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NAME OF AUTHOR........Jan De Jong................

TITLE OF THESIS...Foundation Displacements
...of Multi-Storey.........
...Structures..........................

UNIVERSITY.........Univ. of Alberta..........

DEGREE FOR WHICH THESIS WAS PRESENTED.....Ph. D.......

YEAR THIS DEGREE GRANTED..........1971..................

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THE UNIVERSITY OF ALBERTA

FOUNDATION DISPLACEMENTS OF MULTI-STOREY STRUCTURES

by

JAN DEJONG

A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
OF DOCTOR OF PHILOSOPHY

DEPARTMENT OF CIVIL ENGINEERING
EDMONTON, ALBERTA
SPRING, 1971

UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled FOUNDATION DISPLACEMENTS OF MULTI-STOREY STRUCTURES submitted by Jan DeJong in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

SR Sinclair
Prof. S.R. Sinclair - Supervisor

Peter F. Adams
Dr. P.F. Adams

JR Colbourne
Dr. J.R. Colbourne

N. Morgenstern
Dr. N.R. Morgenstern

S Thomson.
Dr. S. Thomson

J Burland
Dr. J.B. Burland - External Examiner

Date Nov. 24/70

ABSTRACT

This dissertation deals with two subject areas, the first of which is the observation and analysis of foundation displacements of over-consolidated soils. The second area is the interaction of structural frames with their foundation when subjected to differential settlements.

Settlements have been observed on four multi-storey structures. Two buildings are founded on over-consolidated sandy clay till, one on spread footings and the other on Franki end bearing piles. A comparative study of the two foundation types has shown their settlement behaviours to be identical and solely a function of soil deformation parameters. Analysis of the settlement data has shown that 75% of the 25 year settlement takes place during construction and can be estimated using elastic theory.

The two other buildings are founded on spread footings on a preglacial sand deposit which is underlain by a heavily over-consolidated interbedded sandstone and mudstone bedrock formation. The settlement response varies linearly with footing contact pressure during construction, indicative of elastic behaviour. Heave measurements of a 45 foot deep excavation at this site allowed complete overlap of rebound and subsequent settlement. Deformation moduli computed from heave measurements were consistent with those derived from settlement observations. The ratio of field to laboratory moduli proved to be large.

The structural analysis of one of the four multi-storey buildings by ICES-STRUDL has shown that the variation of structural rigidity during construction must be included in any interaction solution. Reactions as high as 44% of the self-weight footing load were computed for the observed differential displacements.

A new "incremental" soil-structure interaction solution is presented. It is based on assumptions of compatibility which dictate that footing loads and settlements must be consistent with the support reactions and displacements of the distorted frame. The method is particularly suited for use in design of moderately stiff, three-dimensional building frames subjected to substantial construction settlements. The computer solution takes into account the variation of structural rigidity during construction and allows an approximation for variation of soil properties over the site.

ACKNOWLEDGEMENTS

It is recognized by the author that without the assistance of the contractors, designers and owners of the four buildings studied, the instrumentation and collection of data would not have been possible.

The owner of the CN Tower is the Canadian National Railways. Abugov and Sunderland, the architects and structural engineers, provided design load information and structural drawings of the building. The interest expressed by Mr. J. Cullen P.Eng. of this firm is acknowledged. Hashman Construction was the contractor and made available their concrete placing records to enable estimation of footing loads.

The owner of the Avord Arms is Avord Holdings Ltd. Read, Jones, Christofferson, the structural consultants, provided design load data and foundation drawings. The building contractor, Christensen and Macdonald, assisted with the installation of settlement plugs. Franki Canada Ltd., the foundation contractors, supplied pile formation records from which an estimate of pile group end areas could be made.

The owner of the Oxford Building is Oxford Leaseholds Ltd. Design load data and foundation drawings were provided by the structural engineers, McBride Ragan. Poole Construction Ltd. was the contractor and made available their concrete records for calculation of footing loads.

The owner of the AGT Tower is the Alberta Government.

Rule, Wynn, Forbes, Lord and Partners, the architects and structural engineers, provided design data and foundation drawings. The assistance given by Mr. H. Feldberg P.Eng. and Mr. J. Kehoe of this firm is acknowledged. The aid given by the contractor, Poole Construction Ltd., in installation of instruments and tabulation of day to day construction records is gratefully acknowledged. Special thanks are due to the project supervisor, Mr. L. Albert, and members of his staff. Their knowledge of construction practice and interest in the research project made field work at the AGT Tower site a pleasure.

The assistance offered by R.M. Hardy and Associates, the foundation consultant for all four buildings, is acknowledged. The settlement records for three of the buildings and a considerable amount of the soil data was provided by this firm. The time given and interest shown by the chief engineer, Mr. M.C. Harris, is acknowledged.

Particular appreciation is due to Professor S.R. Sinclair, the Chairman of the Department of Civil Engineering, under whose direction this research project was initiated. The author is extremely grateful for the guidance given by Professor N.R. Morgenstern whose continuous interest stimulated the author throughout the entire research project. The contributions and helpful criticism offered by several other members of the academic staff of the Department of Civil Engineering is appreciated.

Without the financial assistance of the National

Research Council of Canada this research program could not have been undertaken. The patience and steadfast encouragement extended by my wife has been truly amazing.

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CHAPTER I

INTRODUCTION

1.1 Definition of the Problem

Previous to 1964, the availability of land in the central business district of the City of Edmonton allowed economical construction of small storey buildings of relatively large gross area. In recent years this practice has changed to intense land utilization as a result of a desire towards centralization of business, increasing land prices and a growing shortage of available ground space. This trend is not confined to growth in Edmonton and is prevalent in other urban centres of Canada.

Intense land utilization implies the construction of taller and heavier buildings with accompanying secondary development such as underground and surface parking structures. For foundation engineers the implications of this type of construction are deeper and larger excavations, increased footing contact pressures, greater total and differential settlements and possible increases in structural rigidity depending on the building design.

Most of the new multi-storey buildings in Edmonton will be founded on either an over-consolidated and unsaturated sandy clay till stratum, a preglacial sand deposit or a heavily over-consolidated interbedded sandstone and mudstone bedrock formation (Sinclair and Brooker, 1967). Because the deformation characteristics of these soils were not known with any degree of certainty, the prediction of settlement and heave due to loading and unloading is difficult. A further complication is the absence of unanimity of opinion with respect to the procedures appropriate for calculating the total settlements of buildings founded on over-consolidated soil. Therefore it is of considerable interest to determine the particular mechanisms which control the response of these over-consolidated soils to both loading and unloading in such a way that the results of the study will be of general use in foundation engineering.

1.2 Previous Case Histories

Som (1968) prepared an excellent summary of 13 case histories of settlement of structures founded on over-consolidated clays and 9 case histories of structures founded on normally-consolidated clays.

The progress of settlement, in the synthesis of the data, was classified into three portions. The first is called the "immediate" settlement and refers to the displacements which take place during construction under conditions of no volume change. The second portion is the "consolidation" settlement which is a time dependent phenomena caused by the hydrodynamic time lag which controls the dissipation of excess pore pressure originally generated during construction. The third and final portion is the "secondary" consolidation settlement which is caused by creep of the foundation soil under conditions of constant load and zero excess pore pressure.

Due to the absence of heave measurements Som made the assumption that the heave and subsequent settlement upon restoration of the excavation load is small in comparison to the total settlement. This assumption was considered to be justified where the net foundation pressures are relatively large and where excavations are not left open for too long a period of time. For those cases where the above conditions are satisfied only the "immediate" and "consolidation" settlements under the net foundation load were reported. In light of the experience with excavation heave reported later in this thesis, the preceding assumption made by Som may be questionable.

The general observations of the typical behaviour of over-consolidated clays were as follows.

(1) Structures founded on over-consolidated clays generally settle much less than those on normally-consolidated clays. This is attributed to reduction of compressibility due to over-consolidation.

(2) The "immediate" settlement is a much greater proportion of the final settlement for over-consolidated soils. The average of the 13 cases reported was 57.5 per cent of the 50 year settlement as compared to only 15.5 per cent for normally-consolidated clay. It is known that the "immediate" construction settlement may be influenced by the presence of sand strata, the length of the

construction period and the ratio of clay stratum thickness to the size of the foundation. However, it is considered that in spite of these factors, the large difference in proportion of construction settlement to the 50 year settlement is characteristic of the different response offered by each soil type.

Som attributes this behaviour to the fact that the pore pressure parameter A (Skempton, 1954) is nearer to 1 for normally-consolidated soils and is considerably less for over-consolidated soils (Bishop and Henkel, 1962). Therefore the development of excess pore pressure under identical total stress changes will be much smaller for over-consolidated soil than for normally-consolidated soil. This means that during undrained loading the effective stress will be greater in over-consolidated soil resulting in proportionally more "immediate" settlement and smaller "consolidation" settlement as less excess pore pressure needs to be dissipated.

(3) The progress of "consolidation" settlement is comparatively faster for over-consolidated soils than for normally-consolidated soils as the "secondary" settlement was reached between 1.5 to 7.5 years while the corresponding settlement for normally-consolidated clay was reached between 4 and 25 years. Som attributes this behaviour to greater overall permeability of the over-consolidated soil mass arising from fissures and joints.

(4) "Secondary" settlement of structures on over-consolidated clays is smaller (average 8.8% of the 50 year settlement) than those on normally-consolidated clays (21.6%).

A recent case history reported by Appendino and Samiolkowsky (1969) for the settlement of a 200 m. high chimney founded on clayey and sandy over-consolidated silt supports the preceding conclusions. The settlement of the 15 m. radius base during construction took place rapidly and appeared to be linearly related to footing load. Excess pore pressures were only recorded during the very rapid load application when the footing was poured with no excess pore pressures recorded thereafter. At completion of loading, settlement appeared to cease.

The definition of "immediate" settlement as used by Som (1968) is not entirely accurate. This settlement should not be equated, as is done in his summary of case histories, with the construction settlement. Conventionally the term "immediate" settlement refers only to the instantaneous settlement which takes place under conditions of zero drainage upon application of load.

The difference between "immediate" settlement and the construction settlement can be appreciable, particularly for over-consolidated soils. As pointed out by Skempton and Bjerrum (1957) the pore pressure generated in a deep deposit of over-consolidated soil is considerably smaller than for normally-consolidated soil and as noted by Bjerrum (1963), the proportion of primary consolidation occurring immediately upon load application (and during construction) will be correspondingly greater.

1.3 *Methods of Analysis*

The earliest method of settlement analysis consisted of a one-dimensional consolidation theory which was formulated by Terzaghi in 1924 (Terzaghi, 1924). The ultimate or total settlement (without secondary consolidation) could be estimated using soil parameters found from the laboratory oedometer test. Probably due to sample disturbance and non-representative permeability of the soil mass, this method frequently over-estimated the settlement and under-estimated the rate of consolidation. Although used for many years, and still today, it was recognized that the method was not generally applicable to over-consolidated soils (Terzaghi, 1936). It was not until Skempton and Bjerrum's contribution in 1957 that substantial improvement was achieved (Skempton and Bjerrum, 1957).

During load application the foundation soil undergoes lateral deformation which may result in appreciable immediate settlements. This three-dimensional settlement, assumed to take place under undrained conditions, was taken into account by Skempton and Bjerrum in their approximate procedure. They postulated that the subsequent consolidation takes place under conditions of no lateral deformation and is a function of the excess pore pressure set up in the soil. The pore pressure change is defined in terms of pore pressure parameters which are determined in the triaxial test for appropriate stress increments (Skempton, 1954). Although three-dimensional conditions are partly allowed for by the reasonable estimate of pore pressure, the use of this estimate for one-dimensional consolidation is somewhat illogical (Lee, 1968).

For saturated soils the immediate settlement can be computed by substituting Poisson's ratio μ = 0.5 and E_u, the undrained Young's modulus, into the standard equations of elasticity (Terzaghi, 1943; Scott, 1963 and Harr, 1966). These same equations can be used for non-saturated soils, still with zero drainage. For this case Poisson's ratio will not equal 0.5 but some lower value. With non-saturation the immediate settlement will be considerably larger. In a recent review of settlement prediction methods, Davis and Poulos (1968) point out that the total settlement (without secondary consolidation) can be computed using these same elastic equations, provided the drained Young's modulus, E' and Poisson's ratio, μ' are used.

Since volume change during consolidation is generally not one-dimensional and is often accompanied by significant lateral strain, Lambe (1964) recommended the "Stress Path Method" wherein the strain of an "average element" is determined in the triaxial test under stress conditions which duplicate those in the field. Extrapolation of the axial strain in the laboratory, which includes the immediate and consolidation strain, to the thickness of the deposit undergoing deformation yields the total settlement. An obvious practical difficulty here is the selection of the "average element". The experimental program to support this type of analysis is complex and the appropriate stress paths may be difficult to determine (Lee, 1968). This procedure is perhaps more suited for use with soft, normally-consolidated soils which usually undergo large strains rather than with stiff over-consolidated soils where borehole sampling is more difficult and strains are generally much smaller.

Settlements of over-consolidated soils during construction have been reported to be as high as 75 per cent of the total (Hanna, 1950, 1953). Concern should therefore not rest, as often appears to be the case, with the time-dependent consolidation settlement but rather with the greater portion of the total settlement that takes place during construction. On the basis of no observed excess pore pressures at the completion of construction, one could conclude that the consolidation settlement appears to be virtually absent for some over-consolidated soils. If the over-consolidated soil is not saturated, as is the case with the Edmonton till stratum, the settlement during construction to full occupancy may well represent nearly the entire total settlement. If this can be considered to be true, then three-dimensional elastic theory with drained parameters accounts for all the settlement with the exception of foundation creep.

1.4 Building Settlements and Structural Deformations

Although settlements of structures are calculated for a number of reasons, the primary purpose usually is the determination of whether or not ground movements will impair the aesthetic and/or serviceability requirements set for the building. This includes the performance of special cases, where an effort may be made to eliminate or reduce differential settlements of critical structures such as an accelerator (Ward, Burland and Gallois, 1968) and conventional construction such as ordinary apartment or office buildings. In conventional construction the calculated settlements are usually compared to allowable or tolerable limits. These have been deduced from studies on real buildings which performed satisfactorily or have been damaged (Skempton and MacDonald, 1956).

For most settlement analyses the footing loads which cause settlement are assumed to remain constant while the foundation undergoes deformation. The foundation displacements, calculated by means of this implied structural flexibility, are subsequently examined in terms of distortions of individual structural frame members to determine whether these members have sufficient moment or shear capacity. The anomaly immediately apparent is that structural members cannot be subjected to moments or shear forces by foundation displacements without some alteration of the footing loads which cause those displacements. This problem has led to consideration of flexibility or rigidity in a relative sense and has resulted in the concept of soil-structure interaction for use with those structures where the assumption of perfect flexibility departs significantly from reality.

Meyerhof (1953) and Chamecki (1956) were the first to publish approximate soil-structure interaction solutions for frame structures supported on isolated footings. The work by Meyerhof is based on slope deflection techniques whereby correction moments due to footing settlements are distributed to the superstructure using the Hardy Cross technique. The method does not allow for recalculation of footing settlements using the altered footing loads. Chamecki did consider the effect of load transfer due to structural distortion but limited the redistribution within the structural frame to adjacent members. The basis of his solution was that structural deformations and the footing loads that produce them must be compatible.

With the introduction of computer techniques, soil-structure interaction analyses have become much less tedious and the calculation of stresses in three-dimensional space frames has been facilitated.

Multi-storey structures may tend towards increased rigidity due to large building height. If founded on over-consolidated soil, an appreciable proportion of the total settlement will take place during construction. An ideal soil-structure interaction analysis should therefore be applicable to a three-dimensional space frame and allow variation in structural rigidity as the building frame is distorted by settlement during construction.

1.5 Scope of the Research Program

The three essential elements which make possible the analysis of any soil mechanics and foundation engineering problem are simply, factual field data, geotechnical data and theory. In foundation design it is usually the practice to obtain geotechnical data and utilize this with a theory which is known to be applicable to the problem. If it is desirable to check the performance of the foundation or the accuracy of the design, field observations are usually undertaken in order to obtain factual field data.

The research program described in this thesis is a reversal of the above process as the structural performance was obtained first. At the outset, the mechanisms governing foundation displacements and the applicability and reliability of the available geotechnical data were not clearly understood.

Settlement observations were taken on four multi-storey buildings, all located in the central business district of the City of Edmonton. Two buildings, the 26 storey CN Tower (Fig. 1.1) and the 27 storey Avord Arms (Fig. 1.2), are founded on the over-consolidated sandy clay till stratum. The proximity of these two structures to each other and the fact that the CN Tower is founded on spread footings and the Avord Arms on Franki end bearing piles, made it possible to evaluate in a comparative manner the settlement behaviour of

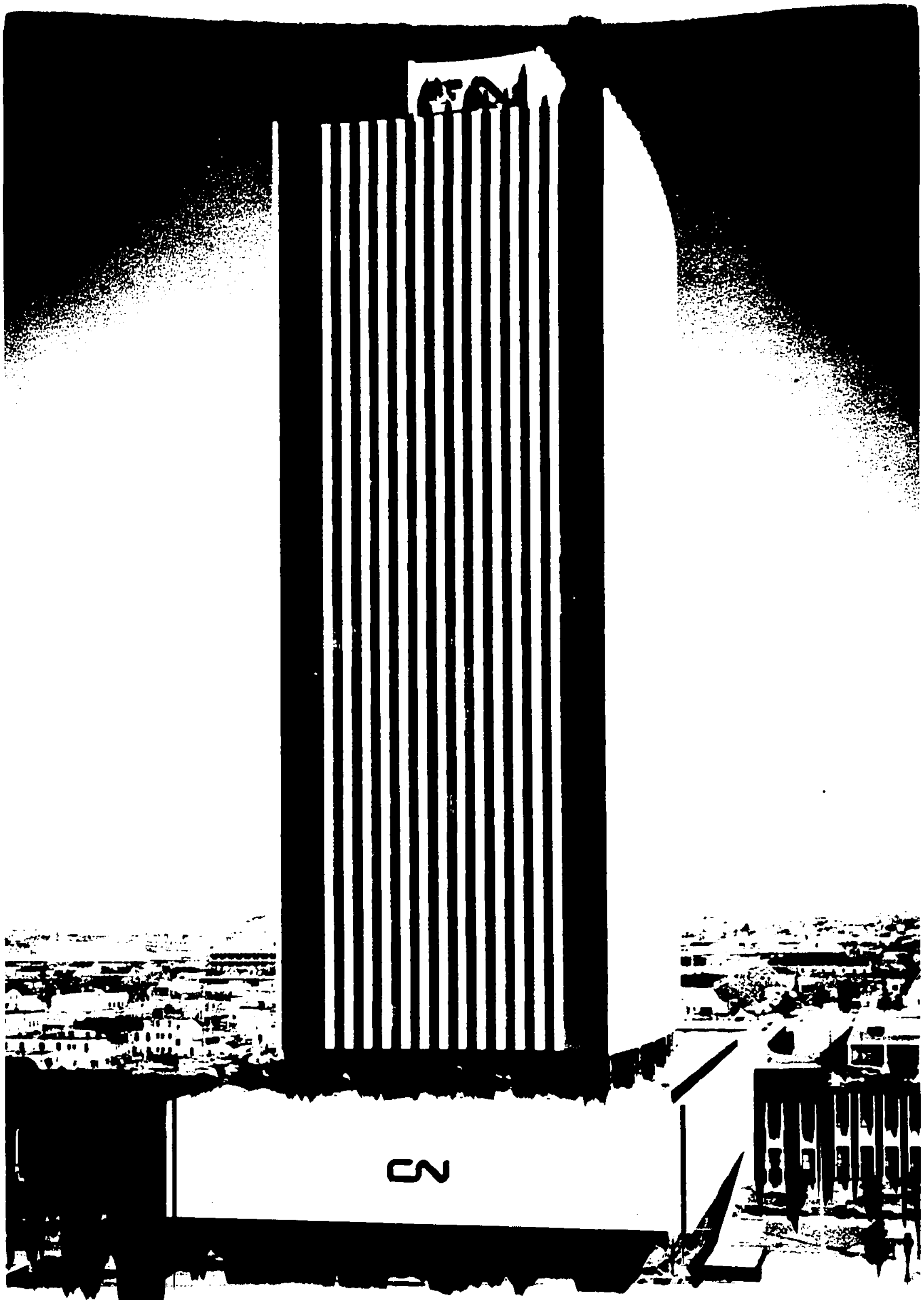


Fig. 1.1 CN Tower - Edmonton, Alberta

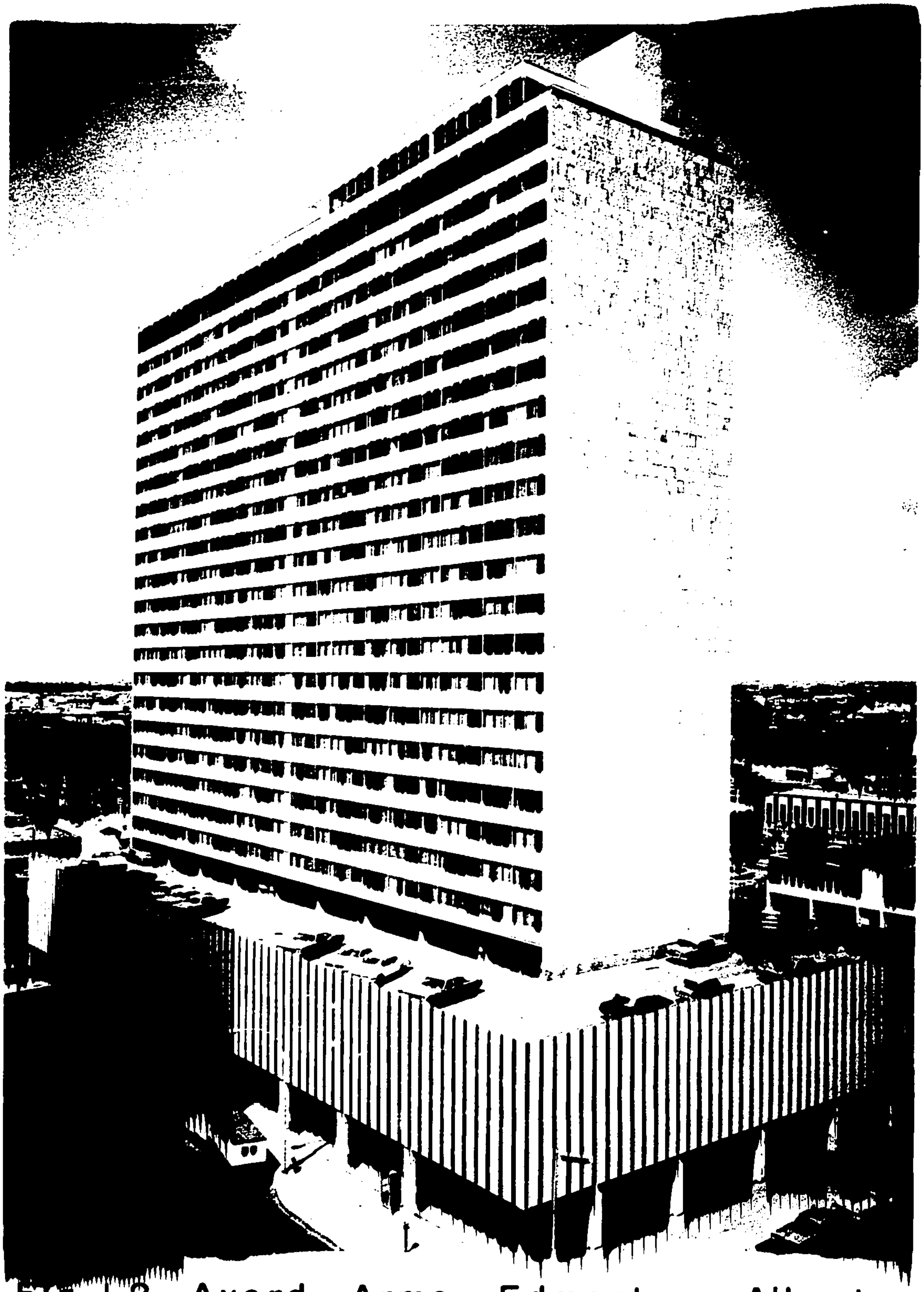


Fig. 1.2 Avord Arms - Edmonton, Alberta

the two foundation types. The other two buildings on which settlement readings were taken are the 26 storey Oxford Building and the 34 storey AGT Tower which together make up the AGT-Oxford Complex (Fig. 1.3). Both of these buildings are founded on spread footings in the same till stratum, but near the underlying preglacial sand deposit and bedrock formation.

During the excavation for the AGT Tower and the surrounding underground parking garage, the rebound of the foundation soil was monitored. Continuation of rebound measurements during construction made it possible to overlap these readings with the settlement observations. This constitutes a unique record of foundation movement.

From the analysis of the foundation displacements the mechanisms of deformation were determined and representative parameters of the soil mass were derived. These were compared with relevant values obtained in the laboratory from tests run on specimens taken at the sites.

Since the rigidity of the building structure can influence the settlement of its supports, the rigidity of the CN Tower is examined in considerable detail. Recognition of the consequence of the combined effects of increasing settlement with varying structural rigidity has resulted in the formulation of a new and more realistic approach to soil-structure interaction analysis.

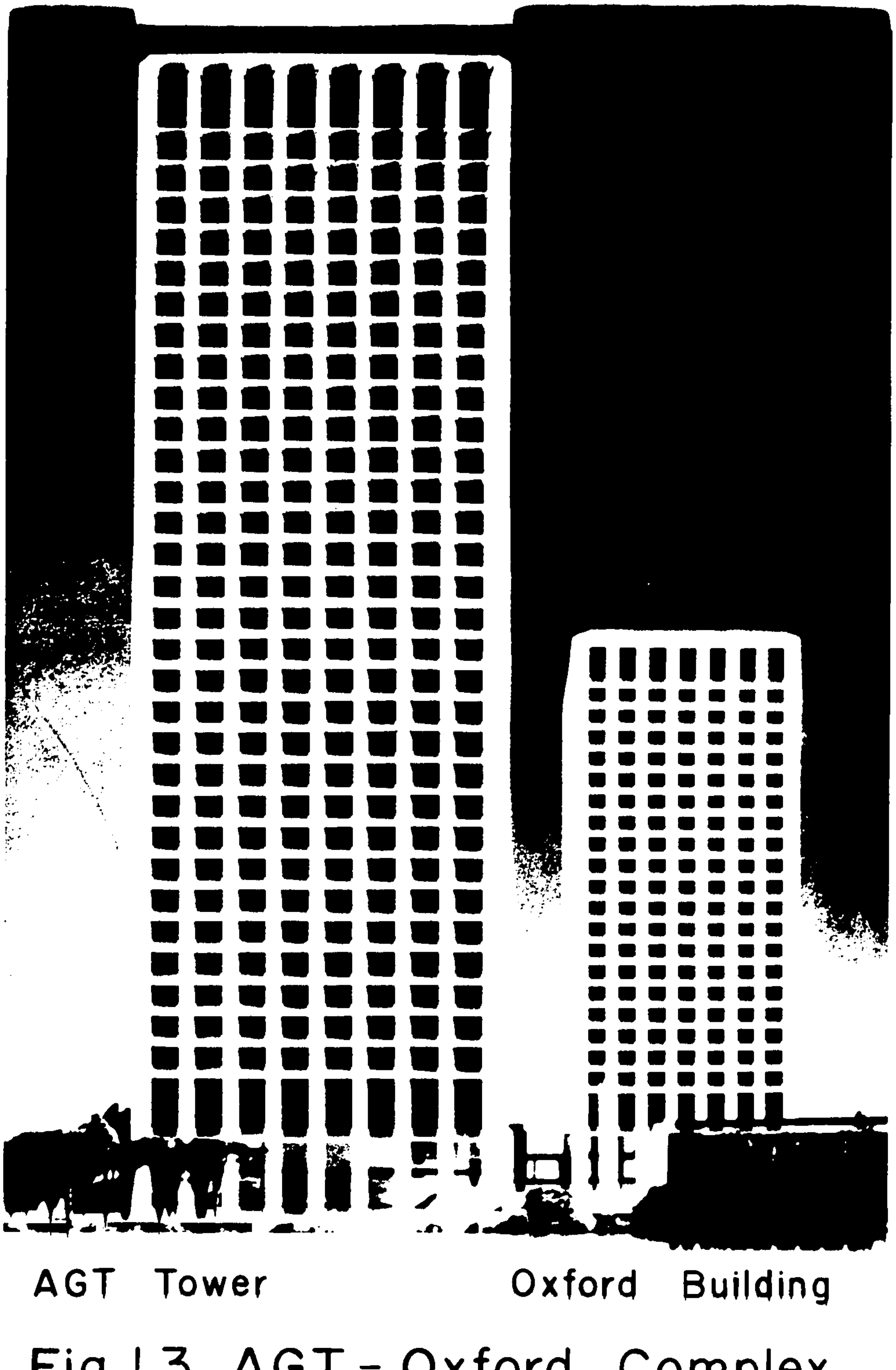


Fig. 1.3 AGT - Oxford Complex, Edmonton, Alberta

1.6 Organization of the Thesis

The thesis begins with a detailed analysis of the settlements at the CN Tower and Avord Arms buildings. Empirical foundation parameters are derived and compared to values obtained in the laboratory.

An evaluation of the influence of the structural rigidity of the CN Tower on the reactions of the building frame is given in Chapter III. The concept of variable structural rigidity is introduced in an incremental method of structural analysis. A mathematical description of the new incremental procedure for soil-structure interaction is presented in Chapter IV with an accompanying sample computer solution.

The procedure used to measure heave at the AGT excavation is described in Chapter V. The analysis of the rebound record is outlined in detail and several implications for construction practice are noted.

The settlement behaviour of the AGT Tower and the Oxford Building is described in Chapter VI. The compression data for the over-consolidated foundation soils is analyzed and representative deformation parameters are described.

A summary of the conclusions from the entire research program and suggestions for further research are given in Chapter VII.

CHAPTER II

SETTLEMENTS AT THE CN TOWER AND AVORD ARMS

2.1 Introduction

In this chapter the settlement behaviour of two buildings, the CN Tower and Avord Arms, is examined. The geology of the site, soil conditions, foundation types and method of instrumentation are fully described.

The analysis of settlement data has been more extensive for the CN Tower. The reason for this is partly because of a more complete settlement load record, readily available structural information and partly because the analysis of the spread footing design is more tractable. Wherever possible, reference to the Avord Arms and comparisons between the two structures will be made.

Although still specific to the area of the two buildings, the description of geology found in this chapter has been expanded to include the entire central area of Edmonton. This is done for reasons of completeness.

2.2 Building Location and Description

The two buildings under consideration are located on the northern edge of the central business district in downtown Edmonton. The specific geographical location is shown on Fig. 2.1.

Although differing in plan, both the 27 storey Avord

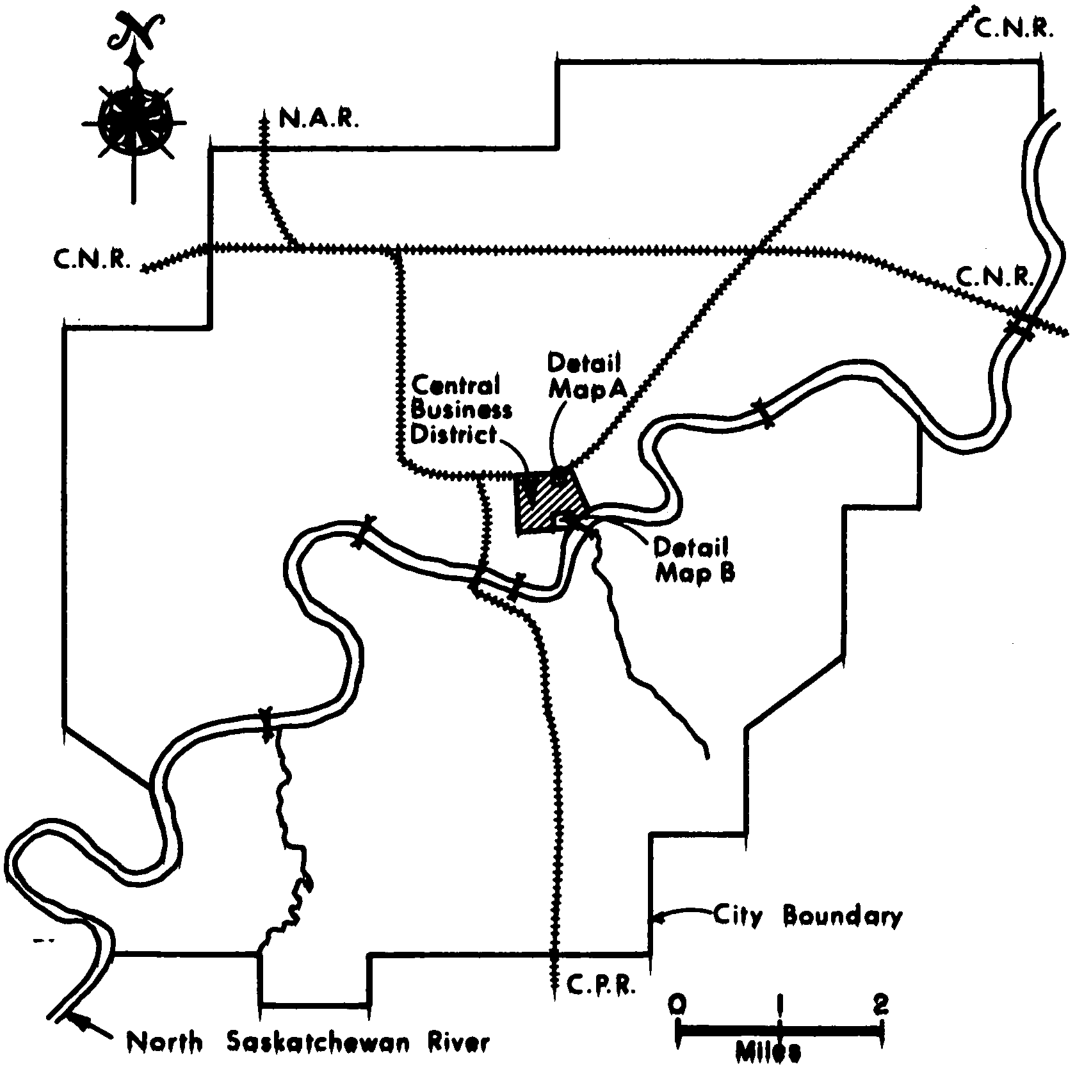


Fig. 2.1 City of Edmonton Outline Map

Arms and the 26 storey CN Tower are reinforced concrete structures consisting of a small number of floors for parking with the remaining upper floors respectively being utilized for apartments and office space. In the case of the CN Tower the four parking floors are cantilevered a distance of 27 feet beyond the west, south and eastern exterior column lines, thereby inducing an increased built-in structural rigidity and greater loads at the supporting columns. An east-west section through this building is shown in Fig. 2.2.

The primary difference between these two buildings is their foundations. The Avord Arms is founded on Franki compacted concrete piles expanded at their base at a depth of sixteen feet below ground surface. Fig. 2.3 shows the foundation plan. A total of 239 piles were used, most of which were 24 inches or 20 inches in diameter, and a small number being 16 inches in diameter.

The CN Tower is founded on spread footings at a depth varying between 26 feet below ground surface at the core to about 22 feet at exterior footings. The central core which contains the elevator shafts and carries the central floor loads is supported on a single mat footing. The footings were formed by neat excavation into the till and an average of eight feet of compacted backfill was placed above the footings to give a net depth of basement of 13 feet below ground surface. Fig. 2.4 illustrates the footing and foundation plan for this structure as well as for the small parking

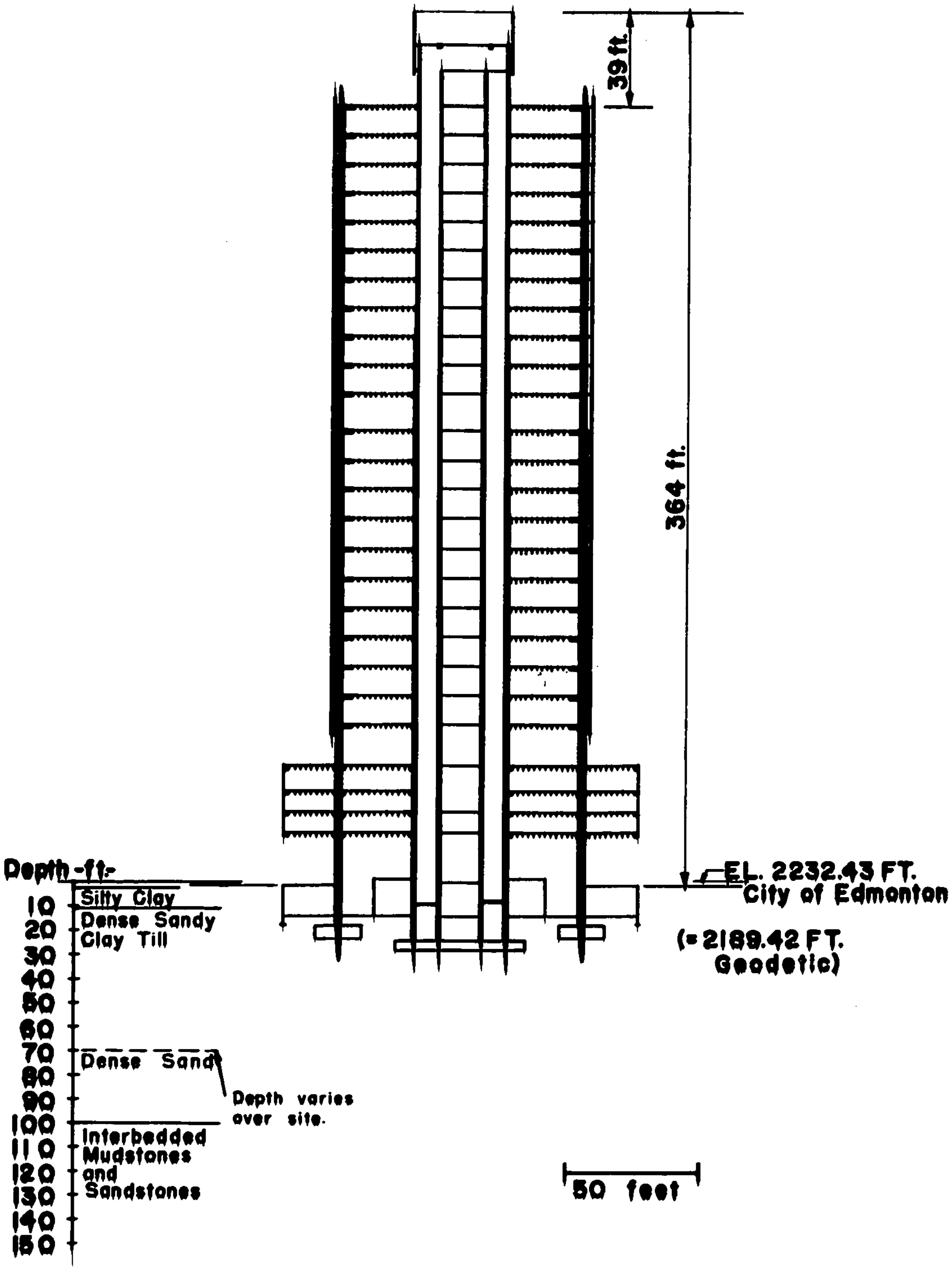


Fig. 2.2 Soil Strata and Building Section
CN Tower

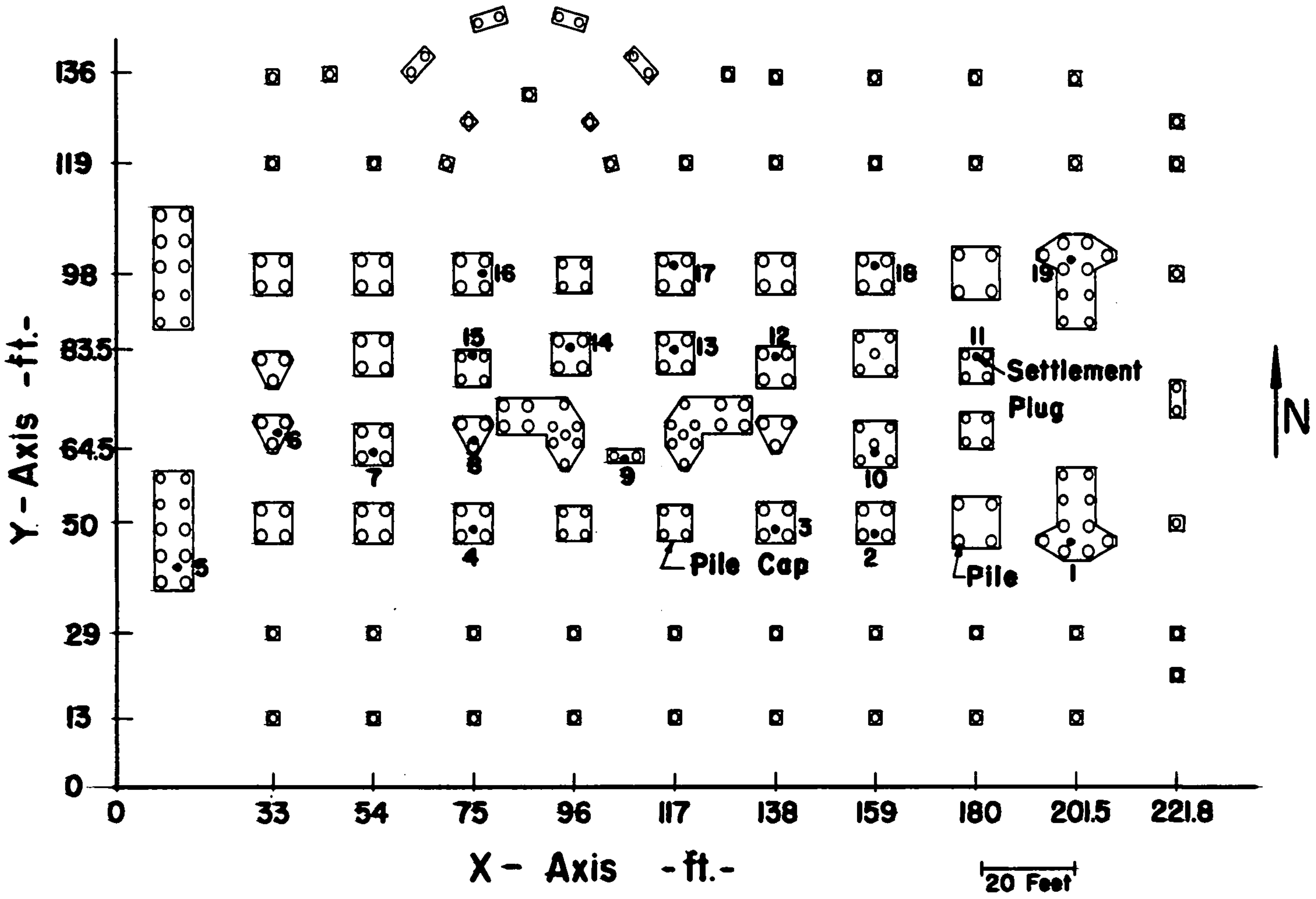


Fig.2.3 Foundation Plan and Settlement Plug Locations
Avord Arms Building

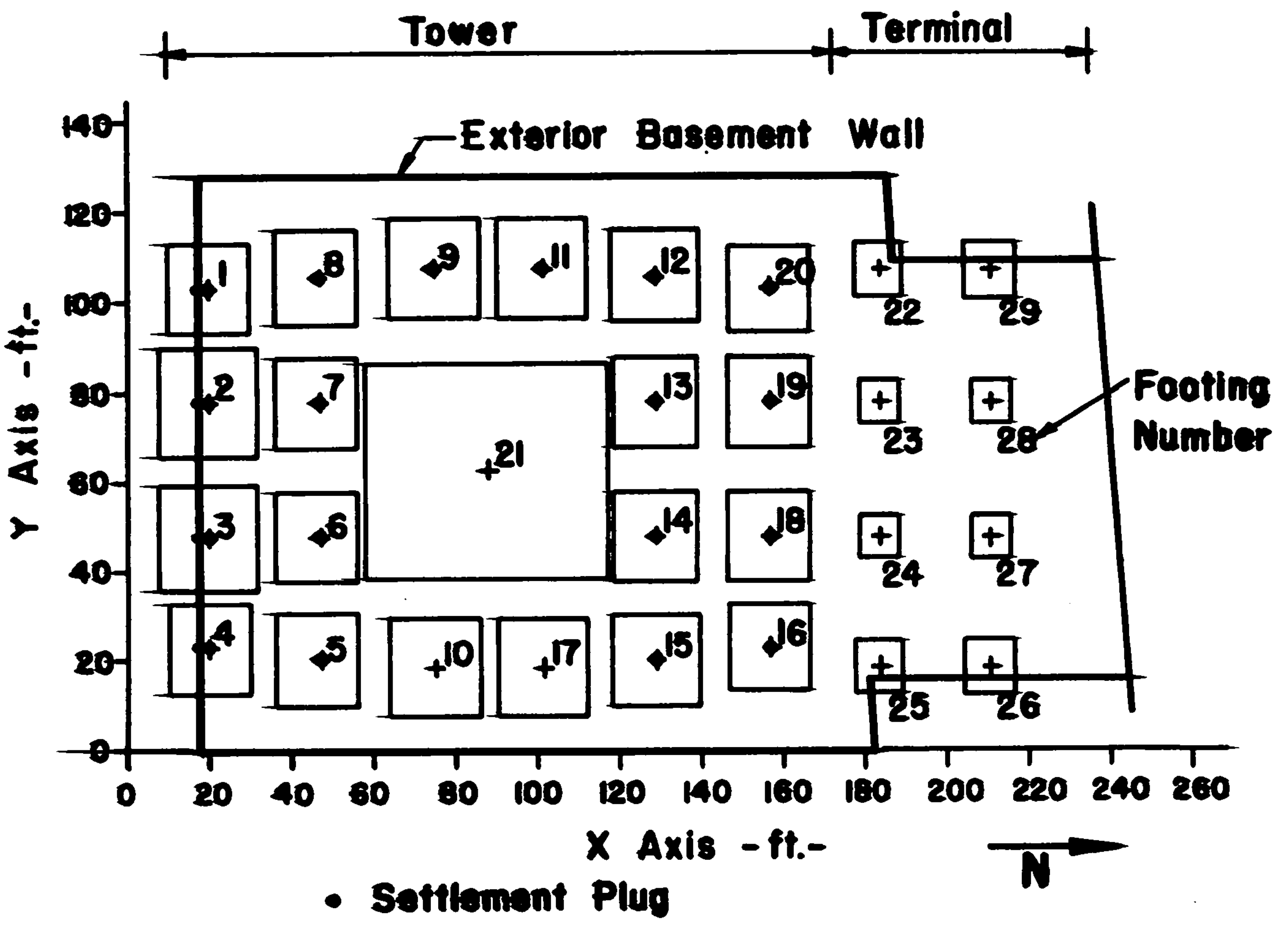


Fig.2.4 Foundation and Footing Plan
Settlement Plug Locations CN Tower

area and terminal extension located immediately to the north.

2.3 Geology

Most of the City of Edmonton is built on a soil profile in descending order, of lacustrine clays, silty clay till, pre-glacial sand and gravel and soft mudstone and sandstone bedrock. In the area of the two buildings, this same soil profile is found; Table 2.1 lists the main units with approximate depths. Fig. 2.2 shows the soil strata in relation to the foundation cross-section. A small amount of fill was encountered at the CN Tower site but was removed during excavation for the basement.

TABLE 2.1

STRATIGRAPHIC SEQUENCE AT THE CN TOWER AND AVORD ARMS

Geologic Description	Material Description	Approximate Depth (ft.)
Lacustrine Clay	Silty-Clay, becomes more silty with depth	0 to 16
Till	Sandy or Silty-Clay containing sand lenses	16 to 70
Saskatchewan Sands and Gravels	Medium sand, some gravel just above bedrock	70 to 100
Edmonton Formation	Interbedded mudstones and siltstones	100 to ---?

Bedrock at the site is described as "poorly consolidated materials comprising the Edmonton Formation of late Cretaceous age" (Carlson, 1966). Within the upper part of the bedrock which might influence the foundation are found interbedded bentonitic shales and sandstones and several continuous bentonite and coal seams. The regional dip of the bedrock surface is about 20 feet to the mile southwestward (Bayrock and Berg, 1966). To the extent that many samples do not show fissility the shales are probably more correctly classified as mudstone and many of the sandstones could more properly be called siltstones. Local usage is to use either the term shale or clay-shale, the latter because of the tendency of the shale to weather to a clay-like material. In its undisturbed state the bedrock is very dense and is a competent foundation material.

The present bedrock surface has been formed by erosion followed by glaciation under ice estimated to have been 5000 feet thick (Bayrock and Hughes, 1962). The present maximum relief in the central Edmonton area is the result of erosion during Tertiary and Pleistocene time. The site of the CN Tower and Avord Arms is located near the valley wall of an ancient pre-glacial valley which was several miles wide (Carlson, 1966). The maximum slope of the bedrock surface in the general vicinity of the sites is in the order of 1% to 2%.

During the erosion cycles that formed the bedrock surface a series of sand and gravel layers were deposited as valley fill (Bayrock and Berg, 1966). The name Saskatchewan Sands and Gravels is applied to these deposits; in central Edmonton sand predominates with some gravel found just above the bedrock. The sand is characterized by high Standard Penetration blow counts usually exceeding 110 blows per foot. Very little sampling or laboratory testing has been carried out on the Saskatchewan Sands and Gravels, partly due to the difficulty of obtaining undisturbed samples, but mainly due to its relatively great depth below most of the foundations used in the Edmonton area.

Recent work has indicated that there are two distinct till sheets of Wisconsin age in the Edmonton area (Westgate, 1969). The identification is largely based on pebble orientation; there does not appear to be any marked distinction in engineering properties between the upper and lower till sheets. Some exposures in central Edmonton show a few feet of sand between the two tills. Although exposures of the unweathered till show rusty fractures, usually in a random orientation, the upper part frequently exhibits a columnar fracture pattern. The till is classified as a sandy-silty clay and contains stones of a wide range in sizes, coal and shale fragments, and sand and silt lenses. The silty clay matrix is usually of low to medium plasticity. The density of the till varies considerably in the Edmonton area but in central Edmonton the density is high as evidenced by Standard Penetration blow counts which range from 60 to 150 blows per foot and average 93 blows per foot. These values do not include the upper two or three feet in the till which is usually softer and has blow counts in the order of 18 to 50 blows per foot. The water content of the till is usually in the range of 10% to 20% averaging 15%.

The till was formed from continental glaciers which advanced from the northeast. The natural drainage systems were blocked off during the retreat of the glaciers resulting in the formation of a number of proglacial lakes. Lake Edmonton was one such lake and covered much of the Edmonton area. During its life, lake sediments were deposited which ranged from clays of high plasticity at the surface to silts and sands just above the till. The upper part of the lacustrine deposit is highly dessicated.

2.4 Geotechnical Properties from Laboratory Tests

Both the Avord Arms and the CN Tower are founded on the till and thus it is the geotechnical properties of this soil that are of greatest interest. The stress influence at bedrock elevation is small; the net change in vertical pressure (Boussinesq) is only 10% of the original total overburden pressure.

A total of five boreholes were drilled at the CN Tower and two at the Avord Arms during the foundation investigation at locations as shown in Fig. 2.7. Because of existing buildings they were drilled around the peripheries of the site. The logs of several boreholes can be found in Appendix A.

Testholes at the site, and in general in the central

Edmonton area, do not show a static water table. Perched water tables are encountered in sand lenses in the till and occasionally the water is under a substantial hydrostatic head. The clay till is commonly unsaturated. Degrees of saturation of 83% and 75% have been found from two tube samples taken at depths of 28 feet and 43 feet in a boring 800 feet west of the CN Tower.

Gradation analyses on block samples taken at footing level at the CN Tower showed an average of 42% sand sizes, 38% silt sizes, and 20% clay sizes (M.I.T. Scale). Consolidated undrained and confined undrained triaxial compression tests and consolidation tests were run on specimens cut from the block samples. Due to the hard and dense nature of the till plus the random occurrence of stones, it was difficult to trim specimens, particularly in the case of the consolidation test where a close fit in the consolidation ring was required.

The results of the undrained triaxial tests on three specimens are summarized in Table 2.2 The tests reflect an uncertain amount of sample disturbance and also the variation that is commonly found in till specimens, even when taken from a single block or tube sample. The tangent modulus of deformation has been determined from the initial nearly straight portion of the stress-strain curve, corrected for seating deformation.

Two consolidated undrained compression tests were carried out at cell pressures of 75 psi and 140 psi. The

TABLE 2.2

CONFINED COMPRESSION TESTS

(CELL PRESSURE - 30 psi)

Sample	Maximum Deviator Stress (psi)	Strain at Failure (%)	Water Content (%)	Initial Tangent Modulus of Deformation (psi)
A	118	15.4	13.2	1460
B	108	14.1	13.5	1160
C	156	13.1	12.4	1480

stress-strain curves for the tests are shown in Fig. 2.5. The numerical results are given in Table 2.3.

It can be seen that consolidation has resulted in much

TABLE 2.3

CONSOLIDATED UNDRAINED COMPRESSION TESTS

Sample	Initial Water Content (%)	Cell Pressure (psi)	Maximum Deviator Stress (psi)	Strain @ Failure (%)	Secant Modulus @ 0.5% Strain (psi)
D	13.3	75	110.5	6.65	14,000
E	13.2	140	197.8	12.3	12,000

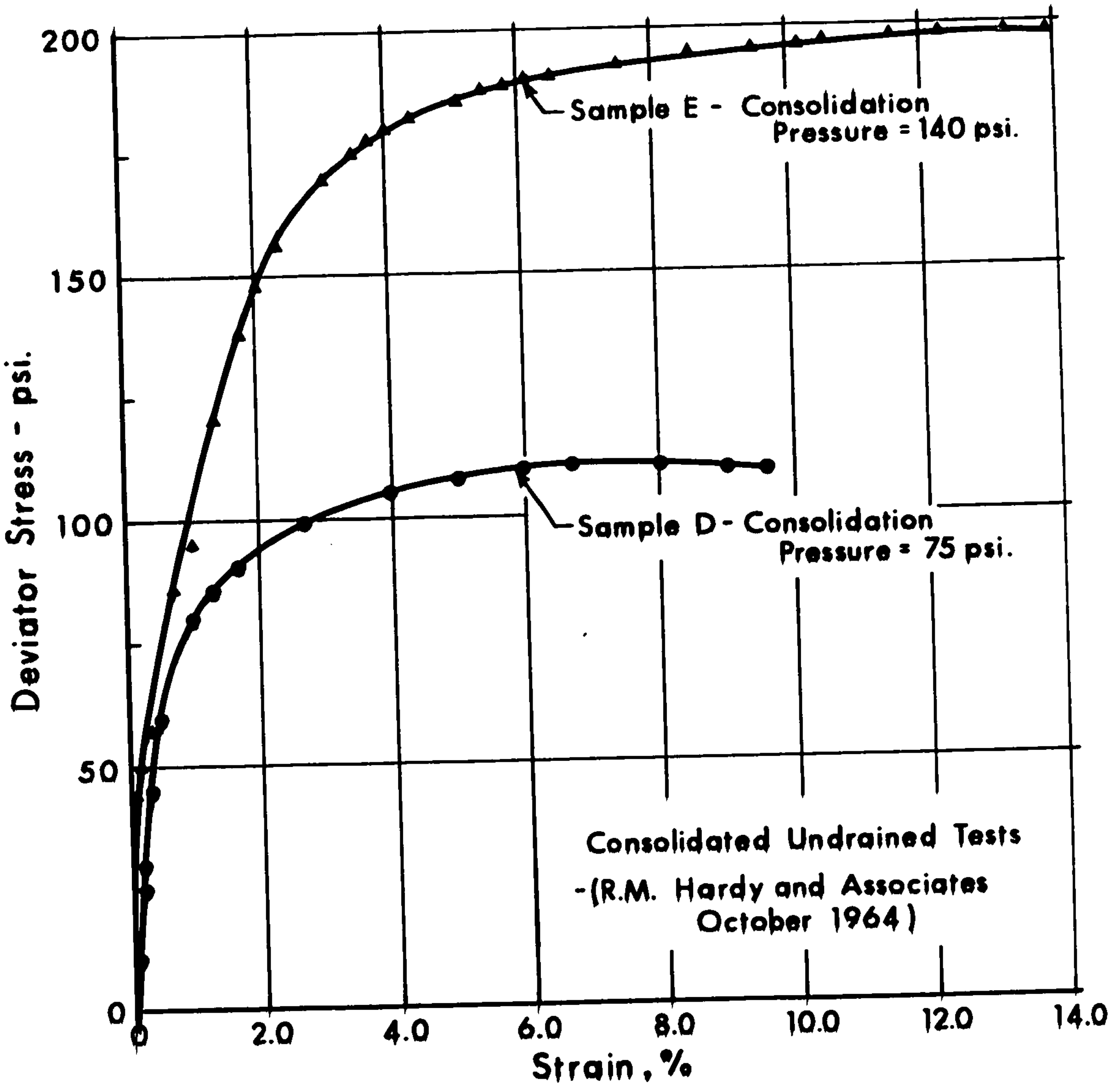


Fig. 2.5 Stress Strain Curves on Till - CN Tower

higher moduli of deformation. There was insufficient material to provide three consolidated, undrained triaxial specimens. On the basis of the two tests the straight line Mohr Envelope had the following parameters:

Total Stress $C = 2$ psi $\phi = 24°$

The results of three consolidation tests are shown in Fig. 2.6. The curves shown have been corrected for compressibility of the testing apparatus but otherwise represent the results of conventional testing methods. Interpretation of the pre-consolidation by accepted methods is difficult, and it is likely that the maximum load reached (14.4 ksf) is not on the virgin compression branch. To give representative numerical results, Table 2.4 has been prepared from interpretation of the data.

2.5 Instrumentation and Data Acquisition

When access became possible, special settlement plugs provided by the Division of Building Research of the National Research Council were installed in the reinforced concrete columns at the lowest floor of the CN Tower and Terminal building.

Three inch ram-set pins were used for reference markers in the Avord Arms building. As construction proceeded, problems of access necessitated the termination of readings on some columns at the CN Tower and installation of a second set of markers in the Avord Arms. In the latter case, two

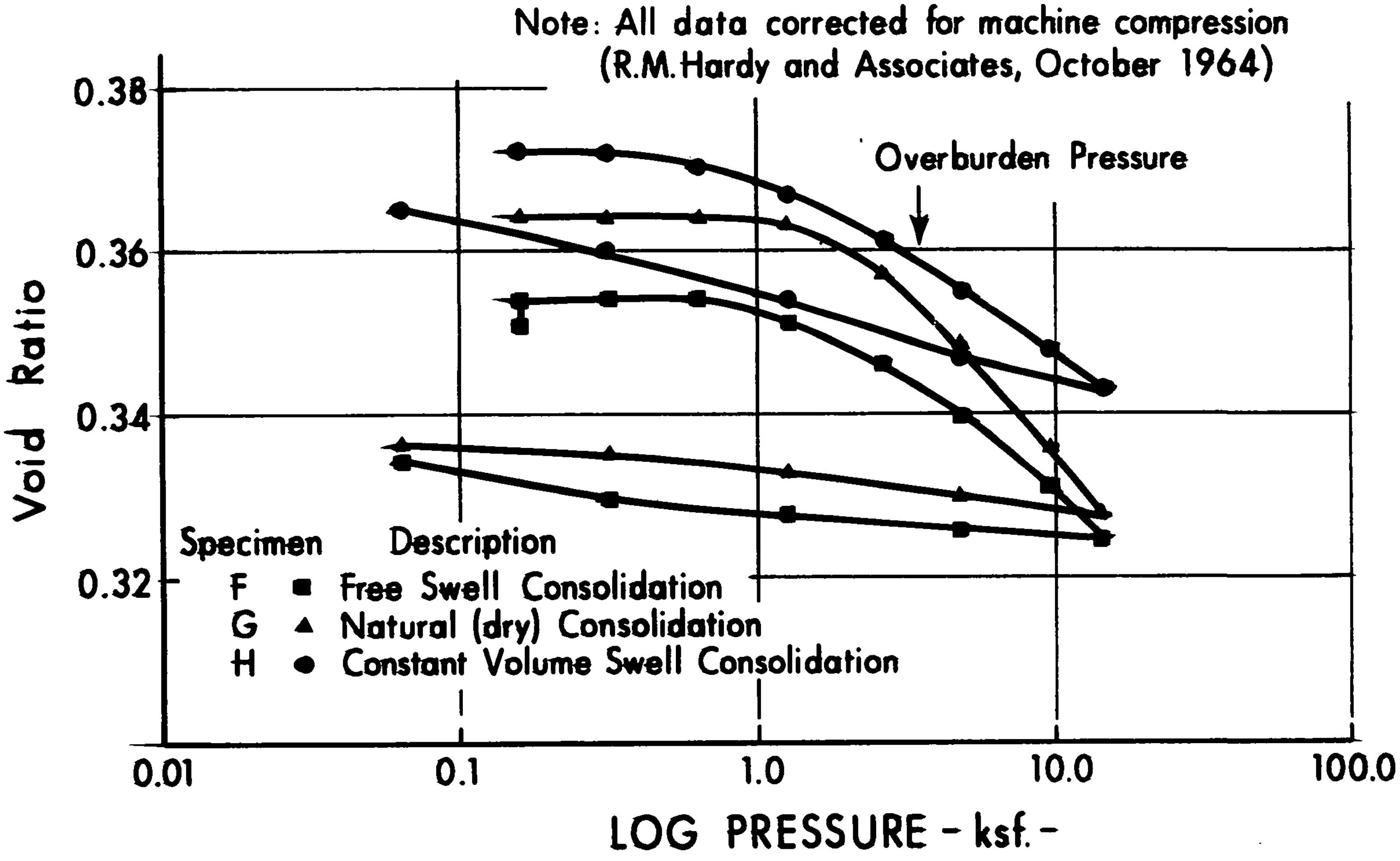


Fig. 2.6 Laboratory Consolidation Tests on Till CN Tower

TABLE 2.4

SUMMARY OF CONSOLIDATION TEST DATA

	Specimen (1)		
	F	G	H
Initial Water Content, W%	11.8	13.0	13.9
Initial Void Ratio, e	.351	.368	.404
Initial Saturation, S%	90.1	94.7	92.0
Compressive Index, C_c (2)			
3.5 ksf to 7.0 ksf	.026	.036	.025
7.0 ksf to 14.4 ksf	.032	.042	.032
Rebound Index, C_r (2)			
14.4 ksf to 7.0 ksf	.006	.006	.011
14.4 ksf to 0.06 ksf	.016	.013	.020
Modulus of Volume Comp., m_v (ft^2/k)			
3.5 ksf to 7.0 ksf	.0017	.0023	.0017
3.5 ksf to 14.4 ksf	.0012	.0017	.0011
Modulus of Deformation, E (psi) (3)			
3.5 ksf to 7.0 ksf	1901	1393	1924
3.5 ksf to 14.4 ksf	2632	1909	2996

1. Test Conditions:

   F. Free swell - sample allowed to swell under token load of 0.16 ksf before consolidation.

   G. Natural consolidation - sample not allowed access to water, drying out prevented.

   H. Constant volume swell - sample maintained at constant volume after given access to water. When tendency to swell completed consolidation test run.

2. Compressive index, C_c, and Rebound Index, C_r, are slope of compression or rebound curves on plot of void ratio versus logarithm of pressure, between points given.

3. Calculated secant modulus of deformation based on an assumed Poisson's Ratio, $\mu = 0.4$.

sets of overlapping readings allowed continuation of the settlement record.

All surveying was carried out using a precise level and a rod scaled in millimeters; possible error in this survey might be as much as ±1 mm. Survey error at the Avord Arms would tend to be somewhat greater due to increased distance from the deep benchmark and greater complexity of the traverse within the building. Readings were not taken at equal intervals of time, but rather in approximately equal intervals of load application; i.e., whenever four or five floors were added to either the Avord Arms or the CN Tower.

Three benchmarks were installed and correlated such that errors due to unreliable benchmarks could be detected. Their locations are indicated on Fig. 2.7. The benchmarks consisted of a 3/4 inch steel rod driven to refusal inside a cased auger hole. Stoppers were used to guide the steel rod down the casing. Sand was used to backfill the annular space between the casing and auger hole. A steel threaded cap served to protect the benchmark at ground level. Benchmark numbers 1 and 2 were driven to refusal at depths of 25 and 29.5 feet respectively in the dense glacial till. Benchmark number 3 located inside the CN building was driven to refusal in the bedrock shale at a depth of 110 feet. A survey traverse taken June, 1966 and June, 1970 resulted in elevation differences of +1.3 mm. and -0.5 mm. respectively, indicating that the movement of benchmark number 3 was negligible.

2.6 Building Loads

The procedure followed in deriving the footing loads

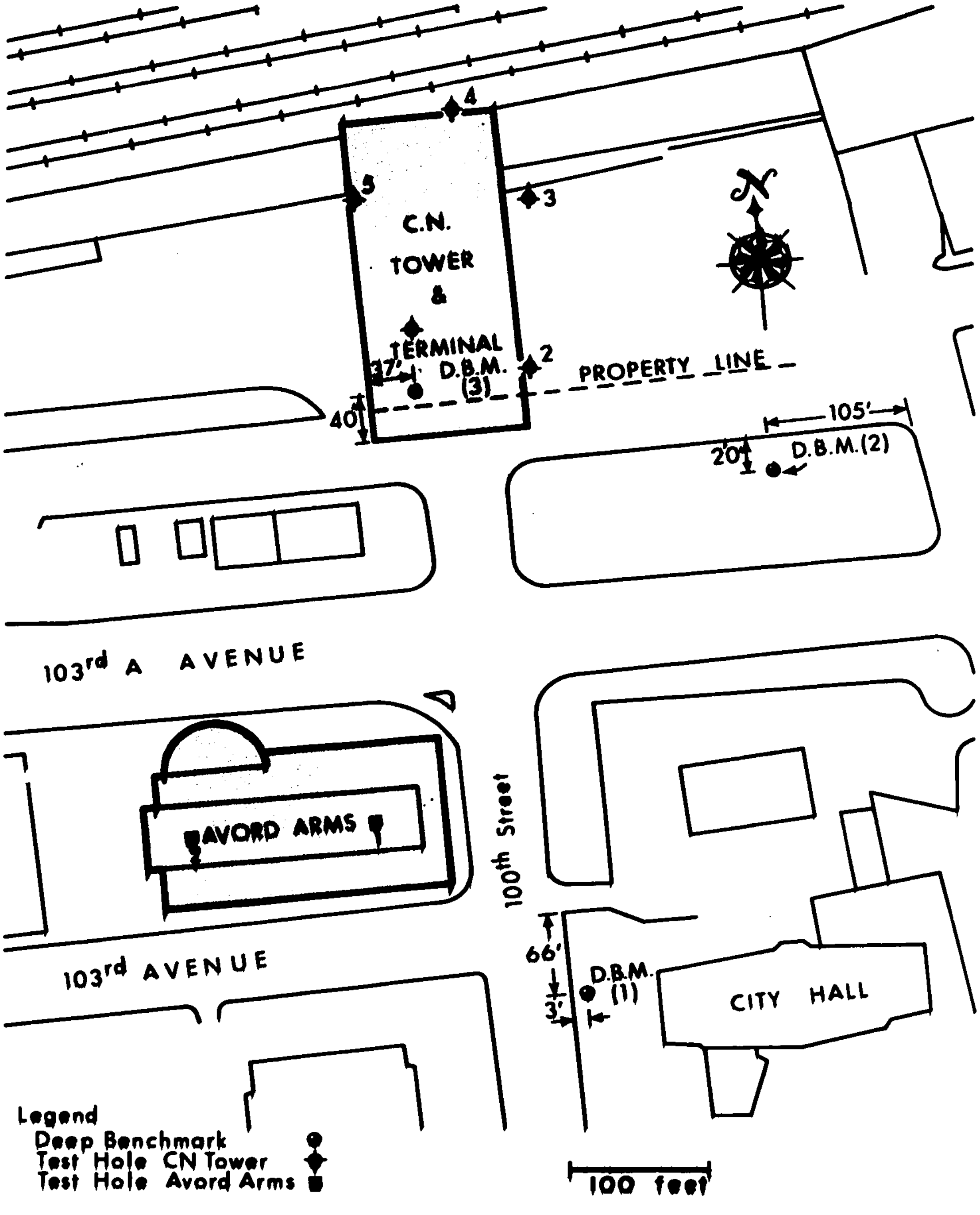


Fig. 2.7 Detail Map A, Location of CN Tower, Avord Arms Building, Testholes and Deep Benchmarks

at the CN Tower was based on three assumptions:

(1) The actual footing dead loads can be most accurately calculated using the concrete quantities delivered to the job.

(2) The proportion of building load carried by a single footing is in accordance with the distribution calculated using structural design loads.

(3) The increase in footing pressure due to live load during the move-in period is equal to the live load computed by the structural designers, the values having been reduced by appropriate factors as outlined in the National Building Code.

Assumptions (1) and (2) had to be made because no columns were instrumented for the purpose of measuring the load they carried. The deadloads computed in this manner nearly equalled the designer's calculated dead load, the latter being greater by about 10%.

The computed reduced live load comprised only 4% of total load carried by individual footings and thus assumption (3) is not critical. The four interior footings are minor exceptions with the reduced live load contributing approximately 10%.

Tabulation of the pile group loads in accordance with the procedure outlined above was not possible for the Avord Arms building as a day to day record of poured concrete quantities was not available. Although the Avord Arms was designed by a different firm, the close agreement of the CN

Tower design loads with the derived loads is an indication that this shortcoming is perhaps not serious. The design pile group loadings were therefore used directly and include an unknown amount of reduced live load. The loading history during the Avord Arms construction period was determined simply by proportioning the total load to the pile group on the basis of the ratio of constructed to maximum storey height.

2.7 Settlement Data

The maximum settlement recorded at the CN Tower to February 25, 1970 is 3.0 cm. The maximum differential settlement observed between two adjacent columns is 1.1 cm. over a span of 37.2 feet. A simple calculation yields an angular distortion value (differential displacement divided by length of span) of S/L of 0.97×10^{-3} which is well within tolerable limits as defined by Skempton and MacDonald (1956), Bjerrum (1963) and Feld (1965).

The variation of both contact pressure and settlement with time for several CN Tower footings is illustrated in Figs. 2.8, 2.9, 2.10 and 2.11. It is quite obvious that the settlement response to increases in footing contact pressure is very rapid and can be termed instantaneous. Most of the settlement takes place during the construction period, the settlement taking place thereafter constitutes less than approximately 13% of the total. Observations reported by Klohn (1965) on tills in Saskatchewan indicate

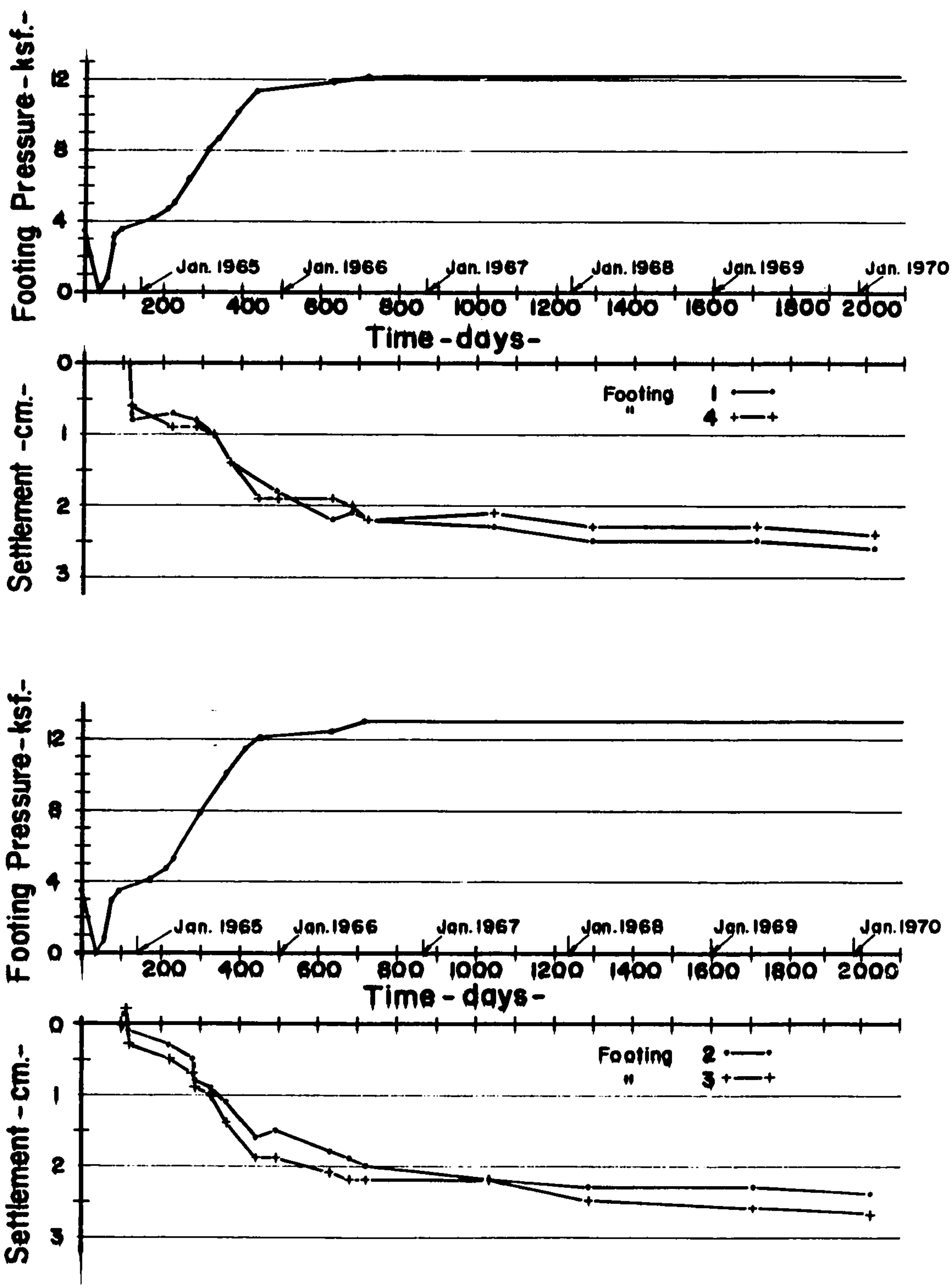


Fig. 2.8 Load and Settlement History, CN Tower

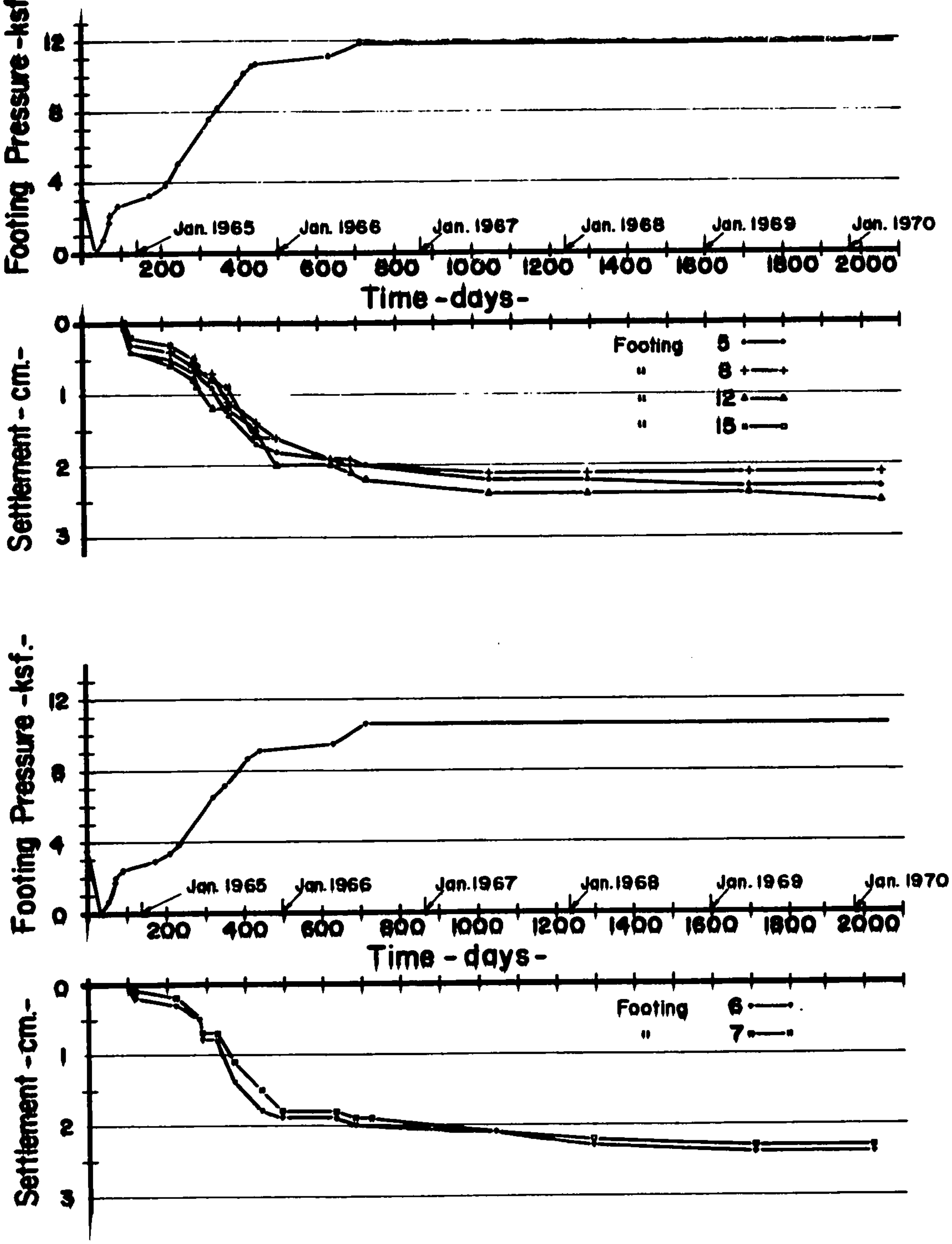


Fig. 2.9 Load and Settlement History, CN Tower

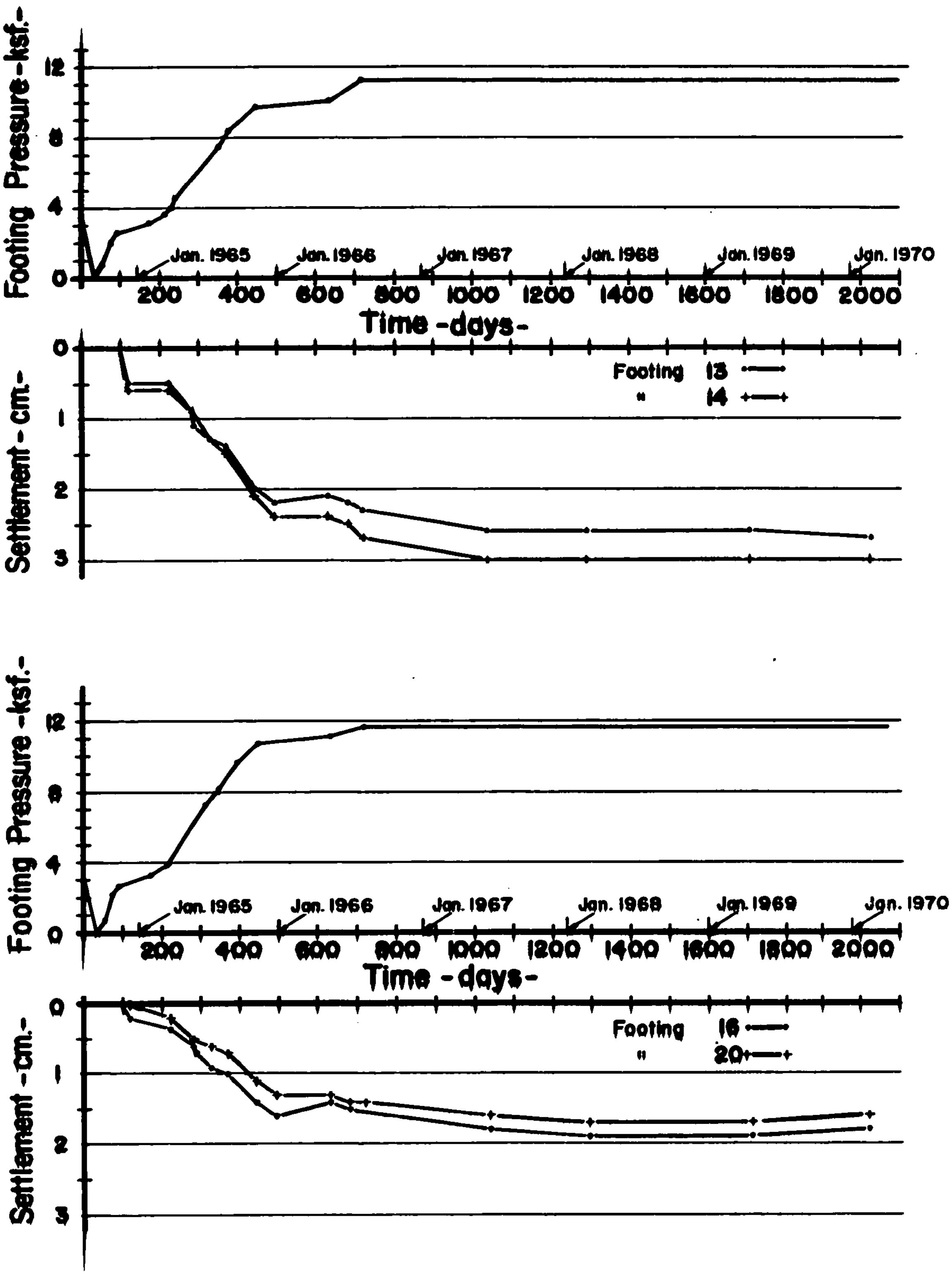


Fig. 2.10 Load and Settlement History, CN Tower

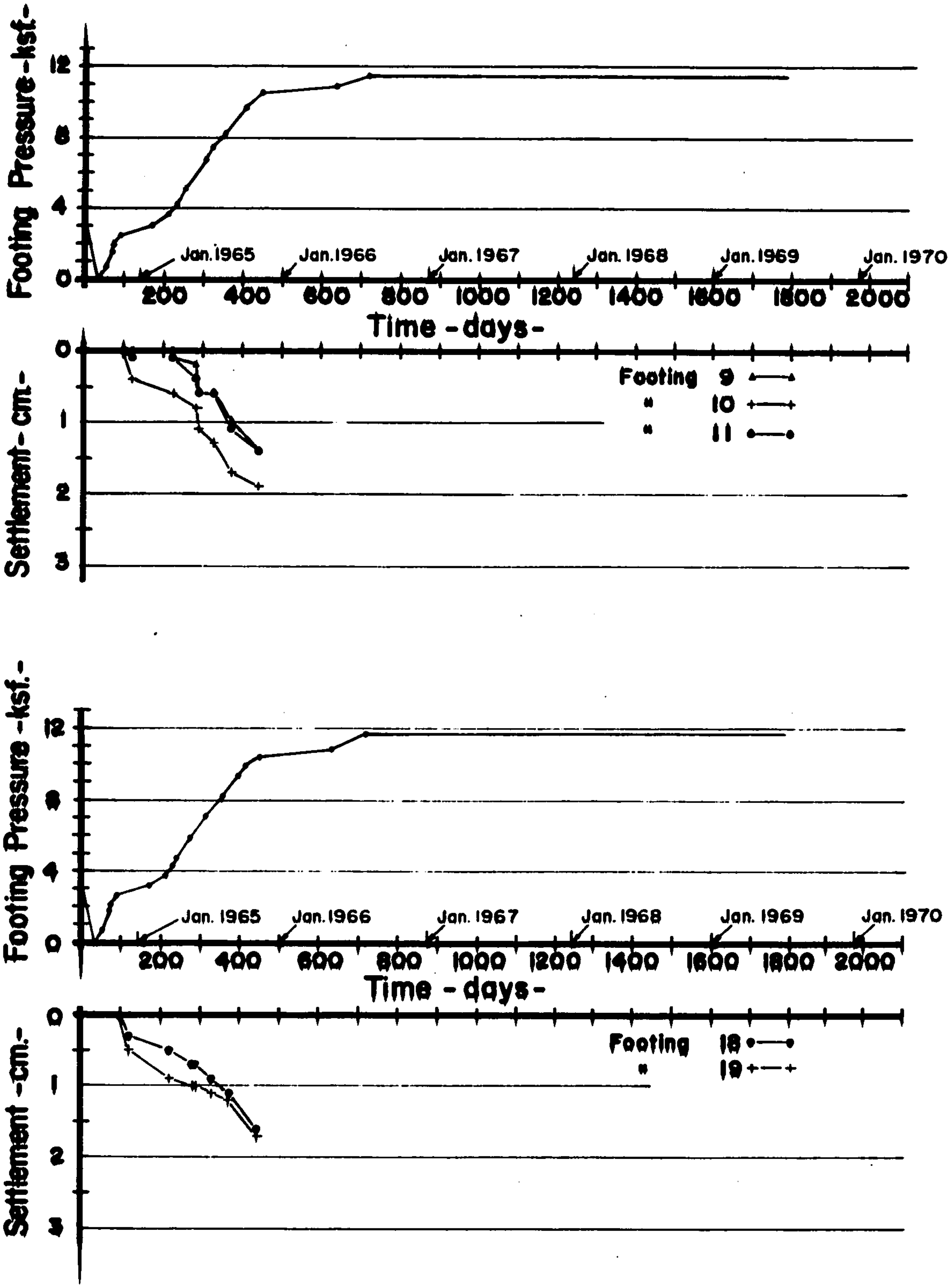


Fig. 2.11 Load and Settlement History, CN Tower

similar behaviour.

