used to determine overburden thicknesses in the Wellington City area. The information was required in connection with an attempted micro-zoning for earthquake effects. An accuracy of 50 feet in overburden depth has been claimed.

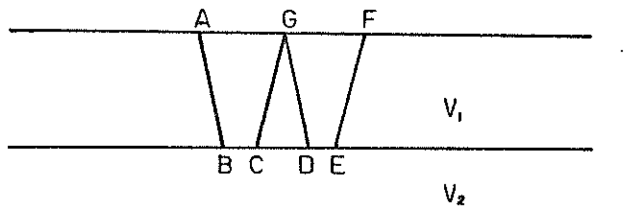

Figure 1a: Ray paths for a single overburden layer

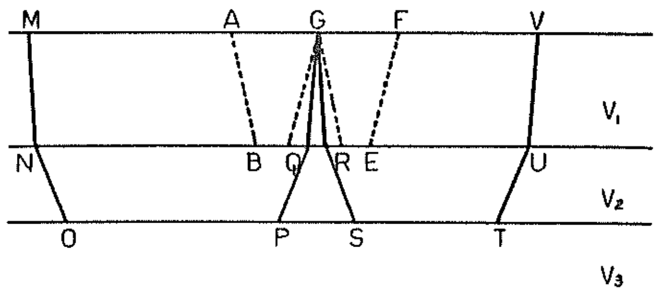

Figure 1b: Ray paths for two overburden layers

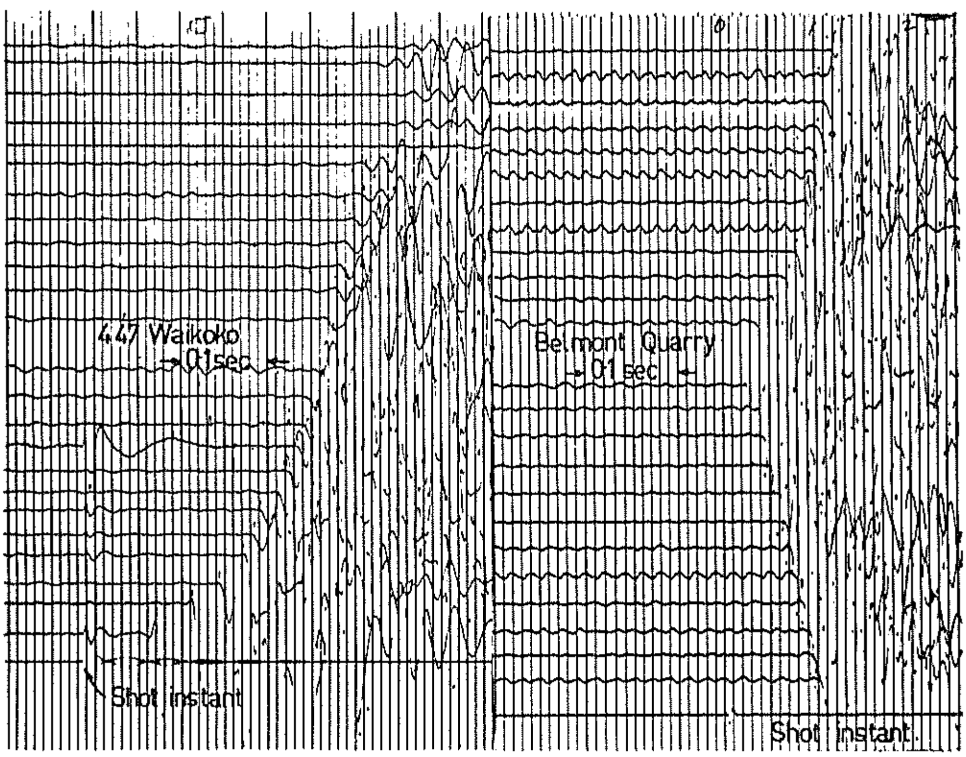

Figure 2a. Geophone spacing 50 ft. Charge 50 lb. Shot depth 12 ft.

Figure 2b. Geophone spacing 50 ft. Charge 1/3 lb. Shot depth 4 ft.

INTRODUCTION OF PAPERS, SESSION 2

CHAIRMAN:

Mr G.D. Mansergh, Geological Survey, Christchurch

GEOLOGY AS AN AID TO SITE INVESTIGATIONS by L.E. Oborn

Mr Oborn introduced his paper by describing how a geologist must study not only the particular site in question but also the surrounding geological environment and the regional geology. He described the types of references and geological maps available for New Zealand. He went on to discuss geological site problems including subsurface topography, slope stability, ground subsidence, ground-water fluctuations and variability of materials. He also mentioned geological hazards such as active faults, volcanicity and Tsunamis. The introduction was illustrated by a series of slides.

GEOPHYSICAL METHODS OF SITE INVESTIGATION by C.E. Ingham

Mr Ingham introduced his paper with an outline of the availability of geophysical methods in New Zealand. He then went on to discuss a new development in which surface waves are used for shear wave determination and hence to evaluate the elastic modulus. He said this was a completely new concept being developed in New Zealand and predicted that although it requires considerable experience and computation, it will be used more in the future.

DISCUSSION, SESSION 2

Mr T. Belshaw (MOW Napier), in a written contribution, said that he had tried a single-channel seismograph in Napier at a point where mixed gravel overburden, with water-table at 8 feet, overlay bedrock at depth 30 feet. He claimed the test was a complete failure and asked Mr Ingham whether the advertising claims of the makers of such instruments were completely misleading.

Mr Ingham said that claims were not entirely false and that they probably can be achieved somewhere but do not have general or easy application.

Mr Cornwell said his laboratory operated a single-channel seismograph and found it a very successful instrument when used sensibly and with recognition of its limitations. He suggested maximum depths of 60 to 100 feet could be explored provided there was no outside noise. Thus their application belonged in rural areas rather than in cities.

Mr R Gilmour asked Mr Oborn whether engineers need to have a fuller course in basic geology with particular reference to a basic knowledge of local stratigraphy. He said that the average engineer does not seem aware that the geologist exists and claimed that all engineers should feed back all information on subsurface exploration to the nearest geologist so that a comprehensive picture could be built up. He said that organised recording of such information was being carried out in Auckland and Christchurch and called for other branches of NZIE to set up similar organisations.

Mr Oborn said he also would welcome much closer communication between geologists and engineers. He suggested this could be achieved by more engineers participating in the Geological Society of New Zealand and detailed a current proposal to form a branch of the International Association of Engineering Geologists.

Mr A.H.C. Smirk (Dunedin City Corporation) agreed that the depth of penetration and general usefulness of the single-channel seismograph was limited by the energy input but that the use of a hammer as an energy source gave advantages as well as disadvantages. He said the fact that explosives are not required greatly simplifies the field operation. Mr Smirk gave details of his experience in two seismic surveys where different instruments and techniques were employed. The first survey was of a site located one mile from the nearest vehicle access and was easily handled by three men. The second site was in a more accessible area but the use of explosives necessitated a five-man team, including a qualified shot-firer. He said that this traverse, probably more difficult than normal, had cost 18.3 cents per foot of length one way and that 50% of this cost was attributable to the use of explosives. He said that the use of multi-channel equipment had reduced the amount of field work but claimed that without explosives the cost would have been much less.

Mr Smirk went on to discuss detections of the subsurface boundaries of a low velocity material sandwiched between two layers of higher velocity material. He said that single-channel seismographs of the chart printing variety record not only the first arrival of the shock wave but also the arrival of all positive waves above threshold value. He claimed that a careful inspection of "late" arrivals can apparently reveal the presence of a low velocity layer.

Mr Ingham agreed that in cases where it can be used, the single-channel seismograph was a good method but said there were not many such cases. He said that multi-channel equipment is more expensive to operate than single-channel except when explosives are used with the latter. He agreed with Mr Smirk that velocity inversions can be measured but said this was by reflection techniques rather than refraction. He said reflection techniques were not generally applicable to site engineering as it requires strata depths of some hundreds of feet. Reflection from shallow boundaries has been achieved with high frequency waves but this is not a common technique.

Mr Blakeley spoke to Mr Oborn's paper, discussing the evaluation of the engineering properties of rock masses. He said that tests on laboratory specimens are not a great deal of use because the properties of the rock mass will be more a property of the discontinuities in the rock than of the intact rock. He mentioned two recent methods used for evaluation and described in the text "Rock Mechanics in Engineering Practice" by Stagy and Zienciewicz. The first method called Rock Quality Designation (RQD) is based on a modified core recovery procedure in which all pieces of core less than 4 inches long are neglected. The lengths of core remaining are summated and expressed as a percentage of the total footage drilled and this is called the RQD. Mr Blakeley said this method is currently being widely used throughout United States. The second method, called "Velocity Ratio", is a comparison of the in-situ compression wave velocity V_F to the velocity of compression waves V_L measured in the laboratory on intact core. The laboratory test must be performed with an axial stress equal to the specimen's natural overburden stress. Mr Blakeley went on to say that initial data indicates that the square of Velocity Ratio may be used interchangeably with RQD. He pointed out that RQD would generally be the less expensive method.

Mr Mansergh said he had viewed cores from the South Canterbury area where no core lengths were greater than 4 inches long. He expressed the opinion that the characteristics of a rock joint such as whether coated with clay or platey minerals or is naturally rough and the areal extent of the joint surface is of far greater significance than the intensity of jointing.

Mr Evans asked Mr Oborn to define an "active" volcano.

To Mr Ingham he said that some investigators using laboratory tests have found that the velocity of waves in soils is influenced by the amplitude of the wave motion. He referred to tests where a change in strain caused by a larger amplitude had influenced the wave velocity by from 10 to 15%. He also said that other studies on stress wave propogation have found that the rate of strain (frequency) and the amount of strain (amplitude) both affect velocity. He suggested that changing the amount of explosive used in a field test may generate waves of different amplitude and asked if any such changes had ever been detected.

Mr Oborn said that an active volcanic district is one where the period since the last eruption is shorter than the period between earlier eruptions.

Mr Ingham could not see why shear wave velocity should change with amplitude - he said it is not supposed to. The measurement of shear wave velocity is very difficult because it is difficult to interpret between shear waves and Rayleigh (surface) waves. He said that the velocity changes with wave period and, for a weak energy source such as a hammer blow, the short wave periods are attenuated quicker than the long periods due to frictional losses. Thus the period being measured will change as the detector moves away from the source.

Mr Alley asked Mr Oborn if there had been any glaciation in the Dunedin area. He claimed to have found a soil with a particle size analysis similar to that predicted by Wu in his text.

Mr Oborn replied that a glacial till grading curve would be very difficult to draw because of extreme variations which occur within a glacier deposit. He said there was no evidence of glaciation in the Dunedin area.

Mr Bullen said he had found resistivity surveys quite successful on large scale situations such as a dam site and asked Mr Ingham to comment on their use in the relatively small scale situations associated with building foundations.

Mr Ingham replied that he was not experienced in resistivity methods. He said interpretation of resistivity data is always based on horizontal layering or approximations thereto and predicted this may lead to trouble in fine scale work.

Dr G.R. Martin (University of Auckland) summarised the session, stressing the need for more co-operation between engineers and geologists and proposed a vote of thanks.

THE DRILLING ORGANISATION

W.L. Cornwell

Ministry of Works, Auckland

This paper discusses drilling for sub-surface investigation and emphasises the important position the driller and drilling unit occupies in the work. It is not proposed to deal in detail with the design of specialised plant and underground equipment or to detail the many techniques the driller uses. Drilling manuals and trade brochures adequately describe these facets of the work and with the wide variety of equipment available at the present time, the specialist in sub-surface engineering is faced with a bewildering range of literature on drilling rigs and sampling devices.

The term drilling unit is used in a broad sense, including both the small unit, frequently operated by the owner and also the more complex organisation involving several machines, drillers, drilling supervisors and management.

Throughout this paper it will be stressed that the drilling programme must not be regarded as either the start or finish of the investigation, but is a stage (and an important stage), being dependent on preliminary surface appraisal and ultimately providing material for classification, testing and evaluation.

THE DRILLING UNIT IN SITE INVESTIGATION

Fookes (1967) recognises three basic stages in investigation in which the drilling unit may be involved.

1. Site Exploration - a preliminary survey which is predominantly a surface assessment but may include preliminary drilling.

2. Site Investigation - a detailed study involving the bulk of sub-surface work.

3. Foundation Investigation - a stage associated with construction.

If these stages are recognised it is then inferred that some organisation and planning of co-ordinated activities is required between surveyors, geologists, engineers, laboratory services drilling teams and designers etc., whose ultimate aim is to fully evaluate the sub-surface conditions of a site for a specific structure. There must be an orderly sequence of events, planned to obtain maximum efficiency from all sections of the group. The extent of planning will be dependent on the size of the project. The purpose of such a plan is to ensure that the objective of the investigation is not lost.

In New Zealand, few organisations are able to provide all the services necessary for a complete site investigation. Drilling units are in general operated on a contract or hire system and therefore, are seldom allied to other service organisations such as soil mechanics laboratories, seismic or geological survey teams. In the overall planning of sub-surface exploration, these restricted services must be taken into consideration.

Drilling services may be associated with both geological mapping, laboratory soil testing and in-situ testing. Some survey (in particular community underground services), sketch design and geology should have been commenced prior to drill rigs moving on to the site. On the other hand laboratory facilities must be available to handle the cores and samples or to carry out field measurements during the drilling operation.

If the maximum information is to be obtained from the bore hole, all service organisations must be co-ordinated to assess the core or samples, as soon as possible after the bore has been completed. The drilling unit will thus require to know where the following responsibilities lie:-

(a) the brief - what is the objective, number and depth of the bore, the sampling requirements;

(b) the setting out and survey for bore holes;

(c) the arrangements for logging and storage of cores and samples;

(d) the requirements for in-situ testing associated with the drilling programme;

(e) whether the bore hole is required for measurements of ground water level.

The drilling operator, if well briefed in the early phases of the work, can maintain a service at the required time, ensure that plant and equipment most suited to the conditions is available, and determine the time involved in the work and hence continuity. There is some responsibility by the planners of site investigations to maintain an even work load on the drilling unit. Heavy, uneven commitments produce additional plant requirements for peak periods and idle machines in the intervening recessions.

A considerable portion of the cost of an investigation can be taken up in drilling. Sound organisation throughout may not materially affect the total cost but will ensure value for financial outlay. It is therefore prudent in planning to specify the details of the drilling requirements.

SOME ASPECTS OF THE DRILLING BRIEF

Detailed specification for the drilling programme in sub-surface exploration will vary from site to site. A more general brief can be formulated for site exploration (Stage I) than for detailed site investigation (Stage 2) but the purpose of the drilling must be made clear.

Drilling operators who specialise in foundation exploration can become highly skilled in core drilling, sampling and in-situ testing. It is prudent to bear in mind that basically, the driller has been trained to drill holes, and he assesses his own efficiency in terms of the daily footage rate achieved. High core recovery is of importance but many drillers regard the slow process of core recovery in difficult strata as a hindrance to obtaining fast footage rates. The geologist and soil scientist are interested in both the quality and quantity of recovered core and are frequently more interested in the core that was <u>not</u> recovered. The drilling brief should enable the driller to establish the relative importance between footage drilled, core recovered, sampling and in-situ testing such that he can measure his performance in terms of the most important activity.

The brief should be informative on topics such as survey, access, general methods of drilling (core, non core) and sampling, instructions concerning core and sample storage and liaison with planning and other independent service organisations. Special conditions concerning underground services or notes on overhead power wires should be drawn to the operator's notice. Sampling frequency and special in-situ testing procedures should be included where possible but in variable sites, some freedom of sampling would be expected and due allowance made in specifications.

Finally, all drilling operators should be required to detail the underground equipment which they have available for use in an investigation. This list should include the specification of the main plant items, detail auxiliary drilling equipment, additional equipment (penetrometers etc.) standard thread sizes, and the facilities which could be employed on inspection shafts. The detailed job specification can then be planned in terms of the contractor's equipment and due allowance made for equipment which will be required from other sources.

