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LIQUEFACTION POTENTIAL OF DENSE BACKFILL
IN ONTARIO MINES

by

PLACER DOME INC.

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LIQUEFACTION POTENTIAL OF DENSE BACKFILL IN ONTARIO MINES

DSS FILE NO. 15SQ23440-6-9039

BY

PLACER DOME INC.

DOME MINE

TIMMINS, ONTARIO

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Sommaire

Ce rapport présente les résultats de l'évaluation du potentiel de liquéfaction des remblais à haute densité. Le projet fut réalisé conformément au contrat avec le ministère des Approvisionnements et Services (No. de contrat 15SQ23440-6-9030).

Le plan de travail consistait en une revue de l'état actuel de la technologie relativement au potentiel de liquéfaction des remblais, des essais en laboratoire et in-situ pour déterminer le comportement des remblais denses et des propriétes spécifiques de ces remblais dans des conditions réelles d'opération et à l'échelle du laboratoire, ainsi que d'effectuer des essais de sautage simulant l'impact de la séismicité sur les remblais asséchés communément appelé pâte de remblais. Les conclusions principales et les recommandations tirées de l'étude sont les suivantes:

1. Il existe peu de données sur l'analyse de la liquéfaction des pâtes de remplissage. La plupart des études récentes sur la liquéfaction portent sur les sols naturels.

2. Une revue de la littérature sur la liquéfaction indique que les essais de déformation, en régime permanent, sur la pâte de remblayage sont les plus appropriés pour identifier le potentiel de liquéfaction du matériel en question.

3. On a identifié les propriétés-clés in-situ et de laboratoire de la pâte de remplissage avec une référence particulière à la liquéfaction. L'avis général est que les pâtes de remplissage "En place" ne sont pas saturées et suivent un comportement dilatant. Ainsi, la possibilité de liquéfaction de ces genres de pâtes de remplissage est minime. Évidemment, le potentiel de liquéfaction est influencé par la cohésion, le rapport volumétrique des vides et des pressions de confinement. Un rapport volumétrique plus bas, un plus haut contenu de ciment et un confinement supérieur réduisent de façon importante le potentiel de liquéfaction.

4. Les pâtes de remplissage sont plus sujettes à la liquéfaction presqu'immédiatement après avoir été mises en place, lorsque les pressions sont encore élevées dans les pores. Du fait que la consolidation a lieu rapidement, le potentiel de liquéfaction baisse de façon significative et, à toutes fins pratiques, est négligeable dans les pâtes de remplissage cimentées.

5. Comme principe général, on recommande un contenu en ciment d'au moins 1,1% afin de minimiser le potentiel de liquéfaction dans les pâtes de remplissage.

6. On peut se servir du cône à pointe conique avec succès dans les pâtes de remplissage fermes afin de fournir des données sur l'évaluation de la liquéfaction in-situ et les propriétés des autres matériaux.

7. Les pâtes de remplissage peuvent soutenir des vibrations allant jusqu'à 2 g sans aucun danger de liquéfaction.

<u>Suivi</u>

Il s'agit d'une étude préliminaire qui exige des travaux additionnels. En particulier, plus de recherche s'avère nécessaire pour approfondir les connaissances sur les techniques dynamiques pour mener des essais sur les pâtes de remplissage en laboratoire afin d'évaluer leur potentiel de liquéfaction, et analyser davantage les méthodes qui ont servi lors de l'étude des propriétés in-situ des remblais miniers.

Executive Summary

This report presents the result on the evaluation of liquefaction potential for high density backfills. The project was carried out under contract to the Department of Supply and Services (Contract No. 15SQ23440-6-9030).

The work program consisted of a state-of-the-art review on liquefaction potential of backfill and soils, laboratory and in-situ tests to determine the behaviour of paste fills and index properties of these backfills under in-situ and laboratory conditions and specific blasting tests carried out to simulate the impact of seismicity on fine grained, dewatered tailings backfill, commonly termed paste backfill. The main conclusions and recommendations of this study are as follows:

1. Very little information on liquefaction analysis of paste backfill exists. Most of the state-of-the-art review on liquefaction is available from natural soils.

2. A review of liquefaction indicates that steady state deformation tests on paste backfill are most appropriate to identify liquefaction potential of this material.

3. Key in-situ and laboratory properties for paste backfills with specific reference to liquefaction potential have been identified. It is generally concluded that the paste backfills are not saturated in-situ and exhibit dilatant behaviour. The potential for liquefaction of these types of paste fills is, therefore, minimal. Obviously, liquefaction potential is influenced by cohesion, void ratio and confined pressures. Lower void ratio, higher cement content and increased confinement significantly reduce the liquefaction potential.

4. Paste backfills are most vulnerable to liquefaction almost immediately after being placed when pore pressures are still high. As consolidation occurs rapidly, the potential for liquefaction decreases significantly and, for all practical purposes, is considered negligible in cemented paste fills.

5. As a general guideline, a cement content of at least 1.1% is recommended to minimize liquefaction potential in paste fills.

6. The cone penetrometer can be successfully used in stiff backfills to provide information on in-situ liquefaction assessment and other material properties.

7. Paste fills can withstand maximum vibrations of up to 2 g without any danger of liquefaction.

Follow-up

This is a preliminary study that requires additional work. Specifically, more research is necessary to investigate dynamic techniques for use in testing paste fills in the laboratory to assess their liquefaction potential, and to expand upon the methods used in this study to examine in-situ properties of mine backfills.

ACKNOWLEDGEMENTS

This report has been completed as a joint effort by several personnel from Dome Mines Limited. The overall coordination and project management was provided by Mr. Dale Churcher. In addition, Dome Mines would like to acknowledge the invaluable assistance by CANMET's scientific authority, Mr. Alfred Annor and McGill University's Dr. F. Hasani and Mr. K Aref.

1.0 INTRODUCTION

This report is submitted in response to a contract between the Department of Supply and Services and Dome Mines Limited (File No. 15SQ23440-6-9309) on the Liquefaction Potential of Dense Backfill in Ontario Mines. The complete terms of reference for the report are briefly summarized below:

1. Carry out a literature search on liquefaction of soil deposits due to seismic activity.
2. Conduct index and physical property analyses related to liquefaction.
3. Study the effect of blast vibration on stability of high density backfills and their potential to liquefy.
4. Install in-situ instrumentation and monitor high density backfill behaviour with specific reference to liquefaction as a result of mining and blasting.

As specified in the original proposal of May 13, 1986 to the Department of Supply and Services, a large proportion of the project was carried out on subcontract by the following organizations:

McGill University	State-of-the-art review on liquefaction
McGill University	Static tests on backfill with specific reference to liquefaction.
Conetec Investigations Limited	Penetration and interpretation of seismic cone penetration test data
CIL Inc.	Characteristics of blasting vibrations in dense backfill.

The complete reports submitted by the subcontractors are attached in the appendices of this report or on file with CANMET. The main conclusions, recommendations and an outline of the project work carried out are summarized in the main report.

1.1 Background

Mill tailings have been used for underground backfill in Ontario mines since the late 1940's. It is estimated that up to 60% of the backfill material placed underground is produced from mill tailings. In the past, these tailings have been processed to remove a large portion of the ultra-fine solids, to produce a free-draining material. The classification process used to remove the fines results in a significant percentage of the solids being rejected. In some operations, up to 70% of the solids have to be disposed on surface. The resultant reduction in quantity of fill material available for underground support can seriously affect mining schedules, particularly in gold mining operations where liberation is only achieved with extremely fine grinding.

In recent years consideration has been given to the use of full stream mill tailings as an alternative support medium to deslimed tailings. The presence of the ultra-fines in this product, however, requires special handling to ensure its safe application in underground stopes. Because fine grained tailings are almost impermeable, excess water must be removed from the fill before placement. This can be accomplished through mechanical dewatering which produces a product with a level of saturation of typically less than 100%.

The Dome Mine operation of Placer Dome Incorporated in South Porcupine, Ontario, has developed a mine backfill system based on the concept of utilizing 100% of the mill tailings in the stope. To date, over 100,000 tonnes of dewatered tailings have been placed in underground stopes, utilizing specialized mechanical dewatering equipment to remove the excess transport water from the product, before placement into the stope. The resulting backfill material, containing less than 22% moisture, has a number of advantages over conventional hydraulically placed, classified fill. These include the following:

1. A larger portion of the mill tailings produced can be recovered for underground backfill, increasing the daily backfilling rates possible and reducing the cost and environmental impact of disposing of tailings on surface.
2. The water:cement ratio resulting from dewatered backfill produces a greater strength gain per unit weight of cement added. Therefore, in most applications less cement can be used to achieve adequate design strengths.
3. The stiff consistency of the dewatered backfill does not require containment in the form of fill fences or bulkheads, once placed into a stope.

It has been shown at Dome Mines that an economic alternative to the use of hydraulic fill exists, one which will result in a very high strength product with significant economic benefits.

The presence of the ultrafine solids in the high density fill, however, has raised a concern about the stability and possible liquefaction of the fill under cyclic loading conditions. Liquefaction can be simply described as the phenomenon wherein a massive soil loses its ability to resist shear forces. This usually occurs when subjected to cyclic or shock loading, which will cause the material to flow in a manner similar to a liquid until the shear stresses acting on the mass are as low as the shear resistance. The liquefaction potential of a given material can be determined through the study of a number of physical properties of that material and conditions to which the material is being subjected.

This report details the work conducted by Dome Mines to investigate the liquefaction potential of densified fine grained tailings backfill. This work was carried out in conjunction with a study commissioned by the Department of Supply and Services (File No. SQ.23440-5-9204) entitled " In Situ Determination of Dewatered Tailings Fill Properties in Ontario Mines" to develop an understanding of the characteristics and support potential of high density backfill at Dome Mine. These two studies were complimentary.

1.2 Scope of Work

The scope of work has been defined by DSS as follows:

1. Conduct a literature search to investigate the theory of liquefaction and determine the mechanisms and test procedures which would apply to the liquefaction potential of high density backfill.

2. Conduct physical properties analysis of the backfill solids being used by Dome Mines for the production of high density paste backfill. A series of index tests would be conducted to identify specific properties of the material.

3. Develop a suitable laboratory procedure to conduct a thorough analysis of the liquefaction potential of dense fill. This procedure must be practical and where possible, applicable to standard soil testing laboratories.

4. Conduct a range of field tests to investigate the in-situ properties of dense fill as related to the critical performance factors determined in the liquefaction investigation. Verification of the laboratory test procedure should rely on field results wherever possible.

5. Conduct dynamic field testing of the resistance of existing high density fill bodies to the effects of blastwave migration.

6. Determine the safe working limits which must be maintained in utilizing fine grained tailings for underground backfill to resist liquefaction.

1.3 Location

During the early history of operations at Dome, shrinkage mining was used extensively for ore extraction. In later years, Dome converted to open longhole stoping. These stopes were left open after mining, leaving a number of large open holes.

The testwork described was carried out concurrently with the placement of high density fill in a shrinkage/longhole stope located below the 600 level at Dome Mine. This stope was being filled as the final phase of a project to investigate and develop a backfill system based on the underground dewatering of full stream mill tailings. Extending from a scram drift below 7 level 180 ft. vertically to 6 level, the stope offered a number of access points for observation and testing of the in place fill, including the drawpoints on intermediate and upper drill subs (Figure 1).

In addition to the work done at Dome Mine, McGill University carried out all of the backfill characterization testing. McGill University Mining and Metallurgy department conducted this work on a subcontract from Dome. Subcontracts were also awarded to CIL Blasting Laboratories in Montreal to monitor blast vibrations in the backfill and to Conetec Inc. for cone penetration testing in the backfill using a seismic cone penetrometer.

2.0 BACKGROUND INFORMATION

2.1 Geology and Stope Geometry

An underground stope, called the 755-852 stope, at Dome Mine was selected to conduct the in-situ tests on paste backfill and study the impact of liquefaction on high density fills. The stoping area lies within massive volcanic units commonly referred to as the Dacite Flow, which has a width of approximately 70 m and a steep northerly dip of 60° - 70°. The flow extends from the greenstone north contact westerly across the entire property.

Local sections of this massive flow are host for a fairly regular series of tension formed quartz veins, which contain 3-5% combined pyrite, pyrrhotite and sphalerite and considerable visible gold. Individual ore zones within the Dacite favoured the more easterly portion of the flow and are extremely irregular in shape. In vertical extension they extend for two to three levels (100 - 150 m). Quartz veins within the Dacite generally have a northeasterly strike and a northwesterly dip of 50° - 60°. Stopes historically advance in cross-cut like fashion through the veins in the north and south extremities or the extension of these veins and often contain ore grades which are left in the walls of shrinkage stopes. The caving of the walls in later years has produced acceptable ore grade sloughage material.

2.2 Mining History

The original shrinkage stope began in 1931, with two small stopes, the 752 and the 755 stopes, which eventually joined up into a larger stoping unit called the 755 stope. More recent development in the area, subsequent to shrinkage mining, included a series of diamond drill programs which outlined ore above and in the walls of the 755 stope. A proposal to place a scram drift at the bottom of the ore, approximately 100 m below the 7 level and to drive a circumventing sill drift above the top of the shrinkage stope, was adopted. A drill drift was also established on the 7 level and from the three elevations, blasthole and drilling programs evolved which resulted in the recovery of 75,000 tons of ore.

2.3 Development of Paste Backfills

In order to stabilize the walls of the mined out stopes, Dome Mines decided to place dewatered backfill using a commercially available dewatering centrifuge called the Joy Tailspinner. A system was needed that could place backfill continuously and concurrently with mining operations and not require backfill fences or bulkheads. A continuous backfilling system, using full stream tailings had great potential, provided that the problems associated with the water transport were eliminated. This required dewatering of tailings prior to placement in the stope. The dewatering centrifuge was selected by Dome Mines to carry out this project. A separate CANMET study on the application of high density backfill using a Tailspinner is being completed for the Department of Supply and Services (DSS File # 06SQ.23440-5-9204).

3.0 STATE-OF-THE-ART REVIEW ON LIQUEFACTION EVALUATION PROCEDURES

A comprehensive review of the alternative liquefaction evaluation procedures available in the industry today was carried out by McGill University on behalf of Dome Mines Limited. The complete report, as prepared by McGill University, is presented in Appendix I. Key points, specifically relevant to paste fill technology and potential for its liquefaction, are discussed here.

Most of the research on liquefaction has centred around natural soils. Soil liquefaction has caused flow slides of natural slopes and earth dams, subsidence of foundations, tilting of buildings and bridge supports, cracking of pavements, collapse of waterfront structures, failures of tailings dams and disruption of other services. A classic example of liquefaction are failures of tailings dams. At least three tailings dam failures have been recorded as having occurred directly as a result of liquefaction. These are the El Cobre Dam in Chile (1965), the Albertam Whales (1966) Dam and the Mochikoshi Dam in Japan (1978). In these tailings dam incidents, 350 lives were lost and the loss of property approximated $150 million.

Paste backfills in underground mining use a high proportion of fine and ultra fine solids. They, therefore, are more susceptible to the potential for liquefaction than conventional hydraulic backfill which uses the coarse fraction of solids recovered from cyclone classification. To date, very little work on the potential of liquefaction of high density backfills has been carried out. The state-of-the-art review evaluates the different methods for assessment of soil liquefaction and geotechnical engineering and the potential application to paste fills in mining. Due to the lack of information rising from very limited research into liquefaction of backfill, McGill University attempted to accumulate and critically review liquefaction of soil materials and case studies which could then be used to assess liquefaction potential for paste fills.

Liquefaction is generally caused by loss of shear strength of a soil deposit due to static or dynamic loading. The soil mass flows when shear stresses acting on the deposit are lower than the shear strength. Any procedure for liquefaction evaluation, therefore, requires accurate information on soil properties. This information obviously may be obtained by field experiments or by measuring these in the laboratory. Alternatively, indirect and critical relationships can also be established. A desirable method for determining liquefaction potential is to compare the shear force that will be exerted on a soil deposit or a paste fill during static or dynamic loading and the shear strength that will be mobilized in the deposit during loading. A review of the soil mechanics literature indicates that two methods have been established to determine the liquefaction potential of a soil.

1. Methods based on the evaluation of stress conditions in the field, and laboratory determination of stress conditions which cause liquefaction.

2. Methods based on observation of the performance of soil deposits during or subsequent to severe seismic activity.

Within the methods based on the evaluation of stress conditions in the field and in the lab, the techniques may be further divided into two major groups.

a) Liquefaction evaluation procedure based on the concept of steady state of deformation and establishment of the critical void ratio.

b) Liquefaction procedure based on cyclical strength tests. This method is based on an evaluation of the cyclic stress or strain conditions likely to be developed in the field and a comparison of these stresses or strains with those observed to cause liquefaction of representative samples of the deposit in some appropriate cyclic laboratory test.

3.1 <u>Steady State of Deformation Tests</u>

This approach to determining the liquefaction potential of a soil has been applied successfully to various dams and structures. The liquefaction evaluation potential is basically an undrained stability failure analysis and consists of the following five steps (Figure 2):

1. Determination of the in-situ void ratio.
2. Determination of steady state void ratio or density as a function of effective stress using compacted specimens.
3. Determination of undrained steady state strength for undisturbed specimens.
4. Correction of measured undrained steady state strengths to in-situ void ratios.
5. Calculation of in-situ driving shear stress and subsequently the factor of safety.

The steady state of deformation for any mass of particles is that state in which the mass is continuously deforming at constant volume, constant normal effective stress, constant shear stress and constant velocity. The steady state of deformation is achieved only after all particle orientation has reached a statistically steady state condition and after all particle breakage, so that the shear stress needed to continue deformation under velocity of deformation remains constant. Examples of steady state of deformation are drained tests on placer sands and undrained tests on soft clays or loose saturated sand. The concept of a steady state of deformation depends on defining a critical void ratio. The critical void ratio is defined as the void ratio at which sand does not change in volume when subjected to shear. Deposits of sand, which have a void ratio larger than the critical void ratio, tend to decrease in volume when subjected to vibration by seismic events. If drainage is unable to occur, the buildup of pore pressure may cause soil liquefaction.

3.2 <u>Cyclic Strength Tests</u> (Figure 3)

Studies of soil failure due to cyclic loading such as earthquakes have shown that a similar behaviour can be observed for soil deposits. Cyclic loading is primarily due to upward propagation of earthquake shear waves in soil deposits. The earthquake loading has a random pattern that is essentially cyclic. Due to this applied cyclic stress, the structure of the soil tends to become compact with a resulting load transfer to the pore water pressure and a reduction in stress on the soil grains. In other words, pore water pressure can build up through a series of cyclic stresses induced by ground shaking in saturated sands of medium to high density. When pore water pressure becomes equal to the confining pressure and some deformations occur, these deposits are said to have liquefied. Three steps are proposed to evaluate the liquefaction potential of a soil based on cyclic tests:

a) An evaluation of the cyclic stresses induced at different levels in the deposit by the cyclic motion.

b) Laboratory investigations to determine the cyclic stress, which at a given confining pressure, will cause the soil to develop peak pore pressure equal to the effective confining pressure or undergo various degrees of cyclic strain.

c) Comparison of cyclic stress induced in the field with the stresses required to cause liquefaction.

Factors which influence cyclic mobility of liquefaction characteristics of soil include grain characteristics, relative density, method of soil formation under sustained load, previous strain history and the lateral earth pressure coefficient and overconsolidation.

3.3 <u>Conclusion</u>

During the past several decades, a considerable amount of research in geotechnical engineering has been directed towards developing better predictive methods and testing techniques for liquefaction potential of soils. In order to evaluate the liquefaction potential of a particular deposit or backfill, two methods have been suggested:

1. Liquefaction evaluation procedure based on the concept of steady state of deformation.

2. Liquefaction evaluation procedure based on the cyclic strength tests.

The liquefaction potential based on the concept of steady state of deformation defines liquefaction as a phenomenon which occurs when the shear resistance of a massive soil decreases when subjected to continuous, cyclic or dynamic loading at constant volume. The mass undergoes very large unidirectional shear strains. It appears to flow until the shear stresses are as low as or lower than the reduced shear resistance.

Cyclic mobility denotes a condition in which cyclic stress applications develop a peak cyclic pore pressure ratio of 100% and subsequent cyclic stress applications cause limited strains to develop, either because of the remaining resistance of the soil to deformation or because of dilation in the soil which causes the pore pressure to drop and the soil stabilizes under applied loads.

For high density paste fills it is recommended that the steady state of deformation method for determining liquefaction potential be used. This method consists of the following five steps:

1. Determination of the in-situ void ratio.

2. Determination of steady state void ratio or density as a function of effective stress using compacted specimens.

3. Determination of undrained steady state strength for undisturbed specimens.

4. Correction of measured undrained steady state strengths to in-situ void ratios.

5. Calculation of in-situ driving shear stress and subsequently the factor of safety.

It should generally be pointed out that very little testing of backfill materials with regards to liquefaction has been carried out. Obviously, comparison testing of soils and typical high density paste fills should be conducted to ensure the relevance of the studies carried out on sands, towards underground mine backfill.

4.0 MATERIAL CHARACTERIZATION

To fully understand the relationship between the physical properties of the tailings material and the effects of external forces causing liquefaction, a detailed analysis of the properties of the solids must be made. This information can be used as a fingerprint to characterize the solids and compare with other materials.

A series of index tests were conducted on the paste fill material to determine the various characteristics of the solid particles on subcontract by McGill University (Appendix II). These tests were designed on standard procedures used in the geotechnical industry. When the performance of consolidated material was being studied, tests were carried out on cemented samples prepared in the laboratory and samples received from the backfilling operations at the mine, either cast and cured underground at the dewatering site or retrieved from the filled stope. Table 1 lists the various index tests performed and the number of samples investigated in each case. Because of the large number of samples involved, rigorous procedures were followed in preparing the samples so that repeatability of results could be guaranteed. Wherever possible test apparatus was connected to a computer logging system for compilation of the data. A summary of the index test methods used and results of analysis of paste backfill from the Dome operation follows.

Particle Size Distribution

- standard ASTM test procedures for screen analyses and hydrometer analysis. Confirmation of results checked against sedigraph tests.
  - coefficient of curvature $C_c > 1$
  - coefficient of uniformity $C_u = 10$
  - see Figure 4 for gradation curve
  - classification-silty sand

Mineralogical Content

- X-ray Diffraction
  - Quartz S_1O_2 70%
  - Dolomite 18%
  - Muscovite 4%
  - Clinachor 3-4%
  - Pyrite trace

Particle Shape

- morphological analysis using a scan electron microscope
  - angular
  - moderate sphericity
  - consistent throughout size range

Plasticity

- Attenburg Limits were determined with cone penetrometer
  - liquid limit = 20%

Bulk Density

- dimensions and weights taken from uniaxial compression tests
  - range of dry unit weight
    1778 kg m^3 - 1201.35 kg m^3
    (111 pcf) (75 pcf)

Direct Shear

- samples prepared from consolidated reconstituted tailings
  - test procedure followed ASTM standards with strain rate of 0.5%/min.
  - friction angle = 30°
  - cohesion = 10 psi

See Table III and Figure 6 for typical test results.

Unconfined Compressive Strength

- samples were prepared from both reconstituted and field cast tailings and cement at various ratios. Procedures for casting and curing tailing samples were established by laboratory and field personnel to simulate standard ASTM procedures where possible.

See Table II and Figure 5 for typical test results.

A more detailed study of the effects of cement content, moisture content etc. on compressive strength was done as part of the liquefaction analysis testing.

4.1 Cone Penetration Tests

In order to determine the in-situ properties of backfill at Dome Mine, a series of cone penetration tests were carried out on subcontract with Conetec Investigations Ltd. These tests were carried out as a trial to evaluate the potential of using the cone penetrometer as a tool to provide the necessary in-situ geotechnical information for the design and verification of backfill properties and provide further determinations of the liquefaction potential of backfill materials.

A cone penetrometer is commonly used in geotechnical analyses on soils for foundation design purposes and soil stability analyses. This device also termed the electronic piezometer friction cone can measure soil parameters on a continuous basis as a unit penetrates the soil. Figure 7 illustrates a cross-sectional view of the cone device showing the different components which relay data back through the drill rods to a logging system for analysis. Information such as angle of friction, modulus of elasticity, water pressure and dynamic shear modulus can be determined from in-situ materials. Interpretation of the cone penetration data is then made with empirical correlation to obtain the geotechnical parameters required.

Conetec used a 10 tonne subtraction type cone for all of the testwork done at Dome. The cone was pushed into the fill using a 10 tonne ram assembly, specially designed for this underground testwork. The cone has a tip area of 10 cm^2 and a friction sleeve of 150 cm^2. Piezometer element thickness of 5 mm is located immediately behind the cone tip. The cone used during the test program was capable of recording the following parameters at 5 cm depth intervals:

a) Tip resistance

b) Sleeve friction

c) Dynamic pressure

d) Temperature

e) Cone inclination

In addition, the cone and accompanying data acquisition system were also capable of recording shear and compression wave arrival times. Cone tests are normally carried out on surface in a vertical orientation. The tests conducted underground at Dome not only represented the first exposure of the cone penetrometer to in-situ testing on high density backfill, but also was the first time Conetec had used the device in a horizontal or inclined orientation. Three cone penetration tests and one seismic cone penetration test were performed. The penetration tests were carried out at depths varying from 6.4 m to 20.5 m below the surface of the backfill.

A comprehensive computer software program and data logging system were used to interpret various in-situ parameters. Despite the harsh environment associated with underground operations, this electronic package functioned well. In the case of naturally occurring soils, tip resistance and friction ratio can be used to establish the following parameters:

- relative density
- friction angle
- elastic modulus
- shear modulus
- undrained shear strength

The absence of detailed correlations on cone penetration test data on manmade materials such as underground mine backfill, restricts the extent of the interpretation of the data. However, some conclusive correlations were possible. For full details of the test program and procedures refer to the Conetec Report in Appendix III.

Interpretation of the cone penetrometer data was done using correlations developed for naturally occurring soil deposits. The correlations used were not for backfill materials, however, based on considerable experience with the cone penetration testing analyses, in surface mine tailings deposits, correlations appear to work reasonably well. Cone soundings were conducted on the paste fill in the test stope at penetration rates of 2 cm/sec. The length of the holes varied from 6.4 m (21 feet), two 28.5 m (92.5 feet). Continuous logging of the cone bearing resistance (QC), sleeve friction (FS), average friction ratio (RF = FS/QC x 100) and the dynamic water pressure (U) was performed in each of the test holes. Figure 8 illustrates a graphical output of typical test results. The data is also presented from the data logger in tabular form with a typical soil behaviour classification based on friction ratio.

The significant variations in friction ratio indicates stratigraphic changes in paste fill material due mainly to changes in cement content. This was also evident from previous studies on drill hole core samples retrieved from the fill. It was found then that the bedding was caused by higher and lower cement addition rates during placement of the fill. The cone test confirmed the presence of bedding throughout the mass of the fill. Presence of strata in this material has a significant effect on stability as will be discussed.

Soil classifications, which are shown in Table 4, are based on the measured value of QC (cone resistance). From previous correlations it has been determined that the soil type and hence behaviour can be identified through cone bearing resistance values. Where QC is < 0.2 MPa, the material is said to be very soft clay. Values ranging up to or > 10 MPa describe the six different soil types through clay silt sand to hard stiff soil. High density backfill has been generally described as sand to silty sand.

Using data generated in laboratories with silica sand, a general correlation of the cone penetration test data suggests the following physical parameters exist in high density backfill.

Friction angle = 38-40°
In-situ relative density = >50%

As seen from Figure 8, a clear picture of the variations in pore pressure with depth can be obtained directly without further correlation. In general, the dense fill in all tests exhibited negative pore water pressure, an indication of a well drained soil.

Peaks of positive pressure were encountered which are considered to represent pockets of uncemented fill (note positive pore pressure with high friction angles indicates a weak material, positive pore pressure with low cone bearing resistance is an indication of a very stiff soil, in this case dense fill with high cement ratios).

The use of the cone penetrometer to monitor seismic wave propagation provides an opportunity to measure the dynamic properties of the fill. An analysis of the seismic data indicates that the shear modulus values in the backfill are relatively small. In addition, shear wave velocities of >400 m/sec.

were recorded. These may well have been influenced by the bedrock interface. Overall, the unusual shear modulus values are consistent with high dense materials.

4.2 Blast Vibration Monitoring

The Explosives Technical Centre of CIL Inc. undertook to measure vibrations in the paste backfill under varying blasting conditions through a series of critically designed field trials carried out in and around 755 stope. The main objectives of the program were as follows:

1. Design suitable transducer and recording systems for simultaneous recording of vibrations at several locations.
2. Specify blasthole diameters and their locations with respect to transducer locations.
3. Carry out vibration measurements for a variety of charge vs. distance configurations so as to characterize the rock mass and dense fill as fully as possible.
4. Analyze the vibration records and provide information on maximum vibration level, its duration and its spectral content as a function of charge size distance, delay sequence in multi-hole blasting and the nature of the transmitting media.

The CIL report is attached in Appendix IV. The general results of the CIL monitoring were somewhat inconclusive. It was evident that significantly greater attenuation of the waves takes place in the backfill material than in rock (Figure 9).

The results also indicate, as expected, that rock can support a much higher frequency than backfill. The dominant frequency for rock at Dome Mines was around 750 Hz, compared with 155 Hz within backfill. Based on the results, CIL established a relationship between scale distance and acceleration (Figure 10). This relationship provides a general reference on the maximum acceleration that can be expected in high density backfills as a result of different charge weights at a given distance. This relationship will be extremely helpful in designing future high density fills and in optimizing blast designs to ensure that the acceleration levels are within acceptable limits.

5.0 STATIC TESTS ON BACKFILL

Based on the theory of steady state of deformation as discussed in Section 3.1, a series of laboratory tests were conducted on samples of consolidated high density backfill. Different moisture contents and cement contents were used to represent field conditions. The tests were conducted according to ASTM procedures reflecting the consolidated condition of the sample. A total of 247 samples were tested as shown below:

Test Type	No. of Samples Tested
Uniaxial compression - unconfined	75
Direct shear box test	72
Triaxial compression	100
Total	247

The complete test results are contained in the McGill University reports Volumes 1 through 5. Highlights of the results are presented in this section.

5.1 Physical and Mechanical Properties of Tested Samples

The physical and mechanical properties of the tested samples are summarized in Table 2. The moisture content ranged from 20 - 24% with a void ratio of between 0.57 to 0.69. The modulus of deformation ranged from 11.5 to 32 MPa, depending on the cement:tailings ratio. The average compressive strength of the sample (unconfined) ranged from 189 kPa to 348 kPa.

5.2 Effects of Variation of Cement Contents, Moisture Contents and Curing Periods on Mechanical Properties

A series of unconfined compression tests conducted to evaluate variations of cement and moisture contents indicate that high strength and modulus values are generally obtained from higher cement and moisture contents. As expected, strength increases with curing time (Figure 5).