This rapid response to loading is shown even more explicity in Fig. 2.12 where settlement is plotted versus the log of time since the start of construction. If it is assumed that the secondary compression continues to be proportional to the log of time for footing 5, then an extension of the line predicts only an additional 0.5 cm. of settlement over the next 25 years. For several footings this value appears to be even smaller.

For all practical purposes it can be said that settlement virtually ceases approximately one year after completion of the structure. Several footings did not exhibit any increase in settlement during the recent April 17, 1969 to February 25, 1970 survey period. The maximum at footing number 14 remained at 3.0 cm.

It is interesting to note the settlement recorded during the seventy day period of live load application, the change in load history being clearly reflected by the three readings taken at that time.

The variation of total applied load and settlement with time for several Avord Arms pile groups is shown in Figs. 2.13, 2.14 and 2.15. The estimated nominal contact pressures are given in brackets. The maximum settlement recorded to May 29, 1970 is 3.2 cm. for pile groups 8, 9 and 10. Again it is obvious that the settlement response to increases in foundation loading is very rapid.

It deserves comment here that despite the similarity

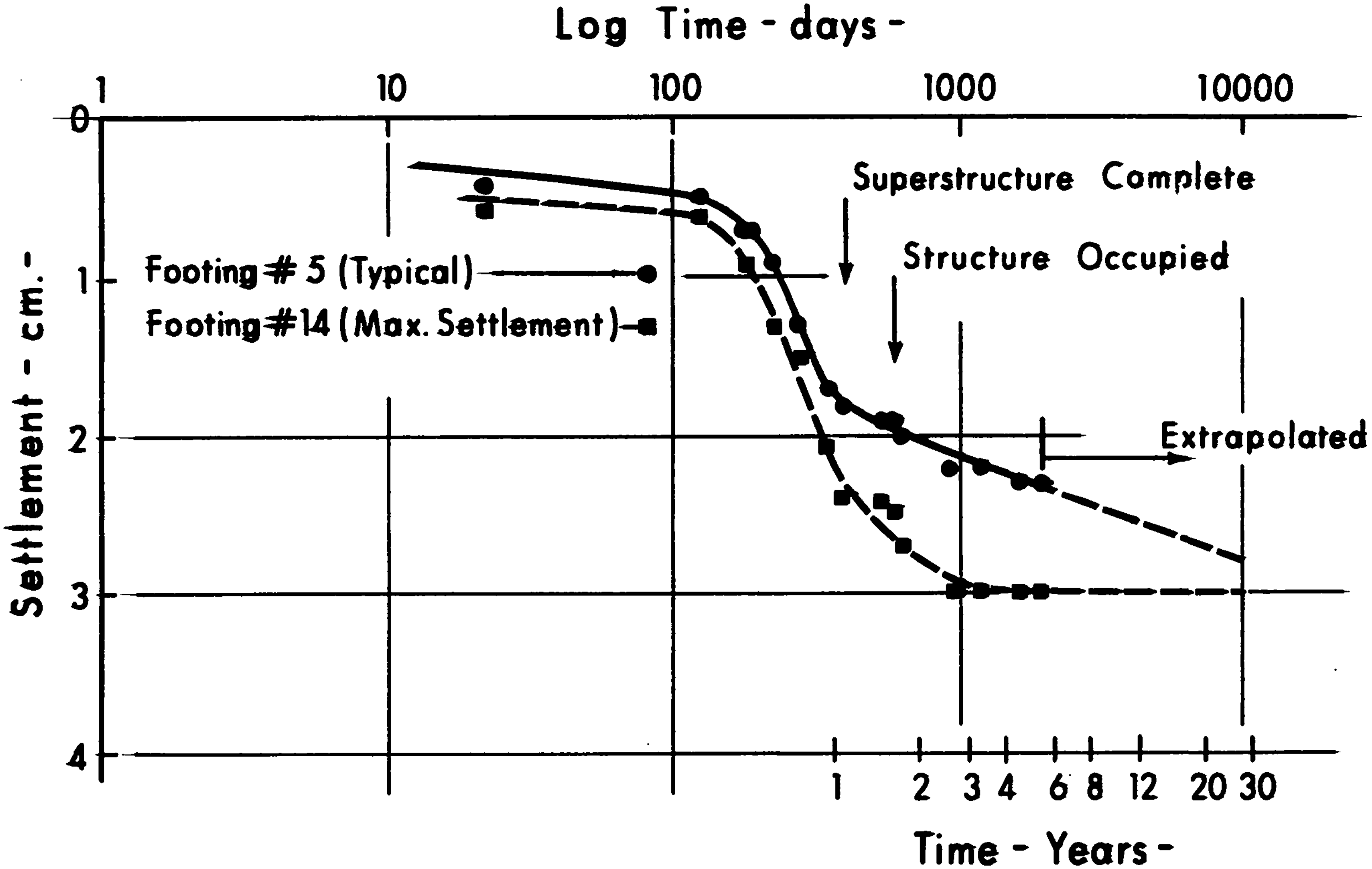


Fig. 2.12 Observed Settlement With Time
CN Tower

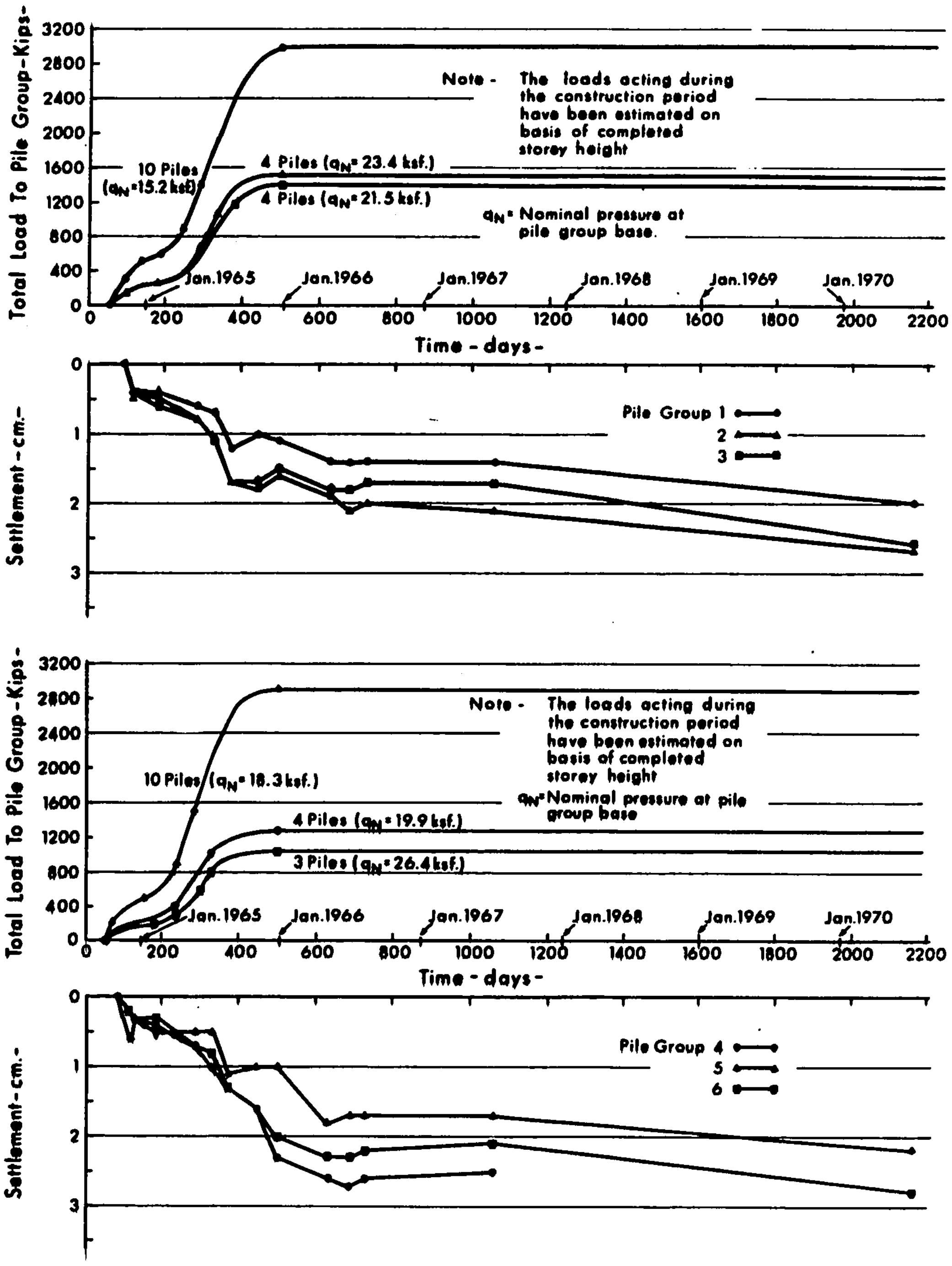


Fig. 2.13 Load and Settlement History, Avord Arms Building

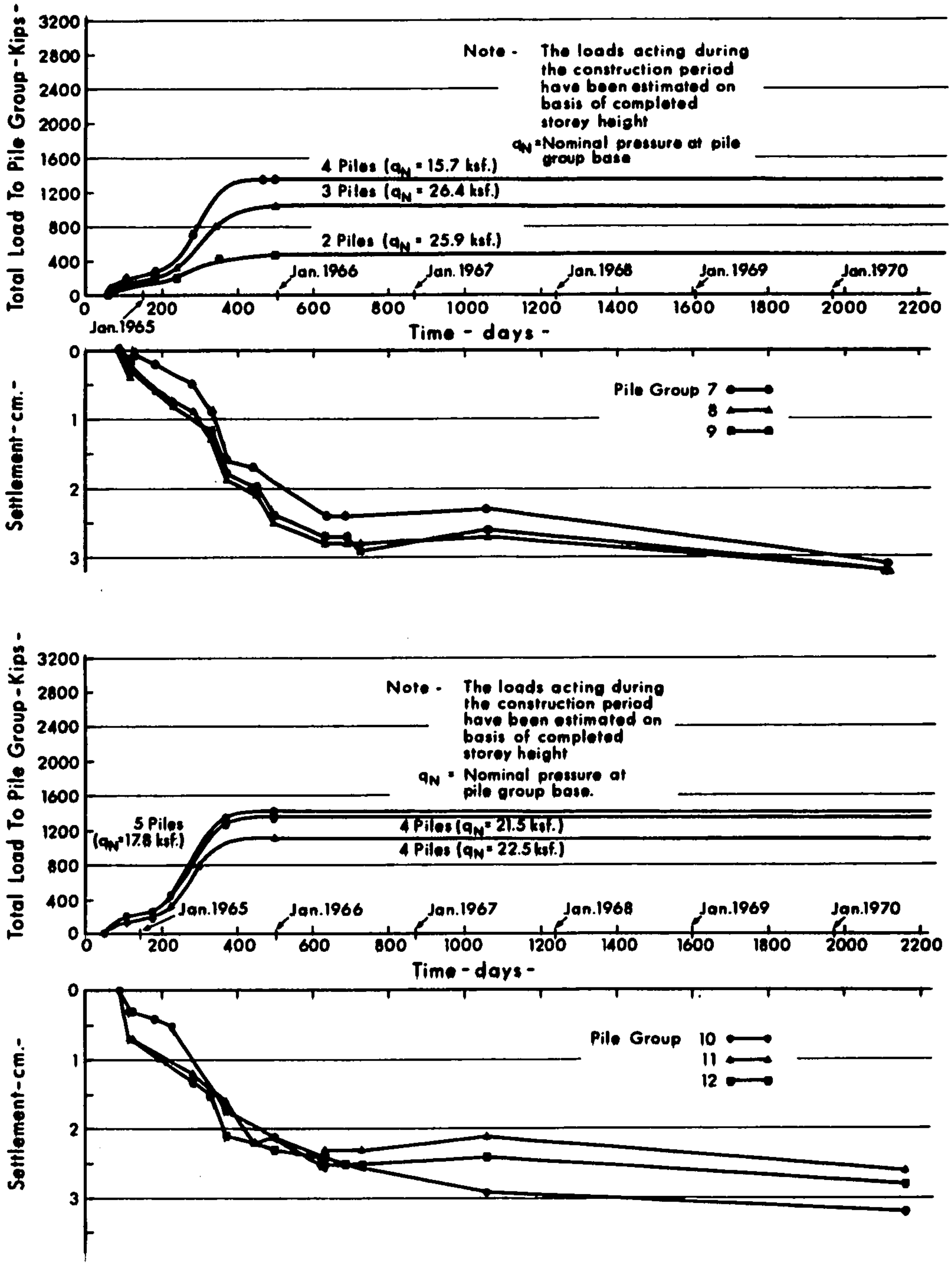


Fig. 2.14 Load and Settlement History, Avord Arms Building

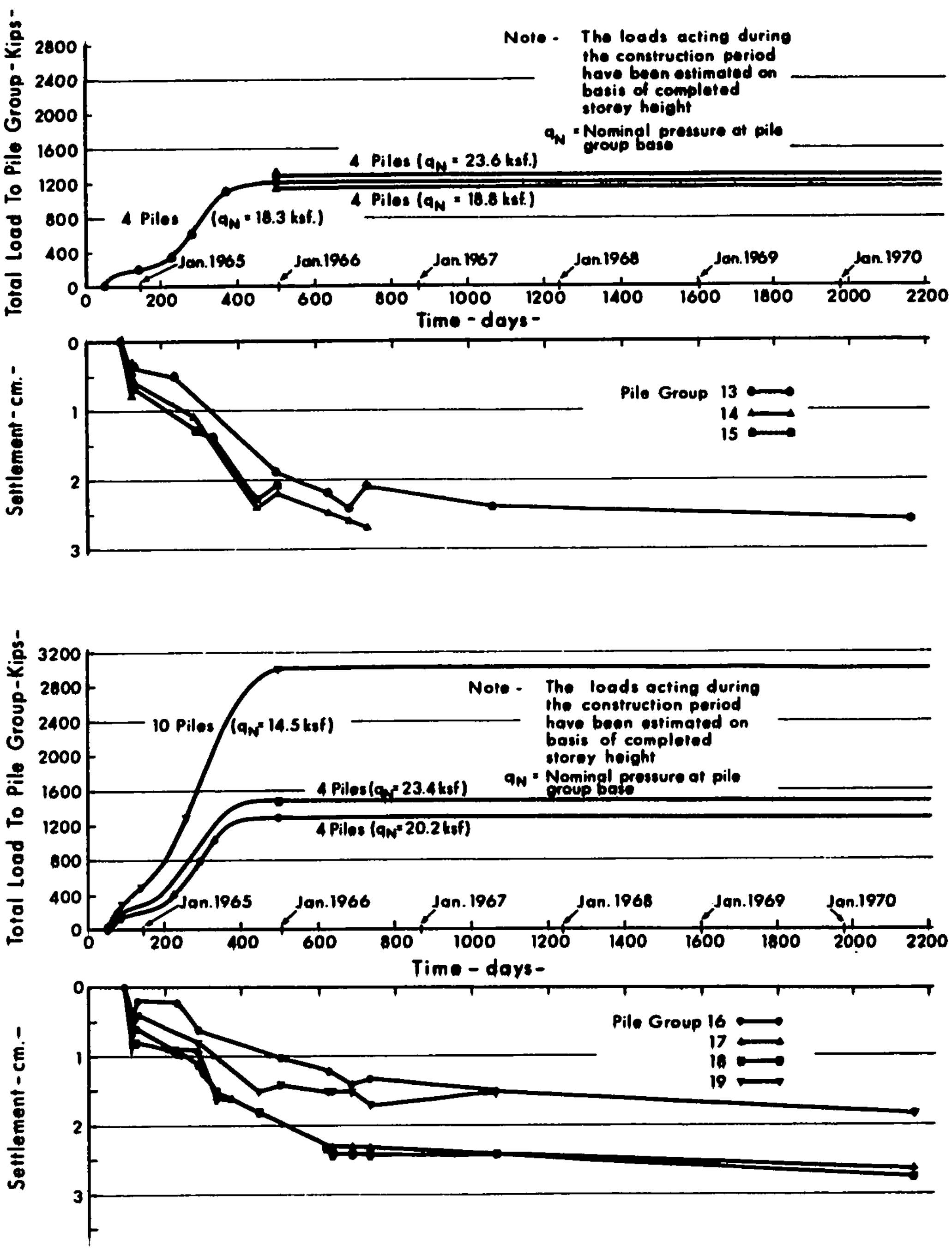


Fig. 2.15 Load and Settlement History, Avord Arms Building

in the maximum settlement recorded at the CN Tower and Avord Arms there is an appreciable difference in average contact pressure; that is, the building load over the total contact area.

In the case of the CN Tower the footing sizes are precisely known. For the Franki pile foundation they are not as the extent of the densified bulb of soil is uncertain. If one takes a particular pile group and encloses the pattern with straight lines always at a distance of 1.5 times the pile shaft radius away from pile centers (the area thus bounded is virtually identical to the pile cap), then one can calculate the end bearing pressure for the pile group. Calculation of the nominal pressure by this procedure is representative as the concrete volumes required to form the "spherical" pile bulbs indicate that no bulb had a radius greater than 1.5 the radius of the pile shaft. As well, for most pile groups the spacing between individual piles was about 3 to 4 feet or about 2 pile diameters. This pile spacing is not unusual in pile foundations.

Table 2.5 gives pertinent data for comparison of the two foundation types. The foundation contact pressures thus calculated for the Avord Arms vary between 12.0 ksf and 26.5 ksf, the average being 19.1 ksf. This differs considerably from the footing contact pressures computed for the CN Tower, the maximum there being 13.0 ksf.

The larger contact pressure variation at the Avord Arms is indicated in Fig. 2.13, 2.14 where large pile groups

TABLE 2.5

PRESSURE SETTLEMENT DATA - CN TOWER, AVORD ARMS

	Average Contact Pressure at end Construction Period (ksf)	Overburden Pressure Removed by Excavation (ksf)	Net Pressure (ksf)	Pressure on Gross Building Area (ksf)	Maximum Observed Settlement to Spring 1970 (cm.)	Settlement Observed During Loading Period (% of Total)
CN Tower	11.3	3.5	7.8	7.9	3.0	87
Avord Arms	19.1	-	19.1	4.6	3.2	80

which carry the greatest load settle the least. Since the greatest portion of settlement of an individual pile is generally due to the load carried by that pile itself, a reduced pile group contact pressure usually results in reduced settlement.

The settlements for four pile groups are plotted in Fig. 2.16 against the log of time. The extrapolated curves indicate that if the secondary compression continues, most pile groups will settle an addition 0.5 cm. over the next twenty-five years. This corresponds to a slope of .65 cm. per log cycle of time in days and is the same as the value found for some CN Tower footings.

Several case records exist which indicate that the preceding observations are typical for buildings founded on over-consolidated soils. A statistical study of case records by Som (1968) for thirteen structures founded on over-consolidated clays and silty clays, indicated that, on average, as much as 58% of total settlement occurs during construction. The percentage for individual cases varied between 31.5 and 80.7 per cent. Since most observations reported in these records were started after construction had progressed to a certain extent, the percentages quoted above could actually be greater.

2.8 Mechanism of Settlement

Since the response to loading was so rapid it was considered worthwhile to plot the CN Tower footing contact pres-

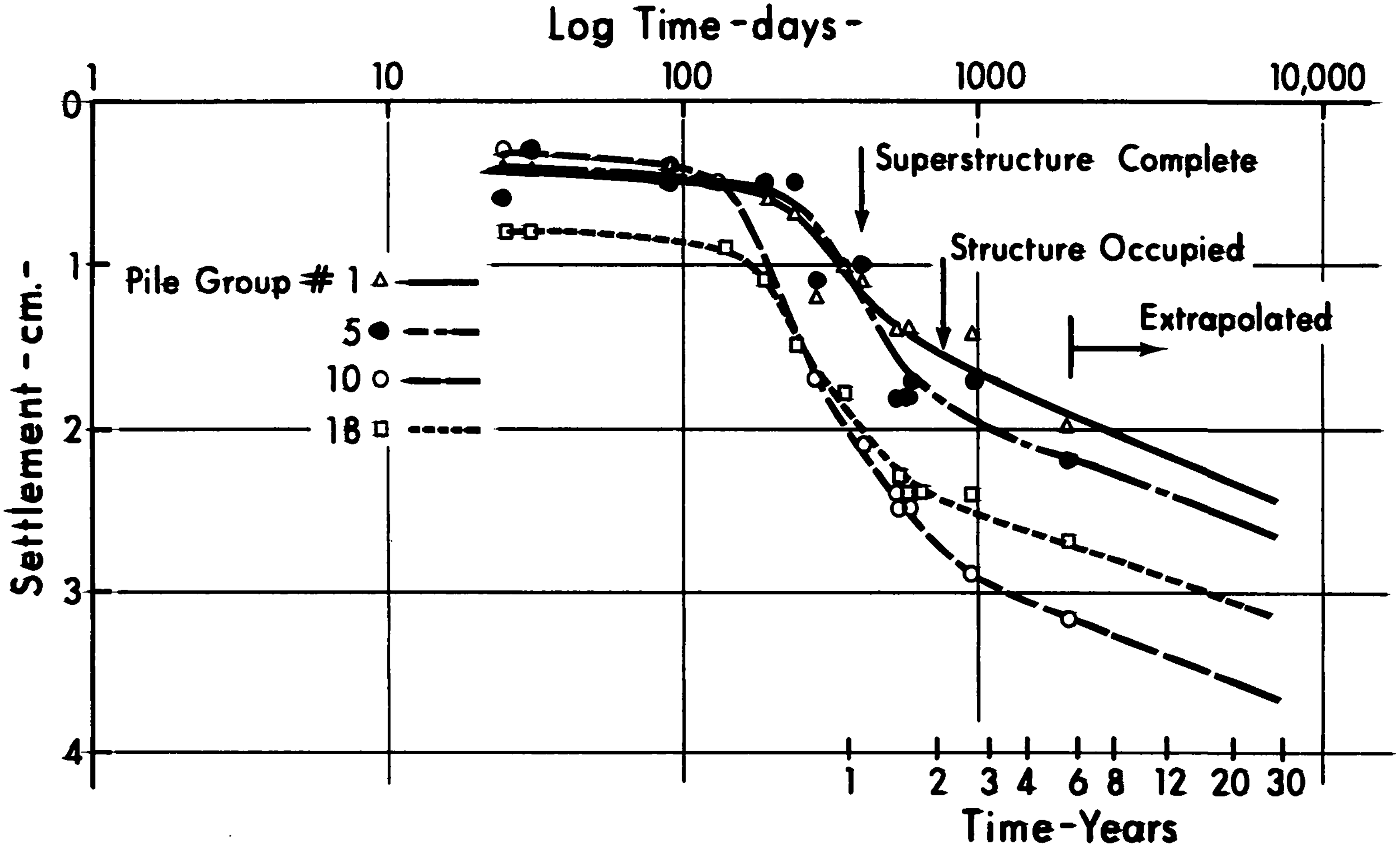


Fig.2.16 Observed Settlement With Time Avord Arms

sures against observed settlements. The parallelism of curves in Fig. 2.17 suggests that there is a relationship between footing pressure and settlement. The slight curvature evident is thought to be due to the inherent non-linearity of the soil response. However, it must be recognized that any time dependent settlement taking place concurrently with the instantaneous response could also produce this curvature. The curves defined in these diagrams are directly analogous to a plot of settlement versus load in a plate bearing test, the only difference being the effect of multiple adjacent footings.

Corner footings and interior footings would by virtue of their relative locations undergo different amounts of settlement. A somewhat superficial method was devised for assessing the effect of footing position. For particular increments of load following the initial settlement reading, the average of the vertical stress distribution below a particular footing, computed using the Boussinesq solution, was plotted versus the corresponding settlement. The average would be greater for interior footings due to the effects of superposition, the settlement tending to be greater here as well. Fig. 2.18 shows that all the settlement values fall within a relatively narrow band, illustrating that the settlement of the building is closely related to the combined effects of footing load and footing location.

To assist in the interpretation for a possible settlement mechanism, normalized plots of the footing pressure and

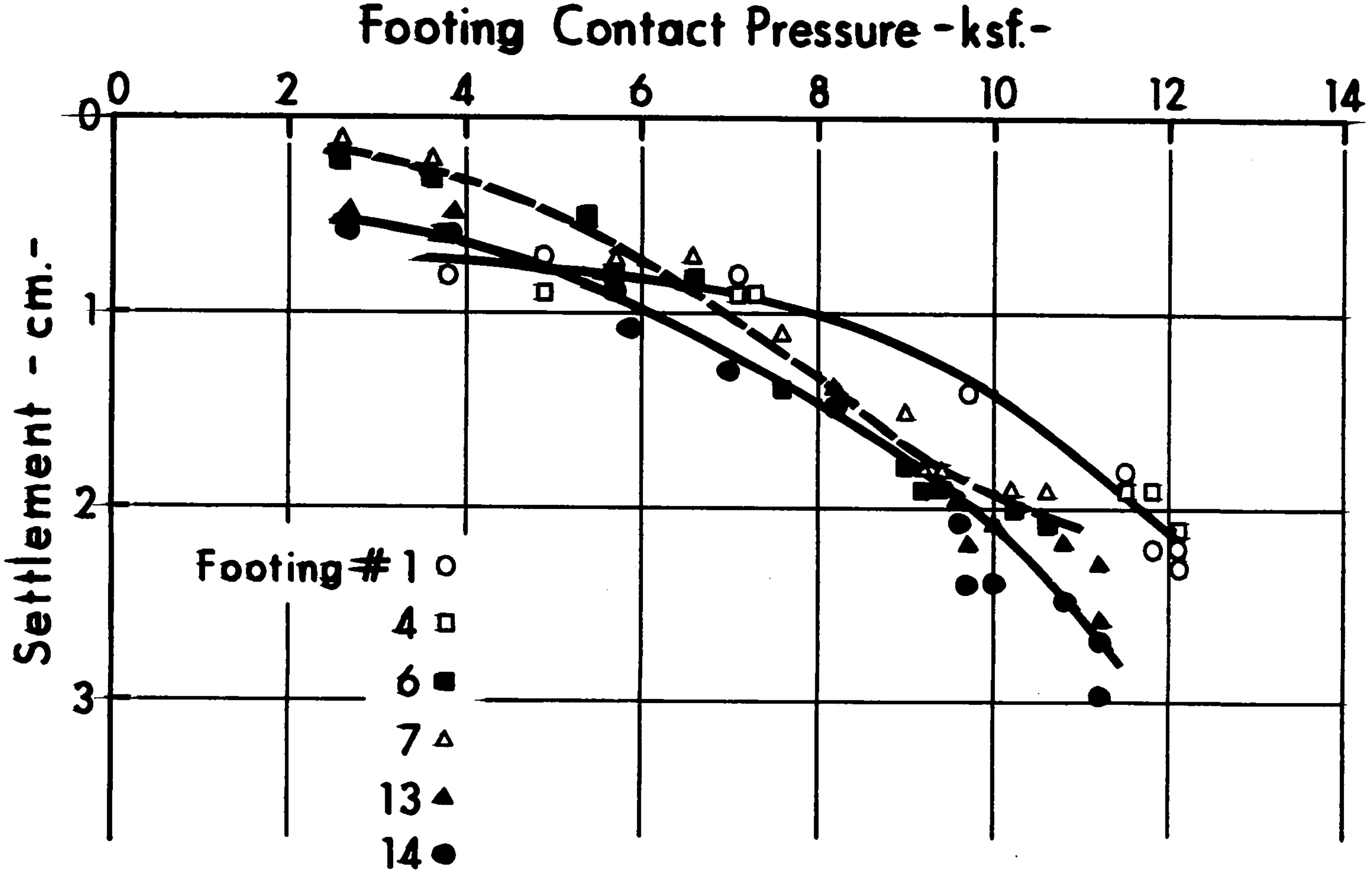
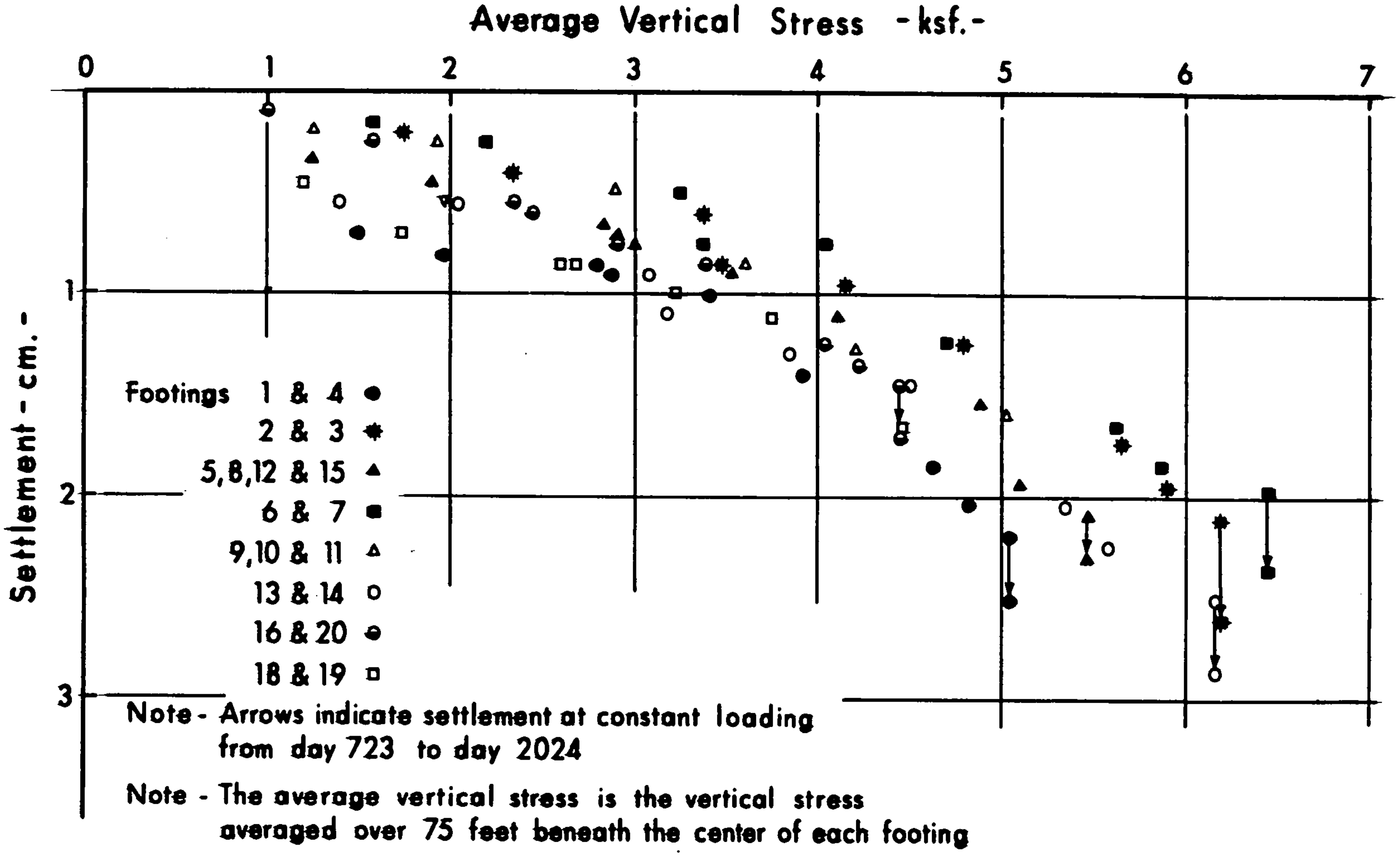


Fig.2.17 Footing Contact Pressure and Settlement CN Tower



Note - Arrows indicate settlement at constant loading from day 723 to day 2024

Note - The average vertical stress is the vertical stress averaged over 75 feet beneath the center of each footing

Fig. 2.18 Relationship Between Settlement and Distribution of Vertical Stress CN Tower settlement record were prepared for the CN Tower construction period. The example shown in Fig. 2.19 for footings no. 13 and no. 14 indicates by virtue of the small deviation about the 1:1 ratio line, that the mechanism of settlement is one of elastic behaviour.

A relatively high initial settlement at light loads occurred at several footings. Although some of this may be due to survey error it is more likely recompression of soil previously heaved by rebound. This recompression would perhaps encompass such phenomena as the closing of fissures opened during excavation. The behaviour is described more fully in a later chapter dealing with rebound observations at the AGT Tower site.

2.9 Analysis of Settlement Data

The equation given by Steinbrenner (1934) and Terzaghi (1943) for the elastic settlement of the corner of a rectangular uniformly-loaded area located on the surface of a semi-infinite solid was used to determine the foundation moduli compatible with the observed settlements. The equation, which is:

$$\Delta = \frac{q(1-\mu^2)BI_\Delta}{E} \qquad (2.1)$$

was programmed on a computer.

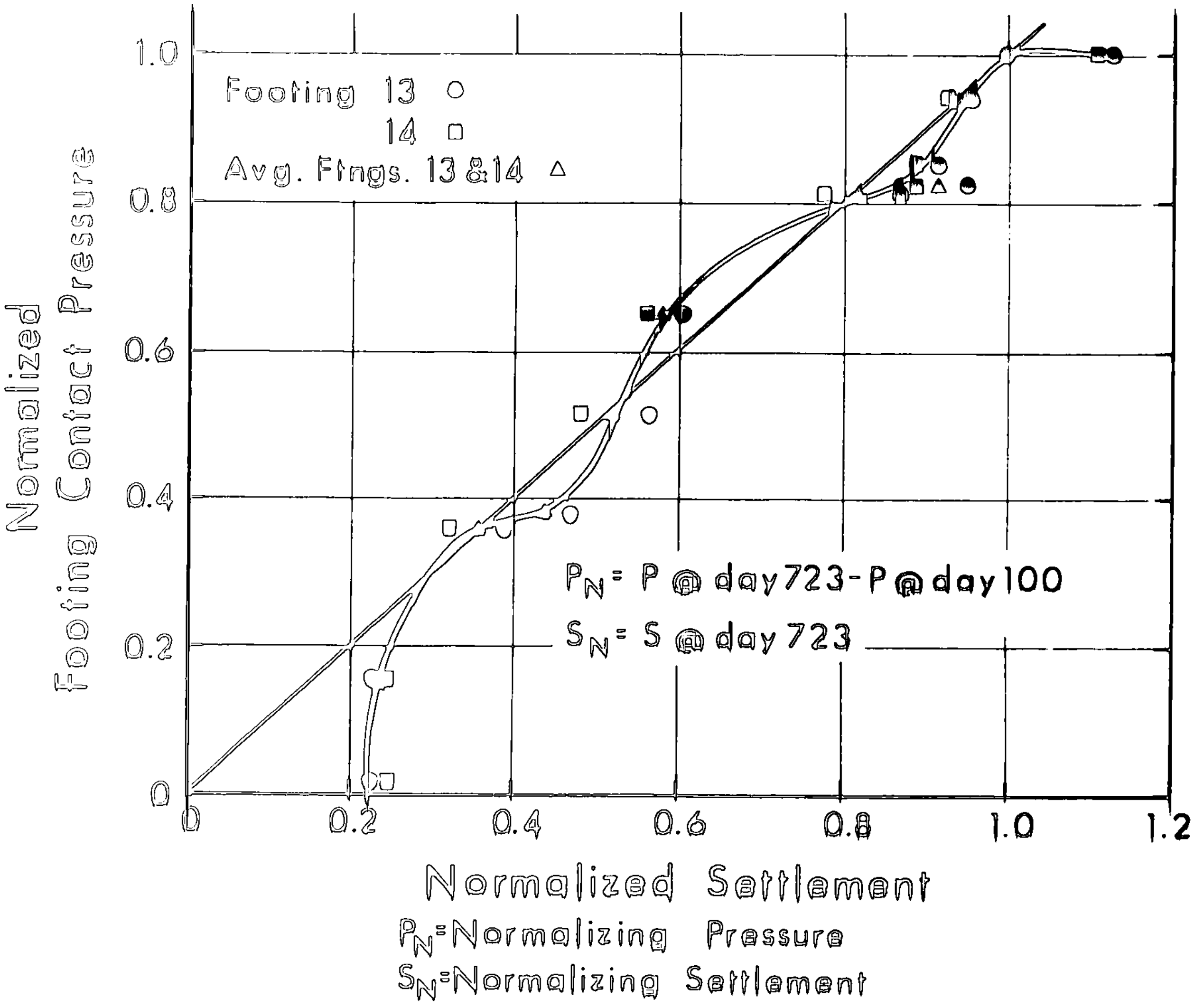


Fig.2.19 Normalized Footing Pressure and Settlement CN Tower

Δ = settlement at the corner

q = uniform footing pressure

μ = Poisson's ratio

B = least dimension of rectangular area

I = influence value for settlement; dependent on footing geometry.

E = modulus of elasticity.

Equation 2 allows computation for the settlement of a single point located inside, outside or on the boundary of a rectangular loaded area (Terzaghi, 1943).

$$\Delta = \frac{q(1-\mu^2)}{E} (B_1 I_{\Delta 1} + B_2 I_{\Delta 2} + B_3 I_{\Delta 3} + B_4 I_{\Delta 4}) \qquad (2.2)$$

The general superposition technique employed here requires definition of 4 area widths B and 4 influence values I_Δ. When a foundation is loaded by 21 CN Tower footings, superposition to include load dispersion effects requires 84 widths, influence values and area loadings to determine the settlement at a single point. The task becomes truly formidable when settlements are desired at all footing centers and at additional other locations.

The computer solution only requires input of footing loads and geometry, namely their centroidal coordinates and dimensions. Computations using a variety of footing load increments were thus easily performed. The procedure followed was to solve for the elastic settlement using values of Poisson's ratio of $\mu = 0.3$, $\mu = 0.4$ and $\mu = 0.5$ and a series of elastic moduli E.

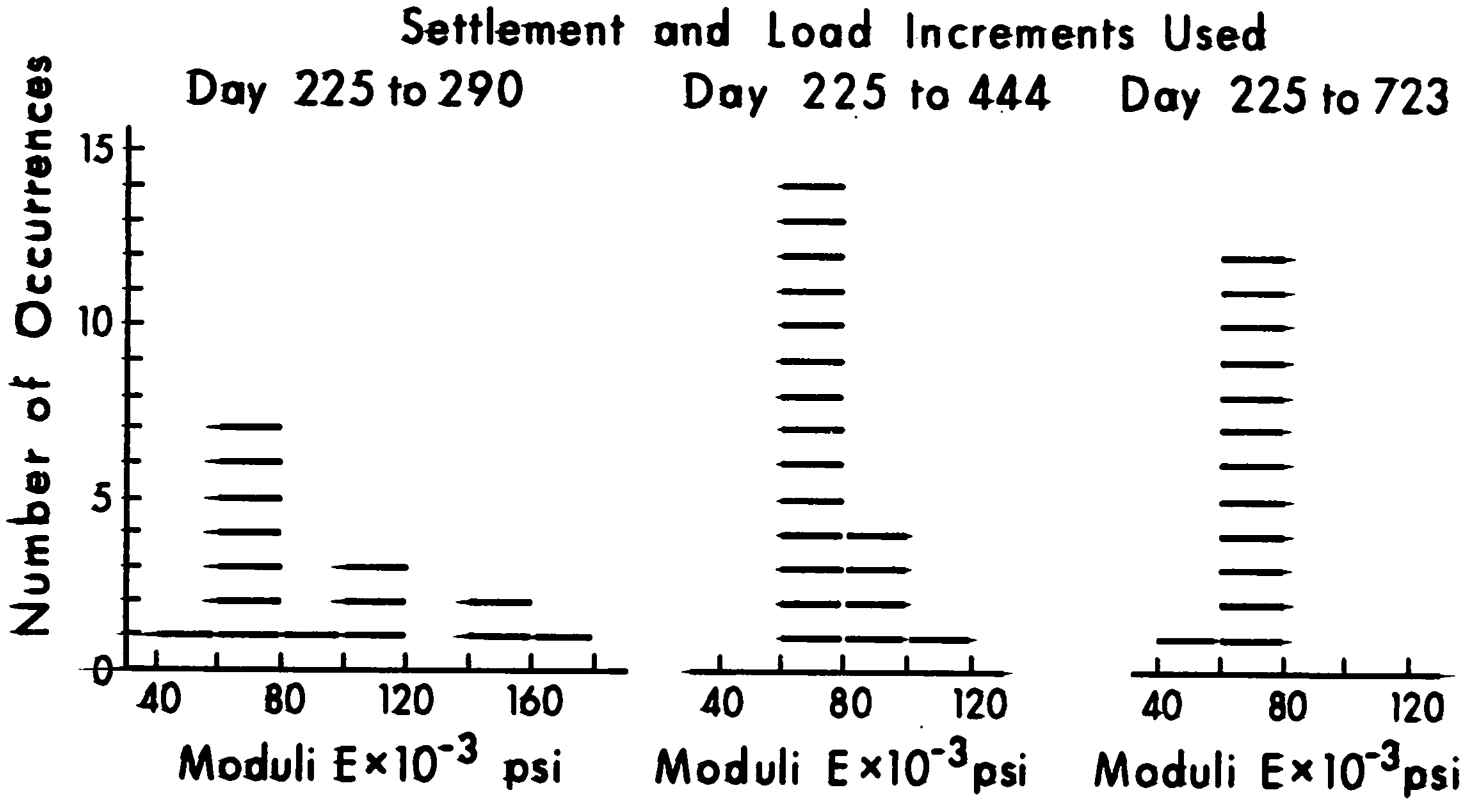
Starting at day 225, when the footing contact pressures first exceeded the original overburden pressure, the load increments used for these calculations varied between day 283 and day 723, thus utilizing only the "net settlement" that occurred subsequent to the reloading portion of the load history. For each load increment the calculated settlements were compared with the actual settlements. By interpolation an elastic modulus was obtained for every footing for which settlement data was available. Three sets of moduli were derived, one set for each Poisson's ratio. Typical histograms are shown in Fig. 2.20 and illustrate the distribution of results.

For the settlement increment to the end of construction and full occupancy, the mean values and standard deviations are given for thirteen footings in Table 2.6.

TABLE 2.6

DERIVED MODULI - DAY 225 to
DAY 723 - CN TOWER

μ	E (psi)	Stnd. Dev. (psi)
0.3	77,400	± 6800
0.4	71,300	± 6000
0.5	64,600	± 5300



Mean = 92,600 psi
Std. Dev. = ±33,900 psi
(a)

= 77,100 psi
= ± 9,800 psi
(b)

= 71,300 psi
= ±6,000 psi
(c)

Fig. 2.20 Histograms of Derived Moduli for Poisson's Ratio = 0.4 CN Tower

As might be expected intuitively from observation of the narrow band of settlement points previously indicated on Fig. 2.18, the standard deviation is small, being only about ± 9% of the derived moduli.*

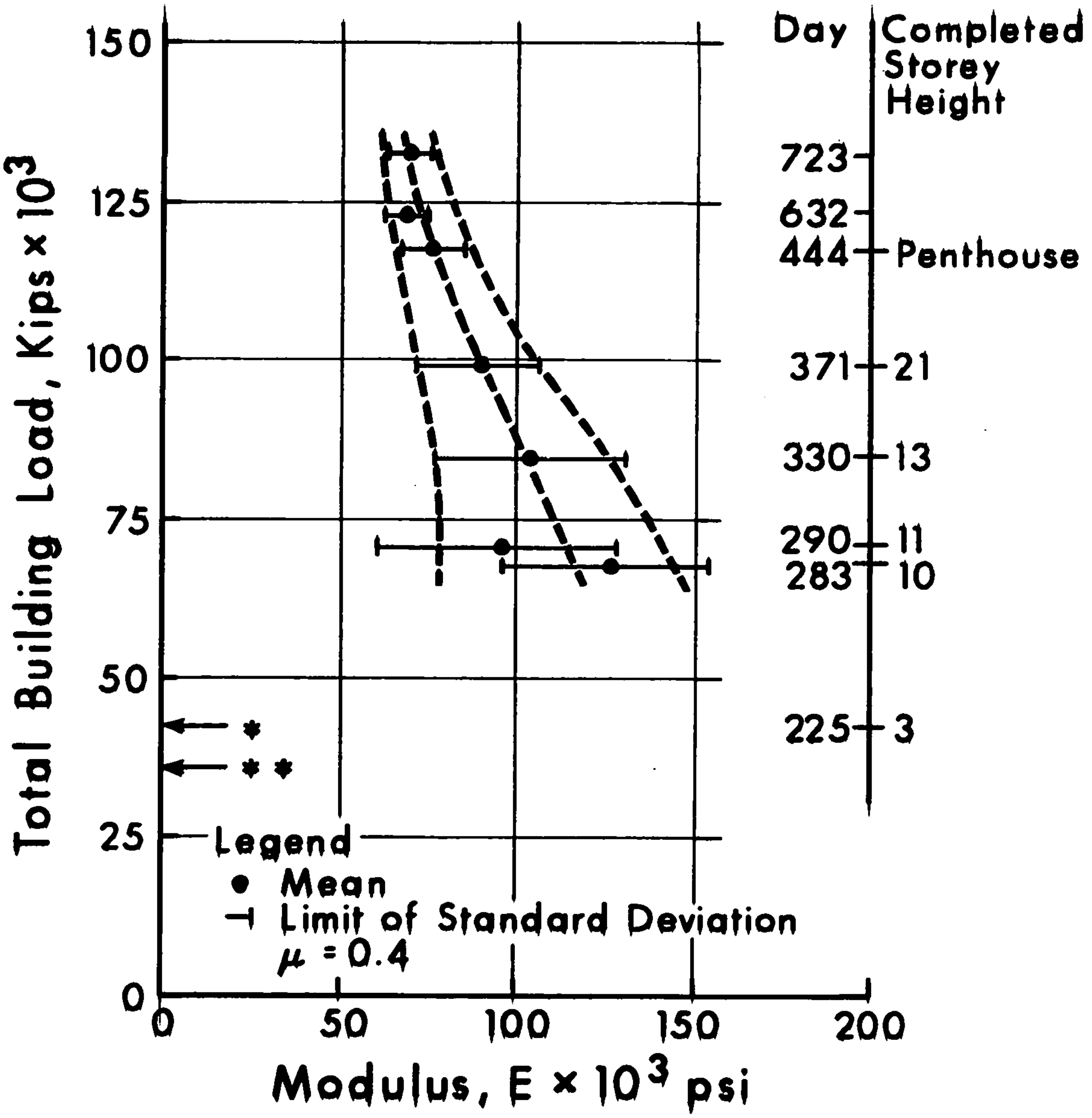
The variation of derived moduli with the total weight of the CN Tower is shown in Fig. 2.21. For convenience the applicable dates and storey heights are indicated in the same diagram. Each plotted point represents the average of the observed moduli calculated for all the footings for which settlement data were available, usually thirteen in total.

The sensitivity of the settlement-load relationship to slight load variations and survey error, when the loads and settlements are small, is reflected by the resulting large standard deviation for the set of derived moduli. This is illustrated in Fig. 2.21 by the broad band width at low building loads. The small standard deviation shown at full building height is indicative that the elastic settlement equation accounts for essentially all the differential settlements observed at the CN Tower.

Of considerable importance is the reduction in derived

---

* The moduli calculated at the CN Tower correspond closely to the initial moduli of elasticity derived from in-situ plate bearing tests by Klohn (1965) for the case of $\mu = 0.3$. The moduli derived by Klohn from settlement data using the approximate solution for an elastic layer resting on a rigid base, are approximately double the CN Tower moduli. This difference is perhaps not surprising as the Edmonton till has a higher natural moisture content, lower density and lower unconfined compressive strength.



Note - * Load Datum Used in Calculations
**Weight of Excavated Soil (Approximate)

Fig.2.21 Derived Moduli With Increasing Building Loads CN Tower moduli, considered to represent non-linear elastic behaviour of the foundation soil with increasing load. The curve is analagous to the secant modulus of a stress strain curve, the modulus being the slope of a line originating at the overburden pressure and passing through points of increasing stress level.

The Avord Arms having no basement, does not undergo rebound and is thus subjected to "net settlement" only. Since the pile group loads were not accurately known during the construction period, only a single set of moduli was derived for a period ending at approximately the time of full occupancy of the building.

Utilizing the same computer program as for the CN Tower analysis, the averages for 17 pile groups are given in Table 2.7. These moduli are lower than the values computed at the CN Tower.

TABLE 2.7

DERIVED MODULI - DAY 91 to
DAY 632 - AVORD ARMS

μ	E (psi)	Stnd. Dev. (psi)
0.3	43,830	± 9650
0.4	40,590	± 8900
0.5	36,200	± 7920

With a modulus that is independent of stress level, it can be shown that for constant load over a given area the settlement computed by the Steinbrenner (1934) solution is virtually independent of footing sizes within the area provided the footings are reasonably close to one another. However, since real soils generally have moduli which decrease with increasing stress level, then at constant load a reduction in footing size results in increased settlement.

This effect is illustrated in Fig. 2.22 where the derived moduli for both the Avord Arms and CN Tower is plotted against the foundation contact pressure in excess of the overburden pressure at footing level. It can be seen that data for the Avord Arms is consistent with that observed for the same soil at the CN Tower. The somewhat larger standard deviation for the Avord Arms average modulus reflects the larger variation in pressures acting at the pile group bases.

The negative pressures represent the difference between the smaller footing contact pressure and the original overburden pressure acting at footing elevation. Although the data is limited, the large settlements observed at low footing pressures indicate that the compressibility of the fissured till is greater in recompression until the original overburden pressure is reached.

2.10 Comparison Laboratory and Empirical Data

For the loading range 3.6 ksf to 11.3 ksf the derived

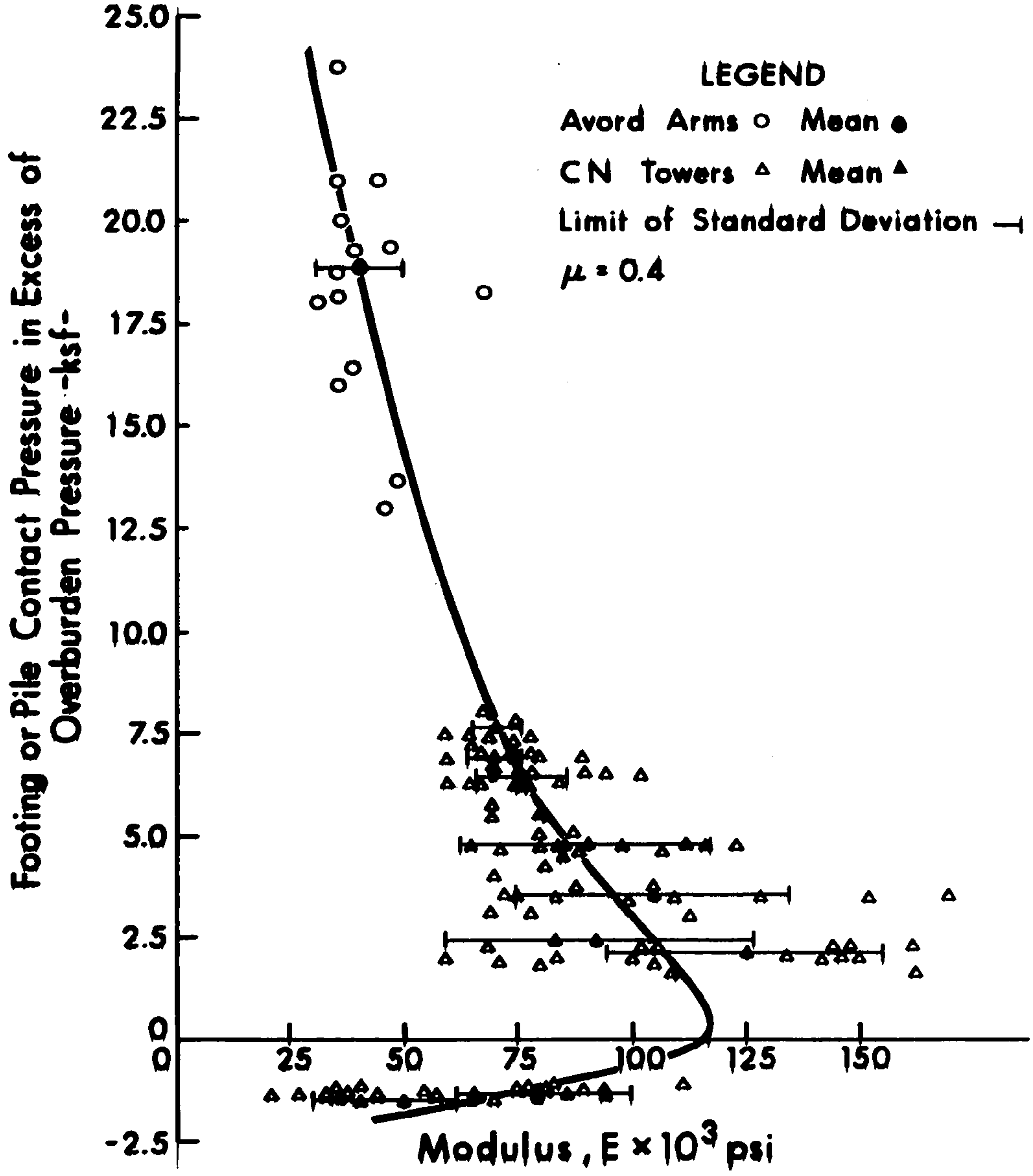


Fig. 2.22 Variation of Modulus With Contact Pressure

moduli for the CN Tower equals 71,300 psi for $\mu = 0.4$. The corresponding laboratory moduli from consolidation tests show that the modulus equals approximately 2500 psi.

Conversion of these results to one-dimensional moduli of volume compressibility is possible by means of the relationship:

$$m_v = \frac{\varepsilon_1}{\Delta\sigma_1'} = \frac{1}{E} \cdot \frac{(1+\mu)(1-2\mu)}{(1-\mu)} \tag{2.3}$$

For $\mu = 0.4$, the empirical m_v value is .000045 ft^2/kip as compared with approximately .0013 ft^2/kip for the laboratory data.

The empirical moduli account for approximately 80 per cent of the settlement observed to date at the CN Tower. For long term conditions this percentage would perhaps be be somewhat lower to account for the additional 0.5 cm. average settlement expected over the next 25 years.

Applying a factor of 0.75 to the derived moduli reduces the discrepancy between field and laboratory only slightly. The ratio of field to laboratory moduli is about 28, and about 21 if the factor 0.75 is applied.

This ratio would be somewhat different if laboratory compression results were used. The ratio would be larger for the confined compression data and equal about 50. The cell pressures used in the consolidated undrained tests exceeded overburden pressures and thus the ratio of field to laboratory moduli of 5 is only an approximation.

This complete lack of agreement between field and laboratory derived compressibilities for the over-consolidated till clearly demonstrates the need for empirical data to accurately predict settlements in the field.

2.11 Procedure for Settlement Analysis from Empirical Data

Since the over-consolidated till exists over a great part of the downtown Edmonton area, the soil properties derived are directly applicable in design.

Consider a hypothetical building founded on unsaturated till and possessing a full basement sixteen feet below ground surface. The allowable bearing pressure designated by the soils consultant is 13.0 ksf. The bulk unit weight of the excavated soil is given as 125 pounds per cubic feet.

Knowing that the water table is at considerable depth near the bedrock surface, the total stress change at footing level is .125 x 16 = 2.0 ksf. Since the excavation will heave, resulting in appreciable recompression upon loading, this portion of settlement is computed by using the modulus taken from Fig. 2.22 for a pressure of -2.0 ksf.

The curve at -2.0 ksf is fairly flat so interpolation requires some judgment. If the excavation is open for a considerable period of time, say four months or more, a modulus of 20,000 psi could be chosen. If construction is rapid with no delays, a modulus of 35,000 psi would be reasonable. Since the till is unsaturated, Poisson's ratio can be taken as 0.4, thereby anticipating volume change. Work by Crawford and Burn (1962) and Klohn (1965) on tills, suggests that this value is appropriate. In any event, the computations are not sensitive to the choice of Poisson's ratio. The choice of modulus will not greatly influence the final settlement calculated, as this settlement is only a small proportion of the total in cases where the excavation is shallow.

The computer program found in Appendix H can be used to perform the necessary calculations. The input required is footing geometry and loads and a soil modulus and Poisson's ratio. Settlements are generated at all footings, as well as other specified points.

The contact pressure in excess of the overburden pressure is 13.0 - 2.0 = 11.0 ksf. Again using the data of Fig. 2.22, the corresponding deformation modulus is 57,000 psi. Specification of Poisson's ratio as 0.4 allows calculation of the "net immediate settlement". The loads now used in the analysis are those in excess of the overburden pressure.

Subtotalling of the two sets of settlements yields the "immediate settlement" of the structure during its construction period to full occupancy.

Long term settlements can be estimated in two ways. The log of time plots indicate that secondary settlements take place at a slope of approximately 0.5 cm. per log cycle of time in days. Referring to the time scale adopted in

Fig. 2.12 an estimate for the long term movement can be made.

The data from the Avord Arms suggests that the slope of 0.65 cm. per log cycle of time in days is true for larger footing pressures. The CN Tower data seems to indicate that several footings have virtually stopped settling. For this reason the intermediate slope of 0.5 cm. per log cycle of time in days was adopted. The calculations are not sensitive to the slope adopted, since the time dependent movements are only a fraction of the total.

The second procedure is by simply using the observed construction settlements in terms of the percentage of the total. For the Avord Arms the value was 80 per cent and applied to "net settlements" only. In the preceding section this value was reduced to 75 per cent. To be conservative, the calculated "net immediate settlements" are divided by 0.75 and, the results added to the "recompression immediate settlements" to arrive at the long term footing displacements.

For a building founded on piles and having no basement, the settlement calculations are simplified, as the applied loads are all in excess of the overburden pressure. For the hypothetical building, a deformation modulus of 50,000 psi from Fig. 2.22 is used corresponding to the allowable pressure of 13.0 ksf.

2.12 Summary

The preceding analyses can be summarized as follows:

(a) The settlement response to loading is rapid for structures founded on the over-consolidated till. The settlements experienced during the construction period comprise about 75% of the total movements and can be calculated using elastic theory.

(b) Elastic properties of the foundation soil, as determined in conventional laboratory tests, give rise to unrealistic settlements and thus cannot be used to perform the required computations. For example, the use of laboratory consolidation data in the prediction of settlements at the CN Tower would yield settlements approximately 25 times greater than those actually observed; that is, 75 cm. rather than 3.0 cm.

(c) The empirical foundation moduli derived from settlement observations on two prototype buildings are stress dependent. The mean empirical elastic moduli derived from settlement observations vary from 105,000 psi to 30,000 psi as contact pressures increase from 2.5 ksf to 23.5 ksf.

(d) The modulus of recompression to the original overburden pressure can be less than those for compression beyond the overburden pressure.

(e) It would appear that the relative merit of end bearing piles to spread footings for the same till formation can be determined only on the basis of tolerable settlements, allowable footing pressures and consideration of the economy of the foundation type.

(f) There is some suggestion that the increment of long term settlement may be slightly greater for foundations with larger contact pressures.

(g) On the basis of experience at the Avord Arms, there is some suggestion that, especially for confined load areas such as pile bases, allowable bearing pressures on the till can be as great as 18.0 ksf without excessive settlement.

CHAPTER III

THE INFLUENCE OF STRUCTURAL RIGIDITY ON CN TOWER FOOTING LOADS

3.1 Introduction

The analysis of the CN Tower settlement data, as described in an earlier chapter, followed a practice often adopted in design by assuming that the building is perfectly flexible. Thus the basic footing loads as calculated from contributing floor areas are assumed to remain unchanged regardless of any differential deformations to which the structure may be subjected. This is to say that the effects of foundation and structure interaction are ignored.

For many structures perfect flexibility is a reasonable approximation. This is particularly so with low structures or flat plate apartment buildings. In contrast, any frame possessing beams of short span lengths and deep section could be considered as a stiff structure. The inclusion of several shear walls throughout the interior of some reinforced concrete buildings contributes greatly to the overall stiffness.

The CN Tower, described earlier in Chapter II, is a reinforced concrete building which possesses a relatively large stiffness. Thick interior walls between the second and fifth floors support a parking garage cantilevered beyond the main tower section. By virtue of being a multi-storeyed building, the column loads, particularly at lower levels, are large. The required column dimensions cause members to possess great resistance to bending. To a lesser extent, the many floor slabs and beams contribute further to overall building stiffness.

Since the assumption of perfect building flexibility used in the analysis of Chapter II may depart appreciably from reality, it is desirable to investigate the relative flexibility of the CN Tower. Of particular interest is the effect of structural rigidity on the footing loads as the frame undergoes differential distortion.

In this chapter the footing design loads, as calculated conventionally from tributary floor areas and column weights, are termed the basic footing loads. Any change in these basic footing loads due to redistribution as differential displacements take place are called footing reactions. The resulting combination of the basic footing load and the footing reaction is defined as the net footing load. Unless otherwise noted all these forces are vertical.

3.2 Analysis of the Structural Frame

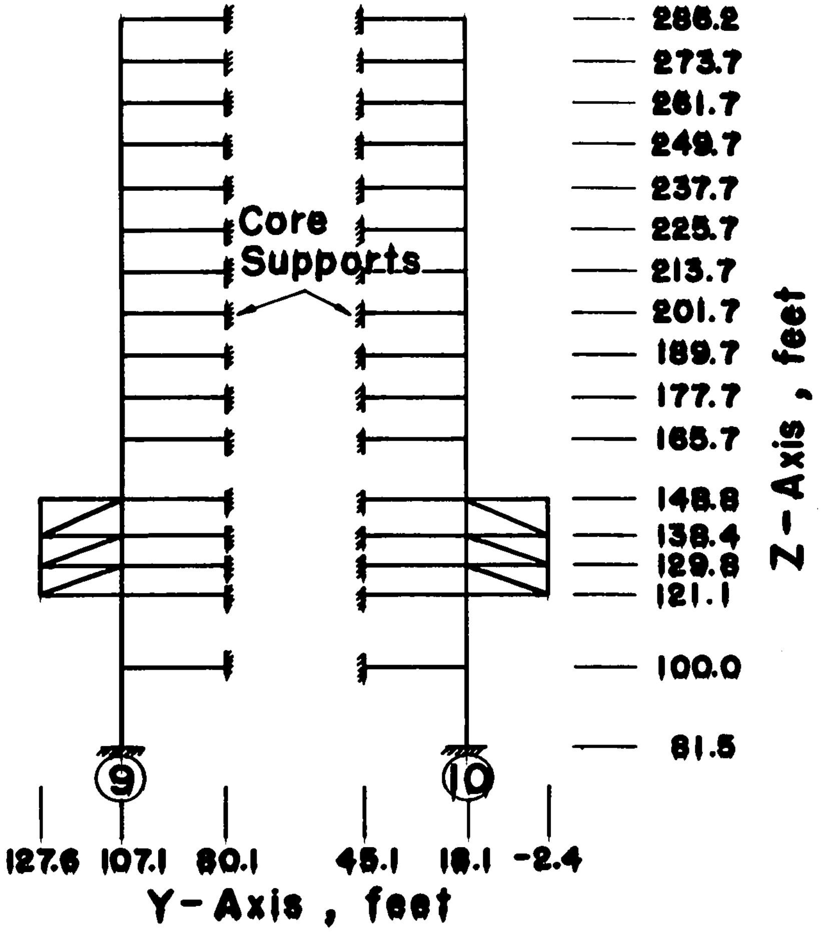
Since the members in the CN Tower structure possess a stiffness of known magnitude, the imposition of any particular set of observed footing settlements on the structure will generate reactions solely due to bending of structural members and thus modify the original estimated basic footing loads.

To solve for these reactions, use was made of Structural Design Language II known by the acronym STRUDL II, developed at the M.I.T. Civil Engineering Systems Laboratory. STRUDL is a general computer program which operates as a subsystem of the Integrated Civil Engineering System (ICES). It can be described simply as a computer language with which a structural engineer can describe a problem, specify its solution procedures and ask for results (Logcher et al., 1968).

Only those techniques particular to the analysis of the CN Tower will be mentioned here. Details of the STRUDL language are described fully in the manuals cited in the references (Roesset and Efimba, 1968).

The complete building was described to the computer as a three-dimensional space frame. All joints were specified in a "global" coordinate system with individual members being defined as a line between any two joints. The CN Tower core wall is a rigid member relative to the surrounding floor slabs, beams and columns. Considering the base of the rigid core fixed vertically, all joints of members framing into it were defined as supports. In this way the true lengths of these members could be maintained. A section through the CN Tower is shown in Fig. 3.1 and indicates the manner in which the space frame is defined in ICES-STRUDL.

The properties of individual members were specified in a "local" coordinate system. For every member type in the CN Tower, the moments of inertia were calculated about the

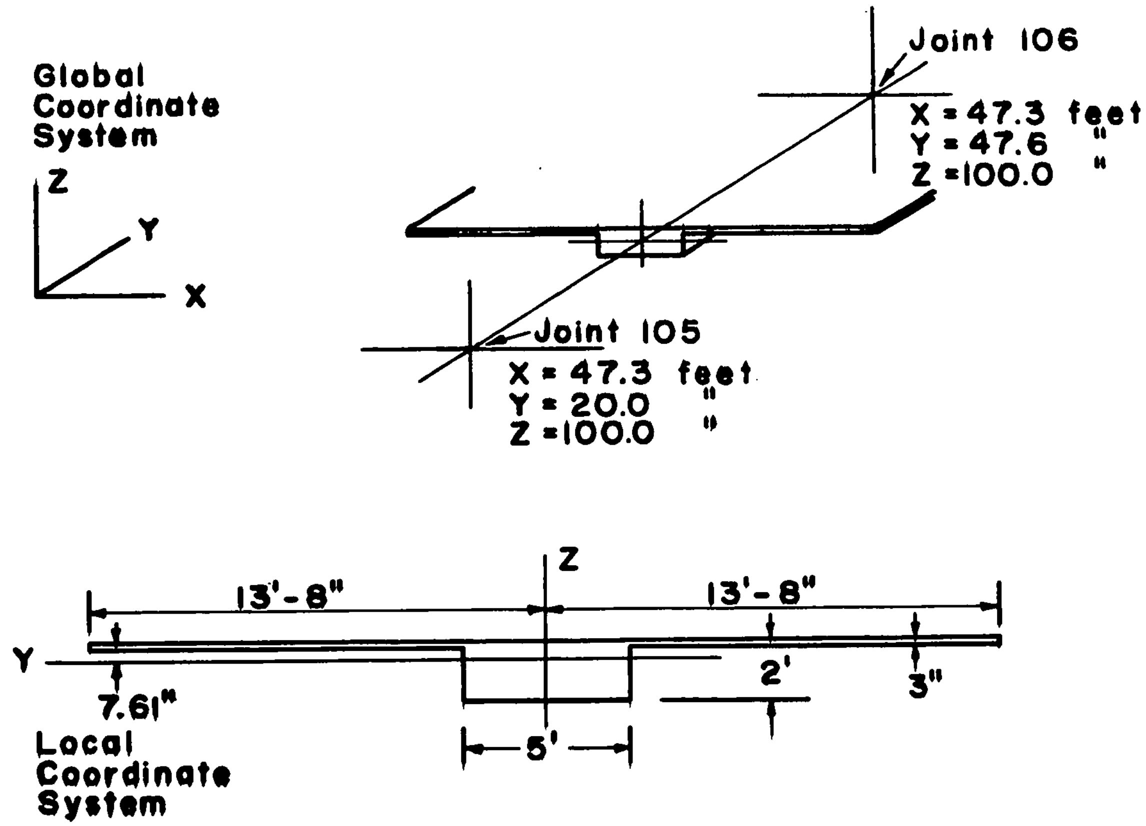


Section — through footings 9 and 10 CN Tower
Trial — No. 5
Storey height — 16 th. floor

Fig. 3.1 CN Tower Space Frame as Defined in ICES-STRUDL two principal axes and the torsional rigidity of the member was determined in accordance with the procedure outlined by Timoshenko (1955). For members which could be classified as thin-walled open sections, the overall rigidity to torsion was considered to be the sum of the torsional stiffnesses of the component sections of the beam (Oden, 1967).

Waffle slabs were considered as beams whose section extended horizontally over the midspan distance to the adjacent beam. Columns or beams framing into walls were considered to have as their section a portion of wall of length four times its thickness. (ACI Standard 318-63, Sect. 906). Non-prismatic members were broken into prismatic segments of specified length, having individual segment properties.

Fig. 3.2 shows a typical member defined in both global coordinates within the frame and individually in local coordinates. The moment of inertia IZ,as indicated in the diagram, was taken to be one order of magnitude greater than those about other axes. Since bending about the Z axis is virtually zero this simplification was made in order to eliminate excessive computation. To eliminate consideration of axial shortening the section area of members was taken to be at least an order of magnitude greater than the actual. This procedure was followed for two reasons, the first being that the bulk of axial shortening would take place due to the basic loading, a loading condition not being applied to the structure in this analysis. The second and more important reason for preventing axial shortening was to simplify the



Section Calculations

Centroid at 7.61" from top surface

Moment of Inertia IY is 131860 in.^{4}

Torsional Rigidity IX is 222500 in.^{4}

Moment of Inertia IZ taken as 9999999 in.^{4}

Section Area taken as 9999999 in.^{2}

Fig. 3.2 Typical Definition of Member and Member Properties modelling of the heavy interior walls.

In the analysis the interior walls were represented by an equivalent truss which would offer the same resistance to differential displacements. As shown in Fig. 3.3 the wall has a diagonal length of d_1 where

$$d_1 = \sqrt{\ell^2 + b^2} \qquad 3.1$$

Application of shear force T gives rise to the shear stress τ where

$$\tau = \frac{T}{b \cdot t} \qquad 3.2$$

By introducing the shear modulus G the shear strain, γ is:

$$\gamma = \frac{\tau}{G} = \frac{T}{b \cdot t \cdot G} \qquad 3.3$$

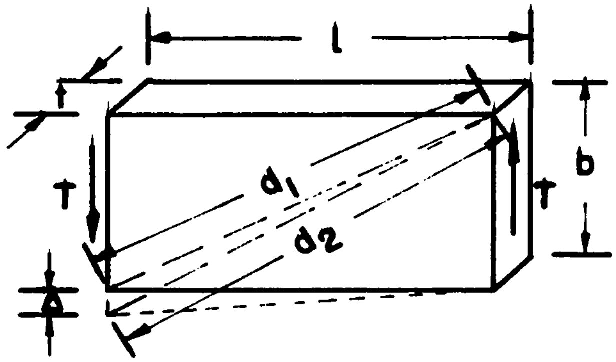
and the vertical displacement Δ at point O as shown in the figure becomes

$$\Delta = \gamma \ell = \frac{T \cdot \ell}{b \cdot t \cdot G}$$

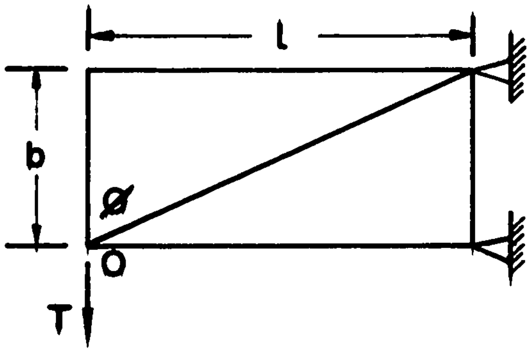
Hence, the new length of the diagonal becomes

$$d_2 = \sqrt{\ell^2 + (b+\Delta)^2}$$

Real Wall



Equivalent Truss



Diagonal has finite cross-sectional area A

$$A = \frac{T\,d_1}{b\,E} \cdot \frac{d_1}{(d_2 - d_1)}$$

where $d_2 = \sqrt{l^2 + (b + \Delta)^2}$

Fig. 3.3 Representation of Wall as Equivalent Truss

With the application of the identical forces on the truss the vertical displacement Δ at O must be duplicated. Only the diagonal has a finite cross-sectional area and thus undergoes axial shortening.

The tension force P carried by the diagonal is

$$P = \frac{T}{\cos \phi} = \frac{T \cdot d_1}{b} \qquad 3.4$$

Note here that cos ϕ is assumed to remain unchanged. Having specified that the displacements of the wall and the truss must be identical, the strain in the diagonal thus equals

$$\varepsilon = \frac{d_2 - d_1}{d_1} \qquad 3.5$$

Introducing the modulus of elasticity E, yields the required cross-sectional area A.

$$A = \frac{P}{E\varepsilon} = \frac{T\,d_1}{bE} \cdot \frac{d_1}{(d_2 - d_1)} \qquad 3.6$$

Since STRUDL requires certain properties for each defined member the imaginary diagonal was assigned moments of inertia equal to unity. In this way the diagonal itself could not offer any appreciable resistance to bending or torsion but could only act in tension or compression to duplicate wall action.

3.3 Definition of Input Displacements

In order to compute reactions due to settlements using the STRUDL program, it was necessary to input the deflections of all the supports of the space frame. However, since displacement measurements were unavailable for some footings they had to be estimated for the purpose of this analysis. Of the nineteen columns instrumented, readings on five had to be terminated shortly after completion of the structure. One column could not be instrumented throughout the entire construction period and thus settlements were estimated for it on the basis of the values recorded at three symmetrically placed positions. Although these estimates were not critical, the absence of readings on the core was.

Since settlements had to be specified for the STRUDL program for the core, these were estimated by plotting the settlement profiles across the core for the footings adjacent to it. Several of these are shown in Fig. 3.4. In most cases, the vertical displacements could be estimated quite easily with an error of about ± .15 cm.

The revealing fact as shown in the figure is that there has likely been a slight tilt of the core in the early stages of construction, prior to the building of the parking garage cantilever area with its heavy interior walls. There is some suggestion that the core tilt of approximately 1/6200 was subsequently reversed slightly by an unknown amount.

Although the estimated tilt is small its effect would be noticeable. Settlement readings on all four sides of the

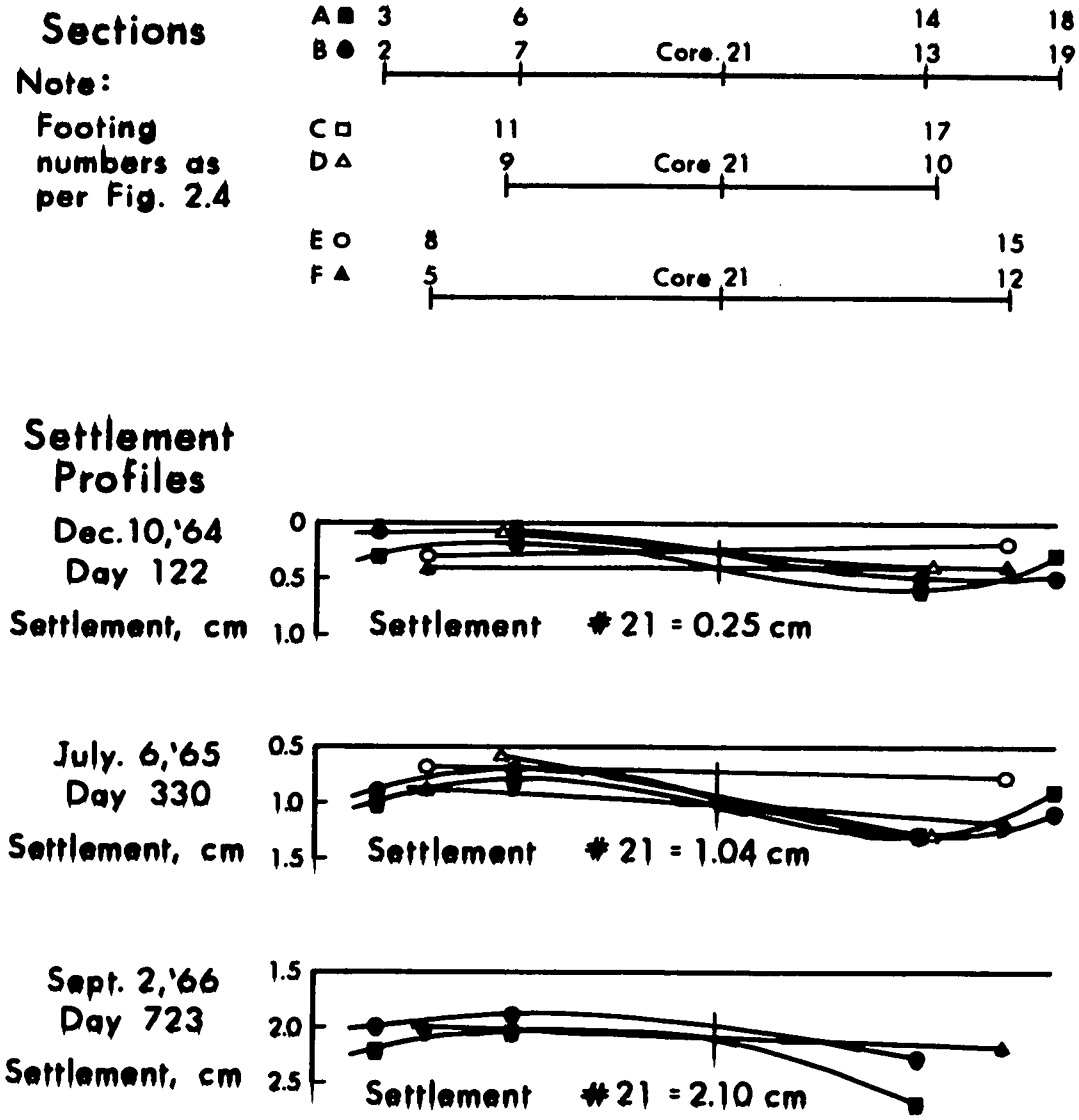


Fig. 3.4 Settlement Profiles Through the CN Tower Core

core would have enabled the author to introduce into the analysis not only vertical displacements but also the horizontal movements at the upper core supports, thus allowing for core tilt.