THE DRILLING UNIT

The basic equipment of the drilling unit can be divided into three main categories:-

(a) the drilling rig (the motorised unit, winches, pumps, shear legs, mast, drop hammers etc.);

(b) the underground equipment (drills, bits, rods, core barrels, bailing gear, casing etc);

(c) the service tender and water supply (water pumps, welding gear, tools, mud mixers etc.).

In addition there should be sufficient backing from an engineering workshop to ensure that equipment is maintained in excellent condition.

Drilling for foundation investigation in New Zealand has been an offshoot of procedures used in the sinking of water-wells.and it is common for machines to be equipped with both rotary turntables and the spudding equipment for percussion drilling. It is only within recent years that core drilling equipment designed for mineral exploration has been imported into the country but the lack of versatility of these machines has not made the machines popular with the operators who still depend to a large extent on water-well installation for their livelihood. Dual purpose machines which are advantageous to site investigation have been developed locally but in many instances the quality of the investigation drilling has suffered because both machines and operators have been geared to drilling for water rather than for sub-surface investigation. The selection of the type of drilling rig by a contractor will depend to a large extent on the most common sub-surface conditions in a district. In the Auckland area, where fine grained alluvial and residual soil types predominate, rotary machines are of greatest value whereas in the Canterbury district, percussion machines will be more suited to the gravel conditions.

There is a marked difference in the machine requirements for mineral prospecting and for site investigations. The former is largely concerned with obtaining cores for analysis and stratigraphical interpretation. The machine rating required for this type of work is based on a depth-diameter relationship, whereas in site investigation this rating has minor significance (tunnel work can be an exception). Site investigation requires numerous relatively shallow boreholes which could be undertaken by small rotary machines, but it is stressed again that the drilling operation does not complete the investigation. Sampling and in-situ testing requirements must be considered in the operator's selection of plant. Weight, winch arrangements, mast height, water pump capacity, power, speed of rotary tables or winches and spudding gear all play an important part in drilling for site investigations.

The selection of plant by an operator will therefore depend on three basic factors:-

(a) the predominant sub-surface conditions in an area;

(b) the type of drilling work most frequently requested;

(c) the demand on the services of the drilling unit.

The financial aspect of the drilling organisation may well be briefly discussed at this point. The capital investment by an owner-operator, equipped to supply the basic plant and specialised underground sampling tools is calculated to be at least $50,000. A drilling operator specialising in site investigation may invest more than this figure as the range of equipment available may include in-situ testing equipment, special core barrels etc. He must also maintain spare parts for machines and equipment and provide for replacements. The more diverse the sub-surface conditions in an operator's area the greater the variety of equipment that must be held in stock.

A well equipped drilling unit would probably consist of three machines:- a percussion rig, a heavy duty rotary machine and a light trailer machine for restricted areas and for sites with difficult access. A heavy duty dual-purpose rotary machine fitted with spudding gear may replace the separate percussion and heavy rotary machines.

The underground equipment required in site investigation will depend on the same factors that govern the drilling rig. Although sampling equipment is dealt with in the following paper by Dr Northey, some reference will be made to diamond bits. The author favours diamond bits for most work, whether soft or hard formations are encountered. A core barrel, designed for high core recovery, is a precision tool and its use with diamond bits enables cores to be cut to prescribed tolerances to suit the barrels. A rough borium tipped barrel frequently produces uneven cutting and tearing of the strata; this in turn distorts the bit and produces excessive wear on the non rotating parts of the core barrel (particularly when clay shoes are used). The preference for bit facings will be dependent on personal experience and also to some extent on the facilities available for maintaining and re-facing bits. Diamond bits however require a high capital outlay and should only be used by skilled operators.

Service equipment should include a four-wheel-drive water tender equipped with a heavy duty winch for hauling purposes. The layout of the tender vehicle can be designed such that auxiliary equipment, drilling clay, mixing machines, welding equipment and tools are available on the drilling site.

The drilling unit has been considered in terms of plant and equipment but the position held by the drilling operator is most important. The operator is responsible for the efficient operation of the machine but in addition, his function is to anticipate the requirements of the site investigation supervisor.

It is preferable that drilling staff are orientated towards soil mechanics procedures to enable liaison between laboratory and field operation. Core logging and sampling are normally part of the laboratory staff's duties and the accuracy of these results depend, to a large extent, on the techniques used by the drilling operator. As with many trades, the skilled operator has to develop a "feel" for his machine and be able to interpret from the drill reaction what is occurring down the borehole. If a drilling unit is to maintain its efficiency, the operator must not only be aware of what the investigation team is trying to accomplish, but also realise that he holds a key position in the final interpretation of sub-surface conditions. Highly specialised equipment in the hands of a poor operator will probably produce poorer results than those obtained with less sophisticated machinery under the control of an expert operator. Underground equipment requires continuous care and maintenance to enable it to operate satisfactorily and when operating, observation and adjustment must be made to the plant. The driller's machine and borehole log is frequently of great use when read in conjunction with the core log and notes on water loss, rate of penetration, water pressures, borehole collapse etc., are important in the overall interpretation of a core. Appendix I is a typical driller's log.

The driller is responsible for analysis of each day's work with a job summary assessing drilling, idle and repair time for the machines such that the total available drilling hours can be equated to the operating hours. Job delays due to maintenance of equipment, cartage of water, access etc. should be reduced to a minimum. The driller should also be required to record the total footage footage of core recovered, number of samples taken and techniques employed for sampling procedure Good recording at the drilling rig may assist planning future work where sub-surface conditions are liable to be similar and can also provide a sound basis for estimating cost. Appendix 2 shows a summary of the driller's daily log sheets and Appendix 3 is a typical job assessment showing costs and job efficiency.

The driller, once in the field, operating a machine under a wide variety of climatic and environmental conditions, becomes a personality. An efficient driller appears to possess certain qualities which may aid or may hinder the performance of a site investigation. He appears to possess good self-control, patience and is in general a long suffering individualist; he is often highly sensitive of criticism and may resent the advice of the non-driller; his skills are accumulated over a long period of time and are regarded as trade secrets; the degree of ingenuity shown may be at times remarkable; he is co-operative but can also be "bloodyminded"; he will always prefer one particular type of rig or drilling method.

As a key man in the investigation the driller alone carries the responsibility of this service unit in the investigation. Generally it can be said that he will perform his duties with the objective of satisfying his client. With enthusiastic and attentive site supervision, the drill operator will persevere to the utmost to achieve the desired results but on the other hand, casual and insufficient briefing without site supervision, will almost invariably result in unsatisfactory drilling performance. The job supervisor should be able to obtain the maximum from the driller without instructing him how to operate his plant. The driller-site supervisor relationship is of the utmost importance when co-ordination of activities between service organisations are required. Appendix 4 attempts to illustrate how the investigation work can be centred around the driller's ability to change procedures frequently during the course of a small job providing planning has been satisfactory. Appendices 5 and 6 outline two problems which were solved by the drilling unit, indicating that the knowledge of drilling specialists in plant and equipment can be of vital importance in difficult sub-surface investigations.

BASIS OF COSTING DRILLING OPERATIONS

The cost of a proposed drilling programme is difficult to estimate. Prices per foot for setting casing and rotary drilling are frequently obtained but the unit rate cost procedure can limit the scope of investigation. This applies particularly to site exploration (Stage 1). When the overall site conditions have been assessed and a site investigation (Stage II) is planned, more detailed specifications can be formulated and unit rate contracts may be applicable.

The cost of drilling can vary between 30 cents per foot and $12 per foot, the higher costs occurring when sampling work is undertaken in difficult materials. Drilling in gravel deposits or broken rock formation may be higher again. The purpose of the investigation is to obtain the maximum information for the minimum cost and therefore the need for assessing the site conditions in preliminary exploration is a necessary precaution against excessive costs. In some investigations, drilling programmes may have to be abandoned and inspection pits substituted when unconsolidated granular material is encountered and it is not unusual to use percussion and rotary drilling together with intermittent sampling. With changing techniques required by the site investigation supervisor, the method of costing on a footage basis has many disadvantages.

The most satisfactory basis for costing is on an hourly rate for machine hire with appropriate allowances for subsidiary drilling equipment and specialised underground tools. This system enables the maximum use of the equipment an operator can supply, whereas on a footage basis the maximum footage with the minimum equipment tends to result. Where the sampling and testing requirements of an investigation is minimal, and the strata are relatively consistent, there may be some benefit in assessing the cost of the work on a footage basis. It must be stressed that there is little advantage in specifying that payment will be made on the footage of recovered core rather than borehole depth. The cost involved in achieving a 100% core recovery rather than, for example, a 90% core recovery may not be economic to the operator. The 10% troublesome material will almost invariably cost greater than 10% of the total cost of the bore, if time is taken to recover the material. From the driller's point of view it is cheaper to ignore the 10% core loss and obtain greater footage in easily cored material.

THE RELATIONSHIP OF DRILLING TO SOILS ENGINEERING

As New Zealand is small but geologically complex, the drilling unit must be versatile with a sufficient range of equipment to cope with drilling operations under a wide range of site conditions.

The drilling unit specialising in site investigation in New Zealand faces a major problem in planning and development. The financial outlay to provide the services of a well equipped, versatile drilling unit is considerable but few operators are prepared to invest in specialised equipment which may have only restricted use. There appears to be a relationship between the extent of sub-surface investigation and the degree of specialisation in drilling equipment. The drilling operator may well ask whether site investigations are being tailored to suit existing drilling facilities or whether advancement in soils engineering will allow the development of more advanced drilling procedures. At the present time, the specialist in soils engineering has to purchase and provide sampling equipment to enable his soil mechanics laboratory to carry out testing work. An assessment of the drilling facilities in New Zealand indicates that sub-surface investigations often involve only the location of a "hard" strata by percussion, wash or core drilling with a limited amount of in-situ testing. Many drilling operators are fully aware that laboratory services are required to enable full utilisation of more sophisticated equipment. The operation of drilling teams and soil mechanics laboratories as complèmentary units can maintain an economic usage of drilling staff and equipment and also laboratory staff.

CONCLUSION

The drilling unit and the drilling operator face uncertainties in extending their services. Soils engineering varies in extent and requirements, depending frequently on the importance placed on foundation investigation by the designer. The drilling operator is faced with the problem of either providing a restricted service or involving himself in high capital expenditure to satisfy the occasional demand for sophisticated site investigation. It would be a profitable avenue for research to compare the cost of site investigation with the construction cost of an engineering or building project and also to relate drilling costs to those of geological, soil mechanics laboratory and allied services in the investigation. The drilling operator would then be able to assess the potential for investigation work and develop his unit with some degree of certainty that high capital expenditure would be fully utilised.

In site investigation, the drilling operator tends to consider that his brief is insufficient and more planning and liaison between service organisations are therefore necessary to enable the most efficient use of equipment at reasonable cost. There is great opportunity in the drilling industry for specialised investigation on which to base foundation design but, at the present time, drilling operators feel that their services are being used only to prove that design is satisfactory.

ACKNOWLEDGEMENT

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APPENDIX I

ENGINEERING LABORATORY, AUCKLAND

DRILLER'S DAILY LOG SHEET

PROJECT AND LOCATION: Ramarama Motorway 161

DRILLER: J.D. May   PLANT: Damco 268161   DATE: 25-2-69

Bore: 21700 E

Bit. & Reamer	Core Barrel	Feet Drilled Fr.	To	Feet	Formation Drilled	Drill Time Fr.	To.	Core Recovery	Samples Taken	Water Loss	Casing Depth	Remarks: e.g. Details of core lost, G.W.L. at E.O.B., Feed and Pump Pressure
	Open	0'	4'	4')	Weathered basalt with sandy sections and hard pockets	1.00	1.15	3'0	4'0		4'	
B.D.	NMS	4'	8'	4')		1.20	1.30	Lost	8'0			Barrel under own weight. Water pressure 50 p.s.i. Penetration insufficient (poor)
Short Shoe-clay	"	8'	12'	4')		1.45	1.55	3'8	12'0			Clay (short) shoe light pressure (better)
"	"	12'	13'	1')		2.05	2.15	1'0	-			Slow-country too hard for shoe. Heavy pressure.
B.D.	"	13'	16'	3'	Blue rock pockets	2.15	2.30	2'6	Rock No test	20%		Pump 20 p.s.i. - revs increased. Safety valve blown - Gauge faulty.
		16'	20'	4')	Consistent semi-hard rock	3.0	3.20	4'0	-			Adjust pump revs. to give more water. Bowden cable not positive.
		20'	25'	5')		3.30	4.00	5'0	-			

Travelling Fr.	To	Hrs	Shifting Fr.	To	Hrs	Nature of Shift	Other lost drilling time Fr.	To	Hrs	Reasons
8.15	9.15	1	12.30	1.00	½	Shift into position	7.30	8.15	¾	Change gear & vehicle. Contact Farmer on best access. Arrange for water & fill.
4.00	5.00	1					9.15	12.00		Recondition NMS barrel & shoe

APPENDIX 2

JOB SUMMARY SHEET

JOB No: 173

SUB SOIL INVESTIGATIONS PROJECT: Auck-Ham. Motorway LOCATION: Ararimu Rd Underpass PLANT: Edeco 454 " 458 CREW: J.May

Date	Available Hrs	Drilling Hrs	Lost time hours Trav.	Lost time hours Wet	Lost time hours Mechanical	Other	Footage/ Drilled	Samples Taken	Remarks
7/8/68	8½	2½	2			4	12	3/1½"	3hrs Stores/W/shops
8	9	7	2				32	3	1hr set up.
9	9	5½	2			1½	11	-	E.O.B.65' 1½hr shift.
12	9	6	2			1	16	4	Water
13	8	-		8				-	
14	9	7	2				37	4	
15	9	4½	2		1¾	¾	22	2	E.O.B. 68' ¾hr shift.
16	8	-	1		7			-	
19	9	4	2			3	15	-	2hrs stores, changing
20	9	4¾	2			2¼	20	-	rig, set up.
21	9	4¼	2		1¼	1½	19	-	
22	9	7	2				25	5	
23	8	6	2				30'6	-	
24	9	5	2			2	22	3	Casing & water.
26	9	3¼	2			3¾	9'6	-	
27	9	7	2				19	5	
28	9	7	2				21	5	
29	9	6	2			1	31	2	
30	8		-	8					
		Rig at Papakura M.C. 31-8-68 to 4-9-68							
5/9	9		2			7			Casing
6	9		2			7			Casing
7	8	5	2			1	12'6		
9	9	5¾	2			¾	19	1	
10	7		2	½		5			
	208½	97½	43	16½	10	41½	373		

APPENDIX 3

SUB SOIL INVESTIGATIONS

Engineering Laboratory,
AUCKLAND.

JOB RECORD

Date: Commenced 7-8-68
Completed 10-9-68

Project: Auck.- Ham. Motorway
Ararimu Rd Underpass

Location: Runciman

Job No: 173

Authority No: 26.80.01

Drilling Details:

Holes No: 5

Total Footage: 373

Plant: Edeco MK VII PN268454
Edeco MK VI PN268458

Crews: J.May & J. Collins
" "

Drilling Times:

		Total	Percentage
Available Hours		208½	100
Drilling Hours		97½	46
Lost Time	Travelling	43	21
	Wet weather	16½	8
	Mechanical	10	5
	Other	41½	20

Charges:

Plant Hire	$442	
Transport	54	
Wages (Field)	484	
Accommodation		
U/Ground Equip. Rep	112	
Total Drilling Costs	1,092	$1,092
Salaries (Lab.)	398	
Stores & Workshops	-	398
Total Costs		$1,490

Drilling Cost/Foot $2.93 Total Cost/Foot $4.00

Remarks:
Average penetration 3.83' per drilling hour.
Clays, silts 0-20' Sandstone, siltstone 20-80'
Hole 575 cased to 87'6, Gravel layer at 80'

APPENDIX 4

DRILLING UNIT BRIEF & DRILLER'S RESPONSIBILITY

Structure: Three Span Bridge

Location: Waipu - 80 miles from Drilling Depot.

Site Information: Site is adjacent to tidal river subject to flooding. Access is suitable for vehicles but difficulties will be experienced in wet weather. Sub-surface conditions are not known but it is expected that alluvial deposits of soft organic clay overlies sand. A penetrometer probe indicates that unconsolidated clays extend to about 25 ft but the penetrometer could not penetrate beyond 30 ft.