5.3 Direct Shear Box Test

Summary of the direct shear box tests is shown on Table 3 and plotted on Figure 6.

5.4 Triaxial Compression Tests

A series of consolidated undrained triaxial tests with pore water pressure measurement were conducted at five different confining pressures. All samples were initially saturated with back pressure followed with a consolidation process prior to shearing. Figure 11 represents a typical stress strain curve and changes of pore pressure against strain. This sample was prepared in the laboratory according to the described sample preparation procedure at a 24% moisture content and a 1:30 cement: underflow ratio. The sample was saturated with back pressure of 138 kPa and consolidated to an effective consolidation pressure of 69 kPa and a void ratio of 0.71. All confined samples demonstrated a distinct non-linear behaviour with the majority of samples failing in a bulging manner with distinct fracture patterns of failure planes.

5.5 Influence of Cement Content on Paste Backfill Behaviour

The influence of cement content on stress-strain strength curves at low confining pressures is shown on Figure 12. It may be noted that the peak strength increases with increased cementation. This type of behaviour was consistent throughout all of the confining pressures applied to paste backfill samples. Failure envelopes for paste backfill determined from triaxial test results are presented in Figure 13.

5.6 Summary

Almost 250 static laboratory tests were carried out on paste backfill samples. These comprise unconfined compression tests, direct shear box tests and triaxial compression tests. Paste backfill samples with cement contents of 1:30, 1:40, 1:50 and moisture contents of 20 and 24% were used.

Under unconfined compression, higher strength and modulus values were generally obtained for higher cement and low moisture contents. Peak strength values were determined for samples tested with the direct shear box. Paste backfill samples exhibited fairly similar angles of internal friction with different cohesion intercepts with changes in cement content.

The results of the consolidated triaxial tests show that paste backfill is brittle at low confining pressures and becomes increasingly more ductile with increasing confining pressure.

Significant differences on shear strength parameters were noticed from the triaxial and shear box test. Due to many shortcomings of the direct shear box test, shear strength parameters of paste backfill obtained under triaxial conditions should be studied.

6.0 LIQUEFACTION EVALUATION ANALYSES

This chapter is intended to provide insights on the liquefaction resistance of paste backfill. Several mechanisms have been identified as being related to the possible failure of paste backfill during and after seismic events. As was stated previously, a logical first step in the analysis of liquefaction evaluation of paste backfill is to assess liquefaction susceptibility based on the concept of steady state of deformation. Results of a comprehensive study on liquefaction evaluation of paste backfill based on the concept of steady state of deformation are presented. A series of consolidated undrained triaxial tests were carried out on paste backfill samples. Influence of cement content on liquefaction potential of paste backfill was evaluated.

The cyclic mobility of paste backfill was evaluated based on the concept of zero effective stress. A number of laboratory tests were conducted on paste backfill with different cement contents. In addition, liquefaction susceptibility of paste backfill was evaluated in-situ. Results of piezometer friction cone tests for liquefaction purposes were discussed in the previous sections of this report.

In order to evaluate the susceptibility of backfill to liquefaction as a result of seismic events such as a blast, rockburst or earthquake, the driving shear stresses within paste backfill should be compared with the undrained steady state strength of paste fill. The undrained steady state shear strength is the minimum strength that paste backfill can have at in-situ void ratio. Typical results of tests on reconstituted samples for undrained consolidated triaxial tests are presented in Figure 14. Dilative behaviour was observed for all reconstituted paste fill samples prepared for the consolidated undrained triaxial tests with reconstituted samples.

A series of undisturbed samples were tested using the same method. These samples were recovered by diamond drilling from the 755-852 stope. The samples had different void ratios and were saturated and consolidated to various effective consolidated pressures. The results of these tests indicate that liquefaction resistance decreases with decreasing cement content.

The influence of void ratio on liquefaction susceptibility of paste backfill samples was also investigated. All samples exhibited dilative behaviour with decreasing excess pore pressure and increasing shear stress. The results indicated that samples with lower void ratios show higher pore pressure reduction as is expected due to the tendency of samples with high densities to develop stronger volumetric expansion during shear.

A series of undrained triaxial tests were also conducted on saturated samples at 76% pulp density and various effective consolidated pressures. It was concluded that due to the higher possibility of volumetric expansion, dilation is significant at lower effective confining pressures, however, an increase in deviatoric stress under undrained conditions required to initiate liquefaction can be seen as the consolidation pressure is increased. The results of these tests thus indicate that liquefaction resistance increases with increasing confining pressures. This is very significant as paste backfill observed in the 755-852 stope has shown low stresses.

The effect of various cement contents on liquefaction has already been discussed. Results show that under low confining pressures and low cement contents, paste backfill could be unstable when placed initially in the stope. Once the fill cures, however, this condition is likely to disappear.

Piezometer Friction Cone Tests

Relative information with regards to liquefaction evaluation using the piezometer friction cone test is discussed below. A dynamic pore pressure of generally less than zero was observed for all the test points examined with the cone on the paste backfill in 755-852 stope. This indicates a dilatant behaviour in the soils as detected by the pore pressure sensing element behind the tip. As previously discussed, liquefaction is a phenomenon wherein the shear resistance of a mass of saturated, lose material decreases when subjected to monotonic cyclic or dy1namic loading at constant volume. Dilative soils are not susceptible to liquefaction because their undrained shear strength is greater than their drained strength. Careful examination of recorded data indicates that paste fill appears to be unsaturated and, therefore, liquefaction potential under present hydrogeological regime is unlikely. In addition, paste fill shows pronounced dilatant behaviour and, therefore, is not susceptible to liquefaction based on the concept steady state of deformation.

LIST OF TABLES

TABLE I
LABORATORY TESTING PROGRAM
Physical Characteristics Tests

Test	Number of Tests
Compositional Analysis	9
Morphological Analysis	9
Particle Size Analysis	9
Atterberg Analysis	9

STATIC LABORATORY TESTING PROGRAM

Test Type	Number of Tests
Unconfined Compression	75
Direct Shear Box (4 normal loads)	72
Triaxial Compression (5 confining pressures)	100

TABLE II

AVERAGE PHYSICAL AND MECHANICAL PROPERTIES OF TESTED SAMPLES

Moisture Content		20%			24%	
Cement:Underflow	1:30	1:40	1:50	1:30	1:40	1:50
Void Ratio	0.62	0.58	0.57	0.68	0.67	0.69
Modulus of Deformation (kPa)	27.3	16.9	11.5	32.0	29.8	14.3
Sample Strength (kPa)	348	268	189	462	368	254

TABLE III

AVERAGE SHEAR STRENGTH PARAMETERS FROM DIRECT SHEAR BOX TESTS

Moisture Content		20%			24%	
Cement:Underflow	1:30	1:40	1:50	1:30	1:40	1:50
Angle of Friction	18	19	23	14	10	7
Cohesion (kPa)	94.7	78	39.5	74.2	65	49.4

PLACER DOME INC.

Contractor ConeTec
On Site Loc: DOME 600-1
Comments : 88-106 xxxxxx
Tot. Unit Wt. (avg) : 19 kPa

CPT Date : 8 MARCH 1988
Cone Used : HT 19? STD 5 MM
Water table (meters) : 28.5

DEPTH (meters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	SPT-N1	CSR
0.50	1.64	125.98	1.68	1.33	4.75	sand to silty sand	>90	>48	31	>50	>0.5
1.00	3.28	178.36	2.28	1.28	14.24	sand to silty sand	>90	>48	45	>50	>0.5
1.50	4.92	141.37	2.30	1.63	23.74	sand to silty sand	>90	46-48	35	>50	>0.5
2.00	6.56	109.34	1.59	1.45	33.24	sand to silty sand	80-90	44-46	27	47	>0.5
2.50	8.20	45.75	0.39	0.86	42.73	silty sand to sandy silt	50-60	38-40	15	23	.34x
3.00	9.84	52.83	0.45	0.86	52.23	sand to silty sand	50-60	38-40	13	18	.19
3.50	11.48	72.18	0.76	1.05	61.73	sand to silty sand	60-70	38-40	18	23	.24
4.00	13.12	94.20	0.92	0.97	71.22	sand to silty sand	70-80	38-40	24	28	.30
4.50	14.76	72.77	0.77	1.06	80.72	sand to silty sand	60-70	36-38	18	20	.21
5.00	16.40	54.13	0.50	1.00	90.21	silty sand to sandy silt	50-60	34-36	18	19	.28x
5.50	18.04	45.30	0.47	1.04	99.71	silty sand to sandy silt	40-50	32-34	15	15	.24x
6.00	19.69	56.79	0.65	1.15	109.21	silty sand to sandy silt	40-50	34-36	19	18	.27x
6.50	21.33	105.71	1.37	1.30	118.70	sand to silty sand	60-70	36-38	26	24	.25
7.00	22.97	106.05	1.33	1.25	128.20	sand to silty sand	60-70	36-38	27	23	.25
7.50	24.61	52.90	0.49	0.93	137.70	silty sand to sandy silt	40-50	30-32	18	15	.23x
8.00	26.25	88.62	1.08	1.22	147.19	sand to silty sand	50-60	34-36	22	18	.19
8.50	27.89	99.65	1.39	1.40	156.69	sand to silty sand	60-70	34-36	25	20	.21
9.00	29.53	93.30	1.09	1.16	166.18	sand to silty sand	50-60	34-36	23	18	.19
9.50	31.17	23.06	0.32	1.37	175.68	sandy silt to clayey silt	UNDFND	UNDFD	9	7	UNDF
10.00	32.81	90.85	1.27	1.40	185.18	sand to silty sand	50-60	32-34	23	17	.17
10.50	34.45	154.41	2.44	1.58	194.67	sand to silty sand	70-80	36-38	39	27	.30
11.00	36.09	63.19	0.90	1.42	204.17	silty sand to sandy silt	40-50	30-32	21	15	.23x
11.50	37.73	126.89	1.63	1.28	213.67	sand to silty sand	60-70	34-36	32	21	.22
12.00	39.37	154.52	1.89	1.22	223.16	sand to silty sand	60-70	34-36	39	26	.27
12.50	41.01	72.50	0.78	1.08	232.66	sand to silty sand	40-50	30-32	18	12	.12
13.00	42.65	62.15	0.87	1.40	242.16	silty sand to sandy silt	40-50	<30	21	13	.22x
13.50	44.29	52.66	0.65	1.23	251.65	silty sand to sandy silt	<40	<30	18	11	.19x
14.00	45.93	93.09	1.27	1.35	261.15	sand to silty sand	50-60	30-32	23	14	.15
14.50	47.57	182.09	2.55	1.40	270.64	sand to silty sand	70-80	34-36	46	27	.30
15.00	49.21	71.32	0.76	1.06	280.14	sand to silty sand	40-50	<30	18	11	.11
15.50	50.85	104.37	1.47	1.41	289.64	sand to silty sand	50-60	30-32	26	15	.16
16.00	52.49	159.88	2.53	1.58	299.13	sand to silty sand	60-70	34-36	40	23	.24
16.50	54.13	164.91	1.94	1.17	308.63	sand to silty sand	60-70	34-36	41	23	.24
17.00	55.77	197.84	3.05	1.54	318.13	sand to silty sand	70-80	34-36	49	27	.30
17.50	57.41	71.97	0.99	1.37	327.62	silty sand to sandy silt	40-50	<30	24	13	.22x
18.00	59.06	180.84	2.66	1.47	337.12	sand to silty sand	60-70	34-36	45	24	.26
18.50	60.70	106.52	1.59	1.49	346.61	sand to silty sand	50-60	30-32	27	14	.15
19.00	62.34	113.64	1.58	1.32	356.11	sand to silty sand	50-60	30-32	30	16	.16

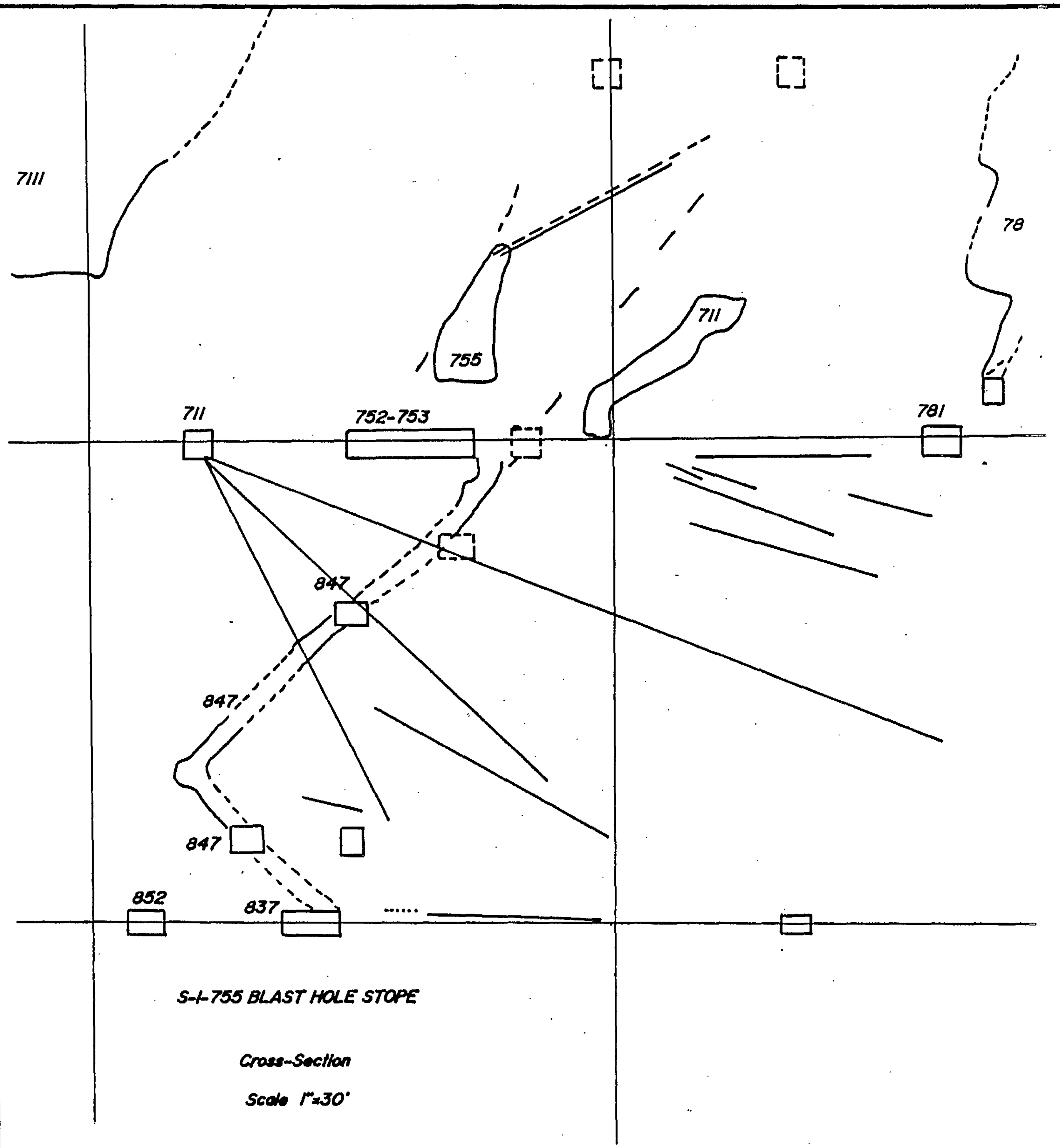
Dr - All sands (Jamiolkowski et al. 1985) PHI - Durgunoglu and Mitchell 1975 CSR: Seed et al. 1983 - M=7.5

x - Seed's correction of 7.5 blows/foot has been applied to N1

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

TABLE IV

LIST OF FIGURES

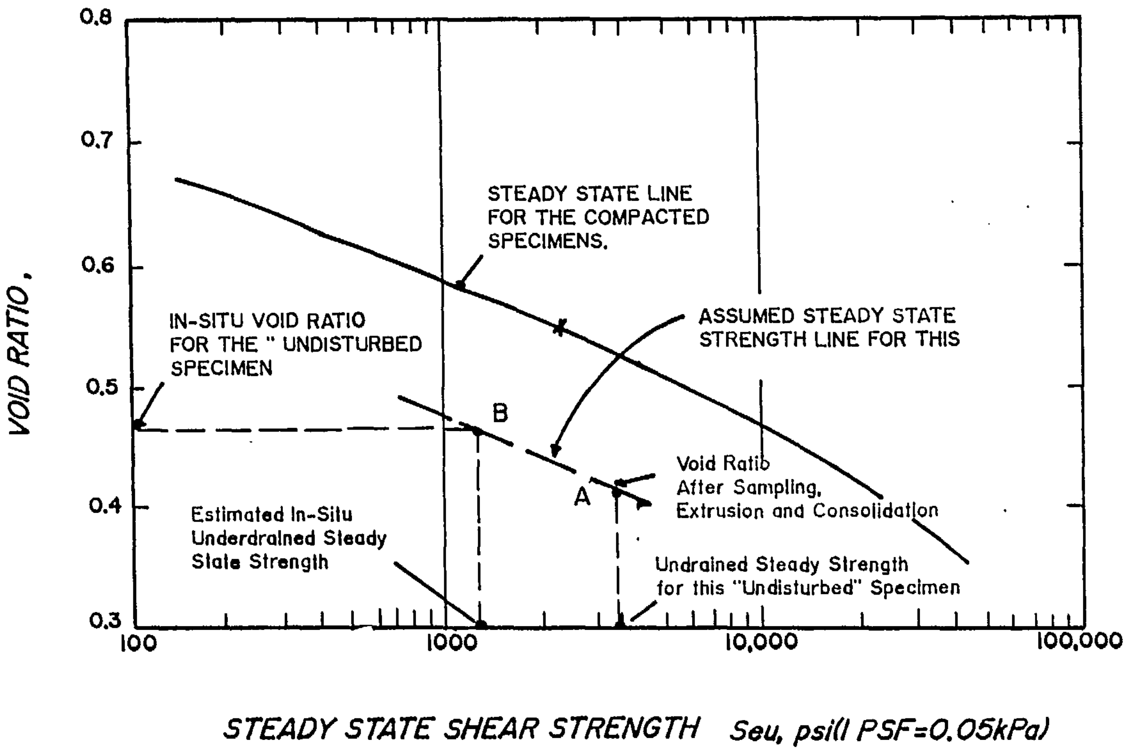


PASTE FILL LIQUEFACTION STUDTY

DOME MINES

CROSS-SECTION OF
755 STOPE

FIGURE 1

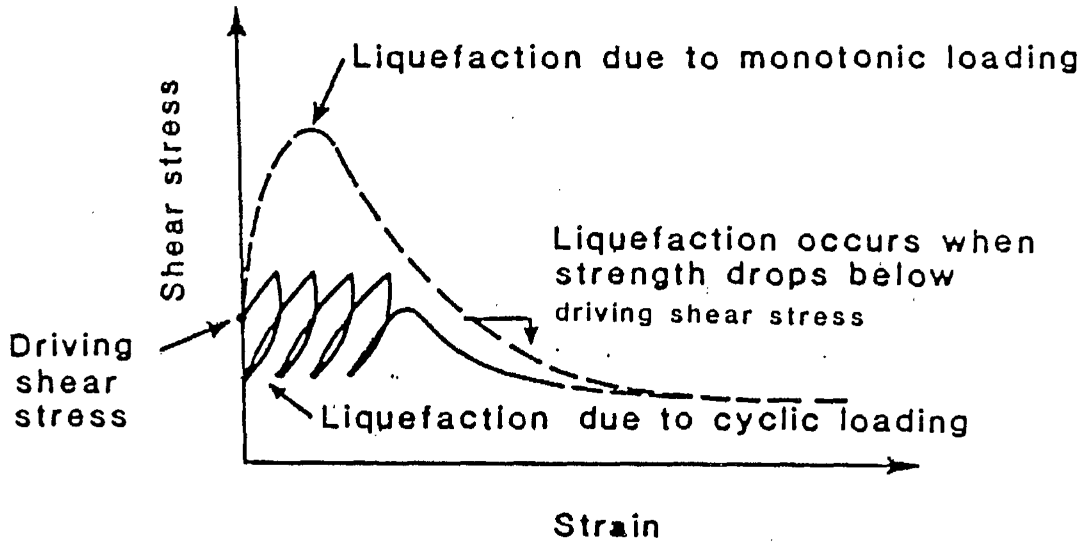


PASTE FILL LIQUEFACTION STUDTY

DOME MINES

Correction of Measured Undrained Steady State Strength for Difference Between I. Situ Void RatioO and during the test. (Polous et al., 1985)

FIGURE 2

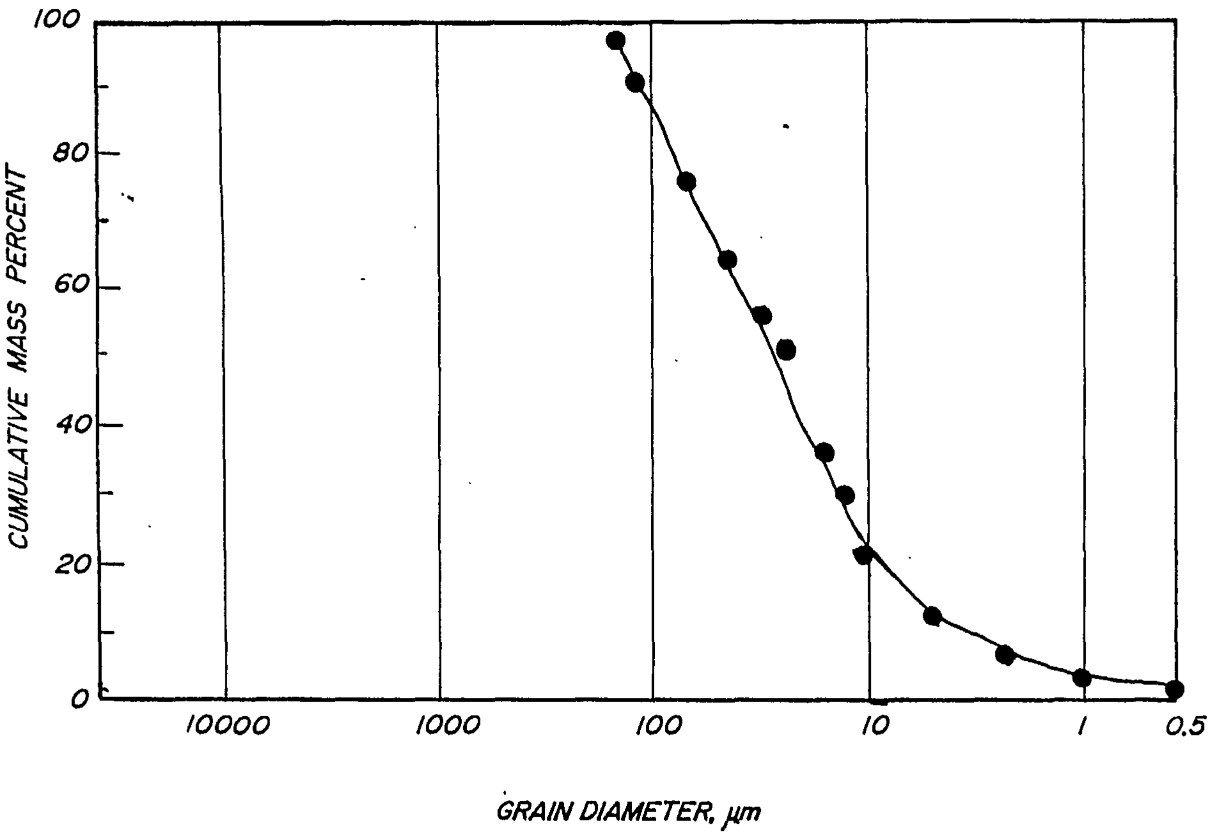


PASTE FILL LIQUEFACTION STUDY

DOME MINES

Liquefaction due to Monotonic or Cyclic loading (Schematic) . (Poulos et al., 1985)

FIGURE 3

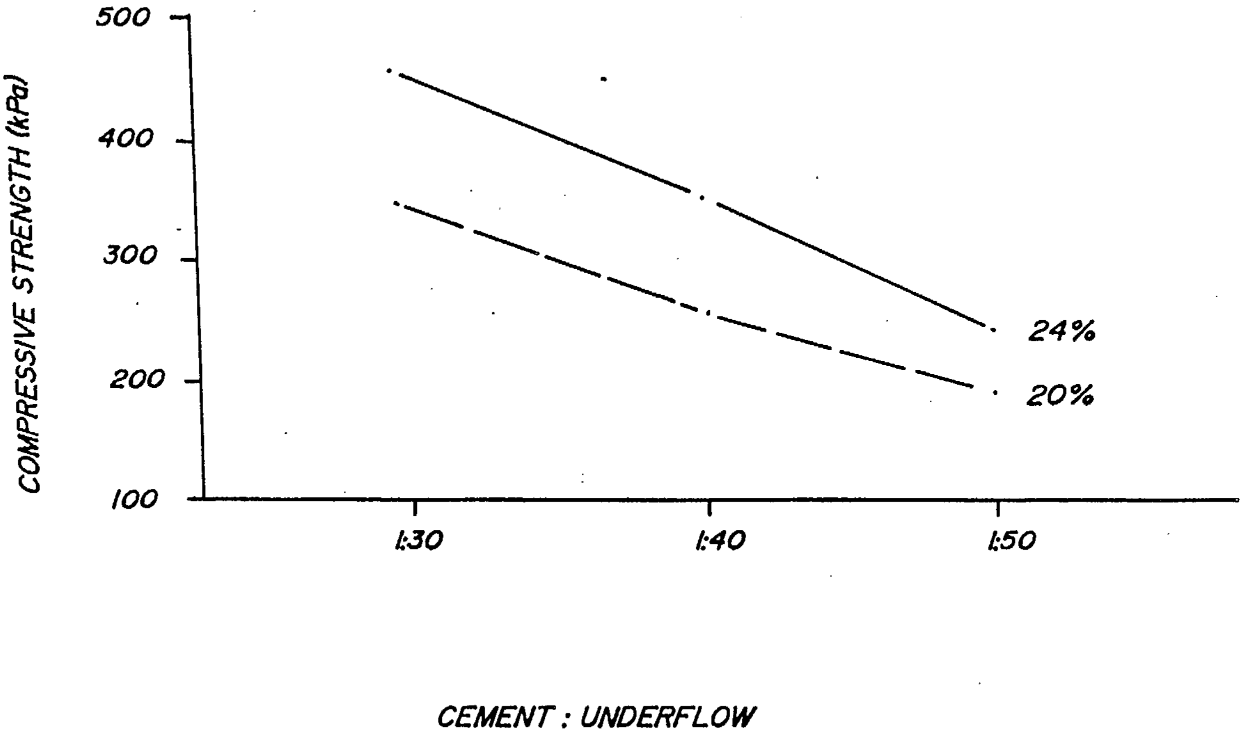


PASTE FILL LIQUEFACTON STUDY

DOME MINES

Grain Size Distribution For Underflow Material

FIGURE 4

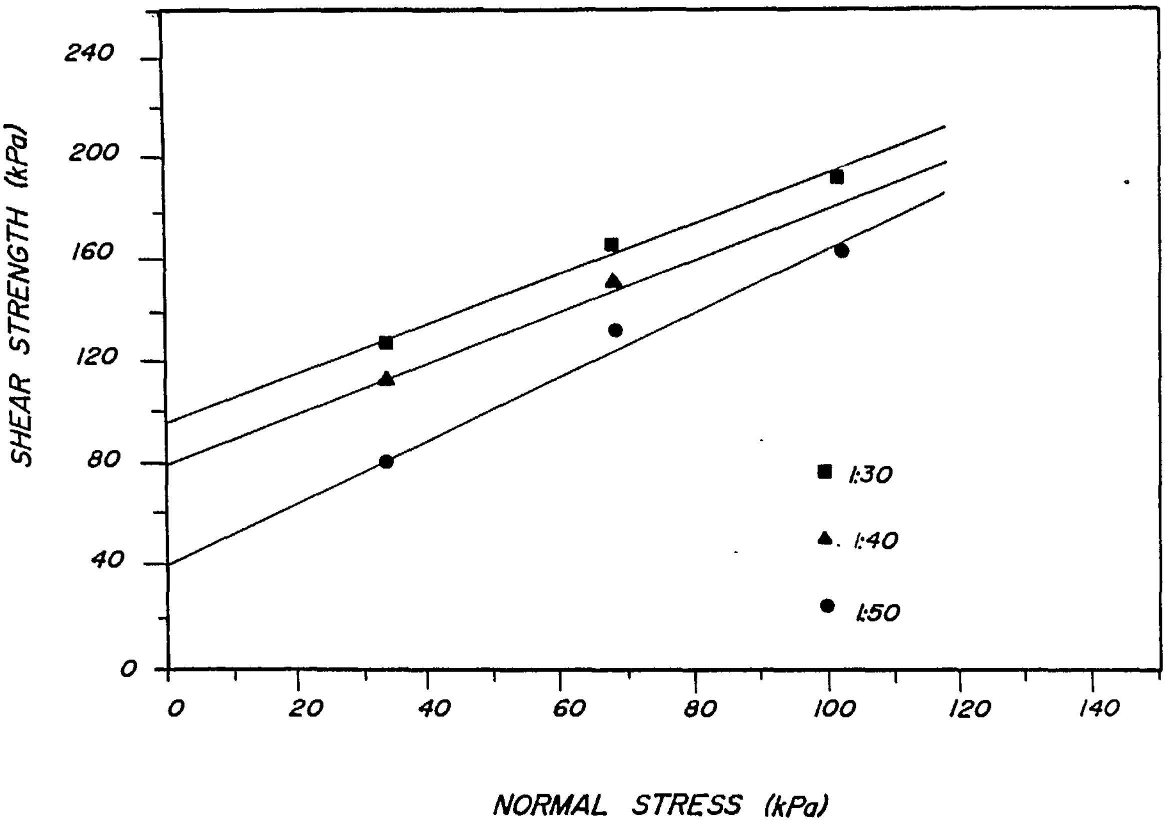


PASTE FILL LIQUEFACTION STUDY

DOME MINES

Variation of Strength with Cement Underflow Ratio's at Different Moisture Contents.