3.4 Significance of Variation of Structural Rigidity with Storey Height

Before discussing the significance of the vertical reactions generated by STRUDL it is important to realize that the reactions are a function of the structural framework analyzed and the displacements imposed upon it. The structural stiffness of a building increases with the completed height by virtue of the additional members added to the total framework as each storey is built.

In Chapter II it was shown that approximately 80 percent of the settlement recorded to February 25, 1970 at the CN Tower took place during construction. These settlements have been observed on a structure whose rigidity is not constant but increases with time.

This concept is of great importance in assessing the soil-structure interaction and it forms the basis for the discussion of Chapter IV. It is also important in the determination of vertical reactions in the STRUDL analysis, as only the observed settlements for a particular portion of the construction period must be imposed on the portion of structure existing at the completion of the period. This is markedly different from conventional soil-structure interaction analyses, which utilize the structural stiffness of the completed building and assume that at completion of construction the loads are suddenly applied with consequent settlements.

The settlement data and the portion of the structure analyzed is shown in Table 3.1. Thus, for example, the first sixteen floors of the CN Tower would be subjected to the observed increments of settlement between May 27th, 1965 and July 6th, 1965. Even this is an approximation, as ideally increments of settlement for every single storey of increased building height should be used, thus following precisely the history of structural stiffness with time.

Note that in trial 8 the increment of settlements, since the completion of the structure to February 25th, 1970, represents the time dependent behaviour of the foundation and structure at constant building height over that period.

Computation time for a sixteen storey CN Tower space frame by means of ICES STRUDL II on an IBM 360/67 using 500 K storage, took approximately 4 hours.

3.5 Analysis of Results

The reactions generated by the STRUDL analysis consist of force components in the three principal (global) directions as well as moments about all three axes. The significance of the horizontal forces and moments will be discussed first before considering in detail the vertical reactions.

In STRUDL the footings were considered as fixed supports

TABLE 3.1

SETTLEMENTS AND COMPLETED STOREY HEIGHT OF THE CN TOWER

Date	Settlement Set No. and Trial No.	Completed Storey Height	Settlement Increment
Dec. 10, 64	1	Ground Floor	Initial to 1
Mar. 23, 65	2	3	1 to 2
May 20, 65	3	10	2 to 3
May 27, 65	4	11	3 to 4
July 6, 65	5	16	4 to 5
Aug. 16, 65	6	21	5 to 6
Oct. 28, 65	7	Roof	6 to 7
Feb. 25, 70	8	Roof	7 to 8

capable of generating other than just vertical reactions. Since the footing sizes of the CN Tower are large, it was found that the moments could easily be generated without significant deviation from the assumed condition of support fixity. For example, the largest moment generated in trial 5 for footing number 3 was 2000 inch-kips about the x "global" axis. An equal but opposite triangular force distribution on each of the two halves of the footing can represent the moment. For the 24 foot by 24 foot support the maximum force per square foot required at the footing edges was approximately .08 kip per square foot. This is very small and can easily be generated without footing rotation.

Since all the input displacements were vertical, the reactions in the two horizontal directions tended to be much smaller in comparison with those in the vertical direction. The footings at the CN Tower are of considerable depth and were formed by neat hand excavation directly into the dense till. Due to the absence of backfill at the sides the small horizontal reactions could easily be generated without significant lateral movements of the footing.

The vertical reactions in terms of the basic loads are plotted in Fig. 3.5 for footings 3, 14 and 18. Since the reactions could be either negative or positive, depending on the relative settlements during the construction increment, the reactions given in Fig. 3.5 are in terms of the absolute value as a percent of the basic footing loads. From the settlement data presented in Chapter II it can be shown that

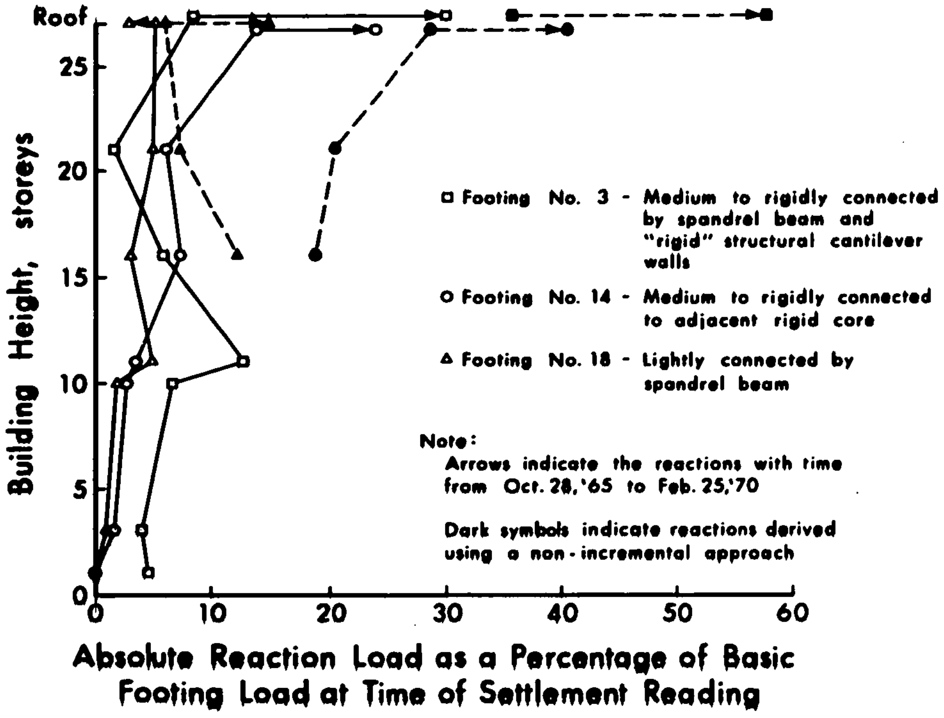


Fig. 3.5 The Variation of Support Reactions Induced by Frame Distortions Observed During Construction, CN Tower

the difference between the maximum and minimum settlement never exceeded 0.9 cm. (.35 inches) for any data set. Fig. 3.5 clearly illustrates that the settlement variation over this limited range can generate appreciable footing reactions.

This conclusion was also obtained by Meyerhof (1953) who observed that structural deformations induced appreciable stresses in spandrel beams and in columns, particularly those in the lower storeys.

The data plotted for footing 18, which is connected to the rest of the structure by means of a relatively flexible beam shows that the load redistributions to or from this footing are small, amounting to only -5.4 per cent at the end of construction, this being reduced to +2.8 per cent over the subsequent 4½ year time span.

In contrast to this is the column framing into footing 3 which is connected to the core through a relatively rigid girder and to the cantilever support walls within the parking garage. At the end of construction the reaction by the incremental procedure is -8.0 per cent, which is increased to -30.4 per cent during the 4½ year time span. A reaction of -30.4 per cent represents a decrease of in footing load of 2271 kips at footing 3.

Starting with trial 5, the total footing displacements experienced were applied to the structure completed at that time. This is parallel with the conventional approach mentioned previously.

From the data given in Fig. 3.5 it is immediately obvious that there is no resemblance between the non-incremental (conventional) approach and the more realistic incremental procedure.

The data presented in Table 3.2 indicates that for footing No. 5 there is at the end of construction a 299 kip reduction in the load at footing level which equals a 6.4% of the basic loading thought to exist on the footing. Using the identical total settlements and the stiffness of the complete building, the conventional procedure predicts an increase in load of 1526 kips which equals 32.6 per cent of the basic loading. This lack of agreement is characteristic of the results calculated.

The reduction in load of 738 kips for the time dependent period is approximately equal to the similar reduction from 1526 to 761 kips for the conventional procedure. This is to be expected as the structural frame used in both trial 7 and trial 8 remains unchanged.

The magnitudes of reactions indicated for footing 5 can be described as typical. In the case of footings 1 to 4, located on the south side of the CN Tower, the reactions are considerably larger due to the proximity of the thick walls required for the support of the overhead cantilever section. February 25, 1970 are -30.4% and -58.1% for the incremental and conventional procedures respectively. These percentages represent loads of -2271 and -4339 kips. The values for footing 2 are +2% and +29.9% for the respective methods of

TABLE 3.2

VERTICAL REACTIONS GENERATED AT FOOTING NO. 5 DUE TO STRUCTURAL DEFORMATIONS OF THE CN TOWER

		Incremental Procedure			Conventional Non-Incremental Procedure		
Trial No.	Date	Reaction Increment kips	Reaction Sub-Total kips	Per cent of the Basic Footing Load Acting at Date Shown	Reaction kips	Per cent of the Basic Footing Load Acting at Date Shown	Completed Storey Height
1	Dec. 10, 64	+8	+8		N.A.		Ground Floor
2	Mar. 23, 65	+110	+118	+6.5	N.A.		3
3	May 20, 65	-523	-405	-14.9	N.A.		10
4	May 27, 65	+131	-274	-9.8	N.A.		11
5	July 6, 65	-301	-575	-17.0	+846	+25.0	16
6	Aug. 16, 65	+212	-363	-9.2	+1280	+32.4	21
7	Oct. 28, 65	+64	-299	-6.4	+1526	+32.6	Roof
8	Feb. 25, 70	-738	-1037	-19.9	+761	+14.6	Roof

calculation. A complete table of results can be found in Appendix C.

The reactions on the central core were much smaller than for most footings. The February 25, 1970 data indicates that for both the conventional and incremental solutions, the basic loading of the core would only be altered by +1.8%. This low redistribution to the core was typical throughout all eight trials.

As can be observed in Fig. 3.4, the settlement pattern of the CN Tower did not conform to a bowl shaped depression. In light of the analysis presented in Chapter II, this is perhaps not surprising, as the contact pressure of the core footing was about 1 ksf. lower than the surrounding column supports.

Because of this somewhat unusual settlement pattern the positive and negative vertical reactions generated at the core supports, due to the relative positions of the surrounding footings, tended to negate one another such that their sum was small. Had the settlement pattern been typical, the maximum settlement of the core would have generated a net negative reaction at the core supports with the adjacent column supports reflecting a net positive reaction.

The reactions induced by the settlements experienced during the period from October 28, 1965 to February 25, 1970 were such that there was an overall reduction of the vertical support loads in the centre of the building with a corresponding increase at the peripheral footings. The settlement of the central part of the building is likely due to non-elastic behaviour of the foundation (creep) and the creep of concrete members.

The reactions computed were not applied to the basic footing loads and used to recalculate the derived moduli of deformation by the procedures outlined in Chapter II. Intuitively, the load percentages quoted above would indicate that the altered basic loads would give rise to an increased scatter in the derived moduli, thereby indicating that perhaps local "soft" or "firm" foundation areas exist within the confines of the building perimeter. However, since the sum of the vertical reactions generated by STRUDL would always equal zero, the effects of load dispersion would cause the average of all the derived moduli to change little from the value previously calculated. For these reasons the reactions were not used for further calculation.

3.6 Some Factors Affecting Interaction Results

The assessment of building rigidity is not a simple task. Deviations from rigid joint action and cracking of members make the structure less stiff than analysis would indicate. In contrast to this tendency towards flexibility, light internal partitioning, architectural facings, mechanical piping, etc. all tend to increase overall stiffness. To take these effects into account is an impossible task.

For long term considerations, yielding of the steel reinforcement and creep of concrete leads to an overall reduction in stress concentrations introduced by differential movements of supports (Neville, 1963). This effect can be approximated by a reduction in the modulus of elasticity used in the structural analysis or by applying a judgment factor to the results. Throughout the entire STRUDL analysis a modulus of elasticity of 3150 kips per square inch was used. No reduction or judgment factor was applied in trial no. 8 to allow for creep effects.

Based on the results of long term deflection tests, the ACI Building Code Requirements for Reinforced Concrete (ACI 318-63, Sect. 909) suggest that multipliers acting on immediate deflections be used to estimate long-time deflections of flexural members under sustained loads. This is comparable to allowing a reduction in loading to maintain a sustained deflection. Thus the use of a multiplier of 2.0 to compute the additional sustained load deflections for the case of no steel compressive reinforcement is similar to a reduction of 2/3 in reactions generated by differential building deflections. This reduction is somewhat severe and perhaps reflects the discrepancy between ultra-conservative conventional approaches to soil structure interaction which predict high support reactions and member stresses and the behaviour of real buildings which do not appear to exhibit signs of distress under these conditions.

Because the mechanism of creep is as yet not fully understood and since the above stated multipliers refer to simply supported beams, considerable judgment must be exercised in applying factors to the results obtained from the analysis of a building frame.

Recent work by Ghali, Dilger and Neville (1969) indicates that the effect of creep on a continuous beam subjected to an ultimate settlement of 0.92 inches occurring in one year results in an induced support reaction approximately 1/2 of the value without creep relief. Although this factor is derived for the case of time dependent settlement it would likely also be applicable to any lower storey beam in the CN Tower which undergoes deflection due to immediate settlements induced by further loading of the supports during construction of the building.

3.7 General Observations and Summary

An analysis of this type wherein the variation in structural rigidity is taken into account becomes particularly attractive where design criteria are critical and where displacements must be controlled. An example of this is perhaps a nuclear reactor installation where differential displacements and cracking of the structure must be strictly limited.

Upon imposition of a shock wave due to earthquake or blast, the behaviour of the structure will be affected by the stresses induced by foundation settlement. To perform the dynamic analysis the effect of differential displacements already existing in the building must be assessed by either the incremental method or the ultra-conservative conventional method of analysis. Where the bulk of settlement takes place during the construction period the incremental procedure would be more appropriate.

The usual structural design practice is to take the settlement pattern estimated from the basic footing loads and calculate the moments generated in individual members due to the differential displacements. If the shear or moment capacity is exceeded, additional reinforcing may be used to satisfy design requirements.

Generally, the axial loads generated by individual beam shear reactions are not computed or used to recalculate new footing loads and therefore a new estimate of settlement. This procedure is conservative, as the new settlement pattern will possess smaller differentials at the supports due to the effect of soil-structure interaction.

The overall factor of safety in a structural frame is probably about 2.0 at the full design load. This means that even if signs of distress were evident, collapse would not occur unless the building was subjected to about twice the design load. Undoubtedly many secondary effects and unknowns can be accommodated by this factor of safety.

From the point of view of axial loading at the supports it would appear that a major component of these unknowns could consist of the reactions generated by differential displacements. This is verified by the support reactions at the CN Tower which were computed to be as high as 40 per cent of the basic footing load. The conventional method of analysis predicts values even greater than these.

Implementation of techniques to compute soil structure interaction effects could lead to a reduction of the amount of "extra building" presently built into structures.

To summarize, the following points can be made:

(a) In performing a soil-structure interaction analysis, the variation in structural rigidity with storey height cannot be ignored, particularly when the displacements during the construction period dominate the settlement history of the building.

(b) The incremental method of analysis, which allows for the variation in structural rigidity, is less conservative but more realistic than conventional approaches. The two methods bear little relation to one another. For example, in some instances the incremental method predicts a negative support reaction whereas the conventional procedure predicts a positive quantity.

(c) The distortions of a real structural frame, induced by small differential settlements at its supports, can generate reactions of appreciable magnitude. The maximum settlement reaction calculated by the incremental method for the CN Tower is +1920 kips at footing number 4 and represents 39.4% of the basic load acting at that footing. The difference between the maximum and minimum settlement which generated these reactions never exceeded 0.9 cm.

(d) Techniques are now available for a complete frame analysis; the reactions can be determined for any set of known displacements imposed at the supports.

CHAPTER IV

SOIL STRUCTURE INTERACTION WITH VARYING STRUCTURAL RIGIDITY

4.1 Introduction

The purpose of this chapter is to introduce the concept of varying structural rigidity into a simple method for the analysis of soil structure interaction which will be useful in design.

The method outlined deals only with the effect of differential settlements of isolated footings on the structural frame. In calculating settlements the variation in rigidity of the structural frame both in plan and in height is taken into account. Variation of soil properties over the site, a major cause of differential displacements, is allowed for in the method. The effect of load dispersion from one footing influencing the settlement of another is included by superposition techniques.

4.2 Previous Work

One of the earliest attempts to depart from the simplifying assumptions of perfect flexibility or rigidity was a study of the rigidity of a circular foundation resting on an elastic medium (Borowicka, 1936). He illustrated that the distribution of the contact soil pressure is a function of Young's modulus and Poisson's ratio of both the circular plate and the underlying foundation soil.

Subsequent work for the case of multi-loaded, semi-flexible rafts, resting on a Winkler medium, neglected the redistribution of moments and loads in the superstructure due to differential settlements (Hetenyi, 1946; Vesic, 1961).

Grasshoff (1957) examined the influence of superstructure rigidity on the contact pressure and bending moments of a combined footing, taking into account the settlement of the raft. His idealizations were perfect flexibility and perfect rigidity of the superstructure, with supports to the foundation being either hinged or fixed-end rigid columns. Many structures do not possess either of these two degrees of rigidity and are not supported by fixed-end rigid columns to the foundation.

For intermediate cases of superstructure rigidity, Sommer (1965) presented a method based on elastic behaviour of both the structure and soil for analysis of foundation beams and slabs, flexible only in one direction.

More recent work by Lee and Harrison (1970) gives solutions based on a Winkler model for combined or two-dimensional raft foundations. The final stress state within the foundation and structure takes into account the combined effects of superstructure and foundation flexibilities.

Meyerhof (1953) and Chamecki (1956) have produced approximate solutions for frame structures supported on isolated footings. The work by Meyerhof is based on slope deflection techniques and follows a procedure whereby correction moments due to footing settlements are distributed to the superstructure by the Hardy Cross technique. The settlements are calculated from structural self-weight loads, neglecting immediate settlement.

Chamecki (1956) considered only the effect of load transfer as a result of soil-structure interaction. Utilizing the fact that footing pressures and the resulting settlements must be compatible, he used an iterative procedure to arrive at final footing loads and final settlements. Simplified structural components were used in the analysis, load transfer being limited only to adjacent columns.

4.3 General Observations

The work of other authors indicate that the areas of concern varied greatly. Some were interested in specific solutions for superstructure bending moments resulting from a particular foundation type. Others were solely interested in settlements. The simplifying assumptions and models used to arrive at specific solutions varied as well. Several conditions of fixity were often investigated and often a variety of "equivalent" beams were used to represent the superstructure.

One concept not recognized in previous work is that the rigidity of any building frame not only varies due to differing assumptions of fixity but also due to the inherent variation caused by the addition of members as the structure increases in height. In fact, the observation can be made that any soil-structure interaction solution which adopts a specific structural model to represent the entire building, and thereafter seeks to utilize elastic theory or a Winkler model to calculate soil settlements, must be in error. Although possibly appropriate when consolidation dominates the settlement response to loading, such a solution must assume that gravity does not act and thus immediate settlements do not take place until the building is structurally complete.

It has been shown in a previous chapter that the settlement behaviour of buildings founded on over-consolidated soils is dominated by the settlements which take place during the construction period. Available solutions to represent soil structure interaction during this period appear inadequate.

4.4 Scope of Soil Structure Interaction Solution

The solution presented here is for the immediate settlement of a multi-storey frame of finite rigidity and founded on a series of rectangular spread footings. The rigidity of the structural frame will vary with height and will depend partially on the conditions of support fixity chosen. The frame may be non-uniform in plan with increasing storey height and may contain a rigid core wall. Variations in soil properties over the site, which lead to larger differential displacements, can be incorporated into the solution.

Most structural design procedures would follow the format of initial sizing of component members as a function of building self-weight and reduced live-load due to occupancy. This first phase would be followed by a check on the tolerance of the structure to wind loading and differential settlement. A proportion of the forces and moments generated by these checks would be added to those calculated in the initial design stage, thus perhaps necessitating re-sizing of some components of the structural frame.

The soil-structure solution given here would require little or no alteration in the procedures presently employed by structural engineers but would greatly facilitate execution of the check for structural behaviour due to settlements.

4.5 The Solution

(a) Foundation Displacements

Foundation displacements during the construction period are termed immediate and can be calculated using elastic theory. The procedure adopted here assumes that the foundation is a homogeneous, linearly elastic medium of semi-infinite extent. For a uniformly distributed vertical load acting on a finite area, the solution for the settlement of a corner of the area as given by Steinbrenner (1934) is

$$\Delta = q\left(\frac{1 - \mu^2}{E}\right)BI_{\Delta} \qquad 4.1$$

where Δ = vertical displacement of the corner

μ = Poisson's ratio

E = modulus of elasticity of medium

B = width of rectangular footing

I_Δ = influence value dependent on ratio of footing length to width.

To allow for the effect of load dispersion, superposition can be used for a group of adjacent footings (Terzaghi, 1943). Thus for N footings subjected to vertical loads R_j the settlement of a particular footing at its centroid can be expressed as:

$$\Delta_i = K \sum_{j=1}^{j=N} \frac{R_j}{A_j} N_{ji} \qquad 4.2$$

where $K = (1 - \mu^2)/E$

A_j = area of footing j

$N_{ji} = B_j^* I_{ji}^*$

N_{ji} is the contribution of footing j to the settlement at i. Since by superposition we normally deal with 4 rectangles, the term N_{ji}

$$N_{ji} = B_j^* I_{ji}^* = \sum_{\ell=1}^{\ell=4} B_{\ell(j)} I_{\ell(ji)} \qquad 4.3$$

For simplification we put the preceding equation into matrix form.

$$\begin{aligned} \Delta_1 &= K[\frac{R_1}{A_1} N_{11} + \frac{R_2}{A_2} N_{21} + \dots + \frac{R_N}{A_N} N_{N1}] \\ &\cdot \\ &\cdot \\ &\cdot \\ \Delta_N &= K[\frac{R_1}{A_1} N_{1N} + \frac{R_2}{A_2} N_{2N} + \dots + \frac{R_N}{A_N} N_{NN}] \end{aligned} \tag{4.4}$$

By letting $\frac{KN_{11}}{A_1} = M_{11}$ we can reduce the matrix still further. Thus,

$$\begin{aligned} \Delta_1 &= R_1M_{11} + R_2M_{21} + \dots \quad R_NM_{N1} \\ \Delta_2 &= R_1M_{12} + R_2M_{22} + \quad R_NM_{N2} \end{aligned} \tag{4.5}$$

etc.

Since the right side of these equations is the product of two matrices we can write:

$$\begin{vmatrix} \Delta_1 \\ \Delta_2 \\ \cdot \\ \cdot \\ \cdot \\ \Delta_N \end{vmatrix} = \begin{vmatrix} M_{11} & M_{21} & M_{31} & \dots & M_{N1} \\ M_{12} & & & & \\ & & & & \\ & & & & \\ & & & & \\ M_{1N} & M_{2N} & & & M_{NN} \end{vmatrix} \begin{vmatrix} R_1 \\ R_2 \\ R_3 \\ \vdots \\ R_N \end{vmatrix} \tag{4.6}$$

and thus $$[\Delta] = [M]\,[R] \qquad 4.7$$

The matrix [M] is symmetrical about the diagonal, the non-diagonal terms generally being an order of magnitude smaller. This dominance of the diagonal indicates that the greatest proportion of settlement at a particular footing is due to the load acting at that footing.

This fact makes possible the approximation for variable site conditions. Rather than using a single E and μ value over the entire site, only those pertinent to a particular footing location can be employed. The matrix now becomes non-symmetrical. To avoid gross error large variations in moduli should not be used. An upper limit of two is suggested for the ratio of largest to smallest E values.

For cohesionless soils Standard Penetration Tests may be used to give the required data (Terzaghi and Peck, 1948). Such a procedure has been used to predict elastic properties over the site for cases where load intensities did not exceed one-third of the ultimate (Farrent, 1963; Webb, 1969). Values of elastic modulus E derived from plate tests can also be used (Burland and Lord, 1970).

(b) Structural Displacements

The procedure used here for relating structural behaviour to support movements is similar to that suggested by Chamecki (1956). A change has been introduced in that load transfer throughout the entire structure is possible and is not limited to immediately adjacent members.

Any structural frame will generate positive or negative vertical reactions at all supports where any single support is displaced downward by a unit amount. These reactions are called reaction coefficients. The subscripts of the reaction coefficient Q_{ji} then refers to reaction at footing support i due to a unit displacement at footing j.

Thus for N footings, subjected to an arbitrary settlement pattern, the net reaction at footing i after displacements take place, is

$$R_i = R^\circ_i + \sum_{j=1}^{j=N} Q_{ji}\Delta_j \qquad 4.8$$

where Q_{ji} = Reaction Coefficient

R°_i = vertical load due to self-weight and no differential settlement of supports

Δ_j = settlement of footing j

Thus we get for N footings

$$\begin{aligned} R_1 &= R^\circ_1 + \Delta_1 Q_{11} + \Delta_2 Q_{21} + \cdots \quad \Delta_N Q_{N1} \\ R_2 &= R^\circ_2 + \Delta_2 Q_{12} + \Delta_2 Q_{22} + \cdots \quad \Delta_N Q_{N2} \qquad 4.9 \\ &\vdots \\ R_N &= R^\circ_N + \Delta_1 Q_{1N} + \cdots \qquad \Delta_N Q_{NN} \end{aligned}$$

This can also be expressed in the form

$$\begin{vmatrix} R_1 \\ R_2 \\ \cdot \\ \cdot \\ \cdot \\ R_N \end{vmatrix} = \begin{vmatrix} Q_{11} & Q_{21} & \cdots & Q_{N1} \\ & & & \\ & & & \\ & & & \\ & & & \\ Q_{1N} & \cdots & \cdots & Q_{NN} \end{vmatrix} \begin{vmatrix} \Delta_1 \\ \Delta_2 \\ \\ \\ \\ \Delta_N \end{vmatrix} + \begin{vmatrix} R^\circ_1 \\ \cdot \\ \cdot \\ \cdot \\ \cdot \\ R^\circ_N \end{vmatrix} \qquad 4.10$$

or

$$[R] = [Q]\ [\Delta] + [R^\circ] \qquad 4.11$$

This equation thus states that the vertical reaction at a particular footing is the sum of self weight loads, assuming no structural distortions, and the load transfer generated by distortion of the frame as differential settlements take place.

Due to the theorem of reciprocity it will be found that the [Q] matrix is symmetrical about the diagonal. Since statics must be satisfied the expansion of [Q] [Δ] and addition of all resulting terms will yield the quantity zero. This in effect states that the net force acting on the frame must remain unchanged and equal to its self-weight. Thus if the sum of the terms in any column or row of the [Q] matrix does not equal zero, an error has been made.

It can also be shown that the load transfer to or from a footing will remain unchanged when an arbitrary frame distortion is altered by an equal amount of settlement to all footings.

(c) Compatibility

From consideration of foundation settlement and structural load transfer, equations 4.7 and 4.11 were derived, both in matrix form.

Compatibility dictates that:

(a) the foundation settlements must equal the structural displacements of the supports and

(b) the vertical foundation loadings must equal the vertical reactions at frame supports.

We can thus combine the equations to arrive at mathematical solutions for final settlements and footing loads.

Substituting equation 4.7 into equation 4.11 yields

$$[R] = [Q]\,[M]\,[R] + [R^\circ] \qquad 4.12$$

Reducing to isolate [R] we get

$$\{\left[\begin{smallmatrix}1&\\&1\end{smallmatrix}\right] - [Q]\,[M]\}\,[R] = [R^\circ] \qquad 4.13$$

and thus $$[R] = [S]\,[R^\circ] \qquad 4.14$$

where $$[S] = \{\left[\begin{smallmatrix}1&\\&1\end{smallmatrix}\right] - [Q]\,[M]\}^{-1} \qquad 4.15$$

with [R] known we can go back to equation 4.7 to solve for settlements.

$$[\Delta] = [M]\,[R] \qquad 4.7$$

Note here that we have arrived at a solution for a structure of specified but arbitrary rigidity undergoing arbitrary settlement of its supports due to the imposition of a set of self-weight loads.

Use of reaction coefficients for the entire structure as well as the total design self-weight loads would correspond to the conventional soil-structure procedures outlined in the earlier discussion.

The solution, taking into account varying structural rigidity, would be identical but only for a single increment of storey height. The settlements would be those generated by the addition of a single storey. The structure undergoing the distortion would be the frame existing at the time of the storey addition.

Repeated use of the solution with increasing magnitudes of reaction coefficients would thus reproduce a realistic variation in rigidity as construction progresses. The addition of increments of storey load would finally yield the correct settlements and load transfers for the end of construction condition.

4.6 Variation of Structural Rigidity with Height

It is time consuming to compute new reaction coefficients every time a building frame increases in height by an additional storey.

For any particular structure or portion thereof, that maintains constant geometry in plan and in height, the reaction coefficients are a linear function of storey height. This can be illustrated by visualizing a single beam of finite stiffness supported at both ends and at midspan. Depending on the support conditions assumed, a forced vertical unit displacement of only the midspan support requires particular positive or negative vertical reactions at each of the three supports. The addition of four beams identical with the first, followed by the same deformation condition will generate vertical reactions exactly five times greater than before at the three supports.

In most structural frames this linearization is nearly rigorously true as spandrel beams, floor slabs, and floor beams are usually typical with height. A slight deviation may result with decreasing column sizes at higher elevations.

Skempton and MacDonald (1956) attempted to relate angular distortions and maximum settlements to maximum storey height, the latter parameter assumed to be indicative of structural rigidity. No pattern was found to exist for the data available. This is likely attributable to the oversimplified assumptions used. Buildings of equal storey height can possess markedly different structural stiffnesses. Furthermore they can exhibit markedly different settlement patterns during construction and afterwards due to differences in foundation types.

Reaction coefficients are computed separately from the soil-structure interaction solution. If the frame is simple, they can be computed by hand. For larger frames it is suggested thatthe ICES-STRUDL II program, available from IBM for use on an IBM 360/67 computer, be used. This method is very simple in that in-depth knowledge of structural engineering or computer programming is not required for derivation of the coefficients.

4.7 Computer Solution

The direct mathematical solution for incremental interaction given in equation 4.14 does not lend itself to use within a computer program. For buildings with say, twenty footings and having a similar number of storeys, the reaction coefficient matrices become unwieldly and the required amount of computer time for matrix inversion becomes prohibitive for commercial purposes.

The iterative technique adopted in the computer program converges quite rapidly to the solution and decreases the storage that would otherwise be required. A brief summary of the iterative procedure follows.

(1) As a first approximation the incremental design loads R°_{i}, when going from storey height $S_{\ell-1}$ to S_{ℓ}, are used to compute a set of Δ_i values. Equation 4.7 yields the results.

(2) Substitution of the set of Δ_i values into the structural relation yields a series of load transfers. The reaction coefficients Q_{ji} are those for a structural frame S_ℓ storeys high.

(3) As indicated by equation 4.11 the load transfers are added to the R°_i values, the new loads being used to recompute a new set of footing settlements Δ_i.

(4) Steps 2 and 3 are repeated until the R_i values repeat themselves within the limits specified by the convergence criteria. The rate of convergence is influenced by the relaxation factor. The number of iterations required to satisfy the convergence criteria is smallest when the relaxation factor equals 0.5. The final Δ_i and R_i values computed are the solution for this particular storey increment.

(5) For increasing storey heights, steps 1 through 4 are repeated and the final settlements and total loads then are

$$\Delta_i \text{ final} = \sum_{S=1}^{S=H} \Delta_i \qquad 4.16$$

$$R_i \text{ final} = \sum_{S=1}^{S=H} R_i \qquad 4.17$$

where H equals building's maximum storey height.

4.8 Input for the Computer Program

The computer program requires five types of information as input.

(1) Statistics for Program Control

Here the total number of footings, storeys, and soil types, are specified. The number of iterations and convergence criteria are listed here as well.

(2) Footing Loads

The total design loads acting on a particular footing for a particular storey height are listed. Omitted values are found by interpolation.

(3) Footing Geometry

Footing dimensions and coordinates of the centroid of the footing are specified.

(4) Reaction Coefficients

For a particular storey height a complete square matrix of reaction coefficients will be required. Omitted matrices will be interpolated on the basis of storey height.

(5) Foundation Properties and Locations

The dimensions and coordinates of the centroid of rectangular areas are specified with the accompanying moduli and Poisson's ratio.

Rules for preparing input data can be found in the detailed description accompanying the program in Appendix F.

A listing of all input data is given in the program output.

Output of results is given for each footing and for both the conventional and new incremental procedures. The loads and settlements for each completed storey are specified.

4.9 Example Solution

An example solution of the soil structure interaction problem using computer techniques was carried out by analyzing a semi-rigid structural frame of non-uniform geometry with height. To accentuate soil-structure behaviour five soil zones were specified such that differential displacements would be maximized.

Convergence criteria specified that all the vertical reactions during the iteration process must repeat themselves with less than 0.05 kip variation before going on to the next storey.

The model frame and location of soil zones is shown in Figure 4.1. We can see that footings 1 and 5 are located on a "soft" and "stiff" soil zone respectively. Hence the relative differential displacement between these two points will tend to be the maximum. For this reason considerable load will be redistributed from footing 1 to footing 5.

The tabulated results for these two footings are shown in Table 4.1. For the case of no soil structure interaction and uniform load increases up to the 5th floor, we have uniform settlement increases of .095 inches per storey. This

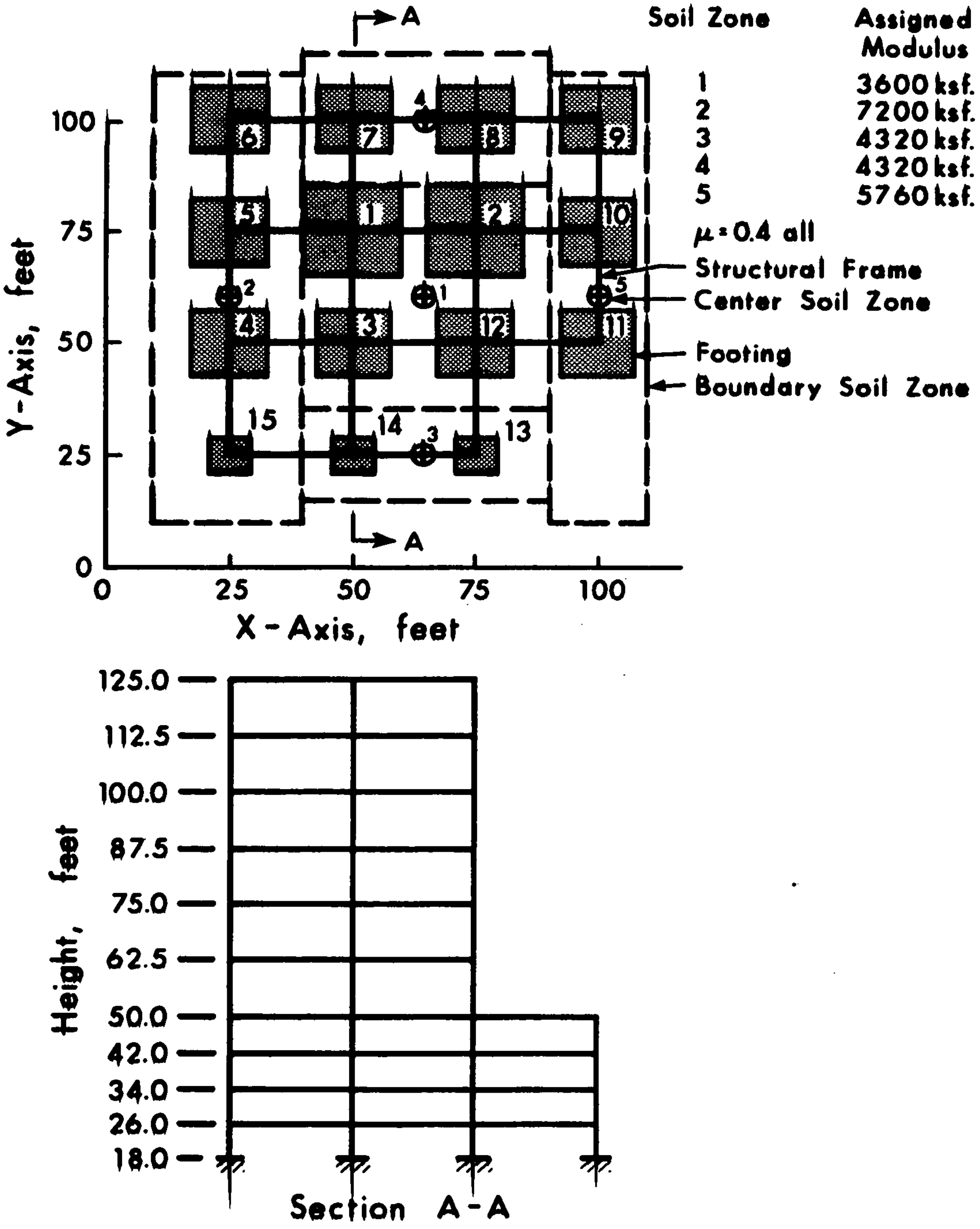


Fig. 4.1 Building Frame and Foundation, Illustrative Example

TABLE 4.1

Footing Loads and Settlements of Model Frame

FOOTING NUMBER 1

	LOADS AND SETTLEMENTS NO SOIL STRUCTURE INTERACTION				LOADS AND SETTLEMENTS WITH INCREMENTAL SOIL STRUCTURE INTERACTION				
STOREY HEIGHT	CUMUL. LOAD	INCREM. LOAD	CUMUL SETT.	INCREM. SETT.	CUMUL. LOAD	INCREM LOAD	CUMUL. SETT.	INCREM. SETT.	NO. OF ITERATIONS
1	250.0	250.0	0.095	0.095	235.4	235.4	0.092	0.092	6
2	500.0	250.0	0.189	0.095	459.4	224.0	0.182	0.090	7
3	750.0	250.0	0.284	0.095	673.2	213.8	0.271	0.089	7
4	1000.0	250.0	0.378	0.095	877.7	204.5	0.358	0.087	8
5	1250.0	250.0	0.473	0.095	1073.7	196.0	0.444	0.086	8
6	1500.0	250.0	0.562	0.089	1262.0	188.3	0.522	0.078	8
7	1750.0	250.0	0.650	0.089	1442.4	180.4	0.599	0.077	7
8	2000.0	250.0	0.739	0.089	1615.6	173.2	0.675	0.076	7
9	2250.0	250.0	0.828	0.089	1782.1	166.6	0.750	0.075	7
10	2500.0	250.0	0.917	0.089	1942.7	160.5	0.824	0.074	7
11	2750.0	250.0	1.006	0.089	2097.6	154.9	0.997	0.073	7
CONVENTIONAL SOIL STRUCTURE INTERACTION									
11	2750.0		1.006		1696.0		0.831		9

ALL RESULTS IN UNITS OF KIPS AND INCHES

FOOTING NUMBER 5

	LOADS AND SETTLEMENTS NO SOIL STRUCTURE INTERACTION				LOADS AND SETTLEMENTS WITH INCREMENTAL SOIL STRUCTURE INTERACTION				
STOREY HEIGHT	CUMUL. LOAD	INCREM. LOAD	CUMUL SETT.	INCREM. SETT.	CUMUL. LOAD	INCREM LOAD	CUMUL. SETT.	INCREM. SETT.	NO. OF ITERATIONS
1	150.0	150.0	0.039	0.039	157.1	157.1	0.040	0.040	6
2	300.0	150.0	0.078	0.039	319.6	162.5	0.080	0.040	7
3	450.0	150.0	0.117	0.039	487.0	167.3	0.120	0.040	7
4	600.0	150.0	0.156	0.039	658.6	171.7	0.161	0.041	8
5	750.0	150.0	0.195	0.039	934.2	175.6	0.202	0.041	8
6	900.0	150.0	0.232	0.036	1011.1	176.9	0.240	0.038	8
7	1050.0	150.0	0.268	0.036	1191.3	180.1	0.279	0.039	7
8	1200.0	150.0	0.305	0.036	1374.3	183.1	0.318	0.039	7
9	1350.0	150.0	0.341	0.036	1560.1	185.8	0.357	0.039	7
10	1500.0	150.0	0.378	0.036	1748.3	188.2	0.396	0.039	7
11	1650.0	150.0	0.414	0.036	1938.8	190.5	0.436	0.040	7
CONVENTIONAL SOIL STRUCTURE INTERACTION									
11	1650.0		0.414		2121.4		0.451		9

ALL RESULTS IN UNITS OF KIPS AND INCHES

accumulates to .473 inches at the 5th floor. Although the load increments remain the same beyond the 5th floor, the absence of load dispersion from footings 13, 14 and 15 results in a uniform reduction in settlements to 0.089 inches per storey with the final settlement accumulating to 1.006 inches. The incremental storey load accumulates to 2750 kips with the completion of the 11th storey. The self-weight footing loading undergoes no change as the structure is considered to be perfectly flexible.

The structural stiffness is taken into account in the incremental soil structure solution. We have at the completion of the first floor a slight reduction in settlement from 0.095 to .092 inches and a reduction in the vertical footing load from 250 kips to 235.4 kips. This reduction is due to the stiffness of a single storey frame causing load to be transferred from footing 1 to adjacent footings. As the frame increases in height and thereby in rigidity, the decrease in settlement per storey becomes greater as does the magnitude of load transfer per storey. The 5th floor increment causes the self weight loading to be reduced from 250 kips to a net footing load increment of 196 kips. Proceeding to the 11th storey the reductions in load carried by footing 1 increase until finally the rigidity of the frame causes only 154.9 kips to be transferred at that floor to the footing. The accumulated footing load equals 2097.6 kips. This represents a 24 per cent reduction of the self weight loading due to the load transfer caused by the differential displacements of the structural supports.

The calculation for the conventional procedure is presented as well. If the entire 11 storey frame could be built as a weightless but semi-rigid structure, the "application" of gravity would cause a much greater amount of load redistribution. The footing load indicated in Table 4.1 has been reduced by 38 per cent from its initial self-weight loading.

The data shown in Table 4.1 for footing 5 is generally similar to that described for footing 1. The footing load increase, instead of reduction, is the result of differential displacements induced by the tendency toward less settlement at the periphery of loaded areas as well as the founding of footing 5 over the "stiff" soil zone.

At footing 5 the incremental procedure predicts an increase in footing load of 17.5 per cent of self-weight loading to 1938.8 kips. The conventional procedure is again more conservative when it predicts the final footing loading of 2121.4 kips.

From this table it is obvious that small differential settlements can give rise to appreciable load transfers when the frame or structure undergoing the distortion is stiff or semi-rigid. The summation of all the final accumulated footing loads will of course equal the total weight of the entire building frame.

Having solved for the support settlements of the semi-rigid frame, it is now possible to compute the effects of differential distortions on individual members. If ICES-STRUDL II was used to derive the reaction coefficient matrices, component members of the frame can be analyzed simply by introducing the support settlements into the already available structural model.

The incremental solution solves the soil-structure interaction problem of moderately rigid structures which settle during construction. It is a considerable improvement over the conventional technique and can be readily applied to design practice.

CHAPTER V

MEASUREMENT AND ANALYSIS OF HEAVE AT THE ALBERTA GOVERNMENT TELEPHONES (AGT) TOWER SITE

5.1 Introduction

One of the questions that had to be answered in the design of the AGT-Oxford Complex was related to the magnitude of foundation displacements which could be expected as a result of excavation at the site and subsequent reloading as construction progressed.

From preliminary examination of the settlement records of the CN Tower located one-half mile to the north it was anticipated that the movements would be dominated by the elastic response of the foundation. However, due to some variation of site stratigraphy between the CN Tower and AGT Tower locations and the absence of any field records on other buildings in the area, there was considerable hesitation in arriving at some representative field modulus which could be used in the analysis. Some laboratory moduli were available and eventually a decision was made such that the analysis could proceed.

Several factors made it desirable to monitor and analyze the rebound during excavation as well as the settlement of the 26 storey Oxford Building and the 34 storey AGT Tower. These include, the unknown reliability of the laboratory derived moduli of deformation, the proximity of existing and to be constructed buildings, the presence of nearby underlying coal workings and the relatively new practice of multi-storey construction on the foundation soil found in the City of Edmonton.

The rebound aspect of the study is presented in this chapter. The observed heaves, their analysis and their implications to design practice are outlined. The instrumentation used and the method of data acquisition are fully described. A discussion of the foundation stratigraphy of the AGT-Oxford Complex site and a description of laboratory derived geotechnical data is included.

5.2 Site Description

The geographical location of the central area of Edmonton within which the AGT Tower excavation is located was shown in an earlier chapter in Figure 2.1. Detail map B shown in Figure 5.1 indicates the location of the AGT-Oxford site, an outline of the excavation, the location of adjacent buildings and the location of the reference benchmark. At the time excavation commenced at the AGT site, the superstructure of the adjacent Oxford Building was nearly complete.

The maximum depth of excavation in the AGT Tower core footing area is 46 feet from the original ground surface. The southern portion of the underground parking area is the shallowest excavation with a depth of 37 feet.

The overall dimensions of the base of the final excava-

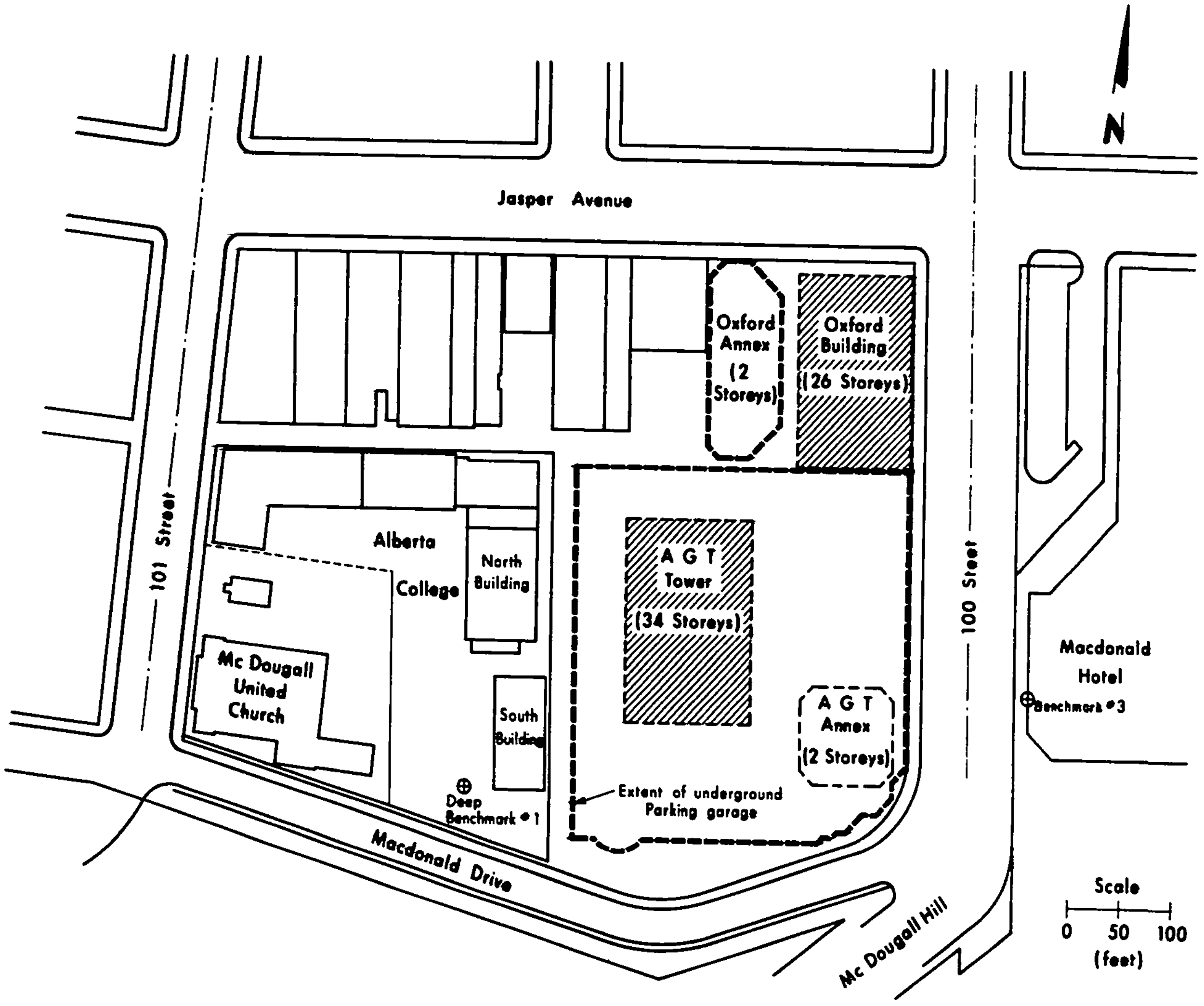


Fig. 5.1 Detail Map B, Location of AGT-Oxford Complex

tion are approximately 330 by 330 feet with construction side slopes of 1 1/2 vertical to 1 horizontal. Although at some locations the side slopes were cut even steeper all stood without collapse over a one year period.

5.3 Site Geology and Stratigraphy

The geology of the central business area in Edmonton was described in some detail in an earlier chapter. The stratigraphic profile at the excavation site remains unchanged in sequence but varies considerably in depth to individual strata.

With the exception of a small number of dry auger test holes, an extensive program of wet-drilling by means of a Failing 1500 drill was carried out by the foundation consultants. Pitcher tube sampling was carried out wherever possible. Several bedrock and till samples were obtained for laboratory testing. The locations of drill holes and stratigraphic sections are shown in Figure 5.2. Several representative borehole logs pertinent to the site are given in Appendix B.

It should be pointed out that the large number of boreholes drilled were put down in an effort to locate an abandoned coal mine thought to exist beneath the building site. Cavities at depths consistent with the coal seams were found but were proved to terminate well away from the tower location.

Borehole resistivity surveys made it possible to deter-

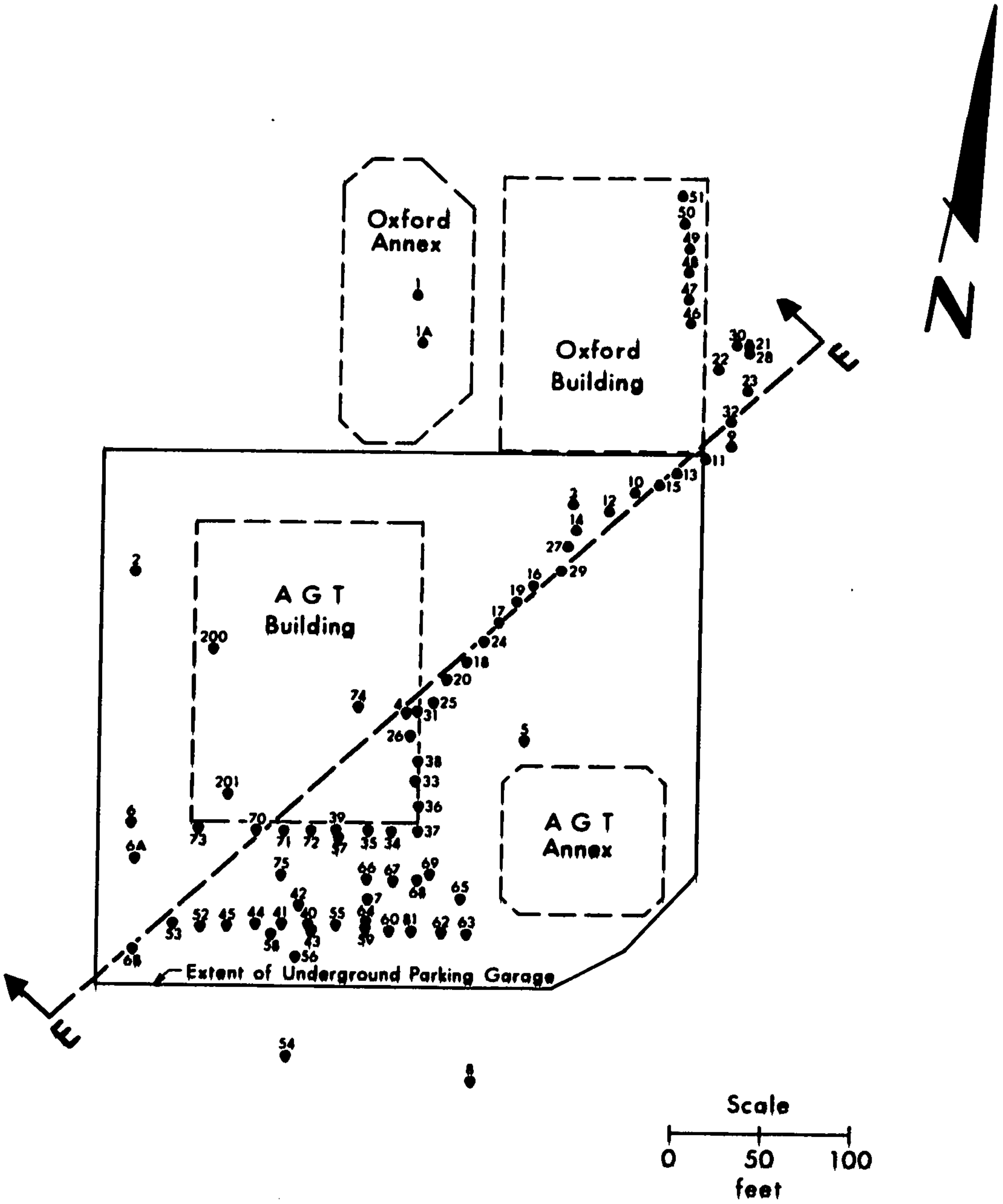
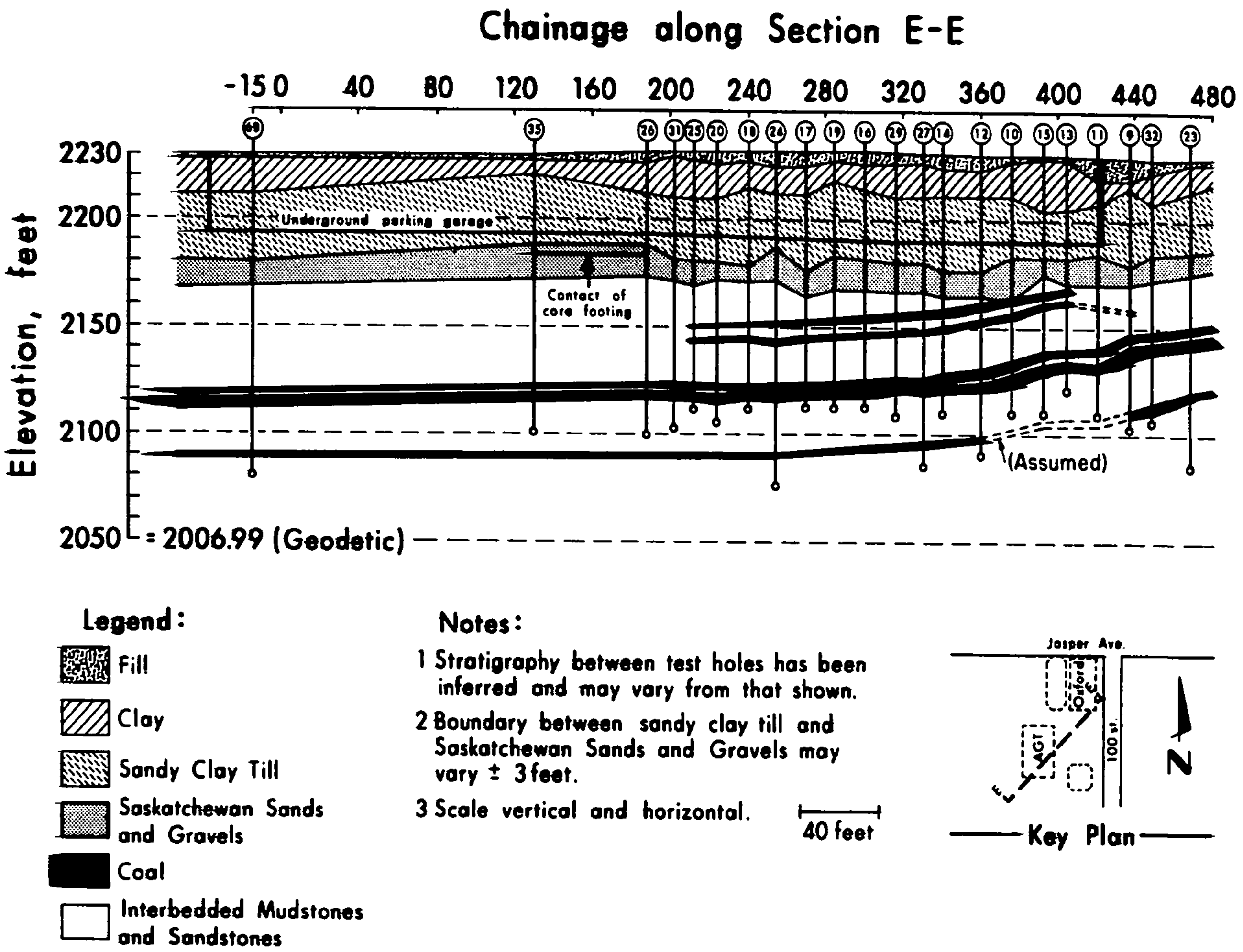


Fig. 5.2 Location of Boreholes and Stratigraphic Section, AGT-Oxford Complex

mine precisely the elevations of specific coal seams and proved to be much more reliable than the determination of elevations from wash borings.

The stratigraphy at the site could be determined with accuracy. The profile consists of strata of surface fill, lacustrine deposit, clay till, Saskatchewan Sands and Gravels and bedrock, the latter locally known as the Edmonton Formation. Cross-section E-E through the site, shown in Figure 5.3, gives the stratigraphic sequence.

The till deposit at the site has an average top surface elevation of 2213 feet (City of Edmonton) and an average thickness of 32 feet. The till matrix consists of a stiff, silty or sandy clay, which contains pockets and lenses of silt and sand, pebbles, stones, coal specks and streaks of iron rust. During excavation some discontinuous sand lenses 2 to 3 feet thick were observed in the till sheet. The colour of the till is usually dark grey-brown, with light brown pockets of sand and silt. The till is extensively fissured throughout the stratum, with a thin layer of iron rust and other minerals deposited within the fissures. The density of till is high, as evidenced by Standard Penetration blow counts at the AGT site varying between 23 to +100 blows per foot, and averaging 80 blows per foot. The lower blow counts occur in the upper portion of the till sheet, the higher values generally occurring at lower elevations and at the location of the AGT Tower footings. The natural water content varies between 7 per cent and 23 per cent and the



Legend:

- ▨ Fill
- ▨ Clay
- ▨ Sandy Clay Till
- ▨ Saskatchewan Sands and Gravels
- ■ Coal
- ☐ Interbedded Mudstones and Sandstones

Notes:

1 Stratigraphy between test holes has been inferred and may vary from that shown.

2 Boundary between sandy clay till and Saskatchewan Sands and Gravels may vary ± 3 feet.

3 Scale vertical and horizontal.

Fig. 5.3 Stratigraphic Section, AGT-Oxford Complex average is usually about 13 per cent. This illustrates the heterogeneity of the deposit.

The Saskatchewan Sands and Gravels is a dense granular deposit with a surface elevation of about 2181 feet (City of Edmonton) and an average thickness of approximately 15 feet at the AGT site. The stratum is mainly dense, medium to fine grained sand, with gravels and rocks found only near the contact with underlying bedrock. Standard Penetration blow counts in the material range from 80 blows per foot to values in excess of 100 blows per foot. The natural moisture content ranges from 4 per cent to 16 per cent, with an average of about 5 per cent. As is typical of dense sand deposits, the stratum at the site is a very competent bearing material.

The bedrock formation consists of interbedded bentonitic mudstones and silty sandstones with some seams of lignite coal. The upper 10 feet of the formation is weathered. Drilling at the site indicated that shattered zones exist deeper within the bedrock mass, probably the result of ice movements during the Pleistocene. Standard Penetration blow counts which range from a low of 16 per foot in a coal seam to 75 to 100 per foot elsewhere, as well as relatigely high sample recovery ratios, indicate that competent bedrock exists at the site.

During the extensive drilling program no permanent water table was observed. Some free water was encountered within sand and silt lenses in the till at the AGT-Oxford site, but in quantity often proved to be small. The water used in the installation of instrumentation drained into the formation over a period of a single day, likely through the fissures in the till. During the excavation at the site, no free water was encountered in the excavation or in the Saskatchewan Sands and Gravel deposit. As is often the case in Western Canada, the free water table is difficult to determine and at the site is thought to exist somewhere in the bedrock formation. The presence of dry cavities within the coal seams suggests that it might be found at depths at least 120 feet below ground surface.

5.4 Geotechnical Properties from Laboratory Tests

From the point of view of computation of heave and settlement, the only property of interest for the upper lacustrine silty clay stratum is its in-situ bulk unit weight. This value used in computing the AGT excavation unloading pressure, varied between 126 pcf and 111 pcf, with an average of 119 pcf. The natural moisture contents of this deposit range from 39% to 16%, averaging between 25 to 30%.

The underlying till deposit is very dense with an in-situ bulk density varying between 135 pcf and 124 pcf and averaging 131 pcf. The heterogeneity of the deposit is reflected in the grading analyses run on samples taken at different elevations. The per cent of sand sizes (M.I.T. Scale) varied between 34% and 73%, silt sizes between 15% and 45% and clay sizes between 12% and 21%. Although somewhat speculative, the till at the AGT site is considered to be somewhat sandier than tills found elsewhere in the Edmonton area. Atterberg limit tests of the till matrix indicated a high and low liquid limit of 38.6% and 22.4% respectively and a high and low plastic limit of 20.5% and 15.1%. The average plasticity index is 10.4%.

Six consolidation tests were run on till samples taken at elevation 2188 feet (City of Edmonton). This elevation is approximately the elevation of the rebound points and the AGT footings. The computed void ratio at a 2 ksf oedometer loading varied between 0.454 and 0.356. By averaging the oedometer deflections subsequent to this load and using the corresponding average void ratio of 0.394, the composite plot shown in Fig. 5.4 was prepared. All data used is corrected for compression of the testing apparatus. Numerical results are given in Table 5.1 for the composite consolidation curve of the six specimens.

The results of 4 unconfined compression tests run on till and pitcher tube samples of bedrock are shown in Table 5.2.

A summary of consolidation test results for bedrock samples is given in Table 5.3. The intermediate rebound and reloading cycle approximately reproduces the vertical stress variation experienced at the sample depths shown due to excavation and reloading. The moduli of deformation over this intermediate reloading range would be approximately twice the values shown at the bottom of the Table for initial

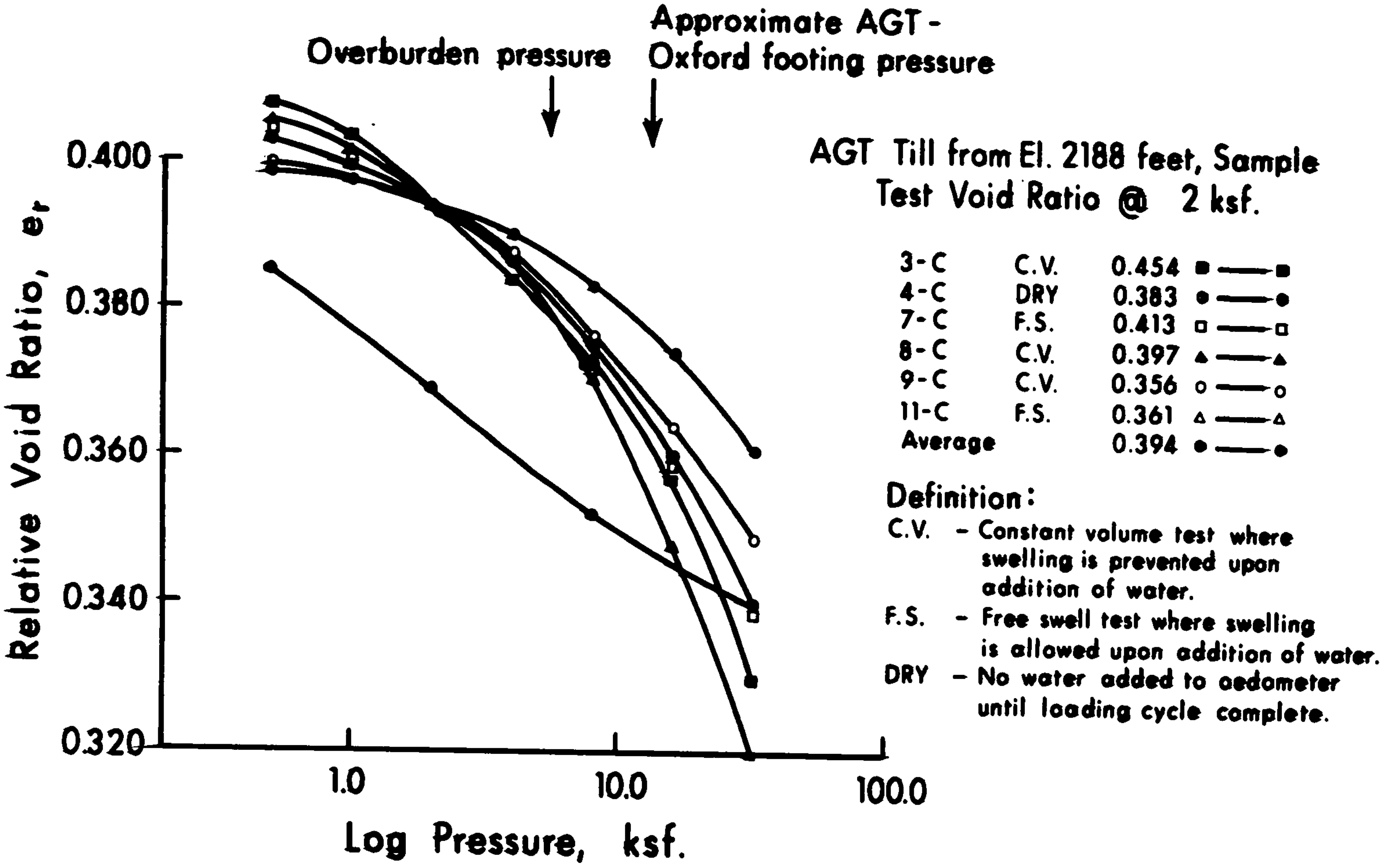


Fig. 5.4 Composite Plot of Consolidation Results For Samples From AGT Tower Site

TABLE 5.1

SUMMARY OF CONSOLIDATION TEST DATA ON TILL

Initial Void Ratio	.382	
Void Ratio at end rebound at 0.5 ksf	.403	
Original overburden pressure	5.40	ksf
Compressive Index, C_c		
0.5 ksf to 2.0 ksf	.013	
2.0 ksf to 5.4 ksf	.030	
5.4 ksf to 13.0 ksf	.044	
Rebound Index, C_R		
5.4 ksf to 0.5 ksf	.028	
Modulus of Volume Comp., m_v		
Rebound 5.4 ksf to 2.0 ksf	.0027	ft^2/k
5.4 ksf to 0.5 ksf	.0043	ft^2/k
Modulus of Volume Comp., m_v		
Compression 0.5 ksf to 5.4 ksf	.0031	ft^2/k
2.0 ksf to 5.4 ksf	.0026	ft^2/k
5.4 ksf to 13.0 ksf	.0016	ft^2/k
Modulus of Deformation, E, $\mu = 0.4$		
Rebound 5.4 ksf to 2.0 ksf	1200	psi
5.4 ksf to 0.5 ksf	752	psi
Modulus of Deformation, E, $\mu = 0.4$		
Compression 0.5 ksf to 5.4 ksf	1044	psi
2.0 ksf to 5.4 ksf	1245	psi
5.4 ksf to 13.0 ksf	2022	psi

TABLE 5.2

UNCONFINED COMPRESSION TEST RESULTS

Sample	Elevation (feet)	Compressive Strength (psi)	Strain at Failure (%)	Water Content (%)	Saturation (%)	Secant Modulus @ 1/4 $(\sigma_1-\sigma_3)_{max}$ (psi)
Till	2206	184	7.2	12.0	85	5,500
Till	2192	203	13.0	7.1	58	9,100
Bedrock (Mudstone)	2140	202	6.6	17.5	85	3,500
Bedrock (Soft SST)	2138	402	5.1	16.7	94	8,000

Note: Secant modulus taken from zero deviator stress to 1/4 $(\sigma_1-\sigma_3)_{max}$

TABLE 5.3

SUMMARY OF CONSOLIDATION TEST DATA ON BEDROCK SAMPLES

	Sample	Specimen				
		13-C Soft Bentonitie Sandstone	14-C Silty Mudstone	15-C Silty Mudstone	16-C Silty Mudstone	TH-39 -103
	Elevation, feet (City of Edmonton)	2169	2141	2141	2157	2128
	Initial Water Content	20.8	19.9	19.9	18.4	19.6
	Initial Void Ratio	.600	.487	.520	.535	.588
Intermediate Rebound and Reloading	Rebound Index, C_r					
	12.0 ksf to 6.0 ksf	N.A.	.013	.019	.020	N.A.
	Compressive Index, C_c					
	6.0 ksf to 16.0 ksf	N.A.	.018	.026	.033	N.A.
	Compressive Index, C_c					
	6.0 ksf to 16.0 ksf	.054	.035	.042	.053	.075
	Modulus of Vol. Comp., m_v, ft^2/k					
	6.0 ksf to 16.0 ksf	.0016	.0010	.0011	.0014	.0021
	Modulus of Deformation, E, psi ($\mu = 0.4$)					
	6.0 ksf to 16.0 ksf	2090	3410	2870	2230	1530

loading.

One interesting feature of the tests run on these bedrock specimens is that the immediate oedometer deflection upon application of the new load increment varied approximately betwen 20 and 40 per cent of the total. In the case of specimen 13-C, primary consolidation was complete in 1 minute with secondary consolidation or creep taking place thereafter. Although influenced by the degree of sample disturbance and fit of the specimen in the oedometer ring, as well as creep of the testing apparatus with time, this rapid response reflects the corresponding behaviour in the field.

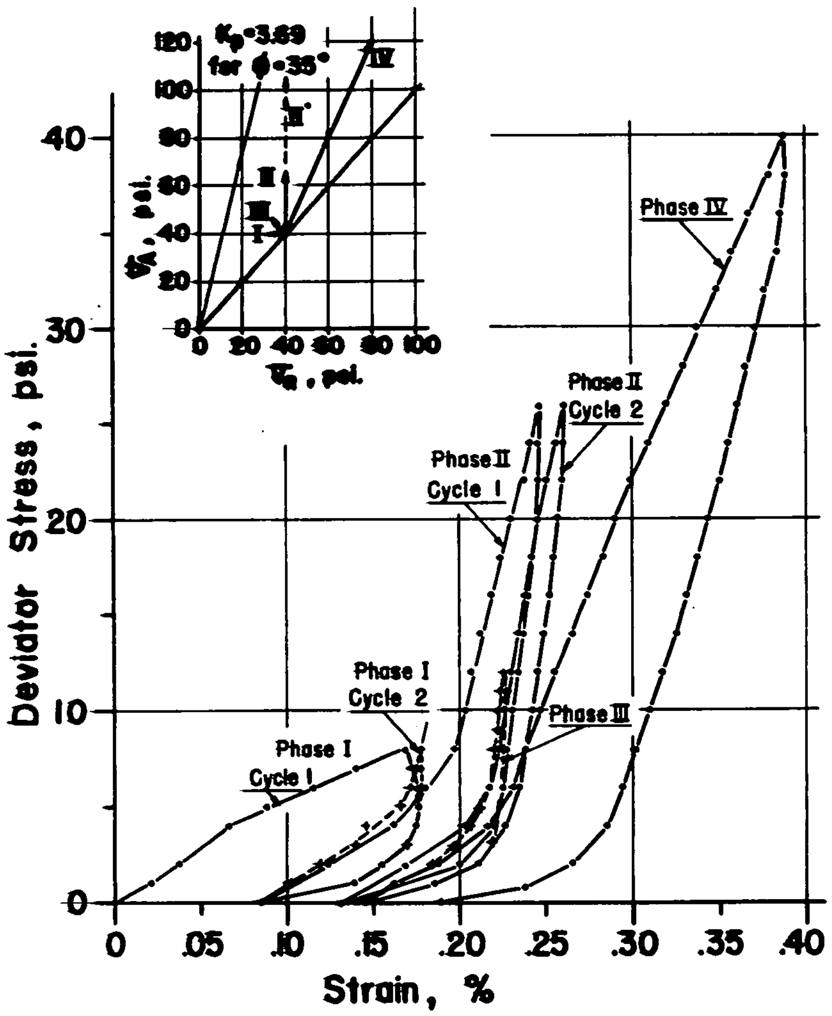
Three small 1.5 inch diameter specimens were cut from a block sample of preglacial alluvial Saskatchewan Sands obtained at elevation 2182 feet (City of Edmonton). The samples were backpressured and consolidated anisotropically to an effective axial pressure of 20.25 psi, with respective lateral pressures corresponding to K_0 values of 0.6, 0.5 and 0.37. After the application of a further deviator stress, the specimens were allowed to creep with time. The first recorded strain at one minute can be used to estimate a modulus of deformation. The derived moduli of the same order noted above are 9500 psi, 14,000 psi and 7,700 psi respectively. Long term creep beyond the one minute strain was small, the maximum recorded was 0.07% for a 5 day period and for the sample subjected to the greatest increment in deviator stress.

At the completion of the creep tests the effective lateral stresses were 29.0 psi, 27.0 psi and 24.0 psi respectively. Maintaining these lateral pressures, the deviator stresses were increased to failure, the resulting angle of internal friction averaging 41.5°. The tangent moduli for the three tests at 1/2 $(\sigma_1-\sigma_3)_{max}$ are respectively 15,000 psi, 26,600 psi and 31,000 psi.

The results of a consolidated undrained compression test run on a 2.8 inch diameter Saskatchewan Sands specimen cut out of a block sample are shown in Fig. 5.5. The isotropic stress path of consolidation and the paths of various loading cycles are indicated. The sample was not back pressured and no pore pressures were recorded at the attached pore pressure transducer. The observed angle of internal friction was 40.4°.

Three compressive strength tests were run on 1.5 inch diameter specimens cut from bedrock core samples taken at elevation 2164 feet (City of Edmonton). The specimens of interbedded mudstone and soft sandstone were consolidated at lateral pressures of 21.0 psi, 29.0 psi and 22.0 psi and taken to failure under fully drained conditions. The maximum observed tangent modulus at 1/2 $(\sigma_1'-\sigma_3')_{max}$ was approximately 5000 psi with peak strengths occurring at strains of about 2.2%. The average effective angle of internal friction observed was 41.5° with no cohesion.

Two consolidated undrained tests with pore pressure measurements were performed on specimens cut from pitcher tube samples of silty soft sandstone taken at elevation 2169



Sample - Saskatchewan Sands, AGT. Site
Elevation - 2186 feet

Phase	Cycle	Modulus of Deformation, psi.	
I	1	4700	Loading
	2	50000	Loading
II	1	36700	Loading
		90000	Unloading
	2	47400	Loading
		78300	Unloading
III	1	100000	Loading
		250000	Unloading
IV	1	21000	Loading
		38200	Unloading
II'	1	47000	Loading to Failure

Fig. 5.5 Cyclic Compression Tests on Sand, AGT Tower

feet and 2167 feet (City of Edmonton. Specimens were anisotropically consolidated under a back pressure of 20 psi with K_0 assumed at 0.6. The applied axial pressures were 65.7 psi and 57.8 psi, the latter value corresponding to the estimated in-situ overburden pressure at elevation 2167 feet. The plotted effective stress paths are characteristic of over-consolidated soil. The tangent moduli at 1/3 $(\sigma_1'-\sigma_3')_{max}$ for the two samples are 14,800 psi and 10,450 psi. The effective angle of internal friction was 37° for both samples, the first specimen showing a cohesion intercept of 13 psi.

5.5 Instrumentation: Installation and Data Acquisition

To record heave within the confines of the excavation area ten rebound points (floating benchmarks) were installed in boreholes at an elevation below the bottom of the excavation and at locations where they would remain accessible as construction progressed. Four settlement pins were installed on the Alberta College buildings located immediately adjacent to the excavation.

These points allowed direct measurement of heave. The settlement pins installed on the south column line of the Oxford Building were used to record approximate or indirect heaves.

The rebound points consisted of a 12 inch high vane possessing 4 flanges made from 1/4 inch steel plate and sharpened at their base. Attached to the flanges at the top was a 3/8 inch circular plate, 3 inches in diameter, which served as a horizontal reference elevation. Fig. 5.6 shows a typical vane and the driving tool used in the installation.

After the boreholes were wet drilled with a Failing 1500 rig to the desired elevation, the vane was lowered to the bottom and pressed into the ground by hydraulically loading the vane with the weight of the truck and drill rig. This usually resulted in 2 or 3 inches of penetration. Using the Standard Penetration Hammer the vane could usually be driven an additional 5 to 6 inches into the dense bedrock or till. Rotation of the driving tool allowed it to be disengaged from the vane.

Each borehole was then backfilled with a bentonite slurry (drilling mud), the consistency of the mixture dependent on the capability of the mud pump to handle the fluid mass. Subsequent examination of the boreholes from the ground surface revealed that excess water had escaped into the till formation, with the remaining bentonite adhering to the borehole walls. Additional slurry was then mixed by hand to a much thicker consistency and poured down the still open boreholes.

As the colour of the slurry was not unlike the dominant colour of the till sheet, it became desirable to use a dye with the bentonite to facilitate relocation of boreholes at a later date. Initial attempts showed that methyl-blue dye was unsuitable. An intense red dye (Erythrosine Dye - colour

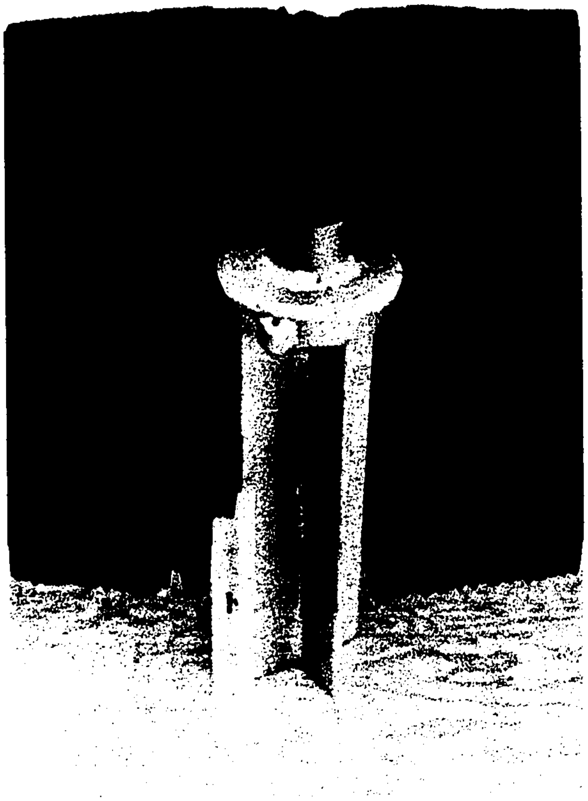


FIG. 5.6 TYPICAL REBOUND VANE WITH ATTACHED DRIVING TOOL

index 773) was added to the bentonite during hand mixing and proved to be very satisfactory. It coloured the slurry a bright pink and stained any soil with which it came into contact.

When some of the small buildings originally on the site were removed, the precise locations of all but two boreholes were determined by triangulation. These two boreholes were referenced to sidewalks and other objects just outside the excavation. The location of all 10 rebound points are shown in Fig. 5.7.