Sub-surface requirements: The depth of sand is required but a bore extending to about 100 feet may be required. Undisturbed 4" samples will be required in the upper soft layer; Raymond penetrometer values in the sand; 4" samples may be required to depth; dutch cone penetrometer work may be required. Detailed sampling depths will be given after the initial bore has been drilled; continuous coring will be required. A technician will be in attendance.

Drilling Unit Organisation:

Plant Requirements:

(a) Heavy duty rotary rig, 30 ft mast, dual winch and cat-head with driving monkey.

(b) Mud mixer and mud pump.

(c) Auxiliary water pump.

(d) Tender with chains and winch.

Drilling Requirements:

(a) 80 ft flush walled NX casing.

(b) 70 ft 6" casing

(c) Casing bailers - mud and sand.

(d) Drill rods to 120 ft.

Sampling Requirements:

(a) Single, double, triple tube core barrels.

(b) Raymond sampler and attachments.

(c) 20 sampling tubes and adaptors.

Dutch Cone Penetrometer: Rods to 120 ft - Six anchor system.

Staff: Operator and assistance, penetrometer crew.

Accommodation: Hotel accommodation.

Daily cost: $80 per day with estimate of 9 hours in the field.

Extra: Pilot for truck with overlength derrick - Auckland to Waipu and on Auckland Harbour Bridge.

APPENDIX 4 - Continued

Additional Work in Area: Quarry investigation 20 miles from Waipu (2 days)
Quarry investigation Dargaville (4 days).

Site Programme:

1. Drill eastern bore to determine soil types - Future site programme to be assessed after 60 ft.
2. Detailed job brief.
3. Drilling procedure based on brief. This is summarised in chart.
4. Dutch Penetrometer at each pier and abutment.

	Soil Profile	Job brief (2 bores)	Drilling Procedure (each bore)
0-25	Soft organic clay	4 x 4" samples at 5,10,15,20 feet.	6" Casing to sampling depth and repeat to gravel layer. Casing additions of 2 ft rqd.
25'-30'	Soft sandy clay		
30'-32'	Fine gravel	Raymond penetrometer in-situ testing	NX casing to 30' test and repeat to 40'. Casing additions in 2 ft lengths.
32'-40'	Medium sand		Stabilise with bentonite before lifting NX casing
40'-105'	Soft silty clay with organic layers	4 x 4" samples at 40,45,50,60 ft.	Set 6" casing to 40' and bail - Sample and repeat.
105'	Dense sand		

Programme Adjustment:

Owing to extremely wet conditions investigation was postponed after bore 1 and 2. Machines diverted to quarry investigation for 6 days. Dutch penetrometer completed programme after first sampling bore and confirmed that undisturbed sampling essential.

APPENDIX 5

THE DRILL OPERATOR'S CONTRIBUTION TO A PROBLEM FORMATION

Throughout North Auckland massive sheets of sandstone, shale and calcareous sediments have slumped across younger formations. During this gravitational movement, the Onerahi Chaos-breccia (Kear 1966) has been sheared, distorted and mixed, giving rise to a formation unstable in both excavations and embankments. As some six structures will be built on the formation in the Silverdale district, laboratory testing was imperative to the design engineer. However, samples could not be obtained by the conventional methods of drilling.

Experiments were conducted by drilling operators with a variety of equipment until some indication of the most satisfactory method was determined. In close co-operation with a drilling equipment manufacturer a 6 inch, triple-tube core barrel was adapted to suit the conditions.

Drilling operators maintained a constant check on fluid pressures, drill pressures, rate of penetration, cutting speeds, machine reaction and washings from borehole such that a correlation could be established between core barrel shoe extension and core recovery in different lithological units. The success of the sampling operation was predominantly in the hands of the operator who understood what was to be accomplished.

From a cost angle, samples for triaxial testing were obtained at approximately $12 per foot. The rate of drilling was slow, extra drilling staff required and the cost of the special equipment was approximately $3,000.

In this case expensive equipment was purchased to use on at least six projects and thus the capital expenditure could be justified as also the high cost of obtaining samples for testing.

APPENDIX 6

THE DRILL OPERATOR IN THIN LAYER SAMPLING

A massive subsidence investigation required information on the shear strength of the sliding surface. The surface had been located and due to limitations in test equipment, 4 inch undisturbed samples were required and taken at an angle of 45° to the shear surface.

The selection of the drilling rig by the operator to perform this work was a heavy weight machine with overhead hydraulic rotary head and hydraulic feed. The derrick could be maintained at a set angle by hydraulic rams.

The procedure adopted was dry drilling to within 12 inches of the shear plane and subsequent sampling by undisturbed four inch sampling tubes across the shear plane. The method used was both cheap and efficient and illustrates the use of versatile machines with competent operators.

DRILLING AND SAMPLING TECHNIQUES

R. D. Northey and R. F. Thomas,

Soil Bureau, D.S.I.R., Lower Hutt

The majority of soil samples taken in New Zealand as part of major building site investigations are probably those taken from boreholes sunk with well-drilling or rotary rock-drilling equipment. On many sites, preliminary shallow subsurface exploration may be made with manual post-hole augers, small power augers and such earth-moving equipment as bulldozers and trenching machines. Samples may well be taken from existing road cuttings and other exposures or from pits, shafts, and hand-drilled holes. While these methods of shallow exploration have a definite and important role in the general pattern of site investigation, this paper is not concerned with them. Test boring with well-drilling or rock-drilling equipment has proved itself most suitable for major building site investigations; greater depths are more readily and economically attainable than by most other methods, the site is not seriously disturbed and the equipment used is usually able to provide sufficient power and reaction for pressing soil samplers into the ground. This paper describes in broad outline the drilling methods in common use in New Zealand, surveys the methods available for disturbed and undisturbed sampling, and describes improvements to the common techniques in order to allow samples of greater engineering significance to be obtained.

DRILLING METHODS

General

As part of a site investigation, holes may be drilled in the ground for a variety of purposes including:

(a) an initial, general appraisal of an area;

(b) location of or sounding to a hard layer, gravel, or bedrock to delineate its surface contours;

(c) taking of representative disturbed samples for identifying soil strata and for simple testing for laboratory classification,

or

(d) advancing and cleaning holes to specific horizons for accurate logging, taking of special samples, or conducting *in situ* tests.

The method of drilling should be matched with the purpose of drilling to ensure satisfactory return for the effort. With few exceptions, the methods used to advance boreholes for site investigation purposes have been developed initially for quite different purposes. Generally the emphasis has been on advancing the hole rather than on identifying clearly the soil material traversed in making the hole. Rate of progress has been an important criterion of a successful drilling method, with little concern for possible disturbance of the material ahead of the drilling tool. However, many properties of soil material are affected in greater or lesser degree by the disturbance of their natural structure. Thus, with conventional well or rock-drilling techniques the success of site-investigation drilling depends in very large measure on the sympathetic understanding, skill, and care of the drilling operator. Relatively primitive techniques may be acceptable for purposes (a) and (b) above but the requirements for purposes (c) and (d) are more demanding.

A clear distinction must still be made between drilling designed to advance a borehole between specific sampling or testing locations, and drilling designed to yield reliable information on the texture, consistency, and thickness of soil strata. These functions may often be carried out simultaneously with some success, but ideally they should be undertaken separately. Where an important building is to be erected, the site investigation should allow a reasonable assessment of the kinds and thicknesses of soil strata before any attempt is made to obtain undisturbed samples of significant materials for critical testing.

Methods Depending on Mechanical Removal of Cuttings

(a) Auger drilling. This includes the traditional British method of advancing a hole in clays, initially by hand, subsequently by power augers. Power augers have not been used extensively for investigation of building sites in New Zealand up to the present time, though single-flight augers designed for installation of posts and power poles have been used in highway surveys and large bucket augers in materials surveys. Continuous-flight augers are widely used overseas and are gaining some acceptance here, especially for holes less than about 50 feet (16 metres) in depth. However, the use of conventional continuous flight augers is an inefficient method for determining the thickness and consistency of soil layers. Despite the too commonly held belief that the depth to soil changes can be assessed from the rate of rotation and feed of the augers and from the arrival of soil at the ground surface, there is no substitute for removing the augers and sampling the bottom of the hole. Material clinging to the lowest flight may prove a useful guide but that in other flights is so mixed that identification and location of soil changes is generally not possible. The removal of a long string of soil-laden augers before sampling may be a time-consuming and tedious operation. Thus the method is not well adapted to sinking holes where a large number of samples is required. Below water table in cohesionless materials it may prove impossible for advancing holes.

Continuous-flight, hollow-stem augers have been designed particularly for engineering site investigation. Essentially, they consist of continuous flight augers constructed round a central access tube instead of a rod. During drilling operations, the mouth of the hollow stem is closed by a pilot bit attached to a string of drill rods extending to the auger head. When a sample is required it is necessary to remove the drill rods only, with the pilot bit attached and to lower a sampling device through the hollow stem to the relatively undisturbed material (Fig. 1). This equipment is rapidly gaining wide acceptance in many countries and has been used to a certain extent in New Zealand (Thomas and Barker, in prep.). Hollow-stem augers constitute a reasonably fast and reliable method of advancing a hole between sampling operations in fine-grained materials and in sands above the water table. They are available with an internal bore large enough to allow the passage of 4-inch (100 millimetres) diameter sampling equipment, but augers of this type are heavy and require the use of very heavy machinery. For investigations requiring samples up to 3 inch (75 millimetres) in diameter, they offer a number of advantages.

Large-diameter bucket or single-flight augers are often used in the construction of piers or caissons for foundations. Their use for site investigation is limited to materials above the water table or to materials of low permeability that can be cased off from any overlying water-bearing stratum. They provide the advantages of rapid progress and the opportunity of visual inspection and access to layers of interest. In situ testing and hand-carved block sampling can be readily carried out in such shafts.

(b) Cable tool drilling. Except in extremely stable ground, this method requires the use of casing which is driven down a distance dictated by the ground conditions or by the depth at which a sample is required. Material below or in the lower section of the casing is churned up or pulverised by lifting and dropping a heavy chopping bit. When drilling above the water table, a small amount of water is added to soften the material being penetrated and to suspend the cuttings; the slurry resulting from the churning action is bailed out of the hole with a sand pump or similar equipment. The casing is driven down and the process repeated. When good-quality samples are required from above the water table the necessary addition of water makes the method of doubtful value. Below the water table it may be more satisfactory but great care is required to ensure that the action of the chopping bit, the lifting of the sand pump, and the driving of the casing do not cause severe alteration to soil properties. Attemps at excessively rapid progress almost certainly cause disturbance of the soil below the casing to depths equivalent to several times the casing diameter. When carefully carried out by a skilled operator, this method may prove reasonably satisfactory both as a means of advancing the hole for undisturbed sampling and as a means of providing information for logging purposes though thin strata may be missed. It is the only practicable method capable of penetrating beds of coarse gravel. On one site known to the authors a thick gravel bed containing boulders up to perhaps 12 feet (4 metres) in diameter was successfully penetrated by cable tools assisted by explosives.

Methods Depending on Fluid Removal of Cuttings

(a) Percussion wash drilling. The hole is advanced by chopping and jetting. The method was once commonly used for site investigation work but fortunately its severe limitations have been recognised and now it is rarely used for identifying soil strata or taking of samples. It still has some value in delineating hard layers.

(b) Rotary water-wash drilling. This method commonly employs a fishtail, multiblade, or roller-cone drilling tool which is rotated to churn or grind the material being drilled. Wash water is usually discharged through downwards directed jets to assist the churning action, to cool the cutting tool, and to carry cuttings to the surface. When the method is used for drilling soils, casing is used to stabilise the hole. A substantial flow of water is necessary to ensure the return of coarse cuttings to the surface and consequently the fluid velocity through the downward directed water passages in the bit may be quite high. Thus when drilling in soft soils, or in sands, this method is likely to produce considerable disturbance of the material below the cutting bit. Identification of the soil being drilled cannot be made reliably from the cuttings returned to the surface in the wash water, but an experienced driller can deduce considerable information concerning the texture and consistency of the soil being drilled from the behaviour of his machinery. Accurate identifications may be made only by using a sampler to obtain material that has not been eroded or sorted by the wash water. Such samples should be taken at regular intervals and whenever the behaviour of the drill or the appearance of the wash water indicates that a change in conditions has taken place. If a clay layer has been penetrated, fresh wash water should be used whenever possible since changes are not readily identified from recirculated water. If wash drilling is being used as a method of advancing a bore hole for undisturbed sampling, satisfactory samples may be obtained only when a clean-out auger (Fig. 2) or similar device, is employed. In this way the disturbed material may be removed from the bottom of the hole without allowing wash water to impinge on the material to be sampled. With care this method may provide a satisfactory method of advancing a borehole. Reverse-flow circulation has some advantages in reducing the downward velocity of water flow at the bit.

(c) Rotary mud-wash drilling. This method is similar to the rotary water-wash method in most respects though circulating fluid velocity is much lower. No casing is needed, drilling mud being used to stabilise the borehole, and consequently the method may sometimes be carried out more rapidly than when water alone is used as a wash fluid Little information regarding changes in the material being drilled can be gained from the appearance of the wash fluid. Samplers for use in mud-stabilised boreholes require either a piston or a large port area to allow the passage of the high-viscosity mud, otherwise slower rates of penetration are needed. The method is not widely used in New Zealand for site investigation work but has a definite place, especially for sand sampling.

(d) Rotary air-flush drilling. The method and the equipment are generally similar to those used for other forms of rotary drilling except that air is used as the circulating fluid with very high rates of flow. Its chief advantage for site investigation is that no water is added to the hole and consequently the water content of soils being drilled in preparation for sampling tends to be relatively unaltered. As noted later, the method has been used in other countries to provide high-quality undisturbed samples in certain circumstances. Earlier comments regarding identification samples and, to a lesser degree, clean-out augers apply also to this method. Again reverse-flow circulation may offer some advantages. Rotary air-flush drilling has rarely been used for site investigation for buildings in New Zealand but its use could be encouraged.

(e) Rotary core barrel samplers. Within certain limitations, some rotary soil samplers, while advancing the hole, may be used to obtain samples suitable for many soil mechanics purposes. Equipment of this type should not be used for soft fine-grained materials or for uncemented sands. A rotary core barrel with extended stationary inner tube may provide satisfactory samples from stiff to hard fine-grained materials. Some overseas experience (Ward et al., 1965) indicates that similar equipment with reverse air-flush in appropriate soils may provide samples of a quality comparable with samples hand-carved from a pit or shaft. Where it is known that no soft or non-cohesive layers are likely to be encountered, equipment of this kind may offer a rapid method of both advancing the borehole and sampling for testing purposes. A calyx or sludge barrel enables lower fluid-circulating velocity to be used with advantage. Larger-diameter samplers of this kind have been designed (Northey, in prep.) incorporating a series of rings in which the sample is retained and transported. A completely satisfactory solution to the problem of transporting and storing cores from smaller-diameter samplers of this kind has yet to be found in New Zealand. American practice of rigid phenolic impregnated paper liners might be an answer. Little or no use has been made in New Zealand yet of auger core-barrels (e.g Aitchison and Lang, 1963) but they seem to combine the advantages of augers with the advantages of improved rotary core barrels. The absence of a circulating fluid should reduce the risk of sample disturbance.

Methods depending on Displacement

Superficially, displacement is the simplest method of drilling to take a soil sample from a particular depth. A closed sampler is driven to the required depth, opened by rotation or surface actuation and driven again to take a sample. The sampler is necessarily restricted to a small diameter but may provide satisfactory identification samples. Several piston samplers of the Porter (1936) type once received widespread use in New Zealand but have fallen into discard. The Dutch-cone sounding apparatus with provision for a displacement sampler (Vermeiden, 1948) has occasionally been used. Considering the simplicity of the concept and the variety of the methods described in recent publications (e.g. Kallstenius, 1953; Begemann, 1961; Sokjer, 1961), it is a little surprising that a wider application of these methods has not been found in New Zealand.