FIGURE 5

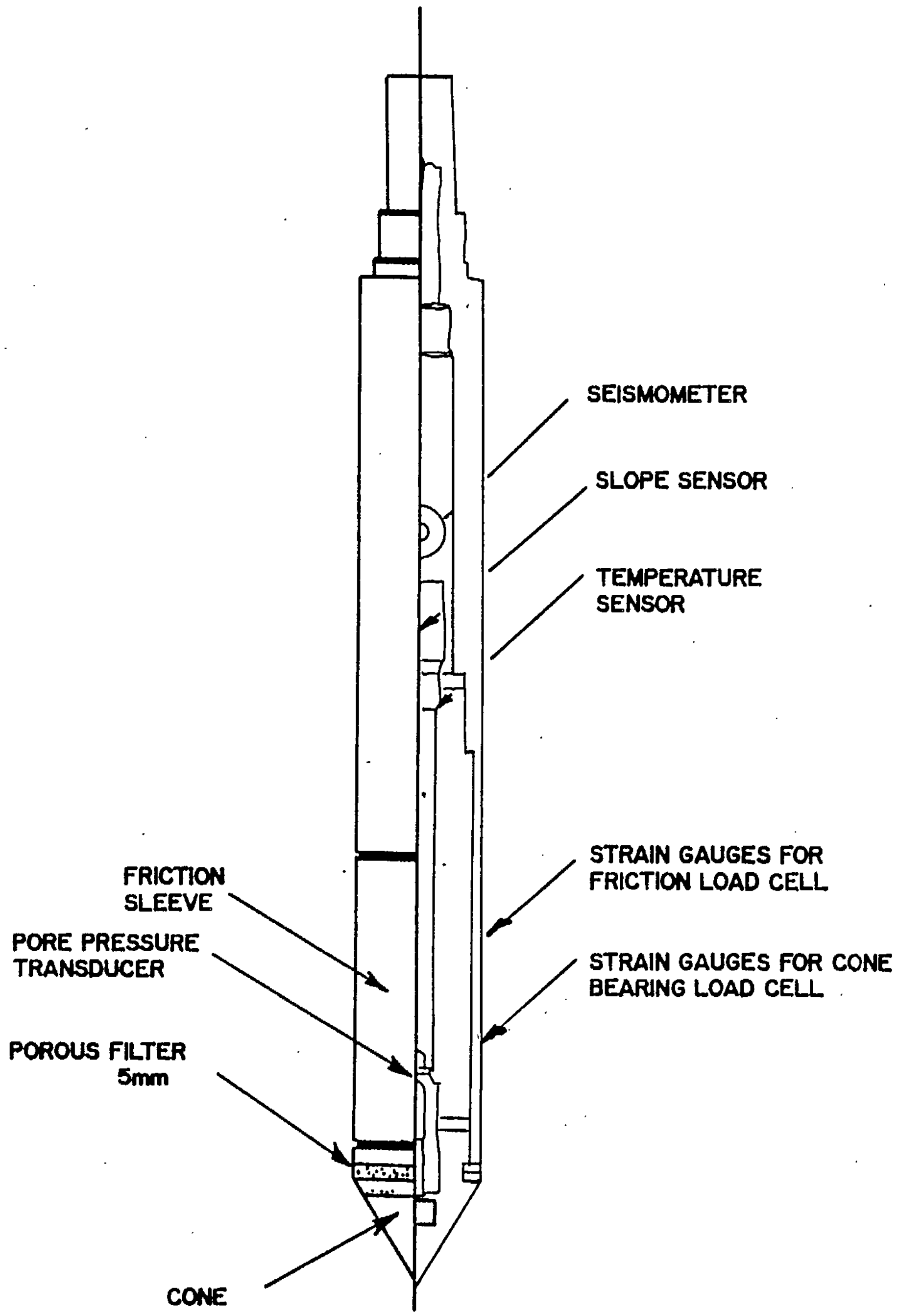


PASTE FILL LIQUEFACTION STUDY

DOME MINES

Peak Strength Values For Paste Backfill Samples at 20% Water Content.

FIGURE 6

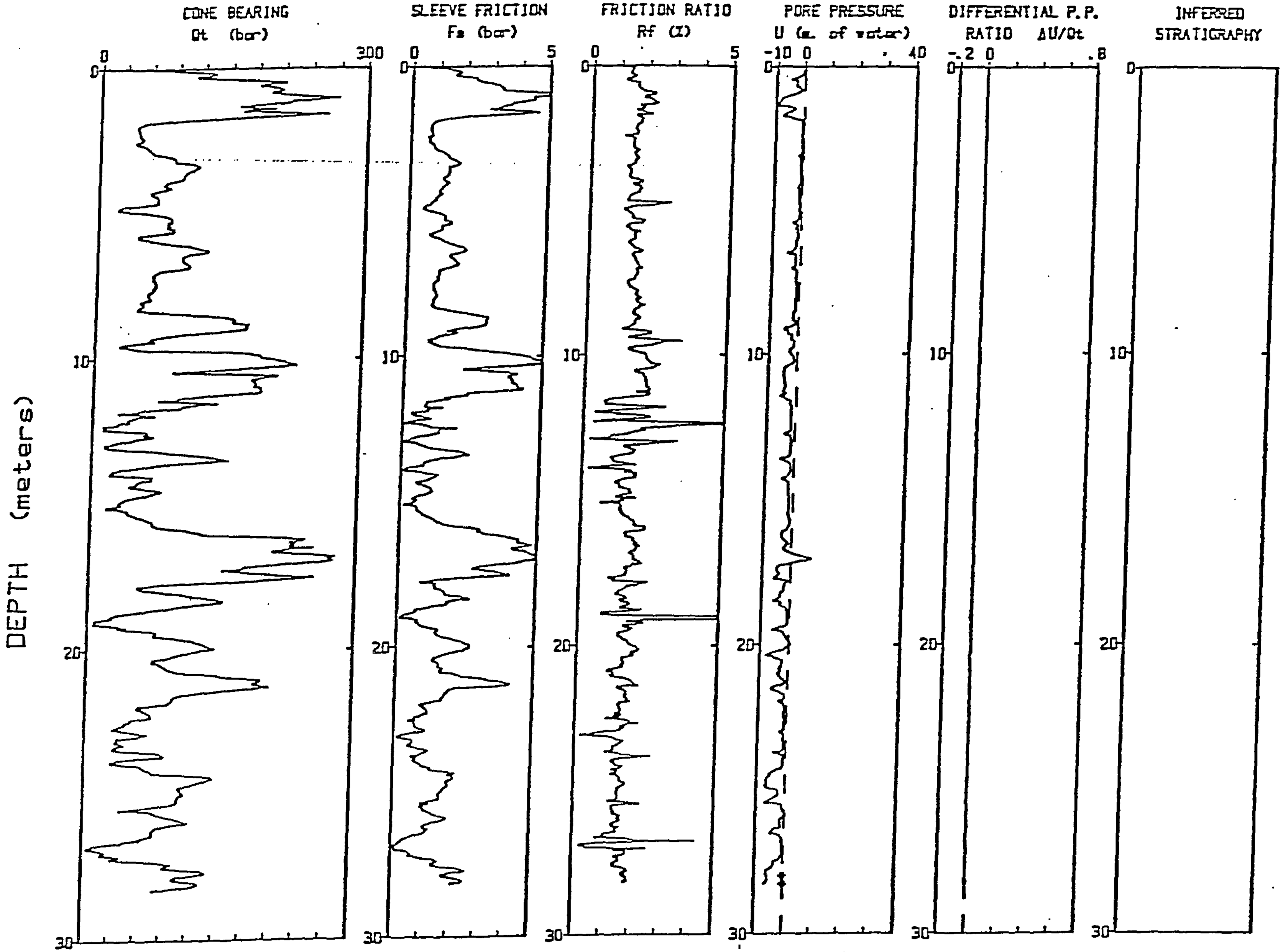


PASTE FILL LIQUEFACTION STUDY
DOME MINES
SEISMIC CONE PENETROMETER
FIGURE 7

PLACER DOME INC.

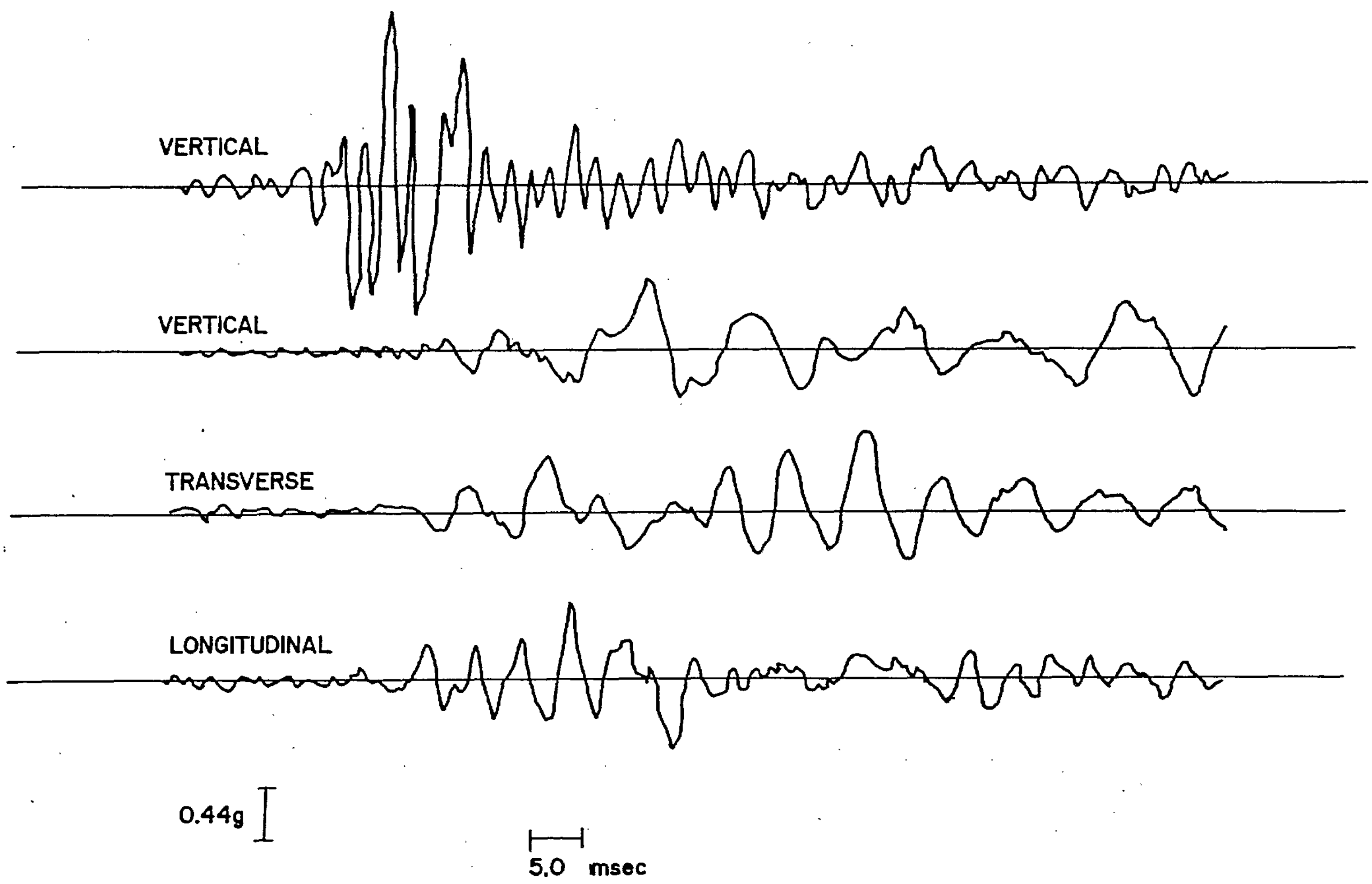
CONTRACTOR: ConeTec DATE: 9 MARCH 1988 Page No: 1 / 1

SITE: DOME 600-2 CONE: HT 192 STD 5 MM JOB # 88-106



Depth Increment : .05 m Max Depth : 28.3 m

FIGURE 8



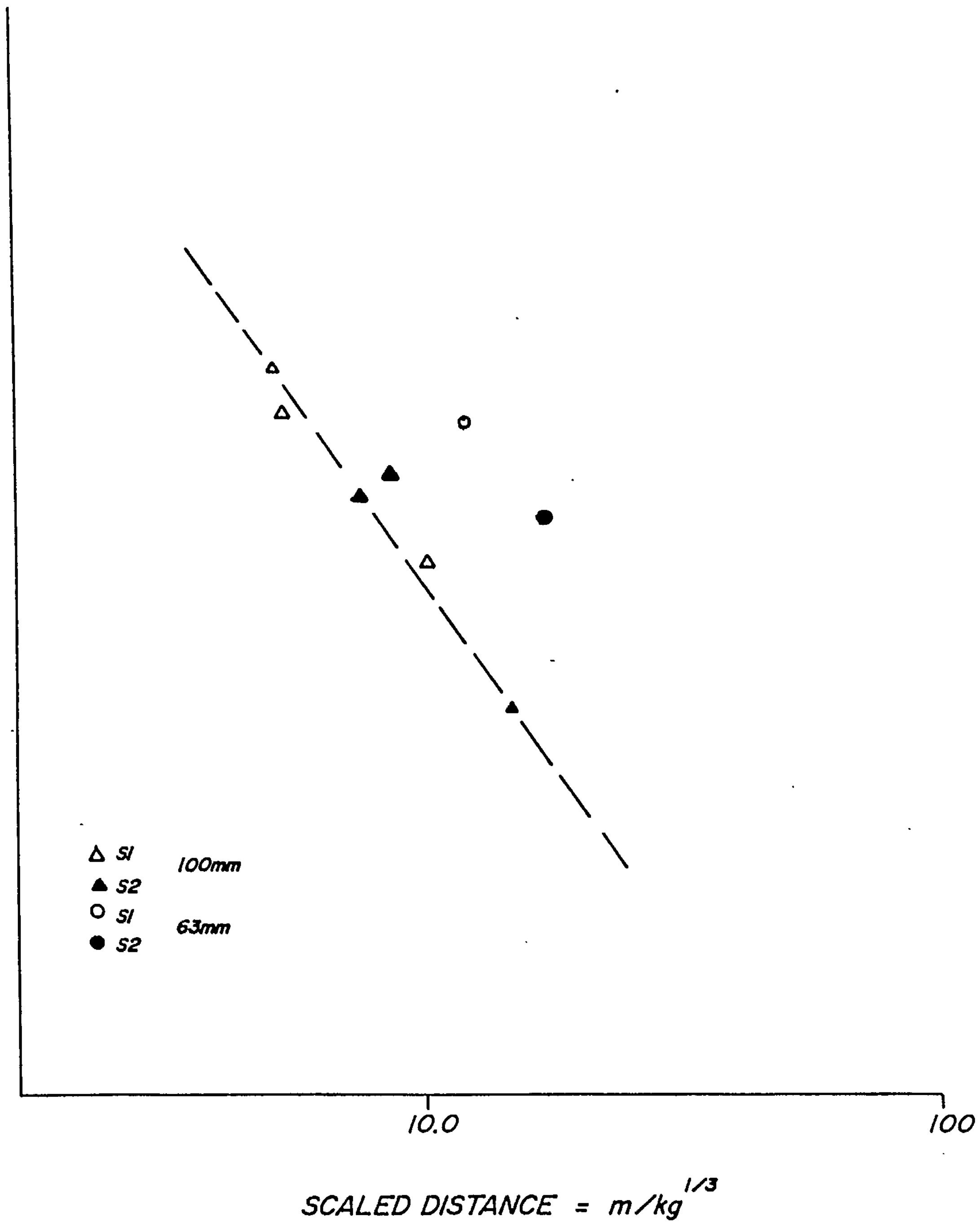
PASTE FILL LIQUEFACTION STUDY

DOME MINES

Comparison of Acceleration Signal Between Stations S1 & S2

Blast No.1

FIGURE 9

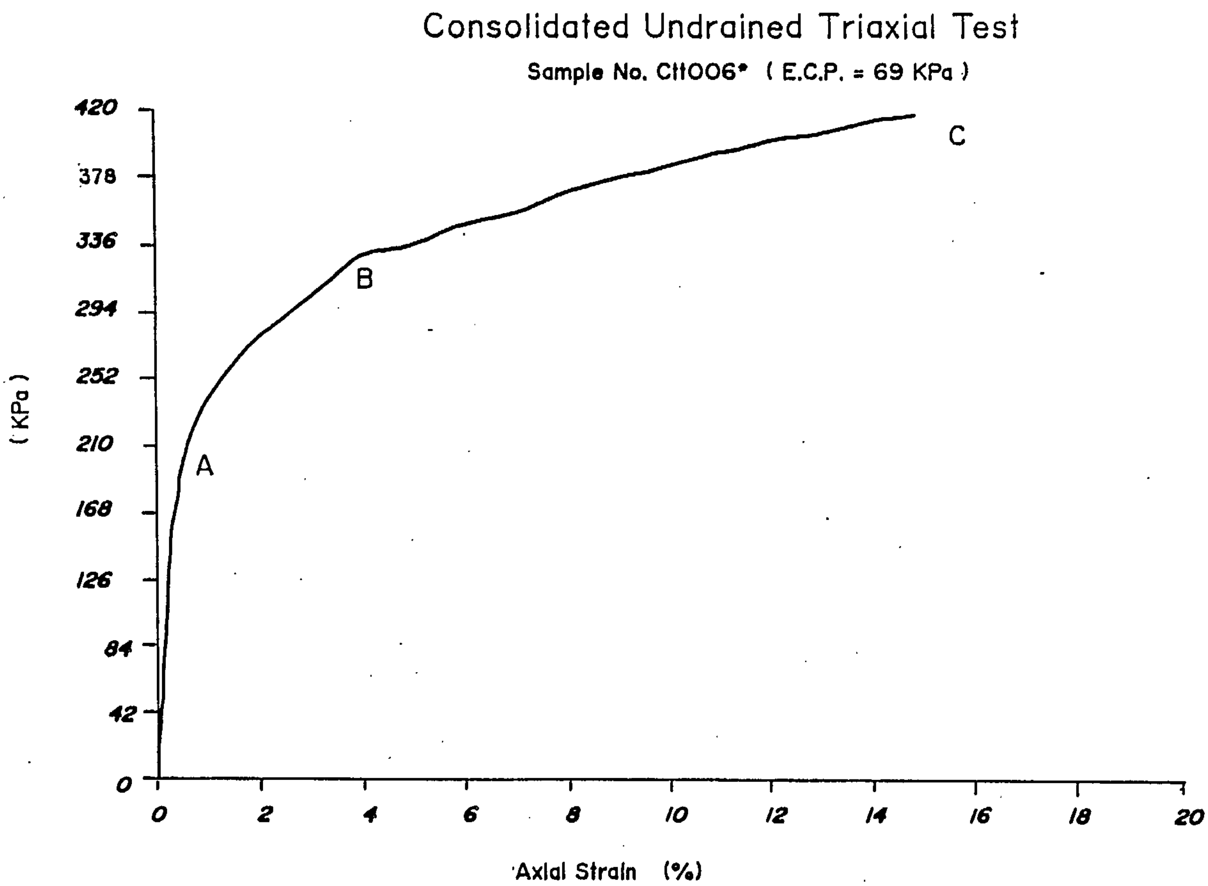


PASTE FILL LIQUEFACTION STUDY

DOME MINES

DOME MINES TRIAL #1
VIBRATION IN DENSE FILL

FIGURE 10

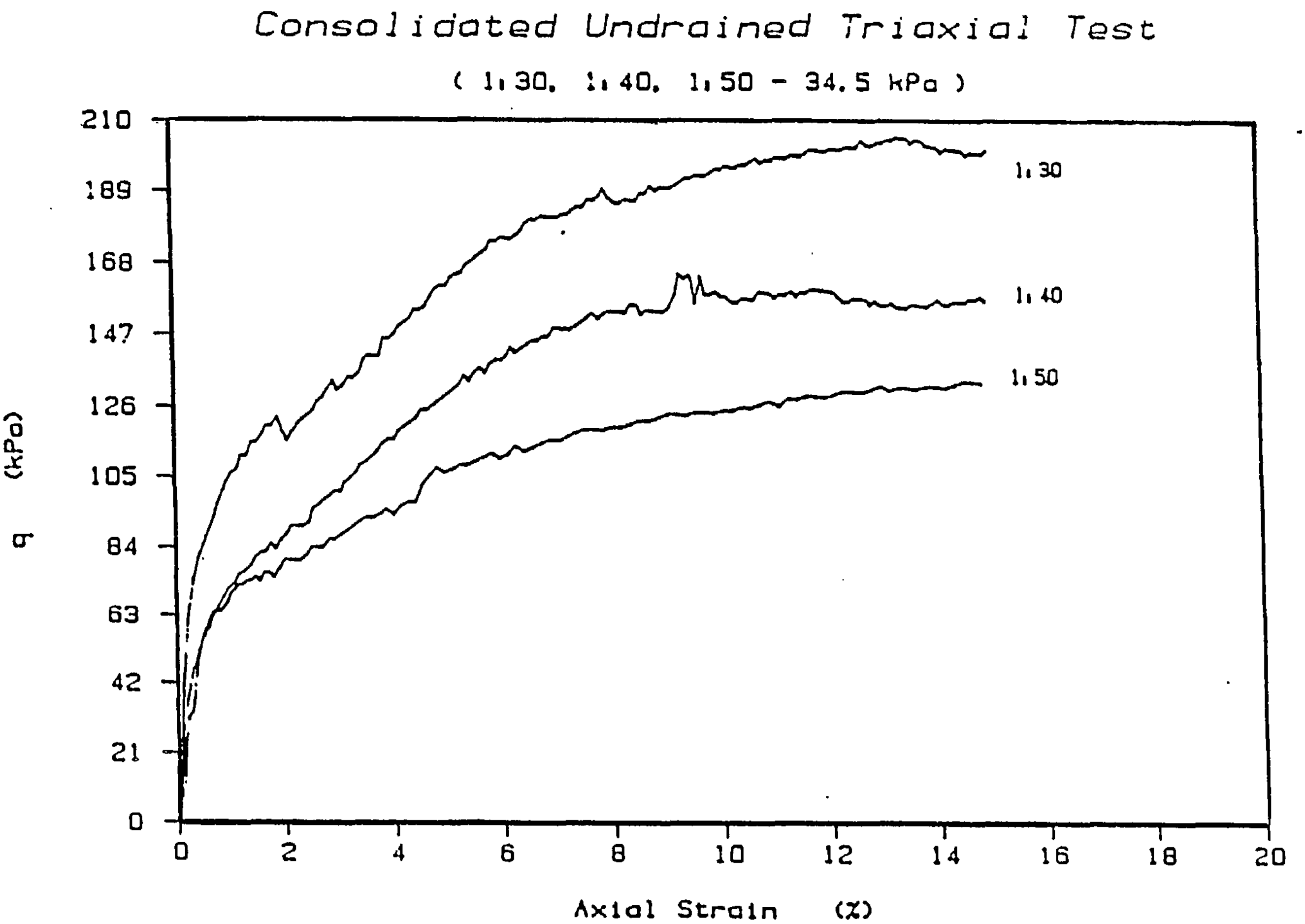


PASTE FILL LIQUEFACTION STUDY

DOME MINES

Typical Stress Strain Curve for Paste Backfill

FIGURE 11



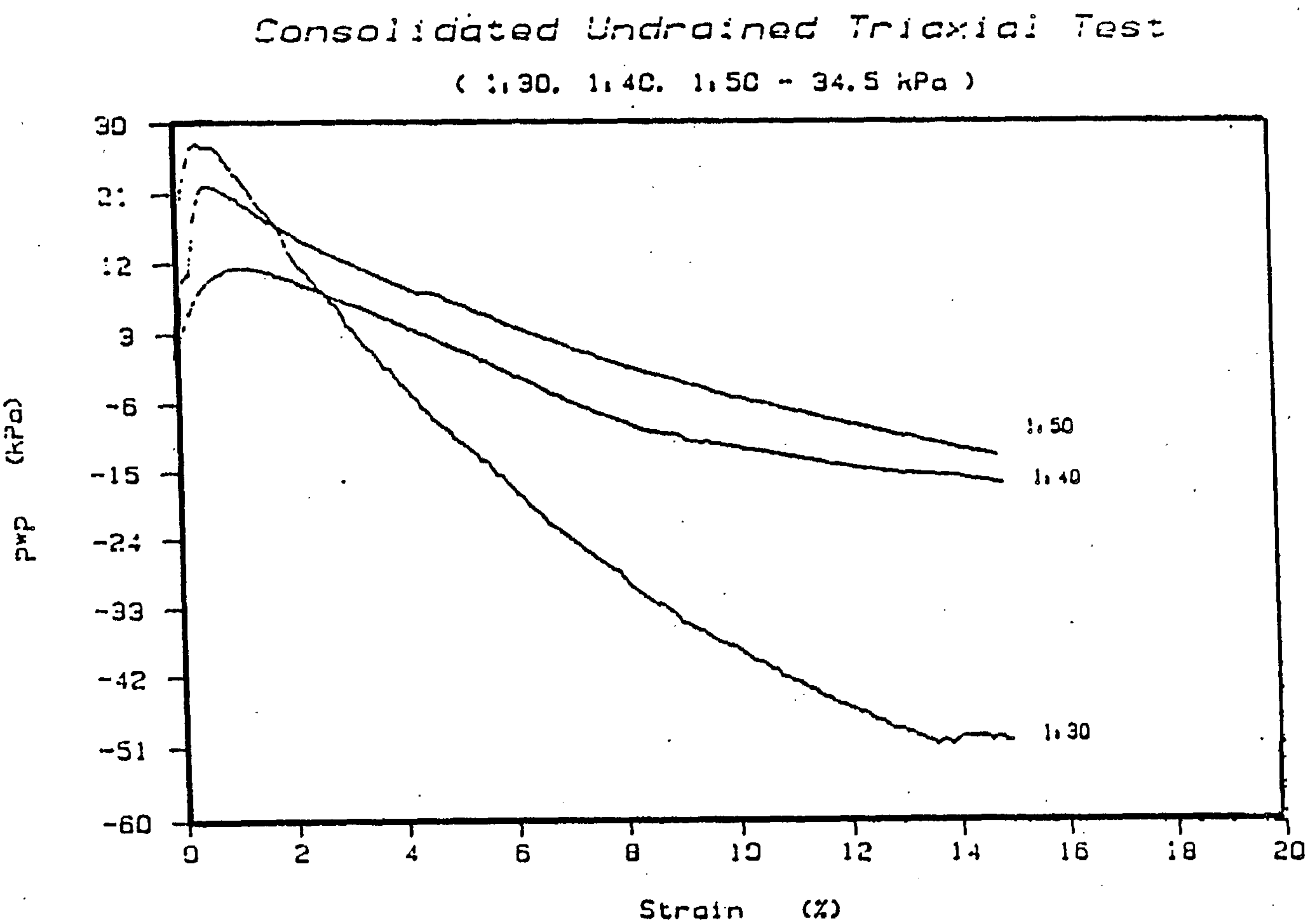
(a): Stress - Strain

PASTE FILL LIQUEFACTION STUDY

DOME MINES

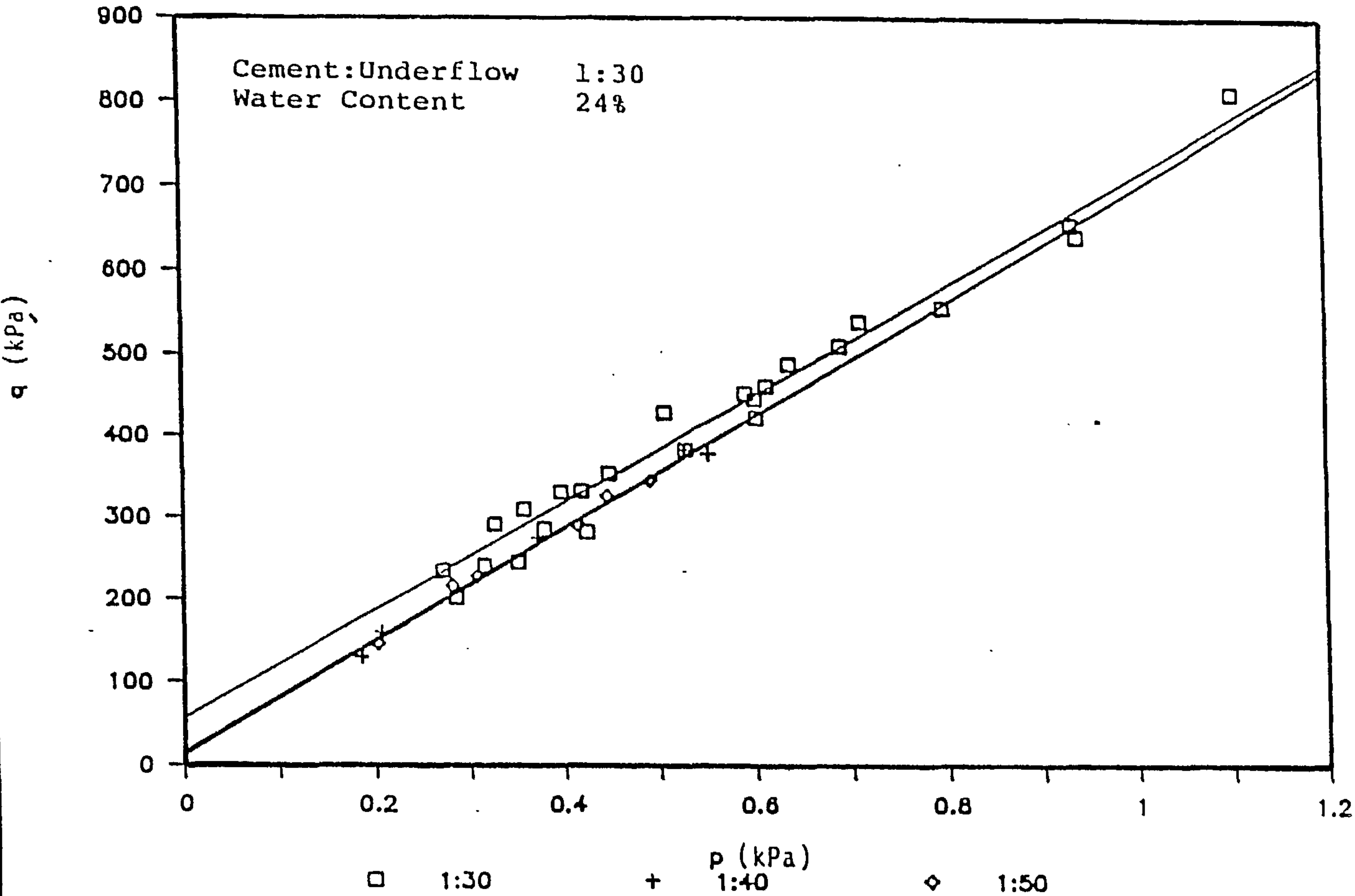
Typical Stress Strain Curves at 34.5 KPa Confining Pressure

FIGURE 12



(d): Pore Water Pressure

PASTE FILL LIQUEFACTION STUDY
DOME MINES
Typical Pore Water Pressure Curves at 34.5 KPa Confining Pressure
FIGURE 12a