From this diagram it is evident that rebound points 3 and 5 are located directly beneath the exterior footings of the AGT Tower. Rebound points 4 and 7 were positioned a horizontal distance only 5 feet apart approximately on the north-south centre line of the AGT Tower. Rebound point 4 was located just below the elevation of the central core footings while point number 7 was located 22.5 feet deeper. The purpose of this was to record the vertical expansion of the 22.5 foot zone as well as its subsequent recompression as construction progressed.

All other points were located within the confines of the lower parking garage outside of the AGT Tower area. Rebound point number 8 was purposely placed near a corner of the excavation. Rebound point 7 was located in the upper bedrock surface, point 4 in the upper few feet of the Saskatchewan Sands and Gravels, and all other points at various levels in the till stratum near its base.

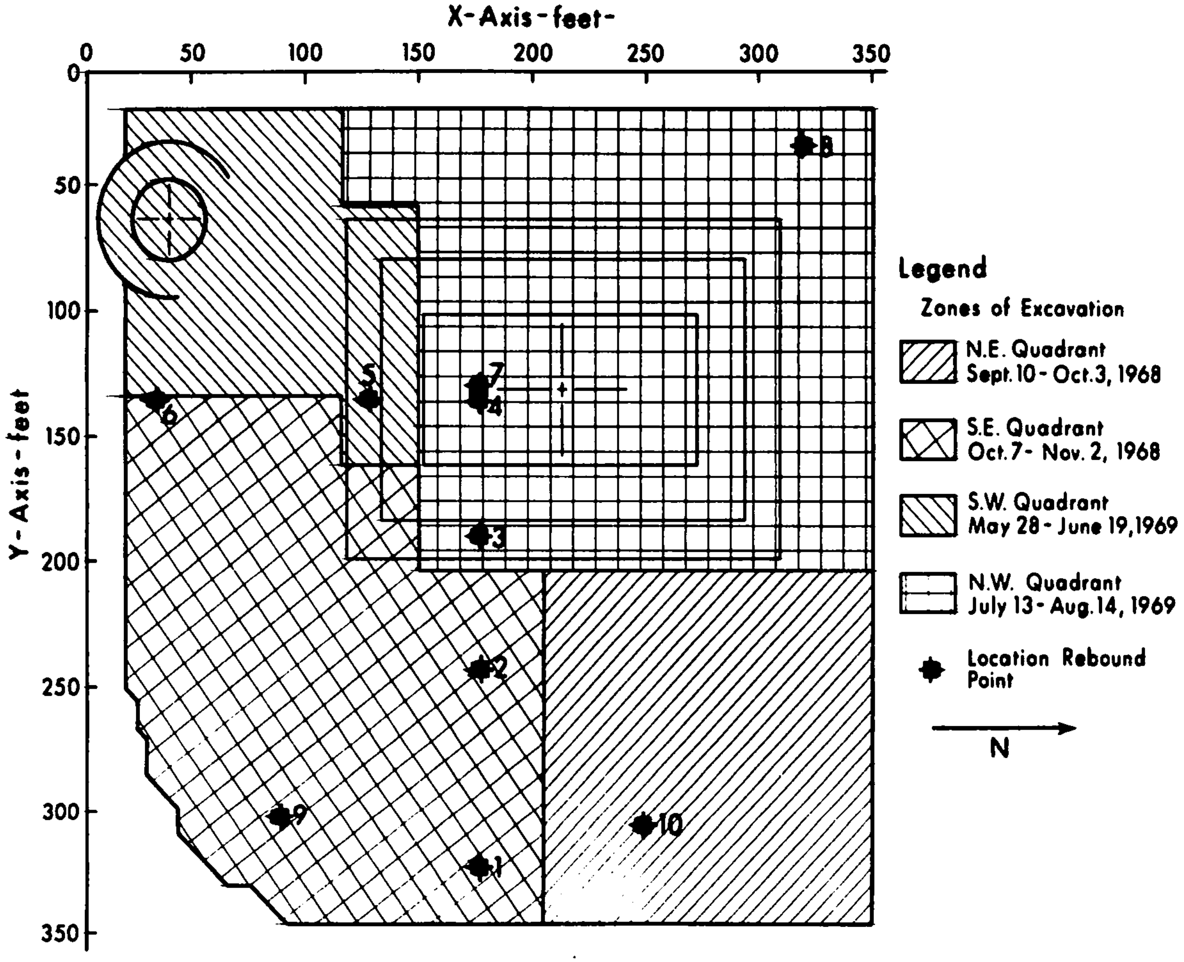


Fig. 5.7 Location of Excavation Zones and Rebound Points AGT Excavation

Prior to any excavation the elevations of all the vane plates were determined as accurately as possible and referenced to the 60 foot deep benchmark (DBM #1) located as shown earlier in Fig. 5.1. This surveying was done by a somewhat unusual procedure which at times was difficult to perform. The record of benchmark control is given in Appendix D. Elevations were taken by using a high precision level (Wild N-3) and reading directly onto a steel tape subjected to a standard 10 pounds of tension and suspended within a 3/4 inch diameter pipe. Fig. 5.8 shows a typical assembly of the apparatus.

The tape was connected to a machined conical probe which fitted the pipe. Individual lengths of pipe were threaded over the tape as the pipe was lowered through the bentonite slurry. Fig. 5.9 shows the operation of pipe coupling and the slotted steel plate holding the pipe in place to prevent it from falling down the hole through the slurry under its own weight. A distinct metallic ring was carried upward through the 3/4 inch piping to the ground surface when contact was made with the reference plate of the rebound point vane. Fig. 5.10 shows a typical rebound plate at the bottom of a borehole at the completion of a 40 foot excavation. Any twist in the tape would be removed, and the tape would then be tensioned.

A reading on the "rod" thus created would be taken to the nearest .001 foot. The pipe assembly would then be listed, rotated and reprobed until three readings were obtained. These would usually repeat themselves with variations rarely

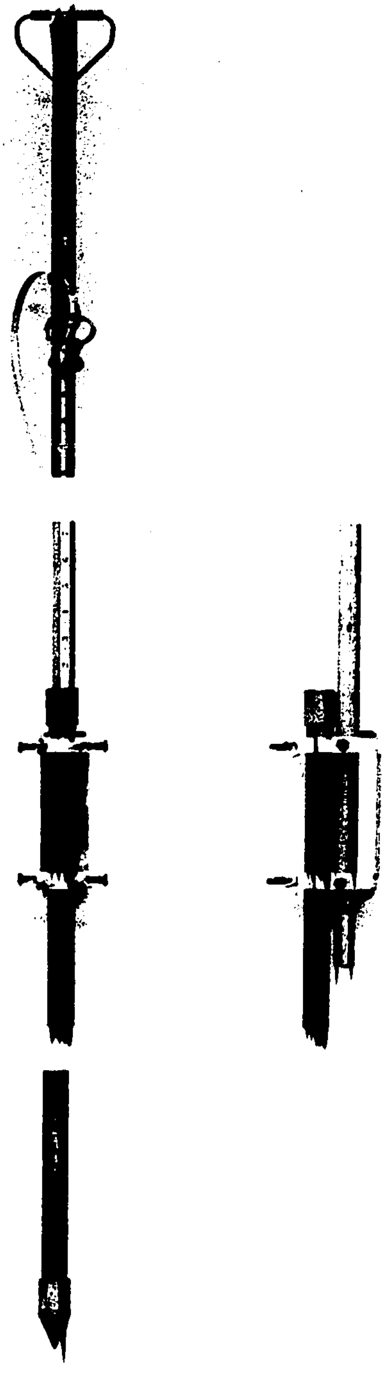


FIG. 5.8 SURVEY APPARATUS

FIG. 5.9 INSERTION PROCEDURE

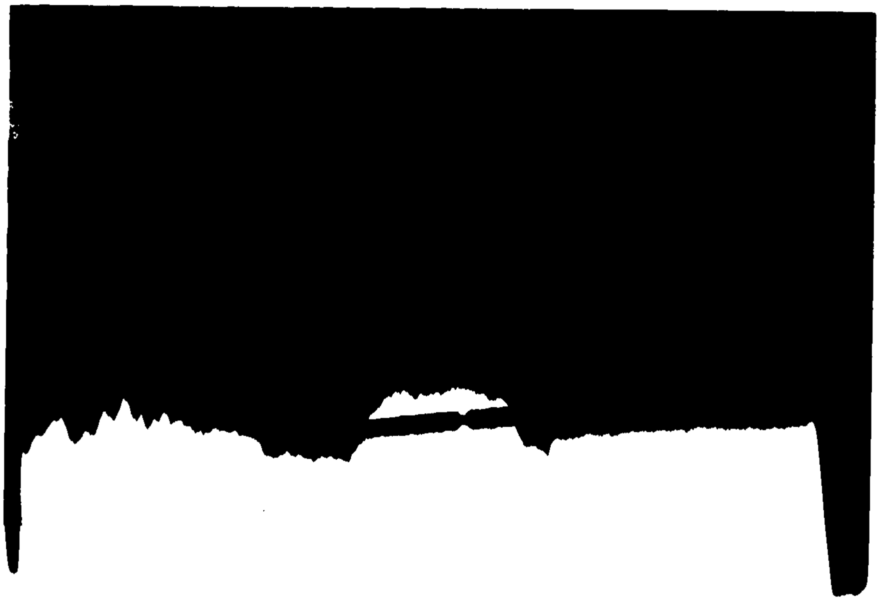


FIG. 5.10 EXPOSED REBOUND PLATE

exceeding ± .001 foot. The overall accuracy of the traverse was considered to be about 0.005 feet with the error of closure rarely exceeding 0.010 feet. The reprobing operation at rebound point 3 is shown in Fig. 5.11. Here about 40 feet of pipe was lifted and reprobed. Immediately behind the technician is the partially constructed underground parking garage located in the N.E. excavation quadrant.

The maximum depth of hole probed was about 75 feet, this being approximately the limiting depth that the pipe could be pushed by hand through the slurry. Removal of the assembly from this depth was difficult, as the pipe had to be held in place and cleaned as individual pieces were disconnected.

As the coefficient of expansion for the steel tape was known, the level readings were corrected for temperature effects. This correction was generally very small but could be significant where the holes were deep or the temperature differentials were severe. Ground temperatures were of the order of 40°F, the standard tape temperature being 68°F. For a 75 foot length of tape suspended in a borehole the level correction would be about .015 foot.

These corrections had to be made as at completion of excavation, a 45 foot distance could now be surveyed in air temperatures, these varying between about 80°F in summer to -10°F in winter.

Some rebound points could not be read in winter as the bentonite slurry and ground was frozen to about a 5 foot

FIG. 5.11 REPROBING PROCEDURE AT GROUND SURFACE

depth. From the method of excavation and subsequent construction followed by the contractor, no vanes were subjected to frost action, as partially constructed areas were enclosed and heated.

Excavation at the site was carried out in 4 stages and the quadrant areas involved are as shown in Fig. 5.7. Elevation changes during the excavation period were recorded by relocating the boreholes and reprobing to the rebound points Rebound points 1 and 9 were lost as the boreholes could not be found. Efforts to find these at the completion of excavation by electronic metal detectors were unsuccessful. The process of relocation and measurement of elevations of the 8 remaining rebound points took two days and was usually done on weekends when excavation equipment was idle. The dimensions and depths of excavation zones corresponding to each heave set were recorded.

At completion, all rebound points except numbers 7 and 8 were located a few feet below the bottom of the excavation and thus probing was now a much simpler task. Rebound point 8 was excavated due to a 1 foot elevation error made during its installation. Access to all remaining points was maintained by sleeving through concrete footings and ground slabs such that data acquisition could continue as construction progressed.

In the case of rebound points 3, 4 and 5 complete overlaps between heave and subsequent foundation settlements was possible, thus providing a continuous record of ground movements at the site.

To reduce survey error, a vertical pipe was installed along a wall such that surveying from the benchmark at ground surface to the underground parking area could be accomplished by means of a single turning point.

A second 100 foot deep benchmark (DBM #2) located in the parking garage basement was included in the rebound point survey traverse to check migration of elevation of this new benchmark. Benchmark #1 has been referenced to a point (BM #3) on a relatively large building located a short distance away. This building is more than 50 years old and thus the reference point is considered to be reliable.

The pins installed in the adjacent Alberta College buildings are called settlement plugs as they are identical to the settlement plugs in the AGT Tower. The installation and recording procedure is described in a later chapter dealing with movements observed there. The elevations of the Alberta College buildings relative to deep benchmark #1 were taken periodically as excavation progressed.

5.6 Observed Foundation Rebound

Excavation work commenced on September 19, 1968 and was completed on August 14, 1969. The heaves measured during the sequence of unloading and partial reloading of the foundation soil are shown in Fig. 5.12.

The real data as obtained in the field has been plotted as distinct points. The lines drawn through these points

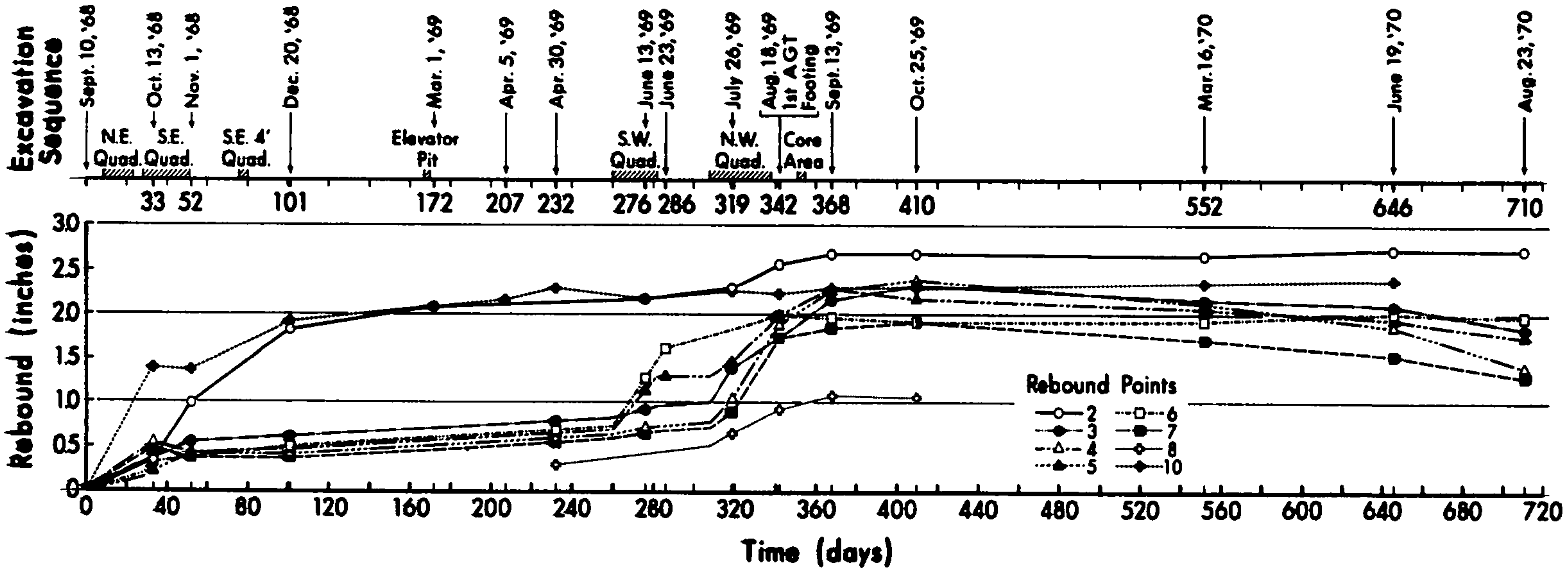


Fig. 5.12 Rebound History at the AGT Excavation

were not drawn simply from point to point but were often extended to reflect the real history of unloading. For example, the parallelism of several lines as given by three rebound sets in Fig. 5.12 was continued beyond the day 101 to day 232 period. During this period and prior to day 260 there was no unloading until the start of excavation of the S.W. quadrant.

One interesting observation that can be made is that the heave at rebound point 7 at day 368 is still 81 per cent of the 2.27 inches recorded for point 4 located 22.5 feet above it.

This means that the bulk of the heave at the AGT site takes place in the bedrock formation. Furthermore, it means that the 22.5 foot zone, essentially the Saskatchewan Sands and Gravels stratum, expanded about 0.42 inches during unloading, as indicated by the gradual separation of plotted data in Fig. 5.12 for points 4 and 7.

The downward movement of rebound points 3, 4, 5 and 7 after day 410 reflects the increasing foundation loading as construction progresses at the AGT site.

Rebound point 10 located within the N.E. quadrant, (see Fig. 5.7) responded very rapidly to the removal of overburden. Rapid response of this type has also been experienced by Bara and Hill, (1967) and Serota and Jennings, (1959). The subsequent 1 inch upward movement of point 2 during the excavation of the S.E. quadrant again reflected this rapid foundation response.

During the seven month winter period prior to excavation of the S.W. quadrant, appreciable time dependent rebound took place at points 2 and 10. The magnitude of this movement equalled the earlier heave recorded during the period of load relief. Similar response has been observed by Endo (1969) for the expansion of a very stiff clay strata due to the excavation of overlying soil. The heave observations at the AGT site are consistent with those reported by Chang and Duncan (1970) for a 200 foot deep excavation in California. Here a deposit of interbedded clays, silts and sands underlain by highly consolidated siltstone and dense silt rebounded as much as 1/3 of the total measured after excavation was completed.

At the completion of the S.E. quadrant the borehole in which rebound point 3 was located, surfaced only about 4 feet from the edge of the 40 foot deep excavation. Probing to this point became difficult at this time. To make proper contact, the piping had to be held against the borehole side and even then the conical probe often slid beside the circular plate that capped the vane. It is considered possible that some lateral distortion of the ground caused the slight out of vertical alignment and hence the difficulty.

The maximum heave recorded at the site after completion of excavation in all four quadrants was 2.7 inches for rebound point 2. Although more centrally located, rebound point 3 heaved a lesser amount of 2.3 inches as extensive time dependent movement was prevented due to reloading by the footing above it.

In general the movements followed the expected theoretical pattern; i.e., the greatest heaves occurring in the centre of the excavation and the least at rebound point 8 located in an excavation corner. An intermediate value would be for rebound point 6, located approximately at the centre line of the excavation but at the side.

The foundation soil at rebound points 2, 6 and 10 is only, if at all, slightly reloaded. These points are located within the underground parking garage area well away from the AGT Tower. Of these three points, point 6 heaved the least because it was closest to the side of the excavation.

Time dependent heave virtually ceased at these points 2 to 3 months after completion of excavation of the overlying soil. This type of movement has also been observed by Bara and Hill (1967), as a result of the 115 foot excavation for the Dos Amigos Pumping Plant foundation in California.

The movements experienced at the Alberta College buildings are shown in Fig. 5.13. The southernmost building paralleled the excavation with the cut exposing the sides of of the foundation. An access ramp located at a depth of 15 feet was supported by vertical sheet piles spaced at 5 foot centres. At a 20 foot distance from the building the cut was vertical to the AGT basement elevation.

The maximum heave experienced by the southernmost building is 0.35 inches. The northernmost building also paralleled the edge of the excavation, but at a distance of

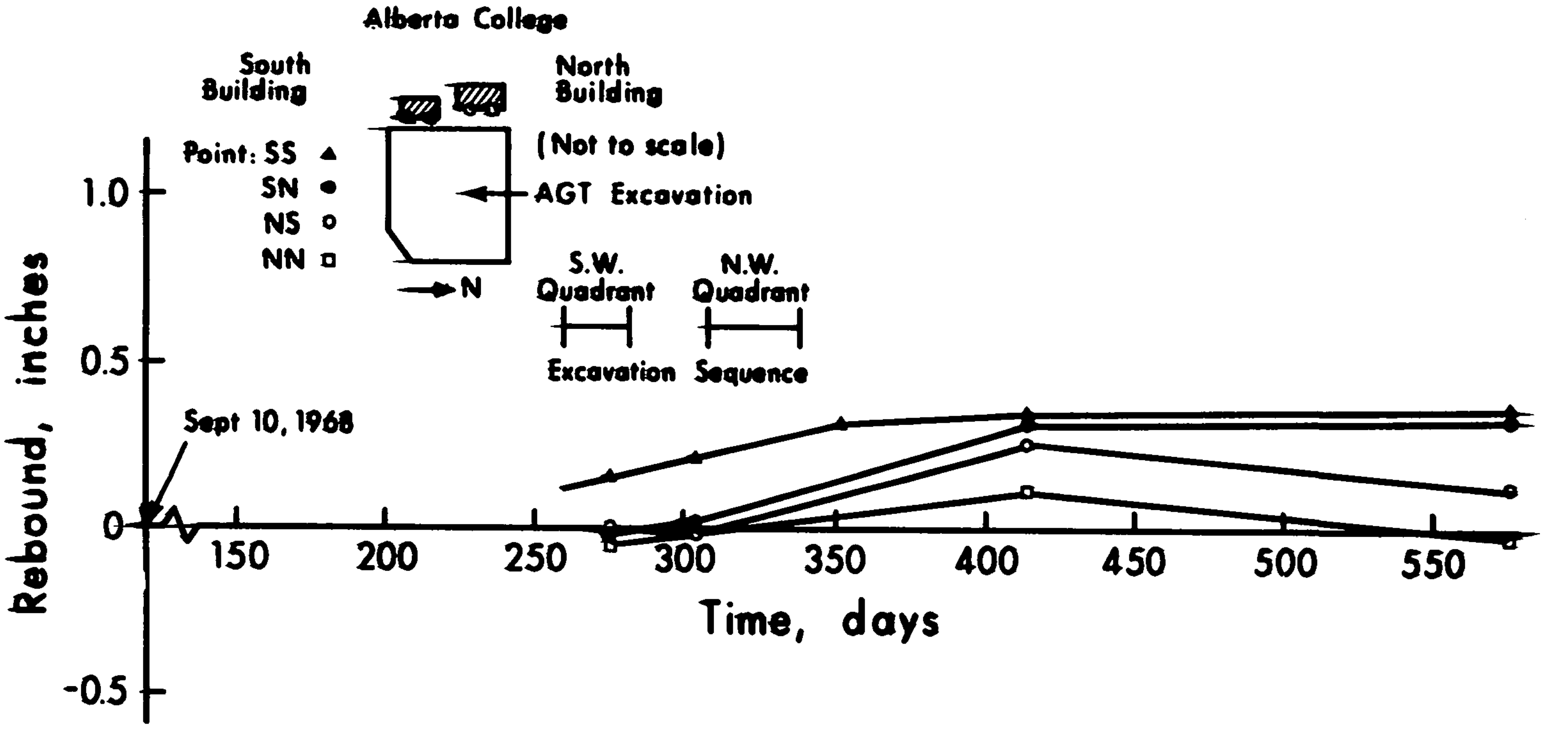


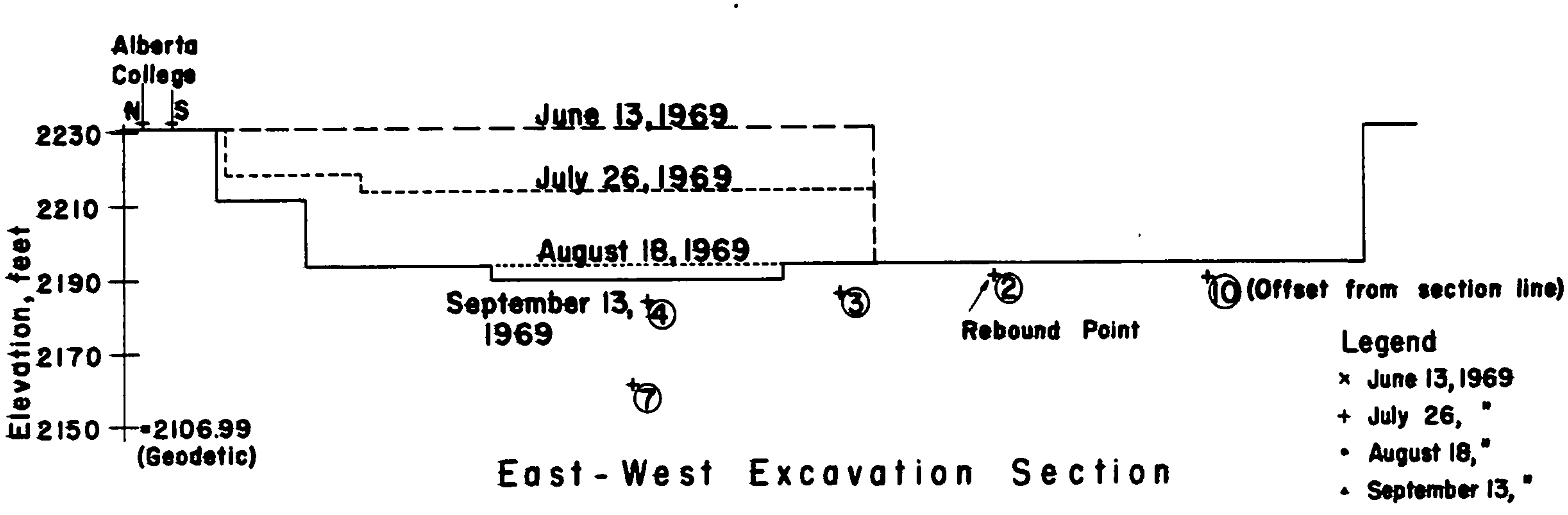
Fig. 5.13 Rebound at Alberta College Due to AGT Excavation

about 15 feet, the excavation side slope was 1 1/2 vertical to 1 horizontal to the bottom of the excavation. The maximum heave recorded at this building was .25 inches and due to later backfilling came down to .12 inches of rebound.

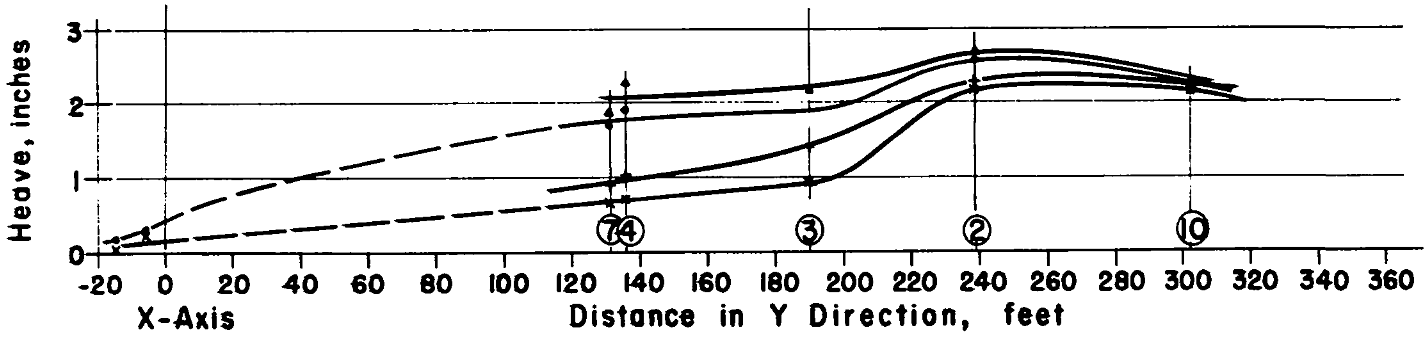
An east west heave profile as well as the corresponding excavation section is shown in Fig. 5.14. The profile indicates that the maximum heave was at the centre of the excavation. The curvesillustrate the sequence of excavation, as the rapid rebound at point 10 was not repeated at points 4 and 7 until excavation of the N.W. quadrant. The data for the Alberta College buildings is included in the figure although the dates of recording do not correspond exactly. The dashed line indicates the hypothetical heave which may have taken place in the area where no rebound points are located.

From these observations on buildings located immediately adjacent to the excavation it is obvious that heave effects are minimal and tend to decrease rapidly with distance away from the excavation. The northernmost Alberta College building is only several feet further from the excavation edge and only undergoes about one-third the heave experienced at the southernmost building.

An indirect indication of heave of areas outside the excavation can be obtained by examining the settlement record of the southernmost Oxford Building footings. The settlement data for settlement plugs 2, 3 and 4 is shown in Fig. 5.15. The heaves recorded at these three locations correspond to



East-West Excavation Section



East-West Rebound Profile

Fig. 5.14 Excavation Section and Heave Profile, AGT-Oxford Complex

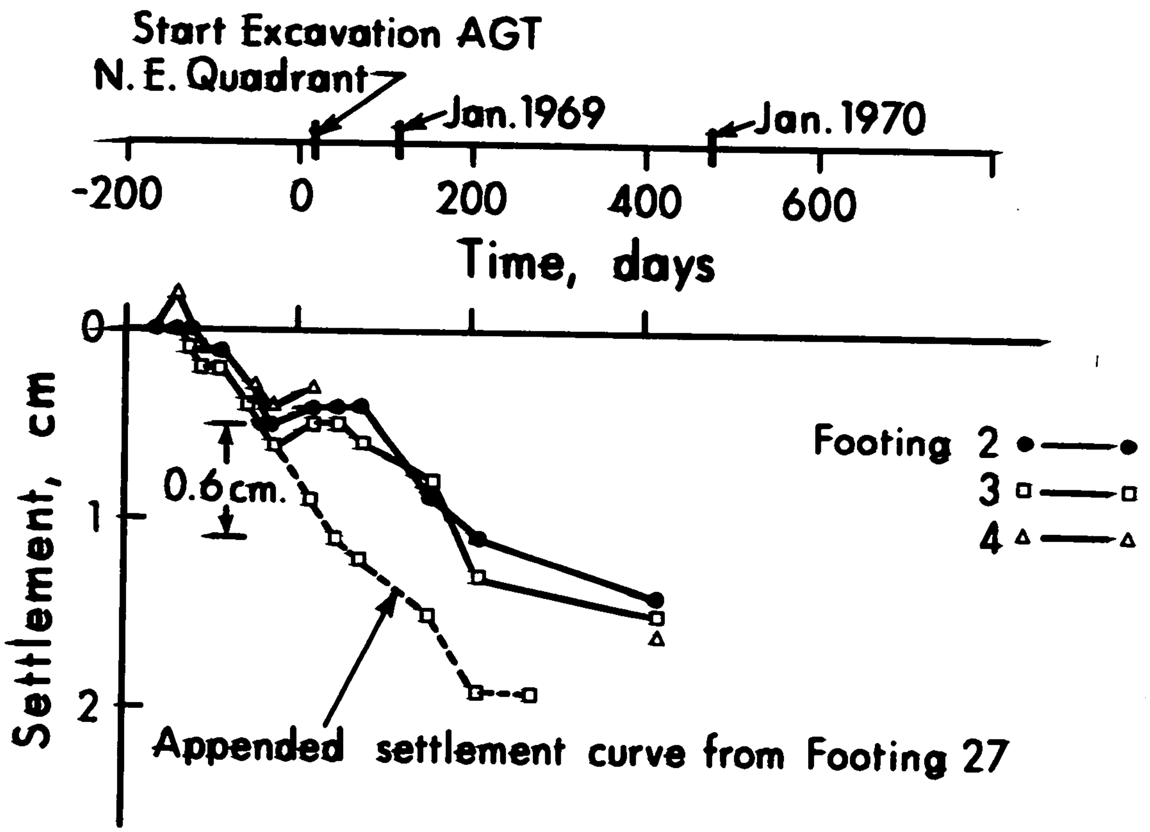


Fig. 5.15 Rebound at the Oxford Building

the onset of excavation at the AGT site. This heave is not discernible at footings located a distance of 40 feet away from the excavation boundary. This again indicates the rapid decay of heave effects at distances away from the excavation boundary. The settlements experienced after day 26 at footing 27, located approximately symmetrically opposite footing 3, is appended to the settlement record of footing 3. The dashed line indicates that approximately 0.6 cm. (.24 inches) heave occurred.

5.7 Mechanism of Heave Response

The response to unloading by excavation is characterized by an immediate upward movement of rebound points which in magnitude usually exceeds 50 per cent of the total heave experienced. For example, rebound point 10 heaved 1.4 inches which is about 60 per cent of the total heave recorded nearly two years later.

After cessation of excavation from a particular quadrant, the time delay to reach approximately 80 per cent of the two year heave is about 2 months. Hence the total rebound response can be termed instantaneous for most practical purposes.

The relative displacements of particular rebound points follow the pattern predicted by elastic behaviour. If the final excavation is assumed to be square with constant depth, then the rebounds experienced at a corner (rebound point 8), at a side (rebound point 6) and at the centre (average rebound of points 2 and 3) can be related simply by means of the theoretical influence values and area widths.

The equation given by Steinbrenner (1934) for the settlement at a corner of a rectangular area is

$$\Delta = qB\left(\frac{1-\mu^2}{E}\right) I_\Delta \qquad 5.1$$

The theoretical ratio of settlement at the centre of the excavation to a corner becomes

$$\frac{\Delta \text{ centre}}{\Delta \text{ corner}} = 2.0 \qquad \text{(theoretical)}$$

The ratio for the observed data yields

$$\frac{\Delta\text{centre}}{\Delta\text{corner}} = 2.2 \qquad \text{(observed)}$$

Using a similar relationship between the centre and a location on a centre line but at an excavation side, yields a theoretical centre to side rebound ratio of

$$\frac{\Delta\text{centre}}{\Delta\text{side}} = 1.5 \qquad \text{(theoretical)}$$

The ratio for observed heave is

$$\frac{\Delta \text{ centre}}{\Delta \text{ side}} = 1.25 \qquad \text{(observed)}$$

The agreement here between the theoretical heave given by elastic theory and the observed heave indicates that the adoption of an elastic mechanism for analysis is appropriate. The rapid response to unloading noted earlier supports this conclusion. This approach to analysis of heave phenomena is not new and has been used by Bozozuk (1963) and Serota and Jennings (1959).

5.8 Method of Heave Analysis

Equation 5.1 allows calculation for heave at the surface of a semi-infinite solid due to a negative vertical load acting upon a finite area. At the completion of excavation when most rebound points are located a few feet from the bottom excavation surface, the solution given by this equation would be applicable. However since the rebounds measured at intermediate stages of excavation correspond to heaves of points located within the elastic body, a different solution must be adopted.

A second equation given by Steinbrenner (1934) can be used to solve for the vertical displacement of a point at any depth beneath a corner of a rectangular area (Harr, 1966).

$$\Delta_c(z) = \frac{aq}{2E}(1-\mu^2)\left(A - \frac{1-2\mu}{1-\mu}B\right) \qquad 5.2$$

where a = width of rectangular area

b = length of rectangular area

z = depth of point below semi-infinite surface

$A = f(m,n)$

$B = f(m,n)$

$m = \frac{b}{a}$

$n = \frac{z}{a}$

Since superposition is valid, the vertical displacement of any point within the mass can be obtained by a summation process. For cases where $z = 0$ equation 5.2 reverts to equation 5.1.

The correction for settlements (or heaves) for areas at depths below the ground surface, as given by Fox (1948), is not incorporated into the solution. For a 330 x 330 foot excavation about 40 feet deep, the correction ratio of mean settlement (heave) of an area at depth C to that located at the surface is 0.985. This means that boundary restraint is small when the overall excavation area is large.

To analyze the rebound at the AGT excavation it is necessary to deal with a large number of excavation areas, each possessing a different bottom elevation. Differences in unit weights of excavated soil, as well as depths, give rise to variations in unloading pressures. Since all rebound points are not at the same elevation, a further complication is introduced. To facilitate analysis of heaves at all rebound points for several excavation configurations a computer solution of equation 5.2 was written.

To solve for the rebound moduli of the AGT foundation, the rebounds at the specific locations of the rebound points were calculated using an arbitrary series of elastic moduli and Poisson's ratios. Interpolation using the observed rebound with the computer derived theoretical rebound set yielded the desired foundation modulus.

5.9 Computer Solution

The computer program, called hereafter the Steinbrenner computer solution, requires four specific sets of information.

i) Soil Profile Data

The elevation of the upper stratum boundary, the stratum thickness and its total unit weight are specified.

ii) Rebound Point Data

Here the X and Y coordinates are specified as well as the rebound point elevations.

iii) Excavation Data

An entire excavation is defined as a number of component blocks, each having a specified length, width, centroidal location and bottom elevation.

iv) Statistics

These control the computer solution. The most important parameter specified is DEL, the depth of individual excavation layers.

The excavation as it appeared on November 6, 1968 is shown in Fig. 5.16. A typical description of the December 20, 1968 excavation in terms of its 8 component excavation blocks is given in Fig. 5.17.

Starting at the ground surface, each excavation block is broken into slabs which possess the same horizontal configuration as their parent block, but are DEL feet thick. An exception is at the base of the block, where due to elevation specifications, the slab may be less than DEL feet thick. Depending on the location of the slab within the soil profile, each slab is assigned a specific weight per square foot, representative of the force it exerts on the next lower slab. If a particular block is shallow, such that an excavation slab at lower elevations is non-existent, an imaginary slab is defined with the assigned unload pressure specified as zero. Thus the entire excavation is defined as a two-dimensional matrix, with each term representing the unload in kips per square foot of a particular component slab.

A test trial run of the solution with variation of DEL between 0.5 and 3.0 feet, indicated that the computed elastic heave over this range was quite insensitive to the value used. A difference of only .02 inches occurred. Throughout the analysis of the AGT excavation the value of DEL was maintained at 1.0 feet.

FIG. 5.16 AGT EXCAVATION, NOVEMBER 6, 1968

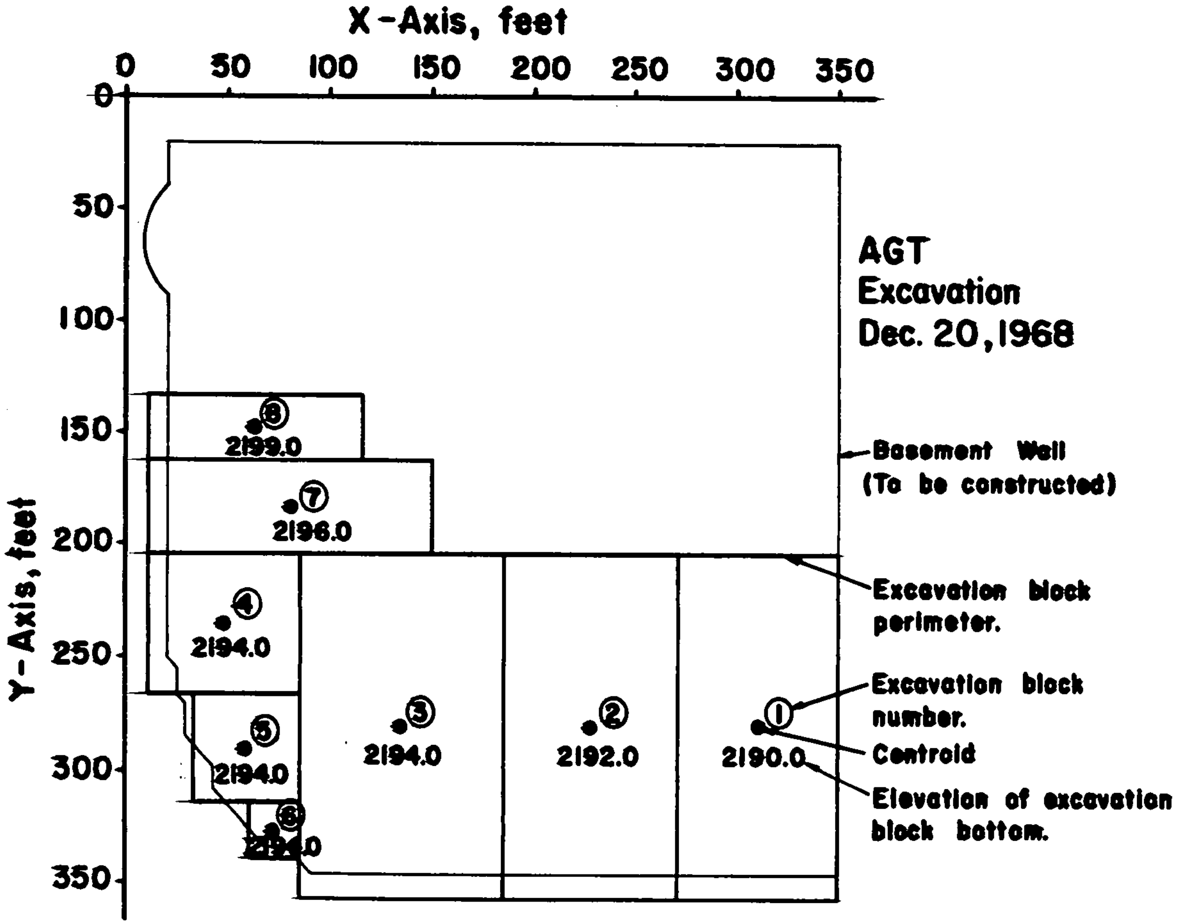


Fig. 5.17 Excavation Description in Terms of Component Blocks

Superposition techniquescan now be used to compute rebound. Rewriting equation 5.2 in a different form, the contribution to rebound at a specific point due to the removal of M excavation slabs can be written as shown in equation 5.3.

$$\Delta_j(z_j) = \frac{1-\mu^2}{E}\left[\sum_{i=1}^{i=M} a_i q_i A_i\right] - \frac{(1-2\mu)(1+\mu)}{2E}\left[\sum_{i=1}^{i=M} a_i q_i B_i\right] \qquad 5.3$$

Starting with the excavation slabs in the uppermost layer at the surface, the rebound Δ_j due to slabs in layer j is then simply a function of z_j, the depth of layer j to the rebound point, and the geometric distribution of the contributing slabs. The q_i surcharge in the equation is the unload pressure of an individual excavation slab and equals a specific term in the previously described excavation matrix.

Using a summation process, the removal of succeeding layers yields the total heave due to the entire excavation configuration. Thus the final heave at a single rebound point is:

$$\Delta = \sum_{j=1}^{j=N} \Delta_j(z_j) \qquad 5.4$$

where N is the total number of layers required to define the entire excavation. Thus, if the maximum excavation block is 40 feet deep and DEL is defined as 2 feet, twenty layers will be required.

To solve for rebounds at other locations, this procedure was repeated using the same excavation matrix but redefining z_j values consistent with the depths of the rebound points.

5.10 Comparison of Solutions

Two alternate finite element solutions were available for comparison with the Steinbrenner solution. For the special case of rebound displacements of an axi-symmetric solid, Wilson's (1965) solution was used.

Fig. 5.18 shows the 38.5 foot deep and the 180.0 foot radius circular excavation used in the axi-symmetric finite element solution. Superimposed on the circular area are twenty one excavation blocks, each 38.5 feet deep, needed for the Steinbrenner solution. Along one radius, points 33 to 45 define locations which were used in the comparison.

Using an elastic modulus of 90,000 psi and a Poisson's ratio of 0.4, the ratio of axi-symmetric finite element heave to the Steinbrenner computer solution heave was 0.80. An exception was point 45 where the ratio was 0.70 due to the imposed boundary conditions existing at that location.

The reasons for the different heave results are due to the following factors.

(a) The Steinbrenner solution assumes a surface plane of infinite extent. The stiffness of the 38.5 foot deep surrounding soil mass in the axi-symmetric finite element

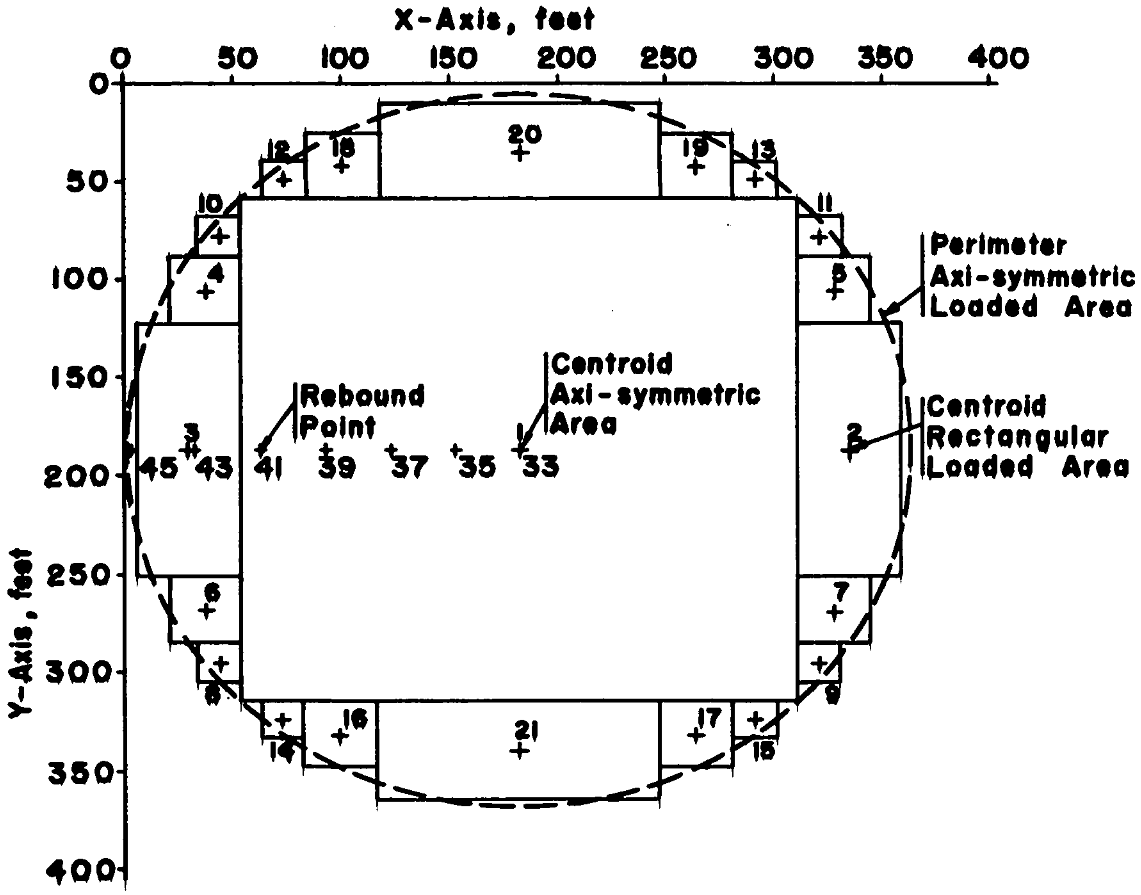


Fig. 5.18 Axi-symmetric Excavation Plan in Terms of Multiple Rectangular Blocks

solution will tend to reduce heave.

(b) The use of radial stress release along the vertical wall would tend to decrease the heave of the axi-symmetric solid, particularly for the excavation bottom near the side of the excavation. The input radial force was calculated using the relation $K_0 = \frac{\mu}{1-\mu}$ for a perfectly elastic medium.

(c) The axi-symmetric solution must be consistent with the boundary constraints of the cylindrical solid undergoing deformation. At a radius of 900 feet the boundary was prevented from undergoing radial distortion. At a depth of 1000 feet the cylindrical bottom was defined as a fixed rigid base. Although this cylinder was large, the use of a stratum of limited thickness and radial extent would give rise to smaller heaves than those calculated in the Steinbrenner solution using a semi-infinite solid.

The influence of the rigid base can be observed by computing the closed form solution for the displacement of a circular area founded on the surface of a layer of infinite lateral extent underlain by a rigid base. Using the influence values computed by Egorov (1958), the displacement at the centre of the circular area is 1.25 inches for a fixed base at 1000 feet. The axi-symmetric finite element solution predicts 1.07 inches, the ratio of axi-symmetric finite element heave to the Steinbrenner computer solution heave is then 0.86. The ratio of 0.86, as compared with 0.80

found earlier, reflects the influence of the rigid base assumption.

The Steinbrenner computer solution which uses superposition of rectangles is compared to the closed form solution also given by Steinbrenner (1934) for the displacement of a circular area founded on a semi-infinite elastic solid. For the centre point, again using a modulus of 90,000 psi and Poisson's ratio of 0.4, the respective heaves were 1.36 and 1.40. This close agreement verifies the Steinbrenner computer solution which was described earlier in this chapter.

A comparison of the Steinbrenner computer solution to Wilson's (1963) two-dimensional finite element plane strain solution was carried out using the real AGT excavation. This will be discussed in a subsequent section.

5.11 Foundation Moduli from Field Excavations

In an earlier section dealing with heave observations, it was noted that heave at points outside the excavation decreased rapidly with distance. This effect can be observed in Fig. 5.14, where rebound point 10, located about 100 feet east of the large N.W. quadrant, did not undergo any appreciable additional heave as this quadrant was excavated.

The Steinbrenner computer solution, as do the finite element solutions, predict appreciable heaves at these "exterior" locations, even though they are not observed in the field. The field moduli, derived from the observed heave response at these locations, are therefore increased.

This effect is indicated in Fig. 5.19, where at the completion of excavation of the N.E. quadrant, the derived foundation modulus for rebound point 10 is about 45,000 psi. The time dependent heave without any change in excavation dimensions, causes the field modulus to decrease to about 35,000 psi. No further heave takes place during excavation of the S.W. and N.W. quadrants. This is immediately reflected in an increased field modulus for rebound point 10, as predicted by the Steinbrenner solution.

The curve shown in Fig. 5.19 for rebound point 10 is very similar to the curve of derived Young's moduli plotted against time, as given by Serota and Jennings (1959) for heave in stiff fissured London Blue Clay. The variation in Young's modulus was attributed to swelling and consolidation, although undoubtedly a great deal was due to the above mentioned effect as excavation progressed at other than the location of the rebound markers. The interpretation and observation of this "artificial" increase in moduli, was greatly simplified at the AGT excavation site due to the greater number of rebound points available.

This same effect can be observed for rebound points 2 and 6, but to a much lesser extent. This is so because both points do undergo some additional heave because of their proximity to the S.W. and N.W. excavation quadrants.

Points 2, 6 and 10 are not subjected to reloading as they are located within the parking garage area. Point 2

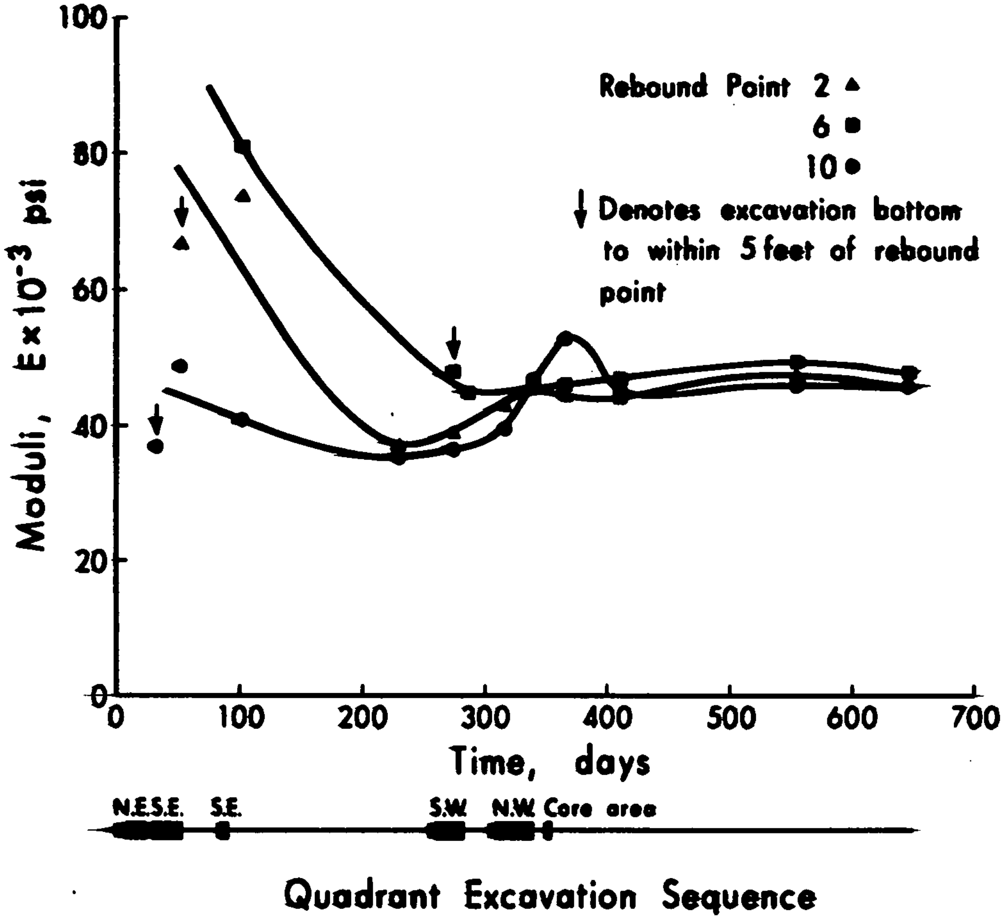


Fig. 5.19 Variation of Derived Modulus With Time, AGT Excavation

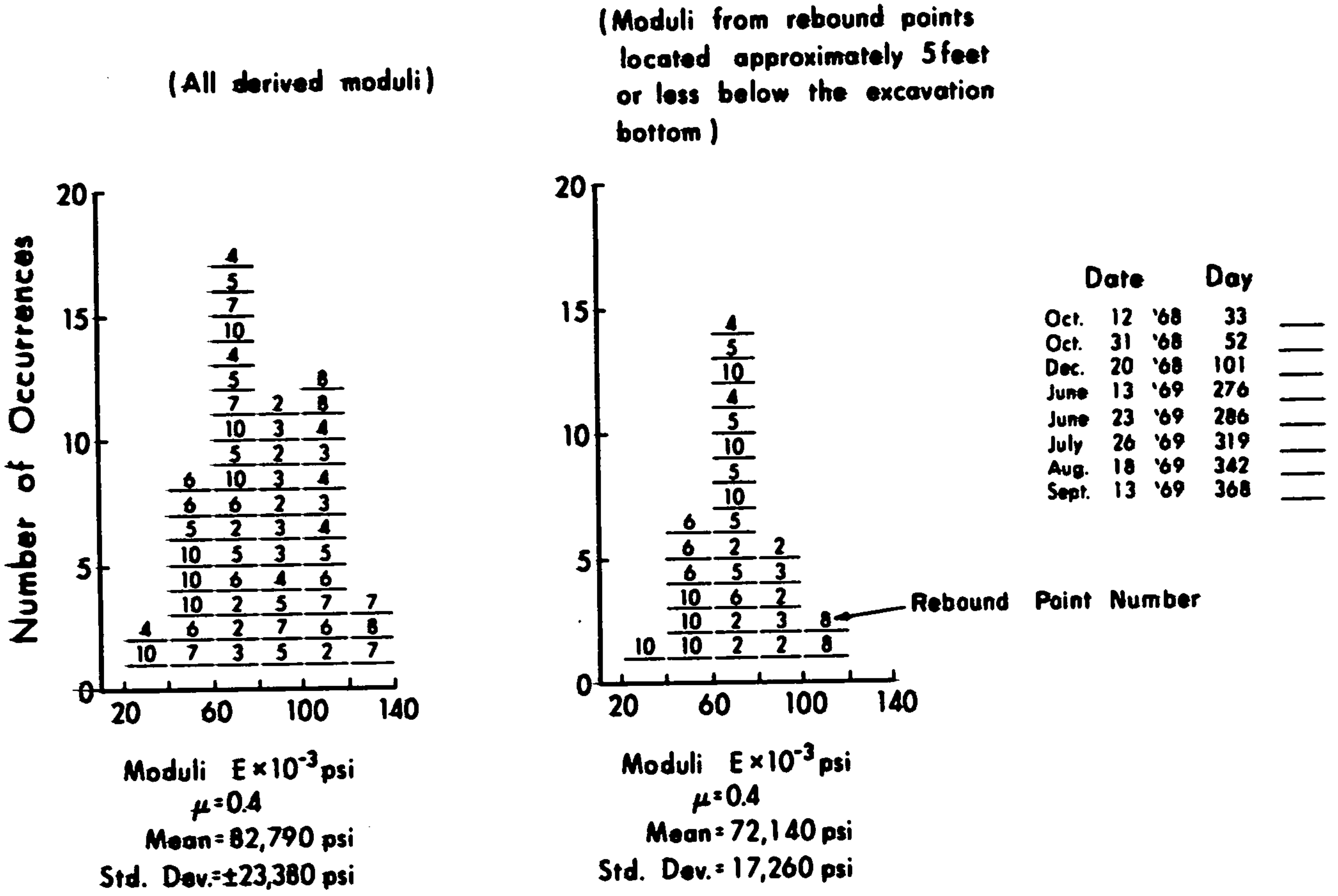
is centrally located, point 6 is near a side and point 10 is located off centre, towards an excavation corner. It is interesting to note that movements at all three points are such that within 60 days after completion of excavation, a uniform prediction of the foundation rebound modulus of about 47,000 psi is indicated.

The complication of the time effect and rapid decrease in heave outside excavation areas has made it necessary to carry out the analysis using two sets of observed rebounds.

The first set of rebounds is the accumulated observed heaves that take place during the excavation periods. This rebound is called "immediate rebound" and is delineated from the total observed rebound by the start and finish dates of quadrant excavations.

Using these rebound values with their corresponding excavation dimensions, the Steinbrenner solution allows the derivation of foundation rebound moduli as shown in Fig. 5.20a. The histogram shows the large standard deviation and reflects the effect of reduced observed rebound of points outside excavation zones.

Since the dates at which the excavation bottom nears the rebound points were known, it was possible to plot the histogram of only those moduli obtained from heave data taken subsequent to these dates. This effectively eliminates the inclusion of "artifically" high moduli computed for heave data obtained from exterior located points. The derived moduli for these computations are shown in Fig. 5.20b. Hence



(a) (b)

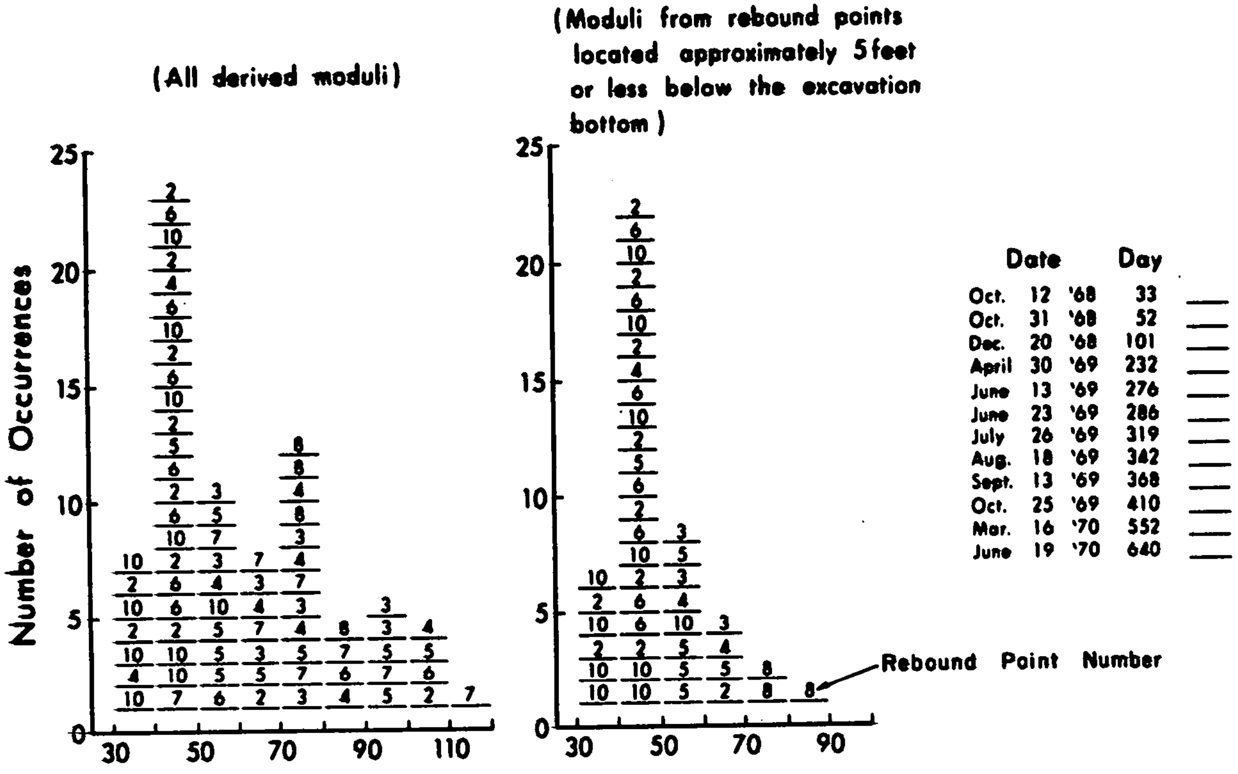
Fig. 5.20 Histograms of Moduli Derived From "Immediate" Rebound Measurements, AGT Excavation the "immediate" heave that can be expected during an excavation period and within the zone of excavation may be computed using a foundation modulus of 72,100 psi, and Poisson's ratio of 0.4, the accuracy of calculation being approximately ± 25 per cent.

The second set of rebounds which can be used in the analysis is the total observed rebound. These heaves include all the time dependent movement recorded during the period of investigation. Data taken subsequent to day 410 for rebound points subjected to reloading were not used in the analysis.

Using the Steinbrenner solution, the computed foundation rebound moduli are as shown in Fig. 5.21a. The removal of the high moduli computed for exterior located points yields the histogram shown in Fig. 5.21b.

From this data it can be concluded that heaves for excavations open for a considerable length of time, say to 6 months or more, can be computed using a foundation modulus of 50,300 psi and a Poisson's ratio of 0.4. This calculation would only be true for locations not subjected to reloading during the period of consideration. The accuracy of this calculation will be about ± 20 per cent.

The excavation on day 368 (September 13, 1969) was nearly complete and approximated a square. A section through the centre, including points 4, 5, 6 and 7, was analyzed by means of the finite element plane strain solution. Although the excavation was about 3 feet deeper at its north



Moduli $E \times 10^{-3}$ psi
$\mu = 0.4$
Mean = 62,490 psi
Std. Dev. = 20,380 psi

(a)

Moduli $E \times 10^{-3}$ psi
$\mu = 0.4$
Mean = 50,290 psi
Std. Dev. = 10,200 psi

(b)

Fig. 5.21 Histograms of Moduli Derived From "Total" Rebound Measurements, AGT Excavation end, averaging of excavation depth and conversion of the square to an equivalent circular area, allowed analysis by the axi-symmetric solution. Using the total displacements for day 368, the moduli computed by the three available methods could be tabulated as shown in Table 5.4.

The elastic solid used in calculating the moduli by means of the axi-symmetric solution is virtually identical to the body used in the comparison described in an earlier section. Factors giving rise to differences in computed moduli are described in that section and will not be repeated here.

The close agreement between the Steinbrenner derived moduli and those from the plane strain solution is likely fortuitous. The section used in the plane strain finite element solution had a rigid base at a depth of 1000 feet with lateral movements restricted at boundaries located at least 200 feet away from the sides of the excavation.

Because the plane strain solution predicts an infinite heave if an infinitely deep section is defined, the computed derived moduli for a rigid base section will be dependent on the chosen depth of the rigid base. Hence, if section depths substantially greater than 1000 feet were used, input of specific rebounds would result in derived moduli of increased magnitude.

One further observation that can be made from the collected rebound moduli is that the modulus of the bedrock formation is likely greater than the modulus of a homogeneous

TABLE 5.4

DERIVED MODULI FOR HEAVE AT DAY 368 BY THREE METHODS

Point	Observed Total Heave in.	Moduli Steinbrenner Solution psi	 Plain Strain Fin. Element psi	 Axi-symmetric Fin. Element psi
2	2.70	44,100	N.A.	35,000
3	2.16	55,900	N.A.	44,300
4	2.27	52,500	56,000	41,200
5	2.29	49,800	53,700	41,000
6	1.97	46,100	47,700	35,000
7	1.85	61,300	66,100	48,100
8	1.07	71,000	N.A.	N.A.
10	2.29	53,300	N.A.	33,800
		Avg. 54,250	Avg. 55,875	Avg. 39,770

"equivalent" foundation which will also include the Saskatchewan Sands and Gravels. This can be seen by comparing the moduli computed from data collected on rebound points 7 and 4, the difference usually being at least 5,000 psi. Although this evidence is only from a single rebound point, the fact that this difference is not greater makes analysis using a single Young's modulus reasonable.

5.12 Comparison of Field and Laboratory Results

The average moduli of "immediate" and "total" rebound can be compared to the moduli obtained in the laboratory from tests run on specimens taken at the AGT-Oxford complex site.

Ratios of field to laboratory moduli are given in Table 5.5 for 4 foundation soil types. The laboratory moduli presented in the table are the largest derived for the soil and test types quoted. As laboratory rebound data was not available for all specimens some moduli were calculated from compression data as indicated.

From the extreme variation it would appear that there is little or no comparison between laboratory or field moduli. The range of ratios is not only an expression of differences in soil type and test type but probably is also a reflection of the variation induced by differences in stress levels, consolidation procedures, sample types, sample disturbance, specimen sizes etc. Work by Ladd (1964) has illustrated that these factors are the primary cause of

TABLE 5.5

COMPARISON FIELD AND LABORATORY MODULI

Empirically derived field moduli
(a) "Immediate" rebound 72,140 psi
(b) "Total" rebound 50,290 psi

Soil Type	Laboratory Modulus psi		Laboratory Test	Ratio Field to Laboratory (a)	(b)
Till	1200	Rebound	Oedometer	60	42
	752	Rebound	Oedometer	96	67
	9100	Comp.	Unconfined Compression	8	5.5
Bedrock (Mudstone)	3500	Comp.	Unconfined Compression	21	14
	3410	Comp.	Oedometer	21	15
(Sandstone)	8000	Comp.	Unconfined Compression	9	6.3
	2090	Comp.	Oedometer	35	24
	41800	Comp.	Consolidated Undrained Triaxial	5	3.4
	5000	Comp.	Consolidated Drained	14	10
Saskatchewan Sands	31000	Comp.	Consolidated Drained	2.3	1.6
	90000	Rebound	Consolidated Undrained Phase II, Cycle 1	0.8	0.6
	78300	Rebound	Phase II, Cycle 2	0.9	0.6
	250000	Rebound	Phase III, Cycle 1	0.3	0.2
	38200	Rebound	Phase IV, Cycle 1	1.9	1.3

differences between field and laboratory moduli.

From the data given in Table 5.5 it is obvious that the use of laboratory moduli would lead to a large over estimate of field rebound. For the case of moduli obtained from unconfined compression tests, this observation was also made by Fadum (1948), where variations of field to laboratory results were reported in excess of 10.

An exception were the laboratory results derived from the cyclic consolidated undrained compression test run on the Saskatchewan Sands. In several instances the laboratory moduli exceeded the field value. Accompanied by great care in sampling and specimen preparation, the field to laboratory moduli ratio close to unity indicates that derivation of representative laboratory moduli is possible. However, any conclusive observation to this effect can only be made after an extensive program of laboratory testing and analysis of results.

One further observation that must be made is the apparent contradiction evident between the relative moduli of deformation obtained for the bedrock and sands in the laboratory and in the field. On the basis of laboratory data it is obvious that the Saskatchewan Sands possess a modulus of deformation at least 2 or more times greater than the values recorded for bedrock specimens. Yet on the basis of actual field behaviour the rebound at point 7 is such that it represents a bedrock rebound modulus about 5000 psi greater than the value recorded at point 4 above it. This means, in effect, that the Saskatchewan Sands possess a real deformation modulus somewhat lower than the bedrock. For the "total" rebound strain between rebound points 4 and 7 at day 342, an approximate estimate of the modulus of deformation of the Saskatchewan Sand deposit is 30,000 psi. However, this result cannot be considered as conclusive because it is based on a single set of horizontally coincident rebound points.

Since this difference in derived moduli becomes more apparent as the excavation approaches rebound point 4, perhaps it can also be attributed to a stress dependent modulus of deformation. At lower stress reductions, the sand and bedrock heave with similar rebound moduli. However, at a greater depth of excavation, the higher elevation of rebound point 4 gives rise to greater stress reduction near that location, thereby causing the relative reduction of field modulus in comparison with the values derived at the deeper rebound point 7.

This anomaly again points out the inconsistency of laboratory and field moduli of these overconsolidated soils and indicates the need for empirical data in solving for real heave response.

5.13 Procedure for Analysis of Heave Using Empirically Derived Moduli

As any deep excavation in the southern portion of the downtown Edmonton area (see Fig. 2.1) will approach the

Saskatchewan Sands deposit and bedrock, the heave response can be expected to be similar to that observed at the AGT-Oxford site. Therefore, the results of the analysis given in this chapter are applicable to practice.

From exploration at any site the stratigraphy can be determined. Samples taken during drilling can be used to derive the total unit weight of the soil to be excavated. From dimensions of the required basement, the excavation can be described in terms of component blocks as outlined earlier in section 5.9. Rebound points can be defined in terms of an arbitrary coordinate system wherever it is necessary or desirable to know the heave. Points inside and outside the excavation boundaries may be specified.

The computer solution as given in Appendix G can be used directly, since heaves are calculated for a specified variation of rebound moduli and Poisson's ratios. Computer output will consist of a listing of all input parameters, a listing of the two-dimensional excavation matrix, and tables of results. For each rebound point, a table of heaves is given with column and row headings corresponding respectively to foundation rebound moduli and Poisson's ratio. As for the analysis, a variation of E from 30,000 psi to 170,000 psi in increments of 10,000 psi is convenient. Because the solution is not very sensitive to variation in Poisson's ratio, specification of Poisson's ratio as 0.3, 0.4 and 0.5 is usually adequate. With this type of output the greatest amount of flexibility is achieved as no decision as yet is required regarding the foundation rebound modulus representative of the location.

For points located at an elevation near the bottom of the excavation and within its boundaries, there are two moduli that can be used to enter the tables. These are approximately 72,000 psi and 50,000 psi respectively for the "immediate" and "total" or relatively long term heave. If the excavation is open for a short period of time and the bottom is loaded rapidly, it will experience heave in accordance with the higher modulus, probably about 70,000 psi. If the excavation is to be left open for, say 6 months or more, or is lightly loaded, the lower modulus of 50,000 psi may be used. For intermediate situations, a modulus between these two values can be chosen to compute the excavation rebound.

If it is desirable to know the rebound at depths considerably deeper, say at more than 1/3 the excavation depth beyond the excavation bottom, then a somewhat higher modulus should be chosen. In the case of "immediate" rebound, an increase of 5,000 psi to 75,000 psi would be representative of field behaviour. For excavations open for a considerable period of time, the increase can be greater, say from 50,000 psi to 60,000 psi.

The greatest need for exercise of judgment is for determining heave of points located outside excavation boundaries. At points outside the excavation, boundary restraint influences heave such that it is much less than computed using the previously given moduli. Several methods can be used to estimate the heave.

The heaves for excavations 45 feet deep seem to decrease very rapidly to near zero at a distance equal to approximately the depth of excavation away from the sides. Using this observation, a linear distribution can be assumed from zero heave to the value calculated for a point located opposite at the excavation boundary. Although somewhat arbitrary, this approximation should be representative of the behaviour of the downtown Edmonton soils.

A second procedure for estimating heaves at these exterior points is to enter the heave table with an "artificially" high modulus. Since time dependent heave at these points appears to be small, differentiation between "immediate" and "total" rebound can be neglected. On the basis of the AGT data, the heave at exterior points takes place with field moduli approximately equal to 120,000 psi or greater. One drawback to this procedure is that heaves will still be specified at distances greater than the depth of excavation, even though they are negligible at these points.

5.14 General Observations and Summary

From the work outlined in this chapter there are several implications of importance to construction practice which should be recognized.

(a) Sequential excavation of specific zones within an excavation boundary implies sequential heave. Although the reduction in heave away from an excavation zone is rapid, concrete structures located immediately adjacent to a zone are subject to heave. At the AGT site this effect is illustrated in Fig. 5.12 at rebound point 2, located about 10 feet away from the toe of the N.W. quadrant excavation zone. The heave there was about 0.3 inches.

Readings taken by the contractor's surveyor on columns placed in cuts in the N.W. quadrant side slope indicated a heave of approximately 3/4 inch. As no effort was made to establish precise control at the survey locations, the reliability of these readings is not known. A slight opening of a construction joint, which disappeared at a distance away from the N.W. quadrant, indicates that this phenomenon definitely takes place.

In order to minimize this effect phased construction should be discouraged at locations at distances less than 1/2 the depth of excavation away from a zone which is to be excavated at a later date.

(b) The occurrence of rapid but still time dependent heave suggests that lightly loaded areas should be constructed as late as possible. This means that slabs on grade should not be poured within two or three months of excavation, as immediate placement will result in heaves which could exceed 1.0 inch.

(c) Points located at or outside the excavation boundary will be subject to heave. This effect should be understood by designers of any project.

(d) If a tower structure and lightly loaded structure are both located within the same excavation, the effect of long term heave of the lightly loaded structure will accentuate the differential displacement between the two as the tower structure settles.

The effect of long term heave will also be observed in the lightly loaded structure as minor fluctuations and variations of elevations. Although not leading to structural distress they should be recognized for what they are.

From the analysis outlined in this chapter the following aspects are the most important and are summarized below.

(a) The heave response due to excavation is rapid. The "immediate" rebound taking place during the excavation period constitutes approximately 60 per cent of the "total" rebound measured over approximately a 1 1/2 year period. This response can be computed for downtown Edmonton locations using elastic theory and the moduli given in the chapter.

(b) Time dependent heave is evident at the AGT-Oxford site and in calculation is included in the "total" rebound quantity. This component of the response is rapid and virtually ceases 2 to 3 months after excavation. At locations well below the bottom of the excavation the time dependent heave is much smaller than that observed near the excavation surface.

(c) Elastic properties as determined in the laboratory differ greatly from field moduli and therefore cannot be used to predict heave movements. There is some indication that cyclic consolidated undrained compression tests may yield realistic data, but additional study is required for confirmation.

(d) The response of structures located at distances away from the excavation is considerably less than predicted from elastic theory. At distances greater than the depth of excavation the rebound response appears to be virtually non-existent.

(e) The greatest portion of the heave recorded at the AGT-Oxford site takes place within the bedrock stratum. The expansion of the Saskatchewan Sands deposit is measurable and cannot be considered to equal zero.

CHAPTER VI

SETTLEMENTS AT THE OXFORD BUILDING AND THE AGT TOWER

6.1 Introduction

In this chapter, the settlement behaviour of the two buildings which comprise the AGT-Oxford Complex will be examined.

The geology of the downtown Edmonton location was described earlier in Chapter II and is therefore not dealt with in this chapter. Discussion of the stratigraphy at the AGT-Oxford Complex site and the relevant geotechnical properties of individual component strata have been given in Chapter V.

6.2 Location and Description of Buildings

The specific location of the AGT-Oxford Complex was shown in Fig. 5.1. The AGT Tower is somewhat the larger of the two with a gross floor area of 21,900 square feet as compared to 17,820 square feet for the Oxford Building.

Although very similar in external architectural appearance the buildings differ structurally. The 26 storey Oxford Building is entirely constructed of reinforced concrete with a centrally located core. Both internal and external columns rise the full height of the building. By external columns are meant all those located at the building perimeter and connected to one another by a 2' - 7" deep by 8" wide spandrel beam. Typical floors consist of a two-way 7" slab with drop panels at all internal and external columns.

The foundation plan of the Oxford Building is shown in Fig. 6.1. The external columns along the south side of the building are supported on a strip footing. The core is founded on a single pad in the centre of the building. Due to legal difficulties in site procurement the northern half of the columns located at the east side of the building were founded on bored piles, hand excavated to a rectangular shape at their base. All other columns were founded on individual spread footings as shown.

The 34 storey AGT Tower possesses only externally located columns connected to one another by a 2' - 7" deep and 1' - 4" wide spandrel beam. As shown in Fig. 6.2 each column is supported by individual footings most of which were poured one immediately adjacent to the next. The rigid core located in the building centre is made up of four units each founded on its own footing pad which is 6 feet thick. The core sections are connected to one another by deep reinforced concrete beams thus essentially resulting in a single rigid monolithic core unit.

The absence of internal columns resulted in a clear span distance of about 39 feet between the core walls and the spandrel beam. For typical floors this distance was bridged by 2' - 2" deep steel frame joists located at 4' - 9" centers. These were covered by metal decking and in turn covered by

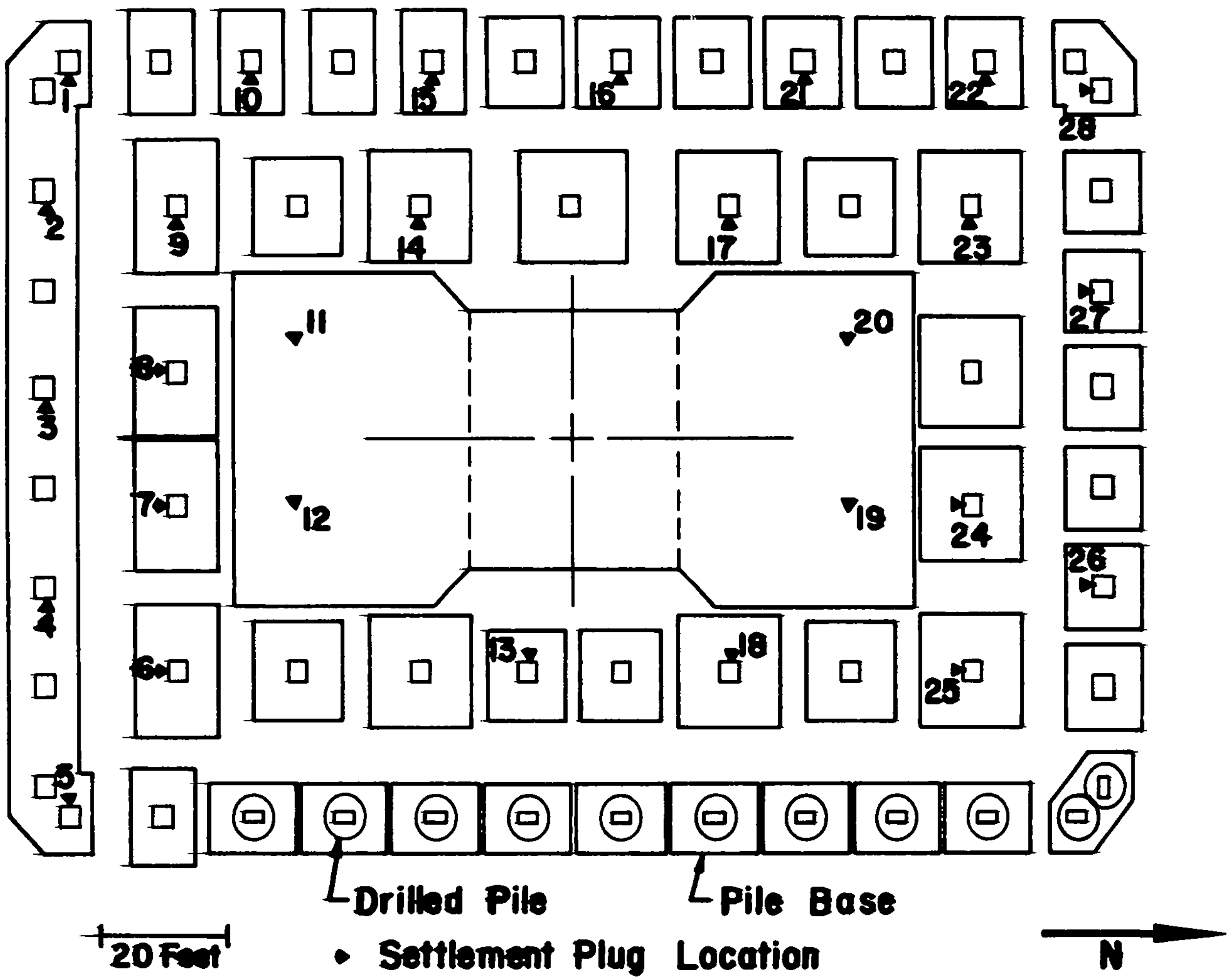
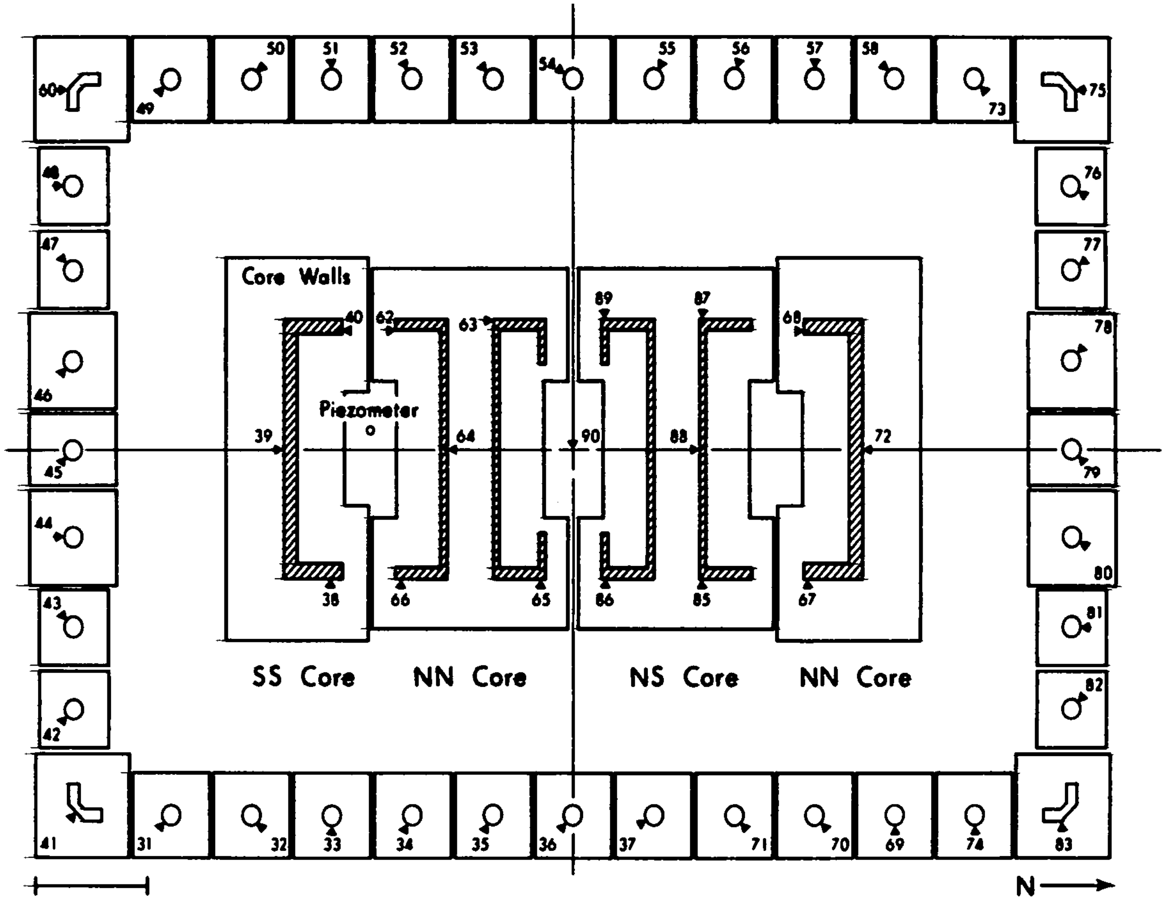


Fig. 6.1 Foundation and Footing Plan
Settlement Plug Locations, Oxford Building



O Indicates Location of Settlement Plug in Lower Parking Garage and Core Area

Fig. 6.2 Foundation and Footing Plan Settlement Plug Locations, AGT Tower a 2 1/2" layer of concrete. The resulting floor system makes the AGT Tower relatively lighter and flexible in comparison with the Oxford Building.