SAMPLING METHODS

The quality of soil samples taken during site investigations varies within wide limits (Northey, 1969). Mixed and even incomplete samples may be used for the identification of the soil layers through which a borehole has passed. Severely disturbed and softened samples, provided they are complete, may be used for the determination of index properties. Distorted samples may be used for the determination of water content and sometimes a field assessment of strength in broad terms. For insensitive soils, a higher degree of disturbance may sometimes be tolerated in determining shear strength than for soft sensitive soils. However, high-quality samples are always necessary for the determination of critical stress/strain relationships and consolidation characteristics. Thus when discussing methods of soil sampling for civil engineering purposes it is necessary to consider sample quality, that is the properties that can be determined on samples taken by each method. In the following discussion, sampling techniques described in each class are the simplest from which a suitable sample can be obtained. When comparing different samplers, the relative wall-thickness (expressed as "area ratio") and the difference in diameter between the cutting edge and the inside of the sample barrel (expressed as "inside clearance ratio") are important parameters. These terms are illustrated and defined in Fig. 3.

Sampling for Layer Identification

Provided the geological pattern is simple, completely disturbed and even incomplete samples may be sufficient to make a reasonable layer identification. Such samples can be obtained from the lowest section of a flight auger, from the bailer used in cable drilling, from a heavy-walled open-drive sampler hammered into the material at the bottom of the borehole or from a suitable side-intake sampler. Samples of non-cohesive material below a water table cannot be obtained from auger flights. Provided the maximum-sized material present can enter the throat, a barrel auger may retain a satisfactory sample from just beyond the mouth of casing or through hollow stem augers.

Sampling for Classification Tests

It is pointless carrying out classification tests on incomplete samples. Further, since the worth of Atterberg limits is reduced unless the natural water content of the sample is also known, samples of fine-grained material should be at natural water content; consequently, samples retrieved from the bailer in cable-tool drilling are not suitable for this purpose. Samples from the lowest section of a flight auger or from a driven thick-walled sampler may be satisfactory, as well as the coarse-grained material from a barrel auger mentioned previously.

Sampling for Estimation of Strength of Fine-Grained Soils

Field and rough laboratory assessments of undrained strength may often be made from samples retained in open-drive samplers (with moderate area ratio) hammered into the ground, provided precautions have been taken to ensure that thoroughly disturbed material has been avoided. Small-diameter samples are usually adequate and 1½ inch (3.8 centimetre) diameter thin-walled open-drive samplers with no inside clearance are commonly used. A stationary inner-tube rotary core-barrel with appropriate bit may retain samples for this purpose from firmer soils. Such barrels of nominal "N" size are widely used giving cores of 1.375 to 2.045 inches (35 to 52 millimetres) in diameter depending on the manufacturer.

Sampling for Laboratory Measurement of Stress/Strain Relationships of Fine-Grained Soils

All samples taken from boreholes are disturbed to some extent, usually by contact with the walls of the sampler and to a lesser degree by direct compression (Fig.4). Disturbance due to immediate stress changes cannot be avoided but subsequent disturbance can be minimised by careful handling and avoidance of delay between sampling and testing. It is generally recognised that, in most fine-grained soils except soft sensitive clays, a sampler with a small effective area ratio and minimal internal clearance ratio pressed into the ground will provide samples suitable for most testing purposes. Some workers commonly retrieve samples of rather greater diameter than that of the ultimate test specimen to enable careful trimming in the laboratory and thus produce a specimen with minimum disturbance. Others retrieve primary samples of the same diameter as the test specimen, recognising that the edges of the sample tested may be rather more disturbed than is desirable. A sample that is removed from a sample barrel in preparation for final trimming undergoes a further redistribution of stresses, and some workers consider avoiding this to be of greater importance than the disturbance that needs to be accepted when no trimming is undertaken. In some circumstances, however there may be distinct practical advantages in taking samples of the same diameter as the test specimens.

The majority of undisturbed samples taken in New Zealand are probably still taken with equipment based on the standard British 4 inch (100 millimetre) diameter barrel. A number of modifications to this equipment have been made (e.g. Northey and Thomas, in prep.) and the use of well designed and maintained equipment of this type should enable good-quality samples to be taken in most circumstances. Good technique requires the use of barrels with a clean, smooth bore concentric with the cutting shoe, a head providing an adequate port area and a valve that is not easily fouled and a sharp cutting shoe which has a low effective area ratio and allows a degree of inside clearance appropriate to the soil being sampled (Fig.5). Provision is also made for accommodating disturbed material above the sample in a sludge barrel or extended head, and ideally for removing the cutting shoe and sludge barrel without rotation of part of the sample. Several crude sampling devices broadly similar to this are used in New Zealand for "undisturbed" sampling. The sampler is pressed into the soil with a single rapid push. The drilling rig needs to provide sufficient weight and power for this to be accomplished. Light rigs may well need ancillary anchorage as in Fig. 6; even with heavy rigs the vehicle springs should be relieved by jacks, before attempting sampling, to avoid a change in the rate of penetration. After pressing the sampler in, a rest period of 15 to 20 minutes should be allowed before attempting withdrawal. Unlike overseas practice, which calls for a sampler to be rotated to shear off the soil at the cutting edge, the authors prefer to withdraw this type of sampler slowly without rotation after the rest period, following their discovery of occasional torsional damage nearly 20 years ago. Provided the inside clearance and area ratio have been well selected, this type of sampler and technique may provide good samples from nearly all fine-grained soils except the very soft and sensitive, and the very stiff to hard. Thin-walled steel and brass open-drive samplers 2 3/8 to 4 inch (60 to 100 millimetre) in diameter, are also in fairly general use for the same purpose, with somewhat similar success.

High-quality sampling of some soft sensitive clays requires equipment that is not available in New Zealand. A thin-wall fixed-piston sampler has been shown to provide adequate samples (Kallstenius, 1963) but probably a sampler incorporating thin foils to eliminate friction between the sample and the inside of the barrel is most suitable (Kjellman et al. 1960). This equipment is expensive but in soft materials may provide continuous cores up to 75 feet (23 metres) long. Since even piston samplers are rarely used here yet, the present status of New Zealand exploration and testing methods allows little call for the foil sampler. Although published 20 years ago, much of the information given by Hvorslev (1949) on undisturbed soil sampling has not yet been fully absorbed into New Zealand practice.

High quality sampling of stiff to hard fine grained soils presents some difficulties. Indeed, according to some overseas writers (Glossop and Meigh, 1962) they can present a larger problem than the soft, sensitive soils. New Zealand practice calls for the use of a rotary core barrel with extended stationary inner tube retaining a core 3-4 inch (75-100 millimetre) in diameter. However, unless the core is retained in rigid rings (Northey, in prep.) difficulties may be experienced getting the sample to a laboratory in a satisfactory condition.

Sampling for Density Determinations

The determination of the *in situ* density of fine-grained materials presents no great problem provided careful sampling techniques are used. Since density is one of the few meaningful measurements that may be made on cohesionless sands below the water table, it is important that undisturbed samples be taken from such materials. Until the advent of the sand sampler designed by Bishop (1948) the sampling of cohesionless sands below the water table was a difficult and costly undertaking. The concept on which the Bishop sampler depends is that when the mouth of a sampling tube containing saturated sand is exposed to air a reversal of capillary tension takes place which provides sufficient "cohesion" to hold the sample in the tube. The Bishop sampler employs a "diving bell" which rests on the bottom of the borehole from which the sampling tube is pushed into the sand, and into which it is lifted after the water in the bell has been replaced by air. The diving bell, with the sampler in it, is then raised to the surface. A feature of the equipment is the facility for disconnecting the drill rods after pressing in the sampler, which allows the bell and sample tube to be raised to the surface on a wire rope, thus avoiding the vibration associated with uncoupling drill rods. Since sand density is easily changed, an accurate volume measurement needs to be made on the sample while still in the tube and before much handling. The equipment and technique call for careful attention to cleanliness and details of operation to ensure successful sampling.

Several modifications of the Bishop sand sampler have been made (Serota and Jennings, 1957; Hodgson, 1967) which greatly simplify its operation but to the authors' knowledge only the basic equipment has been used in New Zealand. Even the Bishop sampler has had little use in recent years. There is no simple method by which similar information can be obtained on gravels at depth. Penetration tests or shafts providing access may allow assessment of density.

CONCLUSION

No single method of drilling can be considered to be ideal for site investigation work in all circumstances. Each of the methods reviewed above has both advantages and disadvantages. In many areas of New Zealand the likelihood of encountering substantial gravel layers makes cable-tool drilling the only practical choice, perhaps in combination with other methods. For layer identification, rotary drilling with frequent dry samples is probably the most economical and satisfactory method where little or no gravels are present. It has some advantage in that the equipment is frequently light and easily set up in difficult areas. When high-quality sampling is required, rotary methods may be satisfactory to advance the hole, provided a clean-out auger is employed, but light rigs may not supply sufficient reaction or power to operate driven samplers in a satisfactory manner. Thoroughly cleaning out the bottom of the borehole before sampling is frequently not possible by cable-tool methods, yet with sufficient care and skill, high-quality driven sampling is possible. Hollow-stem augers are a very convenient boring method for almost all soils above the water table except gravel, but as a means of advancing the hole for high quality sampling generally, they need to be restricted to samples of smaller diameter. Rotary core barrels with an extended stationary inner tube may provide good samples from firmer formations, more particularly when suitable provision is made for the transport and storage of the samples. The use of such samplers with air flush seems worth further consideration when difficult conditions are encountered.

No general conclusion regarding sampling is possible except that the quality of sample required should be assessed in relation to the measurements it is desired to make. The most careful testing carried out on disturbed samples will generally serve only to indicate that the *in situ* soil is worse than it actually is. It is true that results obtained from disturbed samples will generally err on the safe side, but this must be weighed against the extra unknown factor of safety which using the results will introduce. Sampling for critical testing is a skilled occupation requiring well-designed equipment to suit the soil conditions, the sympathetic understanding of a highly-skilled driller, and supervision by a person with suitable field experience who has also had sufficient laboratory experience to appreciate the necessity for extreme care in the field. Application of these requirements cannot be regarded as typical New Zealand practice; there are too many deficiencies in one or more of these requirements in many New Zealand site investigations. This situation will continue until professional engineers, who specify "undisturbed" sampling ensure that samples of the specified quality are actually obtained and tested and that the results are interpreted accordingly.

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CAPTIONS OF FIGURES

Fig. 1 Sampling through hollow-stem augers

Fig. 2 Section of typical clean-out auger

Fig. 3 Dimensions of samplers

Fig. 4 Soil disturbance during sampling

Fig. 5 Typical details of composite open-drive sampler

Fig. 6 A method for providing uninterrupted pressing in of a sampler.

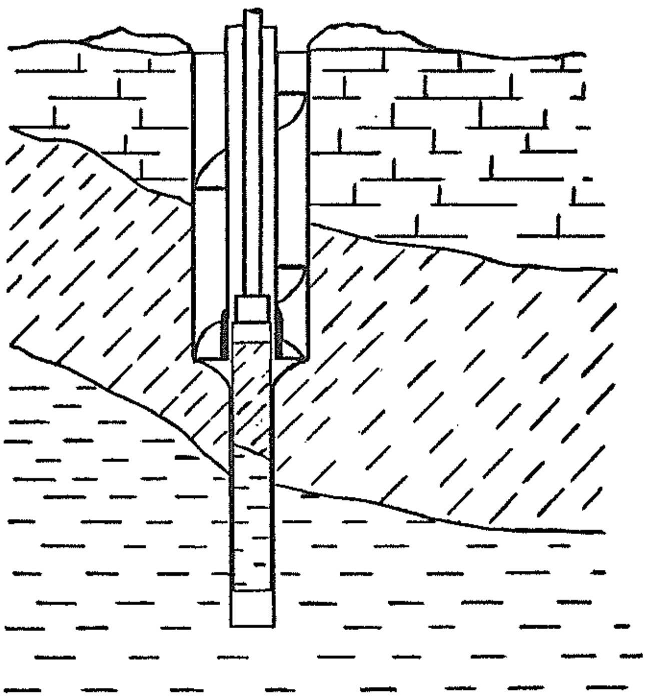


Fig. 1

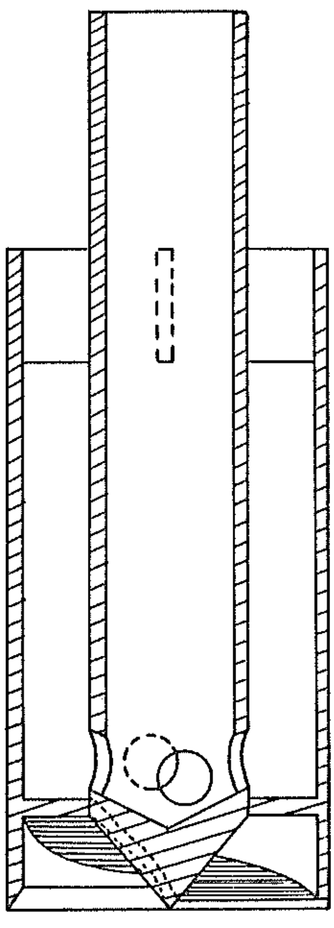
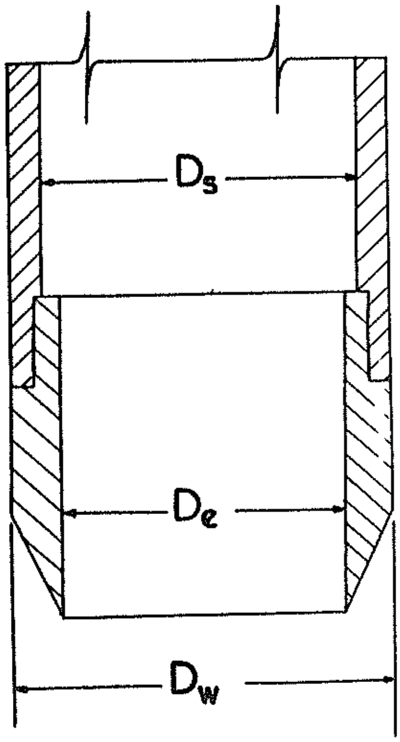