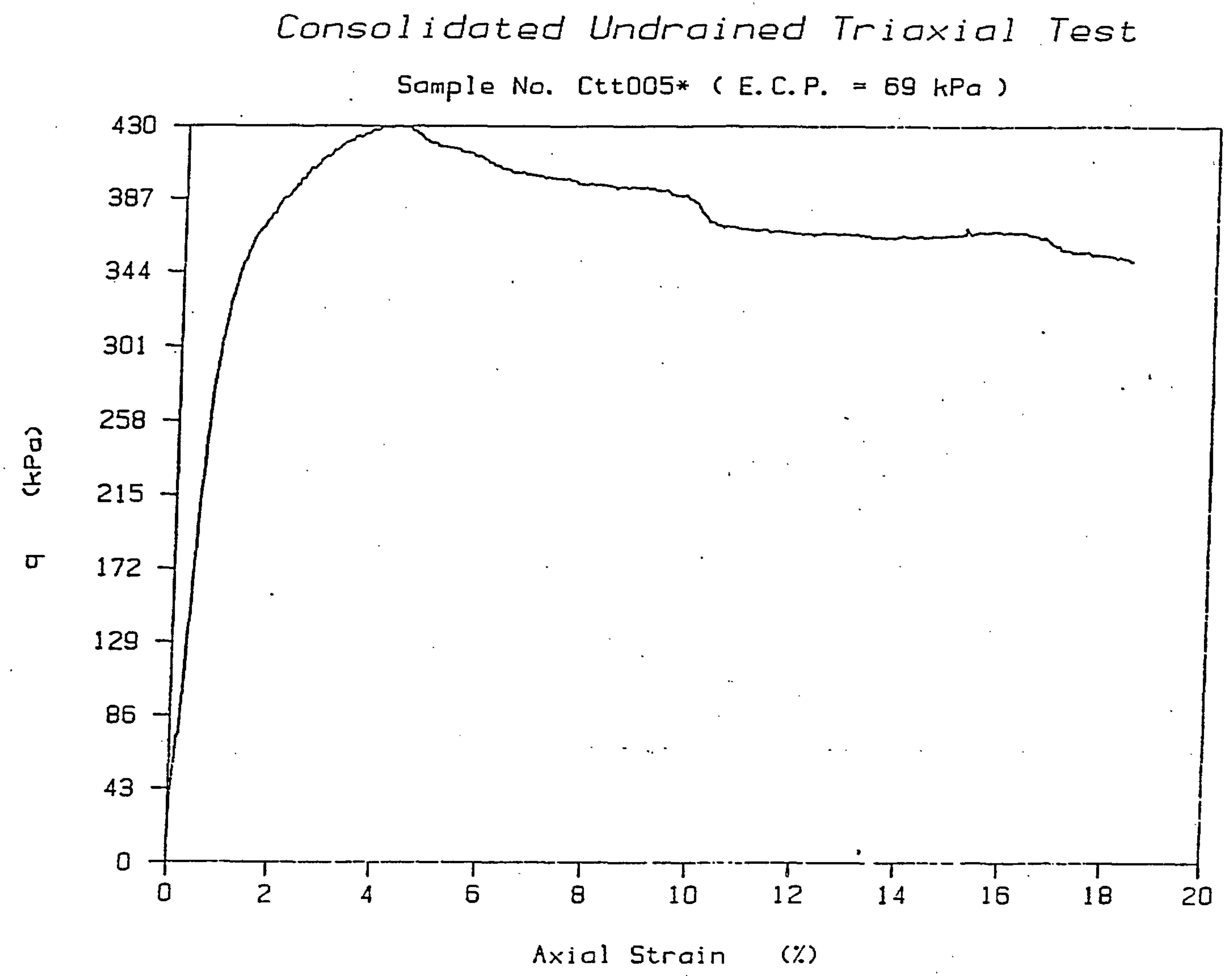


PASTE FILL LIQUEFACTION STUDY

DOME MINES

Failure Envelopes for Paste Backfill

FIGURE 13

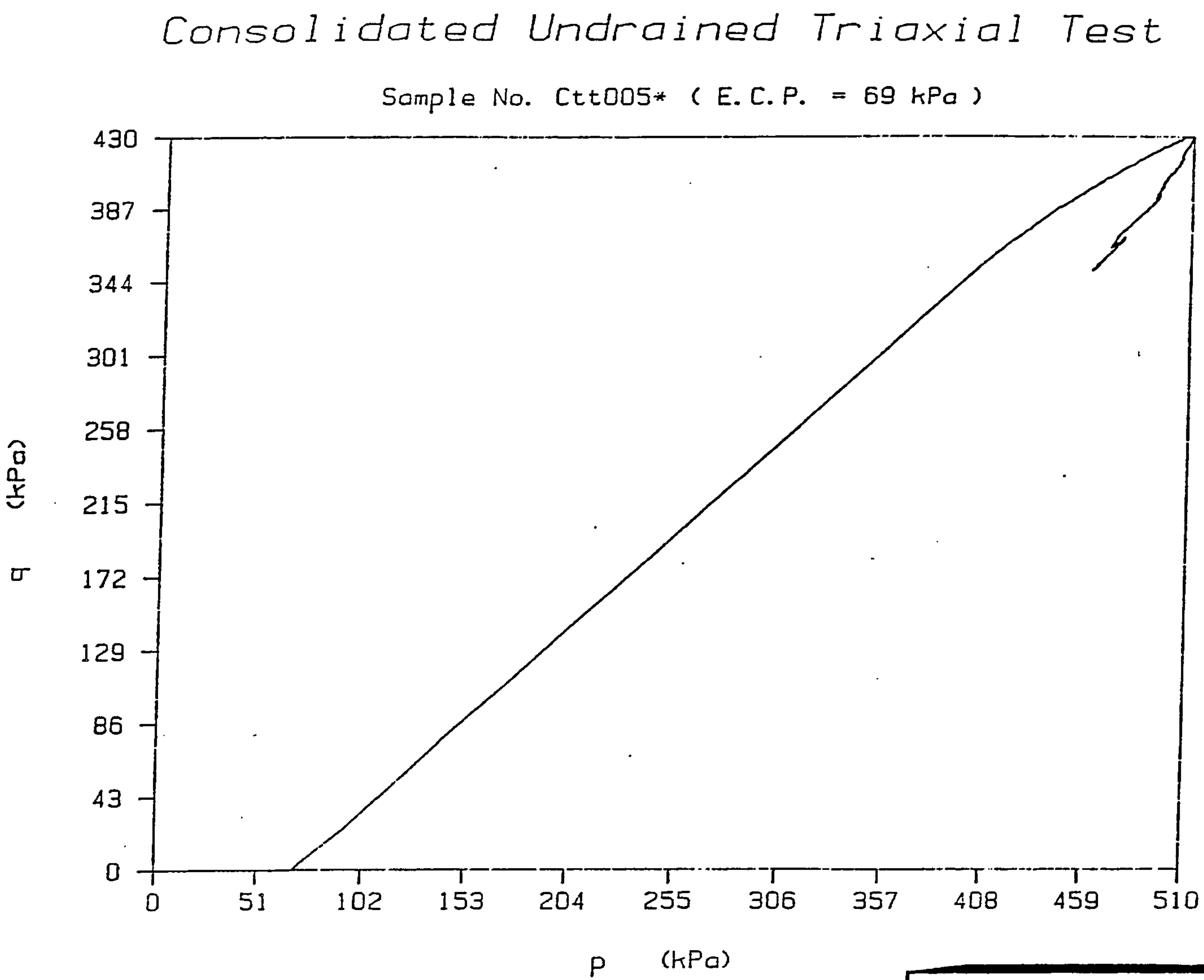


PASTE FILL LIQUEFACTION STUDY

DOME MINES

Stress Strain

FIGURE 14(A)



PASTE FILL LIQUEFACTION STUDY

DOME MINES

Stress Path

FIGURE 14(b)

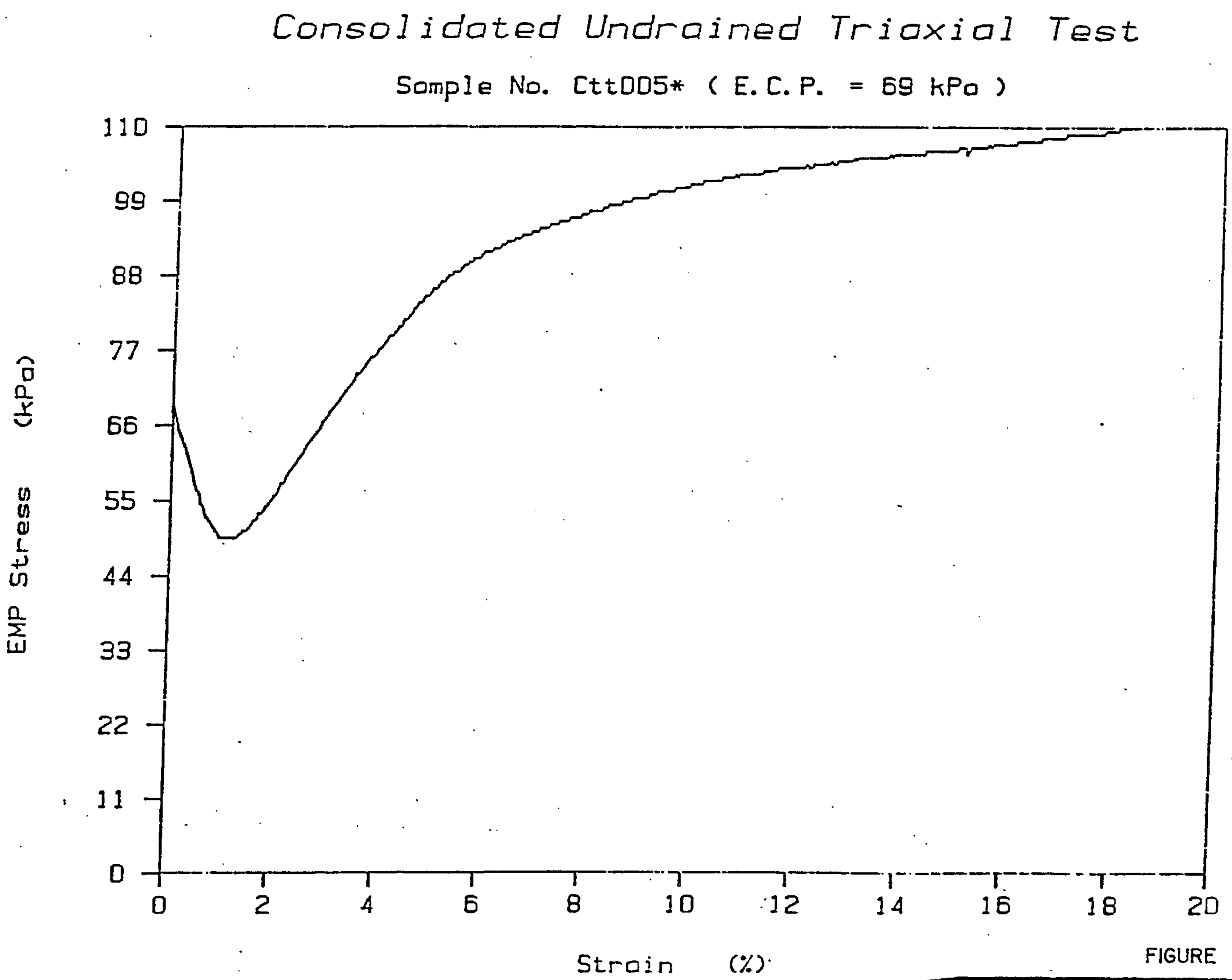


FIGURE 14 (cont.)

PASTE FILL LIQUEFACTION STUDY
DOME MINES
Effective Stress
FIGURE 14(c)

APPENDIX I

CIL REPORT

ON

BLAST VIBRATION MONITORING

CHARACTERISTICS OF BLASTING VIBRATIONS IN DENSE BACKFILL, DOME MINES

(Results of Field Trials, November 16-20, 1987 and March 15-20, 1988)

by

B. Mohanty
Explosives Technical Centre
C-I-L Inc.
McMasterville, Quebec J3G 1T9

July 8, 1988

CHARACTERISTICS OF BLASTING VIBRATIONS IN DENSE BACKFILL, DOME MINES

(Results of Field Trials, November 16 - 20, 1987 and March 15-20, 1988)

INTRODUCTION

The quality of the new dense backfill, now being introduced at the Dome Mines, depends on its ability to withstand the vibrations caused by regular production blasting. Vibrations, under certain conditions, can lead to liquefaction in such unconsolidated materials, thereby threatening the stability of backfills.

The response characteristics of a specific backfill can be modelled to some extent under laboratory conditions, both experimentally and analytically. However, these are obtained on the basis of some arbitrary simple loading conditions such as a 'step', 'ramp' or a triangular load (pressure or acceleration vs. time). It is also possible to model analytically the response due to an impulsive or other complex loading function but none of these realistically duplicate the characteristic loading generated during blasting underground. Since the accuracy of prediction by experimental or theoretical means depends critically on an accurate description of the input waveform, measurements of actual loading conditions, as represented by the blasting vibrations, is considered essential.

The Explosives Technical Centre of C-I-L Inc. undertook to measure these vibration characteristics, under varying blasting conditions, through a series of carefully designed field trials around the 755-stope. This report describes the main findings of the two field trials under this program.

OBJECTIVES

The main objectives of the program were the following,

i) Design suitable transducer and recording systems for simultaneous recording of vibrations at several locations.

ii) Specify blast hole diameters and their locations with respect to transducer locations.

iii) Carry out vibration measurements for a variety of charge size - distance configurations, so as to characterize the rock mass and the dense backfill as fully as possible. This would include short cylindrical charges to long charges, and eventually to multi-hole production blasts with specified charge weight/delay.

iv) Analyze the vibration records, and provide information on maximum vibration level, its duration, and its spectral content as a function of charge size, distance, delay sequence in multi-hole blasting and the nature of transmitting medium (rock or backfill).

v) Assist Dome Mines and McGill University personnel in correlating vibration levels with other measured parameters during the study.

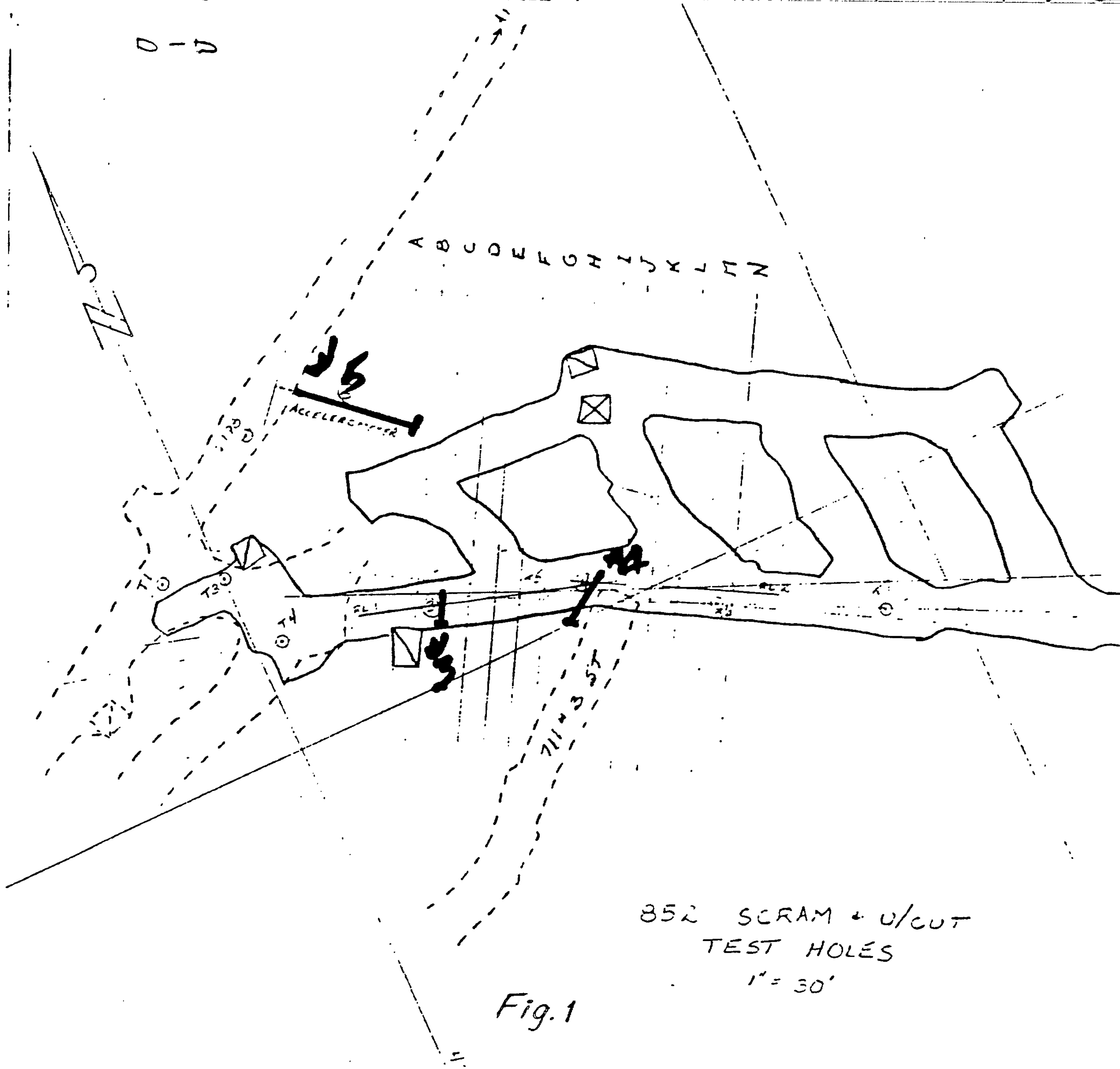

A B C D E F G H I J K L M N
ACCELEROMETER
T1
T3
T4
852 SCRAM + U/CUT
TEST HOLES
1" = 30'

Fig. 1

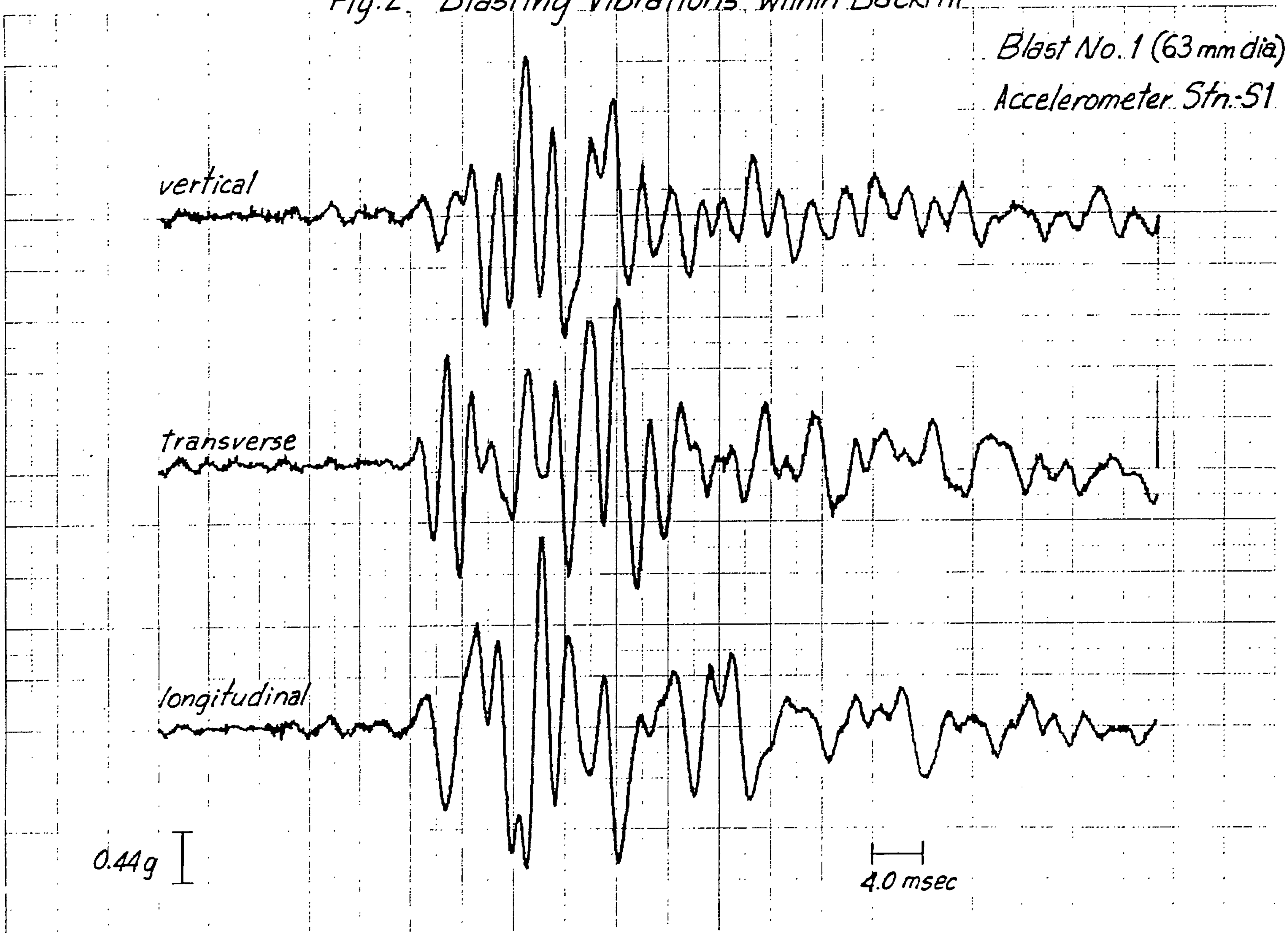
MEASUREMENT OF VIBRATIONS

Instrumentation has been developed to measure either acceleration or particle velocity or both, under far-field and near-field conditions. However, for this study, it was decided to monitor in the acceleration mode.

The vibrations were measured at five locations near the west-end of the stope. Two of the accelerometer stations were placed within the backfill (755 sub-drift). The blast holes (63 mm and 100 mm diameter) were drilled from the 852 scram. Two accelerometers (one placed inside a 100 mm diameter monitor hole, and the other attached to the back of the drift) were placed on this level. The fifth accelerometer station was located on the north side of the stope, designed to study the 'shadow' effect of the backfill on vibrations. Each accelerometer consisted of 3 orthogonally mounted accelerometers. The total system frequency response was d.c. to 10 kHz, the upper end being controlled by the frequency response of the accelerometers.

The blast holes ranged in length from 5 metres to 15 metres and were spaced approximately 1.5 m apart (see Fig. 1). There were a total of 14 blast holes. The charge weight ranged from 5 kg to 55 kg, with MAGNAFRAC 5000 (25 mm x 300 mm) and POWERMEX 500 serving as the test explosives. Specified sections of each borehole were loaded with a cartridge loader. Both single-hole and a 4-hole blasts were carried out during this study. The velocity of detonation was also monitored in the 100 mm diameter boreholes. In addition, two regular production blasts were also monitored during these trials.

Fig.2. Blasting Vibrations within Backfill

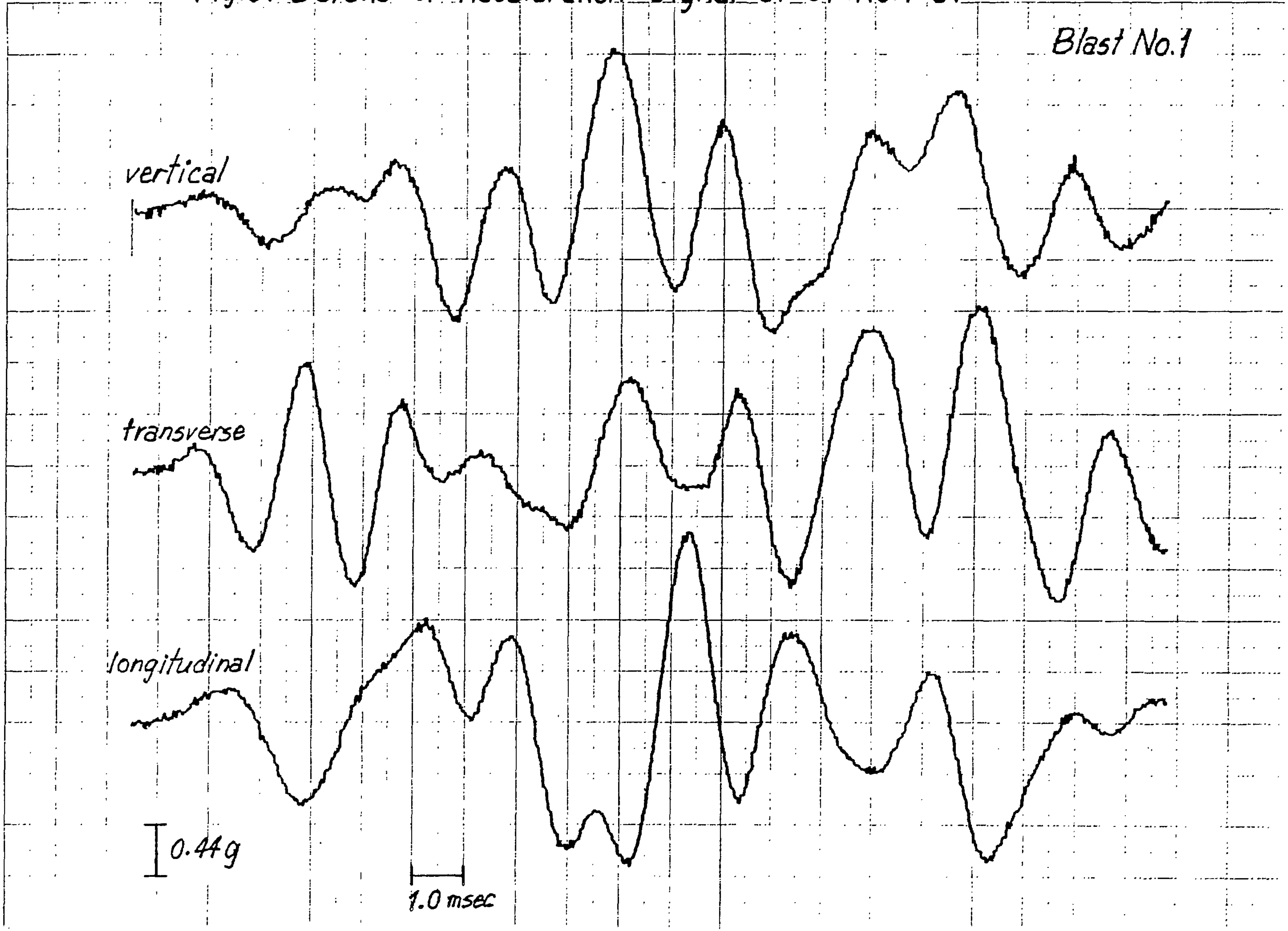


BLASTING VIBRATIONS

The shock waves or seismic waves generated by detonation of explosive in the borehole are collectively known as blasting vibrations. The seismic waves, although more complex than light waves, obey the same laws of optics. They undergo reflection, refraction and diffraction at boundaries and other discontinuities in rock. They can be broadly grouped under 'body waves' and 'surface waves'. The longitudinal (P-wave) and shear waves (S-wave) (and their multiple reflections, conversions and refractions) constitute the body waves, whereas the surface waves are represented largely by the Rayleigh waves. Each of the waves travels at its characteristic velocity in a given medium, with P-wave travelling the fastest and the Rayleigh wave the slowest. Invariably the surfce waves represent the highest amplitude events in a vibration record. However, except for the 'first arrival' represented by the P-wave, it is usually not possible to identify subsequent individual waves or their reflections in a blasting vibration record.

The analysis of blasting vibrations obtained during blasting underground should be viewed in the context of the above statements. A typical underground mine environment presents multiple paths and surfaces to the propagating seismic waves, in addition to the boundaries between backfill and host rock. The vibrations recorded at any location incorporate the collective contributions from these boundaries and discontinuities, in addition to the basic transmission properties of the medium.

Fig.3. Details of Acceleration Signal at Station-S1



RESULTS

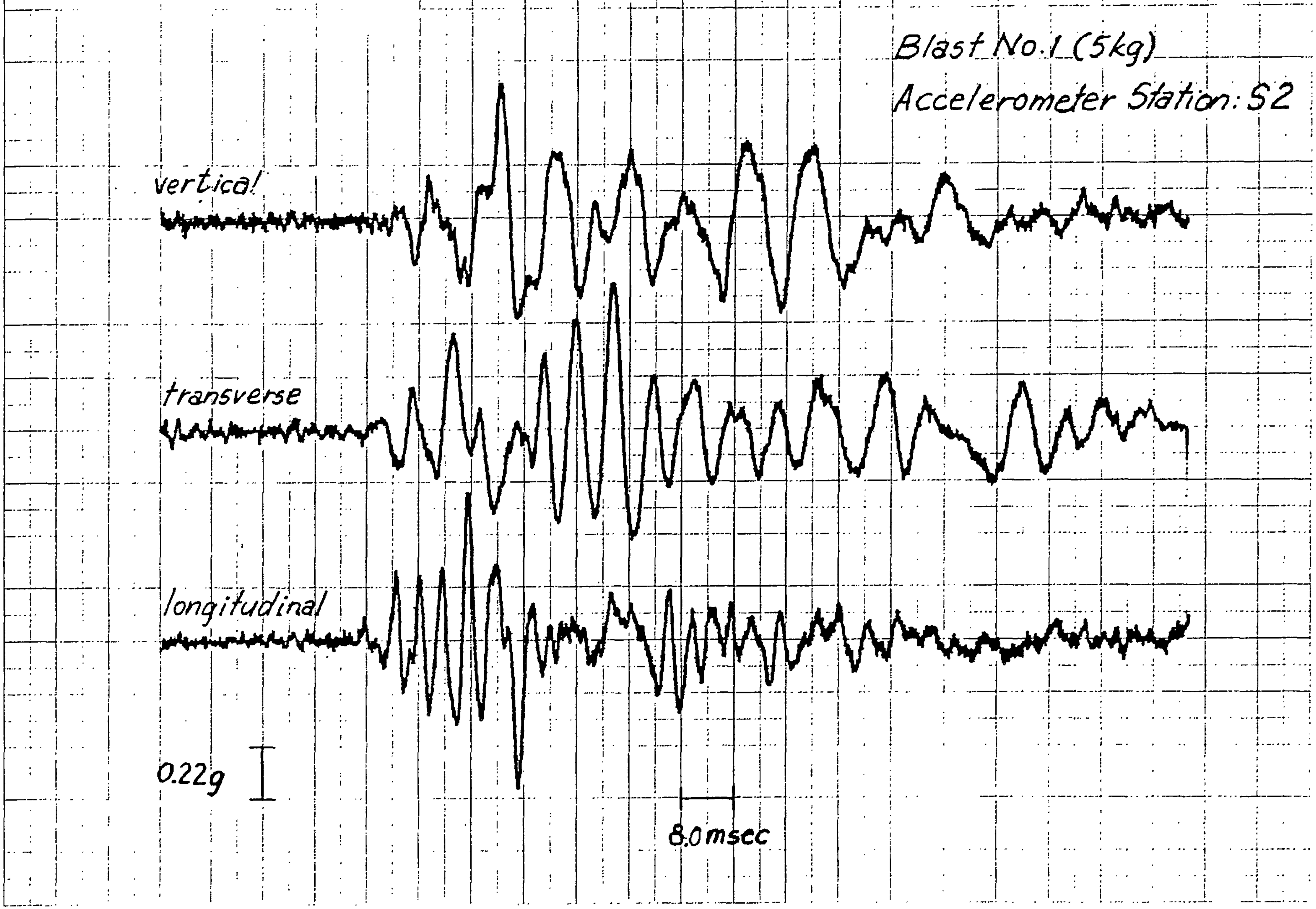
The vibrations recorded by the 3 accelerometers in the deeper acceleromoter station (S-1) in the backfill are shown in Fig. 2. They represent the longitudinal, transverse and vertical components of vibrations, recorded from detonation of a 5 kg charge in a 63 mm diameter borehole (No. N).