A north-south section through the AGT-Oxford complex is shown in Fig. 6.3. The Oxford Building is complete and has been occupied since January 1, 1970. As of September 22, 1970 construction of the superstructure of the AGT Tower had progressed to the 29th floor.

As shown in the section the elevation of the Oxford footings vary considerably in contrast to the relatively uniform elevation of the AGT Tower foundation system. Because the till stratum is a competent bearing material throughout most of its depth and since the proximity of adjacent structures and roads made it undesirable to locate a deep basement under the entire Oxford Building, the south-north transition from the 45 foot deep underground parking garage to the concourse floor was adopted.

The AGT Tower core footings and some exterior column footings were constructed on the Saskatchewan Sands and Gravels. The balance of the footings were excavated entirely in the till usually with only a few feet remaining over the Saskatchewan Sands and Gravel stratum.

For both the Oxford Building and the AGT Tower, footing excavation was carried out by machine with final trimming and cleanup of disturbed soil done by hand. The excavations were not left open for more than one day and were poured neat. An exception to this were the core footings which

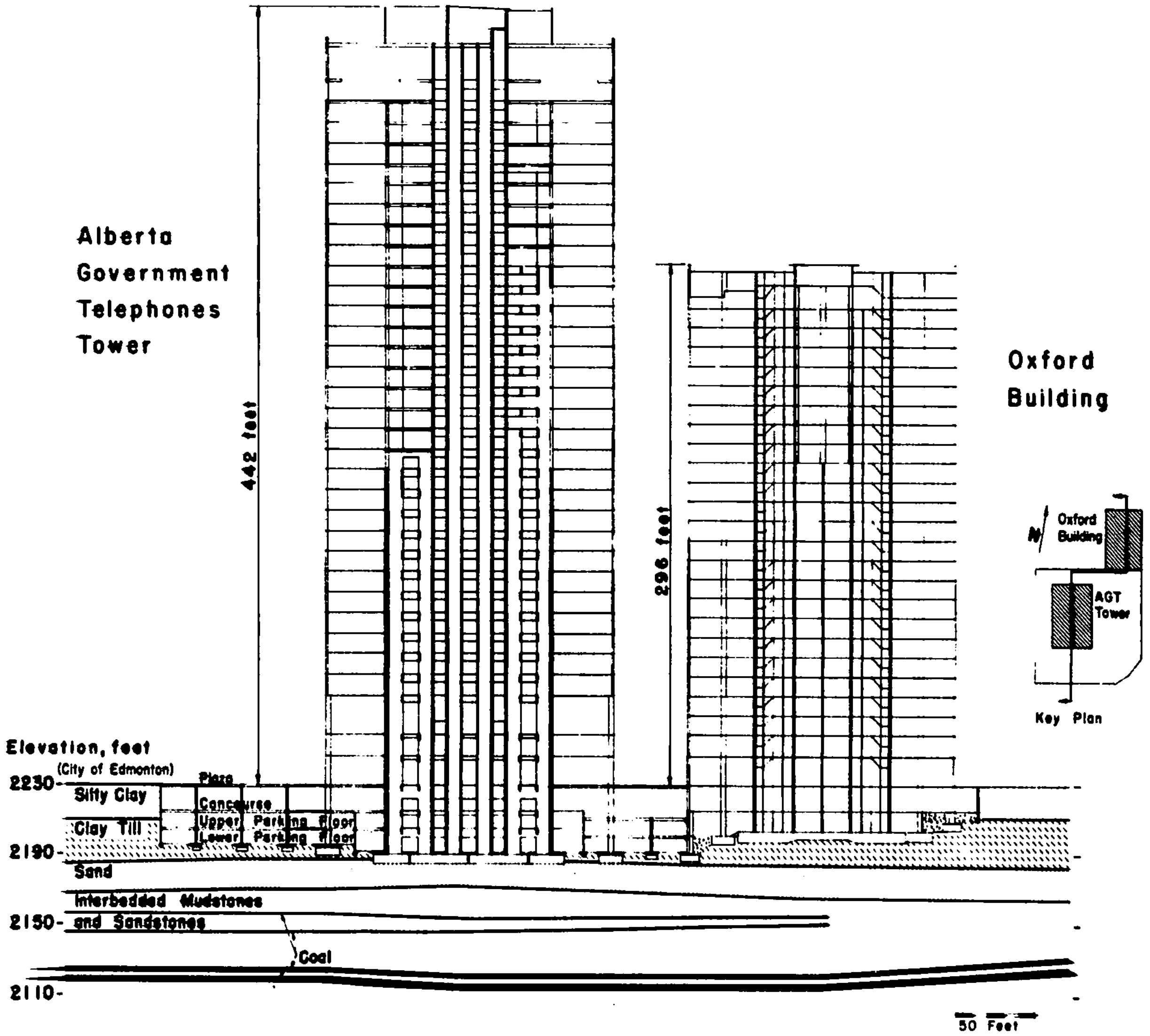


Fig. 6.3 Section Through the AGT-Oxford Complex

were formed at the sides.

6.3 Instrumentation and Data Acquisition

Settlements at the Oxford Building were obtained by measuring the change in elevation of 28 settlement plugs placed in columns at the concourse floor. Their locations are shown in Fig. 6.1.

The installation of settlement plugs made use of commercially available tools and supplies. Upon removal of column forms an impact hammer was used to drill a 5/8" diameter hole into the concrete sufficiently far to ensure that a high strength steel tubular insert would be flush with the concrete surface. A conical expansion slug was used to expand the slotted tube upon insertion so that it became completely immovable relative to the concrete. To obtain elevations a 3/4" diameter stainless steel pin was threaded into the settlement plug and tightened thus allowing a rod to be set on the pin. Upon removal of the steel pin an oiled bolt was used to seal the settlement plug.

Settlement readings on the Oxford Building were referenced to benchmark no. 3 located on the Macdonald Hotel about 220 feet away as shown in Fig. 5.1. Because it is necessary to cross a street and enter the concourse at a lower level in the Oxford Building the survey traverse is complex. Although the traverse usually closed within .1 cm. the error in the survey could exceed this value. Readings were taken by precise level and a rod scaled in millimeters whenever a further four or five floors were added.

At the AGT Tower a total of ninety settlement plugs identical to those at the Oxford Building were located in the lower parking floor of the AGT-Oxford complex. As shown in Fig. 6.2 they were installed in all exterior columns and at several locations in the AGT Tower core area. Settlement plugs were also installed in columns located in the underground parking garage as shown in Fig. 6.4. This allowed observation of settlement differentials between the AGT Tower and the surrounding parking garage. To facilitate surveying all settlement plugs were located on column sides directly visible from a particular instrument location.

Elevations at the AGT Tower were determined by precise levelling techniques making use of a Wild N-3 instrument and a rod scaled in millimeters. Prior to pouring of the slab on grade, survey turning points consisted of 1/2" steel pins driven into the ground. Subsequent turning points consisted of a heavy 5" diameter steel plate which acted as a 1 1/2 inch high tripod and could be located anywhere on the concrete floor slab.

The total length of traverse around the AGT Tower is approximately 800 feet and requires about twelve turning points with foresights and backsights balanced as closely as possible. Readings were visually estimated to the nearest .01 cm. in order to reduce the total traverse error, usually less than ± 0.1 cm.

Elevations for the traverse were referenced to DBM #2

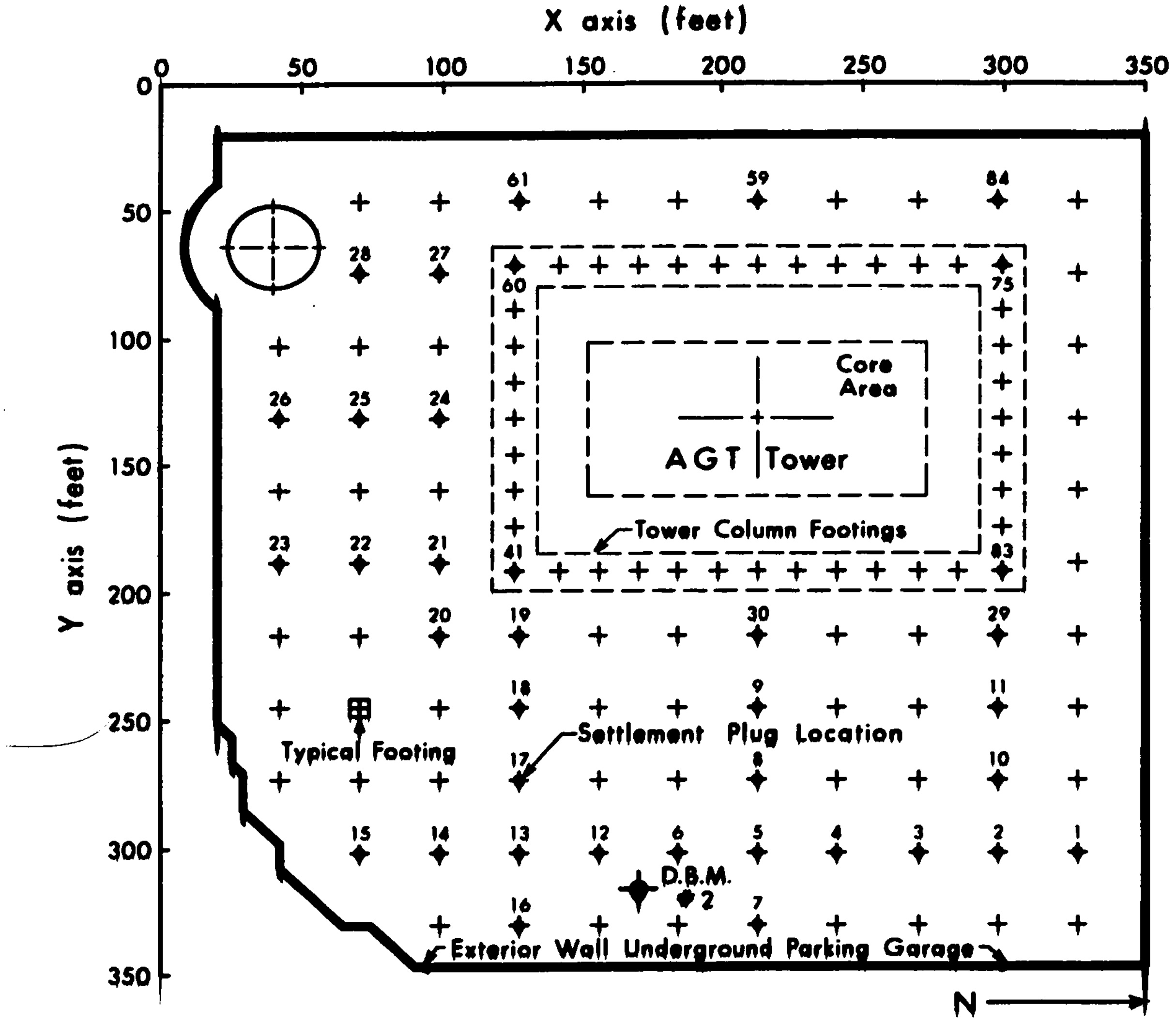


Fig. 6.4 Location of Settlement Plugs in Lower Parking Garage, AGT Tower

located as shown in Fig. 6.4, approximately 120 feet from the AGT Tower. The record of benchmark control is given in Appendix D. The 100 foot deep benchmark was installed at the completion of excavation of the S.E. quadrant (see Fig. 5.7). It consists of a cased 3/4" pipe which tapers to a 1/2" pipe and rises to just below the slab on grade in the lower parking garage. The installation procedure essentially followed that outlined in Chapter II for the deep benchmarks located at the CN Tower.

The initial set of elevations were taken on the AGT Tower columns prior to construction of the first upper parking level floor. Hence the only load acting at the soil footing contact at that time was the weight of the footing itself and the first column or wall resting on it. Subsequent readings were taken at approximately equal intervals of load application; i.e., whenever four or five floors were added to the AGT Tower. Several sets of elevations could therefore be obtained for the 34 storey building during its construction.

As indicated in Fig. 6.2 one piezometer was installed in the lower parking core area between the SS core and SN core footings. The piezometer tip was set in the bedrock formation about 9 feet below the bedrock surface. The piezometer is the University of Alberta transducer type (Brooker et al., 1968) and has been in operation continuously since the start of construction. Readings were taken approximately once every week throughout the period of construction.

6.4 Building Loads

Because no columns were instrumented to provide data regarding the load they carried, the procedure for determining the loading history at the Oxford Building was based on the following three assumptions.

(1) The proportion of building load carried by a single footing is in accordance with the distribution calculated using structural design loads.

(2) Concrete quantities and weight of precast exterior cladding can be used for footing load determination.

(3) The reduced live load acting on specific footings equals 20% of the total dead load carried by the footing.

Use of assumptions 1 and 2 allowed prorationing of building loads in accordance with the percentage carried by an individual footing to the total. Records of concrete volumes and placing of precast were used to provide the total building load at a particular date. Assumption 3 is based on personal communication with the structural consultant and allows for an estimate of the live load at the completion of construction.

A more precise procedure was followed at the AGT Tower since the construction of this building coincided with the period wherein this analysis was undertaken. Day to day records of volumes of concrete and location of placement were available such that loads could be accurately distributed to the appropriate supports. Measurements of brick walls and weights of beams, joists, decking and precast cladding were used to calculate footing loads at particular instances of time. Miscellaneous items such as exterior glass, heavy mechanical equipment and mechanical pads were included upon installation in the building. Of necessity electrical wiring and piping had to be excluded as detailed tabulation of these minor weight components would be a near impossible task. It is considered that the procedure outlined here has allowed the most precise calculation of footing loads possible under conditions of actual construction practice.

6.5 Settlement Observations

The maximum settlement recorded to November 4, 1969 at the Oxford Building was 2.8 cm. for settlement plug 13. The maximum differential settlement of 0.9 cm. over a 25 foot span, at the south-east corner of the building, is considered to be within acceptable limits for a building of this type (Feld, 1965). Since settlements readings were taken on only one-half of the Oxford Building footings it is possible that the maximum settlement and differential settlement is greater than that noted above.

The load and settlement histories for twenty-eight Oxford Building footings during the construction period to completion and full occupancy are illustrated in Figures 6.5, 6.6, 6.7, 6.8 and 6.9. The initial settlement reading was taken on April 2, 1968 with the main floor slab at ground level complete at that time.

The settlement record is similar to that observed at

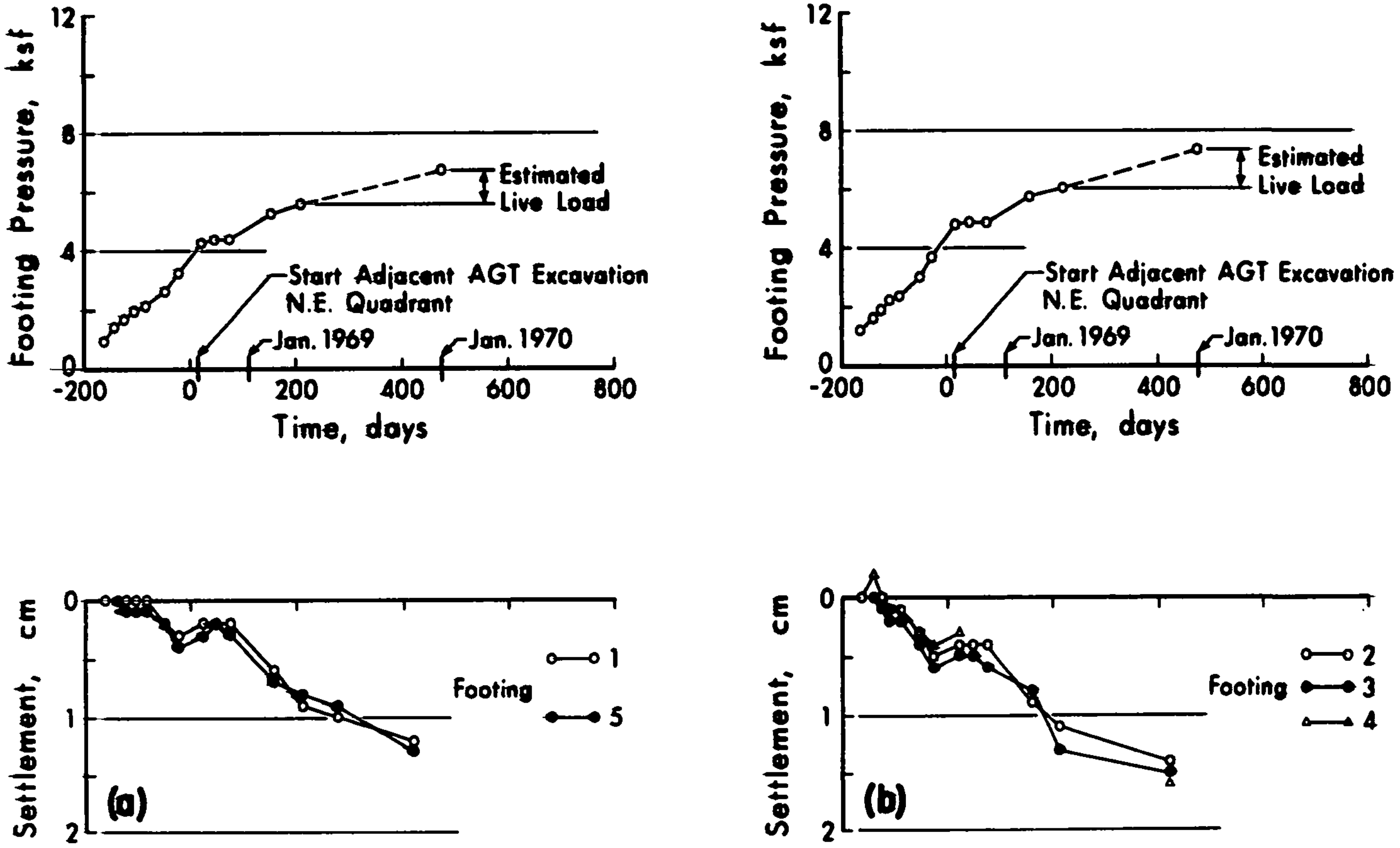


Fig. 6.5 Load and Settlement History, Oxford Building

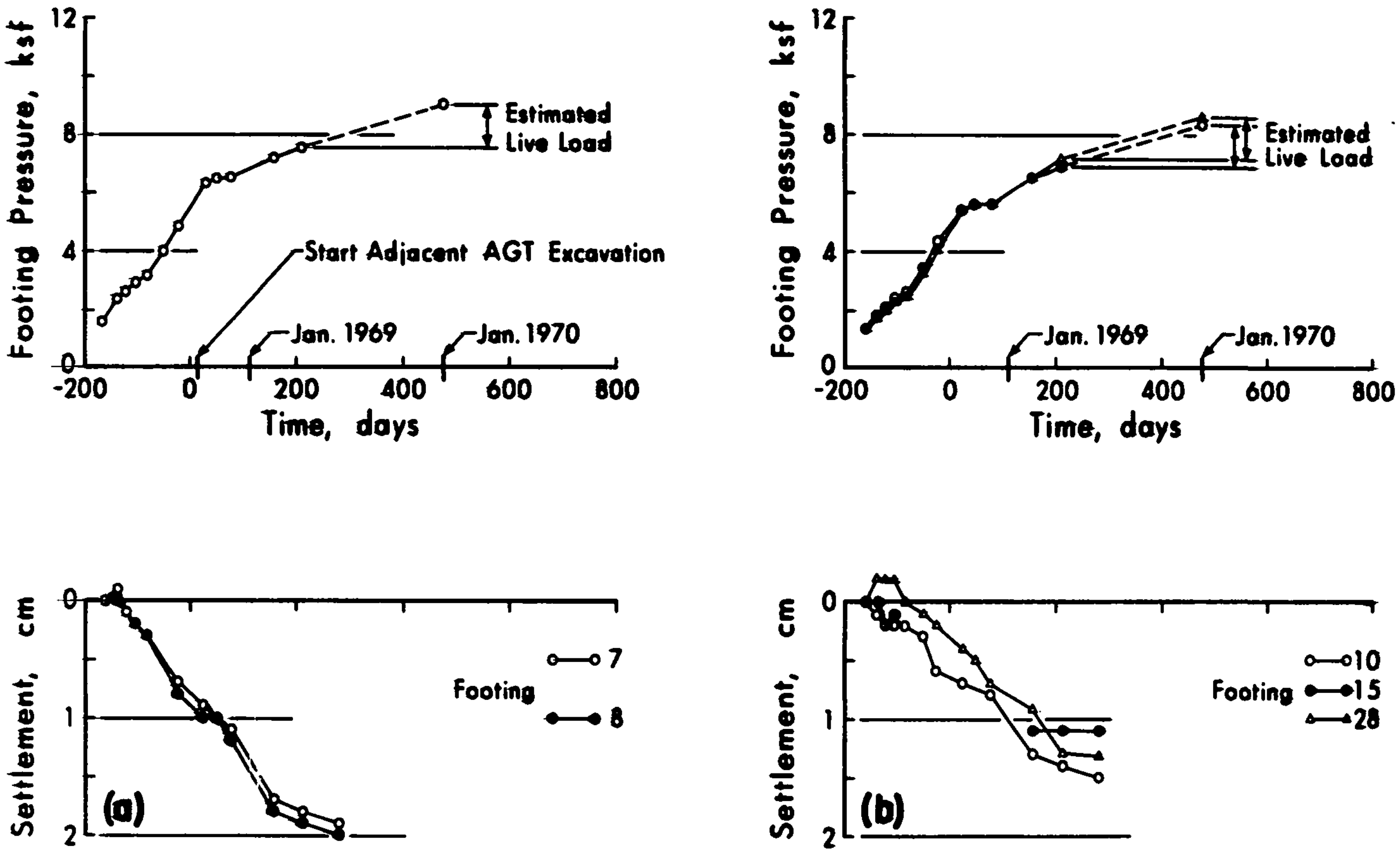


Fig. 6.6 Load and Settlement History, Oxford Building

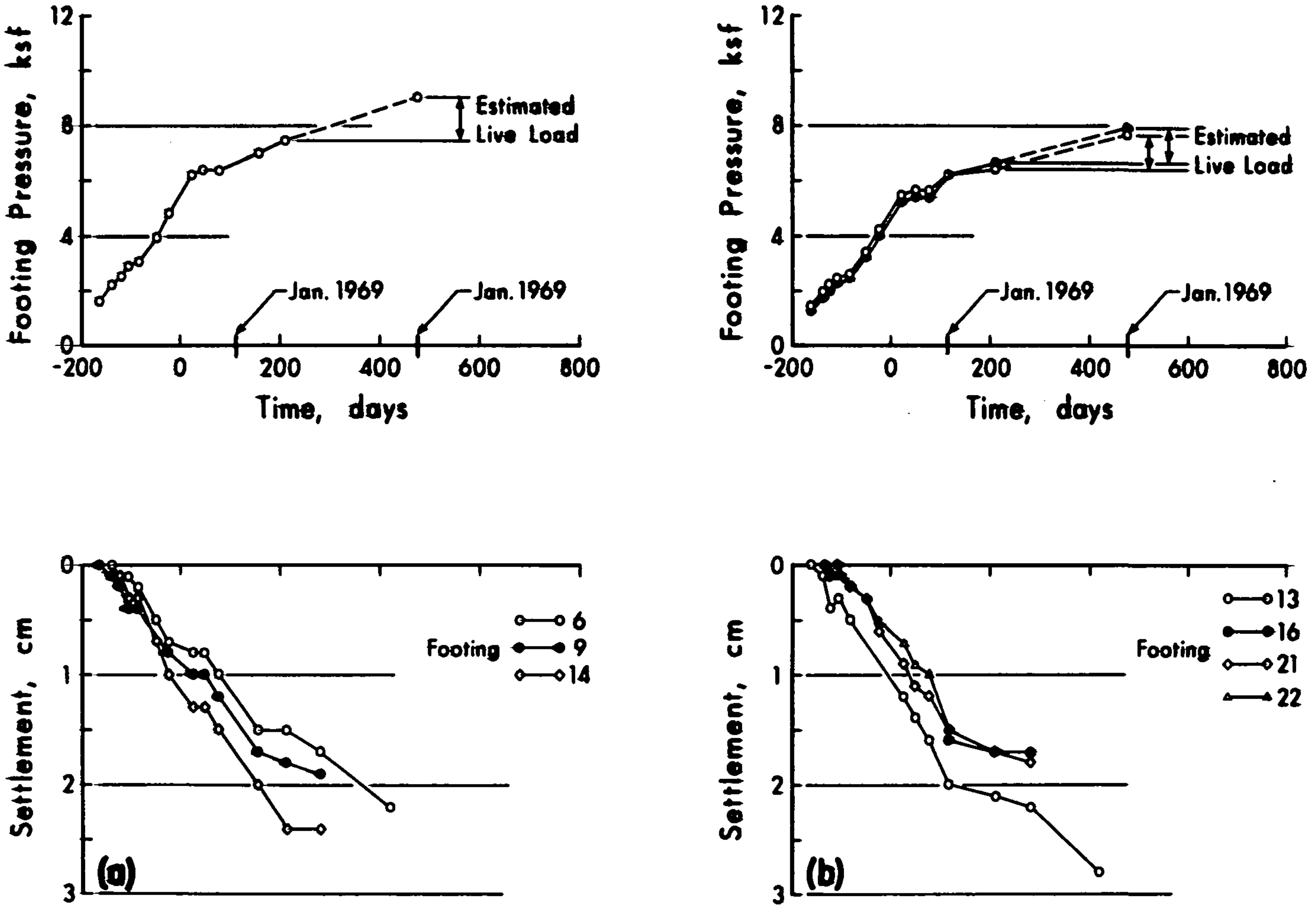


Fig. 6.7 Load and Settlement History, Oxford Building

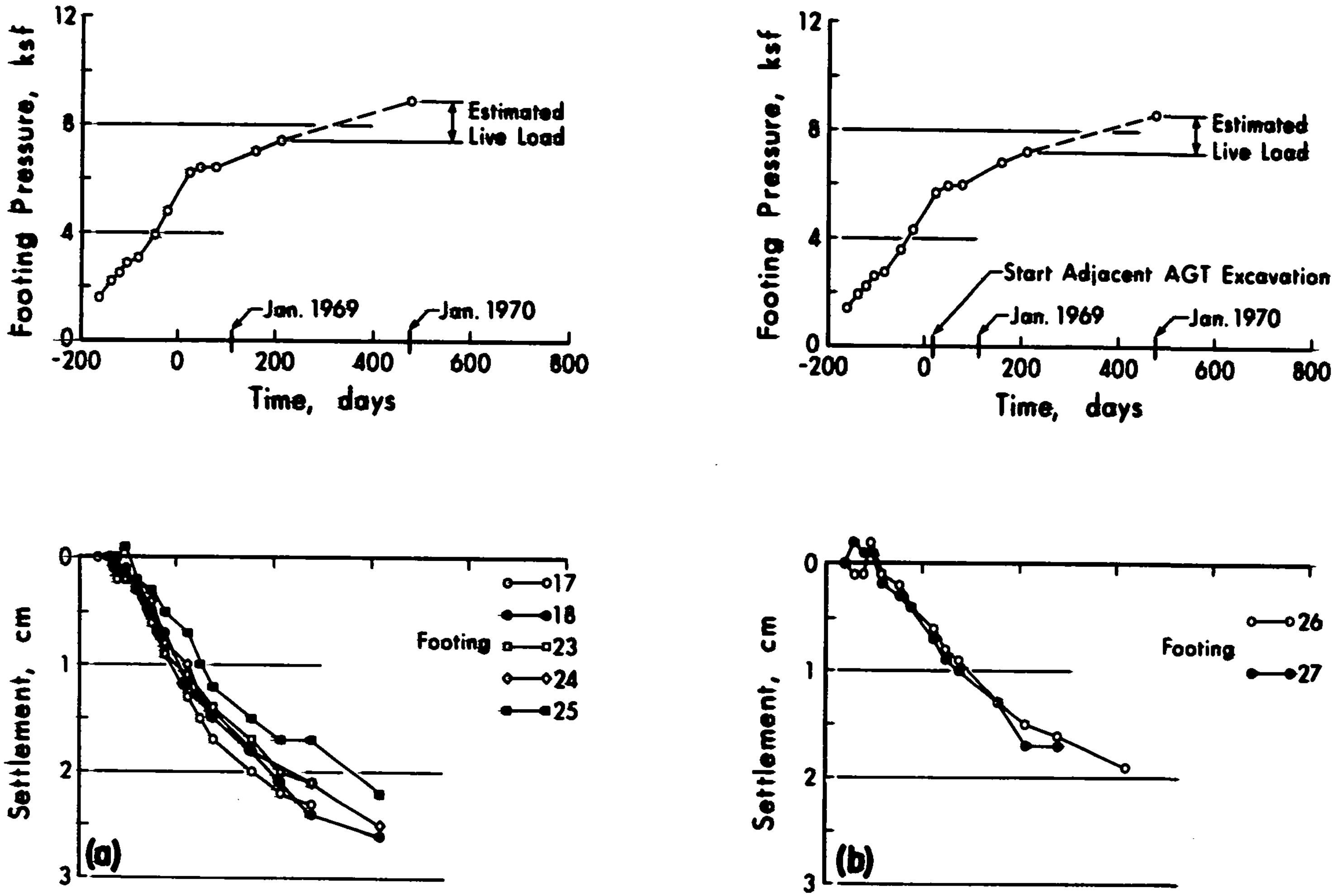


Fig. 6.8 Load and Settlement History, Oxford Building

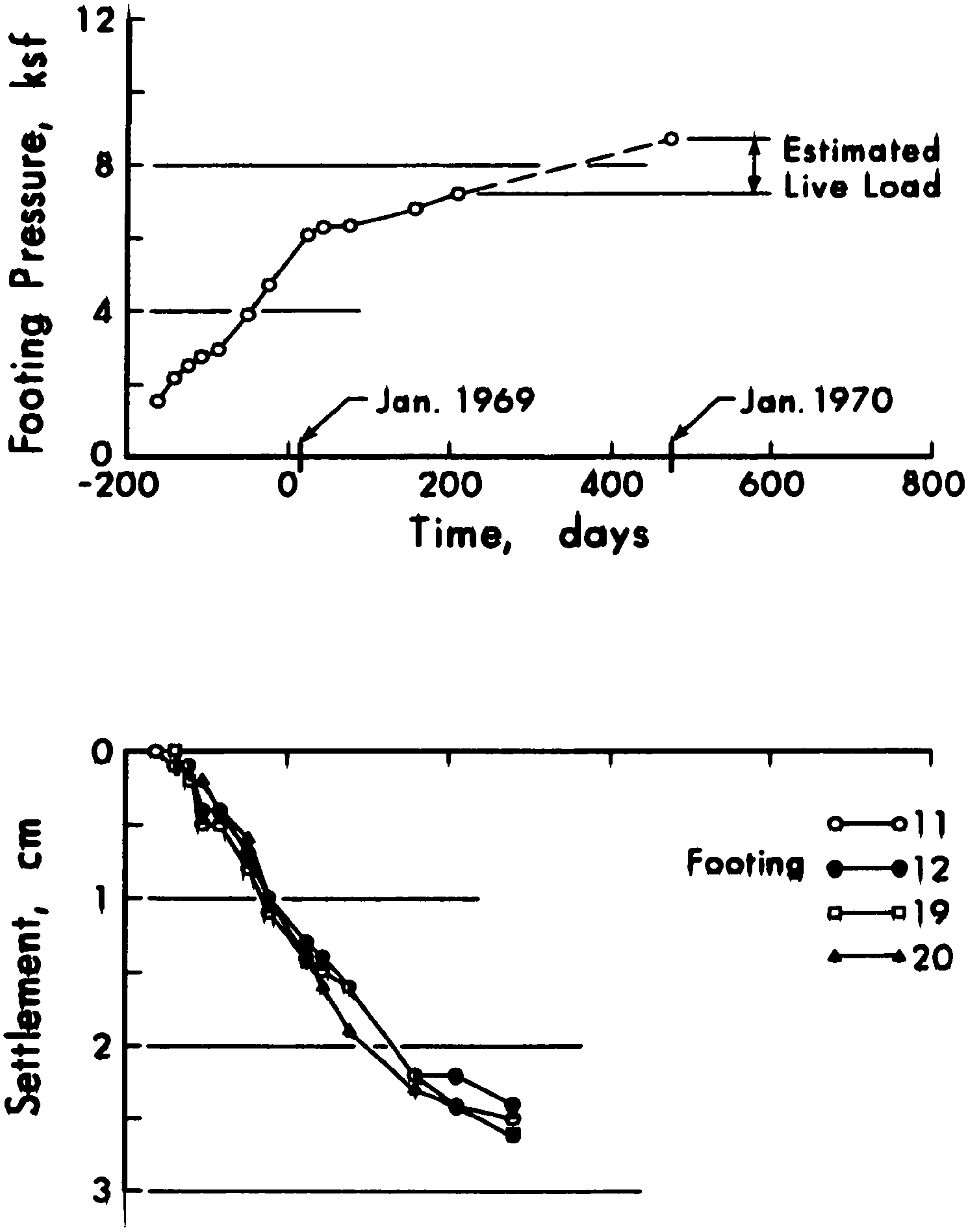


Fig. 6.9 Load and Settlement History, Oxford Building

the CN Tower described earlier in Chapter II. The settlement response of the foundation appears to be related to the footing contact pressure and is quite rapid. This is particularly evident in Figs. 6.8 and 6.9 for footings located in the interior of the building. Although the settlements measured were limited to the construction period there is some evidence of time dependent settlement behaviour. Even though construction ceased during a 29 day strike which started at day 46, settlement continued albeit at a decreased rate. This can be seen in Figs. 6.7 and 6.8 for footings 21, 22, 26 and 27 and is indicative that some time dependent settlement will take place subsequent to full occupancy. Due to the effect of the adjacent excavation for the AGT underground parking garage, which started about 3 weeks prior to the strike, the slight decrease in rate of settlement is completely masked by heave at footings located at the south end of the Oxford Building. The heave of these footings is readily evident in Figs. 6.5 and 6.6 and was described earlier in Chapter V. The north-south settlement profiles through the Oxford Building shown in Fig 6.10 indicate that the settlement pattern is typically "bowl shaped" with some distortion due to heave as shown for the February 14, 1969 data.

The settlement record for the AGT Tower to September 22, 1970 is limited to that occurring during construction up to the 29th floor. Because of the quantity of data, the settlements shown in Figs. 6.11, 6.12, 6.13, 6.14 and 6.15

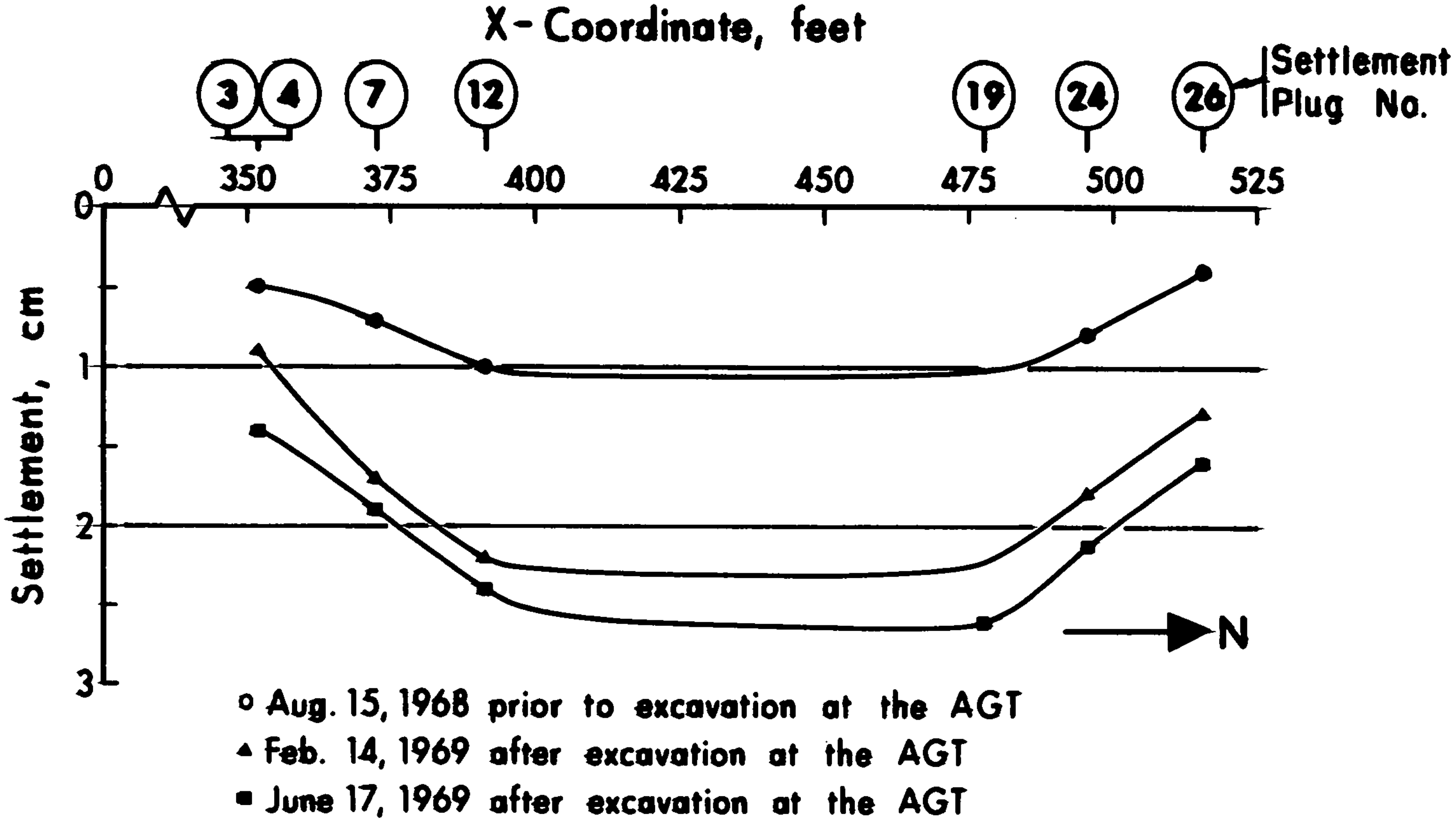


Fig. 6.10 North-South Settlement Profiles, Oxford Building

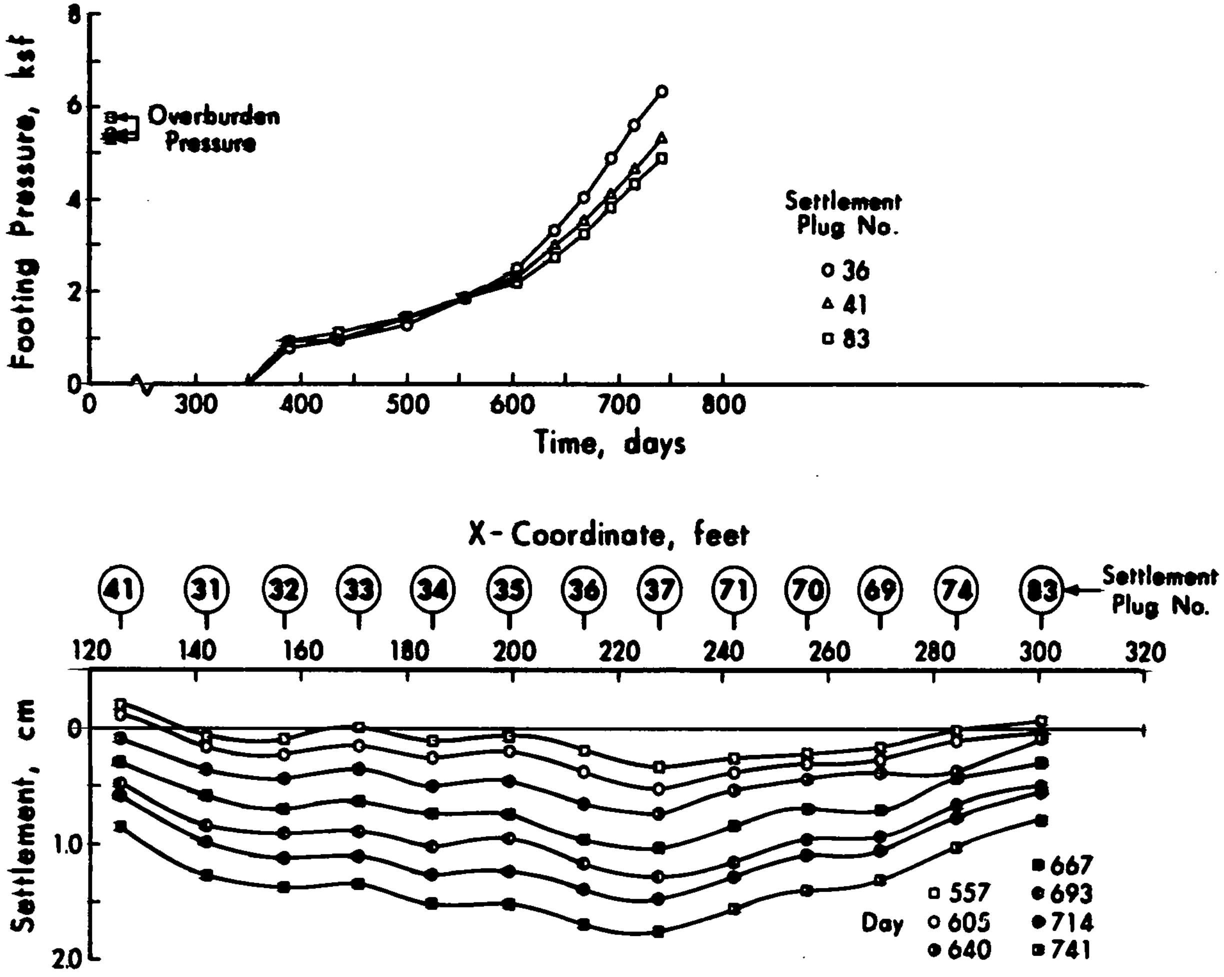


Fig. 6.11 Load History and Settlement Profiles of the East Exterior Column Line, AGT Tower

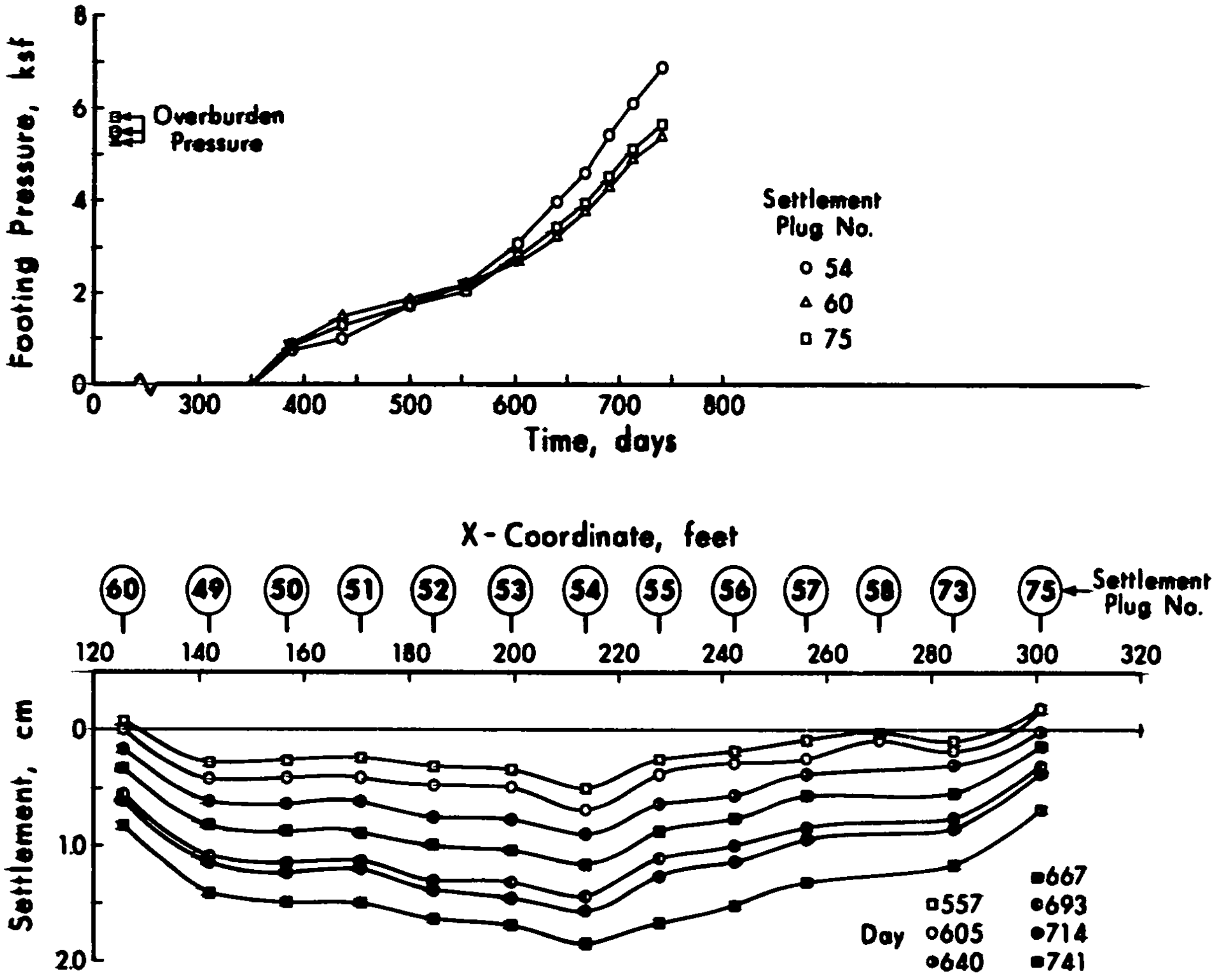


Fig. 6.12 Load History and Settlement Profiles of the West Exterior Column Line, AGT Tower

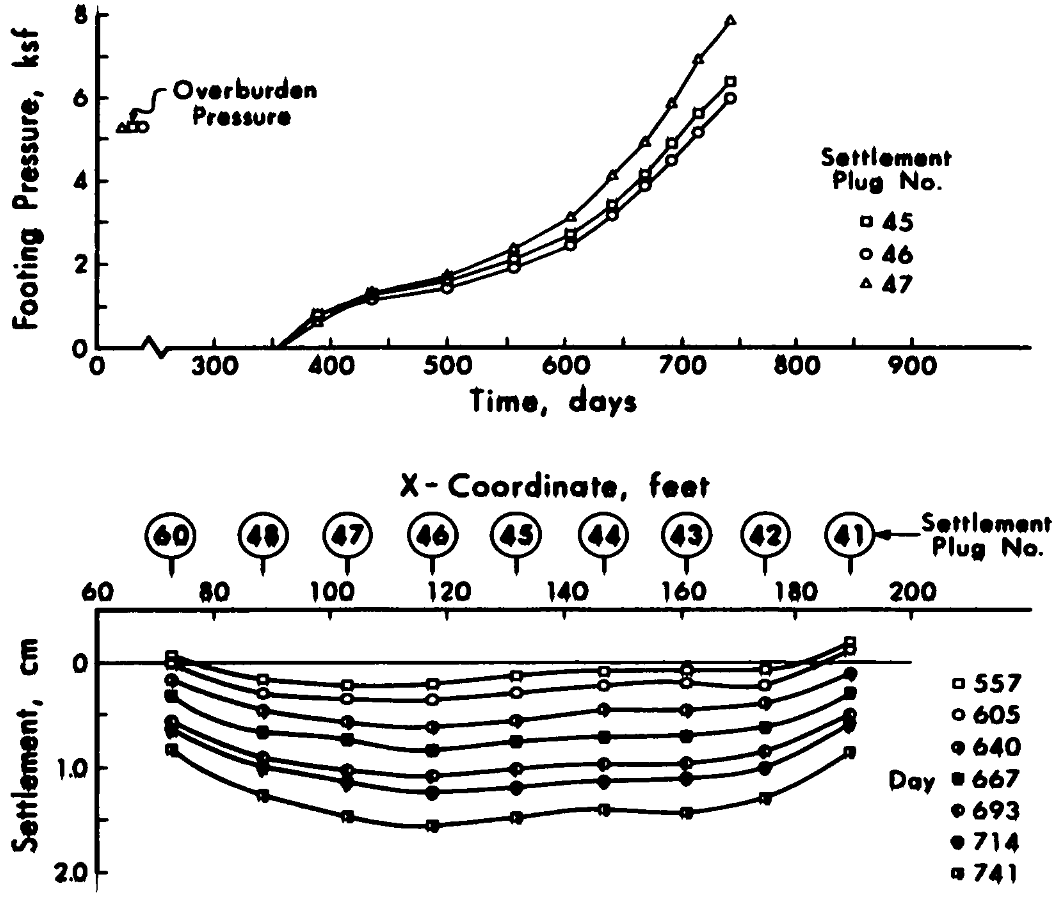


Fig. 6.13 Load History and Settlement Profiles of the South Exterior Column Line, AGT Tower

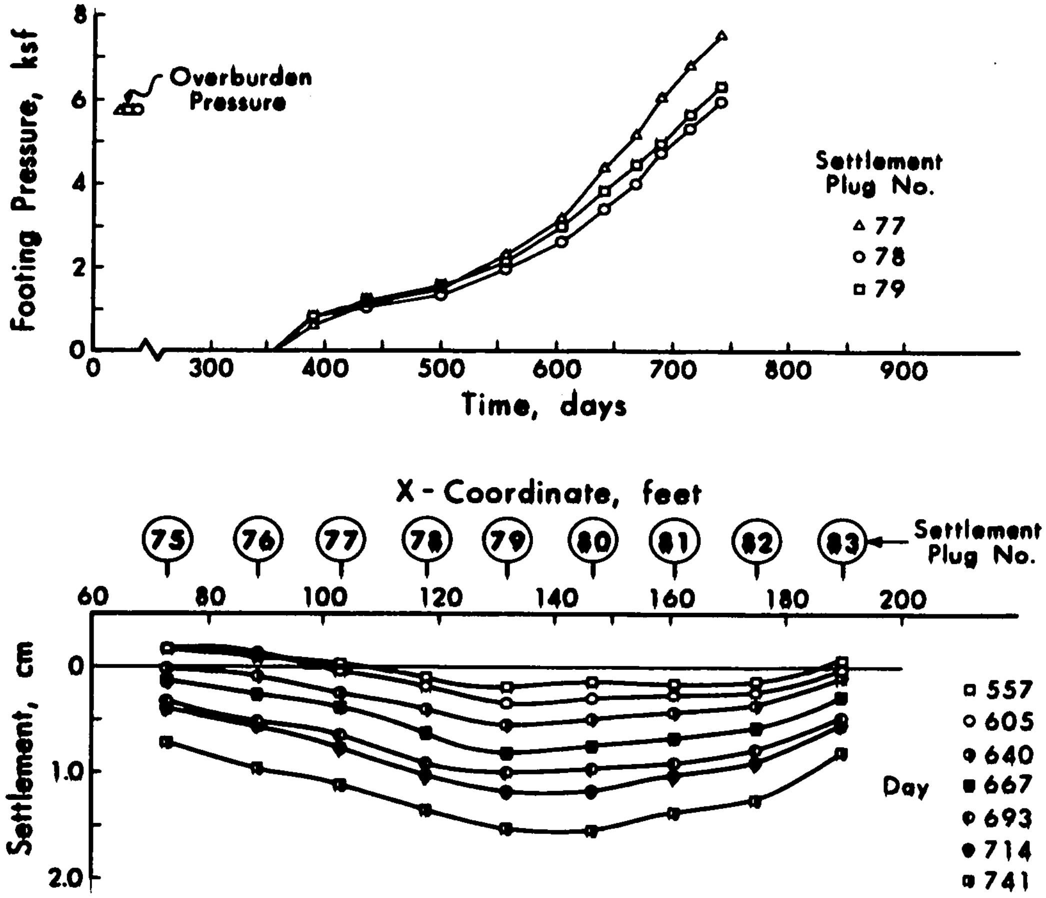


Fig. 6.14 Load History and Settlement Profiles of the North Exterior Column Line, AGT Tower

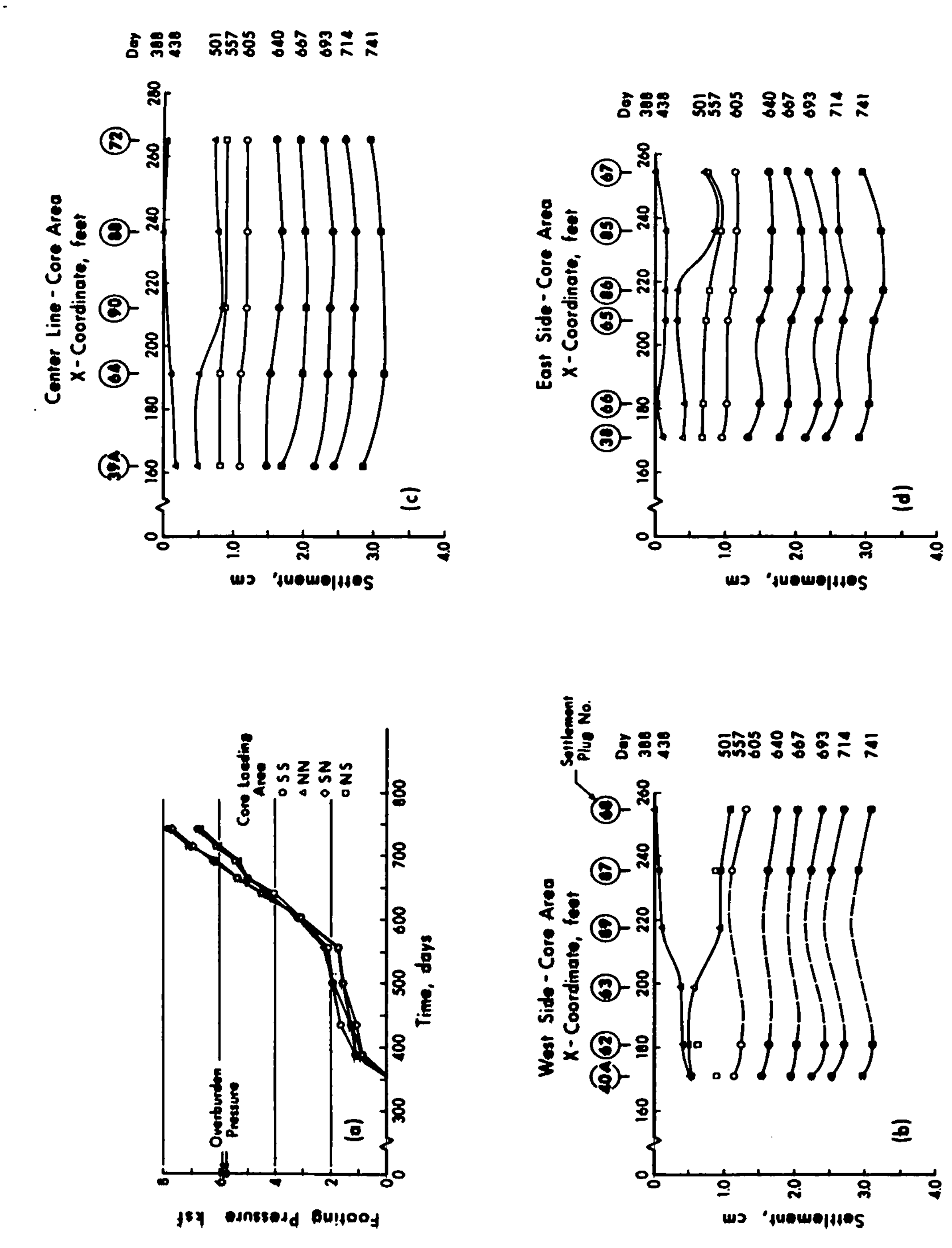


Fig. 6.15 Load History and Settlement Profiles of the AGT Tower Core Area

are presented in terms of settlement profiles along the four exterior column lines and three interior core lines. The time scale used in the accompanying typical loading histories is the same as that used for the Oxford Building.

The maximum settlement observed to September 22, 1970 is 3.2 cm. in the interior core area. The differential settlement between the core and the exterior columns is about 1.4 cm. (0.55 inches) and takes place over a span of 39 feet. As the AGT Tower is a flexible building this differential settlement is quite tolerable although it has caused some difficulty during construction in the determination of ceiling elevations.

As evident in Figs. 6.16 and 6.17 the exterior columns of the building are subjected to time dependent heave of the foundation. This is because of the sequence by which construction took place, the manner by which the initial settlement readings were taken and the geometry of the AGT Tower foundation plan. Upon completion of the excavation of the N.W. quadrant (Fig. 5.7) there was a 2 month delay before all the footings and first columns were poured. At the removal of column forms the settlement plugs were installed and an initial elevation was taken. This results in recording a variable amount of time dependent heave depending upon the date of the settlement plug installation during the 2 month period.

Because the initially lightly loaded exterior footings were surrounded by an expanse of open underground parking

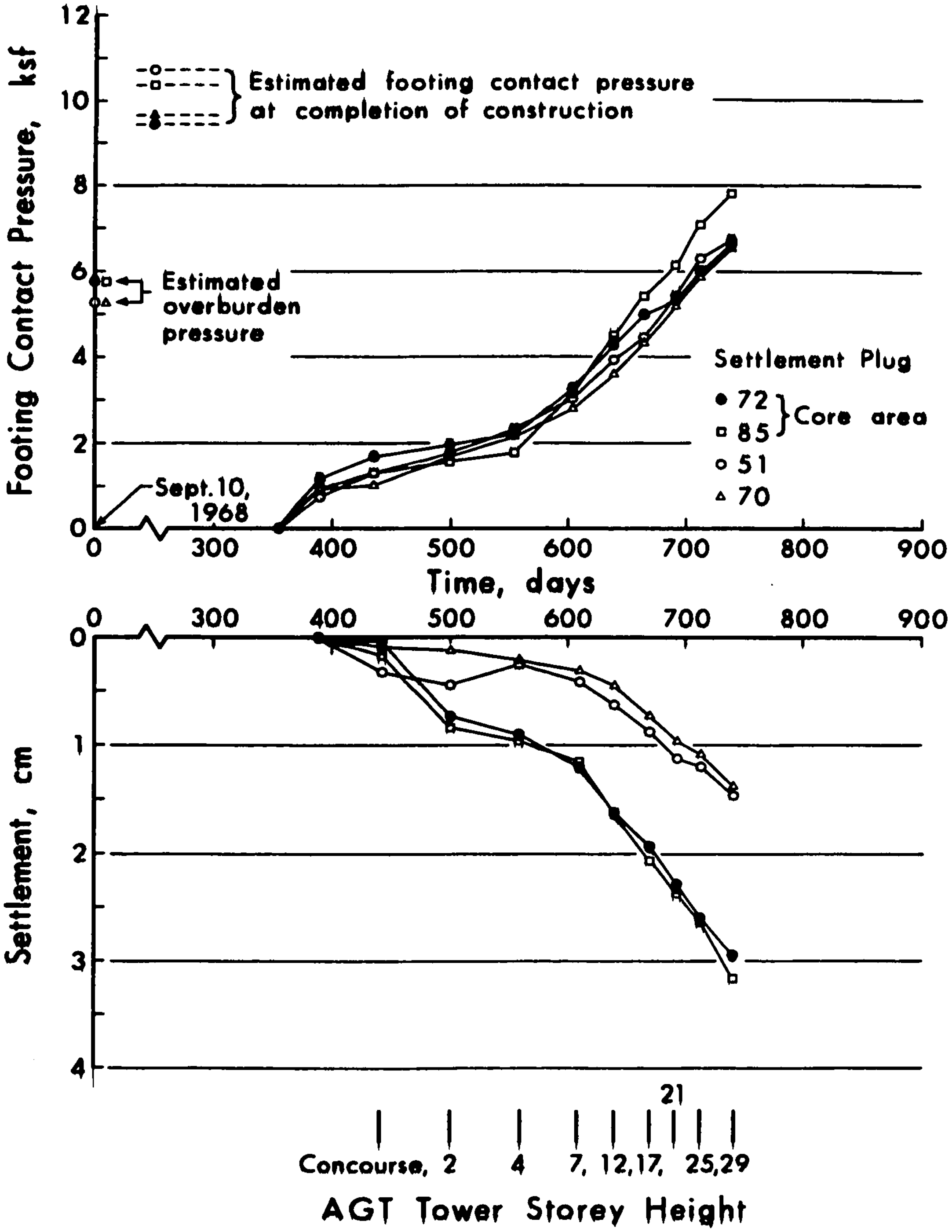


Fig. 6.16 Typical Load and Settlement History, AGT Tower

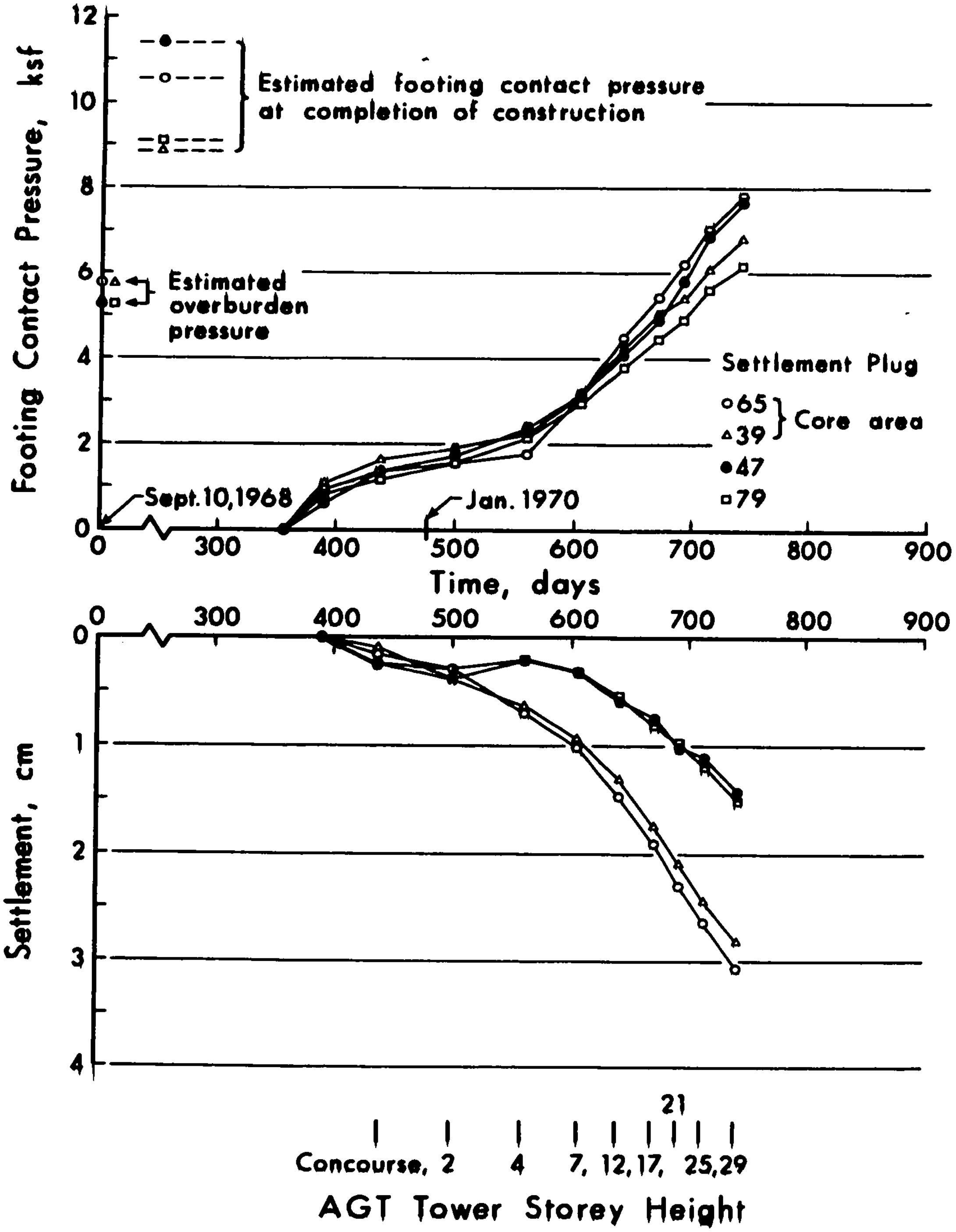


Fig. 6.17 Typical Load and Settlement History, AGT Tower

area, the footings were subjected to rebound as the foundation underwent time dependent heave at the completion of excavation of the N.W. quadrant. This is particularly true at the location of the larger corner footings as evident in the settlement profiles of Figs. 6.11 and 6.14. As indicated in Figs. 6.16 and 6.17 the time dependent foundation heave is suppressed at the much larger massive interior core footings. The diagrams indicate that at day 620, which corresponds approximately to the start of installation of the exterior precast concrete facing, the settlement rate increased until closely corresponding the response observed for the interior core footings. The excavation of the construction ramp near the west side of the AGT Tower probably is the cause of the greater heave indicated for footing 51 in Fig. 6.17.

The settlement profiles of Figs. 6.11, 6.12, 6.13, 6.14 and 6.15 show that the settlement pattern of the AGT Tower foundation is typically "bowl shaped" with the maximum displacement occurring at the center. This is shown explicitly in Fig. 6.18 where the settlement profiles along the two building center lines are drawn using the September 22, 1970 settlement record. The effect of time dependent heave is indicated by the 0.2 cm. uplift of footings 8 and 5 located in the lightly loaded underground parking garage. This is not evident at footings 26 and 25 because the heave was suppressed by the addition of the plaza floor slab after the initial settlement reading. The center line profiles support

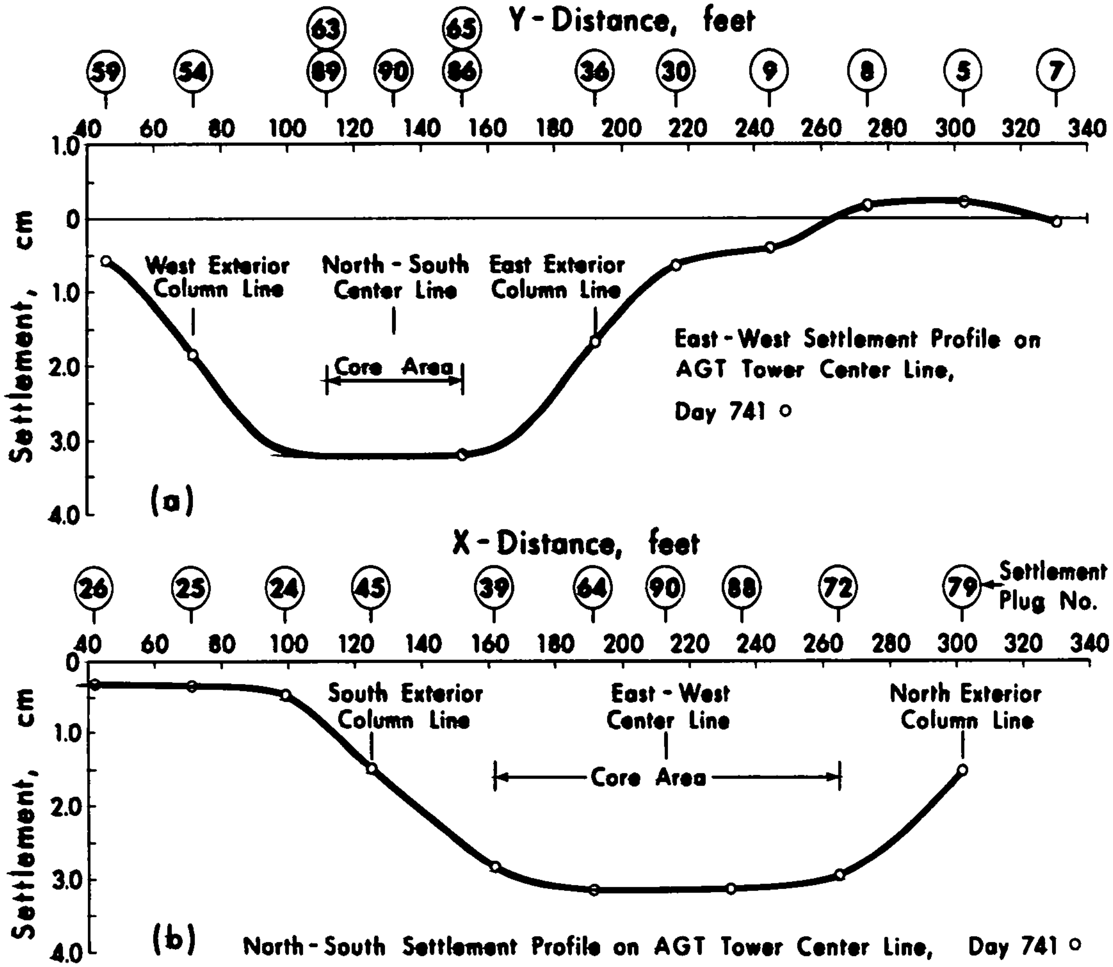


Fig. 6.18 Settlement Profiles on the AGT Tower Center Lines and Through the Underground Parking Garage

the observation made earlier in Chapter V that the differential settlement between a heavily and lightly loaded area, both in the same excavation, will be accentuated by the effect of time dependent heave of the latter.

The overlap of rebound observations on rebound points and settlements on settlement plugs is shown in Fig. 6.19. In the case of rebound point no. 4 the overlap is not perfect as this point is located between the SS and SN core footings (Fig. 6.2). Since this rebound point will not settle quite as much as the adjacent footings, the difference in elevation as given by the two surveying procedures is about 0.5 cm. Rebound points 3 and 5 are located directly under footings 34 and 45 respectively in a small cavity. The differences in elevation by the two procedures are 0.3 cm. and 0.2 cm. respectively. These errors can likely be attributed to a slight variation in elevations between the September 13, 1969 heave readings and the September 30, 1969 initial settlement readings. Some soil intrusion into the small cavity at the base of the footing as the footing is loaded would give rise to additional deviation. Although the overlap at these three locations is not perfect the relatively small differences in elevations at day 710 and the parallelism of the settlement curves after day 650 suggests that complete continuity of foundation displacements has been achieved.

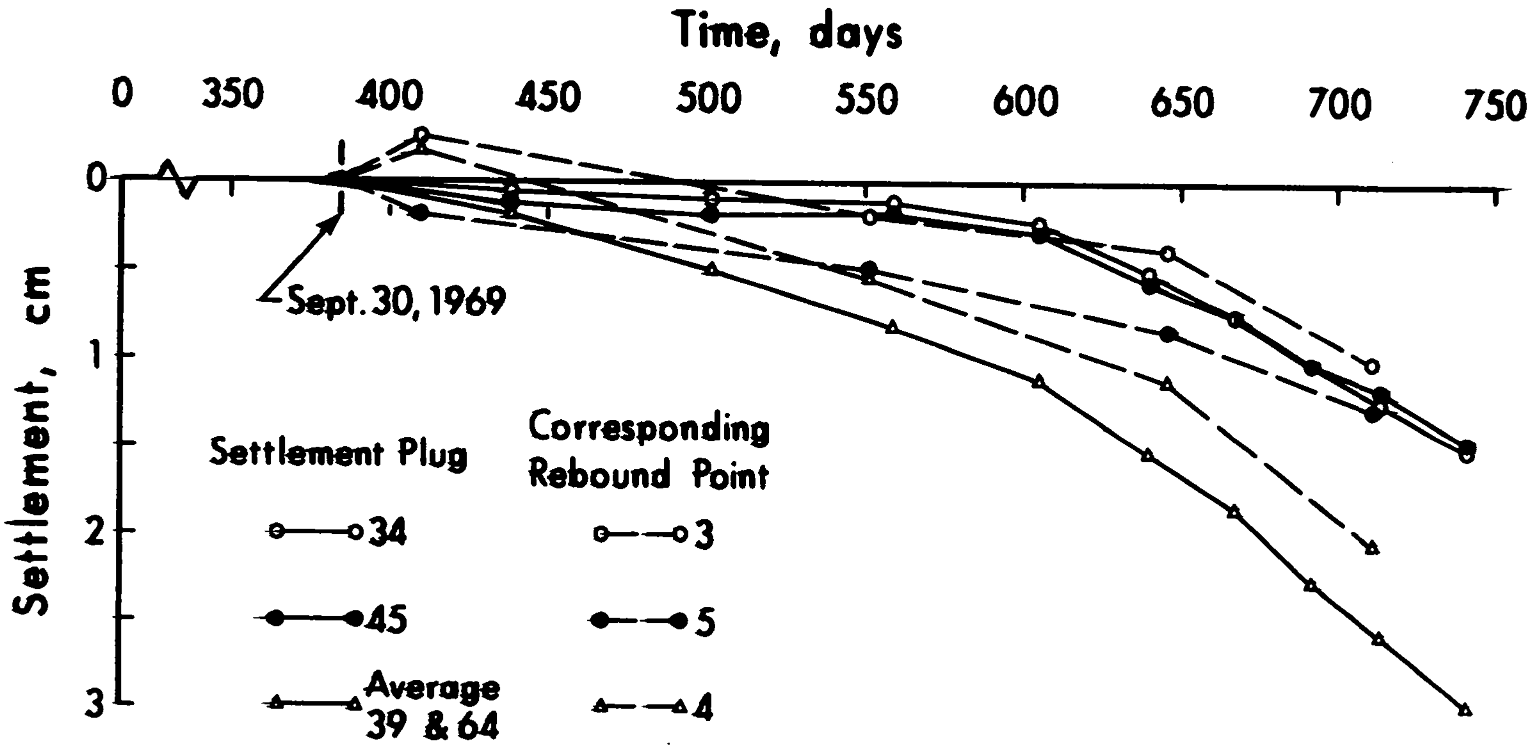


Fig. 6.19 Overlap of Rebound and Settlement Measurements, AGT Tower

6.6 Piezometer Observations

The piezometer installed directly between the SS and SN core footings indicated that no noticeable excess pore pressure was generated in the bedrock formation at the elevation of the piezometer tip throughout construction of the AGT Tower. The transducer type of piezometer, which is capable of measuring small changes in porewater pressure, recorded only minor fluctuations in water pressure as shown in Fig. 6.20. These fluctuations did not exceed ± 2 feet of head from the average for the construction period.

From Fig. 6.20 it is apparent that there is little correlation between precipitation and fluctuations in water pressure. This is perhaps not surprising as after day 425 the entire AGT Tower site was covered by the construction of the first floor slab. Much of the surface runoff from the vicinity thereafter was likely intercepted by the weeping tile installed at the basement elevation. There is some evidence that the 2.66 inch rainfall at about day 355 on the unprotected site did register at the piezometer tip as approximately 2 feet of excess head 10 days after the period of rainfall.

The absence of measurement of excess pore pressure can be attributed to the following causes.

(a) As noted earlier in Chapter V, there is evidence that the bedrock, at least in the upper portion, is extensively shattered. The presence of fractures in this region would greatly enhance the ability of the bedrock

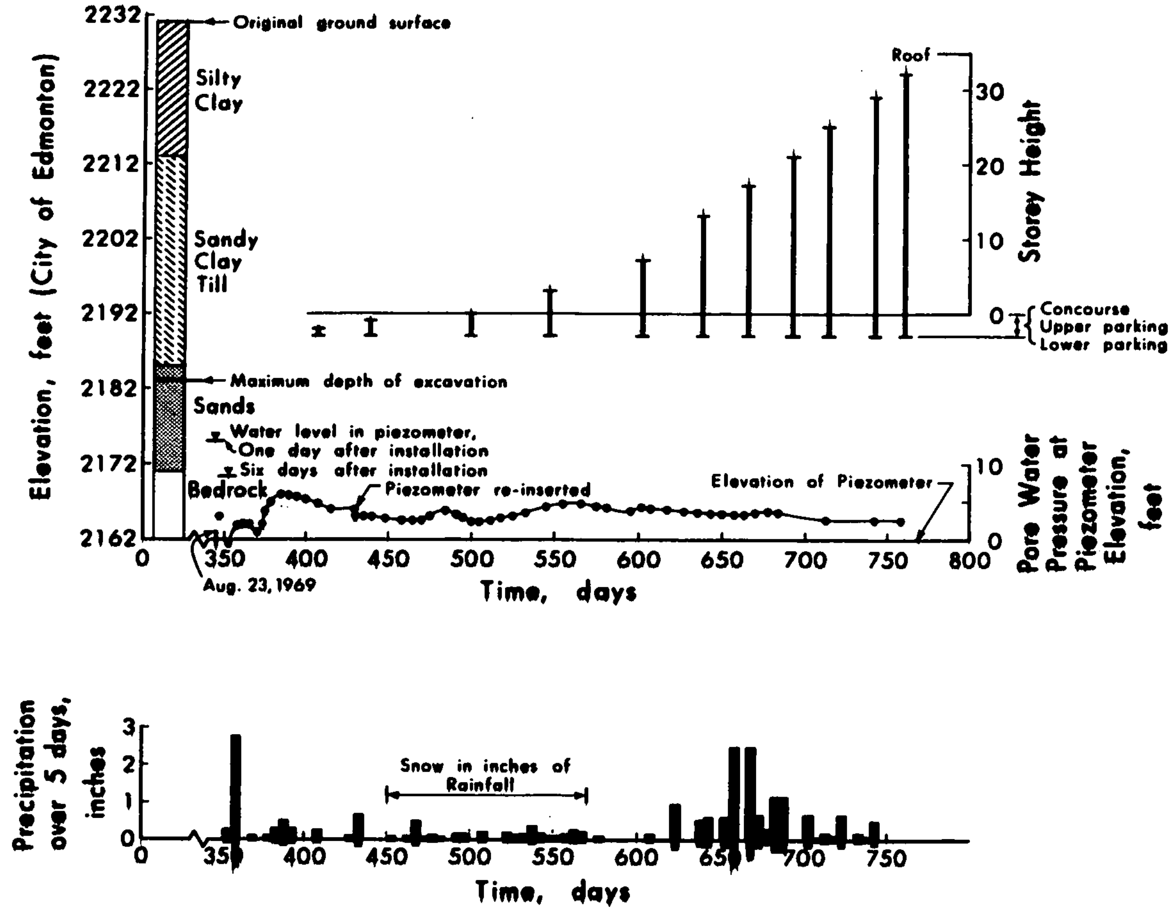


Fig. 6.20 Piezometric Pressure and Precipitation History, AGT Tower

mass to dissipate excess pore pressure by virtue of the increased permeability.