Fig. 2



INSIDE CLEARANCE RATIO $= \dfrac{D_s - D_e}{D_e}$

AREA RATIO $= \dfrac{D_w^2 - D_e^2}{D_e^2}$

Where D_s = INSIDE DIAMETER OF SAMPLE BARREL

D_e = INSIDE DIAMETER AT CUTTING EDGE

D_w = OUTSIDE DIAMETER OF SAMPLER

Fig. 3

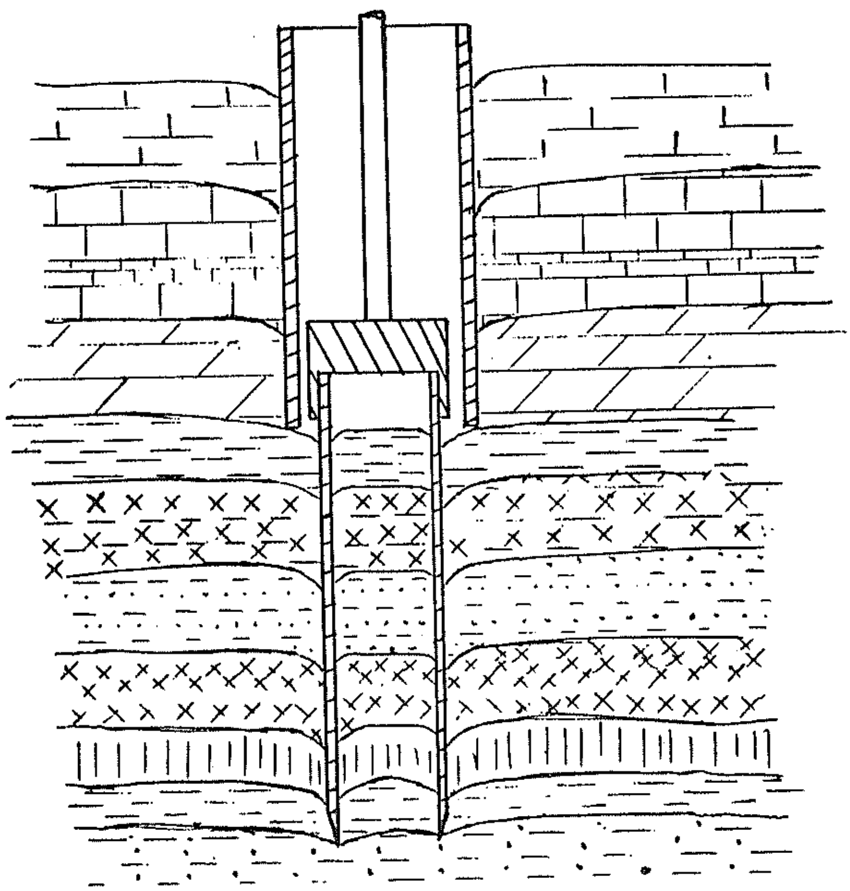


Fig. 4

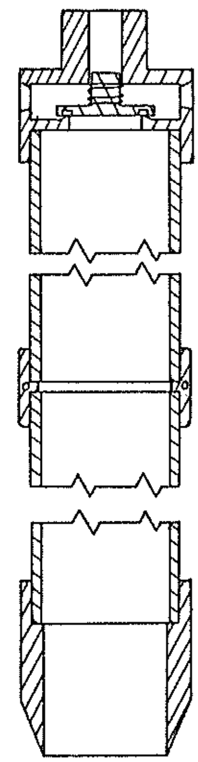


Fig. 5

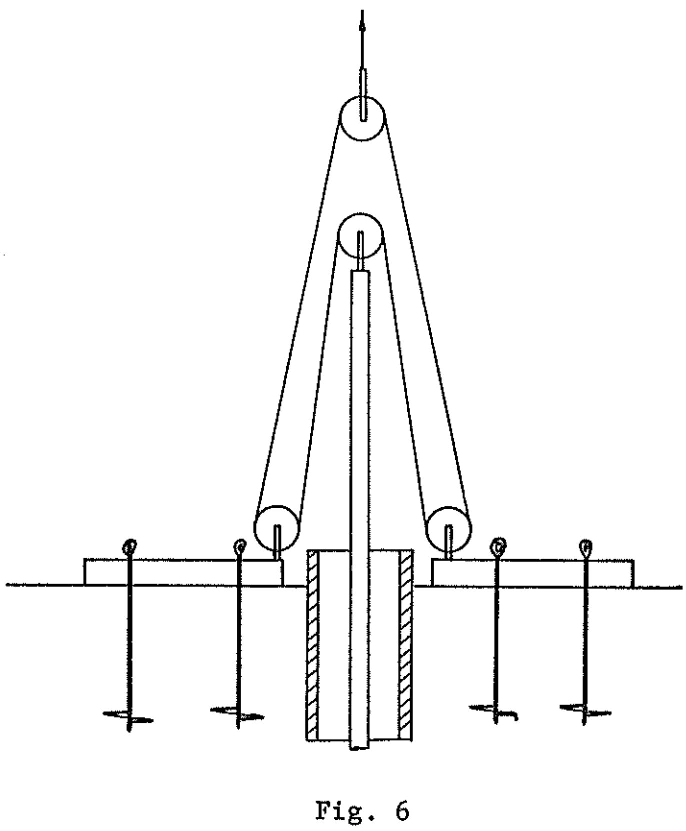


Fig. 6

LOGGING BOREHOLES, HANDLING AND TRANSPORTATION OF SAMPLES

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In considering aspects such as logging boreholes and handling and transportation of samples, it is pertinent to remember that the basic aim with a building foundation investigation is the collection of facts about the nature and condition of the in situ soils. With most building foundations we are concerned with the undisturbed condition of the soils.

Available drilling and sampling techniques produce samples and cores in a variety of ways but these procedures impose quite severe limitations on the form and condition of samples that are extracted from the ground. Similarly, conventional laboratory testing equipment imposes quite definite restrictions on the size and form in which samples must be prepared for testing. These two sets of limitations, together with limitations arising from the nature of a given soil, are very significant factors in determining:-

(a) how a sample or core can be handled from a drill site to the testing laboratory;

(b) the extent to which the sample is disturbed in the transfer process; and

(c) the value and significance of any test results obtained for the samples.

In any discussion of these factors, it is considered particularly important to keep the basic objective to the fore, i.e. the collection and production of facts about the soil and its condition in situ. Therefore in the following, the limitations of procedures, particularly with respect to sample disturbance, will be emphasised.

HANDLING AND TRANSPORTATION OF SAMPLES

Scope and Basic Requirements

When a core or sample is extracted from a borehole, several operations have to be considered. Those that influence the value of the end result are as follows:

(a) Removal of the core or sample from the sampling device. This will be done either at the drill rig or at the laboratory, depending primarily on the type of sampler.

(b) Transport of the core or sample from the drill site to a testing laboratory or suitable storage place.

(c) Storage. It is unlikely that samples will be processed immediately they arrive at a laboratory so they must be stored for a period prior to testing. The type of storage required will depend on the type of sample, its condition when received, and the testing required.

(d) Selection and preparation of test specimens.

When this stage is reached, one has to :

(i) Decide which piece of a sample or core it is most appropriate to select for testing and this often must be done when the soil is not visible.

(ii) Remove the sample from its container, e.g. extrude from a sample tube.

(iii) Form the sample to the appropriate shape and size required for testing. This may be partially achieved during extrusion from a sample tube but other trimming and handling operations are often involved.

In all the above operations there are two basic requirements that are of paramount importance and should be the deciding factors in how samples are treated. These requirements are:

(a) Protection against moisture change. It is particularly important to prevent moisture loss because apart from giving inaccurate information on insitu water contents, it is known that predrying of soils can lead to anomalous test results and difficulties with test techniques. Storage in a humid atmosphere can help increase the safe storage time provided there are precautions against moisture gain. The latter can lead to swelling and softening and be as damaging as drying; it must be avoided.

(b) Support during handling and transportation. The sample or core should not be required to support itself, otherwise deformations and hence disturbances will occur. Similarly, samples should be isolated from vibrations and shocks at all times. "In transit" procedures will vary with circumstances and it is not intended to discuss particular methods in detail. However samples and sample tubes should be firmly held in suitable containers or cradles and varied on or in shock absorbent material (e.g. thick slabs of foam plastic).

Principal Sample Types

The several types of sampler in common use have been described and discussed in the previous paper. These give samples and cores of varying quality and, so far as they affect the handling and transport of samples, can be grouped as follows:

(i) Thick-walled tube
(ii) Thin-walled tube
(iii) Split-tube or -liner
(iv) Ring liners
(v) Special samplers.

The main aspects of these categories are discussed below.

(a) Thick-walled and Thin-walled Tube Samplers

The 4 inch diameter by 18 inch long B.R.S. sample tube is the most common example of a thick-walled tube, while a variety of samplers use fairly long, thin-walled tubes. In most cases the sample is transported and stored in the sampling tube. However the ends of the sample need to be supported, and sealed off to prevent moisture loss. A carefully applied,thick plug of wax gives a satisfactory seal but will not adhere to the metal tube unless it is scrupulously clean and dry (not easy to achieve in the field). Also a ductile wax is needed to accommodate the shrinkage tendency of wax; a 50:50 mixture of paraffin wax and Shell petrolatum is satisfactory particularly when applied in several successive applications at just above its melting temperature.

For a sample to enter a sampling tube without too much drag from the walls, clearance is required on the cutting shoe or cutting edge. Therefore the sample will not be firmly held in the tube and movement and distortion can occur. This problem would not arise with a swelling soil but swelling itself is a form of disturbance.

The worst feature of tube samples is the extrusion method used to extract them. Even if the sample was a loose fit initially it will inevitably adhere to the walls of the tube and force is necessary to remove it. This force causes pressure to develop between the sample and the tube so increasing the resistance. As a result, samples more than a few inches long can rarely be extruded without risk of disturbance. This problem is most severe with B.R.S. tubes. These are often not rust - resistant and are difficult to cut up into shorter lengths. With thin-walled tubes, providing the tube can be sacrificed, the extrusion (and storage) problems can be minimised by cutting the tube and contained sample into short lengths before the sample is extruded.

The other important problem with tube samples is that except for each end, the sample cannot be seen. Therefore it must be extruded before test specimens are selected. This places a severe limitation on the subsequent handling and storing techniques that can be used, and/or on the effectiveness of specimen selection.

(b) Split-Tube Samplers or Split-Liners

Double-tube and triple-tube rotary core barrels are often used for sampling soft rocks and compact or cohesive soils. These core-barrels normally have a split inner tube or liner into which the core slides as it is cored. Clearance between the core and the split-liner tube minimises drag on the core but allows it to be affected by core barrel vibration. This, together with water entering through the split tube, can give considerable disturbance on the core surface. It is not practical to transport the core in the split-liner tube and therefore the following aspects need to be considered:

(i) Opening the split tube. Examination or removal of the core is commonly done after removal of one half of the split tube. This is best done by carefully sliding the top half lengthwise off the core while preventing movement of the core. If the top half of the tube is lifted off, suction and adhesion effects can easily pull friable and granular materials apart.

(ii) As-drilled condition. Good practice requires that the as-drilled condition of the core be logged before removal from the split tube. If pieces of the core are required or are worthy of further testing, these can be selected, removed from the tube, and suitably packed for transport. Disturbed samples are no problem - plastic bags are suitable containers - but if disturbance is to be minimised, then handling, packing and transport require considerable ingenuity, effort and time unless the material is inherently very competent. Without additional sets of split liners, all these operations hold up drilling progress.

(iii) Use of core boxes. In many instances, cores from split-tube liners are transferred immediately to core boxes for transport and storage. Such a core, though apparently intact, will be disturbed in the transfer process, has no protection against moisture change, and almost no support or portection against vibration and shocks. Logging and test specimen selection must therefore be done without delay if valid information is to be obtained. Sometimes cores are inserted into lightweight plastic tubes before being placed in a core box. This helps to reduce moisture losses but almost inevitably results in considerable disturbance. Assessed or measured strengths determined from cores handled and stored in core boxes must be treated with considerable caution.

(c) Ring Liners

Some of the difficulties of handling tube samples and cores in split-tube liners are overcome in samplers or core barrels that have an additional liner composed of a series of short rings or tubes stacked end to end. Ring liners greatly ease and simplify sample handling problems in the laboratory.

One tube sampler in use in New Zealand is a 3" O.D. x 2 3/8" I.D. split-tube containing a liner made up by 12 one inch long brass rings. The sampler is either pushed or driven into the ground. The rings are transported in sets of six, in a plastic bag, inside a neatly fitting metal can.

Another example of the use of ring liners is a triple-tube, rotary core barrel that has had extensive use in the Wellington area. This core barrel takes a 4" diameter core and has a series of two inch long, copper rings inside the normal split-liner. Up to three feet of core can be taken with each coring run. When removed from the core barrel, the rings and contained core need to be continuously supported and clamped tightly together. A metal channel as in Fig. 1 is used for transporting the core from the drill site to the laboratory. Similar channels are used to handle and store the samples in the laboratory; to minimise moisture change the sample plus rings are wrapped in a thin plastic sheet and stored in a humid room. With ring liners, even though the sample or core cannot be seen, and selection of test specimens is still a hit and miss process, the benefits are as follows:

(i) The sample, together with the ring liner, can be removed from the sampler or core barrel and transported back to the laboratory for further processing. The sampler is then free for further use.

(ii) By cutting between rings, suitable test specimen lengths can be obtained. Specimens rarely need to exceed two and one half diameters and can be individually extruded. Extrusion pressures and disturbance are thus minimised.

(iii) If desired, a single ring can be taken for a consolidation specimen and immediately tested in the sample ring without extrusion or trimming.

Laboratory Problems

(a) Storage. As mentioned before, it is unlikely that samples will be processed immediately they arrive at a laboratory and some provision for storing samples is therefore essential. The storage time should be kept as short as possible and in this respect, the particular testing and storage capacity of the laboratory must be taken into account when planning and organising a drilling programme. It is quite useless continuing with drilling if the cores and samples cannot be properly handled and stored. Even with waxed and carefully sealed samples a humid storage room is considered essential particularly while samples are being processed.

(b) Selection of test specimens and sample extrusion. The problem of selecting test specimens when the material cannot be seen and the disadvantages of extruding long tube samples have already been noted. These factors can be very important in determining the effectiveness of a given sampler and should be taken into account when setting up a drilling and sampling programme.

(c) Trimming to Shape. A variety of techniques must be available for trimming specimens to the shapes and sizes required for testing as laboratory equipment can impose severe limitations on test specimen sizes. Details are outside the scope of this paper except to note that the material type can determine how feasible it is to trim an unsupported sample. For example, friable and stoney soils are often difficult to trim smoothly and it may be preferable to test the soil at the cored diameter. Similarly, very soft soils and clean, non-cohesive soils, pose special problems of support during trimming.

LOGGING BOREHOLES

Purpose

The purpose of a borehole log is to convey a concise description of the soils encountered together with some information on their in-situ condition such as compactness, strength, structure, drainage characteristics, etc. In compiling a borelog it is essential to have a standard method of identifying soils and classifying them into categories which have distinct engineering properties. This enables all concerned to speak the same language, thus facilitating the exchange of information and experience.

Identification and Classification System

The "Unified Soil Classification System" (Corps of Engineers, 1953; also Earth Manual, 1960) is recommended as being the most logical and thus the most useful system available. This is a development of Professor Casagrande's Airfield Classification (Casagrande, 1948) which the Bureau of Reclamation and the Corps of Engineers, in consultation with Professor Casagrande, produced and adopted in 1952. It is based on the recognition of the type and predominance of the constituents of soils in terms of grain size, gradation, plasticity and compressibility. It divides soils into three major divisions namely coarse-grained soils, fine-grained soils, and highly organic soils. Fifteen basic groups are recognised and a specific group symbol is assigned to each group. Details of the soil classification system are given in Table 1. The British Standard Code of Practice Classification of Soils for Roads and Airfields (CP 2001:1957) has a similar basis and in fact conforms very closely to Professor Casagrande's original Airfield Classification. However there are significant differences particularly in the gravelly and sandy soils containing fines. When using either classification system, these differences should be clearly understood and as mentioned above, the Unified Soil Classification System is preferred.

Visual Examination

One of the most important aspects of the Unified Soil Classification System is that it enables soils to be identified and classified by means of visual examination and simple manual tests. It is emphasised that not only is it a means of deciding which group symbol to assign to a soil but that it has much more importance as a systematic framework within which word descriptions of soils are compiled. In fact any attempt to use group symbols alone, or as a means of reducing word descriptions, is strongly condemned. Knowledge of, and competence in using the classification system is considered to be very important in borehole logging work, but it should be appreciated that competency in borehole logging is only acquired after considerable training and experience.

Detailed Procedure and Terminology

A detailed treatment of the procedure and data to be recorded in visual identification and classification of soils is given in Designation E-3 of the Earth Manual. However it is stressed that the usefulness of borelogs depends largely on the use of standard terminology. The Unified Soil Classification System defines the granular soil components in terms of grain size, i.e. we have:

Cobbles
Coarse gravel
Fine gravel
Coarse sand
Medium sand
Fine sand

and the "fines" (silt or clay sizes) are defined according to their plasticity characteristics. However there are other properties that can be usefully recorded on a bore log such as grain shape, gradation, more specific comments on plasticity, cohesiveness near the plastic limit, dry strength, compactness (granular soils) and consistency (cohesive soils). All these aspects demand standardised terminology and suggested terms and descriptive adjectives are given in Appendix I. If correctly used, these terms have a fairly specific meaning and can convey a reasonably precise picture. However to give more definite and consistent meaning to some of the terms it is necessary to assign specific values, e.g., for plasticity, consistency and grain size proportions. These values are also given in Appendix I and help calibrate the terminology (see also Burmister, 1948).

Sometimes it may be desirable to include geological and other information (e.g. local soil names) on a borelog. This is admissible provided the additional information is not mixed in with the engineering description. A geological classification should be in terms of geological terminology and as such is a separate entity. Any attempt to merge the two terminologies will only result in confusion and misunderstandings.