The figure shows that the first arrival does not have the highest amplitude, and in fact, each component of vibration is greatly more complex than anticipated. In this case, the duration of actual detonation was only a fraction of one millisecond, whereas the duration of the resulting detonation appears to extend over 60 msec. Although the major portion of the energy can be assumed to be concentrated over a 16 msec range, it is still more than an order of magnitude longer than the duration of detonation.

A time-expanded version of Fig. 2 is shown in Fig. 3. From this an approximate spectral content can be determined. The average dominant frequency of these wave trains is 500 Hz. The maximum acceleration level on any single component is 1.4 g.

The same event, recorded by the shallower accelerometer station (S-2) within the backfill is shown in Fig. 4 (note the different time scale). The characteristics of the vibration after it has travelled about 30 feet (the approximate path difference between S-1 and S-2) in the backfill have undergone significant changes compared to those recorded by S-1 (located closer to the boundary between the host rock and the backfill). The amplitude has fallen by

Fig. 4. Blasting Vibrations within Backfill



about half, as has the frequency. The difference is more clearly illustrated in Fig. 5. The top trace is the vertical component of acceleration as measured at S-1 station; the bottom three traces are vertical, transverse and longitudinal components of acceleration as recorded at the shallower station S-2.

The other notable difference between S-1 and S-2 arrivals is the greatly lengthened pulse train of the latter. This reflects the attenuating property of the backfill material. The maximum acceleration level recorded along any single component at S-2 is 1.1 g, and the dominant frequency is 155 Hz. Both these are considerably lower than recorded at station S-1.

The effect of changing borehole diameter and charge weight on acceleration is shown in Fig. 6. It represents a 33 kg blast in a 100 mm diameter borehole (charge length: 3.6 m), as recorded at station S-1. As expected, the peak values are higher compared to those obtained in the 63 mm diameter blast (Fig. 2). The most pronounced difference however lies in the frequency characteristics of the vibration signal. The damped sinusoids following the main arrivals are almost absent in the larger diameter blast. The latter is characterized by significantly lower frequency content, with the average frequency being 200 Hz, compared to 500 Hz obtained with the 63 mm diameter blast.

Fig. 7 shows the most energetic component of acceleration as recorded by a surface mounted accelerometer located in the 852 scram itself. This is the nature of waveform recorded in host rock, which should be compared with those

Fig.5. Comparison of Acceleration Signal between Stations S1 & S2

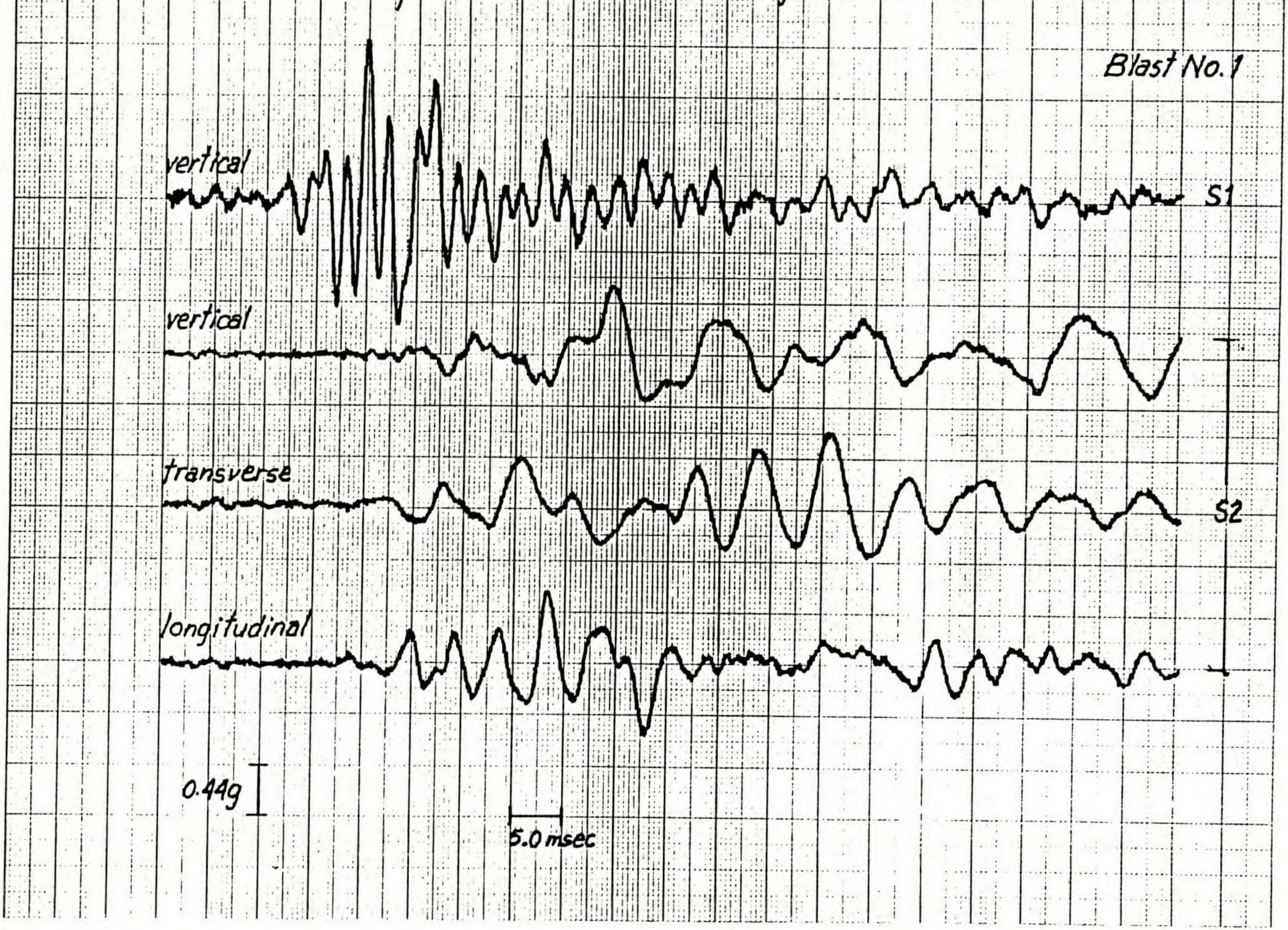


Fig.6. Acceleration Signal within Backfill (Station-S1)

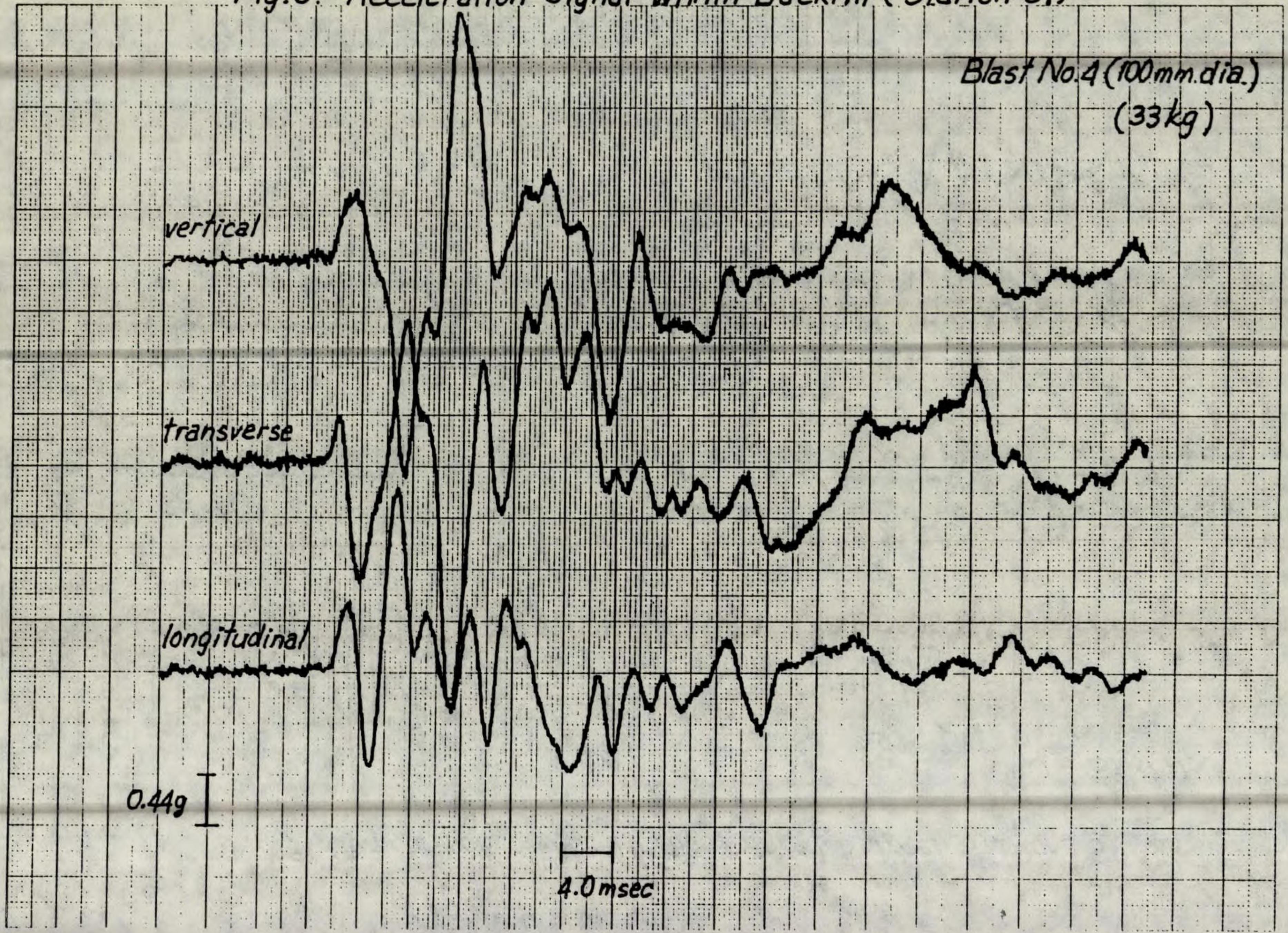
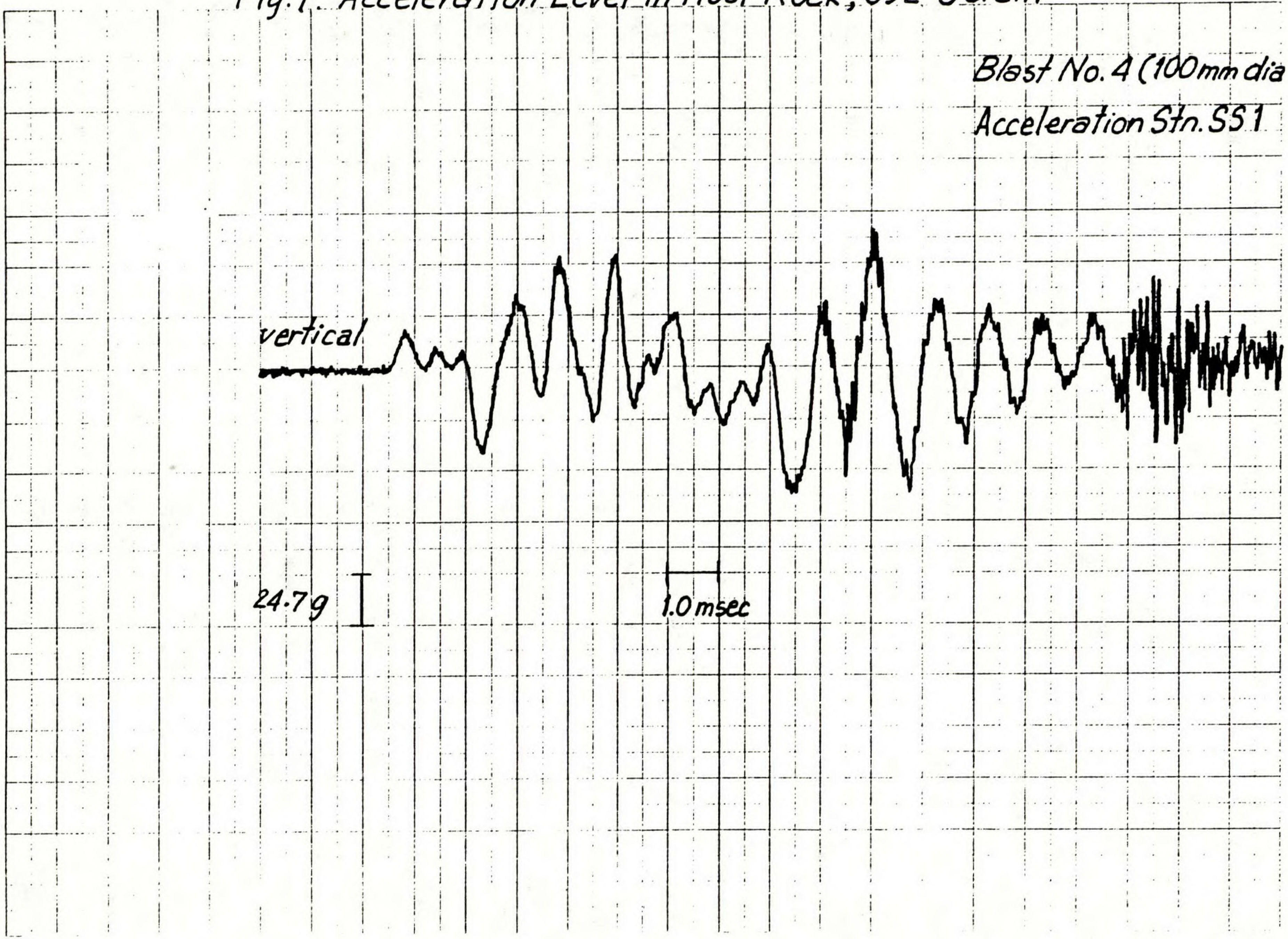


Fig. 7. Acceleration Level in Host Rock, 852 Scram



recorded within the backfill (see Fig. 3). The difference between Fig. 7 and Fig. 3 (note the change in time scale in the former) is obvious. The rock, as expected, can support much higher frequency events than the backfill, the dominant frequency in this case being 750 Hz compared to 155 Hz within the backfill.

There is a noticeable difference between the vibration signals in terms of their spectral content, recorded from small diameter (63 mm) and larger diameter (100 mm) blasts. The former, as expected, yields a higher frequency signal. This difference is shown in Figs. 8 and 9.

The vibration record obtained from a 4-hole blast (all 100 mm diameter) is shown in Fig. 10, for a single component of motion, for both backfill and hard rock. The specified delay times have been compared with those actually recorded. The charge weight for the 3 holes varied within narrow limits (~15 kg for the 2nd and 3rd hole, and ~50 kg for the 1st and the 4th hole).

The resulting acceleration signal is however quite different. Except for the 2nd hole, the scatter in delay firing times is within acceptable levels. For the 2nd hole, the deviation is about 25%. Such deviations for any single detonator are well within the statistics, but have to be accounted for in designing large production blasts.

Even more significant than the firing times are the relative amplitudes of the vibration signals for the 4-hole blast. The 2nd hole appears to be much of greatly reduced strength compared to the rest of the round. This picture is somewhat reversed in the vibration record from the backfill, where the same

Fig. 8 Acceleration in Backfill

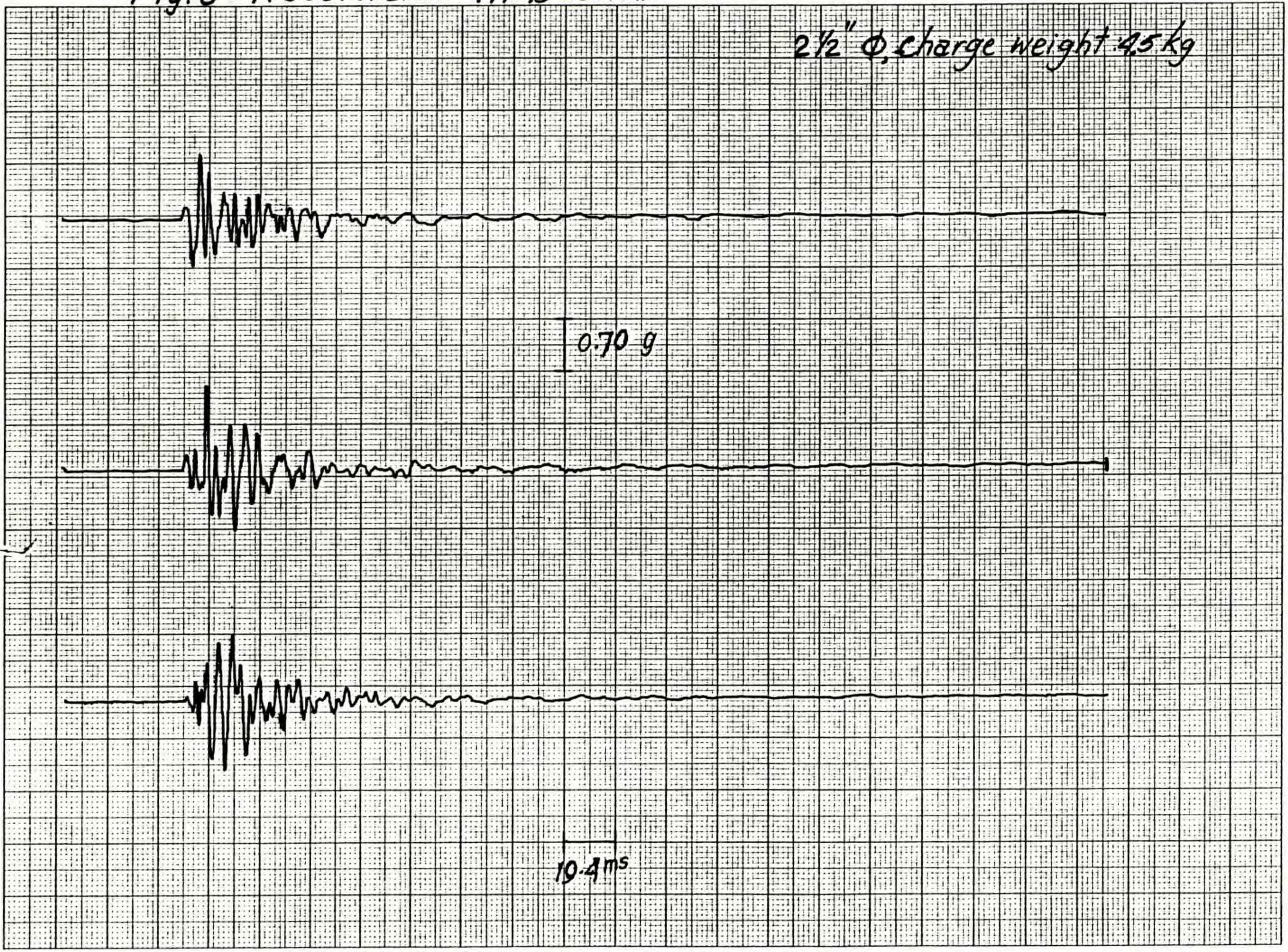
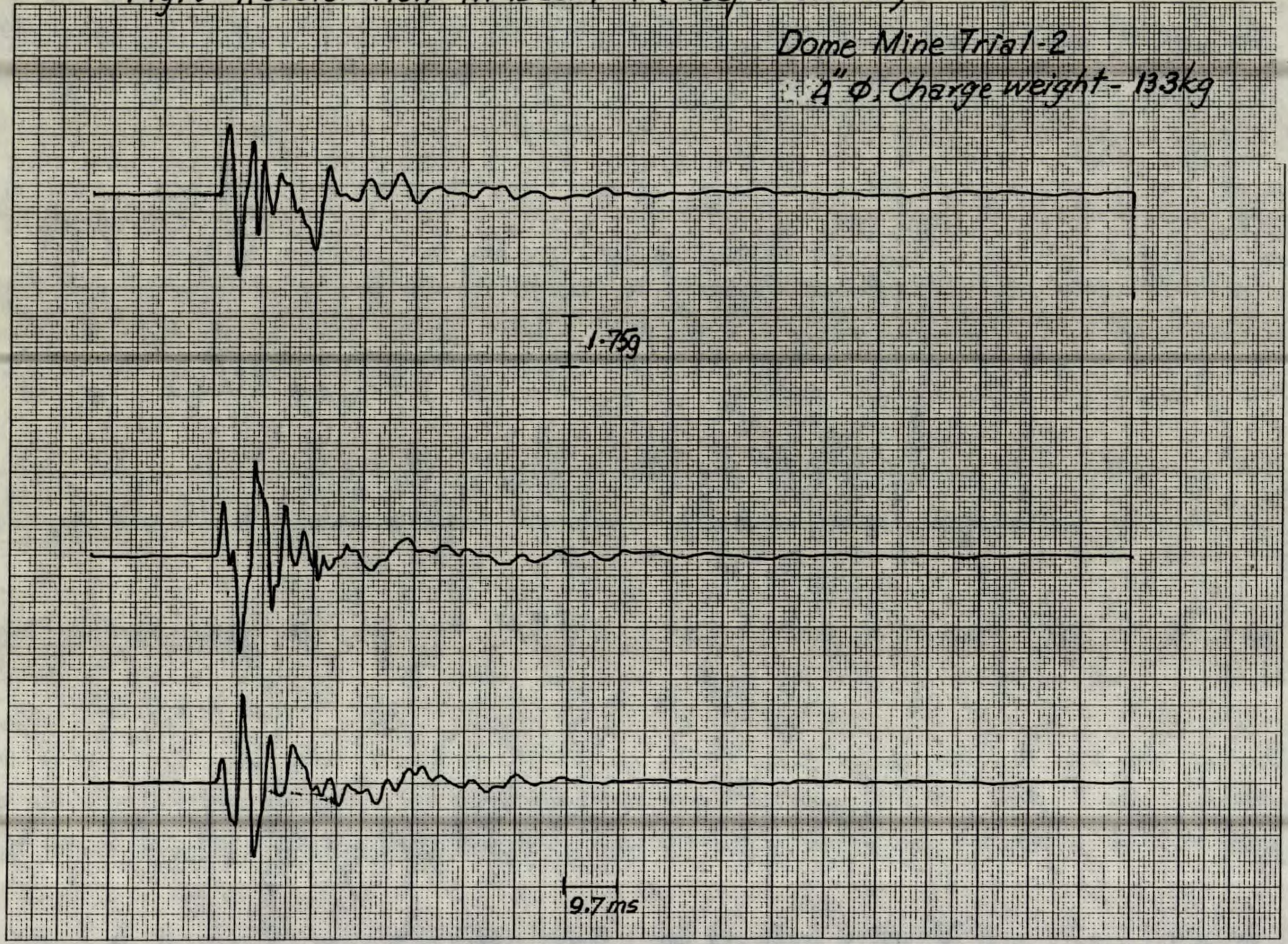


Fig. 9 Acceleration in Backfill (deeper station)



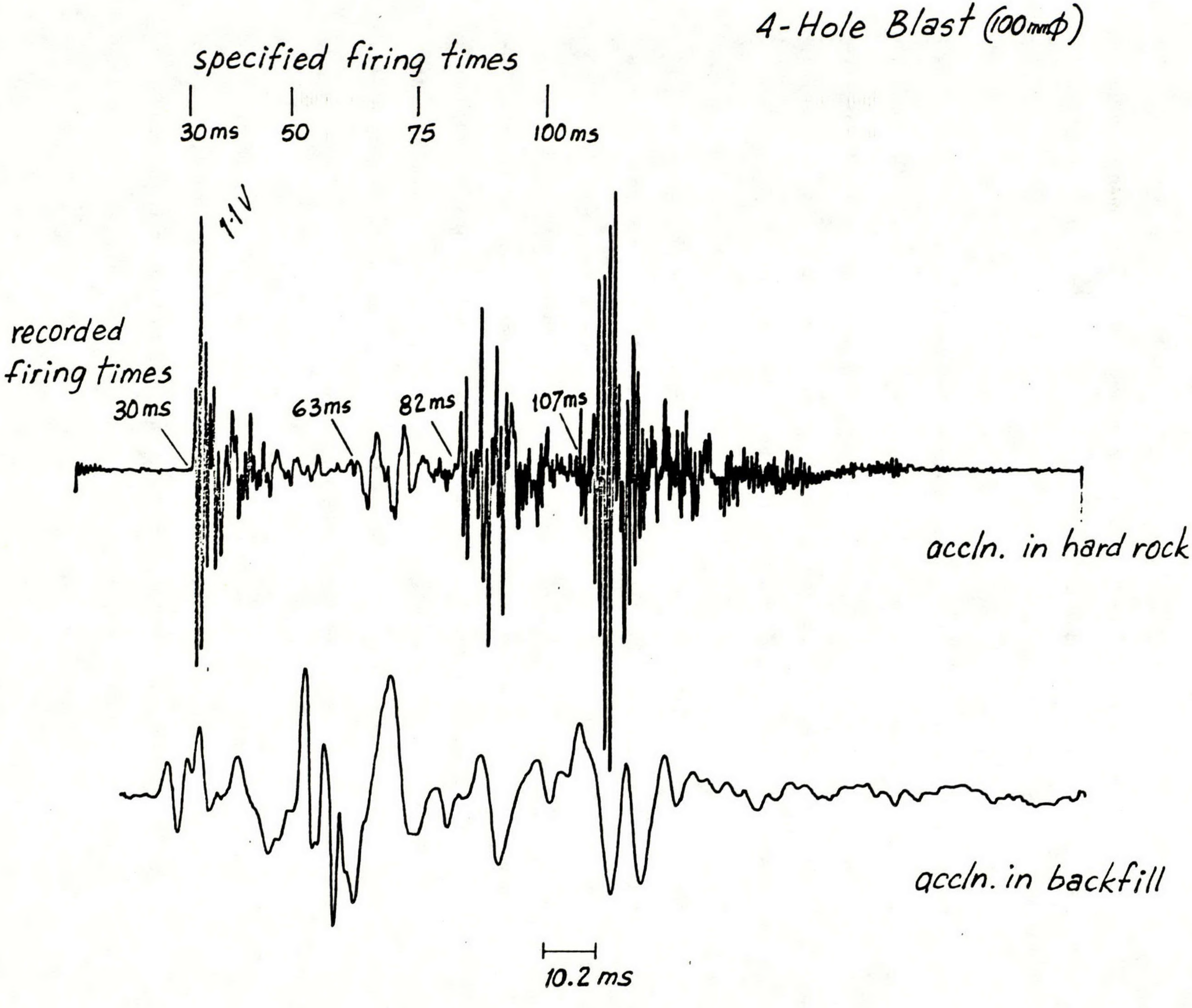

4-Hole Blast (100mmφ)
specified firing times
30ms
50
75
100ms
1·1V
recorded firing times
30ms
63ms
82ms
107ms
accln. in hard rock
accln. in backfill
10.2 ms

Fig 10

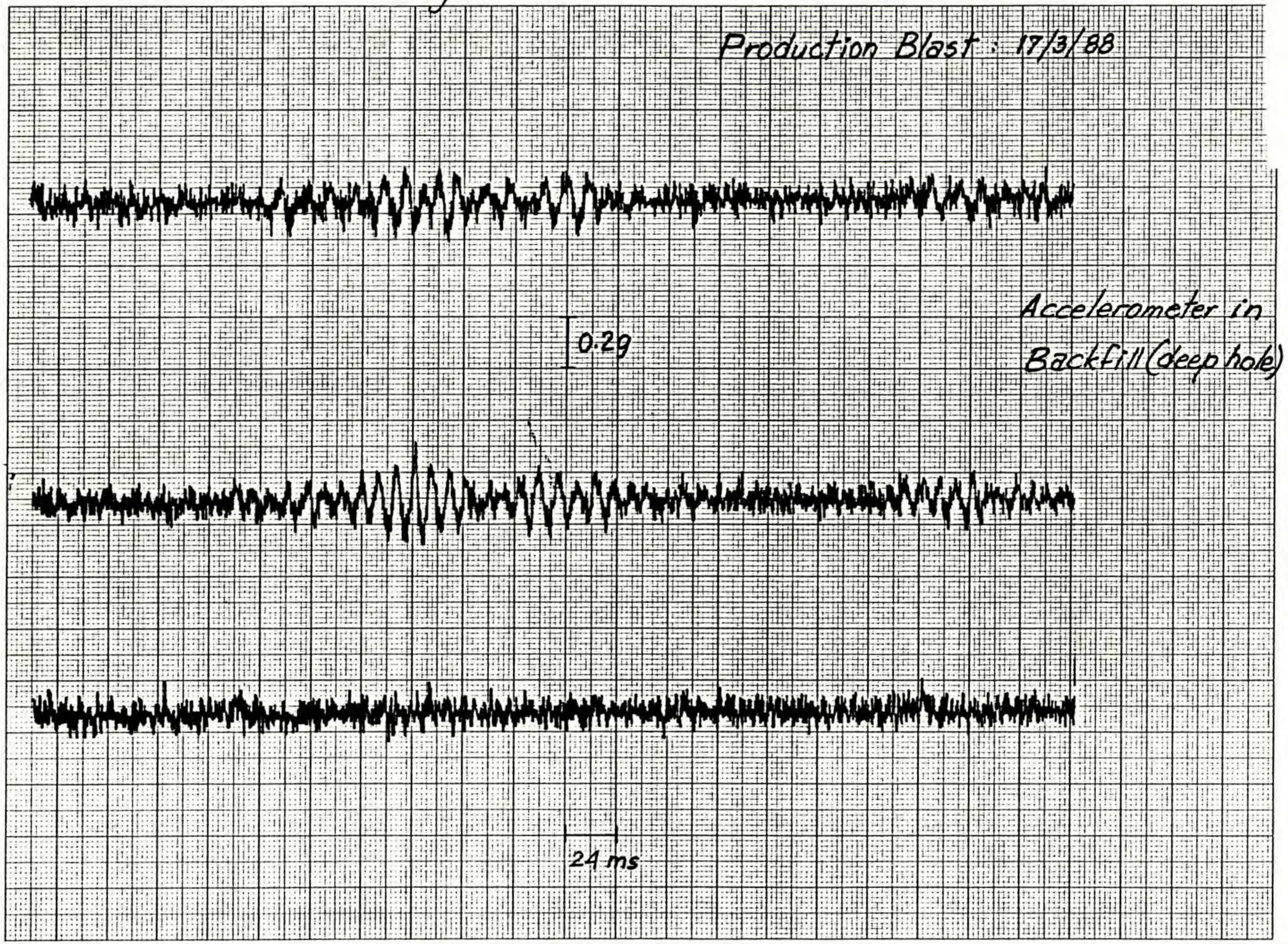
event appears to be the largest. In addition, the individual arrivals are smeared due to attenuation of high frequencies in the backfill material. This results in merging one event into another, and is likely to give rise to unexpected amplification (and also reduction) of vibration signals from multihole production blasts.