(b) The proximity of the Saskatchewan Sands and Gravels and an underlying thin coal seam provide an upper and lower drainage surface within nine feet of the piezometer tip. The drainage path is therefore small and excess pore pressure can dissipate rapidly. From the section drawn in Fig. 6.3 it is obvious that to avoid the proximity of coal seams the piezometer tip would have to be at least 100 feet below the building foundation elevation. Deeper boreholes could perhaps indicate that even this depth is insufficient.

(c) The rate of load application during construction is insufficient to generate measurable excess pore pressure.

(d) The possibility exists that the upper portion of the bedrock is not fully saturated because as was noted earlier in Chapter V, the free water table is likely well below the bedrock surface.

No case records of the measurement of excess pore pressures in the Edmonton bedrock formation due to surface loading have been found.

6.7 Mechanism of Settlement

In Chapter V it was shown that the heave phenomena due to excavation at the AGT-Oxford Complex site can be attributed almost in its entirety to a relatively rapid elastic response of the foundation. The same conclusion can be drawn from the settlement response of the underlying sand and bedrock strata to loading during construction of the AGT Tower. This is shown in Figs. 6.21a and b where the contact pressure of several footings are plotted versus the settlement. It is evident fron the linearity of the plotted data in Fig. 6.21a that the settlement of the massive interior core footings is directly related to the footing contact pressure. The somewhat flatter slope for the exterior footings in Fig. 6.21b reflects the influence of footing geometry and relative location within the entire foundation plan. A factor here also is the time dependent heave noted earlier which takes place initially under light footing loads. If both settlement and contact pressure were normalized the data would obviously plot on or near to the 1:1 ratio line indicative of the linearity of the response.

6.8 Analysis of Settlement Data, Oxford Building

The solution given by Steinbrenner (1934) for the elastic settlement of the corner of a rectangular uniformly-loaded area located on the surface of a semi-infinite solid was used to determine the foundation moduli at the AGT-Oxford Complex site. The method of analysis as well as the applicable equations, equations 2.1 and 2.2, were previously given in Chapter II.

In the case of the Oxford Building the date at which the average footing pressure first exceeded the original overburden pressure was August 15, 1968 which corresponds

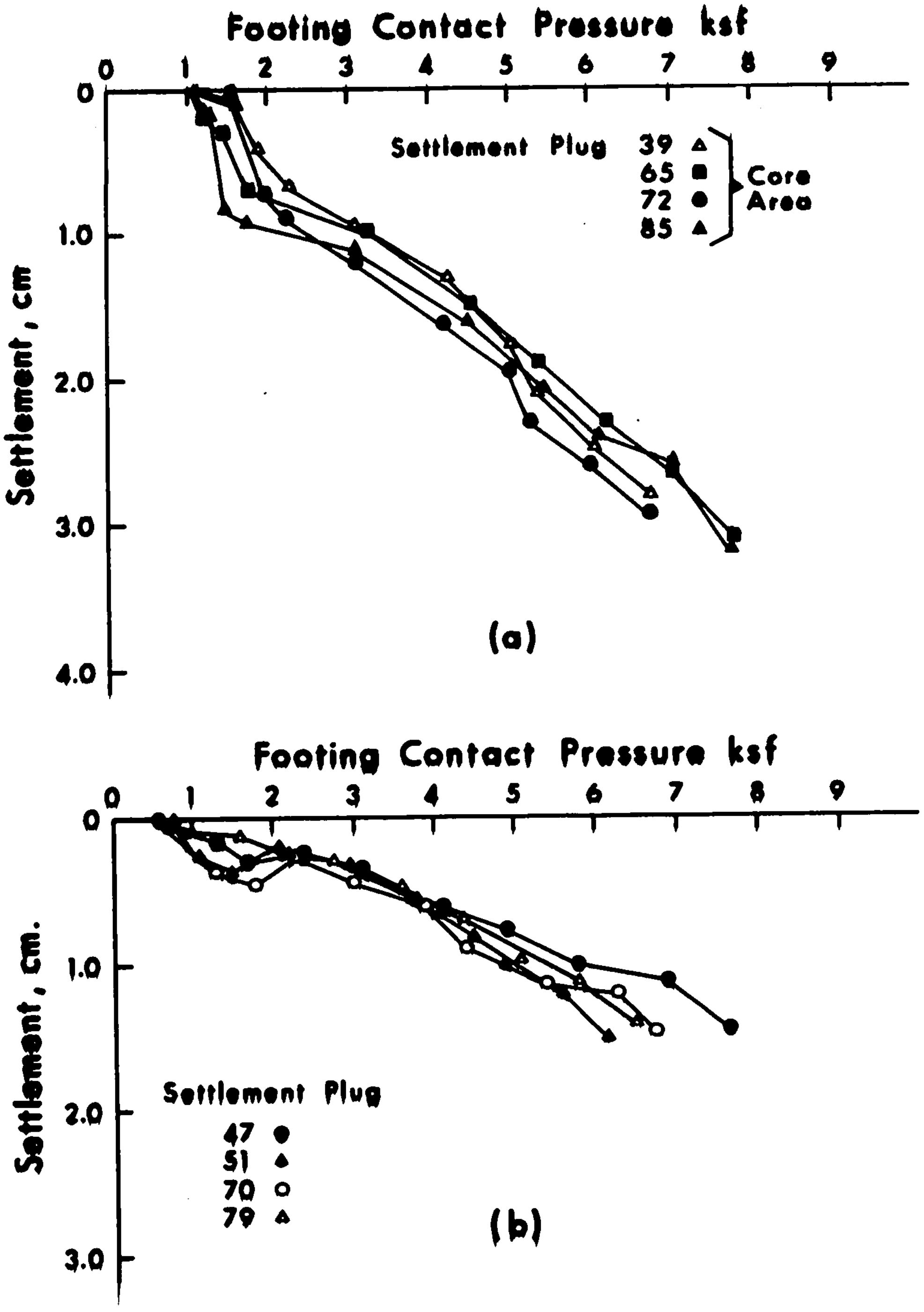


Fig. 6.21 Footing Contact Pressure and Settlement, AGT Tower

to day -26 on the numerical time scale adopted for the entire AGT-Oxford Complex project. The overburden pressure of 4.3 ksf was computed at the contact elevation of the interior core footing. With the footing settlements and loads at day -26 as a reference, the settlement and load increments prior to and after this day were used to compute the foundation moduli.

The settlements observed at the Oxford Building were difficult to analyze because of two factors.

(1) The elevations of the footings were not constant but varied between 2186.5 feet at the south end of the building to 2205.0 feet at the north end. This meant that for some footings the reference average footing pressure at day -26 was above and for others below the actual original overburden pressures. The variation from the average can be as great as 1.2 ksf.

Another aspect of concern here is that the southern footings are in till but only a few feet from the upper surface of the Saskatchewan Sands and Gravel deposit whereas the distance to the sand deposit is considerably greater for several of the footings located at the northern end of the Oxford Building.

(2) The settlements at the Oxford Building after September 10, 1968 were affected by the 40 foot deep AGT excavation immediately adjacent to the south. Correction of these readings for heave is difficult as the Oxford Building is outside the excavation perimeter and is

located at the corner of the excavation area where boundary restraint is maximum.

The first factor represents a deviation from the assumption of loading on a semi-infinite plane. For the purpose of this analysis the variation in footing elevation was not taken into account. The second factor, the fluctuation of settlement due to an external influence, can be taken into account with application of appropriate heave corrections to the settlement data.

The corrections applied to the settlement data taken after September 10, 1968 were computed using the foundation moduli derived from the rebound data and the Steinbrenner computer solution described in Chapter V. The heaves were calculated at specific locations at the Oxford Building and varied in accordance with the changing size of the excavation as construction progressed. Adjustments were made to the calculated heaves by means of the procedures outlined in Chapter V for points located outside the excavation area.

The application of corrections to the footing displacements observed at the Oxford Building after September 10, 1968 resulted in four types of settlement data.

Type I - Corrected settlements using adjusted computed heave values.

Type II - Corrected settlements using adjusted computed heave values; the adjustment different from that of settlement Type I.

Type III - Corrected settlements using the heaves computed from the Steinbrenner solution with no adjustment.

Type IV - Uncorrected settlements.

By means of these four settlement data types the affect of heave at the Oxford Building was bounded the minimum and maximum amount by the application respectively of no correction and an unadjusted correction. Settlement data Types I and II represent two intermediate cases. Fig. 6.22 shows the typical heave corrections and the resultant north-south settlement profiles through the Oxford Building for the settlements observed on day 46, October 25, 1968. Since settlements were recorded at 28 locations, five diagrams similar to Fig. 6.22 were required to correct a single settlement data set.

The uncorrected settlement profile, Type IV, in Fig. 6.22b clearly shows the effect of heave at footings 2 and 9 located at the south end of the symmetrical Oxford Building. Settlement profile Type III infers that 0.17 inches of heave occurred at footing 27, 170 feet from the AGT excavation. This is excessive in light of the small rebounds observed at the Alberta College buildings, described earlier in Chapter V.

Settlement profile Type II represents a linear variation between profiles Type IV and Type III by assuming the heave at footing 27 as zero and the heave at footing 2 as equal to the unadjusted value computed by the Steinbrenner computer solution. The Type II settlement profile seems to

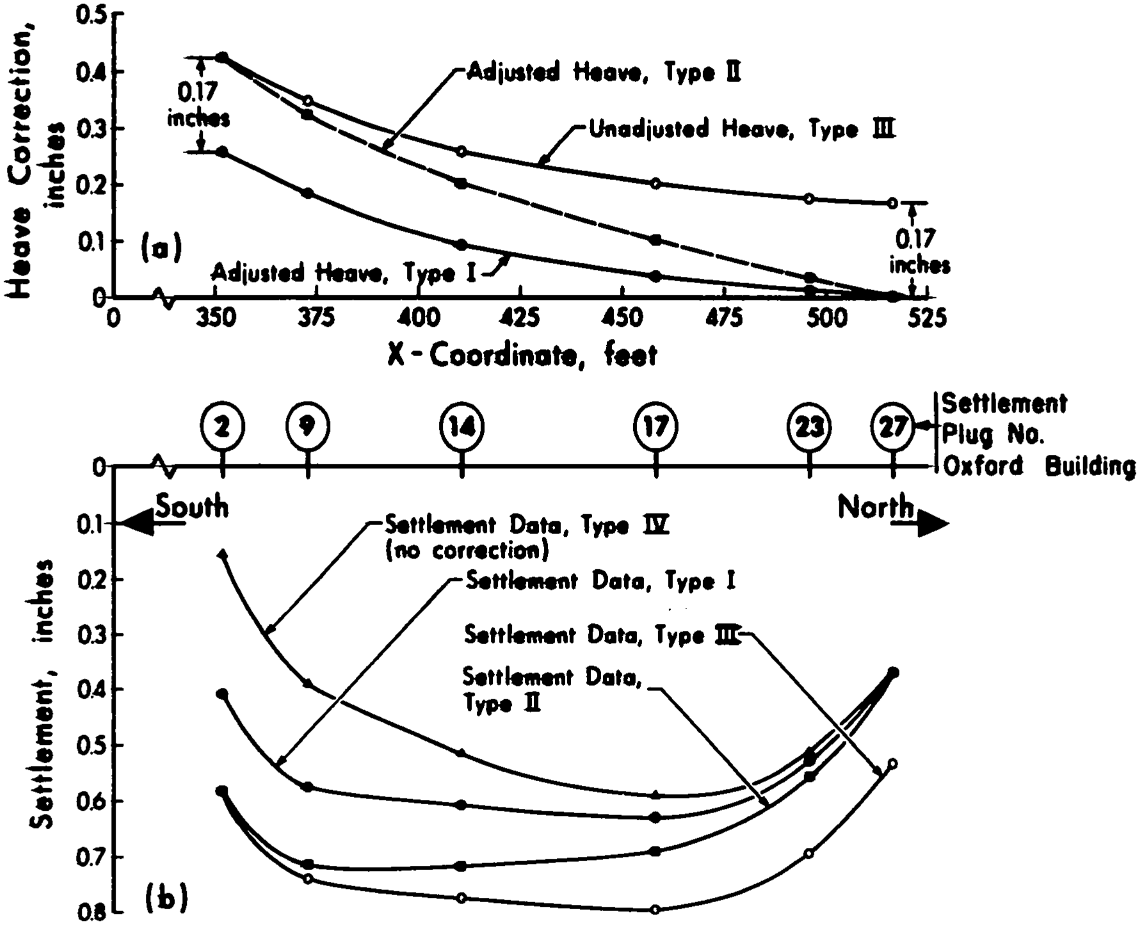


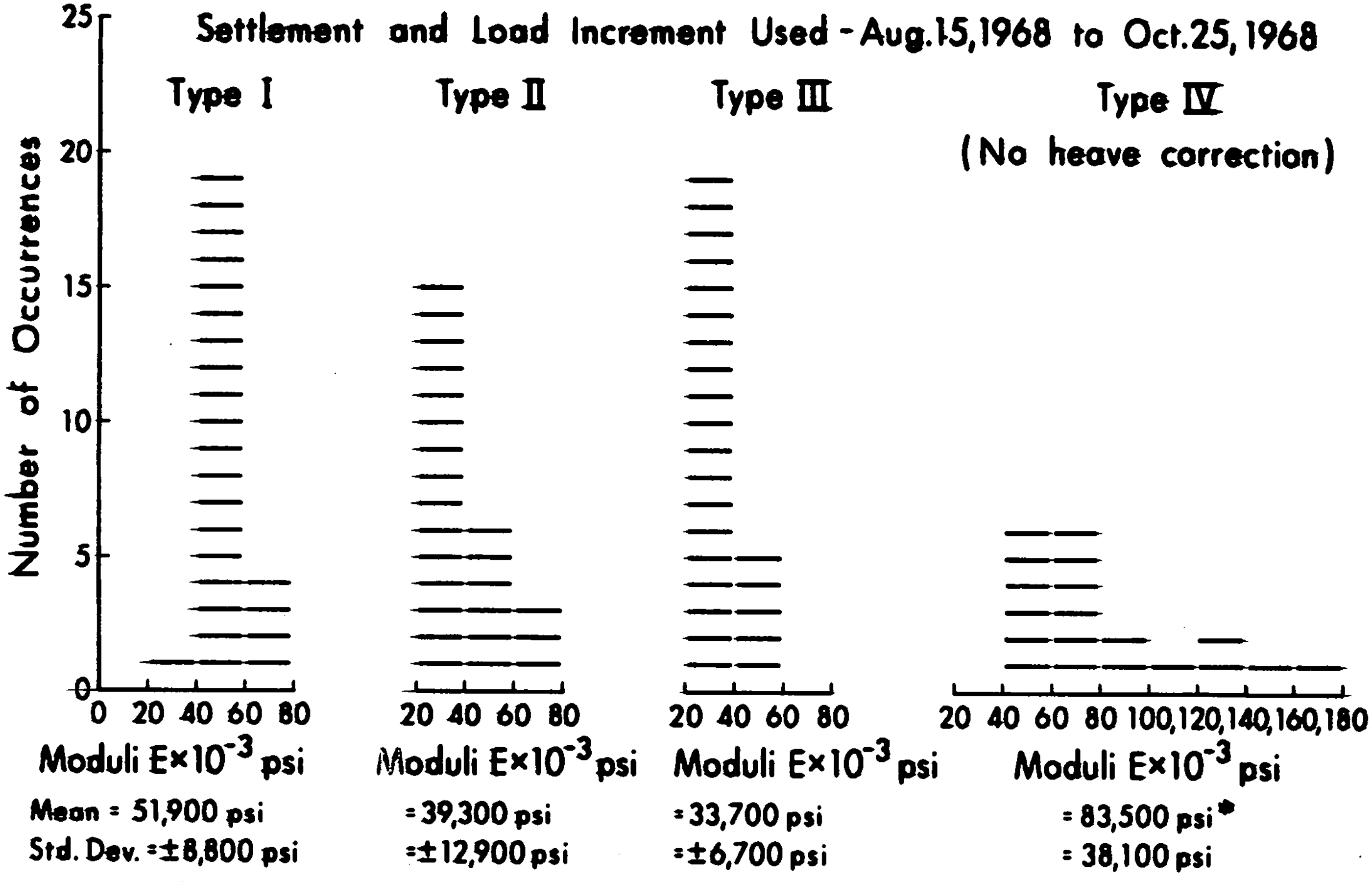
Fig. 6.22 Typical Heave Corrections and Resultant Settlement Profiles, Oxford Building

indicate that the applied heave correction is perhaps too severe as the resulting settlement at the south end of the building is considerably greater than at the north end.

The Type I settlement profile appears to be more reasonable. No heave is assumed to take place at footing 27 and the balance of the profile appears to be approximately symmetrical about the east-west center line of the Oxford Building. The slightly greater settlement at footing 27 as compared with that measured at footing 2 can probably be attributed to the resistance to bending offered by the strip footing and basement wall located at the south end of the building.

The difficulty in analysis of the settlement data is apparent in Fig. 6.22b where some of the heave corrections exceed the measured footing settlements. The fact that heave corrections must be applied to the data is apparent in Fig. 6.23 where the histograms of the computed foundation moduli are given as derived from the four settlement data types for the October 25, 1968 settlement set. The histogram of Fig. 6.23d indicates that there is little relationship between the uncorrected measured settlements, Type IV, and footing loads.

The foundation moduli computed from settlements observed at the Oxford Building at 28 locations between April 2, 1968 to November 4, 1969 are shown in Figs. 6.24 a, b, c and d. The reference datum used in the computations is the August 15, 1968 settlement and load set. The plotted standard

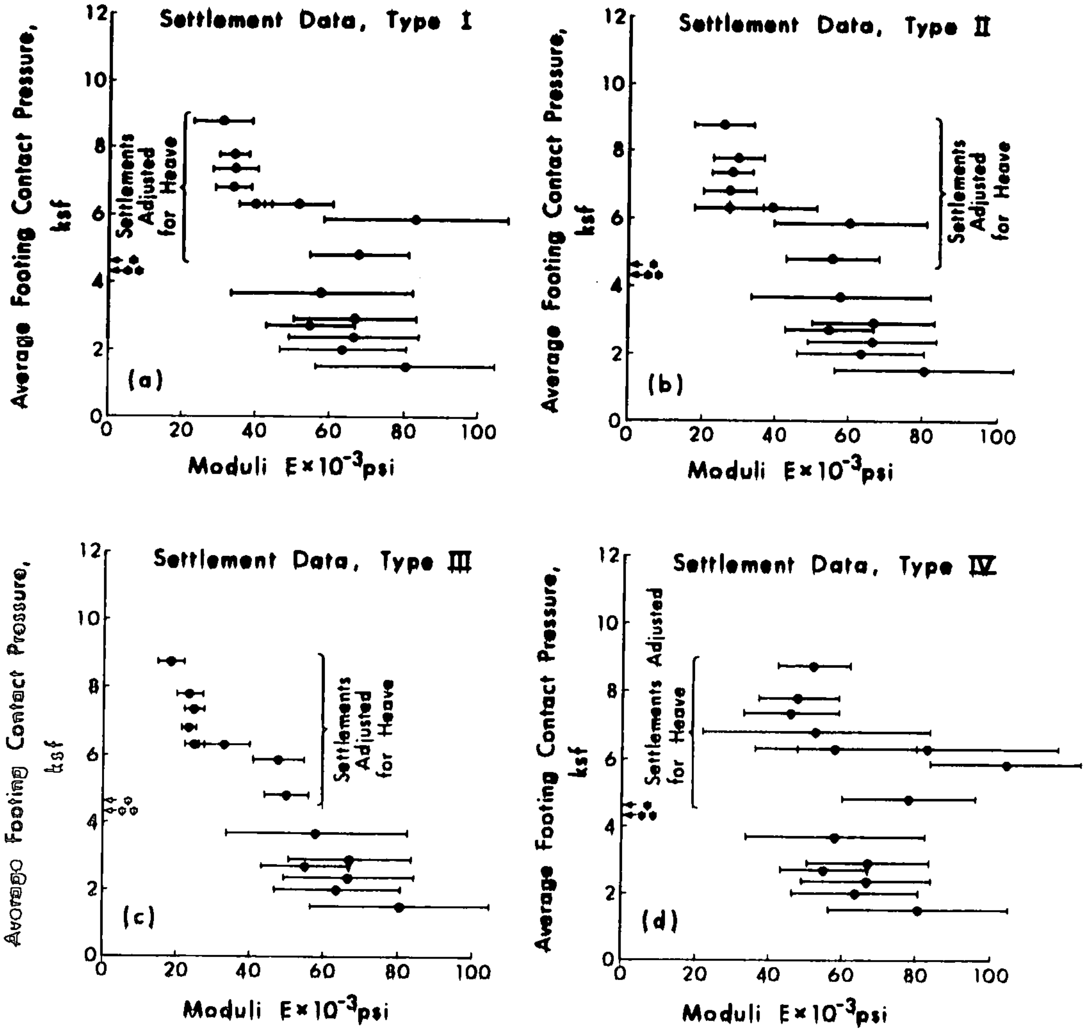


(a) (b) (c) (d)

Note :

* Actual mean is greater as zero settlements correspond to an infinite modulus of deformation

Fig. 6.23 Typical Histograms of Derived Moduli, Oxford Building



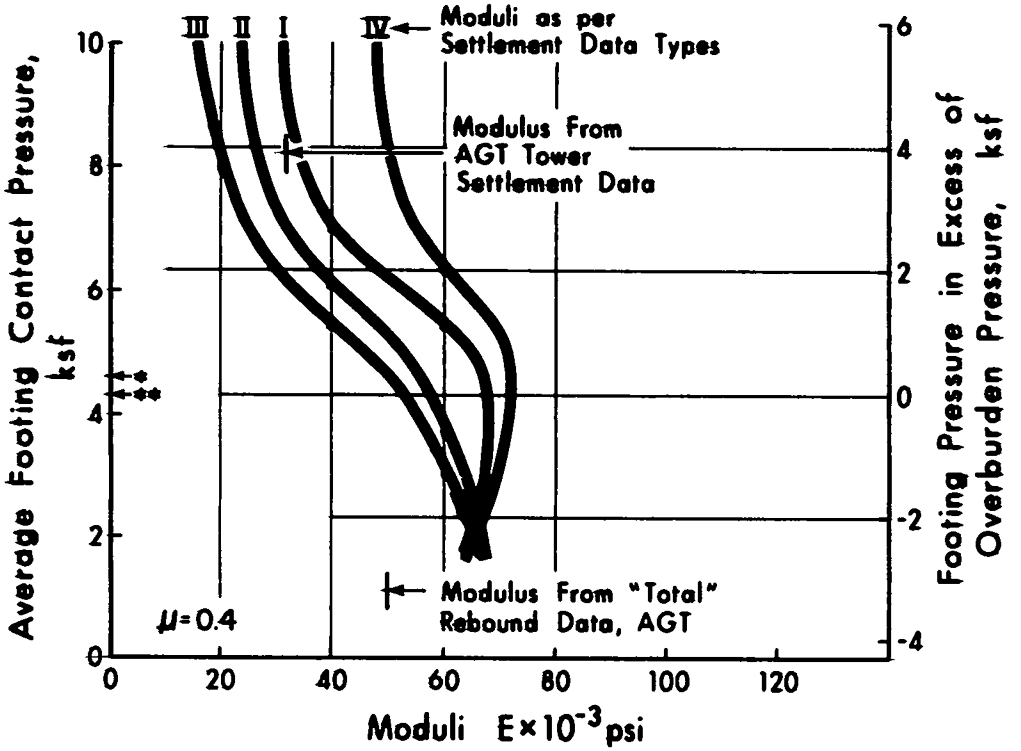
Legend: •Mean, ⊣ Limit of Standard Deviation, μ=0.4
Note: • Pressure Datum Used in Calculations
••Average Overburden Pressure of Excavated Soil (approximate)

Fig. 6.24 Variation of Derived Moduli With Footing Contact Pressure For Four Settlement Data Types, Oxford Building deviations indicate the variation in the derived moduli. Two derived moduli, computed from settlements observed at the start and finish of the 29 day strike, are given at the average footing contact pressure of 6.3 ksf and represent the variation due to time dependent settlement.

A composite diagram of the variation of moduli with footing contact pressure for the 4 settlement data types is given in Fig. 6.25. Since the Oxford Building is adjacent to the AGT Tower, an indication of the computed foundation moduli obtained from settlement observations in this latter building are given for comparison. From the data shown and the previous discussion of settlement profiles it would appear that the curves for settlement data Types I and II are most representative of the variation of foundation modulus with footing contact pressure. The curve labelled I implies that recompression of the foundation to the original overburden pressure takes place at a relatively constant foundation modulus of about 65,000 psi with subsequent loading resulting in settlement which takes place with a progressively decreasing modulus of deformation.

6.9 Analysis of Settlement Data, AGT Tower

The AGT Tower footings were all located at approximately the same elevation with deviation from the average less than ± 2 1/2 feet. The average footing pressure of 5.8 ksf approximately represents the original average overburden pressure over the site on August 26, 1970; day 714. The settlements

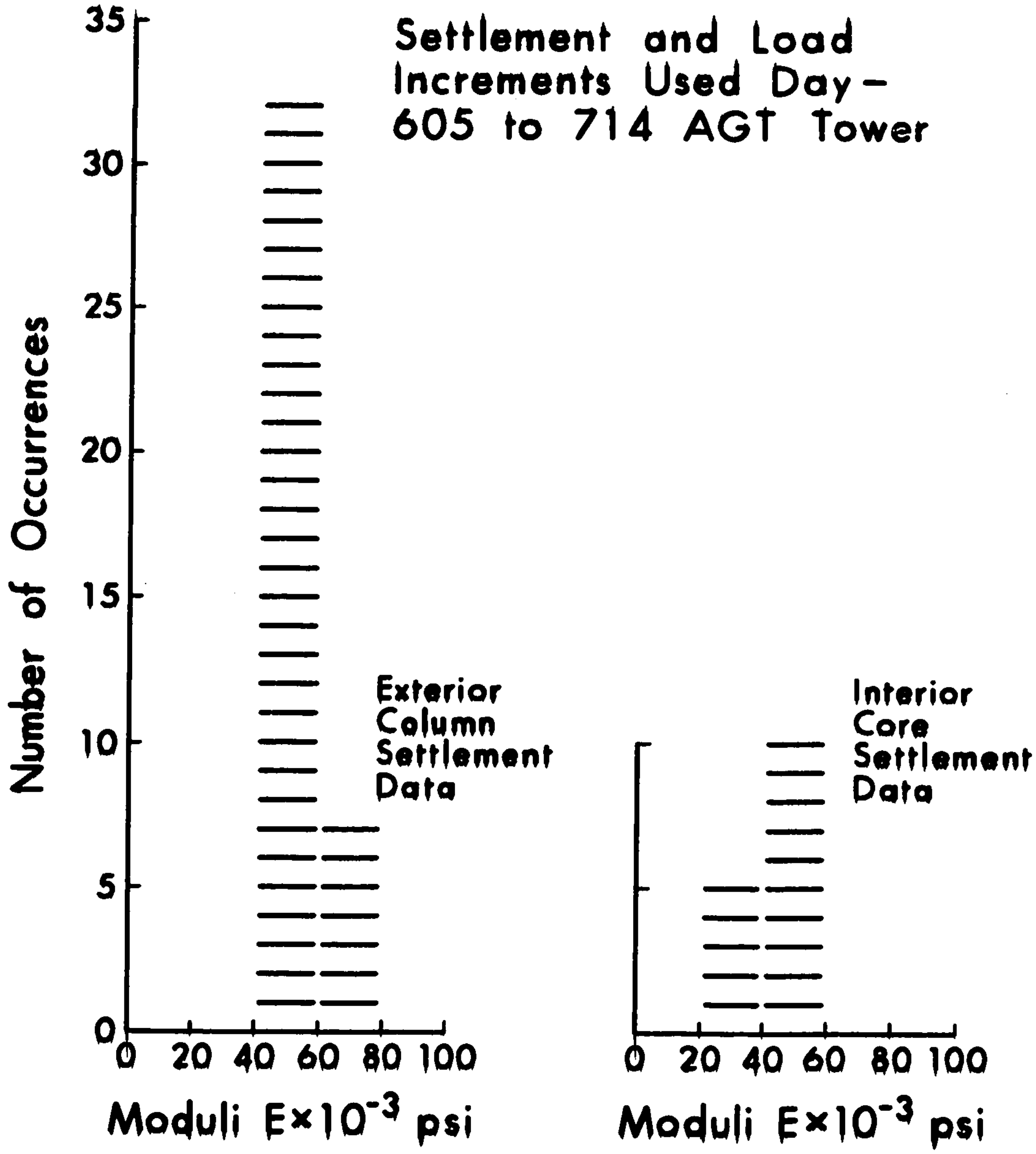


Note: * Pressure Datum Used in Calculations
** Average Overburden Pressure of Excavated Soil (approximate)

Fig. 6.25 Variation of Derived Moduli With Footing Contact Pressure, Oxford Building and loads at this day were used as the reference datum for computing the foundation moduli. The settlements observed at the exterior columns were not corrected for time dependent heave as this effect became negligible at higher footing loads as indicated earlier in Fig. 6.21. The procedure for the derivation of foundation moduli is therefore identical to that outlined in Chapter II for the CN Tower.

Typical histograms of foundation moduli obtained from settlements observed on the AGT Tower core and exterior column footings are given in Fig. 6.26. The precision used in the measurement of settlements and tabulation of load data throughout the entire construction period is evident in the small standard deviation of the derived moduli.

The moduli obtained from the analysis of the entire settlement and load record up to day 741, September 22, 1970 are given in Fig. 6.27. The data indicate that the settlement response of the massive interior core, founded on the 15 foot sand deposit and the underlying over-consolidated bedrock formation, is in accordance with a virtually constant modulus of deformation of about 42,000 psi. The modulus of 50,000 psi obtained from "total" rebound observations at the AGT excavation is given in Fig. 6.27 for comparison. As evident, the agreement is good. This suggests that the monitoring of heave displacements during excavation is an appropriate method for determining the deformation modulus of an over-consolidated foundation soil for which previous construction experience is lacking.



Mean = 55,300 psi
Std. Dev. = ±6,300 psi
(a)

= 42,000 psi
= ± 1,900 psi
(b)

Fig. 6.26 Typical Histograms of Derived Moduli, AGT Tower

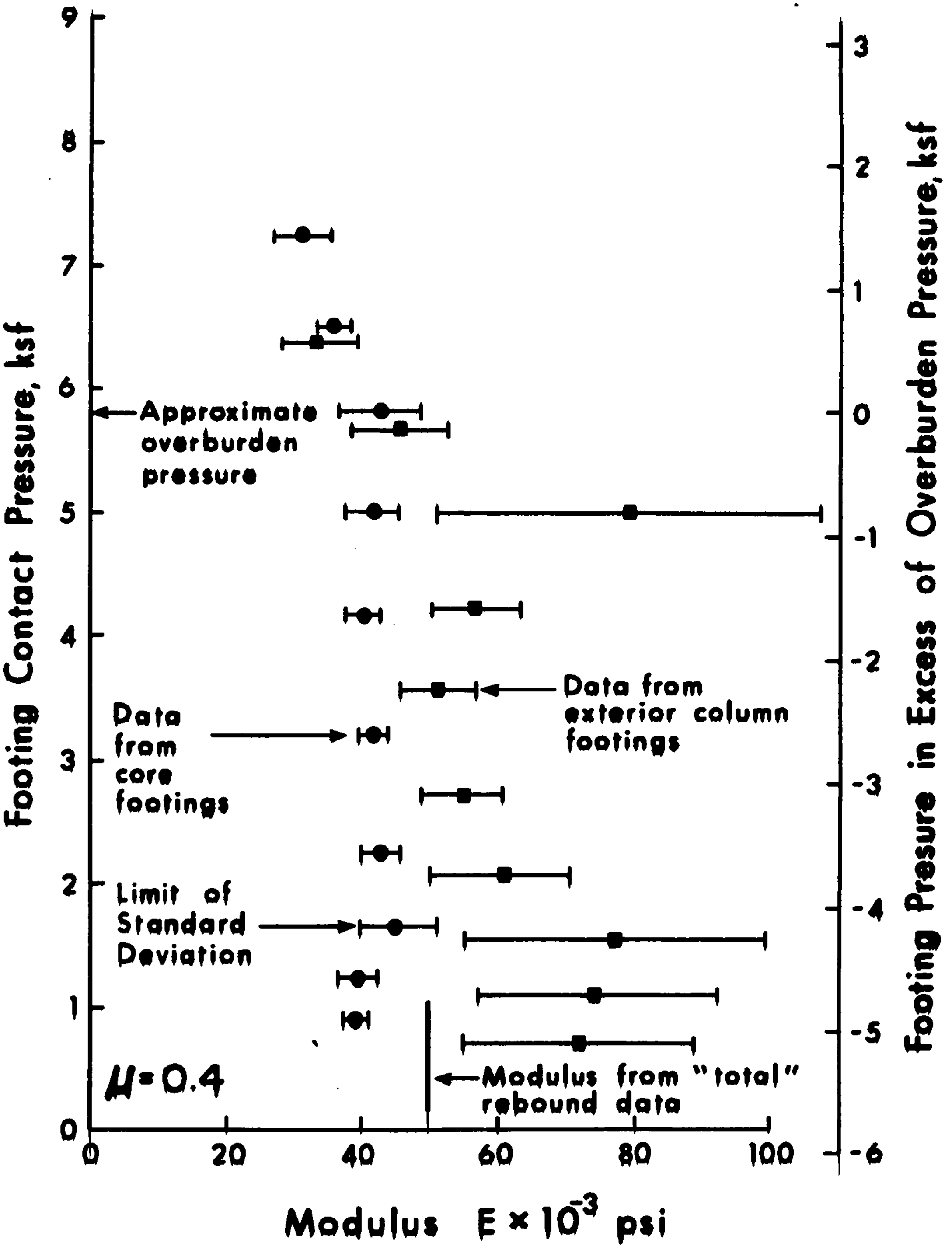


Fig. 6.27 Variation of Derived Moduli With Footing Contact Pressure, AGT Tower

At footing pressures greater than the previous equilibrium overburden pressure the foundation modulus begins to decrease and at a footing contact pressure of 7.3 ksf equals 34,000 psi. The effect of time dependent heave on the settlement of the smaller exterior column footings at light loads is made evident by the higher derived moduli. As footing contact pressures increase the computed modulus of foundation deformation decreases and at the original overburden pressure approximately equals the value given by the core settlement data of 42,000 psi.

6.10 Comparison Laboratory and Empirical Moduli

The modulus of deformation of the foundation at the AGT-Oxford Complex site is about 34,000 psi for footing contact pressures in excess of the original overburden pressure. Although applicable primarily to the Saskatchewan Sand deposit and the bedrock formation, the ratio of field to laboratory moduli is given for 4 foundation soils in Table 6.1 because the Oxford Building is partially founded on the overlying till stratum. The laboratory moduli used in the comparison are the largest derived for the soil and test types quoted.

The variation between laboratory and field moduli is large. As has been noted in Chapters II and V the use of laboratory moduli would lead to a large over estimate of field displacements. The tests run on Saskatchewan Sands are the only exception as the laboratory moduli exceeded the

TABLE 6.1

COMPARISON FIELD AND LABORATORY MODULI

Empirically derived field modulus is 34,000 psi

Soil Type	Laboratory Modulus psi	Laboratory Test	Ratio Field to Laboratory
Till	2,022	Oedometer	17.0
	9,100	Unconfined Compression	3.7
Bedrock (Mudstone)	3,500	Unconfined Compression	9.7
	3,410	Oedometer	10.0
(Sandstone)	8,000	Unconfined Compression	4.3
	2,090	Oedometer	16.3
	14,800	Consolidated Undrained Triaxial	2.3
	5,000	Consolidated Drained	6.8
Saskatchewan Sands	31,000	Consolidated Drained	1.1
	36,700	Consolidated Undrained* Phase II, Cycle 1	.9
	47,400	Phase II, Cycle 2	.7
	100,000	Phase III, Cycle 1	.3
	21,000	Phase IV, Cycle 1	1.6

* See Fig. 5.5 for reference.

field values in several instances. The discussion relating to this observation was given earlier in Section 5.12 and is applicable here as well.

6.11 Procedure for Estimating Settlements from Empirical Data

For buildings founded on the Saskatchewan Sands deposit and the underlying bedrock formation the empirical foundation moduli obtained from the AGT Tower can be used directly for computation of the expected settlements. The procedure is virtually identical to that outlined previously for till in Chapter II, Section 2.11, with the exception that the foundation moduli used are those given in Fig. 6.27.

For structures located in excavations of the same size as the gross area of the building the foundation modulus of 42,000 psi can be used to compute the recompression settlement to the original overburden pressure for all footings within the excavation.

If the structure is located within a significantly larger excavation some judgment must be used. For example, the recompression settlement of rapidly loaded massive core footings can be computed using a deformation modulus of about 42,000 psi. However, relatively lightly loaded and smaller isolated footings may be subjected to time dependent heave and will settle with an "apparent" recompression modulus which varies as shown in Fig. 6.27. The choice of foundation modulus from the diagram of Fig. 6.27 for exterior footings should be made in conjunction with the choice of a heave modulus required in calculating excavation rebound by the procedure outlined in Chapter V. In this manner cognizance is taken of the sequence and rate at which construction progresses from excavation to erection of the structure.

For footing contact pressures greater than the original overburden pressure settlements will take place with a decreasing deformation modulus as shown in Fig. 6.27. At 1.5 ksf in excess of overburden pressure the approximate modulus of deformation is 34,000 psi. Using the footing loads corresponding to the overburden pressure as a reference datum, both the recompression settlement and the settlement at contact pressures exceeding the overburden pressure can be calculated using the computer program given in Appendix E. If footing loads are given simply as a load increment it may be more convenient to use the computer program given in Appendix H where the reference loading is assumed to equal zero.

The moduli of deformation presented in this chapter were derived by assuming Poisson's ratio equal to 0.4. This was considered to be a reasonable value for the sand deposit and the underlying bedrock formation as the settlements represent relatively drained phenomena wherein some volume change undoubtedly takes place. Poisson's ratio of $\mu = 0.5$ is therefore inadmissible. The use of Poisson's ratio $\mu = 0.3$ would yield moduli only 8 per cent different from those computed with $\mu = 0.4$. Since the quantity $(1-\mu^2)/E$ is constant for a specified settlement data set, a choice of $\mu = 0.3$ in the derivation of field moduli would yield identical settlements to those predicted with $\mu = 0.4$ and the moduli given in this chapter.

Although the available settlement data on the AGT Tower and Oxford Building are inadequate for estimating the long term movements at constant load, the time dependent heaves experienced at the AGT site suggest that at the completion of construction and full occupancy at least 75% of the total settlement will have taken place. The accuracy of this estimate can only be assessed when additional settlement readings are obtained at the AGT Tower and Oxford Building in the near future.

6.12 General Observations and Summary

Work by Gibson (1967) showed that the surface displacements of a loaded elastic half-space are appreciably altered from the simpler homogeneous case when the modulus of elasticity is increased with depth. For an incompressible medium Gibson indicated that the settlements of the surface are concentrated under the loaded area. Surface displacements at distances away from the loaded area will be much smaller than those predicted for a homogeneous half-space. With an increasing modulus of elasticity with depth, the surface settlements were also found to be directly proportional to footing pressure and independent of footing size.

The field data presented in this thesis indicates that non-homogeneity is evident but does not dominate the settlement behaviour of the foundation soil. The evidence for this is as follows.

(a) The deep benchmark at the CN Tower did not appear to be influenced by the building footing loads even though it was in the Boussinesq pressure bulb as mentioned in Chapter II. This observation can be attributed to increasing rigidity of the foundation with depth as this tends to reduce the settlement of depth beneath the loaded area.

(b) The influence of excavation on heave decreased more rapidly with distance away from the excavated area than predicted by the Steinbrenner solution. This also points to some non-homogeneity.

(c) As shown in this chapter, the AGT Tower core footings settled more than the exterior column footings. The smaller settlement at peripheral core footings was less than that which could be attributed by elastic theory to relative footing sizes, locations and loadings and is therefore further evidence of some non-homogeneity of the foundation soil.

From observation of the settlement response of the AGT Tower and the Oxford Building there are several implications of importance to construction practice and design.

(a) When a heavily loaded tower area is surrounded by a lightly loaded building, both within the same excavation, the differential displacements between the two may be intolerable and provision must be made to prevent structural distress. At the AGT Tower the differential between the exterior tower columns and the surrounding parking garage is about 1 cm. to September 22, 1970 and will likely be greater at full occupancy of the building. The differential between the two structures in this case is not critical as a simply supported floating slab allows this movement.

(b) Whenever practical, the application of load should be relatively uniform to all loading areas during construction. At the AGT Tower an initially lighter loading was induced at the exterior footings due to the absence of exterior precast concrete cladding until construction of the structural frame had progressed to about the 12th floor. Excessive delay in placing precast of this type tends to accentuate differential settlement between the interior core and the exterior column line. It is therefore desirable to have the height of precast application follow the construction of the upper floors as closely as possible. Needless to say, any heavy exterior precast cladding should be placed around the structure at a uniform storey height if possible, to prevent excessive loading of one side of a building.

(c) Provided the standard deviation of the field moduli calculated from the settlement response observed on prototype buildings is small, it now becomes possible to "tune" the foundation; that is, to adjust footing sizes such that the differential displacements become minimal. This can easily be accomplished with the computer programs given in Appendix G for the heave response of the excavation and in Appendix E or Appendix H for building settlement.

(d) From the point of view of accurate settlement prediction, a spread footing foundation plan where footings vary widely in elevation is less tractable to analysis.

The important aspects of the analysis presented in this chapter are summarized below.

(a) The settlement response of the Saskatchewan Sands deposit and the underlying over-consolidated bedrock formation to loading is rapid. Settlements during construction can be predicted for all practical purposes by means of elastic theory as the time dependent response of the foundation soil during this period appears to be small.

(b) The derived moduli of the sand deposit and bedrock formation possess a modulus of deformation which is virtually constant at footing pressures less than the overburden pressure. At greater pressures the deformation modulus decreases and therefore appears to be stress dependent.

(c) The elastic properties as determined in conventional laboratory tests deviate appreciably with test type and sample quality and are unsuitable for use in settlement prediction. There is evidence that carefully run cyclic loading tests may yield laboratory moduli which approximate those calculated from settlement observations.

(d) Time dependent heave of lightly loaded areas which surround a heavily loaded interior area will accentuate differential displacements between the two. This was implied earlier in the discussion of heave phenomena given in Chapter V and is confirmed from settlement observations reported in this chapter.

(e) No excess pore pressure was measured in the bedrock formation as a result of the application of a surcharge in the form of a building. This can be attributed to the depth of the water table, permeability of the bedrock mass and stratigraphic detail which provides local drainage surfaces.

CHAPTER VII

CONCLUSIONS AND SUGGESTIONS FOR FURTHER RESEARCH

7.1 Conclusions

The analysis and discussion presented in the preceding chapters have dealt with two research topics. The first is the settlement behaviour of multi-storey buildings founded on over-consolidated soils. The second topic is the influence of the interaction of foundation displacements and structural frame distortions on settlement and structural analysis. Only the more important conclusions which relate to these two aspects of the research work will be given here to avoid repetition of detail particular to a specific building or site.

The foundation contact pressures varied considerably for the four buildings studied from a low of about 7 ksf at the Oxford Building to a maximum of about 26 ksf calculated for one pile base at the Avord Arms. With allowable angular distortions noted in the literature as the criteria of acceptance, no observed differential settlements were found to be excessive. All the measured total settlements were relatively small in comparison with those usually observed for deep deposits of normally-consolidated soils subjected to similar loads.

The proximity of end bearing Franki piles at the Avord Arms to the spread footings at the CN Tower, both on the same till stratum, allowed comparison of the settlement behaviour of the two foundation types. It was shown that for identical contact pressures in excess of the original overburden pressure, the settlement behaviour of piles and spread footings is approximately the same and is solely a function of the deformation properties of the till for that increment of contact pressure.

The settlement during construction of the Avord Arms and the CN Tower is known to dominate the settlement history of these two buildings and constitutes about 75 per cent of the 25 year settlement. The construction displacements of buildings founded on this till stratum can be computed using the equations of the theory of elasticity and the deformation moduli deduced in this thesis. The relationship found for the variation of derived moduli with footing contact pressure indicates that the deformation properties of the till are stress dependent.

In contrast to the recompression modulus of till which varies appreciably with footing contact pressure, the recompression modulus of the Saskatchewan Sand and Gravels and the underlying over-consolidated bedrock formation at the AGT-Oxford Complex site is essentially constant at 42,000 psi. However, at footing pressures in excess of the original overburden pressure, the derived moduli also begin to decrease indicating stress dependence. The foundation moduli derived from heave measurements at the same site are shown to be consistent with those obtained from settlement observations.

The deformation moduli obtained from conventional laboratory tests proved to be unreliable when compared to the field moduli. The ratios of field to laboratory moduli are large indicating that the use of laboratory moduli will lead to gross over-estimates of settlement.

At the overburden pressure the modulus of deformation of the till is about 100,000 psi which is appreciably greater than that obtained for the combined sand deposit and bedrock formation. This contrast was not recognized at the time settlements were predicted for the AGT-Oxford Complex and is therefore of considerable interest.

The heave of the sand deposit and the underlying bedrock formation takes place rapidly and can be estimated for excavations at other sites in Edmonton. The displacements can be computed using elastic theory and the parameters given for the over-consolidated soils in Chapter V. For computation of heave outside the excavation, approximate procedures can be used to allow for the effect of boundary restraint.

The role of site investigation for the purpose of settlement prediction of buildings founded on the over-consolidated till, sand deposit and bedrock formation must be modified. The practice of sampling and identification of soil stratigraphy should be extended to include in-situ testing such as plate bearing tests. As well, good samples should be obtained at the sites and should be subjected in the laboratory to the appropriate tests run at stresses representative of those which occur in the field. Although monitoring of heave during excavation can be useful to determine foundation moduli, this procedure should only be used for verification of field parameters as it is likely to be impractical from the viewpoint of scheduling foundation design and construction.

The four multi-storey structures discussed here varied in rigidity; the most inflexible building is thought to be the CN Tower. Analysis of the effect of relatively small differential footing displacements on the structural frame of this building has shown that support loads are altered by as much as 44 per cent at the end of construction as a result of redistribution caused by frame distortion. It has been shown that when the construction settlement dominates the settlement history of a building, the procedures for computation of interaction effects must allow variation of structural rigidity during construction if the results are to be meaningful. "Conventional" methods of computing soil-structure interaction effects, which utilize the entire building in analysis and generally preclude differential displacements during construction, were found to be inadequate and yielded totally different support reactions. The new incremental method of soil-structure interaction analysis presented in this thesis allows variation of structural rigidity during construction and is a considerable improvement over "conventional" techniques. Buildings of moderate rigidity and founded on soils which are conducive to differential construction settlements, should be analyzed using incremental soil-structure interaction techniques. The incremental procedure in design practice will be useful in order to assess the probability of incipient structural distress of individual members within the distorted structural space frame.

7.2 Suggestions for Further Research

From the work described in this thesis it appears that there are several subjects for which further research is appropriate. These are:

(a) the precise determination of heave with distance away from an excavation boundary. This subject is of considerable interest (Peck, 1969) and will allow evaluation of the accuracy of the approximate procedure presented in Chapter V.

(b) the measurement of rebound with depth. At the AGT site only two rebound points were horizontally coincident and further examination of the depth affected in relation to the depth of excavation would be desirable.

(c) The evaluation of in-situ tests for determination of field deformation moduli for sites where previous experience is lacking. Since the settlement behaviour of prototype buildings on the till deposit or sand stratum and bedrock formation are now known, an investigation of the suitability of plate bearing and pressuremeter tests on these soils will be of great value.

(d) the correlation of laboratory deformation moduli with field moduli. The relevance of cyclic consolidated drained tests, or other tests, run on till and bedrock should be investigated. This program should be extensive and include testing of specimens obtained at different depths by pitcher tube sampling, block sampling or other means.

(e) the combination of the analysis of structural stiffness with the incremental soil-structure interaction program. Presently structural stiffness, in terms of reaction coefficients, must be assessed by say ICES-STRUDL and converted to cards for use in the interaction solution. This step can be tedious for multi-footing structures and should be eliminated.

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APPENDIX A

BOREHOLE LOGS
CN TOWER AND AVORD ARMS

R. M. HARDY
&
ASSOCIATES LIMITED
EDMONTON — ALBERTA

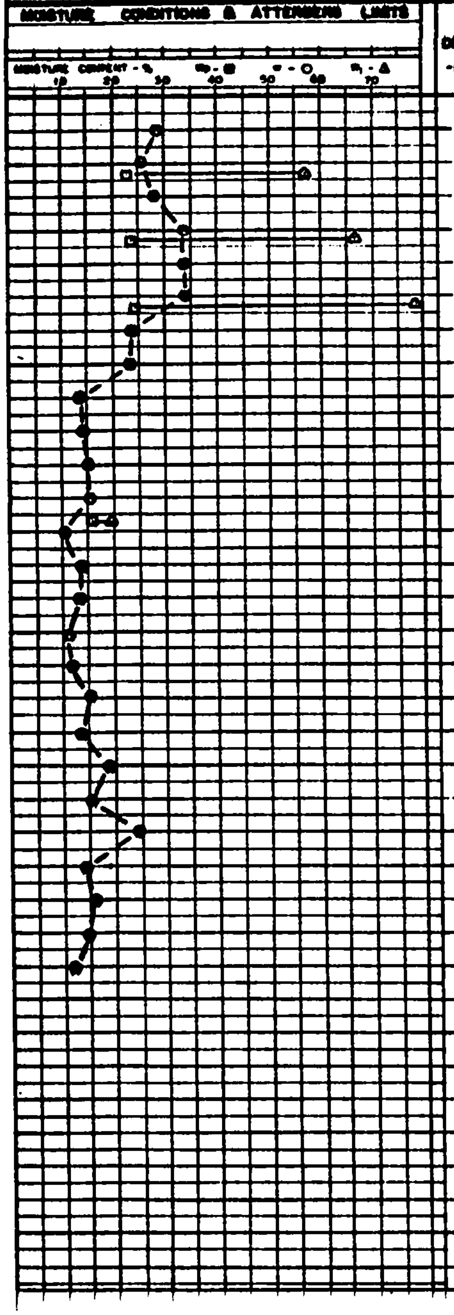
DWN. | CKD.

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT C.N.TOWER & TERMINAL BUILDING

JOB NO. C-1125 | DATE | HOLE 1 | PLATE C-1125-2

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL PROFILE — SOIL DESCRIPTION

DATUM Geodetic
SURFACE ELEVATION 2188.8

DEPTH -FEET-	SOIL DESCRIPTION	TYPE	PENETRATION RESISTANCE	OTHER TESTS
	FILL			
2				
4	CLAY,med.brown, silty, stiff	U_1		q_u = 1.6 tsf
6				
8	slightly laminated, high plastic some traces of rust	U_2		q_u = 1.4 tsf
10				
12		U_3		q_u = 1.2 tsf
14	TILL,clayey,soft, silty,med.brown, rust spots, coal sps			
16		U_4		
18	sandy			
20	clayey,sandy,dense, rust spots, coal specks			
22		DS	121 10"	
24				
26	sand lenses	DS	100 10"	
28				
30	sand lenses	DS	152	
32				
34				
36		DS	100 10"	
38				
40	small lenses of coarse sand and some pea gravel	DS	100 10"	
42				
44	sand water bearing			
46	clayey,very dense traces of clay shale from 47'	DS	100 11"	
48				
50				
52				
54	sandy,clayey,coal specks,sand lenses	DS	100 6"	
56				
58				
60	clayey,dark grey, very dense,stones, coal specks,rust spots			
62		DS	117	
64				
66				
	DRILLED TO 100'			

PLATE C-1125-2

R.M. HARDY & ASSOCIATES LTD.
EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT CN Tower & Terminal Building

R.D.
PLATE 3

SOIL PROFILE

SOIL DESCRIPTION

GEODETIC

SURFACE ELEV. 2188.3

4" CONCRETE

TOPSOIL DRY — TRACE OF GRAVEL

CLAY MED BROWN VERY STIFF SLIGHTLY SILTY,

MORE SILTY

SLIGHTLY NUGGETTED, SALT TRACES

HEAVY RUST STREAKS VERY SILTY

SLIGHTLY WETTER

GLACIAL TILL
MED. BROWN, VERY SILTY CLAYEY, COAL SPECKS, RUST TRACES
SANDY DENSER FROM 15 FT.
MORE DENSE FROM 17 FT.

SANDY, SILTY, DRY

MANY STONES

CLAYEY, SANDY

WET SAND LENSE
SANDY

CLAYEY
OCCASIONAL WET SAND LENSES

SANDY

CLAYEY, DENSE
OCCASIONAL SAND LENSES

CLAYEY, DENSE
SOME SLOUGHING OF SAND ABOVE

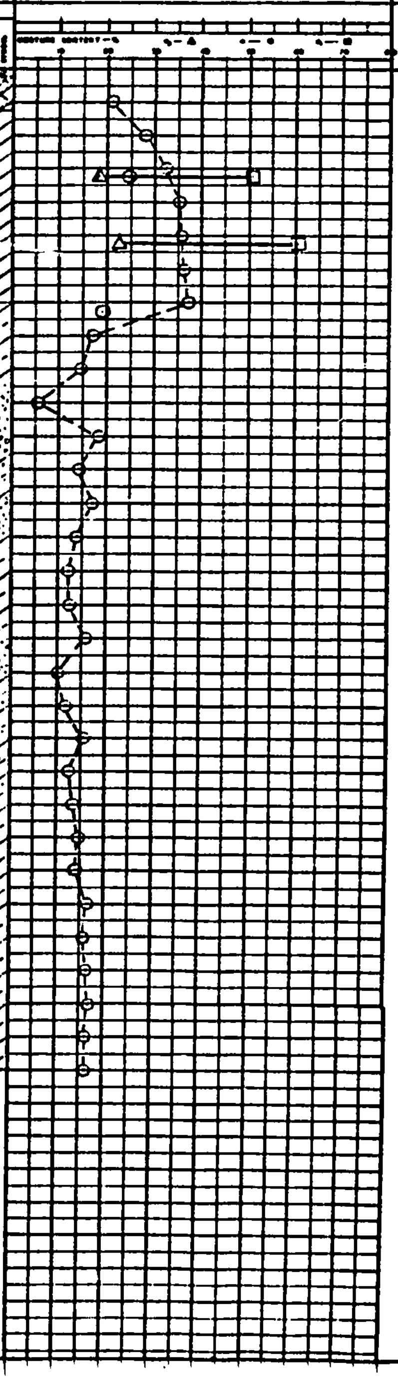
CLAYEY, DARK GREY

CLAYEY,
TRACES OF CLAY-SHALE

END OF HOLE

NOTE: WATER LEVEL 27.6 FT. BELOW SURFACE 20 HRS. AFTER DRILLING

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES

OTHER TESTS

U₁	
U₂	QU = 2.3 T/ft² P.P. = 2.6 T/ft² % SO₄ = 0.01
U₃	
U₄	QU = 2.9 T/ft² P.P. = 3.0 T/ft²
D.S. N=39	
D.S. N=91	
D.S. N=97	
D.S. N=80 for 10"	
D.S. N=125	
D.S. N=84 for 6"	HIT STONE
D.S. N=101	
D.S. N=73	
D.S. N=72	

PLATE 3

R.M. HARDY
& ASSOCIATES LTD.
EDMONTON — ALBERTA

BY R.D.
CK'D

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT CN Tower & Terminal Building

JOB NO. [illegible]
DATE SEPT. [illegible] '67
HOLE 3
PLATE 4

SOIL PROFILE

SOIL DESCRIPTION

ELEV. DEPTH FEET

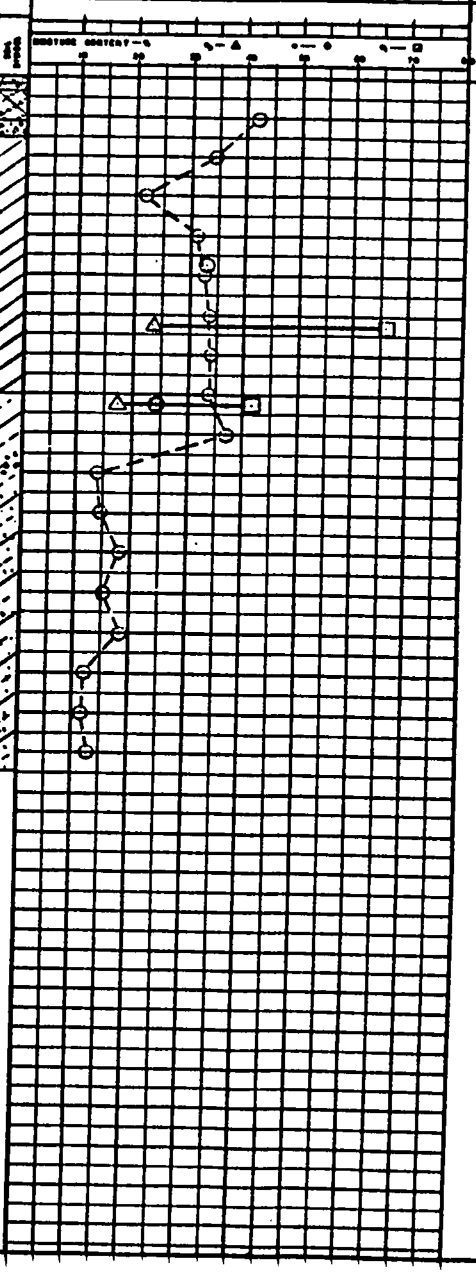
AND GEODETIC

SURFACE ELEV. 2189.1

Depth (feet)	Soil description
	4" CONCRETE
	SAND & GRAVEL
2	TOPSOIL BLACK
	FILL SAND & GRAVEL
4	CLAY MOTTLED SILTY STIFF SAND LENSES, STONES
6	[COULD BE FILL]
8	MED. BROWN, SILTY, STIFF
10	VERY STIFF FROM 10 FT.
12	
14	
16	GLACIAL TILL MED. BROWN, DENSE, SILTY
18	VERY SILTY FROM 17 FT.
	STONES
20	CLAYEY, MOTTLED, SANDY, COAL SPECKS
22	SAND LENSES
24	
26	
28	TRACE OF WATER
30	
32	
34	
36	END OF HOLE

NOTE: SOME SLOUGHING IN TEST HOLE. SLOUGHED TO 25 FT. BELOW SURFACE SOME 15 HRS. AFTER DRILLING.

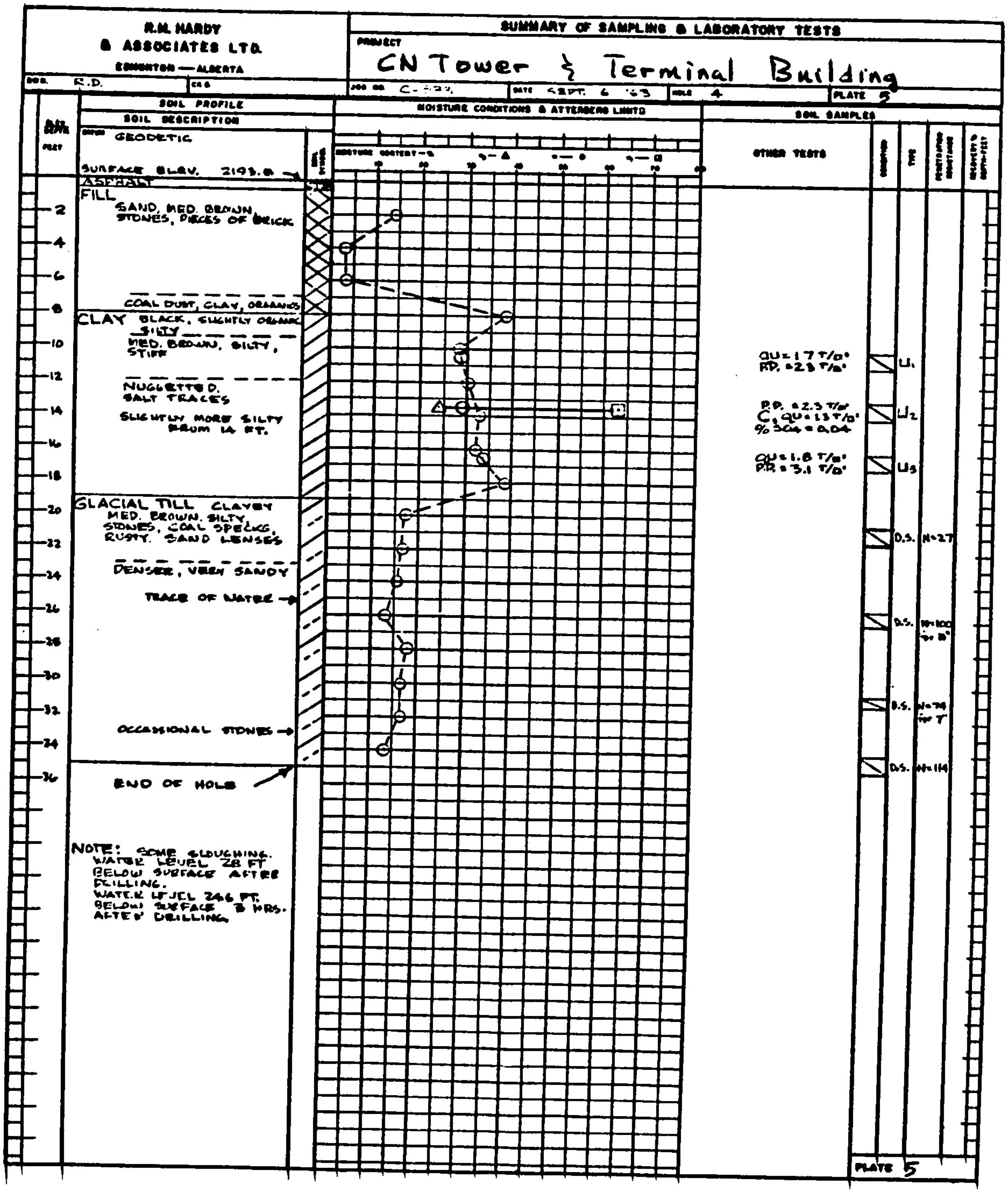
MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES

OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY & DEPTH—FEET
		U_1		
		U_2		
qu = 1.5 T/ft² P.P. = 2.4 T/ft²		U_3		
		U_4		
qu = 1.5 T/ft² P.P. = 2.3 T/ft²		U_5		
HIT STONE		D.S.	N=40 for 9"	
		D.S.	N=86	
		D.S.	N=100 for 9"	
		D.S.	N=100 for 10"	

PLATE 4



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SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT CN Tower & Terminal Building

R.D.

JOB NO. C-[illegible]

DATE SEPT. 6 '63

HOLE 4

PLATE 5

SOIL PROFILE

SOIL DESCRIPTION

GEODETIC

SURFACE ELEV. 2193.8

ASPHALT

FILL

SAND, MED. BROWN, STONES, PIECES OF BRICK

COAL DUST, CLAY, ORGANICS

CLAY BLACK, SLIGHTLY ORGANIC SILTY

MED. BROWN, SILTY, STIFF

NUGGETTED. SALT TRACES

SLIGHTLY MORE SILTY FROM 14 FT.

GLACIAL TILL CLAYEY MED. BROWN. SILTY, STONES, COAL SPECKS, RUSTY. SAND LENSES

DENSER, VERY SANDY

TRACE OF WATER →

OCCASIONAL STONES →

END OF HOLE

NOTE: SOME SLOUGHING. WATER LEVEL 28 FT BELOW SURFACE AFTER DRILLING. WATER LEVEL 24.6 FT. BELOW SURFACE 3 HRS. AFTER DRILLING.

SOIL SAMPLES

OTHER TESTS	TYPE	
QU = 1.7 T/□' P.P. = 2.3 T/□'	U1	
P.P. = 2.3 T/□' C, QU = 1.3 T/□' % SO4 = 0.04	U2	
QU = 1.8 T/□' P.P. = 3.1 T/□'	U3	
	D.S.	N = 27
	D.S.	N = 100 for 10"
	D.S.	N = 74 for 7"
	D.S.	N = 114

PLATE 5

R.M. HARDY & ASSOCIATES LTD.
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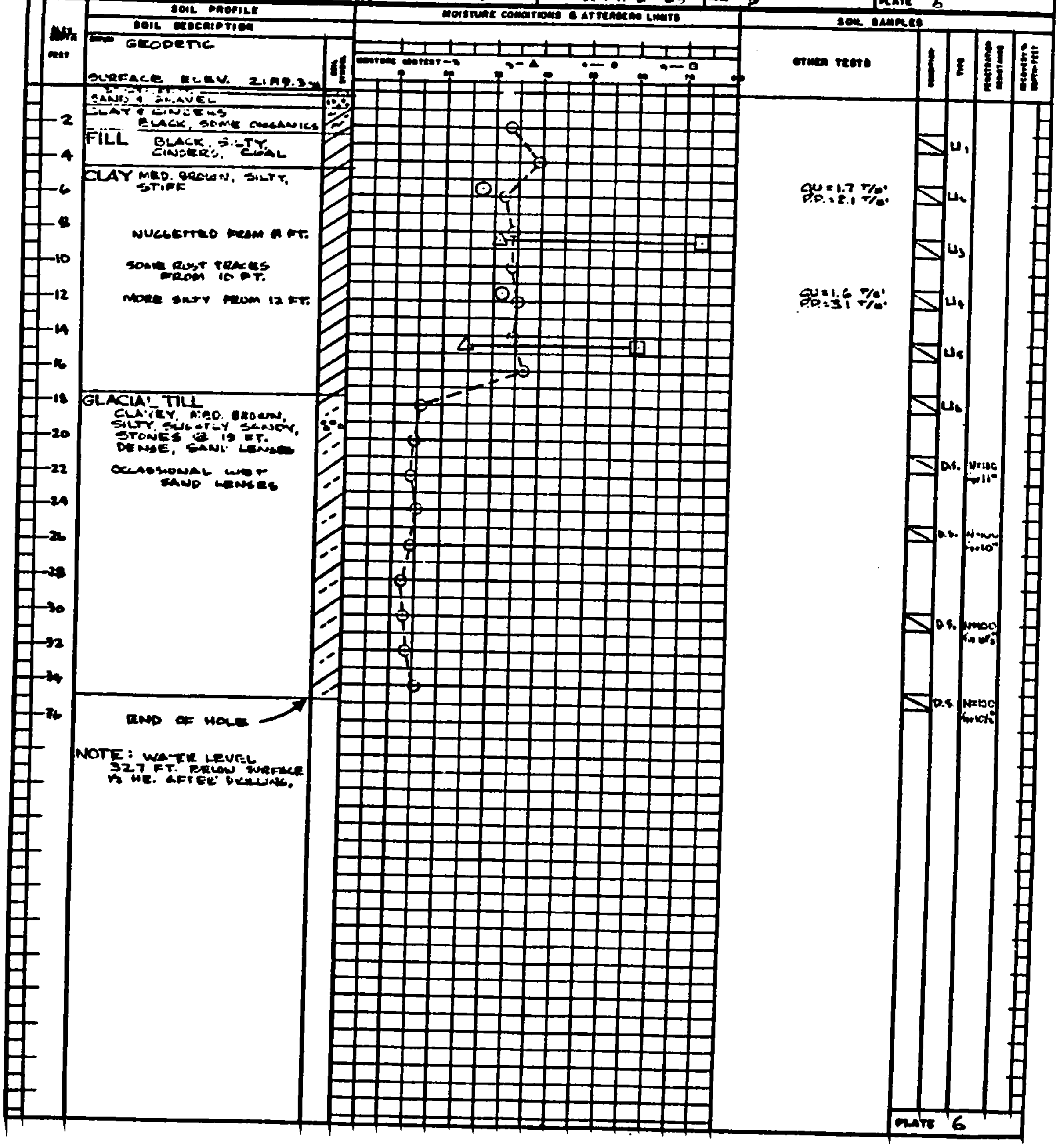
SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: CN Tower & Terminal Building

DWN. G.D	JOB NO. C-433	DATE SEPT. 6 '63	HOLE 5	PLATE 6



GEODETIC
SURFACE ELEV. 2189.3

SAND & GRAVEL

CLAY & CINDERS BLACK, SOME ORGANICS

FILL BLACK, SILTY, CINDERS, COAL

CLAY MED. BROWN, SILTY, STIFF

NUGGETTED FROM 8 FT.

SOME RUST TRACES FROM 10 FT.

MORE SILTY FROM 12 FT.

GLACIAL TILL CLAYEY, MED. BROWN, SILTY, SLIGHTLY SANDY, STONES @ 19 FT. DENSE, SAND LENSES

OCCASIONAL WET SAND LENSES

END OF HOLE

NOTE: WATER LEVEL 32.7 FT. BELOW SURFACE ½ HR. AFTER DRILLING.

OTHER TESTS

QU = 1.7 T/□'
P.P. = 2.1 T/□'

QU = 1.6 T/□'
P.P. = 3.1 T/□'

TYPE	PENETRATION RESISTANCE
U1	
U2	
U3	
U4	
U5	
U6	
D.S.	N=180 for 11"
D.S.	N=[illegible] for 10"
D.S.	N=100 for 1½"
D.S.	N=100 for 1½"

PLATE 6

R. M. HARDY
&
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EDMONTON — ALBERTA

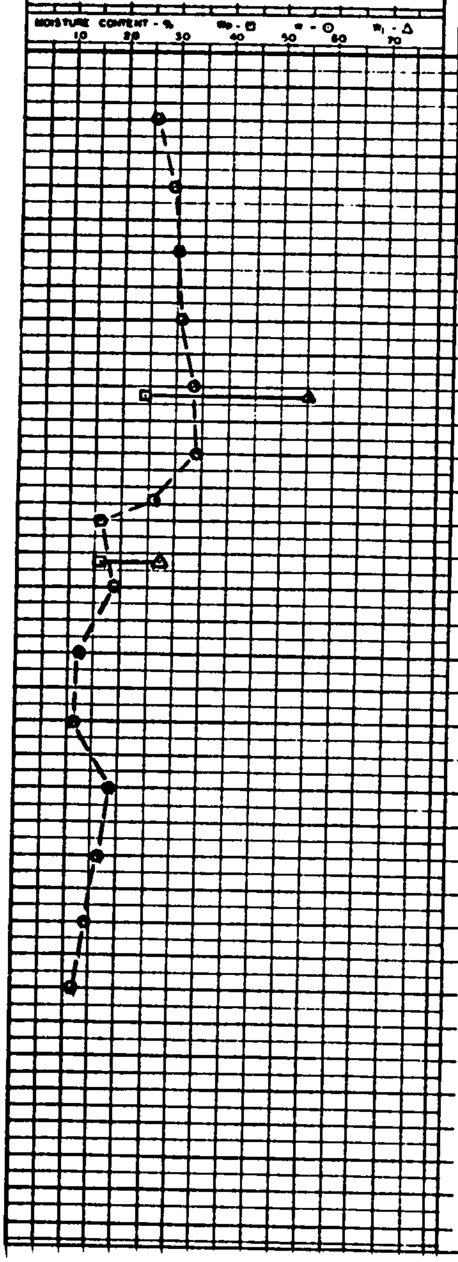
DWN.
CKD.

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT **AVORD ARMS BUILDING**

JOB NO. C-1125
DATE
HOLE 1
PLATE C-1125-3

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL PROFILE — SOIL DESCRIPTION
DATUM City of Edmonton
SURFACE ELEVATION 231.3

DEPTH -FEET-	SOIL DESCRIPTION	SOIL SAMPLES: TYPE	PENETRATION RESISTANCE	OTHER TESTS
	FILL Gravel, Clay			
1	CLAY, med.brown, silty,stiff			
2				
3	slightly nuggetted			
4	from 3'	U1		q_u = 0.5 tsf
5				
6	more silty from 6'			
7	occasional salt traces			
8		U2		q_u = 1.3 tsf SO_4 = 0.02%
9				
10	more silty from 10'			
11	occasional rust traces			
12		U3		q_u = 1.3 tsf.
13				
14	TILL,clayey,silty, medium brown,dense,	U4		q_u = 1.6 tsf
15	slightly sandy,			
16	stones,coal specks,	U5		
17	occasional thin sand lenses	DS	100 9"	
18				
19	med.brown,dense, fine silty sand			
20		DS	111	
21	med.brown,dense, fine sandy silt			
22				
23	clayey,coal specks, mottled, stones			
24		DS	100 8"	hit stone
25	brown,fine silty sand,very dense,dry			
26	clayey,stones,silty, coal specks, rust, salts.			
27				
28				
29	END OF HOLE	DS	102	

APPENDIX B

TYPICAL BOREHOLE LOGS

AGT-OXFORD COMPLEX SITE

Note - The typical borehole logs presented in this Appendix were selected to provide a distribution of the drilling record over the entire AGT-Oxford Complex site. These logs have been printed directly from the drilling technicians notes and therefore include some errors in description (e.g., clay stratum under sandy clay till, page B-18). The logs were not used in this thesis except to provide a broad definition of stratigraphic units and interface elevations.

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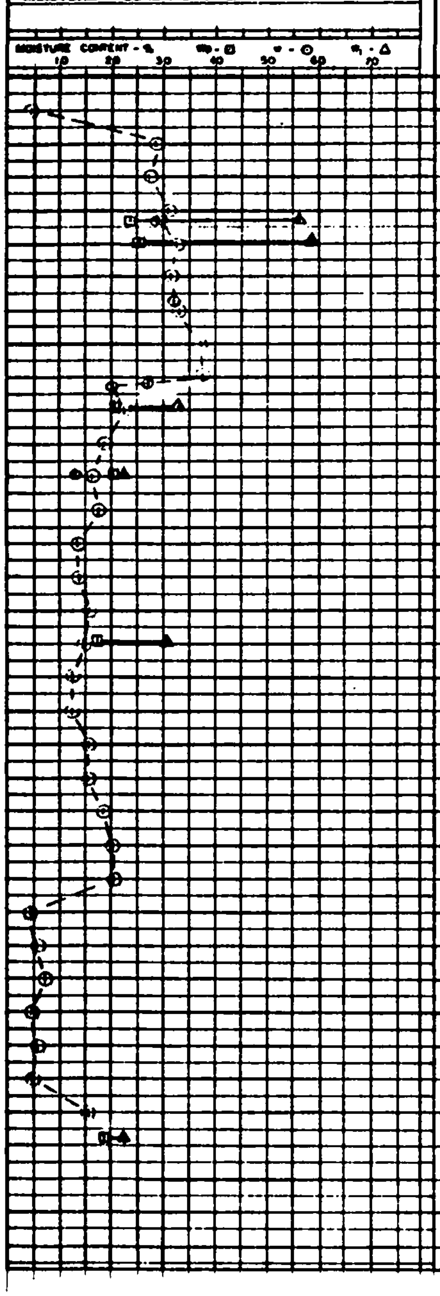
EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T.-OXFORD COMPLEX, EDMONTON, ALBERTA.

DRN. RJB
CKD.
JOB NO. C1534
DATE MAR 67
HOLE 1
PLATE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL PROFILE — SOIL DESCRIPTION

DATUM CITY
SURFACE ELEVATION 2232.5

DEPTH -FEET-	SOIL DESCRIPTION	TYPE	PENETRATION RESISTANCE	OTHER TESTS
2	FILL: GRAVEL, SANDY, CLAYEY			
4	TOPSOIL			
6	CLAY, SILTY MEDIUM PLASTIC MEDIUM BROWN			pp = 2.7 T/□' qu = 2.5 T/□'
8	(CH) MOTTLED NUGGET STRUCTURE STREAKS OF SILT	U1		UU = 3.2 γ = 120 lb/ft³
10	STIFF	DS	15	SO4 = 0.09%.
12				pp = 2.3 T/□'.
14		U2		UU = 2.63 " & qu = 1.6 T/□'. γ = 120 lb/ft³
16	VERY SILTY (CH) LT. BRWN MOTTLED WITH MED. BROWN MED. TO HIGH PLASTIC	DS	15	
18	SILT, CLAYEY	U3		p.p. = 2.2 T/□' & 2.2 T/□' UU = 3.0 " & qu = 1.6 "
20	MEDIUM BROWN (CI) COAL SPECKS	DS	19	γ = 126 lb/ft³
22	CLAY (TILL) (CL) SANDY, SILTY			SO4 = 0.11%. pp = 2.5 T/□'. m/c = 12.7% to 13.4%.
24	SAND LENSES (SM) VERY SANDY	U4 DS	>100	SO4 = 0.09%. pass 200 = 40%.
26	MED. BROWN RUST & COAL SPECKS			
28	(CI) MED. PLASTIC			
30	(SC) VERY SANDY LOW PLASTIC	DS	80	
32	(CI) LOW PLASTIC			
34	VERY DENSE	DS	70	
36				
38	2" SAND LENS	DS	51	
40	GREY BROWN			
42				
44	SAND LENS FINE	DS	90	
46	SILTY GREY BROWN			
48		DS	61	
50	SAND			
52	(SASK. SANDS & GRAVELS) (SW) MEDIUM SIZE, LIGHT TO MEDIUM			
54	BROWN COAL SPECKS	DS	80	SO4 = 0.09%. M.A.
56				
58		DS	88	NOTES: FROZEN TO 4½'
60				
62				
64	(SM) CLAYEY, SILTY	DS	>100	M.A.
66	GRAVEL END OF HOLE			
68				
70				

PLATE

B-2

RMH R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALTA.