Borelog Presentation and Preparation

The usefulness of soil descriptions, as in a borelog, depends a lot on the manner of presentation. The descriptions should be concise, precise, and arranged so that the basic soil description can be seen and assimilated quickly. A description such as "Grey stiff slightly sandy silty clay" tends to hide the principal components of the soil in among the detail of the description. The soil description achieves much more impact if arranged as follows:

Grey	SILTY CLAY; trace fine sand; stiff; soft when remoulded.

A typical page from a preferred form of borelog is shown in Appendix II.

Note that the principal part of the description is presented in capitals and put at the front of the main soil description. When scanning a borelog this arrangement helps the reader to quickly see what materials are involved. Similarly colour (which may or may not be a significant parameter) is separated out into a separate column as is consistency (which, in the example shown, should also be read as compactness) in the undisturbed and disturbed states.

To be able to prepare an accurate borelog, i.e. one where the soil profile is correctly and completely represented and the undisturbed consistency is correctly assessed, one really requires a continuous, undisturbed core. Gaps represent lost, and perhaps vital information, and their occurrence should always be logged. Sometimes the fine detail may not be significant but one rarely knows for certain which facts are significant to the particular problem till all are collected. In short, beware of jumping to conclusions; new information often changes the picture completely. Similarly when compiling and presenting a borelog, ensure that all observed facts are recorded. Too often the borelog that the designer sees is a simplification of the original facts and therefore an <u>interpretation</u> of them; the designer should make his own interpretation of a borelog in terms of other information he has of the site and the structure he is planning.

CONCLUSION

When obtaining and testing borehole samples, the aim should be to have truly undisturbed samples as near as possible. Yet it must be admitted that many of the existing sampling and handling techniques produce test specimens that are nowhere near undisturbed. Therefore in an attempt to highlight the problems, the above discussion has concentrated on the limitations, and sources of sample disturbance, in our current procedures. This was done in the belief that only when the problems are clearly recognised will there be any desire to find and use improved methods. Similarly if the limitations of the results of an investigation are clearly recognised and admitted, there is a much better chance that their significance will be correctly interpreted.

Borehole logging should be carried out within a systematic framework of soil identification and classification. Standard terminology should be used wherever possible together with a systematic method of presentation. Competency in logging boreholes will only be achieved after considerable training and experience.

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APPENDIX I

BOREHOLE LOGGING : STANDARD TERMS AND ADJECTIVES

Grain Size

Very coarse
Coarse
Medium coarse
Medium fine
Fine
Very fine

Grain Shape

Angular
Subangular
Subrounded
Well rounded

Gradation

Very uniform
Uniform
Poorly graded
Fairly well graded
Well graded

Description Terms for Quantities of a Given Size Fraction

Per Cent Passing	Term
1 - 10	Trace
10 - 20	Little or few
20 - 35	Some
35 - 50	And

Other Gradation Adjectives

Gravelly
Sandy
Silty
Clayey

Plasticity

Degree of Plasticity	Plasticity Index
Non plastic	0 - 1
Slight (trace) plasticity	1 - 5
Low plasticity	5 - 10
Medium plasticity	10 - 20
High plasticity	20 - 35
Very high plasticity	Greater than 35

Cohesiveness Near Plastic Limit

Very weak
Weak
Firm
Medium tough
Tough
Very tough

Dry Strength

Very slight
Slight
Medium
High
Very high

Compactness
(granular soils)

Very loose
Loose
Medium dense
Dense; compact
Very dense; well compacted

Consistency
(Cohesive Soils)

Consistency Grading	Unconfined Compression Strength (lbs/sq.ft)	Observed Condition or Behaviour
Very soft	Less than 500	Exudes between fingers
Soft	500 - 1000	Easily moulded
Medium Stiff; Firm	1000 - 2000	Can be moulded with strong finger pressure
Stiff	2000 - 4000	Impossible to mould with fingers
Very Stiff	4000 - 8000	Impossible to mould with fingers
Hard or Extremely Stiff	Greater than 8000	Brittle or very tough

APPENDIX II

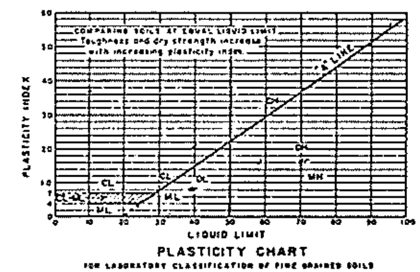
EXTRACT FROM TYPICAL BORELOG

LAB NO. 695.00 BORE NO. PC4

Depth	Sample Type	Colour	Description	Maximum Grain Size	Consistency	
					Undisturbed	Remoulded
60' 0"	MR 1		(This ring was mainly packing)			
	2, 3	Grey	CLAYEY-SILTY SAND; sand sizes are mostly fine.	-	Hard	Friable, but also cohesive
	4, 5	"	GRAVELLY SAND; angular.	1½"	Very Compact	Granular
	6		VERY SILTY FINE SAND; changing to	-		
	7, 8, 9	Grey with brown stones	GRAVELLY SAND; with few silty fines; becomes more gravelly with depth; mostly angular grains; fairly well graded.	1½"	Very Compact	Granular
61'6" – 61'10"	Tin from MS	Grey	VERY WEATHERED GRAVEL & SAND; breaks up to VERY SILTY SAND with some GRAVEL SIZES; angular to subangular grains. (Grading depends on breakup induced)	-	Compact	Friable and granular
62'0"	MR 1, 2, 3, 4, 5, 6	Grey	VERY WEATHERED SAND & GRAVEL giving VERY SILTY SAND; clayey in parts; contains some angular FINE GRAVEL SIZES	½"	Compact & Hard	Friable and mostly granular
63'0" – 63'4"	FB from MS	Grey	FINE GRAVEL & SAND; angular to subangular; with some CLAYEY SILT fines.	½"	(Disturbed)	Friable with some cohesive lumps
64'4"	MR 1, 2	Grey and brown-grey	GRAVEL & SAND sizes; angular; with few silty fines; includes one 3" stone.	Mostly ½"	?	Granular
	3, 4	Grey with orange brown stains	SILTY CLAY; inorganic; medium plasticity	-	Stiff	Firm
	5, 6, 7, 8, 9, 10	Grey (?)	(These rings taken for 1½" triaxial specimens but soil was not logged. Test results suggest a SILTY CLAY of medium plasticity)			
66'0" – 66'4"	Tin from MS	Yellow-grey and blue-grey	VERY SILTY SAND; mostly fine sand sizes AND VERY SANDY SILT; becomes more silty with depth. Also an occasional fine gravel size.	¼"	Very hard	Friable
66'5"	MR 1, 2	Mottled grey & yellow-br.grey	SILTY SAND; fairly well graded	-	Compact & Hard	Granular
	3	Grey	Inorganic CLAY; high plasticity (noted as SPECIAL in test results)		Hard	Stiff
	4, 5	Grey	VERY SILTY FINE SAND or SANDY SILT; low plasticity		Hard	Friable
	6 A	Grey	VERY SILTY SAND; mostly fine and medium sizes but a few coarse sized; subangular rotten grains.	-	Very Compact	Friable
	7 B	"	Thin layers (¼" approx.) SILTY FINE SAND and VERY SANDY SILT	-	" "	"
	8, 9 C		Similar to A in the same run but also an occasional fine gravel size.	-	" "	"
	10					
68'1" – 68'5"	FB from MS	Grey	FINE GRAVEL & SAND; angular to subangular; with few silty fines; plus a 2" stone at one end.	1"	Compact	Friable and granular
68'7"	MR 1, 2, 3, 4, 5	Grey	VERY GRAVELLY SAND; with some silty fines; fairly well graded; angular to subangular.	1½"	Compact	Granular
	6 A		SANDY FINE GRAVEL; subangular & angular; only trace fines	½"	Compact	Granular
	7 B, 8, 9 C	Grey	Thin layers of fairly clean FINE SAND alternating with SANDY SILT; generally becoming more silty with depth.	-	Compact & Hard	Friable

Table 1

UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION

Major divisions		Field Identification Procedures (Excluding particles larger than 3 inches and basing fractions on estimated weights)			Group Symbols ■	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
COARSE GRAINED SOILS — More than half of material is larger than No. 200 sieve size ■ (The No. 200 sieve size is about the smallest particle visible to the naked eye)	GRAVELS — More than half of coarse fraction is larger than No. 4 sieve size. (For visual classifications, the ¼" size may be used as equivalent to the No. 4 sieve size.) — CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			GW	Well graded gravels, gravel-sand mixtures, little or no fines.	Give typical name, indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information, and symbol in parentheses.	Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows:- Less than 5% GW, GP, SW, SP. More than 12% GM, GC, SM, SC. 5% to 12% Borderline cases requiring use of dual symbols. — $C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
	GRAVELS — CLEAN GRAVELS (Little or no fines)	Predominantly one size or a range of sizes with some intermediate sizes missing.			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.		Not meeting all gradation requirements for GW
	GRAVELS — GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).			GM	Silty gravels, poorly graded gravel-sand-silt mixtures.		Atterberg limits below "A" line, or PI less than 4. Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
	GRAVELS — GRAVELS WITH FINES (Appreciable amount of fines)	Plastic fines (for identification procedures see CL below)			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.	Atterberg limits above "A" line with PI greater than 7
	SANDS — More than half of coarse fraction is smaller than No. 4 sieve size. — CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			SW	Well graded sands, gravelly sands, little or no fines.		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
	SANDS — CLEAN SANDS (Little or no fines)	Predominantly one size or a range of sizes with some intermediate sizes missing			SP	Poorly graded sands, gravelly sands, little or no fines	EXAMPLE:- Silty sand, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine; about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	Not meeting all gradation requirements for SW
	SANDS — SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)			SM	Silty sands, poorly graded sand-silt mixtures.		Atterberg limits below "A" line or PI less than 4. Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
	SANDS — SANDS WITH FINES (Appreciable amount of fines)	Plastic fines (for identification procedures see CL below).			SC	Clayey sands, poorly graded sand-clay mixtures.		Atterberg limits above "A" line with PI greater than 7
FINE GRAINED SOILS — More than half of material is smaller than No. 200 sieve size.		IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE						
		DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)				
	SILTS AND CLAYS — Liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.	Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses.	
	SILTS AND CLAYS — Liquid limit less than 50	Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
	SILTS AND CLAYS — Liquid limit less than 50	Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity.	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.	
	SILTS AND CLAYS — Liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.		
	SILTS AND CLAYS — Liquid limit greater than 50	High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays.	EXAMPLE:- Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)	
	SILTS AND CLAYS — Liquid limit greater than 50	Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity.		PLASTICITY CHART FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS
HIGHLY ORGANIC SOILS		Readily identified by color, odor, spongy feel and frequently by fibrous texture.			Pt	Peat and other highly organic soils.		

■ Boundary classifications - Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.
■ All sieve sizes on this chart are U.S. standard.

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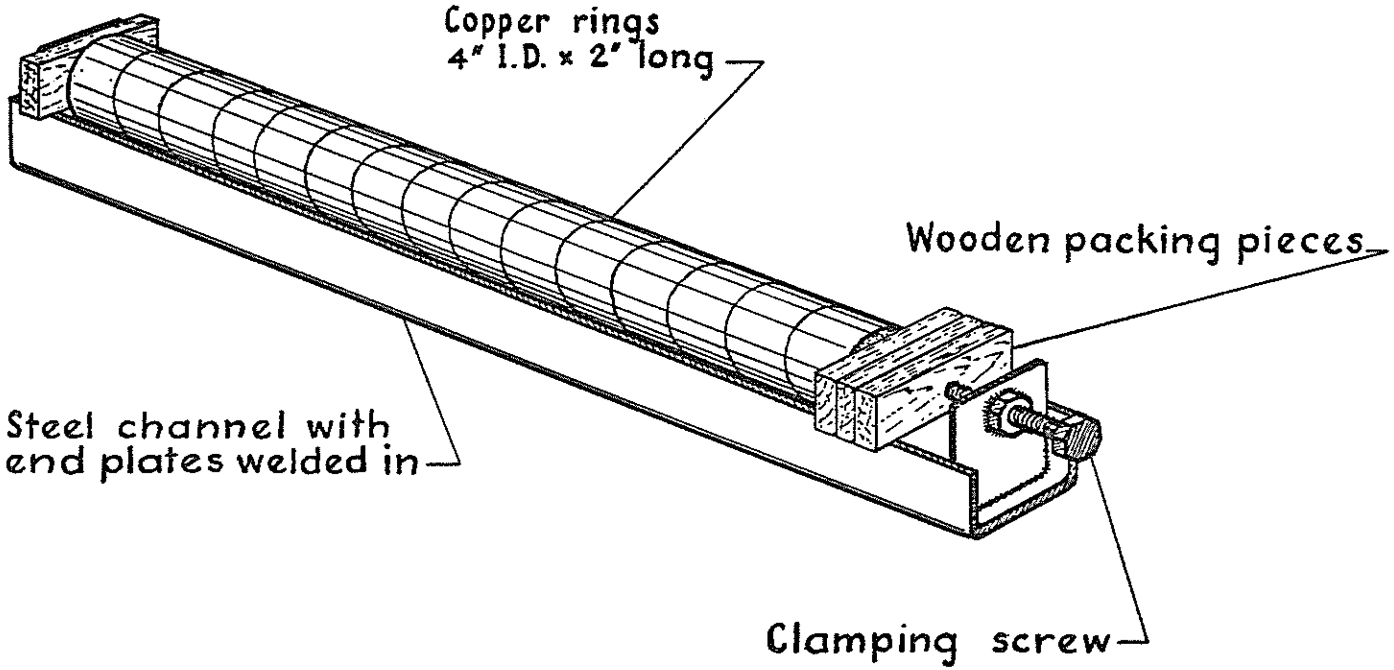

FIG.1 CARRYING CHANNEL FOR RING LINER FROM TRIPLE TUBE BARREL

FIELD TESTING OF SOILS

K. H. Gillespie

Brickell, Moss, Rankine & Hill, Lower Hutt

In the development of practices for evaluating the engineering properties of soils, field test procedures have received much less attention and research than has been accorded laboratory methods. Over the past decade, world-wide effort has been concentrated on refining laboratory techniques and applying the results to conditions assumed to exist in the field. Yet, the inherently variable nature of subsoil and groundwater conditions frequently makes representative sampling and laboratory testing difficult or even impossible. The structural or hydraulic action of a mass of soil may be influenced greatly by undetected and untested strata or discontinuities. Field tests, on the other hand, if properly conducted and interpreted, can reflect this variable nature and indicate the true action of a soil mass.

No soils engineer would ever suggest that field tests should be substituted for laboratory tests on soil samples. Field and laboratory tests should be complementary, each being used at the optimum time in the design-construction sequence. An excellent illustration of this combined function is provided by the design and construction of driven piles. Although the load capacity of a single pile, founded at a given embedment, can be assessed from the results of laboratory tests, it is normal practice on major construction to require the confirming evidence of field loading tests on test piles. These tests are usually performed in the early stages of construction. It is also good practice to observe and record the penetration resistance of all piles throughout the construction period for comparison with test pile data and the laboratory results. Thus the three steps - laboratory soil tests, field loading tests and field measurement of resistance - are used in combination to ensure satisfactory foundation performance. Through engineering imagination and development, similar thoroughness is possible and necessary in other soils engineering applications.

Although geophysical methods of exploration come under the broad title of this paper, they are not discussed as this specialised science has been expounded by a previous speaker.

"DOWN-THE-HOLE" TESTING

The most commonly used methods of field testing are those which can conveniently be performed in or alongside soil exploration borings. These include the dynamic penetrometer (Standard Raymond penetration test) and its many variations, the static penetrometer (Dutch cone apparatus), and the field vane. Both the dynamic and static penetrometer methods involve empirical procedures, whereas the vane test provides direct measurement of soil shear strength. It should be noted that all three methods require interpretation of results based on a knowledge of the actual soils being tested. They therefore cannot be used alone in investigation work and must be considered as supplements to normal drilling, sampling and laboratory testing. Penetrometer techniques have been dealt with previously in papers circulated for discussion within the Society (1,2,3).