During the course of these trials, two regular production blasts were also monitored. A section of the vibration wave train as recorded in the backfill is shown in Fig. 11. The levels are very low, although each arrival from individual delay rounds appears to be 'streched out'. No discernible vibration levels were recorded from the second production blasts.

The true level of measured acceleration is obtained by vector summing of the three components of motion recorded at each station. However, as the figures show, all the acceleration peaks do not align themselves at any single instant of time. The different travel paths traversed by the seismic waves, and the presence of reflecting surfaces, are responsible for this. A practical approach is to select a time 'window' and vectorially add the highest components in this window. A more conservative approach, and the one adopted in this study, is to take the peak acceleration value on a sensor regardless of its location on the trace, and obtain the vector sum.

The resultant peak acceleration (0 to peak value on any trace) obtained by this approach is shown in Fig. 12, as a function of scaled distance ($m/kg^{1/3}$). This is a convenient form to plot the data so as to be able to estimate peak acceleration values at any other explosive charge weight and distance combination. The accuracy of prediction, of course, depends critically on the

Fig. 11


Production Blast : 17/3/88
Accelerometer in Backfill (deep hole)
0.2g
24 ms

degree of scatter in the measured data. The trend line shown in Fig. 7 represents measurements obtained at station S-1 and S-2 within the backfill from over ten blasts. The upper level data represent the 63 mm diameter blast, the rest, the 100 mm diameter blast.

Although the data are limited they show good linear fit, and can be represented by the following equation,

$$a = 44\left(\frac{X}{W^{1/3}}\right)^{-1.53}$$

where 'a' is the peak acceleration in 'g', X is the distance to centre of charge in metres, and W is the explosive charge weight in kg. The higher level accelerations recorded from the 63 mm diameter blast appear anomalous. Clearly more work needs to be done in both 100 mm and 63 mm diameters.

As stated earlier, there is a considerable increase in both amplitude and the frequency of vibration signal in hard rock when compared to those obtained in backfill material. Detailed analysis of the data obtained in hard rock falls outside the scope of this study. However, the amplitude characteristics of acceleration in hard rock at the mine can be estimated from the following.

$$a = 352\left(\frac{X}{W^{1/3}}\right)^{-1.53}$$

CONCLUSION

The acceleration levels within the dense backfill have been successfully measured for a variety of blasting conditions.

A working peak-acceleration vs. scaled distance relation, equating acceleration levels with explosive charge-weight and distance, has been derived. This represents the first step in describing the vibration transmission characteristics of the dense backfill.

The backfill exhibits significant amplitude as well as frequency attenuating properties for the small single-hole blasts (5 kg to 55 kg of explosive) and the distance range studied (up to 35 m). The maximum resultant acceleration recorded just inside the backfill was 2.3 g. Further into the fill it was typically 1.0 g. The corresponding frequency reduced to 155 Hz from 500 Hz. The duration of vibrations within the backfill was 60 msec although the corresponding detonation event giving rise to these vibrations was less than 1 msec in duration. The major fraction of the vibration energy was however limited to about 15 msec.

The acceleration signals are exceedingly complex and cannot be represented by a simple loading function in the laboratory to determine the response of the dense backfill. Any analytical approach would have to take into account this complex waveform as the input loading function.

There appears to be a clear distinction between vibration signals from a small diameter (63 mm) and an intermediate diameter (100 mm) blast, with the latter yielding lower frequency vibrations.

In view of the excessive duration of vibration signals in the backfill even with single-hole blasts, it is certain that some superposition of acceleration would occur in multi-hole delay blasts or regular production blasts. The degree of superposition resulting in possible vibration enhancement would have to be determined by actual monitoring of such blasts.

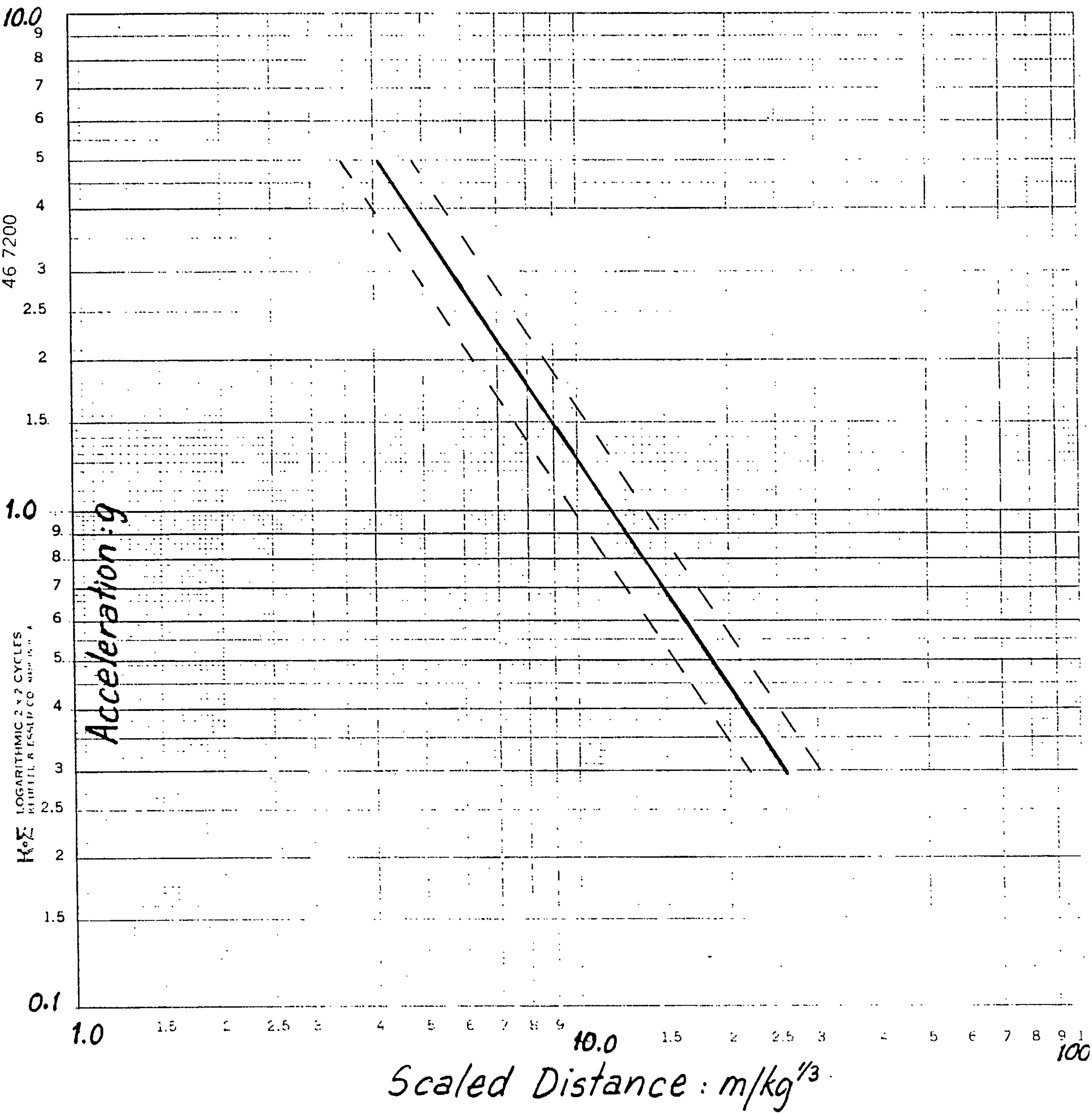
During the course of these, two regular production blasts were also monitored. These blasts produced very low levels of vibration in the backfill.

ACKNOWLEDGEMENTS

I am indebted to Mr. John Vehkala and Mr. Dale Churcher of Dome Mines for making the necessary arrangements and providing the facilities for this field trial, and to Mr. K. Aref of McGill University for leading assistance during the trials. The trials could not have been carried out with such success without the very able assistance of Mr. Mahmued Alam of the Explosives Technical Centre, C-I-L Inc, and Mr. Dale Churcher of Dome Mines.

Dr. B. Mohanty, P. Eng.
Explosives Technical Centre
July 8, 1988

Fig.12 Dome Mines Trials : Vibration in Dense Backfill



APPENDIX II

CONETEC REPORT

ON

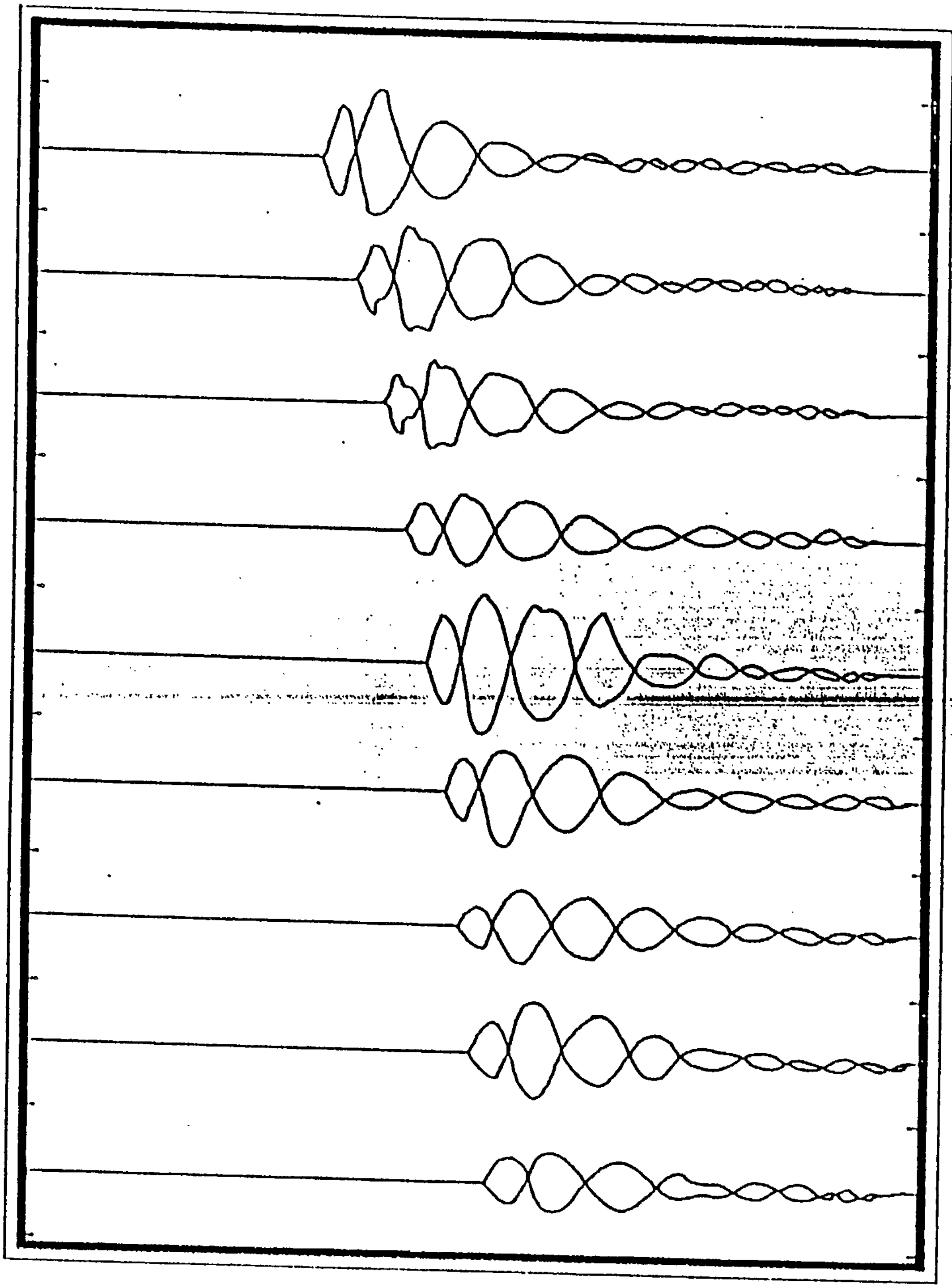
SEISMIC CONE PENETRATION

TEST DATA

PRESENTATION AND INTERPRETATION OF SEISMIC CONE PENETRATION TEST DATA

MINE TAILINGS

DOME MINES, ONTARIO



PREPARED BY:

CONETEC INVESTIGATIONS LTD.
2447 BETA AVENUE
BURNABY, B.C.
V5C 5N1

PRESENTATION AND INTERPRETATION

CONE PENETRATION TEST AND

SEISMIC CONE PENETRATION TEST DATA

PLACER DOME INC., DOME MINE

SOUTH PORCUPINE, ONTARIO

Purchase Order No. 74710

Prepared for:

Placer Dome Inc., Dome Mine
P.O. Box 70
South Porcupine, Ontario
PON 1HO

and

Journeaux, Bedard & Associates
1783 Newman Crescent
Dorval, Quebec
H9P 2R6

April 2, 1988

Prepared by:

ConeTec Investigations Ltd.
2447 Beta Avenue
Burnaby, British Columbia
V5C 5N1

TABLE OF CONTENTS

1.0 INTRODUCTION

This report presents the results of a cone penetration testing (CPT) and seismic cone penetration testing (SCPT) program carried out on modified mine tailings which have been used as underground backfill at Dome Mine in South Porcupine, Ontario. This program was carried out between March 7 and March 11, 1988. The work is part of a detailed geotechnical program being carried out on the backfill material by McGill University for Placer Dome Inc. We understand that additional geotechnical input has also come from Journeaux, Bedard and Associates.

It is our understanding that the purpose of the geotechnical program as a whole is to determine in situ strength, deformation and hydrogeologic parameters. The use of the piezometer cone penetration test was therefore suggested to augment previous testing data. We understand that this short program, consisting of four cone soundings (including one with seismic data collection), was carried out as a trial to evaluate the potential of using the cone as an accurate, reliable and economical means of providing the necessary in situ geotechnical information for design, verification of backfill adequacy, and for determining liquefaction potential of the backfill material.

Upon review of the data presented herein and in light of the logistical success of the cone equipment, we suggest that this program has demonstrated the vast potential for the utilization of the electric piezometer cone (with or without seismic capability) in the underground environment as the premier tool for determining geotechnical parameters of underground backfill.

The funding for this program came from three sources:

(1) Dome Mines (45%)

(2) Journeaux, Bedard & Associates (22%)

(3) ConeTec Investigations Ltd. (33%)

The funding contributions of Dome Mines and Journeaux, Bedard & Associates was through payment to ConeTec Investigations Ltd for services provided. ConeTec's contribution came from a reduction in both fees and equipment modification and construction.

2.0 FIELD EQUIPMENT AND PROCEDURES

2.1 Electric Cone Testing

The cone penetration tests (CPT) were carried out by ConeTec Investigations Ltd. of Vancouver, British Columbia using an integrated electronic cone system. A 10 ton subtraction type cone was used for all of the soundings. This cone has a tip area of 10 sq. cm. and friction sleeve area of 150 sq. cm. A piezometer element 5 mm. thick is located immediately behind the cone tip. The subtraction cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.85. Refer to Figure 1 for an illustration of the subtraction cone.

The cone used during the program was capable of recording the following parameters at 5 cm. depth intervals:

- Tip Resistance (Qc)
- Sleeve Friction (Fs)
- Dynamic Pore Pressure (Ut)
- Temperature (T)
- Cone Inclination (i)

The cone and accompanying data acquisition systems were also capable of recording shear (S) and compression (C) wave arrival times.

The above parameters, excluding the S and P wave velocities were printed simultaneously on a printer and stored on a bubble cassette cartridge for future analysis and reference.

The pore pressure element was located directly behind the cone tip. The pore pressure element was 5.0 mm thick and consisted of porous plastic. Each of the elements were saturated in glycerin under vacuum pressure prior to penetration. Pore pressure dissipations were recorded at 5 second intervals during pauses in the penetration.

A complete set of baseline readings were taken prior to each sounding to determine temperature shifts and any zero load offsets. Establishing temperature shifts and load offsets enables the engineer to make corrections to the cone data where necessary. These corrections are extremely important, especially where the load conditions are relatively low, and generally are the single largest source of error with respect to the accuracy of cone data.

The cone was pushed using a specially designed hydraulic ram assembly with an approximate capacity of 10 tons. The ram assembly was supplied by ConeTec. Dome Mines supplied the hydraulic power pack.

Three cone penetration tests and one seismic cone penetration tests were performed. The penetration tests were carried to depths varying between 6.4 meters and 28.5 meters below the existing ground surface. It is our understanding that the test holes were located accurately in the field by personnel under the direction of Dome Mines.

2.2 Seismic Cone

The equipment and procedures used in this investigation in general were as developed at UBC and reported by Rice, 1984, Laing, 1985 and Robertson et al, 1986. The procedure was incorporated within the cone penetration test (CPT) and conducted when the cone penetration test was stopped to add additional push rods. At the end of the first push, and at one meter intervals thereafter, shear wave velocity measurements were made. The CPT rods are one meter long, and therefore accurate depth intervals were ensured by always pushing the cone rods one meter and ending at the limit of the hydraulic ram system.

Shear waves were generated by striking a steel beam which was held down by rock jacks which were jacked against the back of the stope. This method provided excellent coupling of the beam to the ground surface. The steel beam was approximately 2.3 meters long, 0.2 meters wide and 0.2 meters deep. The centre of the beam was offset horizontally from the cone rods a distance of 3.3 meters. This offset was accounted for in the velocity calculations.

A friction reducer was used to reduce the friction between the soils penetrated and the cone rods behind the cone. The particular reducer used is oversized compared to conventional reducers to facilitate the decoupling of the cone rods from the soils in an effort to produce a high quality signal and produce a high signal to noise ratio.

The beam was struck lightly for shear wave generation using a 7.25 kg sledge hammer in a horizontal direction, parallel to the active axis of the transducer, first from one end and then the other. Each wave was inspected and the procedure was repeated if necessary. Occasionally, excessive vertical components of the striking force resulted in excessive generation of compressional waves and the procedure was repeated. A contact trigger between the beam and the hammer produced accurate triggering times and allowed for the accurate timing of S wave markers.

After each pair of wave traces was recorded, inspected and saved, the two traces were overlain on a digital oscilloscope screen and the arrival and first crossover times were selected.

The shear waves were detected by a horizontally active geophone in the cone which was offset above the cone tip a distance of 0.2 meters. This offset is accounted for in all calculations. The geophone signal was amplified by 100 times prior to processing in the storage oscilloscope. The oscilloscope sampled at a frequency of 50 kHz (ie. 50,000 samples per second). In general, 15,000 sample points were taken per wave trace. The voltage resolution used was 8 bit. In order to maintain acceptable voltage resolution the input sensitivity of the signal was increased with depth to maintain the full voltage resolution.

3.0 CONE PENETRATION TEST DATA AND INTERPRETATION

3.1 CPT Data

The cone penetration test data is presented in graphical form on Figures 2 to 5 inclusive following the text of the report. Penetration data is referenced to existing ground surface. This includes the wall level where horizontal and inclined penetration was carried out for soundings CPT 700-1 and CPT 700-2.

The stratigraphic interpretation is based on relationships between cone bearing, Qc, sleeve friction, Fs and dynamic pore pressure Ut. The friction ratio, Rf (sleeve friction divided by cone bearing) is a calculated parameter which is used to infer soil behaviour type. Generally, cohesive soils have high friction ratios , low cone bearing and generate large excess pore water pressures. Cohesionless soils have lower friction ratios , high cone bearing and generate little in the way of excess pore water pressures.

The interpretation of soils encountered on this project was carried using recent correlations developed by Robertson et al.(1986) (refer to Figure 6). It should be noted that it is not always possible to clearly identify a soil type based on Qc, Fs and Ut. Occasionally soils will fall within different soil categories on the two different charts shown in Figure 6. In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behaviour type. The correlations were developed for naturally occurring soil deposits, not for mine tailings. However, based upon considerable experience with cone penetration testing and analyses in mine tailing deposits, the correlations appear to work reasonably well in these materials. Computer tabulations of the interpreted soil types (based on the upper chart in Figure 6), along with certain other geotechnical parameters for each cone hole is presented in Figures 7 to 10 following the text of the report. Reference should be made to those articles listed in Appendix A for a description of the methods used to calculate the geotechnical parameters.

For this investigation, the soil type was known because the backfill material was placed under controlled conditions; ie. it is not a natural deposit. However, the soil behaviour type interpreted from the cone data will still vary somewhat over the depth of each hole due mainly to cement content of the mine tailings; the cement to tailings ratio does not appear to be constant.

The backfill material for the most part appears to be unsaturated and therefore the potential for liquefaction under the present hydrogeological regime is reduced. However, if these conditions do change, zones of liquefiable materials may be present. On Figures 7 to 10 cyclic stress ratios are presented with the interpreted cone data. These values should be studied with respect to the design seismic event(s) if it is felt that saturation is a possibility for some or all of the backfill. A detailed geotechnical assessment of the liquefaction potential of the backfill material was not part of our terms of reference for this project.

3.2 Seismic Data and Interpretation

The variation in shear wave velocity with depth calculated for CPT 600-2 is shown in Figure 11 following the text of the report. The velocity profile shows the shear wave velocity plotted at a depth midway between the one meter test intervals. The pertinent data for the seismic profile is presented in Tables 1. Based on the seismic cone test data presented, the shear wave velocities in the penetrated tailings backfill vary quite widely between about 170 m/sec and 930 m/sec. The wide range in the in situ shear wave velocities measured is most likely due to the close proximity of the hanging wall of the backfilled stope to the cone sounding location. It is apparent by looking at the results that an irregular hanging wall surface had a significant effect on some the shear wave velocities measured (values of greater than approximately 400 m/sec). Reflected and converted waves which travel much quicker in the rock than in the soil (backfill) have masked the true shear wave arrivals. The dynamic shear modulus (Gmax) at each test location was calculated from the relationship:

$$Gmax = p \times Vs \times Vs$$

where: p = the mass density estimated at 1800 kg/m3

Vs = the shear wave velocity

The variation in the dynamic shear modulus with depth is plotted on Figure 12.

By striking the beam at each end, two wave traces were recorded and overlain. These "polarized" wave traces better define the arrival and crossover markers.

The shear waves generated by this type of source are vertically propagating waves with a horizontal particle motion. They have a small amplitude and are thus representative of a small strain phenomenon.

The shear wave arrivals and crossovers were converted to interval velocity measurements using the following procedure. For the first interval, the first shear wave arrival was used. For subsequent one meter intervals, the crossover times were used to calculate the interval velocity. The use of crossover times was researched at UBC by Rice, 1984, who found that they were much easier to define than arrival times. An example showing arrival and crossover markers for a shear wave is shown in Figure 13. After selecting the first arrival time and crossover times each was corrected for the horizontal offset of the shear source by assuming a straight line travel path. This correction is significant in the upper several meters. From the corrected times and change in depth, the interval velocities were calculated. This interval velocity is representative of the soils between the depths where the measurements were taken.

Overall, the seismic data appears useful and the small strain shear modulus values calculated are generally reasonable. As mentioned above, the shear wave velocities of greater than approximately 400 m/sec. are probably influenced by bedrock interference. Where these higher shear wave velocities are measured, the shear modulus will also be calculated as being higher than is the actual case for the material.

The second geophone (perpendicular to the other in the horizontal plane) essentially showed no response during the testing. This lack of response is as would be predicted for the situation where the cone was properly aligned with respect to the seismic source and no rotation of the cone was permitted during the sounding.

4.0 CLOSURE

This program was unique in both location of the test program (up to 700 feet below ground surface) and the hydraulic assembly used to push and pull the cone and cone rods. Despite these constraints, four successful cone soundings were performed in three days of testing. We are certain that if more testing was to be carried out, significant improvements could be made to the ram assembly to reduce the setup time which comprises well over 50% of the actual production time spent underground. As stated in the introduction to this report, we are confident that the electric cone is the most accurate, reliable and economic method available for providing in situ geotechnical parameters of the tailings backfill material. If more extensive programs using the cone in the backfill are performed, correlations with properties such as cement content, liquefaction potential, shear and volumetric strain potential under cyclic mobility, may be obtained from the cone data in addition to the parameters presented within this report.

In addition, we would be glad to undertake a much more detailed study of the liquefaction/cyclic mobility of the backfill materials on a consulting basis using the cone data available. We would perform parametric analyses by investigating various hypothetical hydrologic situations.

We trust that the information presented in this report is sufficient for your purposes. If you have any questions regarding the contents of the report, please do not hesitate to contact our office.

Yours truly,

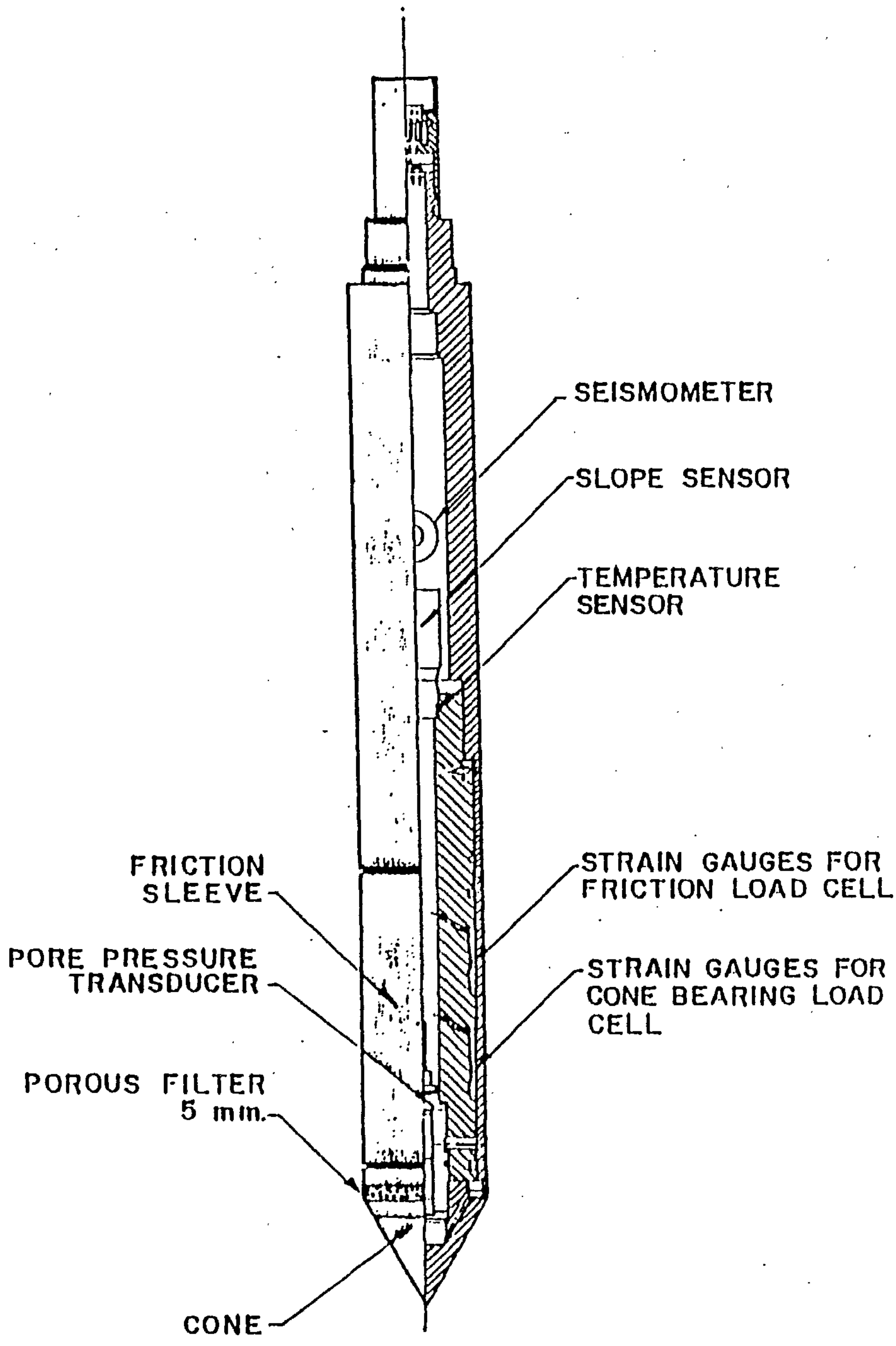
ConeTec Investigations Ltd.

Per:

Michael Davies, M.A.Sc.

Per: for

David Woeller, M.Eng., P.Eng.
President



SEISMIC CONE PENETROMETER

PLACER DOME INC.