DWN GM	CKD [illegible]	DATE JUN 67	HOLE 1-A	JOB NO C1504	PLATE

WATER CONTENT

WATER CONTENT - % 10 20 30 40 50 60 70 w_p - □ w - ○ w_L - △

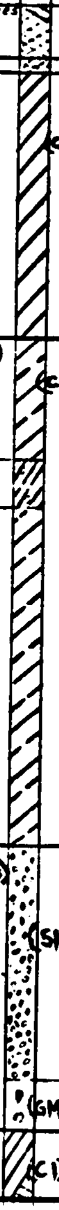
SOIL PROFILE

SOIL DESCRIPTION

DATUM CITY OF EDMONTON

SURFACE ELEVATION 2232.3

SOIL SAMPLES: CONDITION | TYPE | PENETRATION RESISTANCE | RECOVERY % | OTHER TESTS



DEPTH FEET	SOIL DESCRIPTION	UNIFIED CLASSIF.
0–4	FILL – BROKEN RED BRICKS, STONES ASPHALT ORGANIC CLAY, RUST LENS [illegible]	
4	TOPSOIL [illegible] AND [illegible]	
4–20	CLAY - SILTY LIGHT GREYISH BROWN MED. PLASTIC MEDIUM PLASTIC DARK BROWN	(CH)
20–50	SANDY CLAY (TILL) COAL SPECKS ROCKS <¼"ø DARK GREY BROWN LIGHT GREY BROWN. ROCKS <1"ø 75% ROCKS COAL CLAY COAL SPECKS RUST LENS STONES <½"ø CLAY DARK GREY MEDIUM PLASTIC STONES <⅛"ø COAL RUST LENS DARK GREY & LIGHT BROWN	(CL)
50–64	SAND (SASK. SANDS & GRAVELS)	(SM)
64–67	GRAVEL ROUNDED <3"ø COAL	(GM)
67–70	CLAY SHALE (BEDROCK) COAL SPECKS	(CI)

PLATE

B-3

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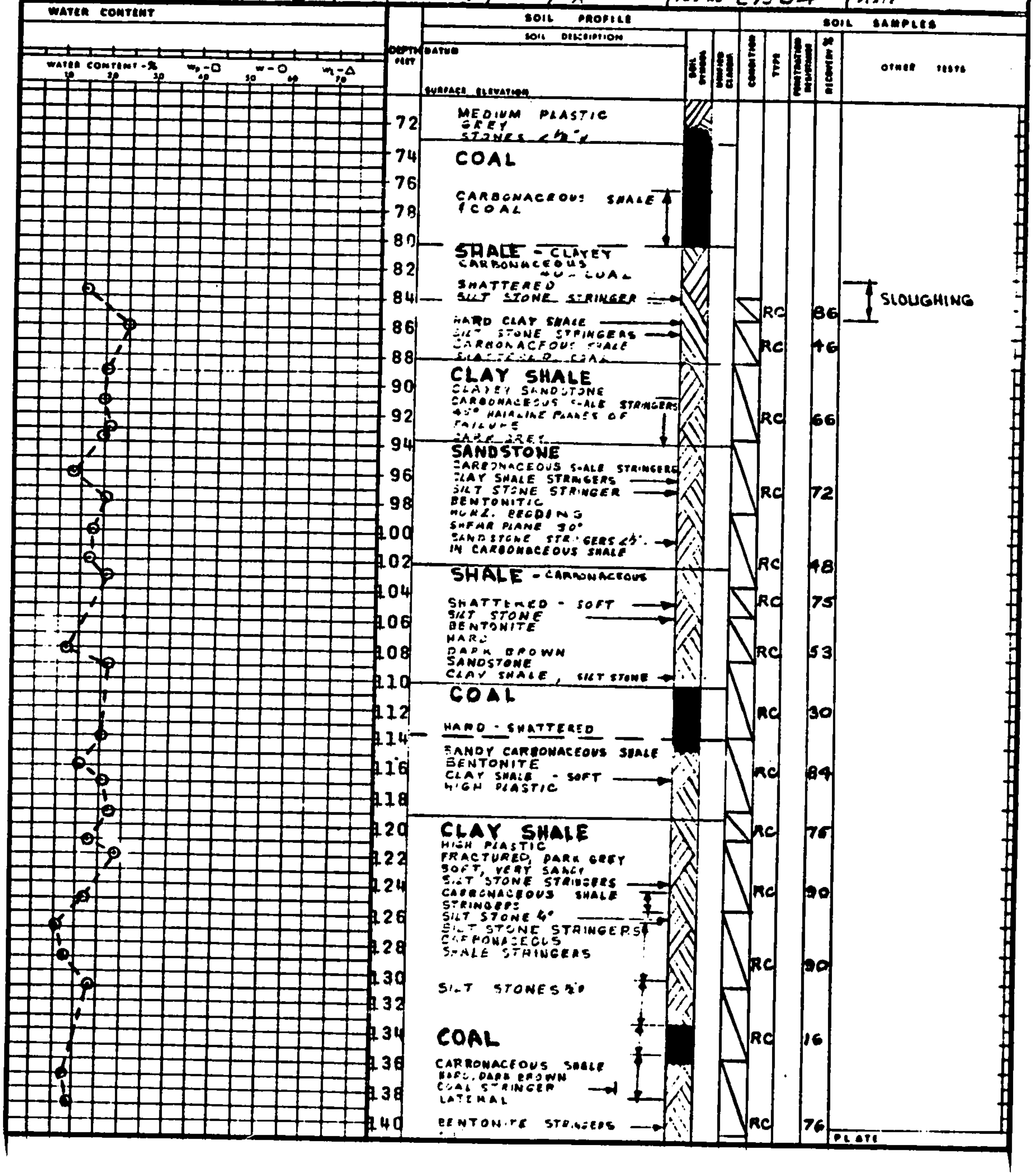
SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN G.M.	CKD RJE.	DATE JUN 67	HOLE 1-A	JOB NO C1504	PLATE



MEDIUM PLASTIC
GREY
STONES <½"

COAL

CARBONACEOUS SHALE
& COAL

SHALE - CLAYEY
CARBONACEOUS
-- COAL
SHATTERED
SILT STONE STRINGER
HARD CLAY SHALE
SILT STONE STRINGERS
CARBONACEOUS SHALE
SHATTERED COAL

CLAY SHALE
CLAYEY SANDSTONE
CARBONACEOUS SHALE STRINGERS
45° HAIRLINE PLANES OF
FAILURE
DARK GREY

SANDSTONE
CARBONACEOUS SHALE STRINGERS
CLAY SHALE STRINGERS
SILT STONE STRINGER
BENTONITIC
HORZ. BEDDING
SHEAR PLANE 30°
SANDSTONE STRINGERS <½".
IN CARBONACEOUS SHALE

SHALE - CARBONACEOUS
SHATTERED - SOFT
SILT STONE
BENTONITE
HARD
DARK BROWN
SANDSTONE
CLAY SHALE, SILT STONE

COAL
HARD - SHATTERED
SANDY CARBONACEOUS SHALE
BENTONITE
CLAY SHALE - SOFT
HIGH PLASTIC

CLAY SHALE
HIGH PLASTIC
FRACTURED, DARK GREY
SOFT, VERY SANDY
SILT STONE STRINGERS
CARBONACEOUS SHALE
STRINGERS
SILT STONE 4"
SILT STONE STRINGERS
CARBONACEOUS
SHALE STRINGERS
SILT STONES 2"

COAL
CARBONACEOUS SHALE
HARD, DARK BROWN
COAL STRINGER
LATERAL
BENTONITE STRINGERS

TYPE	RECOVERY %	OTHER TESTS
RC	86	SLOUGHING
RC	46	
RC	66	
RC	72	
RC	48	
RC	75	
RC	53	
RC	30	
RC	84	
RC	76	
RC	90	
RC	90	
RC	16	
RC	76	

B-4

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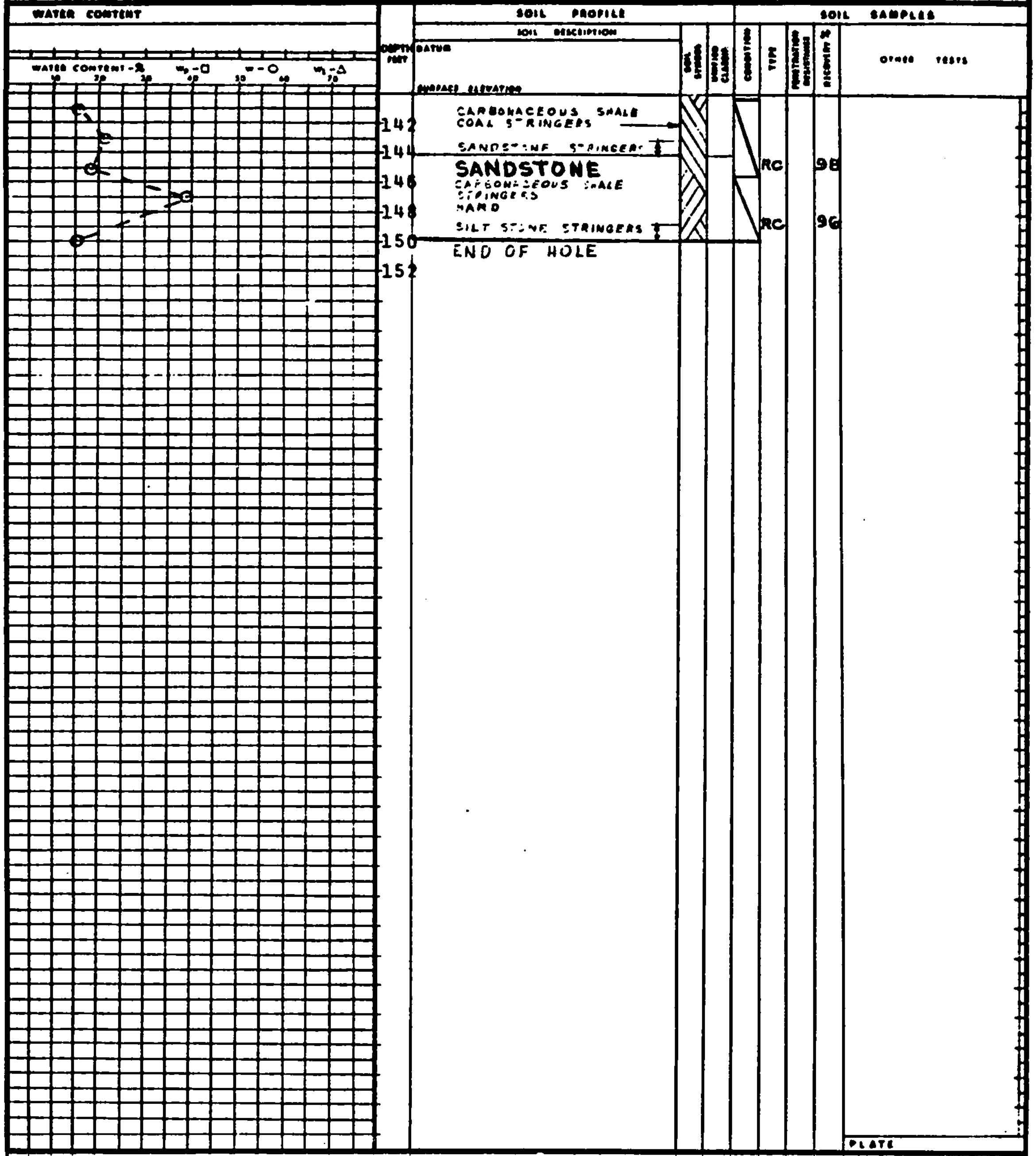
SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN G.M.	CK'D	DATE JUN 67	HOLE 1-A	JOB NO C1504	PLATE



CARBONACEOUS SHALE
COAL STRINGERS
SANDSTONE STRINGERS

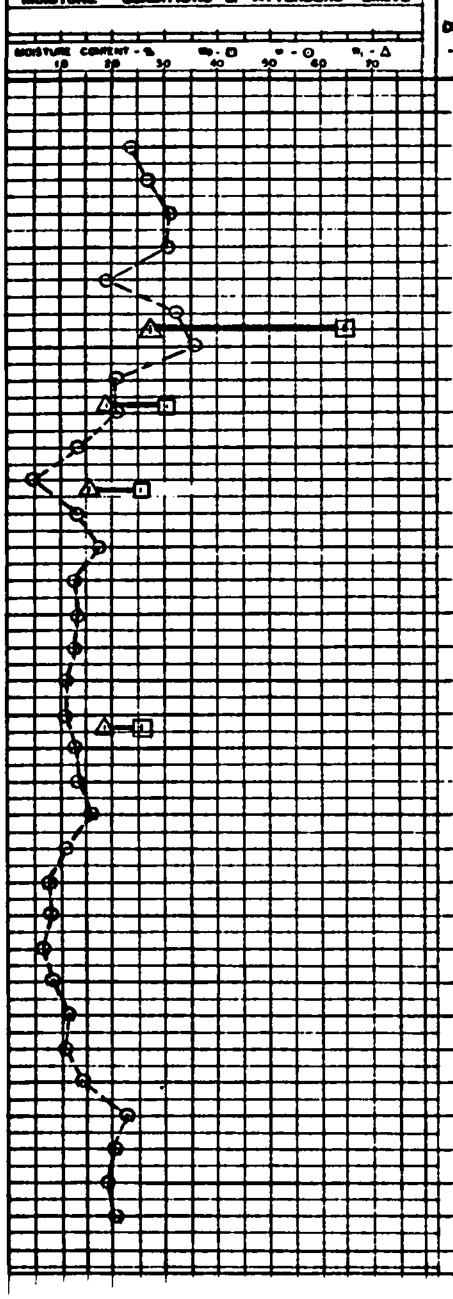
SANDSTONE
CARBONACEOUS SHALE
STRINGERS
HARD
SILT STONE STRINGERS

END OF HOLE

B-5

R. M. HARDY & ASSOCIATES LIMITED EDMONTON — ALBERTA	**SUMMARY OF SAMPLING & LABORATORY TESTS** PROJECT RULE, WYNN , FORBES, LORD & PARTNERS A.G.T.-OXFORD COMPLEX, EDMONTON, ALBERTA.
DWN. RJE	CKD.
JOB NO. C1504	DATE MAR 67
HOLE 2	PLATE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL PROFILE — **SOIL SAMPLES**

DATUM CITY
SURFACE ELEVATION 2253.4

DEPTH -FEET-	SOIL DESCRIPTION	TYPE	PENETRATION RESISTANCE	OTHER TESTS
2	FILL: 2" ASPHALT, 6" CONCRETE, 16" CLAY			
4	SILTY CLAY MEDIUM PLASTIC			
6	MED. BWN MOTTLED WITH LIGHT BROWN (CH) STIFF			
8	SALTS SILT POCKETS	U1		
10		DS	11	
12				
14	PEBBLES, COAL, SALTS	U2 DS	15	SO4 = 0.09 %
16	CH VERY SILTY MED. PLASTIC			
18	CLAYEY SILT CI MED. TO LIGHT BROWN	U3		
20	SOFT, FREE WATER @ 17'	DS	24	
22	CLAYEY SAND (TILL) SAND LENS			
24	SC VERY DENSE LT BROWN & GREY	U4 DS	>100	
26	COAL & PEBBLES			SO4 = 0.09 % M.A.
28	(SW) SAND LENS VERY DENSE LIGHT GREY & BROWN	DS	>100	
30				
32				
34	SILTY, VERY CLAYEY VERY DENSE	DS	>100	
36	PEBBLES & COAL SPECKS MEDIUM BROWN LOW PLASTIC			
38	RUST POCKETS CL-ML	DS	>100	M.A.
40				
42				
44		DS	100	
46				
48	SAND (SASK. SANDS & GRAVEL) FINE TO MEDIUM			
50	(SW) VERY DENSE TRACE OF SILT	DS	>100	
52	GREEN GRAY COAL SPECKS			
54		DS	>100	M.A.
56	(SW)			
58	GRAVEL MD BROWN.	DS	>100	M.A.
60	(GW) STONES <2½"			
62	SANDY CLAY CLAY SHALE			
64	(CH) (EDMONTON FORMATION) WEATHERED	DS	71	
66	SLIGHTLY BENTONITIC MEDIUM GREY			
68		DS	>100	
70	END OF HOLE			PLATE

B-6

R. M. HARDY
&
ASSOCIATES LIMITED
EDMONTON — ALBERTA

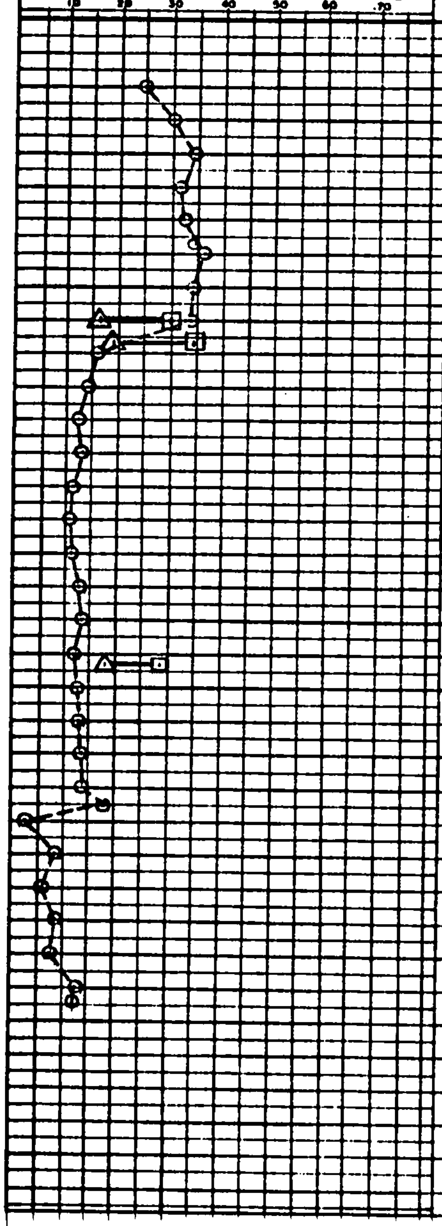
SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T.-OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN. RJE CKD.

JOB NO. C1504 DATE MAR 67 HOLE 3 PLATE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL PROFILE — **SOIL SAMPLES**

SOIL DESCRIPTION

DATUM CITY

SURFACE ELEVATION 2231.9

DEPTH -FEET-	SOIL DESCRIPTION	TYPE	PENETRATION RESISTANCE	OTHER TESTS
2	FILL: GRAVEL CLAY, WOOD DECAYED ORGANICS			
4	SILTY CLAY			
6	(CH) HIGHLY PLASTIC DARK GREY, RUST STREAKS			
8	(CH) MED. PLASTIC LIGHT BROWN, MOTTLED, SALTS, NUGGET STRUCTURE, SILT LENSES	U1		P.P. = 2.5 T/□'
10		DS	14	
12	(CH) MED. BROWN WITH BLUE MOTTLING, MED. TO HIGH PLASTIC, RUST STREAKS NUGGET STRUCTURE			γ_d = 85.2 P.C.F. P.P. = 3.0 T/□' U.U. = 2.38 T/□' q_u = 1.4 T/□'
14		U2		
16		DS	15	
18	CLAYEY SILT (CL) MED. CONSISTENCY	U3		P.P = 4.5+ T/□' PASS #200 = 74%
20	CLAYEY SAND (TILL) (CL) MED. BROWN, DENSE	DS	46	PASS #200 = 66%
22	(SC) SAND LENS, MED. MED. BWN, COAL SPECKS			
24	(CL) VERY DENSE DARK BROWN LOW PLASTIC	U4 DS	>100	PASS # 200 = 72.0%
28	3" WET SAND & GRAVEL	DS	>100	
30	DARK GREY BROWN			
34	BLUE GREY, COAL STONES <1"	DS	>100	
36	SAND LENS FINE, SILTY NON PLASTIC REDDISH MED. BROWN			
38	SC 5" SILT LENS	DS	>100	PASS #200 = 48%
40	DARK GREY VERY DENSE			
44	1" FINE SAND MANY SMALL STONES	DS	56	
46	RUSTY LENSES			
48	SAND (SASK. SANDS & GRAVELS) MEDIUM GRAINED (SW) VERY DENSE PEPPERED GREY BWN. COAL SPECKS COAL LENSES < 1/16" TRACE OF SILT	DS	>100	
54		DS	100	
58		DS	90	
60	END OF HOLE			

NOTES:

1. APPRECIABLE FREE WATER FROM 28', BUT INSUFFICIENT VOLUME TO ESTABLISH STATIC ∇.
2. HOLE COLLAPSED AT 52' AFTER DRILLING
3. FROZEN TO 5'

PLATE

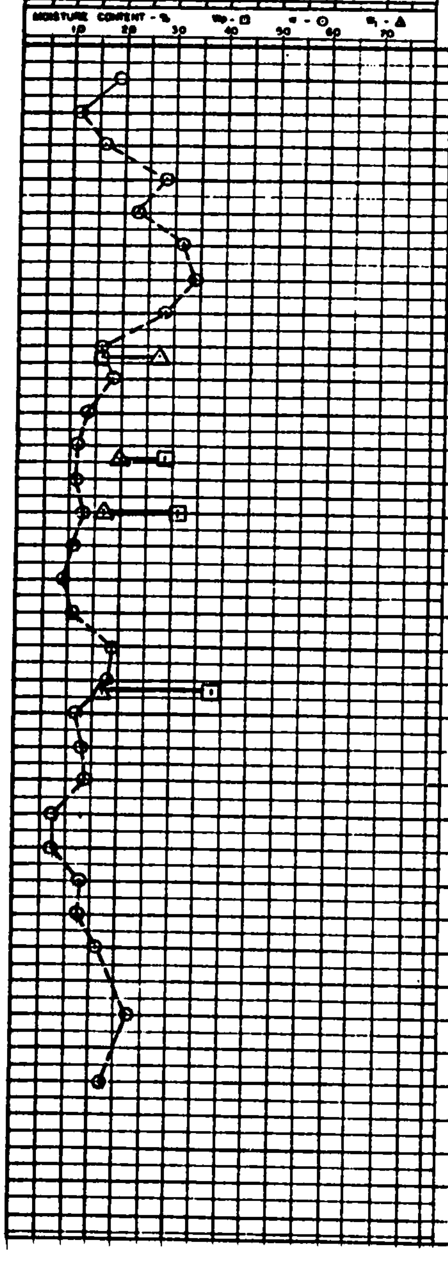
R. M. HARDY & ASSOCIATES LIMITED
EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T.-OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN. RJB
CKD.
JOB NO. C1504
DATE MAR. 67
HOLE 4
PLATE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL PROFILE — SOIL DESCRIPTION

DATUM CITY
SURFACE ELEVATION 2251.8

SOIL SAMPLES

DEPTH -FEET-	SOIL DESCRIPTION	TYPE	PENETRATION RESISTANCE	OTHER TESTS
2	FILL: 6" TOPSOIL 15" SILTY CLAY 2' TOPSOIL			
4		U1		
6	SILTY CLAY 2" SAND LENSE	DS	20	SO4 = 0.09 %
8	(CH) MED. BROWN, SILT LENSES, SALTS ROOTLETS, SAND POCKETS	U2		
10	TILL-LIKE, MD. BWN SANDY, SILTY	DS	17	
12	MED. TO HIGH PLASTIC			
14	(CH) MED. BROWN MOTTLED SILT LENSES FISSURED	U3 DS	 17	P.P. = 3.0 T/□'
16				
18	SANDY CLAY (TILL) (CL) LOW PLASTIC STIFF	U4		
20	CL FINE SAND LENS SILTY LAYERS	DS	23	PASS #200 = 38 %
22	MED TO DARK BROWN PEBBLES, COAL SPECKS (CL)			
24	HARD RUST LENSES	U5 DS	 >100	M.A.
26				
28	CI	U6		γ_d = 121.5 P.C.F.
30	5" fine sand	DS	>100	
32	(SW) SAND LENS FINE, COAL SPECKS VERY DENSE LIGHT TO MED. BWN			
34		DS	>100	PASS #200 = 34 %
36	SM SILTIER WITH DEPTH			
38	(ML) SILT LENS, SANDY NON PLASTIC, WET			
40	CI DARK GREY BROWN MED. PLASTIC SANDY CLAY (TILL) HARD	DS	48	
42				
44		DS	>100	
46	SAND (SASK. SANDS & GRAVELS) (SW) MED., COAL SPECKS & LENSES, VERY DENSE MED. TO DK. BROWN			
48				
50		DS	>100	
52	(SM) SILTY, FINE GREEN BROWN COAL SPECKS			
54	(SW) MEDIUM GREEN BROWN	DS	>100	
56	SILTY CLAY (REWORKED BEDROCK) MED. TO HIGH PLASTIC VERY STIFF			
58		DS	56	
60	SANDSTONE (EDMONTON FORMATION) BENTONITIC, GREY WEATHERED			
62		As		M.A.
64	END OF HOLE			
66	NO FREE WATER			
68	FROZEN TO 1½'			
70				

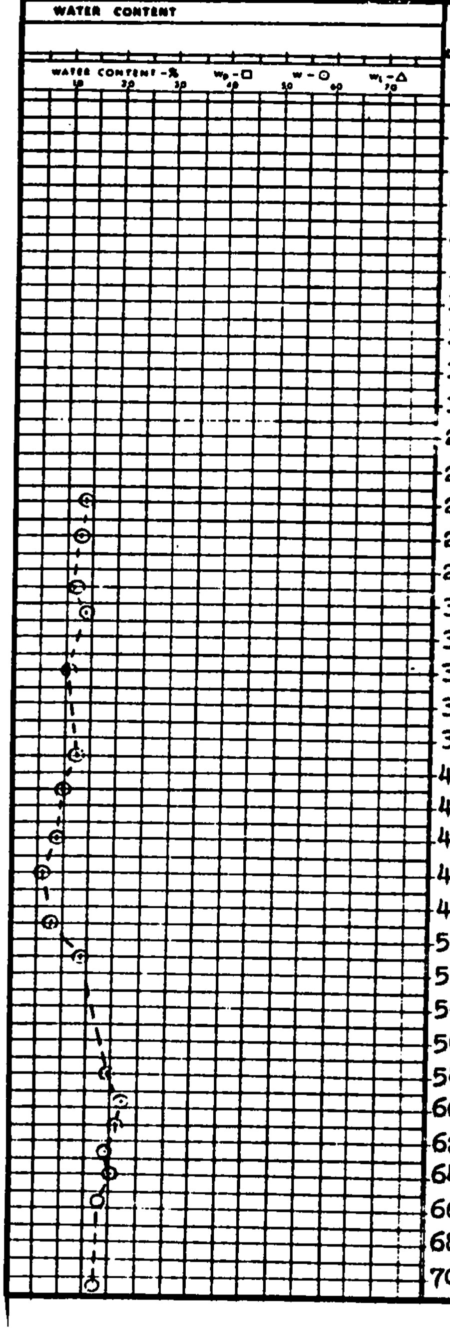
PLATE

RMH R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS
PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN G.M.	CKD.	DATE JUN 67	HOLE 6A	JOB NO. CE04	PLATE

WATER CONTENT



DEPTH FEET

SOIL PROFILE

SOIL DESCRIPTION

DATUM CITY OF EDMONTON

SURFACE ELEVATION 2230.7

DEPTH FEET	SOIL DESCRIPTION	UNIFIED CLASSN.
2 4 6 8 10 12 14 16 18	REFER TO HOLE 6	
20 22 24 26 28 30 32 34 36 38 40 42	SANDY CLAY TILL MEDIUM BROWN LOW PLASTIC LOOSE SANDY SILT HARD DARK GREY BROWN LOW PLASTIC HARD SANDY SILT CLAY ROCKS	(CL)
44 46 48 50	SAND (SASK. SAND & GRAVEL) SILTY NONE PLASTIC MEDIUM GREEN BROWN FINE GRAIN	(SM)
52 54 56 58	GRAVEL CLAYEY GRAVEL LOW PLASTIC	(GM)
60 62 64 66	CLAY SHALE (BEDROCK) HARD MOTTLED DARK GREY & BROWN MEDIUM PLASTIC COAL SPECKS	(CI)
68 70	SANDSTONE BENTONETIC HARD LIGHT CREAM GREY	

SOIL SAMPLES

TYPE	PENETRATION RESISTANCE	RECOVERY %
U1		
U2		
U3		
U4		
U5		
U6		
U7		
U8		
U9		
U10		
U11		
AS		
DS	>90	
DS	63	
U12		
RC		99
RC		14

OTHER TESTS

PLATE

B-9

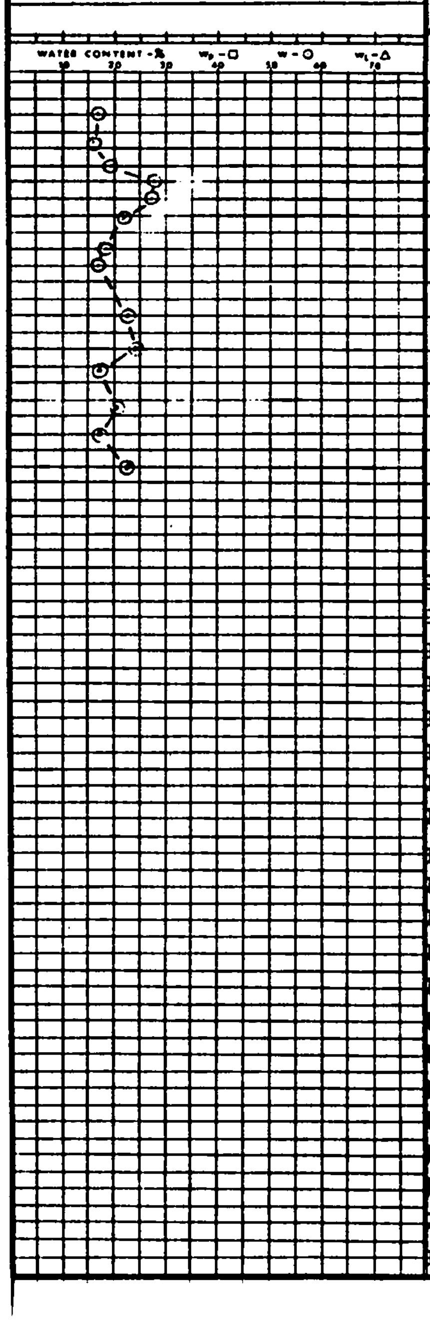
R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN. G. M.
CK'D
DATE JUN 67
HOLE 6A
JOB NO C1504
PLATE

WATER CONTENT



SOIL PROFILE

DATUM

SURFACE ELEVATION

DEPTH FEET	SOIL DESCRIPTION	SOIL SYMBOL	UNIFIED CLASSIF.	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
	ROOT LENS							
72	CLAYEY SANDSTONE MOTTLED LIGHT & DARK GREY.				RC		100	
74								
76	SHALE CARBONACEOUS COAL				RC		62	
78	BENTONITE CREAM COLOR							
80	CARBONACEOUS				RC		77	
82	COAL SPECKS DARK BROWN							
84	SHATTERED SHALE MATRIX							
86	CLAYEY ,CARBONACEOUS				RC		47	
88								
90	SANDSTONE LIGHT GREY FINE GRAIN							
92					RC		100	
94								
96	END OF HOLE							
98								
100								
102								
104								
106								
108								
110								
112								
114								
116								
118								
120								
122								
124								
126								
128								
130								
132								
134								
136								
138								
140								

SOIL SAMPLES

PLATE

RMH R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS
PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN. MSM
CK'D.
DATE 26/6/67
HOLE 9
JOB NO C-1504
PLATE

WATER CONTENT

WATER CONTENT - % 10 20 30 40 50 60 70 w_p - □ w - ○ w_l - △

SOIL PROFILE

DATUM CITY OF EDMONTON

SURFACE ELEVATION 2228.9

SOIL SAMPLES

DEPTH FEET	SOIL DESCRIPTION	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
2	FILL - clay, mottled light & dark brown, sand lenses med. plasticity					
4						
6	dark brown					
8						
10	basement floor @					
12						
14	CLAY - (TILL) - sandy, low plastic light brown rust lens coal specks					
16						
18						
20						
22	dark grey ~30% coal					
24						
26						
28	sandy					
30	stones < ¼" ϕ					
32						
34	SAND - med brown					
36	CLAY - sandy, med. plastic stones ≤ ¼" ϕ					
38	(TILL)					
40						
42						
44						
46	silty, stones ≤ ½" ϕ low plastic					
48						
50						
52						
54	SAND - [illegible] SAND & GRAVEL					
56						
58	SHALE - coal pieces weathered med. plastic - bentonitic					
60						
62						
64						
66	dark grey	☒				
68	SANDSTONE - bentonitic light grey, clayey					
70	silt stone pieces		RC		80	

PLATE

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CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN MSM
CK'D
DATE 26/6/67
HOLE [illegible]
JOB NO [illegible]
PLATE

WATER CONTENT

WATER CONTENT -% 10 20 30 40 50 60 70
w_p - □ w - ○ w_L - △

SOIL PROFILE — SOIL DESCRIPTION — SOIL SAMPLES

DEPTH FEET	DATUM / SURFACE ELEVATION — SOIL DESCRIPTION	SOIL SYMBOL	UNIFIED CLASSIF.	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
	SANDSTONE							
72	SHALE - clay, dark brown	▨						
74	grey, soft sandstone lense - bedding ~∠45°	▨			RC		112	
76	dark grey sandstone stringers soft	▨						
78		▨			RC		76	
80	hard, siltstone ½" ϕ	▨						
82	COAL carbonaceous shale	■						
84		■			RC		38	
86	CARBONACEOUS SHALE - dark brown, soft,	▨						
88	sandstone lens coal stringers	▨			RC		51	
90	COAL carbonaceous shale	■						
92	SANDSTONE	▨			RC		90	
94	SHALE - clay	▨						
96	SANDSTONE - carbonaceous shale lens	▨						
98	SHALE - CARBONACEOUS dark brown, hard	▨			RC		74	
100	CLAY SHALE - bentonite stringers	▨						
102	siltstones ½" ϕ	▨			RC		78	
104	dark grey	▨						
106	carbonaceous stringers sandstone light grey soft dark grey	▨			RC		76	
108	hard siltstone stringers	▨						
110	SANDSTONE - clay shale, stringers, green grey,	▨						
112	SHALE - clay, dark grey, soft	▨			RC		96	
114	SANDSTONE bentonitic	▨						
116	CARBONACEOUS - dark brown	▨						
118	light greenish grey	▨			RC		80	
120		▨						
122	SANDSTONE - greenish grey dark brown, hard	▨			RC		106	
124		▨						
126	END of HOLE							
128								

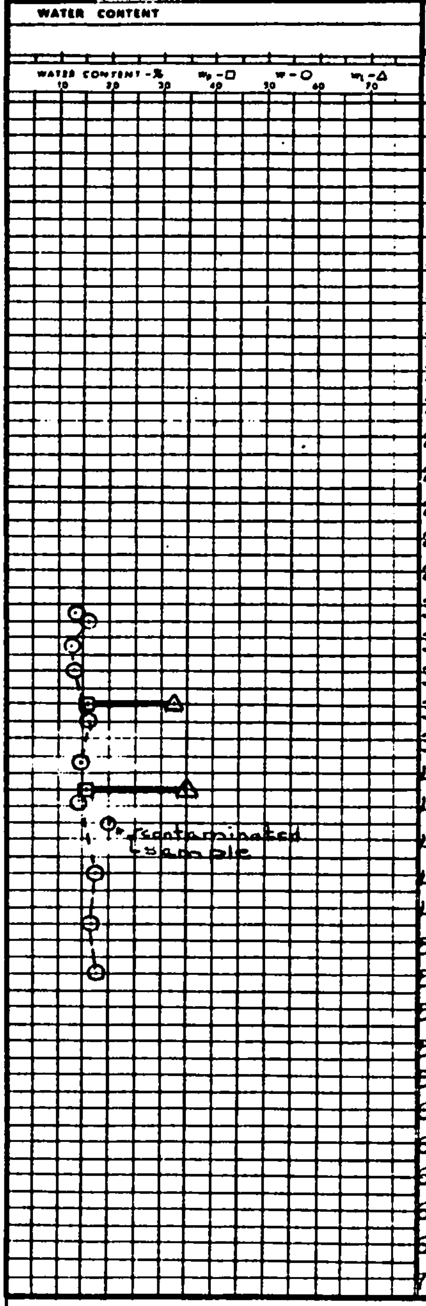
PLATE

RMH R. M HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS, A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN M.J.L. CKD RJE DATE NOV 67 HOLE 39 JOB NO S1504 PLATE



SOIL PROFILE — SOIL SAMPLES

DATUM CITY
SURFACE ELEVATION 2220.9

DEPTH FEET	SOIL DESCRIPTION	UNIFIED CLASS	TYPE	RECOVERY %	OTHER TESTS
0–2	FILL-TOPSOIL				
2–18	CLAY-MEDIUM BROWN, SILTY	(CI)			
16	SAND LENS				
20	CLAY (TILL) STONES < ½" ϕ COAL, RUST SAND LENSES				
28	LOW PLASTIC SANDY	(SC)	U1	50	6" RECOVERY
32			PT	54	26" RECOVERY
34–36	COAL, STONES < ½" ϕ MED. PLASTIC DARK GREY, THIN SAND LENSES	(CI)	PT	60	C
38–42	DARK GREY RUSTY POCKETS VERY STIFF		PT	96	- C
44–48	SAND (S. S. & G.) MED TO DRK. BROWN NON PLASTIC FINE GRAINED MEDIUM BROWN WITH DK BR. LENSES		PT	47	
50		(SM)	PT	70	
52–54	FINE TO MED. NON PLASTIC, DENSE		PT	14	
54–60	GRAVEL DARK TO MED BROWN				
60–66	CLAY SHALE DARK GREY HORIZONTAL SAND LENSES, LOW TO MEDIUM PLASTIC DENSE, CRUMBLY		PT	100	
68–70	LIGHT GREY BENTONIC		PT	95	

PLATE

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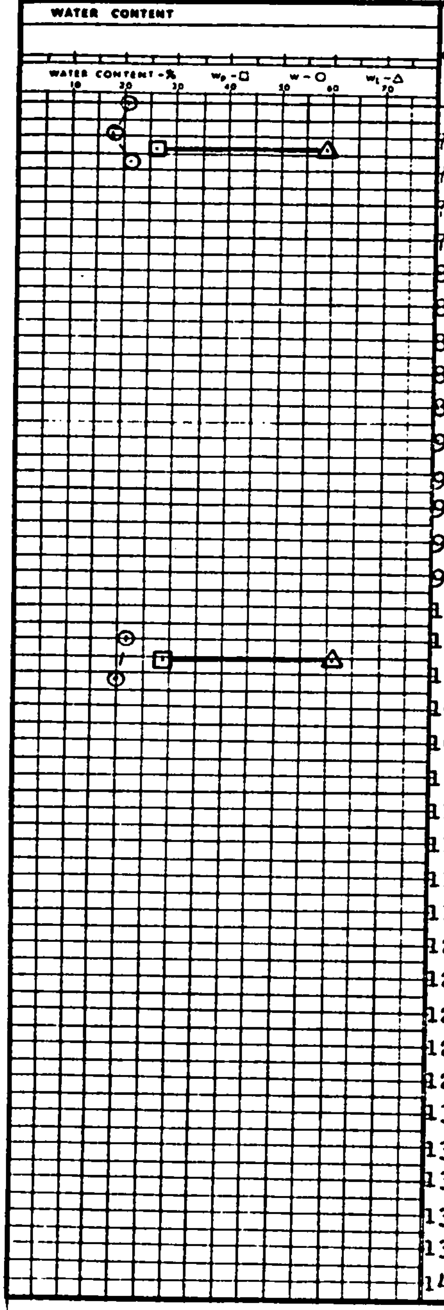
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN MJL	CKD RJB	DATE NOV 67	HOLE 39	JOB NO C1504-	PLATE

WATER CONTENT



SOIL PROFILE — SOIL SAMPLES

DEPTH FEET	SOIL DESCRIPTION (DATUM / SURFACE ELEVATION)	TYPE	RECOVERY %	OTHER TESTS
	CLAY SHALE			
72	MED. GREY, MED. TO	PT	111	C
	HIGHLY PLASTIC			
74	FRACTURED			
	½" BENTONIC STRIN.			
76	COAL	PT	86	
	CLAY SHALE			
78	SLIGHTLY CARBON.			
80	MEDIUM GREY	PT	104	
	MEDIUM PLASTIC			
82	DARK GREY			
	HIGHLY PLASTIC			
84	FRACTURED, CARBON.	PT	100	
	COAL SOFT			
86	CLAY SHALE			
	MEDIUM GREY			
88	HORIZONTAL BEDDING			
	PLANES, SILTIER	PT	60	
90	DENSE			
	SANDSTONE, SILTY			
92	FINE GRAINED			
94	LIGHT GREY, FROM	PT	82	
	90'-92' INTERBEDDED			
96	SHALE & SANDSTONE			
	WATER BEARING			
98	SAND LENSES @ 97'	PT	96	
100	CLAY SHALE			
	SANDSTONE			
102	CLAY SHALE MED			
	BROWN, MEDIUM	PT	71	C
104	PLASTIC			
106	COAL, FAIRLY			
	HARD			
108		PT	71	
	SHALE INTERBEDDED			
110	WITH COAL			
112		PT	100	
114				
	CLAY SHALE			
116	SLIGHTLY CARBON.	PT	100	
	MED. BROWN			
118	MED. GREY HOR.			
	SAND LENSES			
120	END of HOLE			
122				
124				
126				
128				
130				
132				
134				
136				
138				
140				

PLATE

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EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN M.J.L. | CKD RJB | DATE 13 NOV '67 | HOLE 41 | JOB NO 61504 | PLATE

WATER CONTENT

WATER CONTENT -% 10 20 30 40 50 60 70 w_p - □ w - ○ w_L - △

SOIL PROFILE — SOIL DESCRIPTION — DATUM — SURFACE ELEVATION

SOIL SAMPLES

DEPTH FEET	SOIL DESCRIPTION	TYPE	RECOVERY %	OTHER TESTS
	CARBONACEOUS SHALE			
72	DARK GREY	PT	88	60'-80' SLOW LOSS OF WATER
74	MEDIUM PLASTIC SANDY @ 73'			
76	CLAY SHALE SILTSTONE STRINGER	PT	44	
7	COAL			
80	CARBONACEOUS CLAY SHALE			
82	SANDY, MED PLASTIC MED. TO DRK. GREY	PT	78	SLOUGHING
84	CARBONACEOUS			
86	COAL CLAY SHALE	PT	100	
88	SANDSTONE	PT	106	
90	SANDY SHALE			
92	SLIGHTLY CARB. MOTTLED MED. GREY			
94		PT	104	
96				
98	SILTSTONE STRINGER MEDIUM GREY	PT	98	
100	SANDY, CARBON. SHALE LENSES			
102	FIRM,			
104		PT	100	
106	COAL			
10	CARBON. SHALE	PT	102	
11				
112		PT	98	
114	SANDSTONE MED. GREY			
11	CLAYEY			
11		PT	98	
12	CARBON SHALE MEDIUM BROWN			
122	SANDY	PT	107	
124	END of HOLE			
12				
12				
130				
132				
134				
136				
13				
14				

PLATE

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CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN M.J.L.
CKD
DATE NOV 67
HOLE 49
JOB NO C1504
PLATE

WATER CONTENT

WATER CONTENT -% w_p - □ w - ○ w_L - △
10 20 30 40 50 60 70

SOIL PROFILE — SOIL DESCRIPTION

DATUM CITY
SURFACE ELEVATION 2219.6

SOIL SAMPLES: SOIL SYMBOL, UNIFIED CLASS, CONDITION, TYPE, PENETRATION RESISTANCE, RECOVERY %, OTHER TESTS

DEPTH FEET	SOIL DESCRIPTION	TYPE	RECOVERY %
	CLAY SILTY		
2	SANDY, RUSTY BROWN' SAND		
4	LENSES CLAY (TILL) LENS		
6			
8			
10	CLAY - SANDY		
12	SILTY (TILL) NON-PLASTIC, STIFF		
14	STONES, SAND (TILL) AT 12' BR.		
16	TO MED GREY FINE GRAINED		
18	PEBBLES, STONES		
20			
22			
24	LIGHT GREYISH BL.		
26	CLAYEY SILTY SAND (TILL) STONE 1/2"Ø SOFT		
28			
30	GREY STONES NON PLASTIC		
32			
34	SAND (SASK. SANDS (& GRAVELS)		
36			
38			
40			
42			
44	GRAVEL		
46	CLAY SHALE		
48	MED GREY, MEDIUM PLASTIC, HORIZONTAL FRACTURES		
50			
52		PT	100
54			
56	CARBONACEOUS	PT	100
58	COAL		
60	CLAY SHALE	PT	100
62	MED GREY BROWN HIGHLY PLASTIC SLIGHTLY CARBON.		
64		PT	100
66	COAL		
68		PT	80
70	CARBON SHALE		

PLATE

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CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS, A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN M.J.L. CKD DATE NOV 67 HOLE 49 JOB NO C1504 PLATE

WATER CONTENT

WATER CONTENT - % 10 20 30 40 50 60 70 w_p - □ w - ○ w_l - △

SOIL PROFILE

SOIL SAMPLES

DEPTH FEET	SOIL DESCRIPTION DATUM SURFACE ELEVATION	SOIL SYMBOL	UNIFIED CLASSIF.	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
72	CARBON. SHALE MED GREY BROWN	▨		⧅	PT		96	
74	MED PLASTIC	▨						
76	CLAY SHALE MED GREY	▨		⧅	PT		104	
78	MEDIUM PLASTIC HORIZONTAL	▨						
80	FRACTURES.	▨		⧅	PT		104	
82	END of HOLE							
84								
86								
88								
90								
92								
94								
96								
98								
100								
102								
104								
106								
108								
110								
112								
114								
116								
118								
120								
122								
124								
126								
128								
130								
132								
134								
136								
138								
140								

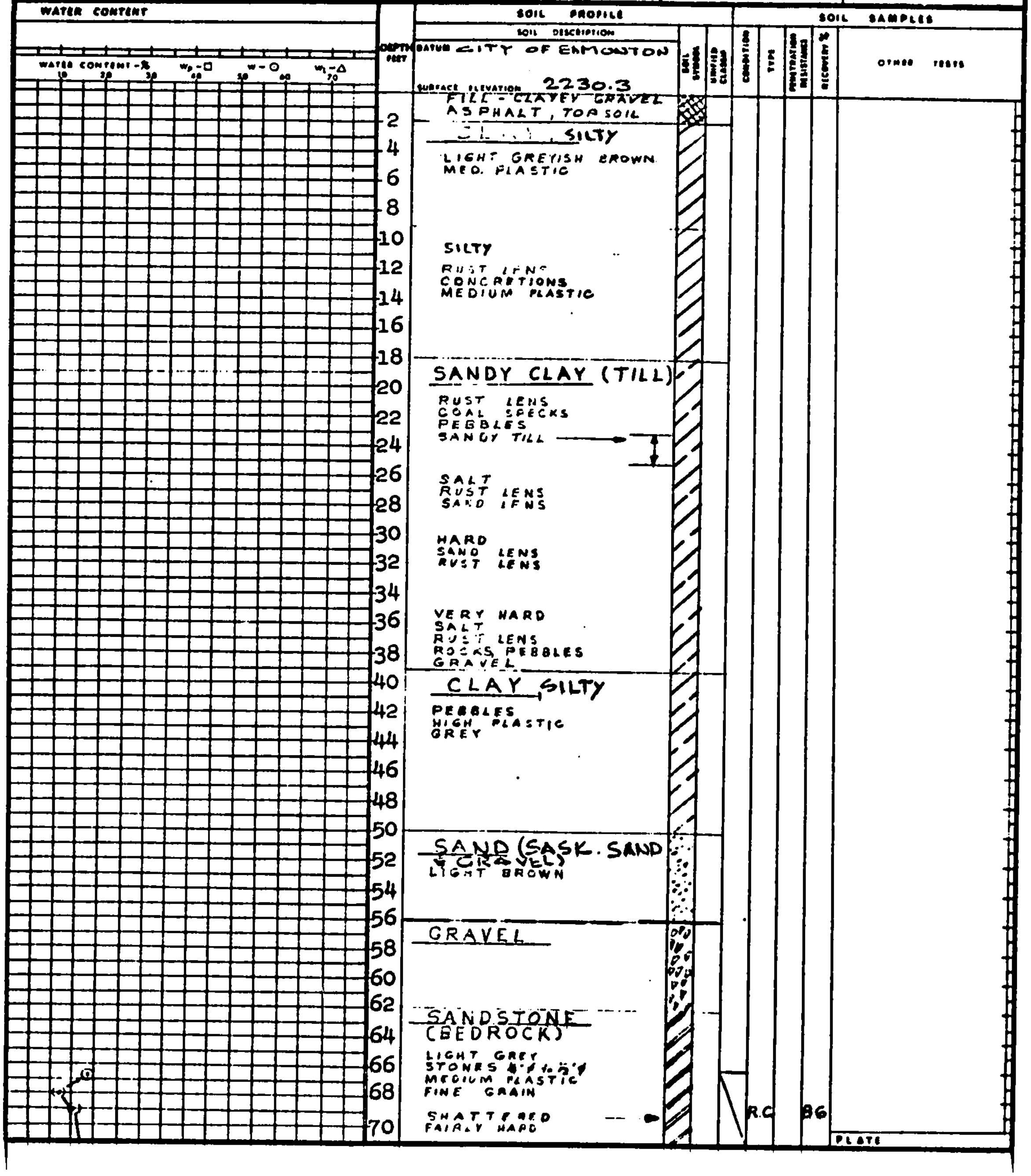
PLATE

RMH R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN. G. M	CK'D	DATE JUN 67	HOLE 68	JOB NO C1504	PLATE



DATUM CITY OF EDMONTON

SURFACE ELEVATION 2230.3

DEPTH FEET	SOIL DESCRIPTION	TYPE	RECOVERY %
0–2	FILL - CLAYEY GRAVEL ASPHALT, TOP SOIL		
2–18	[illegible], SILTY LIGHT GREYISH BROWN MED. PLASTIC SILTY RUST LENS CONCRETIONS MEDIUM PLASTIC		
18–40	SANDY CLAY (TILL) RUST LENS COAL SPECKS PEBBLES SANDY TILL → SALT RUST LENS SAND LENS HARD SAND LENS RUST LENS VERY HARD SALT RUST LENS ROCKS, PEBBLES GRAVEL		
40–50	CLAY, SILTY PEBBLES HIGH PLASTIC GREY		
50–56	SAND (SASK. SAND & GRAVEL) LIGHT BROWN		
56–62	GRAVEL		
62–70	SANDSTONE (BEDROCK) LIGHT GREY STONES [illegible] MEDIUM PLASTIC FINE GRAIN SHATTERED → FAIRLY HARD	R.C	86

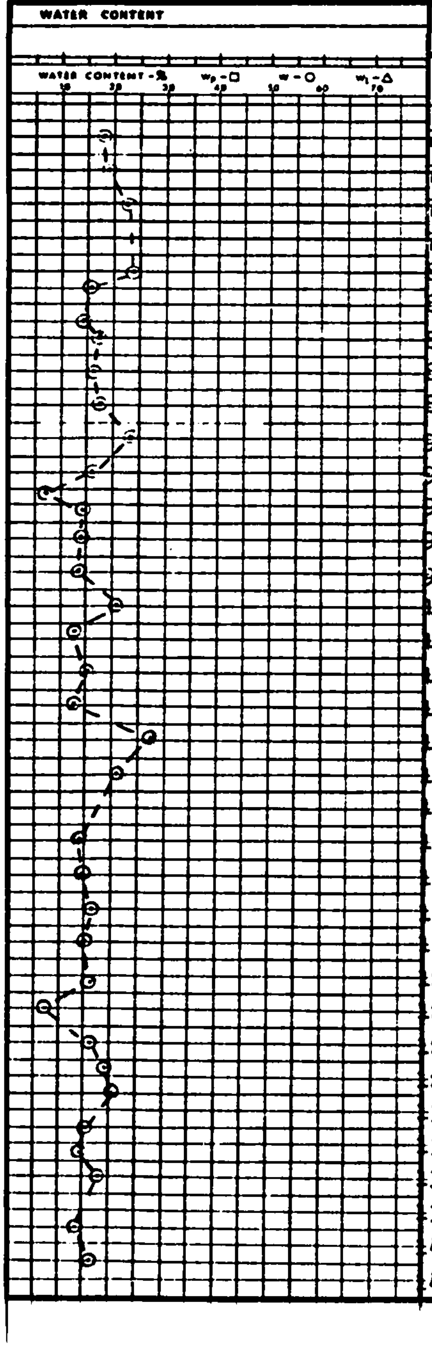
RMH R. M HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN G.M.	CK'D	DATE JUN 67	HOLE 68	JOB NO C1504	PLATE

WATER CONTENT



SOIL PROFILE

SOIL DESCRIPTION

DATUM

SURFACE ELEVATION

DEPTH FEET	SOIL DESCRIPTION
72–78	CLAYEY SANDSTONE GREY FRACTURED CARBONACEOUS SHALE BENTONITE STRINGERS COAL
78–92	CLAY SHALE SOFT COAL BENTONITE STRINGERS BENTONITIC CLAY SHALE SHATTERED HIGH PLASTIC WATER TABLE SANDSTONE STRINGERS COAL SPECKS CARBONACEOUS SHALE DARK BROWN SOFT
92–110	SANDSTONE CARBONACEOUS SHALE STRINGERS HARD LIGHT GREY FINE GRAIN SANDSTONE CARBONACEOUS SHALE STRINGERS CLAYEY SANDSTONE SHATTERED CARBONACEOUS SHALE, HARD, DARK BROWN CLAYEY SANDSTONE DARK GREY, SHATTERED SILT STONE STRINGER LIGHT BROWN CARBONACEOUS SHALE SHATTERED, DARK BROWN TO BLACK.
110–115	COAL HARD · CLEAN
115–125	CLAYEY SANDSTONE CLAYEY SANDSTONE COAL SPECKS CARBONACEOUS SHALE STRINGERS TRACE OF BENTONITE RUST LENS HIGH PLASTIC CARBONACEOUS SHALE STRINGERS CARBONACEOUS SHALE
125–142	CLAY SHALE DARK BROWN HIGH PLASTIC SILT STONE STRINGERS SOFT DARK BROWN HIGH PLASTIC HARD COAL STRINGERS TRACE OF BENTONITE CLAY SHALE COAL STRINGERS SILT STONE STRINGERS

SOIL SAMPLES

DEPTH FEET	SOIL SYMBOL	UNIFIED CLASSIF.	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
74				RC		94	
80				RC		36	
84				RC		90	
89				R.C		42	
94				RC		100	
99				RC		61	
102				R.C		57	
106				RC		46	
110				RC		92	
114				RC		66	
118				R.C		101	
122				R.C		82	
126				R.C		100	
130				R.C		80	
134				R.C		101	
137				RC		68	
142				R.C		96	

PLATE

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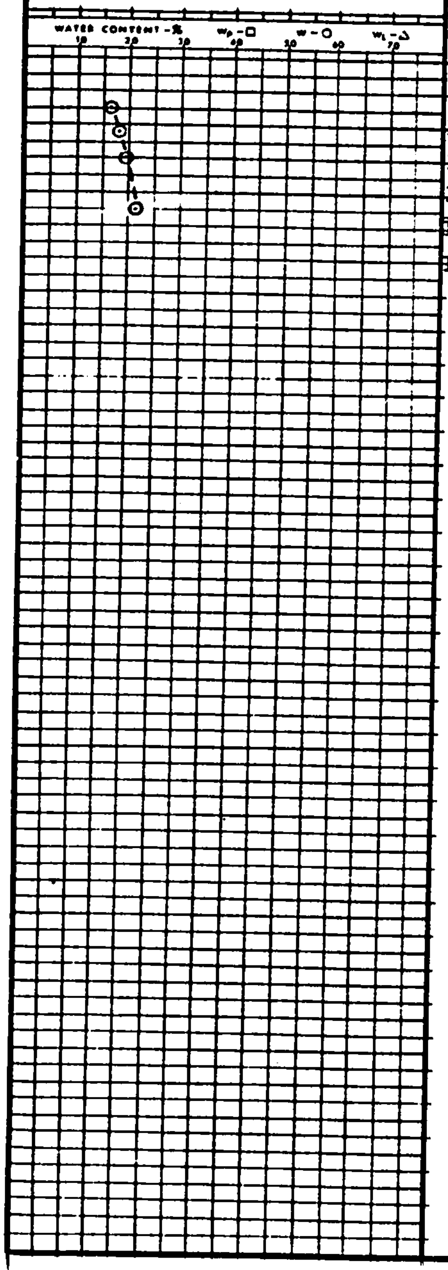
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN , FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN G.M.
CK'D
DATE JUN 67
HOLE 68
JOB NO C/504
PLATE

WATER CONTENT



SOIL PROFILE / SOIL SAMPLES

DATUM
SURFACE ELEVATION

DEPTH FEET	SOIL DESCRIPTION	SOIL SYMBOL	UNIFIED CLASS.	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
142	CARBONACEOUS SHALE							
144	DARK BROWN SANDSTONE LENS							
146	BLACK							
148	SANDSTONE STRINGERS DARK BROWN HARD				RC		96	
150	END OF HOLE							
152								

PLATE

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CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN. M.L.
CK'D.
DATE 20 FEB /68
NO.: 73
JOB NO C1504 A
PLATE

WATER CONTENT

WATER CONTENT - % 10 20 30 40 50 60 70 w_p - □ w - ○ w_L - △

SOIL PROFILE — SOIL SAMPLES

DATUM CITY

SURFACE ELEVATION 2231.0

DEPTH FEET	SOIL DESCRIPTION	SOIL SYMBOL	UNIFIED CLASS'N	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
	FILL 4" →							
2	CLAY, SILTY MEDIUM BROWN MEDIUM PLASTIC							HOLE WAS WET DRILLED AND X-LOGGED
4								
6								
8								
10								
12								
14								
16	MEDIUM GREY							
18								
20	CLAY (TILL) DARK GREY MEDIUM PLASTIC COAL SPECKS RUST LENS STONES < 1/4" ϕ							
22								
24								
26								
28								
30								
32								
34								
36	SANDY, FEW LARGE STONES							
38								
40				☒	DS	69		
42				☒	DS	66		
44				☒	DS	80		For 9"
46	SAND (S.S. & G.)			☒	DS	78		For 9"
48								
50				☒		75		For 9"
52								
54	GRAVEL			☒		82		First 6"
56								LOOSING WATER LOST ≈ 240 GLS.
58								
60	CLAY SHALE MED GREY SANDY							
62								
64								
66								
68								
70								

PLATE

RMH R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN ML
CKD
DATE 20 FEB./62
HOLE 73
JOB NO C1504 A
PLATE

WATER CONTENT: WATER CONTENT - % 10 20 30 40 50 60 70; w_p - □ w - ○ w_l - △

SOIL PROFILE: SOIL DESCRIPTION; DATUM; SURFACE ELEVATION; SOIL SYMBOL; UNIFIED CLASSIF.

SOIL SAMPLES: CONDITION; TYPE; PENETRATION RESISTANCE; RECOVERY %; OTHER TESTS

DEPTH FEET	SOIL DESCRIPTION	OTHER TESTS
72	CLAY SHALE VERY SANDY	
74		
76		
78	COAL	
80	CARBONACEOUS	
82	SHALE	
84	CLAY SHALE	
86	SILTY	
88	COAL CLAY SHALE	
90	DARK GREY	
92	SILTY	
94		
96		
98		
100		
102		
104		
106		
108		
110	COAL	
112	CARBONACEOUS SHALE	
114		
116	COAL	
118	SANDSTONE	
120	LIGHT GREY	
122		
124		
126		
128	CLAY SHALE	NOTE: TOTAL WATER LOSS FOR THIS HOLE WAS 850 GALLONS. HOLE WAS NOT CASED.
130	END of HOLE	
132		
134		
136		
138		
140		

PLATE

RMH R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN. ML
CK'D
DATE 21 FEB/68
HOLE 74
JOB NO C1504 A
PLATE

WATER CONTENT

WATER CONTENT - % w_p - □ w - ○ w_l - △

10 20 30 40 50 60 70

SOIL PROFILE

DATUM CITY

SURFACE ELEVATION 2231.7

DEPTH FEET	SOIL DESCRIPTION	SOIL SYMBOL	UNIFIED CLASS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
	FILL CLAY, SILTY BRICKS							HOLE WAS WET DRILLED AND X-LOGGED
2								
4	CLAY, SILTY MEDIUM GREY							
6								
8								
10								
12								
14	VERY SILTY GREENISH GREY							
16								
18								
20	CLAY (TILL) COAL SPECKS MEDIUM PLASTIC							
22								
24								
26								
28	SAND LENS							
30	SANDY DARK GREY COAL SPECKS RUST LENSES STONES 1"ø							
32								
34								
36								
38								
40								
42				☒	DS	65		For 8"
44	SAND (S S. & G.)			☒	DS	100		First 9"
				☒	DS	65		For 6"
46								
48								
50								
52								
54								
56	GRAVEL			▬	DS	100		First 4"
58								
60	CLAY SHALE DARK GREY							
62								
64								
66								
68								
70	SANDSTONE @ 69'							

PLATE

R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS
PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA.

DWN ML
CK'D
DATE 21 FEB/68
HOLE 74
JOB NO C1504 A
PLATE

WATER CONTENT
WATER CONTENT -% 10 20 30 40 50 60 70
w_p - □ w - ○ w_l - △

SOIL PROFILE
SOIL DESCRIPTION
DATUM
SURFACE ELEVATION

SOIL SAMPLES
SOIL SYMBOL | UNIFIED CLASS'N | CONDITION | TYPE | PENETRATION RESISTANCE | RECOVERY % | OTHER TESTS

DEPTH FEET	SOIL DESCRIPTION
	SANDSTONE
72	MED. GREY
74	
76	
78	
	CARBONACEOUS SHALE
80	
	COAL
82	CARBONACEOUS SHALE
	MED-DARK BROWN
84	CLAY SHALE
86	
88	COAL
90	CLAY SHALE
	SILTY
92	
94	
96	
98	SANDSTONE
100	
102	
104	
106	CLAY SHALE
	GREY
108	
110	COAL
112	CARBONACEOUS CLAY SHALE
114	COAL
116	
118	CLAY SHALE
	SANDY
120	DARK GREY
122	
124	BENTONITIC
126	MED. GREY
128	
130	
	END of HOLE
132	
134	
136	
138	
140	

PLATE

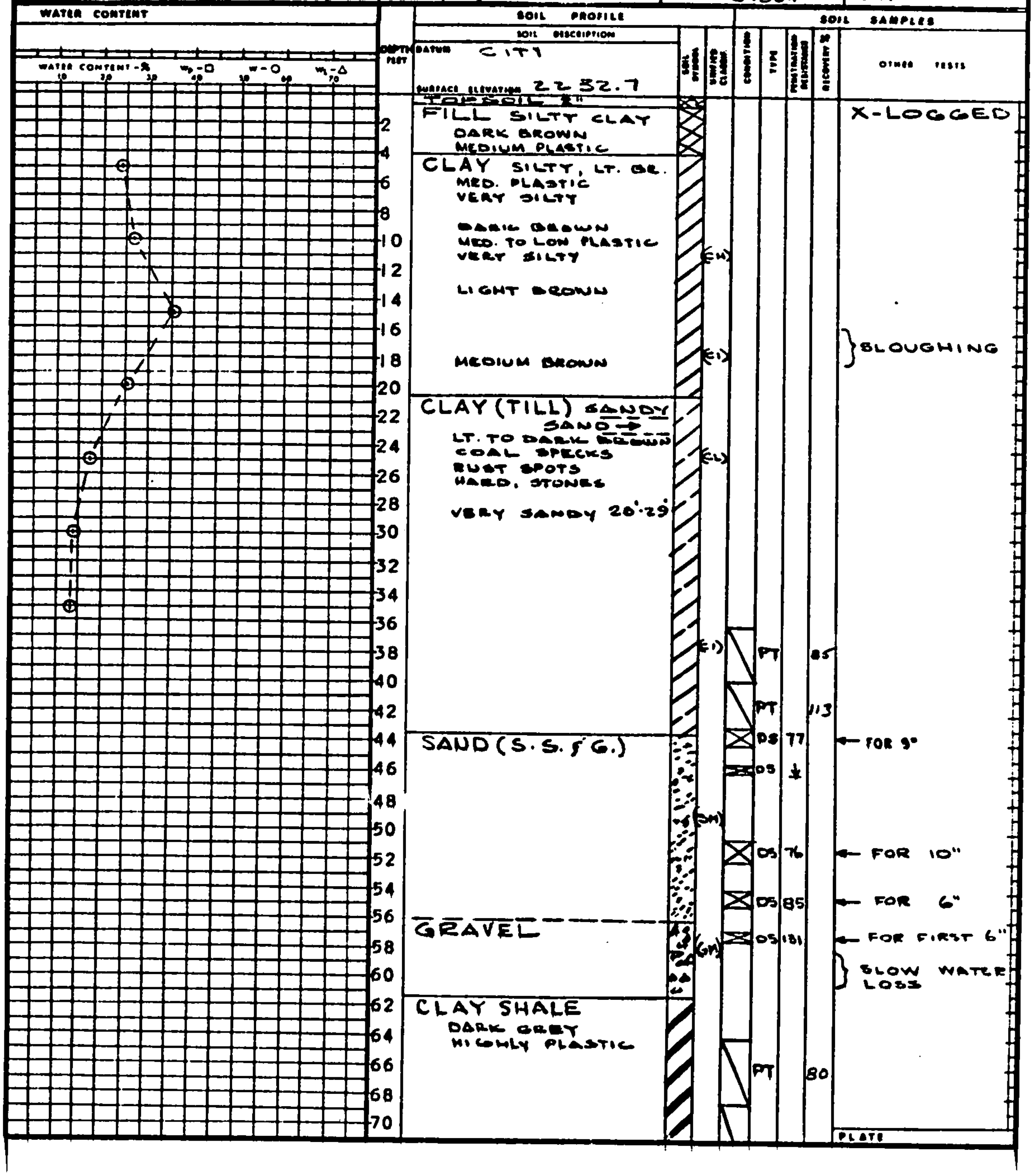
R.M HARDY & ASSOCIATES LTD
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN. M. L.	CK'D R J B	DATE 22 JAN/68	HOLE 200	JOB NO C1504	PLATE

DATUM CITY
SURFACE ELEVATION 2252.7



TOPSOIL 2"

FILL SILTY CLAY
DARK BROWN
MEDIUM PLASTIC

CLAY SILTY, LT. BR.
MED. PLASTIC
VERY SILTY

DARK BROWN
MED. TO LOW PLASTIC
VERY SILTY

LIGHT BROWN

MEDIUM BROWN

CLAY (TILL) SANDY
SAND →
LT. TO DARK BROWN
COAL SPECKS
RUST SPOTS
HARD, STONES

VERY SANDY 28'-29'

SAND (S.S. & G.)

GRAVEL

CLAY SHALE
DARK GREY
HIGHLY PLASTIC

TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
			X-LOGGED
			} SLOUGHING
PT		85	
PT		113	
DS	77		← FOR 9"
DS	⊥		
DS	76		← FOR 10"
DS	85		← FOR 6"
DS	131		← FOR FIRST 6"
			} SLOW WATER LOSS
PT		80	

R.M.H.
R. M. HARDY & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS
EDMONTON ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT RULE, WYNN, FORBES, LORD & PARTNERS
A.G.T. OXFORD COMPLEX, EDMONTON, ALBERTA

DWN G. M.
CK'D R JE
DATE 22JAN'69
HOLE 200
JOB NO C1504
PLATE

WATER CONTENT

WATER CONTENT - % w_p - □ w - ○ w_L - △
10 20 30 40 50 60 70

SOIL PROFILE — SOIL SAMPLES

DATUM
SURFACE ELEVATION

DEPTH FEET	SOIL DESCRIPTION	SOIL SYMBOL	UNIFIED CLASSIF.	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY %	OTHER TESTS
	CLAY SHALE							
72	MED GREY				PT		106	
74								
76					PT		64	
78	BENTONITE COAL							
80	CARBONACEOUS HIGH PLASTIC				PT		93	
82	MED GREY							
84	FRACTURED				PT		67	
86	COAL							
88	CLAY SHALE				PT		38	
90								
92								
94	SILT STONE				PT		111	
96	SANDSTONE							
98	WELL CEMENTED LT GREY				PT		109	
00								
02	CLAY SHALE MED GREY				PT		100	
04	MED PLASTIC							
06	CARBONACEOUS				PT		102	
08	COAL							
10	CARBONACEOUS DK GREY							
12	COAL				PT		93	
14	CARBONACEOUS							
16	SANDSTONE				PT		100	
18	LT GREY							
20	SILTY, MED. GRAIN				PT		104	
22								
24	CLAY SHALE MED GREY				PT		104	
26	MED PLASTIC							
28					PT		89	
30	SANDSTONE LENSES							
32	END of HOLE							
34								
36								
38								
40								

PLATE

APPENDIX C

INTERACTION CHART - RESULTS OF THE STRUCTURAL ANALYSIS OF THE CN TOWER

INCREMENTAL PROCEDURE

Date	Dec. 10, 1964	Mar. 23, 1965	May 20, 1965	May 27, 1965	July 6, 1965	Aug. 16, 1965	Oct. 28, 1965	Total Load Transfer at End of Construction	Total Load Transfer as Percent of Basic Loads	Feb. 25, 1970	Total Load Transfer to Feb. 25, 1970	Total Load Transfer as Percent of Basic Loads
No. of Storeys	Ground Floor	3	10	11	16	21	26 + Roof			26 + Roof		
Trial No.	1	2	3	4	5	6	7			8		
Column	kips	kips	kips	kips	kips	kips	kips	kips	%	kips	kips	%
1	-413	+340	+591	+345	-311	-462	+898	+1016	+20.0	-1260	-244	-5.0
2	+473	-376	+67	-1401	-642	-2858	-1170	-271	-3.6	+285	+14	+0.2
3	+96	-206	-158	+812	-258	-381	-502	-597	-8.0	-1674	-2271	-30.4
4	-190	-144	+793	+390	+252	-151	-260	+690	+14.1	+1238	+1928	+39.4
5	+8	+110	-523	+131	-301	+212	+64	-299	-5.7	-738	-1037	-19.9
6	+10	+250	+598	-1139	+692	-2769	+382	-1976	-44.5	+2621	+645	+14.5
7	+24	+145	-1095	+891	+685	+161	+250	+1161	+26.1	-1048	+121	+2.7
8	+10	-143	-398	+288	+281	-161	-35	-158	-3.0	+1053	+895	+17.2
9	+6	+21	+232	-363	+6	+201	-293	-190	-3.4	+45	-145	-2.6
10	-16	-38	+84	-225	-103	+162	+281	+65	+1.2	+1113	+1178	+21.1
11	+34	+34	-171	+92	+359	-800	+138	-304	-5.6	-106	-410	-7.5
12	-61	-49	+171	+5	-470	+747	+236	+579	+11.1	-788	-201	-3.9
13	-12	+52	-194	-78	+32	+173	-844	-671	-14.9	+280	-391	-8.7
14	-44	+32	-46	+135	-280	-2	-434	-639	-14.2	-452	-1091	-24.3
15	+2	-25	+134	+33	-160	+527	-1055	-554	-10.7	-198	-752	-14.5
16	+4	+14	-185	-77	+49	+44	+632	+561	+13.1	+300	+861	+20.1
17	+28	+36	-117	+30	+396	-661	+503	+207	+3.7	-879	-672	-11.9
18	+12	-3	+24	+60	-25	-246	-91	-249	-5.4	+381	+132	+2.0
19	-31	-56	+227	+26	+101	+124	-54	+337	+7.3	+460	+797	+17.2
20	+57	+31	-185	+40	+180	-238	+27	-88	-2.1	+273	+105	+4.3
21 Core	+12	-51	+70	-39	-393	+662	+1125	+1386	+4.0	-914	+472	+1.6

NON-INCREMENTAL PROCEDURE

Date	Oct. 28, 1965	Total Load Transfer as Percent of Basic Loads	Feb. 25, 1970	Total Load Transfer as Percent of Basic Loads
No. of Storeys	26 + Roof		26 + Roof	
Trial No.	7		8	
Column	kips	%	kips	%
1	-1028	-21.1	-2288	-46.9
2	+1953	+26.1	+2237	+29.9
3	-2666	-35.7	-4339	-58.1
4	-887	-18.2	+354	+7.3
5	+1526	+29.3	+761	+14.6
6	-1801	-40.5	+817	+18.4
7	+2533	+56.9	+1493	+33.6
8	+692	+13.3	+1745	+33.5
9	+36	+0.6	+82	+1.5
10	-1304	-23.3	-102	-1.8
11	+392	+7.2	+282	+5.2
12	-545	-10.5	-1318	-25.3
13	-923	-20.5	-640	-14.2
14	-1286	-28.6	-1839	-40.9
15	-560	-10.8	-64	-1.2
16	+552	+12.9	+537	+12.5
17	+1265	+22.4	+6	+0.1
18	+275	+5.9	+679	+14.6
19	-964	-20.8	-506	-10.9
20	+1412	+32.9	+1683	+39.3
21 Core	+1332	+4.6	+419	+1.5

APPENDIX D

BENCHMARK CONTROL AND AGT TOWER SETTLEMENT DATA

Benchmark Control

AGT-Oxford Complex

Date	Elevation Difference cm.
Sept. 10, 1968	40.7
Oct. 18, 1968	40.3
Nov. 12, 1968	40.6
Mar. 4, 1969	40.5
Apr. 9, 1969	40.7
May 8, 1969	40.6
June 17, 1969	40.7
Nov. 4, 1969	40.8

DBM #1 is higher than BM #3 by these elevation differences - Estimated survey accuracy in this traverse is ± 0.2 cm.

Date	Elevation Difference ft.
June 23, 1969	37.576
July 26, 1969	37.572
Aug. 18, 1969	37.556
Oct. 25, 1969	37.580
Mar. 16, 1970	37.578

DBM #2 is lower than DBM #1 by these elevation differences - Estimated survey accuracy in this traverse is ± 0.010 ft.

Data Obtained Subsequent to Construction of the 29th Floor - AGT Tower

Exterior Columns	Date Oct. 15,70 (764) Floor 32		Date Floor		Date Floor		Date Floor	
	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.
31	6.74	1.40						
32	6.90	1.57						
33	6.83	1.56						
34	6.98	1.75						
36	6.90	1.91						
37	7.02	1.98						
41	5.22	1.00						
42	7.66	1.43						
43	8.41	1.58						
44	6.29	1.57						
45	6.82	1.65						
46	6.28	1.70						

Exterior Columns	Date Oct 15, 70 Floor 32		Date Floor		Date Floor		Date Floor	
	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.
47	8.41	1.57						
48	6.94	1.34						
49	6.98	1.47						
50	7.15	1.67						
51	7.30	1.69						
52	7.23	1.86						
53	7.23	1.91						
54	7.37	2.09						
55	7.22	1.83						
56	7.29	1.71						
57	7.43	1.49						
58	7.18	Covered						
60	5.61	0.89						
69	7.10	1.44						
70	7.07	1.57						

Exterior Columns	Date Oct. 15, 70 Floor 32		Date Floor		Date Floor		Date Floor	
	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.
71	7.07	1.75						
73	7.12	1.27						
74	7.02	1.16						
75	5.85	0.78						
76	7.39	1.01						
77	8.29	1.20						
78	6.42	1.51						
79	6.73	1.69						
80	6.43	1.65						
81	8.48	1.51						
82	7.83	1.36						
83	5.60	0.92						
35	7.08	1.74						

Core Plugs	Date Oct. 15, 70 Floor 32		Date Floor		Date Floor		Date Floor	
	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.	Press ksf.	Settlement cm.
38	7.32	3.22						
39	7.31	3.22						
40	7.32	3.30						
62	8.37	3.55						
64	8.42	Missed						
65	8.37	3.44						
66	8.37	3.37						
67	7.48	3.37						
68	7.48	3.53						
72	7.48	3.41						
85	8.14	3.58						
86	8.14	3.57						
87	8.14	3.33						
88	8.15	3.59						
90	......	Covered						

Derived Foundation Modulus and Standard Deviation - AGT Tower

	Date Oct. 15, 70 Floor 32		Date Floor		Date Floor		Date Floor	
	Derived Modulus	Standard Deviation	Derived Modulus	Standard Deviation	Derived Modulus	Standard Deviation	Derived Modulus	Standard Deviation
Exterior Columns	38,700	4,560						
Interior Core	29,100	1,520						

Day 714 to Day 764

APPENDIX E

ELASTIC SETTLEMENTS OF UNIFORMLY LOADED RECTANGULAR FOOTINGS ON A HOMOGENEOUS, ISOTROPIC, SEMI-INFINITE BODY

Solution Description

The solution presented here is based on the standard equations of elasticity for the settlement of a corner of a rectangular, flexible, uniformly loaded area. The soil medium is considered to be a homogeneous, isotropic, semi-infinite mass. Superposition techniques are used to enable computation of settlements at footing centroids due to multiple footing foundation systems.

The settlement of a specific point k due to N rectangular areas, each with a different uniform load, is given by the equation

$$\Delta_k = \frac{(1-\mu^2)}{E} \sum_{j=1}^{j=N} q_j \sum_{i=1}^{i=4} B_{ijk} I_{\Delta ijk} \qquad 1$$

where Δ_k = Settlement of point k

μ = Poisson's ratio

E = Young's modulus

q_j = Contact pressure of the jth footing

B_{ijk} = Short dimension of a rectangular area

$I_{\Delta ijk}$ = Influence value which is a function of the ratio of the long to short dimensions of a rectangular area.

The expression

$$\sum_{i=1}^{i=4} B_{ijk} I_{\Delta ijk} \qquad 2$$

represents the contribution of footing j to the settlement at point k. Repetition of four such calculations are necessary for superposition.

The influence value for the settlement of a corner of a single rectangle is given by

$$I_{\Delta} = \frac{1}{\pi} \{\ell \ln \frac{1 + \sqrt{\ell^2+1}}{\ell} + \ln(\ell + \sqrt{\ell^2+1})\} \qquad 3$$

where ℓ denotes the ratio of length to width, $\frac{L}{B}$, for the rectangle.

This computer program calculates settlements for an increment of footing load which is determined by subtraction of two sets of input load data. The order of footing identification numbers must be identical. Settlements are computed at the centroids of all input footings. If settlements are desired at other locations, these are specified as "dummy" footings which have a precise coordinate location but have unit length and width and zero load. These "dummy" footings are included at the end of the second data set.

The location of the coordinate axes is arbitrary. To avoid negative and positive reference coordinates it is usually convenient to place the X and Y axes outside the footing system. The only restriction is that the orientation of the axes must be such, that each footing side is parallel to an axis. In other words, the center lines of each footing must be either parallel or perpendicular to a particular axis.

Settlements are computed for a specified variation of Young's modulus and Poisson's ratio and are presented in tabular form for each footing centroid. The variation of the above elastic parameters is defined with three numbers. For example, settlements may be computed for a series of foundation moduli which starts at a particular value and is incremented by a particular value a specified number of times.

Operational Characteristics

- Maximum number of footings for each data set is 90.
- Maximum number of moduli for which settlements may be given is 15.
- Required computer storage = 50 K.

Input

Card (A) Variation of Elastic Parameters.

(F10.0,2X,F8.0,7X,I3,5X,F5.3,5X,F5.3,7X,I3)

Columns	
1-10	Young's modulus in psi at which computations start
13-20	Increment of Young's modulus in psi
28-30	Number of increments $\leq$ 15
36-40	Poisson's ratio at which computations start
46-50	Increment of Poisson's ratio
58-60	Number of increments - quantity not limited.

Card (B) Title Card of First Load Set.

(17A4)

Card (C) Number of Footings in First Load Set.

(2X,I3)

Columns

3-5 Number of footings in first load set.

Card (D) Footing Load Cards - must equal number of footings specified on Card C.

(I3,29X,F7.1) - note - format of second load set can also be used.

Columns

1-3 Footing identification no. - can be in any order

33-39 Footing load in kips.

Card (E) Title Card of Second Load Set.

(17A4)

Card (F) Number of Footings in Second Load Set.

(2X,I3)

Columns

3-5 Number of footings defined in second load set - must be equal or greater than number of footings at first load set specified on Card C.

Card (G) Footing Load Cards - must equal number of footings specified on Card F.

(I3,1X,5F7.1)

Columns

1-3	Footing identification no.
5-11	X - coordinate of footing in feet
12-18	Y - coordinate of footing in feet
19-25	Length of footing in x-direction in feet
26-32	Length of footing in y-direction in feet
33-39	Footing load in kips.

<u>Note</u> - The footing identification numbers of the second load set must be in same order to those specified in the first load set. The "dummy" footings which are added at the end can be in any order.

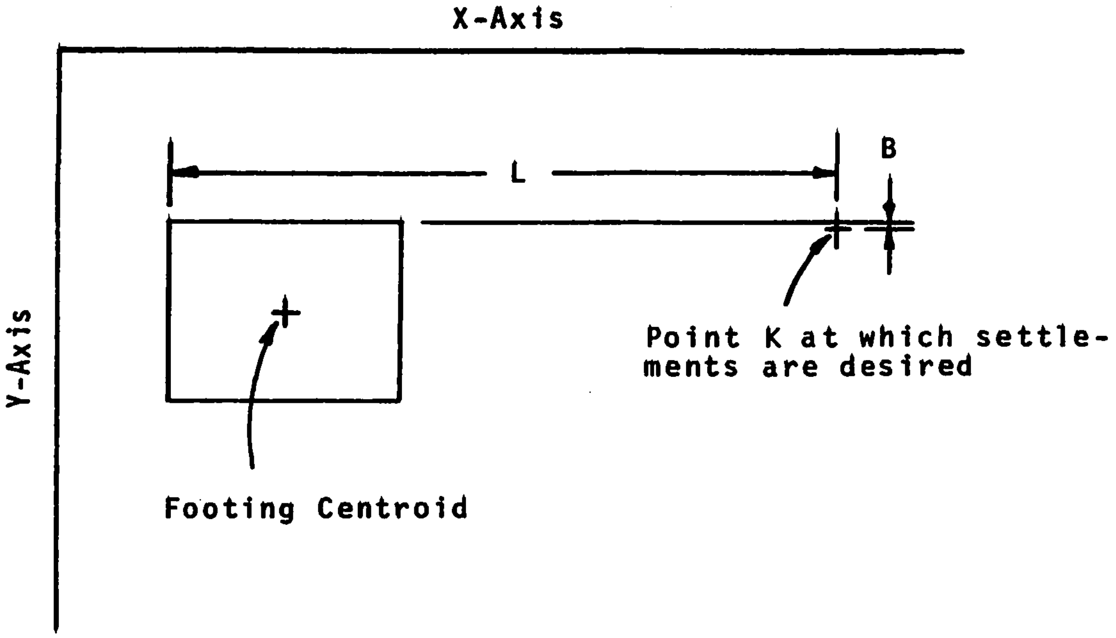
<u>Output</u>

Input data is printed out immediately following read in settlements.

Settlements are printed out in tabular form for each footing specified.

<u>Additional Remarks</u>

(A) Because of the manner in which the influence value is computed error messages may occasionally be given indicating that division by zero has been attempted. This results with footing geometry shown below.



Note that the distance B is very small which when divided into L results in the above error message. If such a condition is observed a slight shift in position of point K, which will make B larger, solves the problem.

(B) The increment of footing load must be either positive for all footings or negative for all footings. Therefore if the first load set has larger footing loads than the second load set, the increments of computed loads must all be negative.

If for some reason a footing is subjected to less load at some later date, the statement card with address 46 must be changed to read 46 PINC=P(J)-PI(J). The second load set must have the larger footing loads for this special case in order to avoid output of negative settlements.

