# A High Capacity Load Test for Deep Bored Piles

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SUMMARY.- The equipment and field procedures used in an end-bearing load test for bored piles founded at depths of from 30 to 50 feet in strong marine deposits are described. Good correlation was obtained between the calculated and actual intensities of applied bearing pressure at failure. Brief reference is made to secondary pull-out tests on the grouted cable anchors used as loading reactions in the pile test. The application of test results in the final design of piles is also described.

## I.- INTRODUCTION

The first stage of construction of the Downtown Redevelopment Scheme in Auckland, New Zealand, commenced in late 1968. The site covers two city blocks and approximately 10 acres; it adjoins the waterfront and includes the commercial area bounded by Queen, Quay and Hobson Streets and Customs Street West. The redevelopment is being undertaken in three stages by the international consortium of Mainline-Dillingham-Fletcher.

Buildings constructed in the first stage include the 21-storey Air New Zealand House, the 14-storey Auckland Travelodge motor-hotel, an extensive multi-storey carparking station for 1700 vehicles and a two-storey tourist centre. All buildings except the tourist centre have full or partial basements. Apart from the parking station which is generally of precast, prestressed shear-wall construction, the buildings are of conventional reinforced concrete construction. All structures were designed to the requirements of the New Zealand code for earthquake resistant construction. Column loads (for dead plus real live loading conditions) ranged from 400 to 2300 kips; these static loads were increased by up to 50 percent for superimposed seismic loadings.

A combination of weak surface soils and structural factors required that the buildings be supported on piles founded in the strong Waitemata series sedimentary deposits which underlie the site at depths of from 20 to 40 feet. Investigation studies had indicated that the three major buildings could be supported satisfactorily and most economically on 190 large diameter drilled-and-cast-in-place piles designed to derive their full capacity from end bearing. Side wall adhesion was neglected as piles in some areas were only expected to penetrate about 7 feet into the dense Waitemata series deposits. Depending on the intensity of applied loading, the piles could be either straight shafted or belled at their bases. The local code ordinance covering foundation design at that time required that applied end-bearing pressures in Waitemata series deposits could not exceed 12 tons per square foot. Accordingly, in view of the number of piles and high order of structural loading involved, it was decided to undertake a comprehensive load test to verify the indicated favourable pile capacities derived by engineering analyses based on laboratory testing results and to persuade the local authority to approve a variation of the code limitations.

## II.- DIMENSIONS

All dimensions of quantities are expressed in terms of feet - pounds - hours, or multiples thereof. "Tons" refer to long tons (2240 pounds). Pressures are quoted as tons per square foot (TSF) or pounds per square foot (PSF). Kips equal 1000 pounds.

## III.-SITE CONDITIONS

Prior to the settlement of Auckland in 1840, part of the site extended on to a flat tidal shelf bordering the original shoreline. The area was initially developed by several stages of reclamation between 1859 and 1923. Research has disclosed that old rock bunds, quay walls, piling and part of a former graving dock were filled over in successive reclamation stages.

Subsoil conditions at the site were explored in two investigations involving 30 borings of from 50 to 82 feet in depth. These studies proved that the site is consistently underlain by:-

(a) surface reclamation fills;

(b) recent marine sediments, and

(c) Waitemata series deposits

The surface fills vary from 14 to 23 feet in depth and consist of weak, poorly compacted intermixed soil, rock and harbour bed deposits. The fills generally overlie up to 20 feet of recent marine sediments comprising soft to moderately firm clays and silts which contain random lenses of sand. At depths of from 18 to 39 feet below present ground level, the surface fills or marine sediments are underlain by dense Waitemata series sediments which extend to the depths explored. These deposits comprise alternating strata of variably cemented sandstones and siltstones which have the appearance, texture and strength characteristics of weak rock. Approximately equal volumes of sandstone and siltstone were encountered in the borings although their distribution tended to vary throughout the site.

Near-continuous cores were recovered from selected borings in the Waitemata series deposits. A large number of unconfined compression, triaxial and direct shear tests performed on samples of the Waitemata series sandstones and siltstones indicated their strength parameters, for purposes of design (Appendix), are of the following orders:-

sandstone: $c_u = 2600$ PSF $\phi_u = 35^o$
siltstone: $c_u = 1000$ PSF $\phi_u = 30^o$

The simplified log of a boring located adjacent to the load test is reproduced in figure 1 together with a summary of the unconfined compression tests performed on selected continuous core samples of the Waitemata series deposits recovered from this hole.