CONTRACTOR: ConeTec — SITE: DOME 600-1

DATE: 8 MARCH 1988 — CONE: HT 192 STD 5 MM

Page No: 1 / 1 — JOB # 88-106

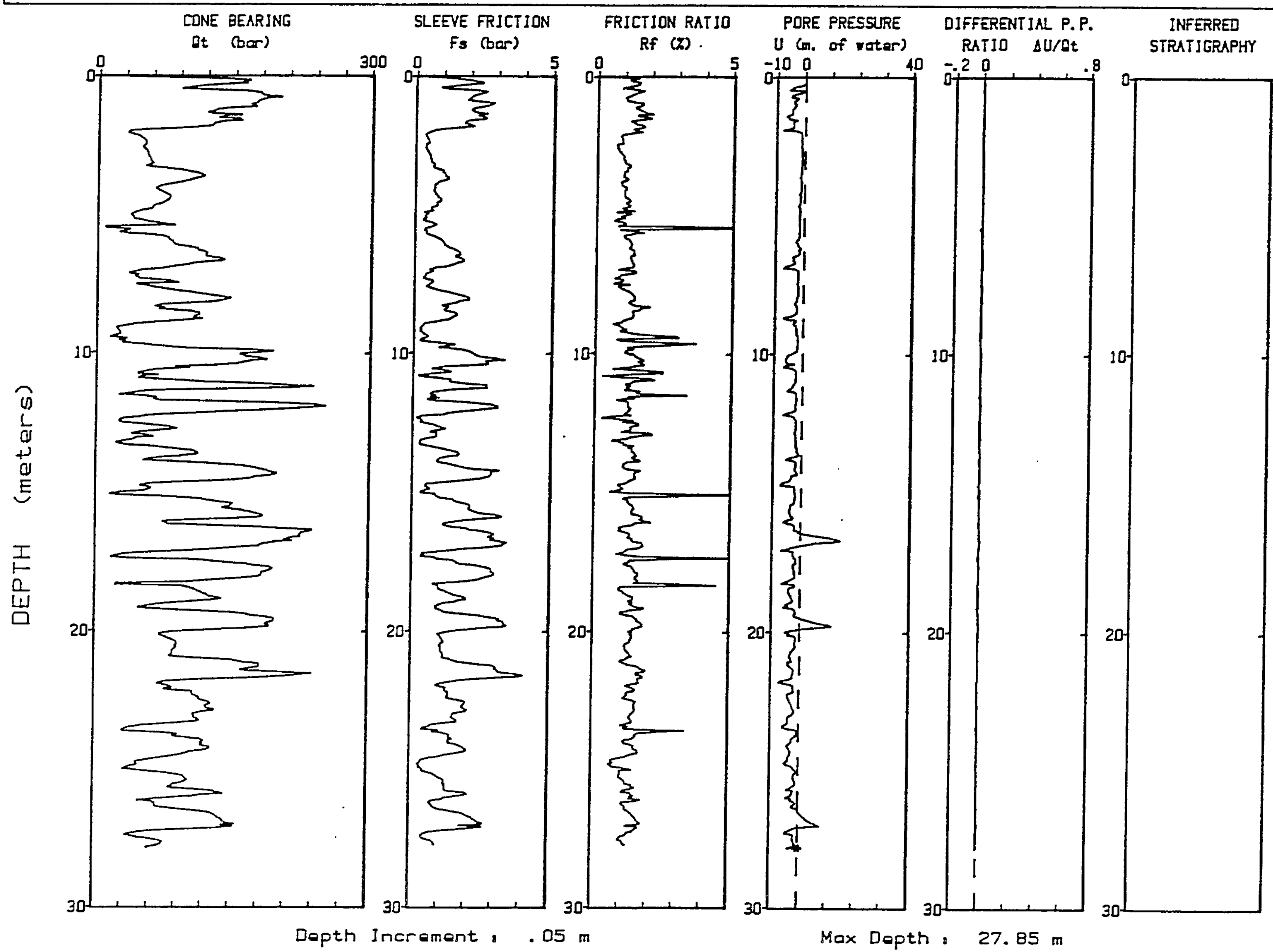


Figure 2.

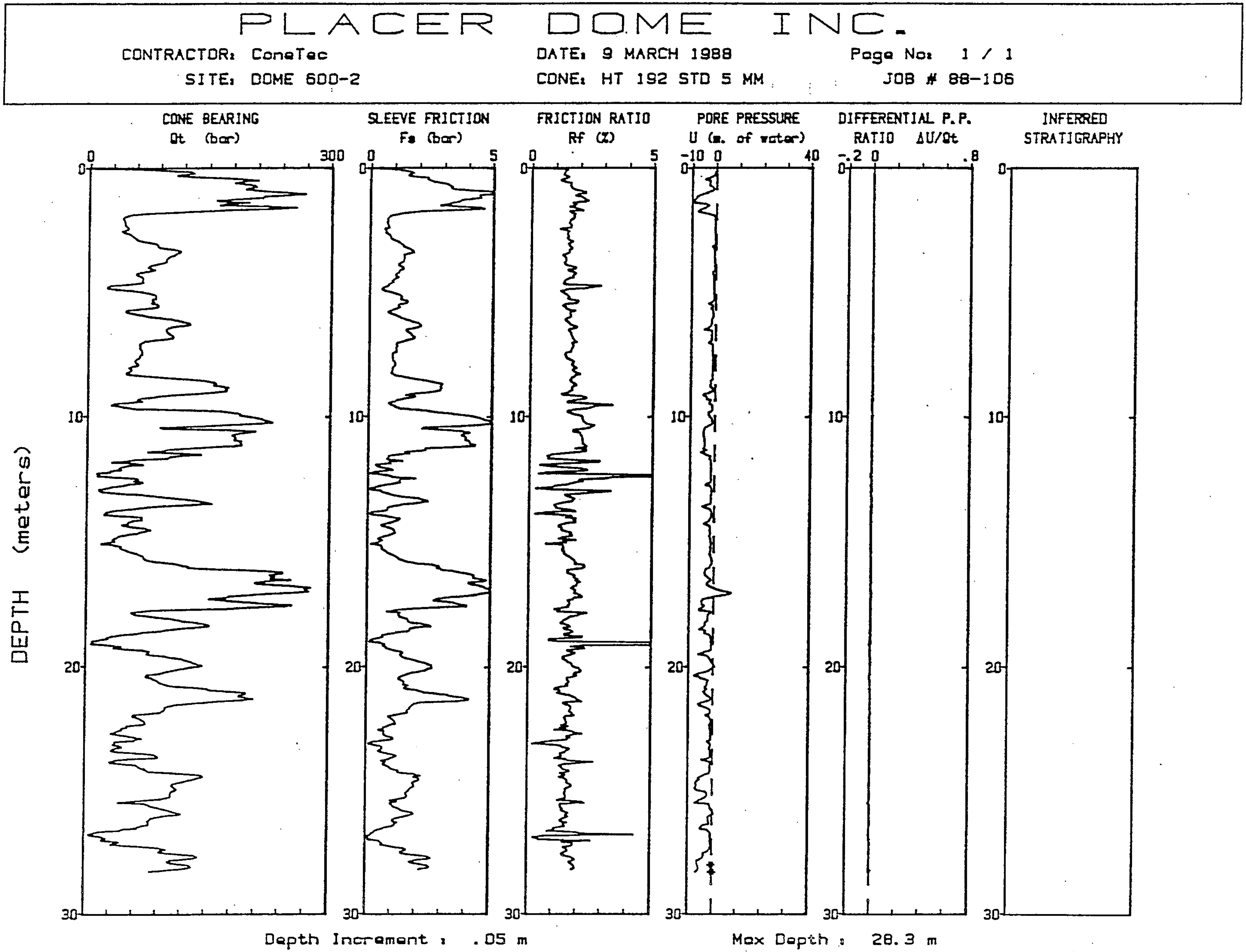
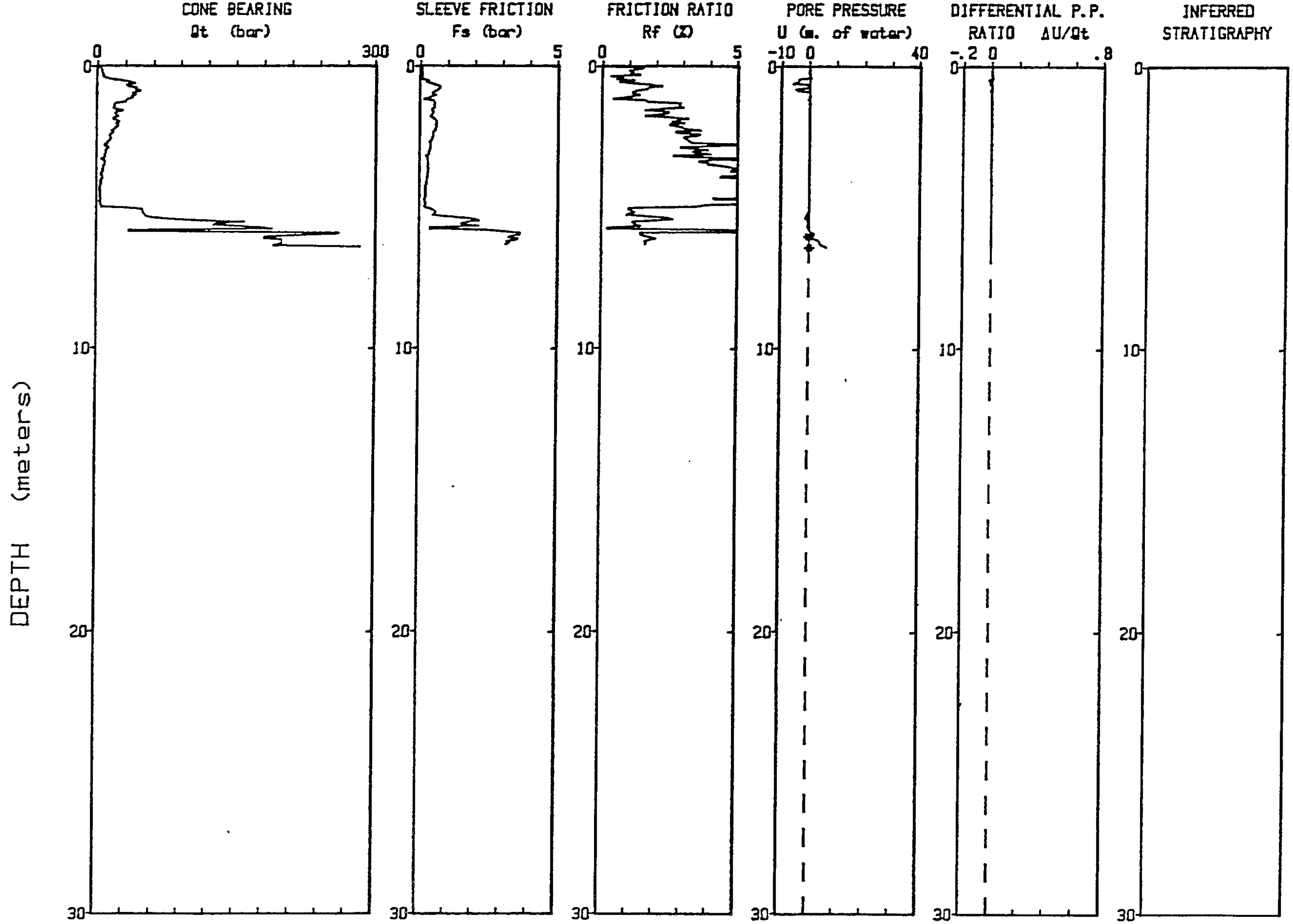

PLACER DOME INC.
CONTRACTOR: ConeTec
SITE: DOME 600-2
DATE: 9 MARCH 1988
CONE: HT 192 STD 5 MM
Page No: 1 / 1
JOB # 88-106
CONE BEARING
Qt (bar)
SLEEVE FRICTION
Fs (bar)
FRICTION RATIO
Rf (%)
PORE PRESSURE
U (m. of water)
DIFFERENTIAL P.P.
RATIO ΔU/Qt
INFERRED
STRATIGRAPHY
DEPTH (meters)
Depth Increment : .05 m
Max Depth : 28.3 m

Figure 3.

PLACER DOME INC.

CONTRACTOR: ConeTec | DATE: 9 MARCH 1988 | Page No: 1 / 1

SITE: DOME 700-1 | CONE: HT 192 STD 5 MM | JOB # 88-106



Depth Increment : .05 m

Max Depth : 6.4 m

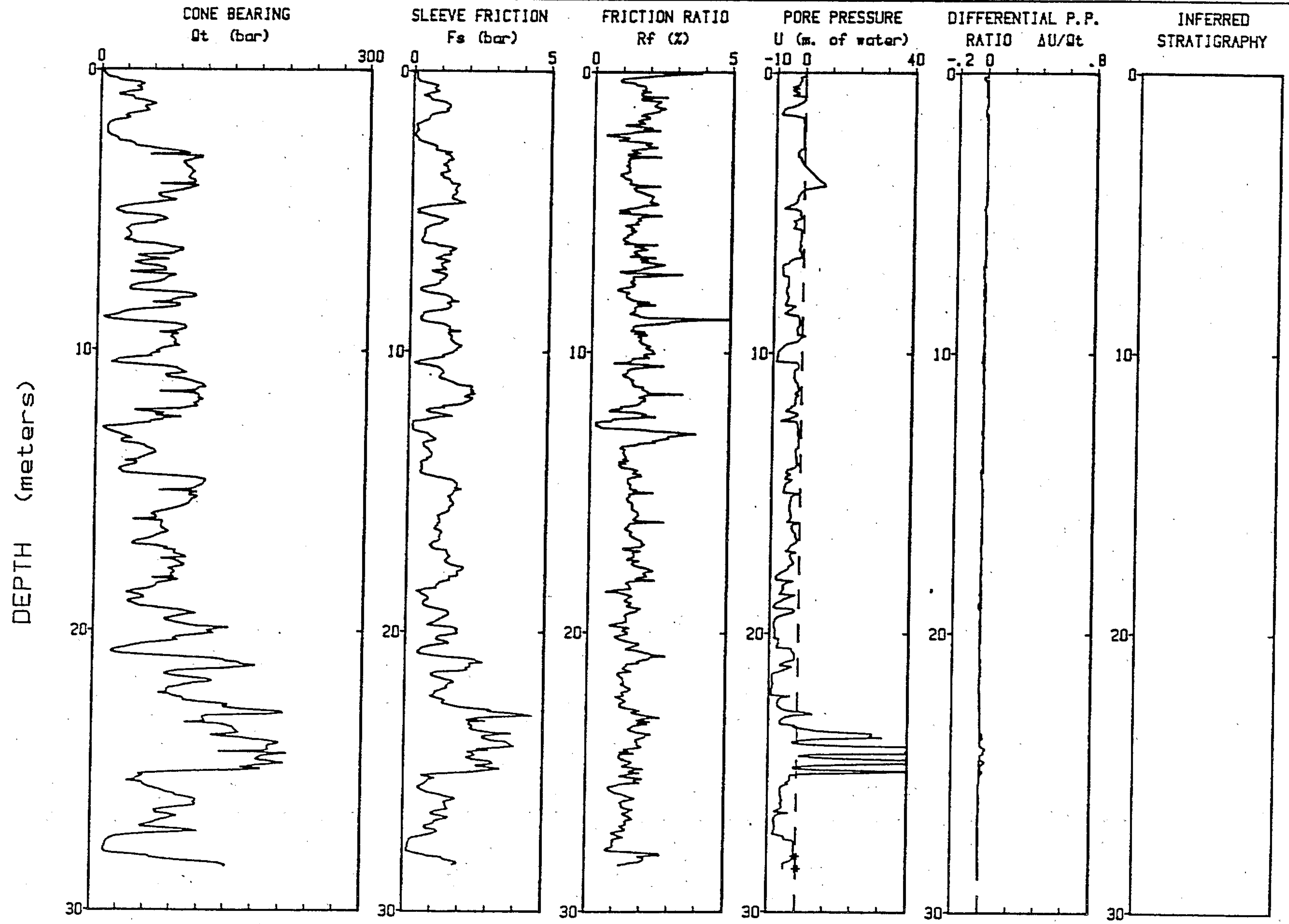
Figure 4.

PLACER DOME INC.

CONTRACTOR: ConeTec
SITE: DOME 700-2

DATE: 10 MARCH 1988
CONE: HT 192 STD 5 MM

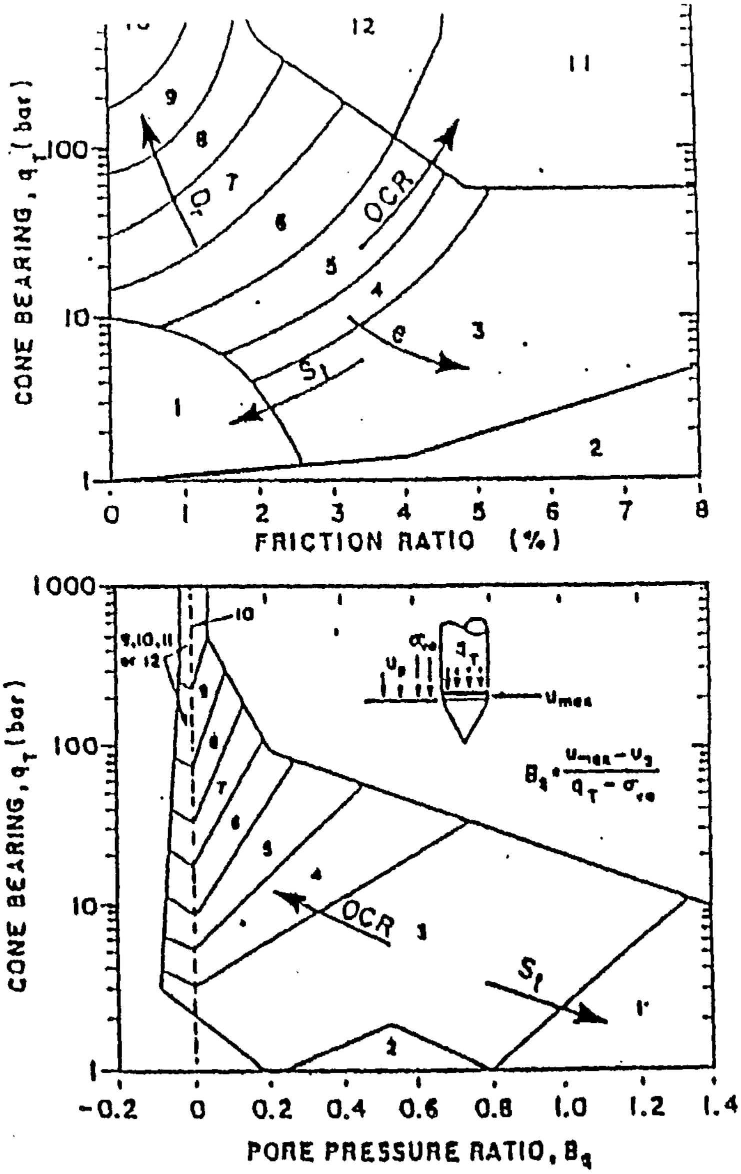
Page No: 1 / 1
JOB # 88-106



Depth Increment : .05 m

Max Depth : 28.45 m

Figure 5.



Zone	Soil Behaviour Type
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained*
12	sand to clayey sand*

* overconsolidated or cemented.

Proposed soil behaviour type classification system from CPTU data.(Robertson et. al,1986)

Figure 6.

PLACER DOME INC.

Contractor ConeTec
On Site Loc: DOME 600-1
Comments : 88-106 xxxxxx
Tot. Unit Wt. (avg) : 19 kPa

CPT Date : 8 MARCH 1988
Cone Used : HT 192 STD 5 MM
Water table (meters) : 28.5

DEPTH meters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	SPT N1	CSR
0.50	1.64	125.98	1.68	1.33	4.75	sand to silty sand	>90	>48	31	>50	>0.5
1.00	3.28	178.36	2.28	1.28	14.24	sand to silty sand	>90	>48	45	>50	>0.5
1.50	4.92	141.37	2.30	1.63	23.74	sand to silty sand	>90	46-48	35	>50	>0.5
2.00	6.56	109.34	1.59	1.45	33.24	sand to silty sand	80-90	44-46	27	47	>0.5
2.50	8.20	45.75	0.39	0.86	42.73	silty sand to sandy silt	50-60	38-40	15	23	.34x
3.00	9.84	52.83	0.45	0.86	52.23	sand to silty sand	50-60	38-40	13	18	.19
3.50	11.48	72.18	0.76	1.05	61.73	sand to silty sand	60-70	38-40	18	23	.24
4.00	13.12	94.28	0.92	0.97	71.22	sand to silty sand	70-80	38-40	24	28	.30
4.50	14.76	72.77	0.77	1.06	80.72	sand to silty sand	60-70	36-38	18	20	.21
5.00	16.40	54.13	0.58	1.08	90.21	silty sand to sandy silt	50-60	34-36	18	19	.28x
5.50	18.04	45.30	0.47	1.04	99.71	silty sand to sandy silt	40-50	32-34	15	15	.24x
6.00	19.69	56.79	0.65	1.15	109.21	silty sand to sandy silt	40-50	34-36	19	18	.27x
6.50	21.33	105.71	1.37	1.30	118.70	sand to silty sand	60-70	36-38	26	24	.25
7.00	22.97	106.85	1.33	1.25	128.20	sand to silty sand	60-70	36-38	27	23	.25
7.50	24.61	52.90	0.49	0.93	137.70	silty sand to sandy silt	40-50	30-32	18	15	.23x
8.00	26.25	88.62	1.08	1.22	147.19	sand to silty sand	50-60	34-36	22	18	.19
8.50	27.89	99.65	1.39	1.40	156.69	sand to silty sand	60-70	34-36	25	20	.21
9.00	29.53	93.38	1.09	1.16	166.18	sand to silty sand	50-60	34-36	23	18	.19
9.50	31.17	23.06	0.32	1.37	175.68	sandy silt to clayey silt	UNDFND	UNDFD	9	7	UNDF
10.00	32.81	90.85	1.27	1.40	185.18	sand to silty sand	50-60	32-34	23	17	.17
10.50	34.45	154.41	2.44	1.58	194.67	sand to silty sand	70-80	36-38	39	27	.30
11.00	36.09	63.19	0.90	1.42	204.17	silty sand to sandy silt	40-50	30-32	21	15	.23x
11.50	37.73	126.89	1.63	1.28	213.67	sand to silty sand	60-70	34-36	32	21	.22
12.00	39.37	154.52	1.89	1.22	223.16	sand to silty sand	60-70	34-36	39	26	.27
12.50	41.01	72.50	0.78	1.08	232.66	sand to silty sand	40-50	30-32	18	12	.12
13.00	42.65	62.15	0.87	1.40	242.16	silty sand to sandy silt	40-50	<30	21	13	.22x
13.50	44.29	52.66	0.65	1.23	251.65	silty sand to sandy silt	<40	<30	18	11	.19x
14.00	45.93	93.89	1.27	1.35	261.15	sand to silty sand	50-60	30-32	23	14	.15
14.50	47.57	182.09	2.55	1.40	270.64	sand to silty sand	70-80	34-36	46	27	.30
15.00	49.21	71.32	0.76	1.06	280.14	sand to silty sand	40-50	<30	18	11	.11
15.50	50.85	104.37	1.47	1.41	289.64	sand to silty sand	50-60	30-32	26	15	.16
16.00	52.49	159.88	2.53	1.58	299.13	sand to silty sand	60-70	34-36	40	23	.24
16.50	54.13	164.91	1.94	1.17	308.63	sand to silty sand	60-70	34-36	41	23	.24
17.00	55.77	197.84	3.05	1.54	318.13	sand to silty sand	70-80	34-36	49	27	.30
17.50	57.41	71.97	0.99	1.37	327.62	silty sand to sandy silt	40-50	<30	24	13	.22x
18.00	59.06	180.84	2.66	1.47	337.12	sand to silty sand	60-70	34-36	45	24	.26
18.50	60.70	106.52	1.59	1.49	346.61	sand to silty sand	50-60	30-32	27	14	.15
19.00	62.34	119.64	1.58	1.32	356.11	sand to silty sand	50-60	30-32	30	16	.16

Dr - All sands (Jamiolkowski et al. 1985) PHI - Durgunoglu and Mitchell 1975 CSR: Seed et al. 1983 - M=7.5

x - Seed's correction of 7.5 blows/foot has been applied to N1

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

Figure 7.

DEPTH eters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	SPT N1	CSR
19.50	63.98	112.01	1.51	1.35	365.61	sand to silty sand	50-60	30-32	28	15	.15
20.00	65.62	176.90	2.92	1.65	375.10	sand to silty sand	60-70	32-34	44	23	.24
20.50	67.26	82.08	1.10	1.34	384.60	sand to silty sand	40-50	<30	21	10	.11
21.00	68.90	89.38	1.15	1.28	394.10	sand to silty sand	40-50	<30	22	11	.11
21.50	70.54	173.38	2.66	1.53	403.59	sand to silty sand	60-70	32-34	43	21	.22
22.00	72.18	137.66	2.29	1.67	413.09	sand to silty sand	50-60	30-32	34	17	.17
22.50	73.82	103.42	1.38	1.34	422.58	sand to silty sand	40-50	<30	26	12	.13
23.00	75.46	124.52	1.92	1.54	432.08	sand to silty sand	50-60	<30	31	15	.15
23.50	77.10	92.77	1.26	1.35	441.58	sand to silty sand	40-50	<30	23	11	.11
24.00	78.74	78.83	1.21	1.53	451.07	silty sand to sandy silt	<40	<30	26	12	.21x
24.50	80.38	115.30	1.75	1.52	460.57	sand to silty sand	40-50	<30	29	13	.14
25.00	82.02	51.69	0.47	0.91	470.07	silty sand to sandy silt	<40	<30	17	8	.16x
25.50	83.66	88.31	0.99	1.12	479.56	sand to silty sand	40-50	<30	22	10	.10
26.00	85.30	110.24	1.55	1.40	489.06	sand to silty sand	40-50	<30	28	12	.13
26.50	86.94	73.05	0.97	1.33	498.55	silty sand to sandy silt	<40	<30	24	11	.19x
27.00	88.58	138.72	2.28	1.64	508.05	sand to silty sand	50-60	<30	35	15	.16
27.50	90.22	83.03	1.21	1.46	517.55	silty sand to sandy silt	<40	<30	28	12	.20x

Dr - All sands (Jamiolkovski et al. 1985) PHI - Durgunoglu and Mitchell 1975 CSR: Seed et al. 1983 - M=7.5

x - Seed's correction of 7.5 blows/foot has been applied to N1

*** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

Figure 7. cont.

PLACER DOME INC.

Contractor ConeTec
On Site Loc:DOME 600-2
Comments :88-106
Tot. Unit Wt. (avg) : 19 kPa

CPT Date :9 MARCH 1988
Cone Used :HT 192 STD 5 MM
Water table (meters) : 28.5

DEPTH eters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	SPT N1	CSR
0.50	1.64	133.84	1.76	1.32	4.75	sand to silty sand	>90	>48	33	>50	>0.5
1.00	3.28	204.75	3.63	1.77	14.24	sand to silty sand	>90	>48	>50	>50	>0.5
1.50	4.92	198.58	3.88	1.95	23.74	sand to silty sand	>90	>48	50	>50	>0.5
2.00	6.56	133.03	2.35	1.77	33.24	sand to silty sand	>90	44-46	33	>50	>0.5
2.50	8.20	45.22	0.69	1.52	42.73	silty sand to sandy silt	50-60	38-40	15	23	.34x
3.00	9.84	55.95	0.86	1.53	52.23	silty sand to sandy silt	50-60	38-40	19	26	.40x
3.50	11.48	99.68	1.51	1.51	61.73	sand to silty sand	70-80	40-42	25	31	.36
4.00	13.12	89.97	1.30	1.44	71.22	sand to silty sand	60-70	38-40	22	26	.28
4.50	14.76	68.42	1.12	1.64	80.72	silty sand to sandy silt	50-60	36-38	23	25	.39x
5.00	16.40	47.28	0.71	1.51	90.21	silty sand to sandy silt	40-50	34-36	16	16	.25x
5.50	18.04	81.72	1.31	1.60	99.71	silty sand to sandy silt	60-70	36-38	27	27	.46x
6.00	19.69	66.60	1.01	1.52	109.21	silty sand to sandy silt	50-60	34-36	22	21	.31x
6.50	21.33	110.01	1.76	1.60	118.70	sand to silty sand	60-70	36-38	28	25	.27
7.00	22.97	95.97	1.62	1.69	128.20	silty sand to sandy silt	60-70	36-38	32	28	>0.5
7.50	24.61	65.63	1.06	1.62	137.70	silty sand to sandy silt	50-60	32-34	22	18	.28x
8.00	26.25	60.40	0.98	1.63	147.19	silty sand to sandy silt	40-50	32-34	20	16	.25x
8.50	27.89	64.64	1.14	1.77	156.69	silty sand to sandy silt	40-50	32-34	22	17	.26x
9.00	29.53	161.84	2.76	1.71	166.18	sand to silty sand	70-80	38-40	40	31	.35
9.50	31.17	83.54	1.43	1.71	175.68	silty sand to sandy silt	50-60	32-34	28	21	.31x
10.00	32.81	121.23	2.31	1.91	185.18	silty sand to sandy silt	60-70	34-36	40	29	>0.5
10.50	34.45	181.65	4.16	2.29	194.67	silty sand to sandy silt	70-80	36-38	>50	43	>0.5
11.00	36.09	186.64	3.82	2.05	204.17	sand to silty sand	70-80	36-38	47	32	.38x
11.50	37.73	141.56	3.01	2.12	213.67	silty sand to sandy silt	60-70	34-36	47	32	>0.5
12.00	39.37	77.19	0.82	1.07	223.16	sand to silty sand	40-50	30-32	19	13	.13x
12.50	41.01	30.86	0.77	2.50	232.66	sandy silt to clayey silt	UNDFND	UNDFD	12	8	UNDF
13.00	42.65	42.82	0.71	1.65	242.16	silty sand to sandy silt	<40	<30	14	9	.17x
13.50	44.29	99.19	1.61	1.62	251.65	silty sand to sandy silt	50-60	32-34	33	21	.30x
14.00	45.93	54.69	0.64	1.18	261.15	silty sand to sandy silt	<40	<30	18	11	.19x
14.50	47.57	59.80	0.94	1.57	270.64	silty sand to sandy silt	<40	<30	20	12	.20x
15.00	49.21	50.47	0.78	1.55	280.14	silty sand to sandy silt	<40	<30	17	10	.18x
15.50	50.85	42.62	0.59	1.39	289.64	silty sand to sandy silt	<40	<30	14	8	.16x
16.00	52.49	89.84	1.74	1.94	299.13	silty sand to sandy silt	40-50	30-32	30	17	.26x
16.50	54.13	210.06	3.79	1.81	308.63	sand to silty sand	70-80	34-36	>50	30	.33
17.00	55.77	248.76	4.73	1.90	318.13	sand to silty sand	70-80	36-38	>50	35	.46
17.50	57.41	197.41	3.65	1.85	327.62	sand to silty sand	60-70	34-36	49	27	.29
18.00	59.06	129.51	1.99	1.53	337.12	sand to silty sand	50-60	30-32	32	17	.18
18.50	60.70	128.30	1.94	1.51	346.61	sand to silty sand	50-60	30-32	32	17	.18
19.00	62.34	45.42	0.75	1.66	356.11	silty sand to sandy silt	<40	<30	15	8	.16x

Dr - All sands (Jamiolkowski et al. 1985) PHI - Durgunoglu and Mitchell 1975 CSR: Seed et al. 1983 - M=7.5

x - Seed's correction of 7.5 blows/foot has been applied to N1

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

Figure 8.