COMPUTER PROGRAM LISTING

```
G LEVEL 1, MOD 4             MAIN                DATE = 70310          00/23/15
C
C     ****************************************************************
C     ELASTIC SETTLEMENT OF RECTANGULAR FOOTINGS ON A SEMI-INFINITE
C     PLANE -REQUIRES- TWO LOAD SETS AND SET OF SPECIFICATIONS FOR
C     VARIATION OF YOUNGS MODULUS AND POISSONS RATIO: J. DEJONG
C     ****************************************************************
C
      DIMENSION IR(90),X(90),Y(90),A(90),B(90),P(90),Q(90),SETI(90),QIN(
     190),EV(30),TITLE(17),TLE(17),II(60),PI(60)
C
C     ****************************************************************
C     READ AND PRINT DATA
C     ****************************************************************
C
      READ(5,12)ELAS,DELAS,NEL,PR,DPR,NPR
      READ(5,40)(TITLE(I),I=1,17)
      READ(5,10)K
      READ(5,9)(II(J),PI(J),J=1,K)
      READ(5,40)(TLE(I),I=1,17)
      READ(5,10)N
      READ(5,11)(IR(J),X(J),Y(J),A(J),B(J),P(J),J=1,N)
      WRITE(6,41)
      WRITE(6,42)(TITLE(J),J=1,17)
      WRITE(6,43)
      WRITE(6,44)(II(J),A(J),B(J),X(J),Y(J),PI(J),J=1,K)
      WRITE(6,37)
      WRITE(6,45)(TLE(J),J=1,17)
      WRITE(6,43)
      WRITE(6,44)(IR(J),A(J),B(J),X(J),Y(J),P(J),J=1,N)
      WRITE(6,37)
    9 FORMAT(I3,29X,F7.1)
   10 FORMAT(2X,I3)
   11 FORMAT(I3,1X,5F7.1)
   12 FORMAT(F10.0,2X,F8.0,7X,I3,5X,F5.3,5X,F5.3,7X,I3)
   37 FORMAT(31X,'NOTE - ALL UNITS ARE IN TERMS OF KIPS AND FEET',/)
   40 FORMAT(17A4)
   41 FORMAT(1H1,20X,'FOOTING SETTLEMENT ON ELASTIC SOLID',//)
   42 FORMAT(23X,17A4,/)
   43 FORMAT(25X,'FTG. LENGTH  WIDTH  X COORD. Y COORD.  LOAD',/)
   44 FORMAT(24X,I3,1X,F7.1,1X,F7.1,1X,F7.1,2X,F7.1,2X,F7.1)
   45 FORMAT(//,23X,17A4,/)
C
C     ****************************************************************
C     CHECK ORDER OF FOOTINGS AND COMPUTE INCREMENT OF FOOTING CONTACT
C     PRESSURE.
C     ****************************************************************
C
      WRITE(6,49)
      DO 30 J=1,K
      IF(II(J).EQ.IR(J)) GO TO 46
      WRITE(6,47)
      CALL EXIT
   46 PINC=ABS(P(J)-PI(J))
      Q(J)=PINC/(A(J)*B(J))
      WRITE(6,48)II(J),PINC,Q(J)
```

```
    30 CONTINUE
       WRITE(6,37)
    47 FORMAT(10X,'FOOTING IDENTIFICATION NUMBERS DO NOT MATCH.',/,10X,'C
      1ARDS ARE NOT IN ORDER.')
    48 FORMAT(24X,I3,2X,F7.1,3X,F5.2)
    49 FORMAT(//,23X,'LOADING INCREMENT',//,25X,'FTG.  INCR.   PRESS.',/)
C
C      ******************************************************************
C      COMPUTE SETTLEMENT MATRIX (INCLUDE Q(I) IN MATRIX)
C      ******************************************************************
C
       DO 29 J=1,N
       QIN(J)=0.0
    29 CONTINUE
       DO32I=1,K
       RR=A(I)/2.0
       SS=B(I)/2.0
       DO31J=1,N
       DA=ABS(X(I)-X(J))
       DB=ABS(Y(I)-Y(J))
       FI=FE (RR,SS,DA,DB)
       QIN(J)=QIN(J)+(Q(I)/0.012)*FI
    31 CONTINUE
    32 CONTINUE
C
C      ******************************************************************
C      COMPUTE SETTLEMENT FOR COMPUTED VARIATION OF ELASTIC PARAMETERS
C      ******************************************************************
C
       DO 22 L=1,N
       WRITE(6,23)IR(L)
       DO 20 J=1,NPR
       CD=J-1
       POR=PR+CD*DPR
       DO 21 I=1,NEL
       CEL=I-1
       EV(I)=ELAS+CEL*DELAS
       SETI(I)=QIN(L)*((1.0-POR**2)/EV(I))
    21 CONTINUE
       IF(J.NE.1) GO TO 25
       WRITF(6,24)(EV(I),I=1,NEL)
    25 WRITE(6,26)POR,(SETI(I),I=1,NEL)
    20 CONTINUE
       WRITE(6,38)
    22 CONTINUE
    23 FORMAT(//,1X,'SETTLEMENT POINT',I4,//,1X,'POIS. RATIO',30X,'MODULI
      1')
    24 FORMAT(11X,15(1X,F7.0))
    26 FORMAT(3X,F5.2,2X,15(3X,F5.3))
    38 FORMAT(7X,'NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES')
       STOP
       END
```

```
G LEVEL 1, MOD 4              FE                  DATE = 70310            00/23/15
      FUNCTION FE (X,Y,A,B)
C
C     ***************************************************************
C     CALCULATE INFLUENCE COEFFICIENT COMPONENTS
C     ***************************************************************
C
      DIMENSIONRF(4)
      DO7M=1,4
      GOTO(1,2,3,4),M
    1 W=A+X
      V=B+Y
      GOTO5
    2 W=ABS(A-X)
      GOTO5
    3 V=ABS(B-Y)
      GOTO5
    4 W=A+X
    5 IF(W.EQ.0.0.OR.V.EQ.0.0)GOTO12
      IF(W.LT.V)GOTO6
      R=W/V
      RF(M)=V*FG (R)
      GOTO7
    6 R=V/W
      RF(M)=W*FG (R)
      GOTO7
   12 RF(M)=0.0
    7 CONTINUE
      IF(A.LE.X.AND.B.LE.Y)GOTO8
      IF(A.LE.X)GOTO9
      IF(B.LE.Y)GOTO10
C     CASE 1
      FE=RF(1)-RF(2)+RF(3)-RF(4)
      GOTO11
C     CASE 2
    9 FE=RF(1)+RF(2)-RF(3)-RF(4)
      GOTO11
C     CASE 3
   10 FE=RF(1)-RF(2)-RF(3)+RF(4)
      GOTO11
C     CASE 4
    8 FE=RF(1)+RF(2)+RF(3)+RF(4)
   11 RETURN
      END
```

```
/ G LEVEL 1, MOD 4            FG                  DATE = 70310            00/23/15
      FUNCTION FG (R)
C
C     ***************************************************************
C     ARITHMETIC FUNCTION
C     ***************************************************************
C
      A=SQRT(R**2+1.0)
      B=R*ALOG((1.0+A)/R)
      C=ALOG(R+A)
      FG=(B+C)/3.1416
      RETURN
      END
```

TYPICAL RESULTS

```
SETTLEMENT POINT  75

POIS. RATIO                                MODULI
             10000.  20000.  30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000.
   0.30      2.750   1.375   0.917   0.687   0.550   0.458   0.393   0.344   0.306   0.275   0.250   0.229   0.212   0.196   0.183
   0.40      2.530   1.269   0.846   0.635   0.508   0.423   0.363   0.317   0.282   0.254   0.231   0.212   0.195   0.181   0.169
   0.50      2.266   1.133   0.755   0.567   0.453   0.378   0.324   0.283   0.252   0.227   0.206   0.189   0.174   0.162   0.151
     NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES

SETTLEMENT POINT  76

POIS. RATIO                                MODULI
             10000.  20000.  30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000.
   0.30      2.971   1.465   0.990   0.743   0.594   0.495   0.424   0.371   0.330   0.297   0.270   0.248   0.229   0.212   0.199
   0.40      2.742   1.371   0.914   0.686   0.548   0.457   0.392   0.343   0.305   0.274   0.249   0.229   0.211   0.196   0.183
   0.50      2.448   1.224   0.816   0.612   0.490   0.408   0.350   0.306   0.272   0.245   0.223   0.204   0.198   0.175   0.163
     NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES

SETTLEMENT POINT  77

POIS. RATIO                                MODULI
             10000.  20000.  30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000.
   0.30      3.150   1.575   1.050   0.788   0.630   0.525   0.450   0.394   0.350   0.315   0.286   0.263   0.242   0.225   0.210
   0.40      2.908   1.454   0.969   0.727   0.582   0.485   0.415   0.363   0.323   0.291   0.264   0.242   0.224   0.208   0.194
   0.50      2.596   1.298   0.865   0.649   0.519   0.433   0.371   0.325   0.288   0.260   0.236   0.216   0.200   0.185   0.173
     NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES

SETTLEMENT POINT  78

POIS. RATIO                                MODULI
             10000.  20000.  30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000.
   0.30      3.157   1.578   1.052   0.789   0.631   0.526   0.451   0.395   0.351   0.316   0.287   0.263   0.243   0.225   0.210
   0.40      2.914   1.457   0.971   0.728   0.583   0.486   0.416   0.364   0.324   0.291   0.265   0.243   0.224   0.208   0.194
   0.50      2.602   1.301   0.867   0.650   0.520   0.434   0.372   0.325   0.289   0.260   0.237   0.217   0.200   0.186   0.173
     NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES

SETTLEMENT POINT  79

POIS. RATIO                                MODULI
             10000.  20000.  30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000.
   0.30      3.198   1.599   1.066   0.800   0.640   0.533   0.457   0.400   0.355   0.320   0.291   0.267   0.246   0.228   0.213
   0.40      2.952   1.476   0.984   0.738   0.590   0.492   0.422   0.369   0.328   0.295   0.268   0.246   0.227   0.211   0.197
   0.50      2.636   1.318   0.879   0.659   0.527   0.439   0.377   0.329   0.293   0.264   0.240   0.220   0.203   0.188   0.176
     NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES
```

APPENDIX F

SOIL STRUCTURE INTERACTION COMPUTER PROGRAM "INCREMENTAL" AND "CONVENTIONAL" SOLUTIONS

Description

The computer solution given here solves for the support settlements of a three-dimensional space frame taking into account the influence of structural rigidity. Since footing settlements are computed using elastic theory, including the effect of load dispersion in the foundation, settlements during construction will be immediate as will be the differential displacements of the frame's supports. The rigidity of the frame will therefore not be constant but will vary with storey height. Distortions of the structural frame are assumed to take place elastically.

This method is based on compatibility requirements which dictate that foundation settlements must equal the support displacements of the structural frame. As well, the vertical footing loads must equal the vertical reactions at the frame's supports. To account for the influence of variable site conditions, an approximation is used for calculating foundation settlements whenever different foundation deformation parameters are specified.

The "incremental" solution given here takes into account the variation of structural rigidity during construction. The "conventional" solution is also given and assumes the entire building is distorted immediately upon completion of construction. No support settlements are assumed to occur during this period.

Support settlements can be expressed in matrix form as

$$[\Delta] = [M][R] \qquad 1$$

where $[\Delta]$ denotes the column matrix of support settlements

$[M]$ denotes a square matrix which includes the elastic foundation parameters, footing areas, the influence of load dispersion, and the approximation for variable site conditions

$[R]$ denotes a column matrix of resultant footing loads.

For an arbitrary frame distortion the influence of structural rigidity is taken into account in the expression

$$[R] = [Q][\Delta] + [R°] \qquad 2$$

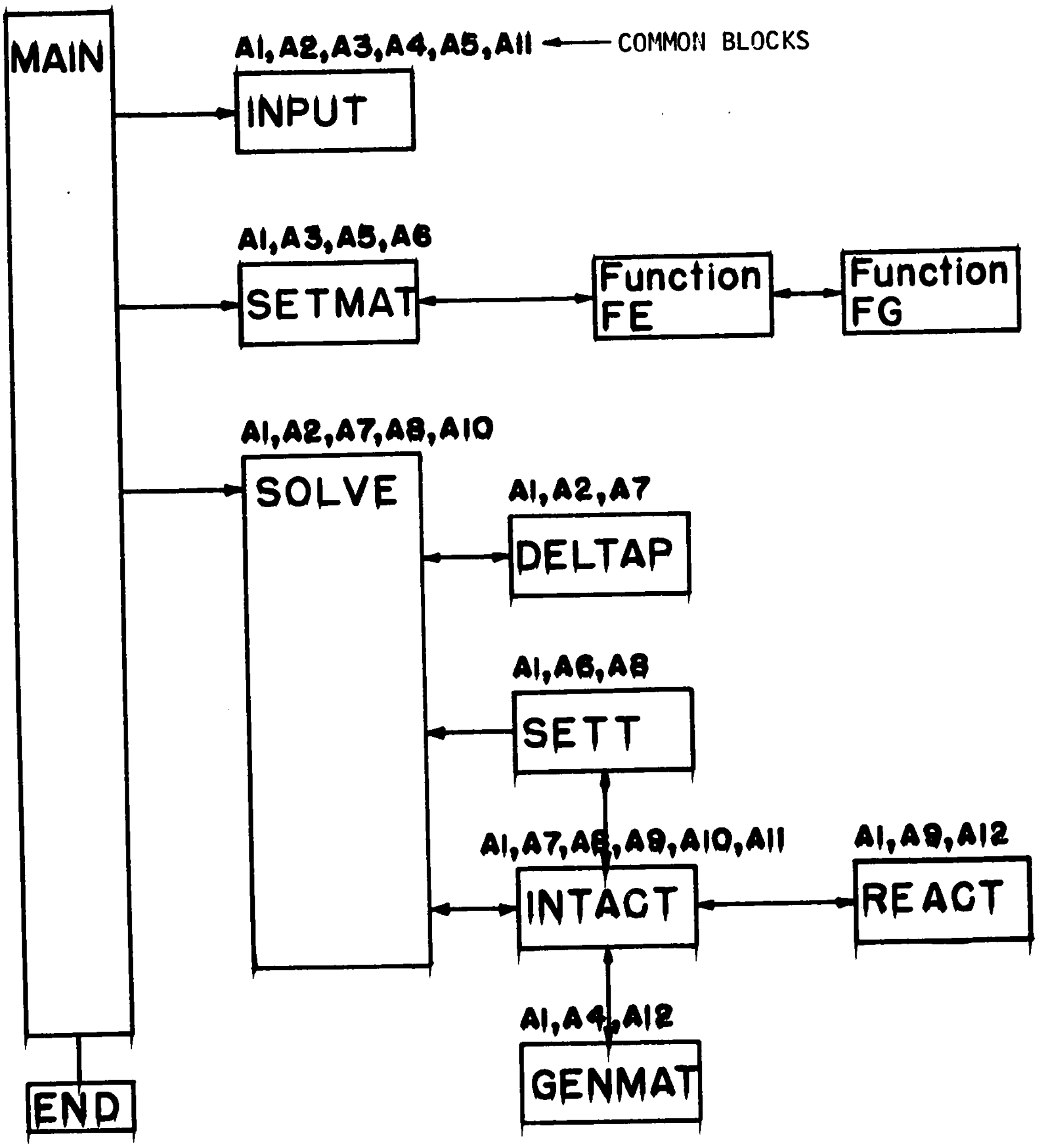
where $[Q]$ denotes a square matrix which specifies the vertical frame reactions at N supports for a unit displacement of a single support

$[R°]$ denotes a column matrix of the footing loads due to the self-weight of the structural frame under conditions of zero differential settlement.

Program Subroutines

The flow chart indicates the sequence of subroutines and functions which are used in the computer solution. Subroutine "INPUT" inputs the required data and prints it immediately thereafter. The input data is described in a later section. Subroutine "SETMAT" computes the square matrix

Soil Structure Interaction Flow Chart



[M] for the case of rectangular spread footings located on the surface of a semi-infinite, homogeneous, isotropic, elastic body. This matrix is stored in common block A6. Subroutine "SOLVE" controls the "incremental" and "conventional" soil-structure interaction techniques by performing the calculations storey by storey. The self-weight structural load per storey is computed by "DELTAP". The resulting column matrix [R°] is stored in common block A7. Subroutine "SETT" computes the settlement in accordance with equation 1 for any specified set of footing loads. The results of this calculation are stored in common block A8. Subroutine "GENMAT" generates the square matrix of reaction coefficients [Q] from the input data for a particular structural frame of specified storey height. The matrix is stored in common block A12. Subroutine "REACT" computes the vertical reactions generated at the supports due to distortion of the structural frame. This corresponds to multiplication of [Q][Δ] as indicated earlier in equation 2. The results are stored in common block A9. The iterative computations are carried out in subroutine "INTACT". The maximum number of iterations can be controlled and the rate of convergence can be altered by means of the relaxation factor used in this subroutine. The convergence criteria is specified by the program user for flexibility. At completion of iteration the final footing loads and settlements for a particular storey are stored in common block A10. The number of iterations required to satisfy the convergence criteria is stored here as well.

Operational Characteristics

Number of footings ≤ 30.

Number of storeys ≤ 30.

Number of input footing loads with height of building per footing ≤ 7.

Number of input reaction coefficient matrices with height of building ≤ 10.

Number of different soil zones ≤ 10.

Total storage requirement = 150 K.

Input

Card (A) Control Cards.

(18A4/3I5/F5.2,I5,F6.4)

(i) Column

1-72	Heading or title.

(ii) Column

1-5	Number of footings
6-10	Number of storeys
11-15	Number of soil zones.

(iii) Column

1-5	Relaxation factor (e.g., 0.5 gives good rate of convergence)
6-10	Maximum number of iterations allowed (e.g., 30 for large space frame)
11-16	Convergence criteria in kips (e.g., .05 kip; footing loads must be repeated within this limit of variation).

Card (B) Footing Load Cards - number of cards must equal the number of footings and be in order of footing identification numbers.

(1X,I3,1X,7(I2,F8.0))

Column	
2-4	Footing identification number
6-7	Storey height
8-15	Total Footing Load of the building with construction complete to the specified storey height
16-17	Storey height
18-25	Total Footing Load of the building with construction complete to the specified storey height.
etc.	

Note - Although 7 footing loads may be specified to define the curve of self-weight footing load with storey height, a lesser number can also be used. The last footing load specified must always equal the footing load corresponding to the maximum height of building. At footings located under a part of the structure which is smaller in height (e.g., a lower wing), the maximum footing load at the upper storey height of the wing must be duplicated to correspond to the maximum storey height of the entire frame.

Card (C) Footing Geometry and Coordinate Cards - number of cards must equal the number of footings and be in order of footing identification number.

(2X,I3,4F10.1)

Column	
3-5	Footing identification number
6-15	X - coorindate of the footing centroid
16-25	Y - coordinate of the footing centroid
26-35	Footing length in X direction
36-45	Footing length in Y direction.

Note - Coordinate axes must be parallel or perpendicular to all footing sides.

Card (D) Storey height of first input reactions coefficient matrix

(2X,I3)

Column	
3-5	Storey height.

Card (E) Reaction Coefficients - first matrix

(8F10.1)

Column	
1-10	Reaction at footing 1 due to a unit displacement at footing 1
11-20	Reaction at footing 2 due to a unit displacement at footing 1
21-30	Reaction at footing 3 due to a unit displacement at footing 1
71-80	Reaction of footing 8 due to a unit displacement at footing 1
1-10	(Next card). Reaction at footing 9 due to a unit displacement at footing 1

etc.	Unit reaction at last footing is specified for the unit displacement at footing 1
1-10	Reaction at footing 1 due to a unit displacement at footing 2
etc.	As for unit displacement of footing 1.

<u>Note</u> - For a structure with 25 footings the reaction coefficient matrix will be specified on 3 x 25 = 75 cards.

<u>Note</u> - The sign of the reactions must be included as the sum of any column or row in the matrix must equal zero.

<u>Note</u> - Reaction coefficients are in kips for unit displacements of 1 inch.

<u>Card (F)</u> Storey height of second input reaction coefficient matrix

(2X,I3)

<u>Card (G)</u> Reaction Coefficients - second matrix

(8F10.1)

<u>Note</u> - Cards F and G correspond to Cards D and E respectively.

<u>Note</u> - A maximum of 10 reaction coefficient matrices may be entered. Because of the volume of cards required it is desirable to keep the number of matrices as small as possible.

<u>Note</u> - The last matrix entered must be the matrix for the maximum building height.

Card (H) Soil Parameters - number of cards must equal the specified number of soil zones.

(3X,I3,1X,5F10.1,3X,F5.3)

Column	
4-6	Soil zone identification number
8-17	X - coordinate of soil zone centroid
18-27	Y - coordinate of soil zone centroid
28-37	Soil zone length in X direction
38-47	Soil zone length in Y direction
48-57	Soil zone Young's modulus in psi
61-65	Poisson's ratio of soil zone.

Note - Soil zones must be rectangular with sides parallel or perpendicular to coordinate axes. The perimeters of soil zones must not overlap. All footings must be enclosed by a soil zone or a number of soil zones. The perimeter of a soil zone must not fall on the centroid of a footing. The ratio of elastic moduli of two adjacent soil zones should not exceed 2.0. If the approximation for variable soil conditions is not used a single soil zone enclosing all footings must be specified.

Special Options

The approximation for variable site conditions may be used without taking into account the rigidity of the structure. For this case, no reaction coefficients need be specified as the structure is assumed to be perfectly flexible.

Card D, the input storey height, must read zero. The cards which must follow are those described by Card H. The in between cards, Cards E, F, G, are omitted.

COMPUTER PROGRAM LISTING

```
G LEVEL 1, MOD 4              MAIN              DATE = 70311          03/27/41

 C      SOIL STRUCTURE INTERACTION: J. DEJONG
 C      "INCREMENTAL" METHOD INCLUDES VARIATION OF FRAME RIGIDITY DURING
 C      CONSTRUCTION.
 C      SETTLEMENTS ARE COMPUTED FOR THE CASE OF RECTANGULAR FOOTINGS
 C      LOCATED ON AN ELASTIC, HOMOGENEOUS, ISOTROPIC, SEMI-INFINITE
 C      FOUNDATION.
 C      APPROXIMATION FOR VARIATION OF FOUNDATION PROPERTIES OVER THE SITE
 C      IS OPTIONAL.
 C      SETTLEMENT COMPUTATION WITH THE ASSUMPTION OF PERFECT STRUCTURAL
 C      FLEXIBILITY IS OPTIONAL.
        CALL INPUT
        CALL SETMAT
        CALL SOLVE
        STOP
        END
```

```
      SUBROUTINE INPUT
C
      COMMON/A1/NUMCOL,NUMSTR,NUMFPR
     1/A2/NCOL(30),ISTOR(30,7),DLOAD(30,7)
     1/A3/NFTG(30),XCOOR(30),YCOOR(30),DX(30),DY(30)
     1/A4/QCOEF(10,30,30),IST(10)
     1/A5/NUME(10),XZON(10),YZON(10),XDZO(10),YDZO(10),ZMOD(10),POIS(10)
     1/A11/RELAX,IT,CRIT
      DIMENSION HED(18)
      IFLAG=0
      READ(5,1000)HED,NUMCOL,NUMSTR,NUMFPR,RELAX,IT,CRIT
      WRITE(6,2000)HED,NUMCOL,NUMSTR,NUMFPR,RELAX,IT,CRIT
C
C     ******************************************************************
C     READ AND PRINT LOAD DATA
C     ******************************************************************
C
      WRITE(6,2002)
      DO 1 J=1,NUMCOL
      READ(5,1001)NCOL(J),(ISTOR(J,I),DLOAD(J,I),I=1,7)
      WRITE(6,2001)NCOL(J),(ISTOR(J,I),DLOAD(J,I),I=1,7)
    1 CONTINUE
C
C     ******************************************************************
C     READ AND PRINT FOOTING GEOMETRY
C     ******************************************************************
C
      READ(5,1002)(NFTG(J),XCOOR(J),YCOOR(J),DX(J),DY(J),J=1,NUMCOL)
      WRITE(6,2003)
      WRITE(6,2004)(NFTG(J),XCOOR(J),YCOOR(J),DX(J),DY(J),J=1,NUMCOL)
C
C     ******************************************************************
C     READ AND PRINT REACTION COEFFICIENTS
C     ******************************************************************
C
      DO 3 JJ=1,10
      READ(5,1003)IST(JJ)
C     CHECK FOR OPTION OF SETTLEMENT COMPUTATION ASSUMING PERFECT
C     STRUCTURAL FLEXIBILITY.
      IF(IST(JJ).EQ.0) GO TO 14
      WRITE(6,2005)
      WRITE(6,2006)IST(JJ),(J,J=1,NUMCOL)
      DO 2 J=1,NUMCOL
      READ(5,1004)(QCOEF(JJ,J,I),I=1,NUMCOL)
      WRITE(6,2007)J,(QCOEF(JJ,J,I),I=1,NUMCOL)
    2 CONTINUE
      DO 7 J=1,NUMCOL
      TOT=0.0
      DO 8 I=1,NUMCOL
      TOT=TOT+QCOEF(JJ,J,I)
    8 CONTINUE
      TOT=ABS(TOT)
      IF(TOT-1.0) 7,7,10
   10 WRITE(6,2011) J,JJ
      IFLAG=1
```

```
    7 CONTINUE
      DO 9 I=1,NUMCOL
      TOT=0.0
      DO 11 J=1,NUMCOL
      TOT=TOT+QCOEF(JJ,J,I)
   11 CONTINUE
      TOT=ABS(TOT)
      IF(TOT-1.0) 9,9,12
   12 WRITE(6,2012) I,JJ
      IFLAG=1
    9 CONTINUE
      IF(IST(JJ)-NUMSTR) 3,4,5
    3 CONTINUE
      GO TO 4
   14 DO 15 J=1,NUMCOL
      DO 16 I=1,NUMCOL
      QCOEF(1,J,I)=0.0000000000
   16 CONTINUE
   15 CONTINUE
      IST(1)=NUMSTR
    4 CONTINUE
C
C     ******************************************************************
C     READ AND PRINT FOUNDATION PROPERTIES
C     ******************************************************************
C
      WRITE(6,2008)
      READ(5,2009)(NONE(J),XZON(J),YZON(J),XDZO(J),YDZO(J),ZMOD(J),POIS(
     1J),J=1,NUMFPR)
      WRITE(6,2009)(NONE(J),XZON(J),YZON(J),XDZO(J),YDZO(J),ZMOD(J),POIS
     1(J),J=1,NUMFPR)
      IF(IFLAG.EQ.1) GO TO 13
      GO TO 6
    5 WRITE(6,2010)
   13 CALL EXIT
 1000 FORMAT(18A4/3I5/F5.2,I5,F6.4)
 1001 FORMAT(1X,I3,1X,7(I2,F8.0))
 1002 FORMAT(2X,I3,4F10.1)
 1003 FORMAT(2X,I3)
 1004 FORMAT(8F10.1)
 2000 FORMAT(1H1,18A4//
     13X,'NUMBER OF COLUMNS      =',I4//
     23X,'NUMBER OF STOREYS      =',I4//
     33X,'NUMBER OF DIFF. SOILS =',I4//
     43X,'RELAXATION FACTOR      =',F5.2//
     53X,'NUMBER OF ITERATIONS  =',I4//
     63X,'CONVERGENCE CRITERIA  =',F6.4)
 2001 FORMAT(5X,I2,3X,7(I4,F9.0))
 2002 FORMAT(1H1,3X,'LOAD DATA',//,3X,'COLUMN  STOREY LOAD  STOREY LOAD
     1 STOREY LOAD  STOREY LOAD  STOREY LOAD  STOREY LOAD  STOREY LOAD',
     1//)
 2003 FORMAT(1H1,3X,'FOOTING GEOMETRY',//,3X,'FTG.     X ORD.     Y ORD.
     1WIDTH X  LENGTH Y',//)
 2004 FORMAT(3X,I3,4F10.1)
 2005 FORMAT(1H1,3X,'REACTION COEFFICIENT MATRIX',//)
 2006 FORMAT(3X,'STOREY HEIGHT=',I3,/,(10X,12I10))
 2007 FORMAT(5X,I3,2X,12F10.1,/,(10X,12F10.1))
 2008 FORMAT(1H1,3X,'FOUNDATION PROPERTIES',//,2X,'ZONE      X ORD.    Y
     1ORD.  WIDTH X  LENGTH Y  MODULUS  POIS. RAT.'/)
 2009 FORMAT(3X,I3,1X,5F10.1,3X,F5.3)
 2010 FORMAT(3X,'ERROR - LAST STOREY READ EXCEEDS BLDG. HEIGHT')
 2011 FORMAT(3X,'ROW',I3,' IN REACTION COEFFICIENT MATRIX',I3,' DOES NOT
     1 HAVE A SUM EQUAL TO ZERO')
 2012 FORMAT(3X,'COLUMN',I3,' IN REACTION COEFFICIENT MATRIX',I3,' DOES
     1NOT HAVE A SUM EQUAL TO ZERO')
    6 RETURN
      END
```

```
G LEVEL 1, MOD 4              SETMAT              DATE = 70311           03/27/41
      SUBROUTINE SETMAT
C
      COMMON/A1/NUMCOL,NUMSTR,NUMFPR
     1/A3/NFTG(30),XCOOR(30),YCOOR(30),DX(30),DY(30)
     1/A5/NONE(10),XZON(10),YZON(10),XDZO(10),YDZO(10),ZMOD(10),POIS(10)
     1/A6/CN(30,30)
C
C     ******************************************************************
C     CALCULATE INFLUENCE VALUES FOR SETTLEMENT
C     ******************************************************************
C
      DO 1 I=1,NUMCOL
      RR=DX(I)/2.0
      SS=DY(I)/2.0
      DO 2 J=1,NUMCOL
      DA=ABS(XCOOR(I)-XCOOR(J))
      DB=ABS(YCOOR(I)-YCOOR(J))
      FI=FE (RR,SS,DA,DB)
      CN(I,J)=FI/(DX(I)*DY(I))
    2 CONTINUE
    1 CONTINUE
C
C     ******************************************************************
C     FIND SOIL MODULUS PERTINENT TO FOOTING
C     ******************************************************************
C
      IF(NUMFPR.NE.1) GO TO 5
      DO 7 J=1,NUMCOL
      DO 8 I=1,NUMCOL
      CN(I,J)=(CN(I,J)*1000.0*(1.0-POIS(1)**2))/ZMOD(1)
    8 CONTINUE
    7 CONTINUE
      GO TO 12
    5 NCOUNT=0
      DO 4 LL=1,NUMFPR
      RR=XDZO(LL)/2.0
      SS=YDZO(LL)/2.0
      DO 9 J=1,NUMCOL
      DA=ABS(XZON(LL)-XCOOR(J))
      DB=ABS(YZON(LL)-YCOOR(J))
      IF(RR.EQ.DA.OR.SS.EQ.DB) GO TO 11
      IF(DA.GT.RR.OR.DB.GT.SS) GO TO 9
      NCOUNT=NCOUNT+1
      DO 10 I=1,NUMCOL
      CN(I,J)=(CN(I,J)*1000.0*(1.0-POIS(LL)**2))/ZMOD(LL)
   10 CONTINUE
    9 CONTINUE
    4 CONTINUE
      IF(NCOUNT.LT.NUMCOL) GO TO 13
      GO TO 12
   11 WRITE(6,2001)LL,J
 2001 FORMAT(3X,'ERROR - BOUNDARY OF SOIL ZONE',I4,' LOCATED ON CENTER O
     1F FOOTING',I4)
      CALL EXIT
   13 WRITE(6,2002)
 2002 FORMAT(3X,'ERROR - ONE OR MORE FOOTING CENTERS ARE NOT LOCATED WIT
     1HIN A SOIL ZONE')
      CALL EXIT
   12 RETURN
      END
```

```
G LEVEL 1, MOD 4              FE                 DATE = 70311          03/27/41

            FUNCTION FE (X,Y,A,B)
      C
            DIMENSIONRF(4)
      C
      C     *********************************************************************
      C     CALCULATE INFLUENCE COEFFICIENT COMPONENTS
      C     *********************************************************************
      C
            DO7M=1,4
            GOTO(1,2,3,4),M
          1 W=A+X
            V=B+Y
            GOTO5
          2 W=ABS(A-X)
            GOTO5
          3 V=ABS(B-Y)
            GOTO5
          4 W=A+X
          5 IF(W.EQ.0.0.OR.V.EQ.0.0)GOTO12
            IF(W.LT.V)GOTO6
            R=W/V
            RF(M)=V*FG (R)
            GOTO7
          6 R=V/W
            RF(M)=W*FG (R)
            GOTO7
         12 RF(M)=0.0
          7 CONTINUE
            IF(A.LE.X.AND.B.LE.Y)GOTO8
            IF(A.LE.X)GOTO9
            IF(B.LE.Y)GOTO10
      C     CASE 1
            FE=RF(1)-RF(2)+RF(3)-RF(4)
            GOTO11
      C     CASE 2
          9 FE=RF(1)+RF(2)-RF(3)-RF(4)
            GOTO11
      C     CASE 3
         10 FE=RF(1)-RF(2)-RF(3)+RF(4)
            GOTO11
      C     CASE 4
          8 FE=RF(1)+RF(2)+RF(3)+RF(4)
         11 RETURN
            END

G LEVEL 1, MOD 4              FG                 DATE = 70311          03/27/41

            FUNCTION FG (R)
      C
      C     *********************************************************************
      C     ARITHMETIC FUNCTION
      C     *********************************************************************
      C
            A=SQRT(R**2+1.0)
            B=R*ALOG((1.0+A)/R)
            C=ALOG(R+A)
            FG=(B+C)/3.1416
            RETURN
            END
```

```
      SUBROUTINE SOLVE
C
      COMMON/A1/NUMCOL,NUMSTR,NUMFPR
     1/A2/NCOL(30),ISTOR(30,7),DLOAD(30,7)
     1/A7/DELP(30)
     1/A8/SET(30)
     1/A10/PD(30),SE(30),ITRIAL
      DIMENSION CONLCU(30,30),CONLIN(30,30),CONDCU(30,30),CONDIN(30,30),
     1SSILCU(30,30),SSILIN(30,30),SSIDCU(30,30),SSIDIN(30,30),IR(30)
      DIMENSION CONSET(30)
C
C     ******************************************************************
C     INITIALIZATION FOR FIRST STOREY
C     ******************************************************************
C
      JSTOR=1
      CALL DELTAP (JSTOR)
      CALL SETT (DELP)
      DO 1 I=1,NUMCOL
      CONLCU(I,1)=DELP(I)
      CONLIN(I,1)=DELP(I)
      CONDCU(I,1)=SET(I)*12.0
      CONDIN(I,1)=SET(I)*12.0
    1 CONTINUE
      CALL INTACT (JSTOR)
      DO 2 I=1,NUMCOL
      SSILCU(I,1)=PD(I)
      SSILIN(I,1)=PD(I)
      SSIDCU(I,1)=SE(I)*12.0
      SSIDIN(I,1)=SE(I)*12.0
      IR(JSTOR)=ITRIAL
    2 CONTINUE
C
C     ******************************************************************
C     SOIL STRUCTURE INTERACTION (INCREMENTAL METHOD) TO FULL BLDG. HGT.
C     ******************************************************************
C
      DO 3 JSTOR=2,NUMSTR
      CALL DELTAP (JSTOR)
      CALL SETT (DELP)
      DO 4 I=1,NUMCOL
      CONLIN(I,JSTOR)=DELP(I)
      CONLCU(I,JSTOR)=CONLCU(I,JSTOR-1)+CONLIN(I,JSTOR)
      CONDIN(I,JSTOR)=SET(I)*12.0
      CONDCU(I,JSTOR)=CONDCU(I,JSTOR-1)+CONDIN(I,JSTOR)
    4 CONTINUE
      CALL INTACT (JSTOR)
      DO 5 I=1,NUMCOL
      SSILIN(I,JSTOR)=PD(I)
      SSILCU(I,JSTOR)=SSILCU(I,JSTOR-1)+SSILIN(I,JSTOR)
      SSIDIN(I,JSTOR)=SE(I)*12.0
      SSIDCU(I,JSTOR)=SSIDCU(I,JSTOR-1)+SSIDIN(I,JSTOR)
      IR(JSTOR)=ITRIAL
    5 CONTINUE
    3 CONTINUE
```

```
C
C     ******************************************************************
C     CHECK SOIL STRUCTURE INTERACTION (CONVENTIONAL METHOD)
C     ******************************************************************
C
      DO 6 I=1,NUMCOL
      DO 7 J=1,NUMSTR
      IF(ISTOR(I,J).NE.NUMSTR) GO TO 7
      DELP(I)=DLOAD(I,J)
    7 CONTINUE
    6 CONTINUE
      CALL SETT (DELP)
      DO 8 I=1,NUMCOL
      CONSET(I)=SET(I)*12.0
    8 CONTINUE
      CALL INTACT (NUMSTR)
C
C     ******************************************************************
C     OUTPUT RESULTS
C     ******************************************************************
C
      DO 10 I=1,NUMCOL
      SE(I)=SE(I)*12.0
      WRITE(6,2001)I
      WRITE(6,2002)(J,CONLCU(I,J),CONLIN(I,J),CONDCU(I,J),CONDIN(I,J),SS
     1ILCU(I,J),SSILIN(I,J),SSIDCU(I,J),SSIDIN(I,J),IR(J),J=1,NUMSTR)
      WRITE(6,2003)
      WRITE(6,2004)NUMSTR,DELP(I),CONSET(I),PD(I),SE(I),ITRIAL
      WRITE(6,2005)
   10 CONTINUE
 2001 FORMAT(1H1,10X,'FOOTING NUMBER',I3,//,22X,'LOADS AND SETTLEMENTS',
     121X,'LOADS AND SETTLEMENTS WITH',/,19X,'NO SOIL STRUCTURE INTERACT
     2ION',13X,'INCREMENTAL SOIL STRUCTURE INTERACTION',/,6X,'----------
     3--------------------------------',5X,'----------------------------
     4----------------------------',/,8X,'STOREY   CUMUL.   INCREM.    C
     5UMUL    INCRFM.',9X,'CUMUL.   INCREM      CUMUL.   INCREM.    NO. O
     6F',/,8X,'HEIGHT   LOAD       LOAD       SETT.      SETT.',10X,'LOAD
     7     LOAD        SETT.      SETT.   ITERATIONS',/)
 2002 FORMAT(9X,I3,3X,F9.1,1X,F9.1,2X,F6.3,4X,F6.3,8X,F9.1,1X,F9.1,2X,F6
     1.3,4X,F6.3,6X,I3)
 2003 FORMAT(1X,/,61X,'CONVENTIONAL SOIL STRUCTURE INTERACTION',/)
 2004 FORMAT(9X,I3,3X,F9.1,12X,F6.3,18X,F9.1,12X,F6.3,16X,I3,/)
 2005 FORMAT(9X,'ALL RESULTS IN UNITS OF KIPS AND INCHES')
      RETURN
      END
```

```
G LEVEL 1, MOD 4              GENMAT              DATE = 70311          03/27/41
      SUBROUTINE GENMAT (JSTOR)
C
      COMMON/A1/NUMCOL,NUMSTR,NUMFPR
     1/A4/QCOEF(10,30,30),IST(10)
     1/A12/QCOF(30,30)
C
C     ******************************************************************
C     GENERATE REACTION COEFFICIENT MATRIX
C     ******************************************************************
C
      DO 1 K=1,10
      IF(JSTOP-IST(K))3,2,1
    3 IF(K.EQ.1) GO TO 4
      RAT=FLOAT(JSTOR-IST(K-1))/(IST(K)-IST(K-1))
      DO 5 I=1,NUMCOL
      DO 6 J=1,NUMCOL
      DCOEF=QCOEF(K,I,J)-QCOEF(K-1,I,J)
      QCOF(I,J)=QCOEF(K-1,I,J)+RAT*DCOEF
    6 CONTINUE
    5 CONTINUE
      GO TO 7
    4 RAT=FLOAT(JSTOR/IST(1))
      DO 8 I=1,NUMCOL
      DO 9 J=1,NUMCOL
      QCOF(I,J)=RAT*QCOEF(1,I,J)
    9 CONTINUE
    8 CONTINUE
      GO TO 7
    2 DO 10 I=1,NUMCOL
      DO 11 J=1,NUMCOL
      QCOF(I,J)=QCOEF(K,I,J)
   11 CONTINUE
   10 CONTINUE
      GO TO 7
    1 CONTINUE
    7 RETURN
      END
```

```
G LEVEL 1, MOD 4              DELTAP              DATE = 70311          03/27/41
      SUBROUTINE DELTAP (JSTOR)
C
      COMMON/A1/NUMCOL,NUMSTR,NUMFPR
     1/A2/NCOL(30),ISTOR(30,7),DLOAD(30,7)
     1/A7/DELP(30)
C
C     ******************************************************************
C     COMPUTE STOREY LOAD
C     ******************************************************************
C
      DO 1 I=1,NUMCOL
      DO 2 J=1,7
      JJ=J
      IF(JSTOR.LE.ISTOR(I,J)) GO TO 3
    2 CONTINUE
    3 IF(JJ.NE.1) GO TO 4
      DELP(I)=DLOAD(I,JJ)/ISTOR(I,JJ)
      GO TO 1
    4 DELP(I)=(DLOAD(I,JJ)-DLOAD(I,JJ-1))/(ISTOR(I,JJ)-ISTOR(I,JJ-1))
    1 CONTINUE
      RETURN
      END
```

```
G LEVEL 1, MOD 4              INTACT             DATE = 70311           03/27/41
      SUBROUTINE INTACT (JSTOR)
C
      COMMON/A1/NUMCOL,NUMSTR,NUMFPR
     1/A7/DELP(30)
     1/A8/SET(30)
     1/A9/DISP(30)
     1/A10/PD(30),SE(30),ITRIAL
     1/A11/RELAX,IT,CRIT
      DIMENSION PLD(30)
      CALL GENMAT (JSTOR)
C
C     ******************************************************************
C     ITERATE TO CONVERGE UPON REPEATING LOADS
C     ******************************************************************
C
      DO 9 I=1,NUMCOL
      PLD(I)=DELP(I)
    9 CONTINUE
      DO 1 ITR=1,IT
      ITRIAL=ITR
      IFLAG=0
      CALL REACT (SET)
      DO 2 I=1,NUMCOL
      SE(I)=SET(I)
      PD(I)=DELP(I)+DISP(I)
      DIFF=ABS(PLD(I)-PD(I))
      IF(DIFF.GT.CRIT)IFLAG= 1
    2 CONTINUE
      IF(IFLAG.EQ.0) GO TO 5
      CALL SETT (PD)
      DO 4 I=1,NUMCOL
      SET(I)=SE(I)+RELAX*(SET(I)-SE(I))
      PLD(I)=PD(I)
    4 CONTINUE
    1 CONTINUE
    5 DO 8 I=1,NUMCOL
      SE(I)=(SE(I)+SET(I))/2.0
    8 CONTINUE
      IF(ITRIAL.EQ.IT) GO TO 6
      GO TO 7
    6 WRITE(6,2001) JSTOR
 2001 FORMAT(3X,'THE SOLUTION FOR SOIL STRUCTURE INTERACTION REQUIRED TH
     1E MAXIMUM NUMBER OF ITERATIONS SPECIFIED FOR STOREY',I4)
    7 RETURN
      END
```

```
G LEVEL 1, MOD 4              REACT              DATE = 70311           03/27/41
      SUBROUTINE REACT (S)
C
      COMMON/A1/NUMCOL,NUMSTR,NUMFPR
     1/A9/DISP(30)
     1/A12/QCOF(30,30)
      DIMENSION S(30)
C
C     ******************************************************************
C     CALCULATF REDISTRIBUTED LOAD
C     ******************************************************************
C
      DO 1 I=1,NUMCOL
      DISP(I)=0.0
      DO 2 J=1,NUMCOL
      DISP(I)=QCOF(I,J)*S(J)*12.0+DISP(I)
    2 CONTINUE
    1 CONTINUE
      RETURN
      END
```

```
G LEVEL 1, MOD 4              SETT                DATE = 70311          03/27/41
        SUBROUTINE SETT (P)
  C
        COMMON/A1/NUMCOL,NUMSTR,NUMFPR
       1/A6/CN(30,30)
       1/A8/SET(30)
        DIMENSION P(30)
  C
  C     *********************************************************************
  C     CALCULATE SETTLEMENT
  C     *********************************************************************
  C
        DO 1 J=1,NUMCOL
        SET(J)=0.0
        DO 2 I=1,NUMCOL
        SET(J)=(CN(I,J)*P(I))/1000.0+SET(J)
      2 CONTINUE
      1 CONTINUE
        RETURN
        END
```

APPENDIX G

ELASTIC REBOUNDS DUE TO EXCAVATION OF RECTANGULAR BLOCKS

Solution Description

The solution presented here is based on the standard equations of elasticity for the settlement of a point located within an elastic body below a corner of a rectangular, flexible, uniformly loaded area. The soil body is considered to be homogeneous and isotropic. For heave computations the direction of the loads are simply reversed. Due to the complex shapes of some excavations, excavations are defined in terms of a number of component blocks, each having a specific length, width, centroidal location and bottom elevation. Superposition techniques are used to compute heave at any point within the elastic body.

The computer solution takes the excavation blocks and divides them into a series of slabs, the thickness of which is controlled by the program user. As each slab is located at a different elevation, and therefore in different strata, the force per square foot and the vertical distance to a specified rebound point will be different for each slab. Superposition within the computer solution will therefore not only be in the usual horizontal sense but also in the vertical.

The location of the reference coordinate axes must be outside the entire excavation area. Their orientation should be such that they parallel the exterior boundary of the excavation. The sides of the rectangular excavation blocks must be either parallel or perpendicular to a particular axis.

The contribution to rebound at a specific point due to the removal of M excavation slabs, all at the same elevation, can be written as

$$\Delta_j(Z_j) = \frac{1-\mu^2}{E}\left[\sum_{i=1}^{i=M} a_i q_i A_i\right] - \frac{(1-2\mu)(1+\mu)}{2E}\left[\sum_{i=1}^{i=M} a_i q_i B_i\right]$$

where Z_j = the depth of the point below the bottom of the excavation slab

a = width of the rectangular excavation slab

b = length of the rectangular excavation slab

q = unload of slab in kips per square foot

A = $f(m,n)$

B = $f(m,n)$

M = $\frac{b}{a}$

n = $\frac{Z}{a}$.

The removal of succeeding excavation slabs yields the total heave.

$$\Delta = \sum_{j=1}^{j=N} \Delta_j(Z_j)$$

where N = the total number of excavation layers.

Rebound points at which heaves are computed can be at any location. However, the method of computation is dependent on this location of the rebound point. If the point is located at the ground surface, the contribution to heave from all excavation slabs is computed with Z = 0.0. If the point is located within the soil body but above the maximum depth of excavation, the contribution of the slabs above the point will be computed with a variable depth Z. The contribution to heave of slabs lower than the point will be calculated with Z = 0. If the rebound point is located below the maximum depth of excavation, the contribution from all slabs will be calculated with a variable depth Z.

Settlements are computed for a specified variation of Young's modulus and Poisson's ratio and are presented in tabular form for each footing centroid. The variation of the above elastic parameters is defined with three numbers. For example, settlements may be computed for a series of foundation moduli which starts at a particular value and is incremented by a particular value a specified number of times.

Operational Characteristics

Maximum number of soil types $\leq$ 10.
Maximum number of rebound points $\leq$ 50.
Maximum number of excavation blocks $\leq$ 100.
Required computer storage = 100 K.

Input

Card (A) Variation of Elastic Parameters.

(F10.0,2X,F8.0,7X,I3,5X,F5.3,5X,F5.3,7X,I3)

Columns	
1-10	Young's modulus in psi at which computations start
13-20	Increment of Young's modulus in psi
28-30	Number of increments $\leq$ 15
36-40	Poisson's ratio at which computations start
46-50	Increment of Poisson's ratio
58-60	Number of increments - quantity not limited.

Card (B) Depth of Excavation Layers.

(6X,F4.1)

Columns	
7-10	Excavation layer depth in feet (e.g., 1.0 foot).

Card (C) Number of Soil Types in Stratigraphic Profile.

(3X,I2)

Columns	
4-5	Number of soil types; $\leq$ 10.

Card (D) Soil Types - Number of cards must equal value indicated on Card C. Cards must be in order of increasing depth below ground surface.

(A4,6X,F6.1,4X,F5.1,5X,30X,F5.3)

Columns	
1-4	Alphanumeric name of soil type
11-16	Elevation of upper surface of stratum; in feet
21-25	Thickness of stratum; in feet
61-65	Total unit weight of excavated soil; in kips per cubic foot.

Note - Only the strata in which excavation takes place need be entered. If the thickness of the lowest stratum is not known an arbitrary value can be specified. The resulting bottom elevation of the stratum must be below the bottom elevation of the deepest excavation block.

Card (E) Number of Rebound Points.
(3X,I2)

Columns	
4-5	Total number of points at which rebounds are to be calculated; $\leq$ 50.

Card (F) Rebound Point Locations.
(I2,3X,F6.1,4X,F6.1,4X,F6.1)

Columns	
1-2	Rebound point identification number
6-11	X - coordinate of point; in feet
16-21	Y - coordinate of point; in feet
26-31	Elevation of point; in feet.

Card (G) Title of Excavation Data.

(17A4)

Card (H) Number of Excavation Blocks.

(3X,I2)

Columns	
4-5	Number of excavation blocks; ≤ 100.

Card (I) Excavation Block Data.

(F5.1,5X,F5.1,5X,F6.1,4X,F6.1,4X,F6.1)

Columns	
1-5	Length of block in X-direction; in feet
11-15	Length of block in Y-direction; in feet
21-26	X - coordinate of excavation block centroid; in feet
31-36	Y - coordinate of excavation block centroid; in feet
41-46	Elevation of bottom of excavation block; in feet.

COMPUTER PROGRAM LISTING

```
G LEVEL 1, MOD 4                MAIN                DATE = 70316            18/43/51
C
C     ******************************************************************
C     ELASTIC REBOUND OF THE FOUNDATION DUE TO EXCAVATION: J. DEJONG
C     THIS APPROXIMATE SOLUTION IS BASED ON THE ASSUMPTION THAT THE BODY
C     UNDERGOING REBOUND IS HOMOGENEOUS,ISOTROPIC AND OF SEMI-INFINITE
C     EXTENT.
C     ******************************************************************
C
      DIMENSION NAME(10),TITLE(17),QCOMP(100),REBI(30),EV(30)
      COMMON/SOIL/ELEV(10),T(10),G(10)
      COMMON/RBPD/IRB(50),XR(50),YR(50),RB(50)
      COMMON/LOAD/DA(100),DB(100),X(100),Y(100),EEXC(100)
      COMMON/LOLA/BOT(100),CALC(100,100),M,NUMB
C
C     ******************************************************************
C     READ AND PRINT INPUT DATA
C     ******************************************************************
C
C
C     READ VARIATION OF ELASTIC PARAMETERS.
C
      READ(5,12)ELAS,DELAS,NEL,PR,DPR,NPR
C
C     READ DEPTH OF EXCAVATION SLAB.
C
      READ(5,2)DEL
    2 FORMAT(6X,F4.1)
   12 FORMAT(F10.0,2X,F8.0,7X,I3,5X,F5.3,5X,F5.3,7X,I3)
C
C     ******************************************************************
C     READ SOIL DATA
C     ******************************************************************
C
      READ(5,3)L
      READ(5,4)(NAME(J),ELEV(J),T(J),G(J),J=1,L)
    3 FORMAT(3X,I2)
    4 FORMAT(A4,6X,F6.1,4X,F5.1,5X,30X,F5.3)
C
C     ******************************************************************
C     READ REBOUND POINT DATA.
C     ******************************************************************
C
      READ(5,3)K
C
C     READ REBOUND POINT LOCATIONS.
C
      READ(5,31)(IRB(J),XR(J),YR(J),RB(J),J=1,K)
   31 FORMAT(I2,3X,F6.1,4X,F6.1,4X,F6.1)
C
C     ******************************************************************
C     READ EXCAVATION BLOCK DATA.
C     ******************************************************************
C
      READ(5,30)(TITLE(I),I=1,17)
      READ(5,3)M
```

```
      READ(5,32)(DA(J),DB(J),X(J),Y(J),EEXC(J),J=1,M)
   30 FORMAT(17A4)
   32 FORMAT(F5.1,5X,F5.1,5X,F6.1,4X,F6.1,4X,F6.1)
C
C     ******************************************************************
C     OUTPUT SOIL AND REBOUND POINT DATA.
C     ******************************************************************
C
      WRITE(6,34)(TITLE(I),I=1,17)
      WRITE(6,33)L
      WRITE(6,35)
      WRITE(6,36)(NAME(J),ELEV(J),T(J),G(J),J=1,L)
      WRITE(6,37)
      WRITE(6,38)
      WRITE(6,39)
      WRITE(6,40)(IRB(J),XR(J),YR(J),RB(J),J=1,K)
      WRITF(6,41)(TITLE(I),I=6,17)
   33 FORMAT(28X,'SOIL PROFILE ¬ ',I2,' LAYER SYSTEM',/)
   34 FORMAT(1H1,25X,17A4,//)
   35 FORMAT(30X,'STRATUM ELEVATION  DEPTH   UNIT',/,38X,'TOP BOUND.
     1     WEIGHT',/)
   36 FORMAT(31X,A4,4X,F6.1,4X,F5.1,3X,F5.3)
   37 FORMAT(31X,'NOTF ¬ ALL UNITS ARE IN TERMS OF KIPS AND FEET',//)
   38 FORMAT(28X,'REBOUND POINT DATA',/)
   39 FORMAT(30X,'POINT   X COORD. Y COORD. ELEVATION',/)
   40 FORMAT(31X,I2,5X,F6.1,3X,F6.1,4X,F6.1)
   41 FORMAT(//,28X,'EXCAVATION DATA',12A4,//,30X,'EXCAV. LENGTH  WIDTH
     1 X COORD. Y COORD. BOT. ELEV.  LOAD'/)
C
C     ******************************************************************
C        COMPUTE THE REDUCTION IN VERTICAL STRFSS QCOMP AT ELEVATION
C        EEXC DUE TO EXCAVATION TO THIS ELEVATION.
C     ******************************************************************
C
      DO 10 I=1,M
      QCOMP(I)=0.0
      DO 11 J=1,L
      EL=ELEV(J)-T(J)
      IF(EEXC(I).LT.EL) GO TO 11
      DO 21 KI=1,J
      IF(KI.NE.J) GO TO 13
      QCOMP(I)=QCOMP(I)+G(KI)*(ELEV(KI)-EEXC(I))
      GO TO 10
   13 QCOMP(I)=QCOMP(I)+G(KI)*T(KI)
   21 CONTINUE
   11 CONTINUE
   10 CONTINUE
C
C     ******************************************************************
C     OUTPUT EXCAVATION DATA.
C     ******************************************************************
C
      WRITE(6,42)(J,DA(J),DB(J),X(J),Y(J),EEXC(J),QCOMP(J),J=1,M)
C
C        FIND EMAX AND EMIN, RESPECTIVELY THE ELEVATION OF THE
```

```
C         SHALLOWEST AND THE DEEPEST EXCAVATION.
C
      EMIN=EEXC(1)
      EMAX=EEXC(1)
      DO 9 KK=1,M
      IF(EMIN-EEXC(KK)) 7,9,8
    8 EMIN=EEXC(KK)
      GO TO 9
    7 IF(EMAX-EEXC(KK)) 6,9,9
    6 EMAX=EEXC(KK)
    9 CONTINUE
      WRITE(6,1)EMAX
      WRITE(6,50)EMIN
      WRITE(6,44)DEL
    1 FORMAT(/,28X,'MIN. DEPTH OF EXC. ',F6.1)
   42 FORMAT(31X,I2,4X,F5.1,3X,F5.1,2X,F6.1,3X,F6.1,5X,F6.1,3X,F5.2)
   44 FORMAT(/,28X,'IN THIS RUN DEL=',F4.1)
   50 FORMAT(28X,'MAX. DEPTH OF EXC. ',F6.1)
C
C     ******************************************************************
C     DETERMINE EXCAVATION UNLOAD MATRIX AND OUTPUT EXCAVATION LOAD
C     CONFIGURATION.
C     ******************************************************************
C
      WRITE(6,18)(J,J=1,M)
      DAT=ELEV(1)
      RMEM=0.0
      DO 80 IN=1,100
      XYZ=0.0
      DAT=DAT-DEL
      BOT(IN)=DAT
      DO 70 J=1,L
      EL=ELEV(J)-T(J)
      IF(DAT.LT.EL) GO TO 70
      DO 71 KI=1,J
      IF(KI.NE.J) GO TO 72
      XYZ=XYZ+G(KI)*(ELEV(KI)-DAT)
      GO TO 73
   72 XYZ=XYZ+G(KI)*T(KI)
   71 CONTINUE
   70 CONTINUE
   73 POT=DAT+DEL
      DO 74 I=1,M
      IF(EEXC(I).LT.POT) GO TO 75
      CALC(IN,I)=0.0000
      GO TO 74
   75 IF(EEXC(I).LE.DAT) GO TO 76
      FRAC=(POT-EEXC(I))/DEL
      CALC(IN,I)=(XYZ-RMEM)*FRAC
      GO TO 74
   76 CALC(IN,I)=XYZ-RMEM
   74 CONTINUE
      WRITE(6,77)BOT(IN),(CALC(IN,I),I=1,M)
      NUMB=IN
      RMEM=XYZ
```

```
      IF(DAT.LE.EMIN) GO TO 78
   80 CONTINUE
   18 FORMAT(1H1,28X,'LOAD CONFIGURATION',//,2X,'LAYER',24(1X,I3,1X))
   77 FORMAT(1X,F6.1,24F5.2)
C
C     ******************************************************************
C     COMPUTE CONTRIBUTION OF ALL EXCAVATION BLOCKS TO REBOUND AT POINT
C     "IRB(LI)".
C     ******************************************************************
C
   78 WRITE(6,5)
      DO 14 LI=1,K
      WRITE(6,23)IRB(LI)
      CA=0.0
      CB=C.0
      IF(RB(LI).LT.ELEV(1)) GO TO 28
      RB(LI)=ELEV(1)
   28 CONTINUE
      CALL RBNDC (CA,CB,LI)
C
C     COMPUTE HEAVE FOR VARATION OF ELASTIC PARAMETERS.
C
      DO 20 J=1,NPR
      CD=J-1
      POR=PR+CD*DPR
      APOR=1.0-POR**2
      BPOR=(1.0-2.0*POR)*(1.0+POR)
      DO 17 I=1,NEL
      CEL=I-1
      EV(I)=ELAS+CEL*DELAS
      REBI(I)=((APOR*CA)-(BPOR*CB))/EV(I)
   17 CONTINUE
      IF(J.NE.1) GO TO 25
    5 FORMAT(1H1,1X)
   23 FORMAT(//,1X,'REBOUND POINT',I3,//,1X,'POIS. RATIO',30X,'MODULI')
C
C     ******************************************************************
C     OUTPUT REBOUND RESULTS FOR POINT "IRB(LI)".
C     ******************************************************************
C
      WRITE(6,24)(EV(I),I=1,NEL)
   25 WRITE(6,26)POR,(REBI(I),I=1,NEL)
   20 CONTINUE
      WRITE(6,29)
   14 CONTINUE
   24 FORMAT(11X,15(1X,F7.0))
   26 FORMAT(3X,F5.2,2X,15(3X,F5.3))
   29 FORMAT(7X,'NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES')
      STOP
      END
```

```
      SUBROUTINE RBNDC (CA,CB,IN)
C
C     ***************************************************************
C     COMPUTE ELASTIC HEAVE FOR REBOUND POINT "IN".
C     ***************************************************************
C
      DIMENSION RF(4),RG(4)
      COMMON /LOAD/A(100),B(100),C(100),D(100),E(100)
      COMMON /RBPD/IR(50),F(50),G(50),H(50)
      COMMON /LOLA/AA(100),BB(100,100),MM,NN
      DO 16 J=1,MM
      RP=A(J)/2.0
      SS=B(J)/2.0
      DA=ABS(C(J)-F(IN))
      DB=ABS(D(J)-G(IN))
      DO 18 NOT=1,NN
C        DETERMINE WHETHER THE EXCAVATION OF EXC. AREA J AND
C        SPECIFICALLY ITS NTH SOIL LAYER CONTRIBUTES TO THE TOTAL
C        REBOUND.
      IF(BB(NOT,J).LT.0.0001) GO TO 16
      Z=AA(NOT)-H(IN)
      IF(Z)14,15,15
   14 Z=0.00000
   15 CONTINUE
      DO 7 MR=1,4
      GO TO (1,2,3,4),MR
    1 W=DA+RR
      V=DB+SS
      GOTO5
    2 W=ABS(DA-RR)
      GOTO5
    3 V=ABS(DB-SS)
      GOTO5
    4 W=DA+RR
    5 IF(W.EQ.0.0.OR.V.EQ.0.0)GOTO12
      IF(W.LT.V)GOTO6
      R=W/V
      T=Z/V
      S=V
      GO TO 17
    6 R=V/W
      T=Z/W
      S=W
      GO TO 17
   12 RF(MR)=0.0
      RG(MR)=0.0
      GO TO 7
   17 Q=SQRT(R**2+T**2+1.0)
      QA=ALOG((Q+R)/(Q-R))
      QB=R*ALOG((Q+1.0)/(Q-1.0))
      RF(MR)=S*(QA+QB)
      IF(Z.EQ.0.0) GO TO 13
      VAL=R/(T*Q)
      RG(MR)=Z*ATAN(VAL)
      GO TO 7
```

```
   13 RG(MR)=0.0
    7 CONTINUE
      IF(DA.LE.RR.AND.DB.LE.SS) GO TO 8
      IF(DA.LE.RR) GO TO 9
      IF(DB.LE.SS) GO TO 10
C     CASE 1
      FE=RF(1)-RF(2)+RF(3)-RF(4)
      FG=RG(1)-RG(2)+RG(3)-RG(4)
      GOTO11
C     CASE 2
    9 FE=RF(1)+RF(2)-RF(3)-RF(4)
      FG=RG(1)+RG(2)-RG(3)-RG(4)
      GOTO11
C     CASE 3
   10 FE=RF(1)-RF(2)-RF(3)+RF(4)
      FG=RG(1)-RG(2)-RG(3)+RG(4)
      GOTO11
C     CASE 4
    8 FE=RF(1)+RF(2)+RF(3)+RF(4)
      FG=RG(1)+RG(2)+RG(3)+RG(4)
   11 CA=CA+BB(NOT,J)*FE
      CB=CB+BB(NOT,J)*FG
   18 CONTINUE
   16 CONTINUE
      CA=(CA/3.14159)*(1000.0/24.0)
      CB=(CB/3.14159)*(1000.0/24.0)
      RETURN
      END
```

COMPUTER RESULTS

```
REBOUND COMPUTATION - EXCAVATION AGT - SEPT. 13,1969

  SOIL PROFILE ¬  4 LAYER SYSTFM

    STRATUM ELEVATION  DEPTH    UNIT
            TOP BOUND.          WEIGHT

     LEC      2231.0     18.0   0.119
     TILL     2213.0     32.0   0.131
     SS&G     2181.0     10.0   0.125
     BEDR     2171.0    500.0   0.135
     NOTE ¬ ALL UNITS ARE IN TERMS OF KIPS AND FEET

  REBOUND POINT DATA

    POINT    X COORD. Y COORD. ELEVATION

      1       358.0    239.9     2186.5
      2       351.8    255.3     2186.5
      3       351.8    283.8     2186.5
      4       351.8    312.3     2186.5
      5       358.0    342.0     2186.5
      6       372.6    324.2     2196.0
      7       372.6    300.4     2196.0
      8       372.6    281.4     2196.0
      9       372.6    257.7     2196.0
     10       384.4    239.9     2195.0
     11       391.6    276.7     2197.7
     12       391.6    315.2     2197.7
     13       427.2    324.2     2200.0
     14       410.6    257.7     2198.5
     15       412.9    239.9     2201.0
     16       441.4    239.9     2205.0
     17       458.1    257.7     2198.5
     18       458.1    324.2     2198.5
     19       477.1    315.2     2197.7
     20       477.1    276.7     2197.7
     21       469.9    236.9     2205.0
     22       498.4    236.9     2205.0
     23       496.1    257.7     2198.5
     24       496.1    300.4     2198.5
     25       496.1    324.2     2198.5
     26       516.8    312.3     2202.5
     27       516.8    269.6     2205.0
     28       514.8    239.9     2205.0

  EXCAVATION DATA- EXCAVATION AGT - SEPT. 13,1969

    EXCAV. LENGTH  WIDTH  X COORD. Y COORD. BOT. ELEV.  LOAD

      1      79.6   210.0   310.2    241.0     2190.0    5.15
      2      85.5   170.0   227.7    261.0     2192.0    4.89
      3     100.0   170.0   134.9    261.0     2194.0    4.63
      4      65.0   266.0    52.4    164.0     2194.0    4.63
      5      52.0    48.0    58.9    291.0     2194.0    4.63
      6      24.8    25.0    72.5    327.5     2194.0    4.63
      7      10.0   181.0    14.9    174.0     2213.0    2.14
      8      65.0    31.0    52.4     15.5     2228.0    0.36
      9      72.0    34.0   121.0     17.0     2225.0    0.71
     10      32.0    26.0   173.0     21.0     2221.0    1.19
     11      32.0    26.0   205.0     21.0     2213.0    2.14
     12      32.0    26.0   237.0     21.0     2205.0    3.19
     13      32.0    26.0   269.0     21.0     2197.0    4.24
     14      26.0    26.0   298.0     21.0     2200.0    3.84
     15      39.0    26.0   330.5     21.0     2210.0    2.53
     16      56.0   102.0   298.0     85.0     2190.0    5.15
     17      24.0   102.0   338.0     85.0     2221.0    1.19
     18      57.5   141.0   113.3    105.5     2194.0    4.63
     19      42.5    54.0   163.5     62.0     2194.0    4.63
     20      85.5    54.0   227.7     62.0     2192.0    4.89
     21     128.0    87.0   206.0    132.5     2190.0    5.15

  MIN. DEPTH OF FXC. 2228.0
  MAX. DEPTH OF EXC. 2190.0

  IN THIS RUN DEL= 1.0
```

LOAD CONFIGURATION

LAYFR	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
2230.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2229.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2228.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2227.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2226.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2225.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2224.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2223.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2222.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2221.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
2220.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12
2219.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12
2218.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12
2217.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12
2216.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12
2215.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12
2214.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12
2213.0	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.0	0.0	0.12	0.12	0.12	0.12	0.12	0.12	0.0	0.12	0.12	0.12	0.12
2212.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.13	0.13	0.13	0.0	0.13	0.13	0.13	0.13
2211.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.13	0.13	0.13	0.0	0.13	0.13	0.13	0.13
2210.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.13	0.13	0.13	0.0	0.13	0.13	0.13	0.13
2209.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2208.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2207.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2206.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2205.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2204.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2203.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2202.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2201.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2200.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.13	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2199.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2198.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2197.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2196.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2195.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2194.0	0.13	0.13	0.13	0.13	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.13	0.13	0.13	0.13
2193.0	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.0	0.0	0.13	0.13
2192.0	0.13	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.0	0.0	0.13	0.13
2191.0	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.0	0.0	0.0	0.13
2190.0	0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.13	0.0	0.0	0.0	0.0	0.13

```
REBOUND POINT  1

POIS. RATIO                                   MODULI
               30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000. 160000. 170000.
    0.30       2.747   2.060   1.648   1.373   1.177   1.030   0.916   0.824   0.749   0.687   0.634   0.589   0.549   0.515   0.485
    0.40       2.557   1.918   1.534   1.279   1.096   0.959   0.852   0.767   0.697   0.639   0.590   0.548   0.511   0.479   0.451
    0.50       2.311   1.733   1.386   1.155   0.990   0.866   0.770   0.693   0.630   0.578   0.533   0.495   0.462   0.433   0.408
       NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES

REBOUND POINT  2

POIS. RATIO                                   MODULI
               30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000. 160000. 170000.
    0.30       2.848   2.136   1.709   1.424   1.221   1.068   0.949   0.854   0.777   0.712   0.657   0.610   0.570   0.534   0.503
    0.40       2.655   1.992   1.593   1.328   1.138   0.996   0.885   0.797   0.724   0.664   0.613   0.569   0.531   0.498   0.469
    0.50       2.404   1.803   1.442   1.202   1.030   0.901   0.801   0.721   0.656   0.601   0.555   0.515   0.481   0.451   0.424
       NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES

REBOUND POINT  3

POIS. RATIO                                   MODULI
               30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000. 160000. 170000.
    0.30       2.730   2.048   1.638   1.365   1.170   1.024   0.910   0.819   0.745   0.683   0.630   0.585   0.546   0.512   0.482
    0.40       2.545   1.909   1.527   1.273   1.091   0.955   0.848   0.764   0.694   0.636   0.587   0.545   0.509   0.477   0.449
    0.50       2.304   1.728   1.383   1.152   0.988   0.864   0.768   0.691   0.628   0.576   0.532   0.494   0.461   0.432   0.407
       NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES

REBOUND POINT  4

POIS. RATIO                                   MODULI
               30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000. 160000. 170000.
    0.30       2.547   1.911   1.528   1.274   1.092   0.955   0.849   0.764   0.695   0.637   0.588   0.546   0.509   0.478   0.450
    0.40       2.374   1.781   1.425   1.187   1.018   0.890   0.791   0.712   0.648   0.594   0.548   0.509   0.475   0.445   0.419
    0.50       2.149   1.611   1.289   1.074   0.921   0.806   0.716   0.645   0.586   0.537   0.496   0.460   0.430   0.403   0.379
       NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES

REBOUND POINT  5

POIS. RATIO                                   MODULI
               30000.  40000.  50000.  60000.  70000.  80000.  90000. 100000. 110000. 120000. 130000. 140000. 150000. 160000. 170000.
    0.30       2.160   1.620   1.296   1.080   0.926   0.810   0.720   0.648   0.589   0.540   0.499   0.463   0.432   0.405   0.381
    0.40       2.007   1.505   1.204   1.004   0.860   0.753   0.669   0.602   0.547   0.502   0.463   0.430   0.401   0.376   0.354
    0.50       1.808   1.356   1.085   0.904   0.775   0.678   0.603   0.542   0.493   0.452   0.417   0.387   0.362   0.339   0.319
       NOTE - MODULI IN PSI.; SETTLEMENT IN INCHES
```

APPENDIX H

ELASTIC SETTLEMENTS OF UNIFORMLY LOADED RECTANGULAR FOOTINGS ON A HOMOGENEOUS, ISOTROPIC, SEMI-INFINITE BODY

Solution Description

The solution presented here is based on the standard equations of elasticity for the settlement of a corner of a rectangular, flexible, uniformly loaded area. The soil medium is considered to be a homogeneous, isotropic, semi-infinite mass. Superposition techniques are used to enable computation of settlements at footing centroids due to multiple footing foundation systems.

The equations used in the solution are identical with those described earlier in Appendix E. This computer program uses a single load set and calculates settlements for a single value of Young's modulus and Poisson's ratio. Settlements are computed at the centroids of all input footings. If settlements are desired at other locations, these are specified as "dummy" footings which have a precise coordinate location but have unit length and width and zero load. These "dummy" footings are included at the end of the footing load data set.

The restrictions given in Appendix E for definition of coordinate axes relative to footings are applicable here as well.

Operational Characteristics

Maximum number of footings $\leq$ 50.

Required computer storage = 100 K.

Input

Card (A) Deformation Parameters.

(1X,4X,F10.0,F5.2)

Columns	
6-15	Young's modulus in psi
16-20	Poisson's ratio.

Card (B) Title, Load Data Set.

(17A4)

Card (C) Number of Footings.

(2X,I3)

Columns	
3-5	Number of footings.

Card (D) Footing Load Data - number of cards must equal the value specified on Card C.

(I3,1X,5F7.1)

Columns	
1-3	Footing identification number
5-11	X - coordinate of footing in feet
12-18	Y - coordinate of footing in feet
19-25	Length of footing in x-direction in feet
26-32	Length of footing in y-direction in feet
33-39	Footing load in kips.

Output

The input data and the results of the calculation are printed out on a single table as shown in the accompanying program listing.

Additional Remarks

For a specific geometric configuration of footings and points at which settlements are computed, there is the possibility that error messages may be given. The method for correcting this situation was presented in Appendix E.

```
 NOV 68)                            OS/360  FORTRAN H

CMPILER OPTIONS - NAME=  MAIN,OPT=02,LINECNT=59,SOURCE,EBCDIC,NOLIST,NODECK,L
C
C     ******************************************************************
C     ELASTIC SETTLEMENT OF RECTANGULAR FOOTINGS ON A SEMI-INFINITE
C     PLANE -REQUIRES- SINGLE LOAD SET, YOUNGS MODULUS AND POISSONS
C     RATIO
C     ******************************************************************
C
      DIMENSION X(50),Y(50),A(50),B(50),P(50),Q(50),SETI(50),TITLE(17),
     1IR(50)
C
C     ******************************************************************
C     READ DATA
C     ******************************************************************
C
      READ(5,15)E,U
      READ(5,90)(TITLE(I),I=1,17)
      READ(5,10)N
      READ(5,11)(IR(J),X(J),Y(J),A(J),B(J),P(J),J=1,N)
   10 FORMAT(1X,1X,I3)
   11 FORMAT(I3,1X,5F7.1)
   15 FORMAT(1X,4X,F10.0,F5.2)
   90 FORMAT(17A4)
C
C     ******************************************************************
C     INITIALIZE AND COMPUTE FOOTING CONTACT PRESSURE
C     ******************************************************************
C
      DO30J=1,N
      SETI(J)=0.0
      Q(J)=P(J)/(A(J)*B(J))
   30 CONTINUE
C
C     ******************************************************************
C     COMPUTE SETTLEMENT
C     ******************************************************************
C
      DO32I=1,N
      IF(P(I).EQ.0.0)GOTO32
      RR=A(I)/2.0
      SS=B(I)/2.0
      DO31J=1,N
      DA=ABS(X(I)-X(J))
      DB=ABS(Y(I)-Y(J))
      FI=FE (RR,SS,DA,DB)
      SETI(J)=SETI(J)+(Q(I)/0.144)*((1.0-U**2)/E)*FI*12.0
   31 CONTINUE
   32 CONTINUE
C
C     ******************************************************************
C     OUTPUT INPUT DATA AND RESULTS
C     ******************************************************************
C
      WRITE(6,17)E,U
      WRITE(6,90)(TITLE(I),I=1,17)
      WRITE(6,14)
      WRITF(6,13)
      WRITE(6,12)(IR(J),X(J),Y(J),A(J),B(J),P(J),Q(J),SETI(J),J=1,N)
```

```
      12 FORMAT(1X,6X,I2,2X,F7.1,1X,F7.1,2X,F7.1,1X,F7.1,3X,F7.1,5X,F5.2,14
        1X,F5.2)
      13 FORMAT(11X,7HX (FT.),1X,7HY (FT.),2X,7HA (FT.),2X,7HB (FT.),2X,8HP
        1 (KIPS),3X,8HQ (KSF.),'              INCHES',/)
      14 FORMAT(/,6X,'FTG.   COORDINATES       DIMENSIONS       LOAD      PRE
        1SSURE          SETTLEMENT')
      17 FORMAT(1H1,6X,'MODULUS OF ELASTICITY =',F10.0,1X,'PSI.   POISSONS
        1RATIO = ',F5.2,//)
         STOP
         END

: COMPILATION ******
```

```
 NOV 68)                                     OS/360  FORTRAN H

)MPILER OPTIONS - NAME=  MAIN,OPT=02,LINECNT=59,SOURCE,EBCDIC,NOLIST,NODECK,L
          FUNCTION FE (X,Y,A,B)
    C
    C     *****************************************************************
    C     CALCULATE INFLUENCE COEFFICIENT COMPONENTS
    C     *****************************************************************
    C
          DIMENSIONRF(4)
          DO7M=1,4
          GOTO(1,2,3,4),M
        1 W=A+X
          V=B+Y
          GOTO5
        2 W=ABS(A-X)
          GOTO5
        3 V=ABS(B-Y)
          GOTO5
        4 W=A+X
        5 IF(W.EQ.0.0.OR.V.EQ.0.0)GOTO12
          IF(W.LT.V)GOTO6
          R=W/V
          RF(M)=V*FG (R)
          GOTO7
        6 R=V/W
          RF(M)=W*FG (R)
          GOTO7
       12 RF(M)=0.0
        7 CONTINUE
          IF(A.LE.X.AND.B.LE.Y)GOTO8
          IF(A.LE.X)GOTO9
          IF(B.LE.Y)GOTO10
    C     CASE 1
          FE=RF(1)-RF(2)+RF(3)-RF(4)
          GOTO11
    C     CASE 2
        9 FE=RF(1)+RF(2)-RF(3)-RF(4)
          GOTO11
    C     CASE 3
       10 FE=RF(1)-RF(2)-RF(3)+RF(4)
          GOTO11
    C     CASE 4
        8 FE=RF(1)+RF(2)+RF(3)+RF(4)
       11 RETURN
          END

= COMPILATION ******
```

```
NOV 68)                              OS/360  FORTRAN H

OMPILER OPTIONS - NAME=  MAIN,OPT=02,LINECNT=59,SOURCE,EBCDIC,NOLIST,NODECK,L
           FUNCTION FG (R)
    C
    C      ****************************************************************
    C      ARITHMETIC FUNCTION
    C      ****************************************************************
    C
           A=SQRT(R**2+1.0)
           B=R*ALOG((1.0+A)/R)
           C=ALOG(R+A)
           FG=(B+C)/3.1416
           RETURN
           END

F COMPILATION ******
```

```
       MODULUS OF ELASTICITY =    40590. PSI.  POISSONS RATIO =  0.40

   LOAD DATA - AVORD ARMS BLDG. CONSTRUCTION COMPLETE

       FTG.    COORDINATES       DIMENSIONS        LOAD      PRESSURE             SETTLEMENT
             X (FT.) Y (FT.)  A (FT.)  B (FT.)  P (KIPS)   Q (KSF.)               INCHES

         1    201.5    46.5     16.0      8.0    1942.0     15.17                  0.74
         2    159.0    50.0      8.0      8.0    1496.0     23.38                  0.92
         3    138.0    50.0      8.0      8.0    1376.0     21.50                  0.91
         4     75.0    50.0      8.0      8.0    1276.0     19.94                  0.87
         5     12.0    44.0      8.0     13.0    1900.0     18.27                  0.74
         6     33.0    68.0      7.0      5.5    1017.0     26.42                  0.87
         7     54.0    65.5      8.0      8.0    1336.0     20.88                  0.92
         8     75.0    68.0      7.0      5.5    1017.0     26.42                  1.02
         9    106.5    63.0      7.0      2.5     454.0     25.94                  0.90
        10    159.0    65.5      9.0      9.0    1440.0     17.78                  0.94
        11    180.0    80.0      7.0      7.0    1100.0     22.45                  0.89
        12    138.0    80.0      8.0      8.0    1376.0     21.50                  1.02
        13    117.0    82.5      8.0      8.0    1172.0     18.31                  0.96
        14     94.3    82.6      8.0      8.0    1200.0     18.75                  0.96
        15     75.0    80.0      7.0      7.0    1156.0     23.59                  1.00
        16     75.0    98.0      8.0      8.0    1300.0     20.31                  0.88
        17    117.0    98.0      8.0      8.0    1292.0     20.19                  0.90
        18    159.0    98.0      8.0      8.0    1496.0     23.38                  0.92
        19    180.0    98.0     10.0     10.0    1450.0     14.50                  0.79
        20    201.5    56.5      8.0      8.0    1056.0     16.50                  0.74
        21    180.0    50.0     10.0     10.0    1450.0     14.50                  0.79
        22    117.0    50.0      7.0      7.0    1132.0     23.10                  0.91
        23     96.0    50.0      7.0      7.0    1120.0     22.86                  0.90
        24     54.0    50.0      8.0      8.0    1212.0     18.94                  0.83
        25     33.0    50.0      8.0      8.0    1344.0     21.00                  0.81
        26     12.0    56.5      8.0      8.0    1018.0     15.91                  0.71
        27     94.0    67.0      7.5     11.0     984.0     11.93                  0.91
        28    119.0    67.0      7.5     11.0     984.0     11.93                  0.92
        29    138.0    68.0      7.0      5.5    1017.0     26.42                  1.05
        30    180.0    68.0      7.0      7.0    1100.0     22.45                  0.89
        31     84.0    70.5      8.0      8.0    1298.0     20.28                  1.03
        32    129.0    70.5      8.0      8.0    1298.0     20.28                  1.05
        33     33.0    80.0      7.0      5.5    1017.0     26.42                  0.87
        34     54.0    82.5      8.0      8.0    1336.0     20.88                  0.92
        35    159.0    82.5      9.0      9.0    1440.0     17.78                  0.94
        36     12.0    91.5      8.0      8.0    1018.0     15.91                  0.71
        37     12.0   104.5      8.0     13.0    1900.0     18.27                  0.74
        38     33.0    98.0      8.0      8.0    1344.0     21.00                  0.81
        39     54.0    98.0      8.0      8.0    1212.0     18.94                  0.83
        40     96.0    98.0      7.0      7.0    1152.0     23.51                  0.90
        41    138.0    98.0      8.0      8.0    1376.0     21.50                  0.91
        42    201.5    91.5      8.0      8.0    1056.0     16.50                  0.74
        43    201.5   101.5     16.0      8.0    1942.0     15.17                  0.74
```