Dynamic Penetrometer

The so-called Standard dynamic penetrometer involves a 2 inch O.D., 1.3/8 inch I.D. open nose hollow spoon which is driven into the subsoils by a 140 pound weight free-falling through 30 inches. The spoon is driven 18 inches into the subsoils and the number of blows to drive the final 12 inches is recorded as the penetration resistance ("N" count).

Without doubt, the dynamic penetrometer test is the most mis-used and abused tool employed in site investigations in New Zealand. Even in this current age of supposed enlightenment, drillers are regularly engaged to drill borings to depths that are determined even before investigation starts and only to carry out dynamic penetration tests at 5 feet depth intervals, regardless of the soil conditions. The resulting information is, in many cases, useless and indeed even dangerous. It can only be assumed that penetration tests are made because drillers charge nominal amounts over and above normal drilling rates for carrying them out. In considering the large number of high-rise buildings that have been erected on alluvial areas of New Zealand using foundations designed from such inadequate data, it is indeed marvellous that there have been no catastrophic failures.

Extensive research has been made, particularly in United States, to correlate penetration resistance with relative density when testing sands. Although apparently conclusive results have been published in a number of papers based on such research,these can only be applied in practice with extreme caution.

As with all empirical testing, the primary requirement when using a dynamic penetrometer is to follow every aspect of the standard procedure rigorously. Most soils engineers will have supervised some form of dynamic penetration testing but how many have ever taken the time to measure the weight of the drop hammer, the length of its free fall or even the diameters of the penetrometer spoon? Similarly, although the standard test should be performed with size "A" drill rods, it is common to use whatever rods the driller happens to carry.

Many ad hoc variations of the standard dynamic penetrometer have been developed and subsequently applied to more general practice. In dense or stiff soils where a very high blow count of say more than 100 per foot is indicated by initial driving, it is often prudent to limit the number of blows to on the order of from 50 to 60 and to measure the resulting penetration. The main purpose of limited driving is to prevent damage to equipment but it has also been found that the drill string can generally be withdrawn from the boring again by one or two winch ropes, thus avoiding the time-consuming problems of having to jack the string out.

In dense, gravelly soils the nose of a hollow spoon penetrometer is frequently damaged during driving. Where this damage is caused by a large piece of gravel being driven ahead of the penetrometer, the resulting blowcount may err excessively on the high side. A variation introduced to overcome this problem is to fit a solid 60^{o} conical tip to the penetrometer. A similar variation of the test, known as the continuous dynamic probing, comprises a 2 inch diameter 60^{o} cone which is driven into the ground on the end of a string of size "A" drill rods. By plotting penetration resistance in blows per 6 inches against depth, some relative indication is obtained of variations in subsoil strata. At best, this procedure is only suitable for infill work after normal drilling and sampling procedures have established general subsurface soil conditions.

Static Penetrometer

Static sounding tools were originally evolved in Scandinavian countries early in the century. Subsequent to these, the Netherlands Department of Public Works developed the now widely used "Dutch Cone" apparatus. In its most elementary form this consists of a 1.4 inch diameter 60^{o} cone attached to the lower end of a 5/8 inch diameter rod surrounded by ¾ inch gas pipe. A record is kept of the force required to push the cone into the ground at a constant velocity; after short lengths of penetration, the gas pipe is pushed down to the cone and the cycle is repeated. Various refinements have been developed for the Dutch Cone, particularly to reduce friction between components of the apparatus.

The prime use of the Dutch Cone is for rapid infill work between exploratory borings which indicate subsurface soils comprising soft silts, clays and peats. All apparatus required for sounding such soils to depths of up to 40 feet can be carried in the field by hand. Although the apparatus is widely used in firm or stiff fine grained soils and in sands, the equipment required to penetrate the cone through these materials is beyond the scope of hand-work and it is usually then set up on a drill rig.

As with dynamic penetrometers, extensive research has been carried out on the Dutch cone to relate penetration resistance to soil strength parameters. Some workers in the field claim the Dutch Cone can be used to evaluate foundation bearing capacities and settlements and even to classify soils. Although such claims can possibly be proven within local areas of uniform soil conditions, it is necessary to constantly bear in mind that the Dutch Cone is an empirical test.

Field Vane Test

The field vane has been developed for obtaining the in-place shear strength of soft to moderately firm, fine grained soils. The equipment consists of four thin rectangular blades connected to a small circular shaft to make a cross in section. Generally the height of the vane is about twice its width. The vane is pushed into the soil and then twisted at a controlled rate of rotation until the soil is ruptured. The failure occurs on the surface of a cylinder which has diameter equal to and height very nearly equal to those of the vane. From the maximum moment needed to fail the soil and the surface area of the cylinder, the shear strength at rupture is readily computed.

The vane can be used in a cased or uncased hole which has been advanced by any form of drilling, provided it is pushed straight into the bottom of the hole a sufficient distance to be beyond those soils disturbed by the drilling operations. Alternatively, it can be used in soft, fine grained soils, without drilling a hole,by forcing the vane and rods into the soils. In either case the soil is tested by applying a moment at ground level until a maximum value is reached; after recording this value, the soil in the test zone is remoulded by turning the vane rapidly through several revolutions and repeating the test procedure to obtain a form of remoulded strength.

The calculation of shearing strength from vane test results is made on the assumption that the block of soil being tested is entirely homogenous. If the soil contains thin lenses of stronger soil or of sand, then the results of the test may be very misleading.

In-Place Permeability Tests

The permeability of soils is a most important physical property since some of the major problems of soils engineering originate from drainage problems encountered in the construction of structures. These problems include drainage of highways and airports, seepage beneath earth dams, uplift beneath structures below ground water level, dewatering of excavated sites and seepage pressures causing earth slides and failures of retaining walls. In all of these, the permeability of soils has a considerable influence on the effective strength of the soils and hence on their responses under application of stress. Whereas free draining soils will act as open systems and have a fully effective shearing strength, soils of low permeability may act as closed systems under rapid application of stress with the development of pore-water pressures and consequent reduction in shearing strength.

Permeability tests are generally carried out in the laboratory but there are occasions, particularly in granular, non-cohesive soils, when the natural structure of the soil has an important influence on the test results. In-place field tests are then necessary.

In concept and execution, field techniques are simple and inexpensive and, if reasonable care is taken, fairly reasonable values can be computed. The test is performed by first cleaning carefully to the bottom of a boring which has been drilled and cased to a particular depth, as is customary before sampling. The water level in the casing is then brought to ground surface by the addition of water and maintained until all entrapped air has escaped. The water level is then allowed to drop for a period of 10 to 30 minutes, the rate of drop being recorded at frequent intervals. It is also possible to run the test by adding water at a rate just sufficient to maintain a constant level in the casing. In either case, the water in the boring may contain silt in suspension which would tend to settle to the bottom of the hole as the test proceeds and affect the test results. This can be overcome by initially bailing water from the borehole and measuring the subsequent rate of rise of water level, but great care is necessary to avoid piping. It should be noted that these techniques are usually only suited to soils which range in permeability from about 0.1 to 0.0001 cm/sec. To extend the range in which the method could be used during a normal site investigation would require refinements and expense out of proportion to the potential accuracy of the final results. For special cases, several pieces of apparatus have been devised and are described in soils mechanics literature.

BEARING TESTS

An important area of field testing which appears to receive only limited attention in New Zealand is that of bearing tests. These include shallow and deep founded plate bearing tests and the many forms of pile load tests. Bearing tests are used by designers and soils engineers to confirm their design assumptions but these are more often than not severely limited by financial considerations and, as a result, tend to be too few in number and conducted for too short a testing period.

The aim of a bearing test is to measure the performance, under load, of a prototype foundation on or in a given soil. Such a test is rather elementary in performance, but the interpretation of its results is not simple as there are no invariable relations between the soil response in a small scale test and the overall performance of a complete structure. Factors which influence bearing tests and must be fully considered include soil character, preconsolidation, imposed stress conditions, rigidity of bearing area and confinement conditions.

For fullest economic utilisation of foundations, bearing tests must accurately determine the yield point load of the prototype, foundation. Yield point load may be approximately defined as that point on the load - settlement curve at which an increase in load produces a disproportionate increase in settlement. In practice this definition is difficult to recognise and several alternative methods have been formulated. Probably the most successful of these is that developed by W.J. Housel; it involves plotting the rate of settlement for the last 30 minutes of each load increment against total load. For loads less than the yield value, the rate of settlement is usually found to be negligible. Beyond the yield value, the rate increases rapidly. Two lines drawn through the points thus plotted will intersect at the yield point load. Having accurately defined the yield value and by selecting a factor of safety commensurate with the design and end-use of the structure, bearing values may be calculated.

POST-CONSTRUCTION OBSERVATIONS

After a comprehensive site investigation, laboratory testing and analyses, a soils engineer is able to predict the settlement performance of a proposed structure. This information is accepted by the structural engineer and allowed for in design, but is rarely confirmed during and after construction. By closer liaison between all parties concerned, valuable information could be obtained at only small expense. The problem is that of responsibility and rather than being left to individuals should be administered at local or preferably national government level.

REFERENCES

1. "Notes on the Use of the Standard Penetration Test (SPT)" by R.M. Tonkin.

2. "The Dutch Static Soil Penetrometer" by L.D. Wesley.

3. "Use of the Dutch Deep Sounding Penetrometer In New Zealand" by T. Belshaw.

INTRODUCTION OF PAPERS, SESSION 3

CHAIRMAN:

Mr R.Shepherd, University of Canterbury

THE DRILLING ORGANISATION by W.L. Cornwell

Mr Cornwell discussed the important position which the drilling unit occupies in sub-surface exploration. He outlined the basic stages of site investigation and stressed the need for the drilling team to receive adequate briefing. He described the normal type of drilling unit operating in New Zealand and the range of equipment available. He also gave details of a system of field records for bore-hole data and job costing.

DRILLING AND SAMPLING TECHNIQUES by R.D. Northey and R.F. Thomas

The paper was introduced by Mr Thomas who described the various methods of bore-hole drilling that are available in New Zealand. He then went on to discuss the quality of sampling with relation to the tests which are to be made. He suggested that there was a lack of understanding of the skill and equipment required to obtain undisturbed samples, to carry out laboratory tests and to interpret the results.

LOGGING BOREHOLES, HANDLING AND TRANSPORTATION OF SAMPLES by R.O. Bullen

Mr Bullen said that the basic aim of site investigations was to obtain data on the in-situ soils. He described the processes that a core or sample passed through between recovery at the bottom of a borehole and testing in a laboratory machine. For borehole logging, Mr Bullen recommended the use of the Unified Soil Classification System. He emphasised the need for clear and accurate logs based on a systematic method of presentation.

FIELD TESTING OF SOILS by K.H. Gillespie

Mr Gillespie detailed various forms of field testing and said these comprised only part of an investigation, being complementary to good drilling, logging and laboratory testing. He discussed field bearing tests for shallow and deep foundations and gave details of a high capacity pile load test carried out recently in Auckland.

DISCUSSION, SESSION 3

Mr T. Belshaw, in a written submission, suggested that Mr Bullen had stressed the triple tube barrel sampler a little too much, since the majority of foundation problems are caused by alluvium. He asked Mr Bullen to explain why the method of logging appeared to ignore the Raymond penetrometer.

To Mr Gillespie he stated that the "Dutch Cone" should not be mixed up with the precise deep sounding cone developed by Delft. He claimed that few worthwhile soils procedures are not empirical.

Mr Bullen said he had not intended to over-stress the triple tube sampler or any other sampler; each has its own application. He said that, as a sampler, the Raymond penetrometer was almost useless because of the high end area ratio.

Mr Gillespie said that the static penetrometer, no matter what it was called, was an empirical test and was not the answer to every soils problem. He said it was of assistance as an in-fill tool between properly drilled and logged boreholes.

Mr Taylor spoke on the cost of investigation work. He said there was a need for quality control on a rational basis which would enable sound engineering at reasonable cost. He pointed out the prohibitive expense involved in a suggestion by Mr Bullen that soil samples should not be carried by public transport.

He emphasised his earlier remarks on two-stage investigations, saying that the first stage should determine what the subsoils are and the second stage should be to sample the critical strata. He suggested that all gear for advancing the borehole and for first stage soil recovery should be provided by the driller and that equipment for second stage sampling should be provided by the laboratory.

To Mr Thomas he suggested the use of a vacuum pump to assist in retaining a sample within the sampling tube as it is withdrawn.

To Mr Cornwell he said the work load of a drill rig is not improved by integrating laboratory and driller.

He said there is not a sufficient volume of soils exploration work available in any one area of New Zealand and a specialist laboratory/driller team would have to travel extensively to be fully employed. He emphasised that the driller's responsibility goes beyond just running the rig and must include bringing cores or samples back to the ground surface as required.

Mr J.D. Moss (Brickell, Moss, Rankine, & Hill, Lower Hutt) agreed with Mr Taylor's remarks on the costs of investigation work. He said he had analysed the costs of site investigations he had carried out over several years and found the following:

Function	Percentage of Total Cost
Field (including driller)	55 to 75
Laboratory	5 to 10
Engineering Analysis	5 to 15
Consultation and Report	10 to 15

He said that in his experience, the cost of a foundation investigation for a commercial building would be from 0.35 to 0.75 percent of capital cost of the building. For engineering structures such as bridges the cost may be up to 1 percent of capital cost.

Mr Moss pointed out that in private practice one must always look to cost and therefore seek a compromise between cost and quality. He said it was essential to provide a field supervisor to work alongside the driller to log the boring, determine the frequency and type of sampling and pack the samples.

Referring to sampling techniques, Mr Moss said that his firm had adopted an American method of thick-walled sampling which was a compromise of other systems. He illustrated the sampling equipment and a variety of bits suitable for various soil conditions. He said the sample enters the sampling tube and is permanently retained in brass rings or split-tube liners. This provides specimens which can be directly used, without re-extrusion, for strength and consolidation testing and reduces both sample disturbance and costs.

<u>Mr Bullen</u> referred to Mr Moss' contribution and pointed out a potential problem in laboratory testing if the soil is not tight fitting in the liner ring. He also claimed that it was essential to obtain a continuous undisturbed core from a borehole.

To Mr Taylor he said that a two-stage investigation is sound practice but not always possible.

<u>Mr Thomas</u> referred to Mr Moss' remarks on costs. He said that he spoke as a scientist and was therefore obligated to advocate the best of practices. He pointed out that it was up to the engineer to determine any compromise between quality and cost. He claimed that non-extrusion from liner rings before testing could leave up to 5 percent of soil which may be disturbed.

<u>Mr Cornwell</u> spoke on evening out the work load for drillers. He said that site investigation work is not a full time occupation for New Zealand drillers and they must therefore rely on other employment such as well-boring.

He agreed with Mr Moss that it was essential to have a field supervisor working with the driller.

<u>Mr Blakeley</u> asked about the relative merits of payment for drilling on footage or on hourly rates. He said that payment on a footage rate encourages high drilling speeds and this must affect the quality of samples. He claimed that payment on the basis of footage of core recovered was an even worse method. Mr Blakeley advocated that all drilling should be paid for on hourly rates with the actual hours worked being agreed daily on site between the driller and the field supervisor.

<u>Mr Cornwell</u> replied to Mr Blakeley by quoting Mr Taylor's paper,page 1-8 and his own paper, page 3-6 where both advocated payment by hourly rates. He pointed out that if payment was made on core recovered, it was just not economic for a driller to waste time trying to recover a difficult stratum that comprised only a small percentage of the borehole. However this is invariably the very stratum which the soils engineer wishes to have sampled.

Mr Blakeley asked for an indication of the frequency with which payments were made on footage and on hourly rates.

Mr Richardson spoke in support of payments on a footage basis. He quoted a recent job in Wellington where the driller was employed on an hourly basis and the actual costs worked out at between $30 and $50 per foot of hole drilled. He claimed that the job should have cost approximately $5 per foot.

To Mr Bullen he said that the Raymond penetrometer is useful as a form of sampler because, although the soil may be disturbed it can at least be visually classified.

Mr Richardson stressed the need for hand-excavation if the first few feet of any boring where underground services were likely to be present.