PLACER DOME INC.

ntractor ConeTec On Site Loc:DOME 600-2 Page No. 2

DEPTH (ers)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	SPT N1	CSR
19.50	63.98	47.05	0.98	2.09	365.61	sandy silt to clayey silt	UNDFND	UNDFD	19	10	UNDF
20.00	65.62	123.74	2.05	1.65	375.10	sand to silty sand	50-60	30-32	31	16	.16
20.50	67.26	95.56	1.86	1.95	384.60	silty sand to sandy silt	40-50	<30	32	16	.25x
21.00	68.90	121.01	1.68	1.39	394.10	sand to silty sand	50-60	30-32	30	15	.16
21.50	70.54	182.49	3.20	1.75	403.59	sand to silty sand	60-70	32-34	46	22	.24
22.00	72.18	89.58	1.56	1.74	413.09	silty sand to sandy silt	40-50	<30	30	15	.23x
22.50	73.82	64.65	0.98	1.52	422.58	silty sand to sandy silt	<40	<30	22	10	.19x
23.00	75.46	50.44	0.80	1.59	432.08	silty sand to sandy silt	<40	<30	17	8	.16x
23.50	77.10	40.47	0.50	1.24	441.58	silty sand to sandy silt	<40	<30	13	6	.14x
24.00	78.74	62.16	0.90	1.46	451.07	silty sand to sandy silt	<40	<30	21	10	.18x
24.50	80.38	104.19	1.54	1.48	460.57	sand to silty sand	40-50	<30	26	12	.12
25.00	82.02	117.51	1.93	1.64	470.07	sand to silty sand	40-50	<30	29	13	.14
25.50	83.66	91.42	1.44	1.58	479.56	silty sand to sandy silt	40-50	<30	30	14	.22x
26.00	85.30	99.63	1.48	1.48	489.06	sand to silty sand	40-50	<30	25	11	.11
26.50	86.94	80.49	1.19	1.48	498.55	silty sand to sandy silt	<40	<30	27	12	.20x
27.00	88.58	26.23	0.26	1.00	508.05	silty sand to sandy silt	<40	<30	9	4	.12x
27.50	90.22	64.87	1.08	1.66	517.55	silty sand to sandy silt	<40	<30	22	9	.18x
28.00	91.86	120.52	2.10	1.75	527.04	silty sand to sandy silt	40-50	<30	40	17	.26x

Dr - All sands (Jamiolkowski et al. 1985) PHI - Durgunoglu and Mitchell 1975 CSR: Seed et al. 1983 - M=7.5

x - Seed's correction of 7.5 blows/foot has been applied to N1

** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

Figure 8. cont.

PLACER DOME INC.

Contractor ConeTec
On Site Loc:DOME 700-1
Comments :88-106
Tot. Unit Wt. (avg) : 19 kPa

CPT Date :9 MARCH 1988
Cone Used :HT 192 STD 5 MM
Water table (meters) : 6.5

DEPTH (meters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL-BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	SPT N1	CSR
0.50	1.64	8.94	0.09	0.96	4.75	clayey silt to silty clay	UNDFND	UNDFD	4	20	UNDF
1.00	3.28	38.22	0.52	1.37	14.24	silty sand to sandy silt	60-70	42-44	13	33	>0.5
1.50	4.92	25.31	0.43	1.69	23.74	sandy silt to clayey silt	UNDFND	UNDFD	10	21	UNDF
2.00	6.56	20.41	0.48	2.37	33.24	clayey silt to silty clay	UNDFND	UNDFD	10	18	UNDF
2.50	8.20	16.53	0.50	3.01	42.73	clayey silt to silty clay	UNDFND	UNDFD	8	13	UNDF
3.00	9.84	10.13	0.35	3.49	52.23	silty clay to clay	UNDFND	UNDFD	7	9	UNDF
3.50	11.48	7.24	0.28	3.83	61.73	clay	UNDFND	UNDFD	7	9	UNDF
4.00	13.12	5.00	0.26	5.26	71.22	clay	UNDFND	UNDFD	5	6	UNDF
4.50	14.76	3.06	0.20	6.47	80.72	clay	UNDFND	UNDFD	3	3	UNDF
5.00	16.40	6.37	0.19	3.01	90.21	clay	UNDFND	UNDFD	6	7	UNDF
5.50	18.04	70.54	1.09	1.55	99.71	silty sand to sandy silt	50-60	36-38	24	23	.35x
6.00	19.69	156.38	2.20	1.41	109.21	sand to silty sand	70-80	40-42	39	37	>0.5

Dr - All sands (Jamiolkowski et al. 1985) PHI - Durgunoglu and Mitchell 1975 CSR: Seed et al. 1983 - M=7.5

x - Seed's correction of 7.5 blows/foot has been applied to N1

*** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

Figure 9.

PLACER DOME INC.

Contractor ConeTec
On Site Loc:DOME 700-2
Comments :88-106
Tot. Unit Wt. (avg) : 19 kPa

CPT Date :10 MARCH 1988
Cone Used :HT 192 STD 5 MM
Water table (meters) : 29

DEPTH eters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	SPT N1	CSR
0.50	1.64	18.74	0.28	1.49	4.75	sandy silt to clayey silt	UNDFND	UNDFD	7	34	UNDF
1.00	3.28	36.18	0.66	1.81	14.24	sandy silt to clayey silt	UNDFND	UNDFD	14	38	UNDF
1.50	4.92	51.82	1.16	2.23	23.74	sandy silt to clayey silt	UNDFND	UNDFD	21	42	UNDF
2.00	6.56	20.84	0.36	1.73	33.24	sandy silt to clayey silt	UNDFND	UNDFD	8	14	UNDF
2.50	8.20	11.52	0.14	1.21	42.73	sandy silt to clayey silt	UNDFND	UNDFD	5	7	UNDF
3.00	9.84	58.97	0.92	1.57	52.23	silty sand to sandy silt	60-70	38-40	20	27	.45x
3.50	11.48	97.30	1.27	1.31	61.73	sand to silty sand	70-80	40-42	24	31	.34
4.00	13.12	103.25	1.22	1.18	71.22	sand to silty sand	70-80	40-42	26	30	.34
4.50	14.76	87.77	1.56	1.77	80.72	silty sand to sandy silt	60-70	38-40	29	32	>0.5
5.00	16.40	54.55	1.06	1.95	90.21	silty sand to sandy silt	50-60	34-36	18	19	.28x
5.50	18.04	54.30	0.86	1.58	99.71	silty sand to sandy silt	40-50	34-36	18	18	.27x
6.00	19.69	35.42	0.46	1.30	109.21	silty sand to sandy silt	<40	30-32	12	11	.19x
6.50	21.33	63.99	1.03	1.61	118.70	silty sand to sandy silt	50-60	34-36	21	19	.29x
7.00	22.97	60.04	1.14	1.90	128.20	silty sand to sandy silt	40-50	32-34	20	18	.27x
7.50	24.61	70.31	1.12	1.59	137.70	silty sand to sandy silt	50-60	32-34	23	20	.29x
8.00	26.25	56.14	0.71	1.27	147.19	silty sand to sandy silt	40-50	30-32	19	15	.24x
8.50	27.89	90.90	1.43	1.58	156.69	silty sand to sandy silt	50-60	34-36	30	24	.36x
9.00	29.53	29.24	0.57	1.93	166.18	sandy silt to clayey silt	UNDFND	UNDFD	12	9	UNDF
9.50	31.17	87.25	1.43	1.64	175.68	silty sand to sandy silt	50-60	32-34	29	22	.32x
10.00	32.81	83.37	1.63	1.96	185.18	silty sand to sandy silt	50-60	32-34	28	20	.30x
10.50	34.45	44.29	0.82	1.84	194.67	silty sand to sandy silt	<40	<30	15	10	.19x
11.00	36.09	84.14	1.14	1.35	204.17	sand to silty sand	50-60	32-34	21	15	.15
11.50	37.73	108.17	2.01	1.86	213.67	silty sand to sandy silt	50-60	32-34	36	24	.37x
12.00	39.37	113.44	1.88	1.66	223.16	sand to silty sand	50-60	32-34	28	19	.20
12.50	41.01	74.26	0.83	1.12	232.66	sand to silty sand	40-50	30-32	19	12	.12
13.00	42.65	28.92	0.34	1.17	242.16	silty sand to sandy silt	<40	<30	10	6	.14x
13.50	44.29	41.98	0.70	1.67	251.65	silty sand to sandy silt	<40	<30	14	9	.17x
14.00	45.93	54.96	0.67	1.22	261.15	silty sand to sandy silt	<40	<30	18	11	.20x
14.50	47.57	43.05	0.63	1.47	270.64	silty sand to sandy silt	<40	<30	14	9	.17x
15.00	49.21	112.37	1.71	1.52	280.14	sand to silty sand	50-60	32-34	28	17	.17
15.50	50.85	107.84	1.56	1.45	289.64	sand to silty sand	50-60	30-32	27	16	.16
16.00	52.49	76.71	1.28	1.67	299.13	silty sand to sandy silt	40-50	<30	26	15	.23x
16.50	54.13	73.59	1.10	1.50	308.63	silty sand to sandy silt	40-50	<30	25	14	.22x
17.00	55.77	58.13	1.00	1.72	318.13	silty sand to sandy silt	<40	<30	19	11	.19x
17.50	57.41	89.43	1.40	1.57	327.62	silty sand to sandy silt	40-50	<30	30	16	.25x
18.00	59.06	91.88	1.81	1.97	337.12	silty sand to sandy silt	40-50	<30	31	17	.25x
18.50	60.70	76.91	1.20	1.56	346.61	silty sand to sandy silt	40-50	<30	26	14	.22x
19.00	62.34	46.64	0.61	1.31	356.11	silty sand to sandy silt	<40	<30	16	8	.16x

Dr - All sands (Jamiolkovski et al. 1985) PHI - Durgunoglu and Mitchell 1975 CSR: Seed et al. 1983 - M=7.5

x - Seed's correction of 7.5 blows/foot has been applied to N1

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

Figure 10.

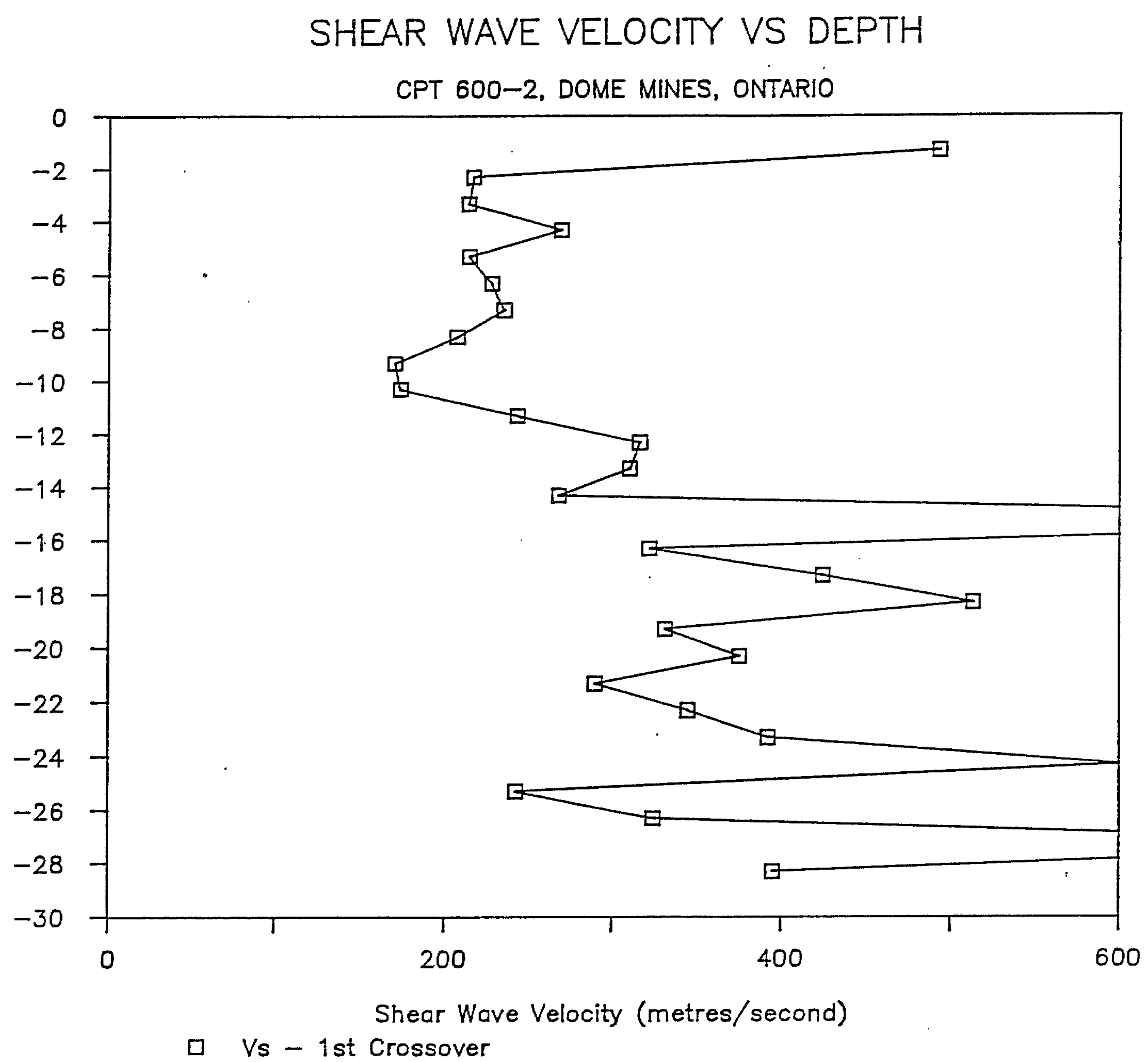
DEPTH eters)	(feet)	Qc (avg) (bar)	Fs (avg) (bar)	Rf (avg) (%)	SIGV' (kPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	SPT N1	CSR
19.50	63.98	82.09	1.24	1.51	365.61	silty sand to sandy silt	40-50	<30	27	14	.23x
20.00	65.62	107.05	1.33	1.24	375.10	sand to silty sand	50-60	<30	27	14	.14
20.50	67.26	97.10	1.32	1.36	384.60	sand to silty sand	40-50	<30	24	12	.13
21.00	68.90	49.49	1.01	2.04	394.10	sandy silt to clayey silt	UNDFND	UNDFD	20	10	UNDF
21.50	70.54	142.95	2.07	1.45	403.59	sand to silty sand	50-60	30-32	36	18	.18
22.00	72.18	107.60	1.42	1.32	413.09	sand to silty sand	40-50	<30	27	13	.13
22.50	73.82	89.49	1.18	1.31	422.58	sand to silty sand	40-50	<30	22	11	.11
23.00	75.46	166.36	2.81	1.69	432.08	sand to silty sand	60-70	30-32	42	20	.21
23.50	77.10	134.21	2.53	1.89	441.58	silty sand to sandy silt	50-60	30-32	45	21	.31x
24.00	78.74	167.28	3.23	1.93	451.07	sand to silty sand	60-70	30-32	42	19	.20
24.50	80.38	193.07	2.90	1.50	460.57	sand to silty sand	60-70	32-34	48	22	.23
25.00	82.02	187.35	2.93	1.56	470.07	sand to silty sand	60-70	32-34	47	21	.22
25.50	83.66	60.79	0.97	1.60	479.56	silty sand to sandy silt	<40	<30	20	9	.17x
26.00	85.30	84.65	1.10	1.30	489.06	sand to silty sand	<40	<30	21	9	.10
26.50	86.94	100.86	1.41	1.39	498.55	sand to silty sand	40-50	<30	25	11	.11
27.00	88.58	77.42	1.15	1.49	508.05	silty sand to sandy silt	<40	<30	26	11	.20x
27.50	90.22	63.34	0.74	1.16	517.55	silty sand to sandy silt	<40	<30	21	9	.17x
28.00	91.86	16.81	0.25	1.48	527.04	sandy silt to clayey silt	UNDFND	UNDFD	7	3	UNDF

Dr - All sands (Jamiolkowski et al. 1985) PHI - Durgunoglu and Mitchell 1975 CSR: Seed et al. 1983 - M=7.5

x - Seed's correction of 7.5 blows/foot has been applied to N1

*** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

Figure 10. cont.



SHEAR WAVE VELOCITY VS DEPTH
CPT 600–2, DOME MINES, ONTARIO
Depth (metres)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-28
-30
0
200
400
600
Shear Wave Velocity (metres/second)
Vs – 1st Crossover

Figure 11.

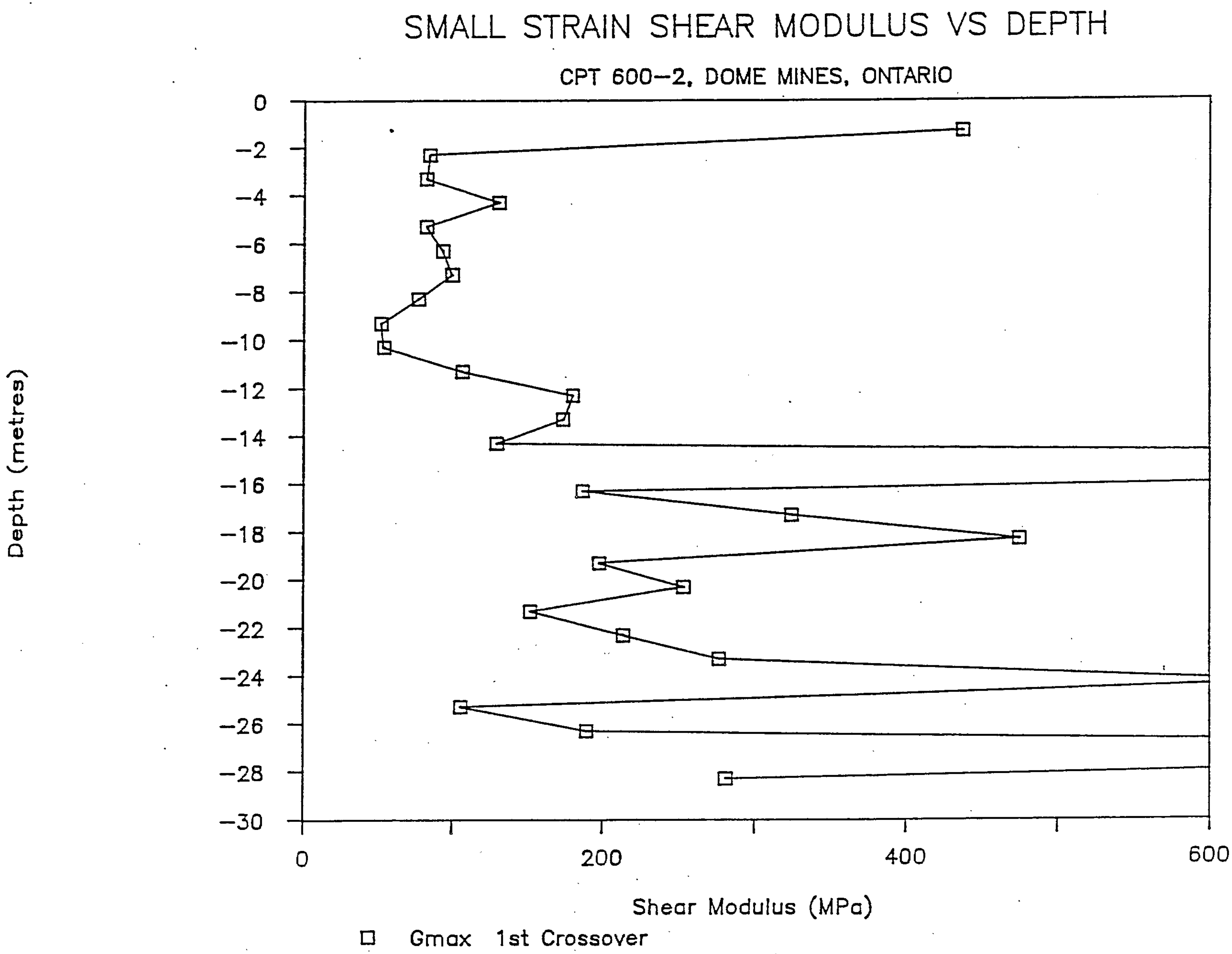

SMALL STRAIN SHEAR MODULUS VS DEPTH
CPT 600-2, DOME MINES, ONTARIO
Depth (metres)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-28
-30
0
200
400
600
Shear Modulus (MPa)
Gmax 1st Crossover

Figure 12.

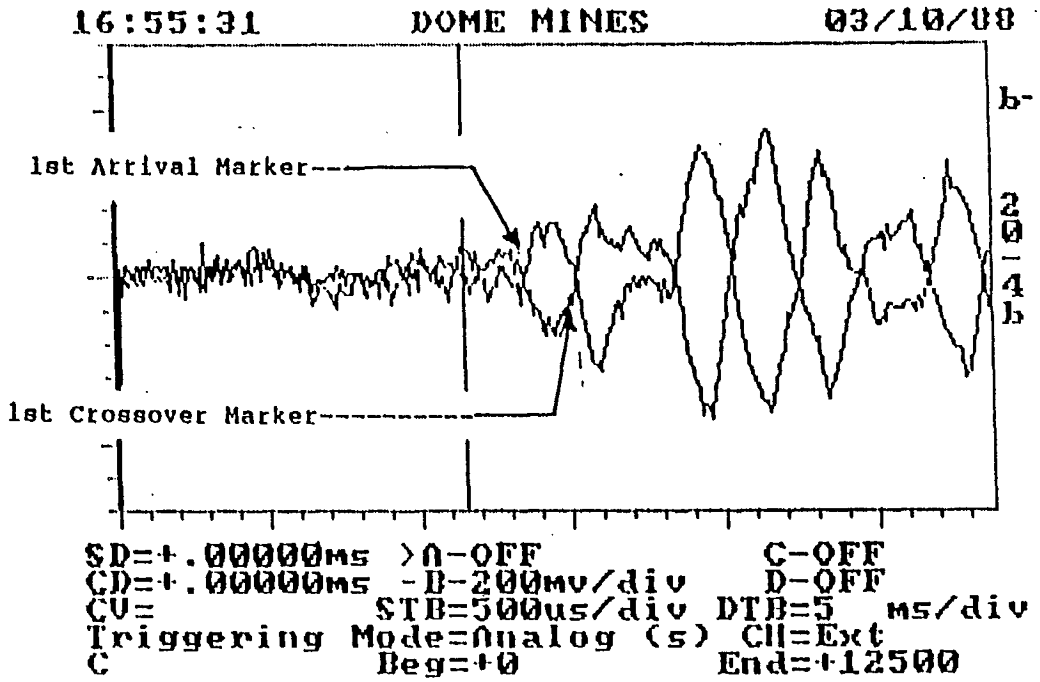

16:55:31
DOME MINES
03/10/88
1st Arrival Marker
1st Crossover Marker
SD=+.00000ms >A-OFF C-OFF
CD=+.00000ms -B-200mv/div D-OFF
CV= STB=500us/div DTB=5 ms/div
Triggering Mode=Analog (s) CH=Ext
C Beg=+0 End=+12500

Figure 13.

ConeTec Investigations Ltd.
Seismic Cone Data Reduction
March 10, 1988
Placer Dome Mines, Timmins Ontario 600 Level
Hole # 600-2
Shear Source Offset .92m/seismometer .20m above tip
Source Hammer HT 192

seismometer Depth (m)	Distance (m)	Arrival (msec)	Crossover (msec)	Corrected Arrival (msec)	Corrected Crossover (msec)	Average Depth of sensor (m)	Velocity from arrival	Velocity from crossover	Velocity Profile (m/sec)	Density (kg/m3)	Gmax Profile (MPa)	Average Depth of sensor (m)
1.3	3.546829	7.2	10.8	2.638976	3.958464	0.65	492.6152		492.62	1800	436.81	-0.65
2.3	4.022437	11.2	15	6.404077	8.576889	1.8	265.5971	216.5240	216.52	1800	84.39	-1.8
3.3	4.666904	14.5	18.75	10.25304	13.25825	2.8	259.8097	213.6130	213.61	1800	82.13	-2.8
4.3	5.420332	16.3	21.4	12.93094	16.97681	3.8	373.4278	268.9207	268.92	1800	130.17	-3.8
5.3	6.243396	19.4	25.5	16.46860	21.64687	4.8	282.6728	214.1303	214.13	1800	82.53	-4.8
6.3	7.111961	22.4	29.4	19.34262	26.04344	5.8	296.3817	227.4497	227.45	1800	93.12	-5.8
7.3	8.011242	26.7	33.25	24.32956	30.29804	6.8	222.8693	235.0396	235.04	1800	99.44	-6.8
8.3	8.931965	32	37.8	29.73589	35.12552	7.8	184.9681	207.1473	207.15	1800	77.24	-7.8
9.3	9.868130	35.3	43.5	33.26769	40.99560	8.8	283.1416	170.3554	170.36	1800	52.24	-8.8
10.3	10.81572	40	49.1	38.09266	46.75875	9.8	207.2551	173.5163	173.52	1800	54.19	-9.8
11.3	11.77200	43	53	41.27590	50.87495	10.8	314.1456	242.9423	242.94	1800	106.24	-10.8
12.3	12.73499	46	55.95	44.42877	54.03890	11.8	317.1719	316.0604	316.06	1800	179.81	-11.8
13.3	13.70328	48.6	59	47.16971	57.26364	12.8	364.8378	310.1027	310.10	1800	173.09	-12.8
14.3	14.67583	53.8	62.6	52.42224	60.99688	13.8	190.3843	267.8635	267.86	1800	129.15	-13.8
15.3	15.65183	56	63.5	54.74117	62.07258	14.8	431.2329	929.6294	929.63	1800	1555.58	-14.8
16.3	16.63069	57.2	66.5	56.06260	65.17767	15.8	756.7599	322.0520	322.05	1800	186.69	-15.8
17.3	17.61192	59.6	68.75	58.54441	67.53235	16.8	402.9312	424.6857	424.69	1800	324.64	-16.8
18.3	18.59516	61.1	70.6	60.13016	69.47936	17.8	630.6179	513.6080	513.61	1800	474.83	-17.8
19.3	19.58009	64.8	73.55	63.87304	72.49787	18.8	267.1739	331.2898	331.29	1800	197.56	-18.8
20.3	20.56647	67.55	76.15	66.67476	75.16333	19.8	356.9233	375.1696	375.17	1800	253.35	-19.8
21.3	21.55411	69.4	79.55	68.58179	78.61212	20.8	524.3760	289.9566	289.96	1800	151.33	-20.8
22.3	22.54284	72.4	82.4	71.62005	81.51232	21.8	329.1353	344.8035	344.80	1800	214.00	-21.8
23.3	23.53253	74.3	84.9	73.56582	84.06108	22.8	513.9354	392.3483	392.35	1800	277.09	-22.8
24.3	24.52305	77.85	86.5	77.14191	85.71323	23.8	279.6351	605.2706	605.27	1800	659.43	-23.8
25.3	25.51430	79.9	90.6	79.22887	89.83899	24.8	479.1657	242.3795	242.38	1800	105.75	-24.8
26.3	26.50622	84	93.65	83.34645	92.92137	25.8	242.8608	324.4246	324.42	1800	189.45	-25.8
27.3	27.49872	84.5	94.8	83.88933	94.11490	26.8	1842.027	837.8550	837.86	1800	1263.60	-26.8
28.3	28.49175	87.8	97.3	87.20909	96.64515	27.8	301.2268	395.2167	395.22	1800	281.15	-27.8

Table 1.

APPENDIX A

CPTINTR1 version 3.02

written by JAMES GREIG

CPTINTR1 is a highly interactive program to provide basic inter-etation of CONE PENETRATION TEST (CPT) data. It was written at the iversity of British Columbia, Vancouver Canada and is compatible th the data format of the Hogentogler & Co. Field Computer Systems. e interpretation methods used provide an estimate of the following il parameters (appropriate references are given below):

) Soil Behaviour Type: - Robertson and Campanella 1983

:) Equivalent Relative Density :

i) Ticicno Sand - Bellotti et al. 1985
ii) Hokksund Sand - Bellotti et al. 1985
iii) Ottawa / Hilton Mines Sands - Schmertmann 1976
iv) All Sands (average) - Jamiolkowski et al. 1985

I) Angle of Internal Friction

i) Robertson and Campanella 1983
ii) Durgunoglu and Mitchell 1975
iii) Janbu and Senneset 1974 - Beta=+15,0, and -15 degrees

1) Equivalent SPT N value - Robertson et al. 1983

5) Corrected SPT N1 value - N1=Cn*N (where Cn=SIG^(-.7) and SIG is in Kg/cm^2)

5) Cyclic Stress Ratio (CSR) to cause liquefaction (M=7.5) - Seed et al. (1983)

7) Undrained Shear Strength (Su) - Su=(Qc-SIGV)/Nk where SIGV is the total overburden stress

TABULATED DATA is based on values averaged over a a specified depth range and thus the influence of extreme values may be subdued. The ENGINEER should be using the tabulated output ONLY AS A GUIDE for the interpretation of the PLOTTED CPT DATA. It is important that the engineer fully understand the limitations of the CPT test.

IMPORTANT NOTE

c is believed that the calculations performed by the routine are one correctly and reliably. However, no resonsibility is assumed ' either the University of British Columbia or the author for any :rors that may occur with the use of this program. It is also eft up to the engineer to decide which, if any, of the correlation echniques are applicable to the specific soil conditions.

......references

REFERENCES

Bellotti, R., Crippa, V., Pedroni, S., Baldi, G., Fretti, C., Ostricati, D., Ghionna, V., Jamiolkowski, M., Pasqalini, E.,1985, "Laboratory Validation Of In-Situ Tests", Italian Geotechnical Society Jubilee Volume for the XI ICSMFE, San Francisco, Cal.

Durgunoglu, H.T. and Mitchell, J.K., 1975, "Static Penetration Resistance of Soils: I-Analysis", Proceedings of the ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Raleigh, North Carolina, Vol. I.

Janbu, N. and Senneset, K., 1974,. "Effective Stress Interpretation of In Situ Static Penetratuion Tests", Proceedings of the European Symposium on Penetration Testing, Stockholm Sweden, Vol. 2.2.

Jamiolkowski, M., Ladd, C.C., Germaine, J.T., Lancellotta, R., 1985,"New Developments in Field and Laboratory Testing of Soils", State of the Art Adress for XIth ICSMFE, San Francisco.

Robertson, P.K. and Campanella, R.G., 1983, "Interpretation of Cone Penetration Tests - PART I (SAND)", Canadian Geotechnical Journal, Vol. 20, No. 4.

Robertson, P.K., Campanella, R.G., and Wightman, A., 1983, "SPT - CPT Correlations", Journal of the Geotechnical Division, ASCE, Vol. 109, Nov.

Schmertmann, J.H., 1976, "An updated Correlation between Relative Density, Dr and Fugro-type Electric Cone Bearing, qc" Department of Civil Engineering Report, University of Florida, July.

Seed, H.B., Idriss, I.M. and Arango, I., 1983, "Evaluation of Liquefaction Potential Using Field Performance Data", Journal of Geotechnical Engineering Division, ASCE, Vol. 109, No. 3, March 1983, pp. 458-482.