The ground water level, measured in numerous exploratory borings throughout the site, consistently stabilised at depths of from $7\frac{1}{2}$ to 8 feet below the present surface. Although the site is within several

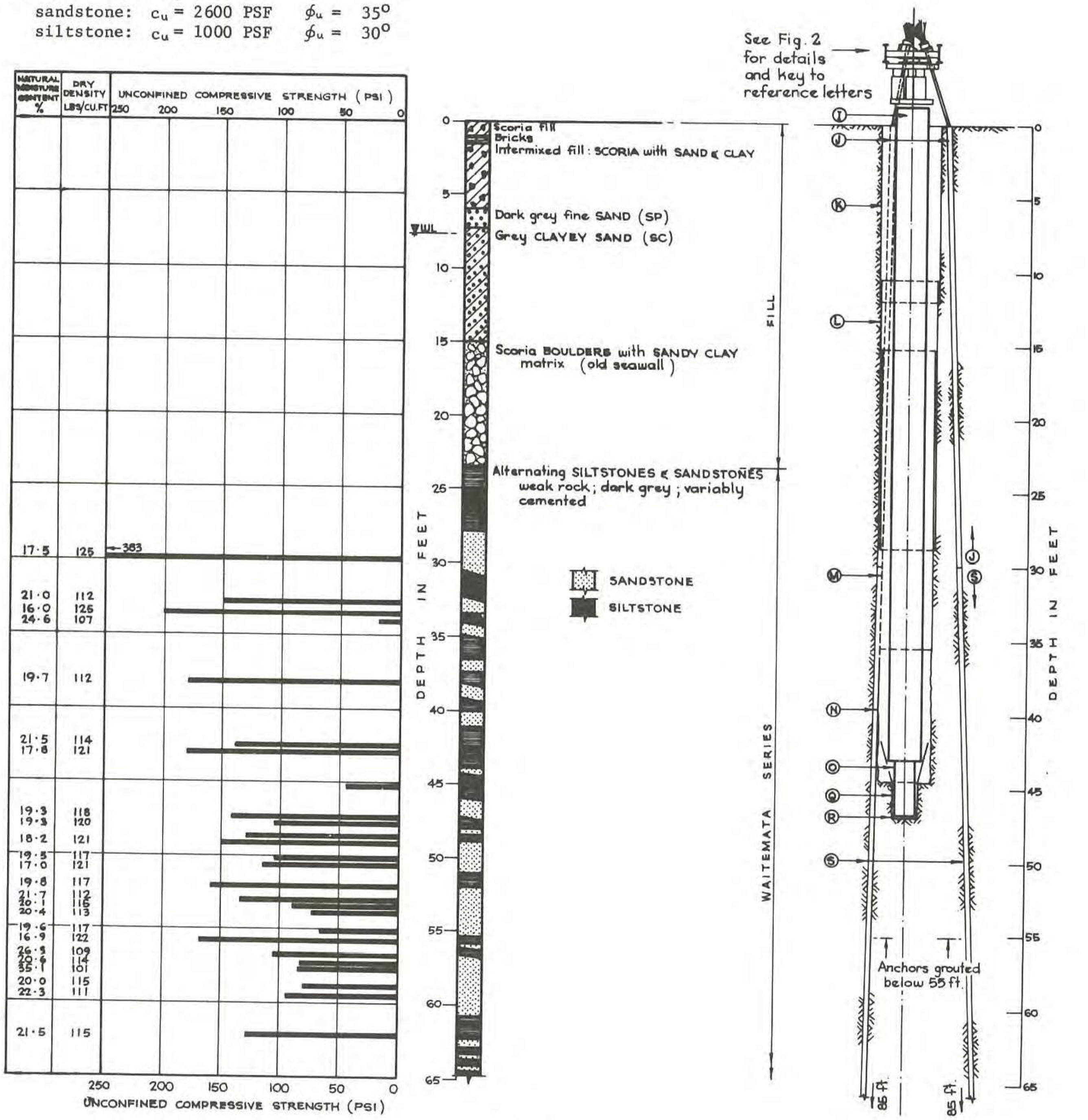


Fig.1

BORING LOG, TYPICAL STRENGTH DATA AND ELEVATION OF TEST PILE hundred feet of the harbour, continuous ground water level observations over several tide cycles did not reveal any significant fluctuation of the ground water table due to tidal influence.

## IV.- PLANNED SCOPE OF TEST

The purpose and scope of the load test were to determine the following information for use in design and construction:-

(a) A basis for estimating the load-settlement performance of deep end-bearing drilled piles founded in Waitemata series deposits of average strength;

(b) The relationship between the apparent yield and ultimate bearing values of the Waitemata deposits, as applicable to drilled piles;

(c) Correlation between the theoretical and actual ultimate bearing capacity of an individual pile;

(d) Construction problems involved in drilling large diameter shafts through the variable fills and natural deposits at the site; and

(e) Ultimate capacities of grouted cable anchors in Waitemata series deposits.

## V.- THEORETICAL CAPACITY OF TEST PILE

In order to assure that base failure would occur under test loading, it was essential that the dimensions of the transfer pile and bearing plate be correctly proportioned and that adequate reaction capacity be provided. For these reasons, the theoretical ultimate capacities of various sizes of end-bearing piles were calculated prior to undertaking the load test. These calculations were based on the Terzaghi "General Shear Failure" formula and assumed strength parameters for the supporting Waitemata series deposits as determined from prior laboratory studies.