Referring to Mr Bullen's paper and transportation of samples, Mr Richardson said there was great need for education of both drillers and field supervisors. He said his firm commonly encountered jobs where the field supervisor damaged samples through complete lack of transportation facilities.

To Mr Gillespie he said that the static penetrometer is a very limited tool. He said it may be satisfactory as an in-fill tool to follow the surface of an underlying strata such as a layer of gravel but the results could be very misleading if such as a buried log or isolated boulder was encountered.

Mr Bullen replied to Mr Richardson about the Raymond penetrometer as a sampler. He said that if this was the only form of sampling carried out then the engineer was deceiving himself.

Mr Cornwell described the extent to which his organisation sometimes had to go to obtain samples of a particularly difficult soil stratum. He said that, in isolated cases, this could raise drilling costs up to as much as $90 per foot but was only carried out if fully warranted.

Mr Dodd asked Mr Bullen to explain the way in which samples should be placed for storage. He said it appeared inevitable that some drainage must occur. He also asked Mr Bullen how part of a sample could be removed from a BRS tube without extrusion.

Mr Dodd said he was pleased to see that the classification system advocated by Mr Bullen did not include "loam". He suggested that a useful addition to the system would be an intermediate plasticity section for silts and clays (liquid limit 35 to 50).

Mr Bullen replied that the position of sample storage was not important provided it fitted snug in the tube and was firmly supported at both ends. He said that the most satisfactory method of cutting a sampling tube was with a slow-moving band saw.

Mr Bullen agreed with Mr Dodd over words such as "loam" and "pug" and went on to advocate that boring logs should describe the soils strictly in engineering terms. If geological classification was also required this should be completely separate.

Mr Galloway pointed out that a further misnomer was the term "well-graded" which in geology and engineering had completely separate meanings.

Mr Oborn said that the value of interpretation of a boring would be enhanced if geological terms were incorporated in the soil log. He said a correct description of colour was of primary importance and quoted the instances where this was of considerable geological significance.

Mr Faulkner said that drillers often became discouraged about quality of sampling because of poor handling by field supervisors. He quoted a case where cores had been left lying near the borehole for several months before being collected by the engineer.

Mr G.O. Woodward (Andrew Murray and Partners, Auckland) asked Mr Gillespie to comment on pocket type equipment such as penetrometers and shear vanes.

Mr Gillespie replied that such equipment was of very limited use as it was easily upset by small local variations in soils. He quoted a field use for the pocket shear vane in testing cores at regular intervals from a boring drilled in very soft muds. He said this provided a relative but not quantitative measure.

Mr H. Prestney criticised the smaller engineering practice who never supervised drilling or examined cores. On the question of hourly and footage rates he said drillers prefer hourly rates as this allows them to give a quality product.

Mr I.G.B. Wilson (Davies, Lovell-Smith and Partners, Christchurch) closed the session with a vote of thanks to the speakers.

SYMPOSIUM REVIEW

R.J.P. Garden, Consulting Engineer

INTRODUCTION

To attempt a comprehensive discussion of the lectures would take too long and would overlap undesirably with the contributions from the audience which are also to be reproduced. This reviewer has been asked to represent the views of an engineer who works outside the laboratory of the specialist, and the following remarks will be mainly devoted to expressing some contrast in view point.

BASIC OBJECTIVES

Out of the many aspects covered by Mr Taylor in the wide and well balanced presentation given in his paper, matters related to settlement repeatedly claimed attention. On this subject, there are two points worthy of emphasis. Observations of old buildings reveal that quite large distortions are often accepted without noticeably serious effects. Designers of new buildings can envisage large differential soil settlements and allow for them either by providing a flexible building (Fig. 1) or by having built-in means of relevelling the bases with hydraulic jacks, sand jacks etc.

The several references to "briefing", to the tailoring of building design to suit soils, and discussion of the need to have an appreciation both of the building and of soil properties, are relevant to the whole question of specialisation. As the specialist in soil mechanics will not often be the structural designer, and the structural designer will not often be a soils specialist, there is the problem of making decisions based upon a deep appreciation of both disciplines.

There is commonly an Architect involved just as deeply. The structural man is rather in the middle, and it is suggested herein that the structural engineer ought to have considerable competence in matters of soil mechanics. It is important that he should be able to recognise when a specialist is needed, and it is best if he has the competence to judge the significance of specialist advice.

In soil mechanics work there are times when meticulous testing and intensive theoretical analysis should be employed; there are times when one should not try to be too clever, when straight forward solutions supported by sound precedent are available at comparable final cost.

SAMPLING PRACTICE

Dr Northey's paper on overseas sampling practice gave us something of a refresher course on the need to avoid disturbance of samples. It would appear that soils may suffer a serious change in their basic properties, if given a slight nudge, and members of the audience were clearly concerned to follow the ramifications of this.

Typical of the aspects on which we asked for enlightenment are the following. If rigidity of soils can be greatly changed by disturbance, what do we know about behaviour during an earthquake disturbance and of the soil characteristics subsequent to an earthquake? If the "very highest" standards are necessary in obtaining undisturbed samples for determination of permeability, with what accuracy can we predict the residual permeability after distortion by consolidation settlement and other distortions of the soil mass in the field? One would have liked to hear the significance of thixotropy and other autogenous recovery in this connection. A speaker provided an interesting item of evidence related to this. He said that the Mecklenberg earthquake did not cause failure in muds which happened to be preloaded to approximately failure point, but on the other hand some old slip circles were reactivated and moved two feet.

One does not doubt the truth of the statement made to us that sampling of a suitably high standard would enable lower factors of safety to be used in design, but it is surely also true that a suitably full understanding of both the laboratory and the field conditions would be necessary to justify the use of low safety factors with soils so neurotic as to demand a feather bed in the laboratory.

It was unfortunate that Dr Northey could not be with us to present and amplify his important paper, but we were grateful to Mr Thomas for answering so many questions.

GEOLOGY

Aspects of Geology make fascinating lectures and Mr Oborn deserves special thanks for the outstanding quality of the many illustrations he projected on the screen. It appears that engineers in this country do not avail themselves of the available geological services to the extent which their works merit. If this is so, engineers should develop the habit of calling in a geologist. Whether paid for, or free, the advice can be most valuable, and Mr Oborn's lectures would convince his audience of this.

The effect of rock attitude upon the value of its load resistance and the degree of tsunami risk to which various parts of our coast-line is exposed, are two samples from the lecture to illustrate matters of direct engineering interest covered by his lecture.

The remark made at the Symposium is repeated. The inclusion of a unit of General Geology in the course for B.E. would contribute both technical value and liberal education.

GEOPHYSICS

The seismic, electrical and magnetic methods of site investigations, well explained by Mr Ingham and invoking discussion from the audience were well placed as a transition from geology towards the next three papers on drilling and sampling. The foundation engineer must keep in mind that geophysical methods are some times the best and some times the only appropriate method of site investigation. The lecture devoted time to explaining when and how these methods could best be used.

DRILLING OPERATORS

At the Symposium there was insufficient time for discussion with the drilling contractors who attended. This reviewer strongly supports the comment by Mr Moss that engineering staff must be kept in attendance on field drilling and sampling operations. The validity of the results being recorded is of such importance that every check is demanded. It is equally important to obtain the help from a competent drill operator, the devotion of both members of the team is needed to avoid the all too common misinterpretations. The man who should not be allowed on the job is the know-all engineer or operator, who will not keep an open mind.

Given a little personal vigour, manual methods of exploration can be used to do certain jobs more promptly and cheaply than by machine power. We have, in answer to an urgent summons, carried out hand drilling and continuous probings to 25 feet depth within the space of time one would need to find out how many weeks it would be before the nearest rig would be available.

It is a pleasure to watch the skill with which office staff can repeatedly throw the 140 lb. weight of the penetrometer accurately up to the chalk mark. It has been observed that this rythmical exercise has a beneficial effect upon costive draughtsmen, but it is claimed to be harmful to those who earn their living by manual work. Even those who train all year round to gain a place in the local rugby team would be unlikely to join drilling teams one has used overseas to drill and continuously sample, without the help of any machine power whatever, cased holes of over 200 feet depth.

DRILLING AND SAMPLING

Messrs Cornwell, Northey/Thomas, Bullen and Gillespie in their four papers, each had the task of making short presentations on matters that can be argued at length. The papers contain, nevertheless useful summaries of fact and also expressions of opinion worthy of respect.

Along with the versatility of equipment demanded by Mr Cornwell, it is essential that the investigator should keep wide awake during all stages of the exploration and sampling, so that procedures can be changed immediately upon the need for such change being discernible by the operator, by the field engineer or by the laboratory. Out of experience, it is suggested that the best operator is the owner of a drilling rig, running the rig himself in person. No other class of operator has so great an interest in giving satisfactory service or in accumulating knowledge of the techniques.

The Engineer should bear in mind the differences between site testing and laboratory testing and should know when each is appropriate. Field testing will evoke an intelligent interest from a good operator. Similarly, for quality control during construction, the foreman can be expected to be a better member of the team if he can carry out site testing himself. Designers should remember that quality control during construction is primarily the contractor's responsibility. The Contractor's activities becomes more meaningful, the more he is involved in checking on the quality of his own achievements, and engineers who administer contracts should realise the long term ill-effects of displacing the contractor in this activity.

TESTING NON-COHESIVE SOILS

It is desired to take up some points regarding the testing of granular soils. Messrs Northey and Thomas state that "density is one of the few meaningful measurements that may be made on the cohesionless sand below the water table". Mr Gillespie's paper criticises strongly the misuse of the dynamic penetrometer and says "it is indeed marvellous that there have been no catastrophic failures" among the large number of high-rise buildings on alluvial areas of New Zealand using foundations designed from such inadequate data. There may be truth in these statements from both papers, but as they stand they are incomplete.

How often can density measurements be accurately obtained on a sample of non-cohesive material? Even when using elaborate means of sampling, such as impregnation in situ with a chemical grout, we have found the soils to exist in layers of significantly differing grading. Consequently the density can vary every inch and when the sample is remixed for laboratory determination of maximum and minimum densitites the results obtained cannot even be considered a useful average of the properties of the whole sample. In addition to this inadequacy of test upon the sample, the variation from sample to sample over the one site is likely to show such irregularity as to compound the difficulty of obtaining representative information. Much the same objection holds for load tests. In art. 54 of "Soil Mechanics in Engineering Practice", Terzaghi and Peck, discuss the shortcomings of SPT procedures, but say "Nevertheless the test results are a far more reliable basis for judging the allowable soil pressure than the results of a few conventional load tests".

Alluvial and littoral deposits of non cohesive soils generally show great irregularity of grading. In such soils, the laboratory tests for relative density, and the relatively few loading tests that can be economically made, must be regarded as attempts to calibrate penetrometer tests which can be economically carried out in considerable numbers. The penetrometer readings are at least actual readings of a resistance; the density tests in variable materials are lacking in both empirical and theoretical support. Of course, if one is fortunate enough to have uniform non-cohesive soils of sedimentary or aeolian origin, one is fortunate!

An interesting article by Hanns Simons, published this year discusses non-cohesive soils with special reference to driven piles. He says that penetrometer tests have not been looked at with great favour because a margin of 100% had to be allowed in properties determined. He says that it is now possible to predict vertical and horizontal load capacity of piles in non-cohesive soils with adequate provision by soundings or SPT, but states that estimates of settlement are highly inexact. Philcox (2) is another who has used SPT readings in connection with a large volume of piling work, and he has sought to record the relation between the N count and the load/settlement behaviour of piles. Sutherland (3) has also sought to tabulate and study correlations between N counts and settlement. Despite the crude nature of the penetrometer tests and the dangers of omitting to supplement with other testing, the quotations in this section of this review will illustrate that foundation engineers in five scattered parts of the globe, from Tiwai Point to Dusseldorf, find the need to resort to dynamic or static penetrometers.

CLAY SOILS

The appearance of precision given by the figures for strength of clays, factors of safety stated for slip circle failure etc., which are sometimes given out by enthusiastic investigators, does not inspire the experienced engineer with confidence. We are used to finding considerable scatter in the test results for the properties of steel, concrete, wood, brick etc even when our test pieces are of prismatic shape and the loads applied under the best control that can reasonably be contrived. When one considers the imprecision of knowledge of shape of failure surface, variation of moisture content and other characters of the clays which are likely to exist at the time of failure, claims of accurate prophecy of failure behaviour are best treated as evidence of unreliability of the prophets. The quotation from Peck, given at the close of Mr Taylor's lecture can be read with this comment.

ECONOMICS OF TESTING

In considering how much testing should be done on a site, how much design investigation should be done of economical alternatives, and what construction processes should be called for to make best use of the soil conditions of the site, the paramount factor is usually the time that will be consumed. With each year the national economy becomes less dependent upon the speed at which grass can be made to grow, and more dependent upon the speed of development of secondary industry. Any project with a capital cost of say $1 million must be worth at least $5000 for each month it is in service; otherwise the project is not worth initiating. For industrial projects, indeed, the value will on the average be much greater than is represented by that monthly figure of 0.5% of capital value. One must therefore bear in mind that any elaboration of testing, of design or of construction which is intended to save money, must allow in its saving achieved, for the cost of any protraction of completion date at a rate of $5000 to $10,000 per month for a $1 million project.

RESEARCH AND DESIGN

The aspect of economics mentioned in the foregoing will emphasise that however desirable it is to lose no opportunity to further research on our jobs, we can not justify delaying construction progress for the sake of research. Research needs time and money both of which are usually denied to the engineers who design and build. While practicing engineers will, if they are keen men, squeeze some research into their budgets of time and money, they must rely upon institutions set up for the purpose to carry out most research work. It may also be said that it is the duty of the research men to shape, check and reshape the results of their research so that they are brought into forms suitable for instant use by the design engineer. It may be regrettable that theory and practice should be so separated, but this is the direction in which we have been moving for many centuries.

The continuation of observations of the behaviour of structures after their construction is also most desirable, but is not easy to have carried out. Few owners of buildings have much interest in such activities and this attitude can extend to public bodies who "own" bridges.

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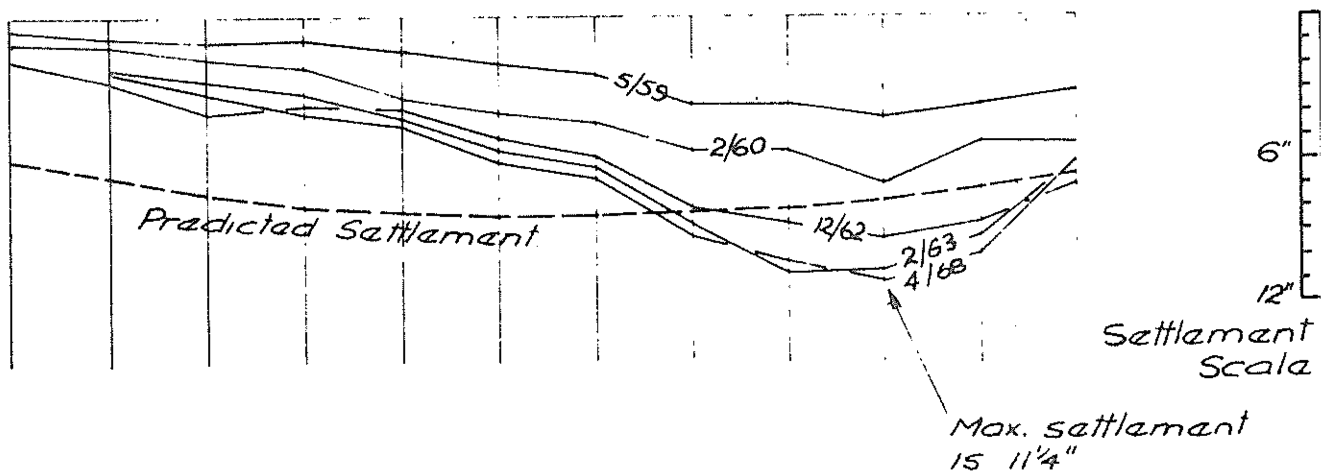

Plot of Settlements; showing month and year of Successive Measurements.
Concrete Wall 220 feet length, carrying 90 feet span roof.

Fig. 1