As described below, the test pile was founded at a depth of 47 feet in predominantly sandstone deposits and had an end-bearing area of 2.05 square feet. The ultimate capacity of this pile was calculated to be 370 tons; this capacity was equivalent to an end-bearing pressure of 181 TSF.

The approximate yield point strength of the Waitemata series sandstones was assessed from the results of a high-capacity consolidation test. A sample of cored sandstone recovered from a depth of 50 feet in the boring adjoining the load test site was loaded to pressures in excess of 500 TSF. The resulting plot of applied pressure against percentage consolidation gave straight line relationships from 4 to 64 TSF and from 128 to 512 TSF. The assumed yield occurred at 95 TSF corresponding to 5 percent vertical strain. It was concluded that between pressures of 64 and 128 TSF the natural cementing bonds progressively yielded and, at higher pressures, the sandstone acted as a non-cemented material. It was further concluded that apparent yielding of the founding materials could be expected to occur under applied test pressures of from 90 to 110 TSF.

## VI.- TEST PILE INSTALLATION

The load test assembly was selected from several alternatives on the basis that it appeared to satisfy all prime objectives of the study. The basic configuration was planned to determine the end-bearing capacity of a free-standing load transfer pile located within an oversized, partially cased shaft. Loading was applied at the ground surface by hydraulic jacking against a steel frame anchored by grouted cables. Details of the shaft, pile, loading frame, anchor cables and measuring equipment are illustrated in figures 1, 2, 3 and 4.

Supplementary comments on the assembly, including its sequence of installation, are as follows:-

Three symmetrically spaced, 4-inch diameter anchor holes were drilled on a 5-foot diameter circle centred on the selected test location. The holes were raked at an inclination of 1 horizontal to 30 vertical and extended to depths of approximately 85 feet. These holes were cased through the surface filling to a depth of 30 feet. On completion of drilling, the anchor holes were reamed and flushed clean, the cables were inserted and their lower 30 foot lengths were pressure grouted. Minor difficulties were experienced in cleaning the base of one hole due to seepage through the casing seal; in this case, the cable terminated 4 feet higher than those in the other anchor holes. The anchor cables consisted of twelve ½-inch diameter high tensile wire strands. In the grouted section of the cable, the individual strands were held symmetrically on a 3-inch diameter pitch circle by two types of spacers, located alternately at 3-foot intervals. The upper ends of the cables terminated in anchor grips at the surface load frame. The cables were installed and grouted 18 days prior to loading.

Following installation of the cable anchors, a partially cased, truly plumb shaft was drilled to a depth of 47 feet. As drilling proceeded, the uppermost section of the shaft was lined with a 12-foot length of 48-inch diameter casing. After this starter casing had been installed, the next section of shaft was advanced and lined with a prepared length of 46-inch diameter "core barrel" casing to a depth of 29 feet. The leading edge of this latter casing was tipped with tungsten steel cutting teeth to assist drilling through the basalt and scoria boulders encountered between depths of from 15 to 25 feet. The "core barrel" casing was advanced through the boulders by rotary drilling methods and had to be withdrawn periodically for dressing or replacing of the cutting teeth. A third section of 42-inch diameter casing was then inserted and advanced to a depth of 35½ feet, some 12 feet below the surface of the Waitemata series deposits, to ensure a seal against peripheral seepage. The shaft was then pumped dry and advanced to a depth of 44 feet by a combination of machine drilling and hand excavation. Although an effective seal was obtained at the base of the inner casing, a minor inflow of ground water occurred between the 42-inch and 46-inch diameter casings. Although attempts to seal the seepage between the casings were not fully effective, the inflow was reduced to less than several cubic feet per hour. Following completion of the main shaft, a concentric 20-inch diameter cased shaft was drilled from depths of 44 to 47 feet. The casing for this secondary shaft extended to within 9 inches of the base. The bottom of the shaft was cleaned by hand trimming and then covered with a 1-inch thick layer of sand.

A special precast concrete pile was manufactured to serve as the load transfer unit. This member consisted of a standard 26-inch square, prestressed concrete pile with a 4-foot long, 16-inch diameter reinforced concrete extension cast monolithically at its base. A 19$\frac{5}{8}$ -inch diameter, 2-inch thick steel plate was bolted concentrically to the base of the pile extension. A number of small fillets were welded to the projecting upper face of the bearing plate to prevent binding of the pile with shaft casing during withdrawal.

The pile was installed by hoisting from two lifting lugs. The insertion of the base plate extension into the close-fitting base shaft was assisted by the provision of four steel guides welded to and projecting from the base shaft casing. The vertical alignment of the load transfer pile was established by

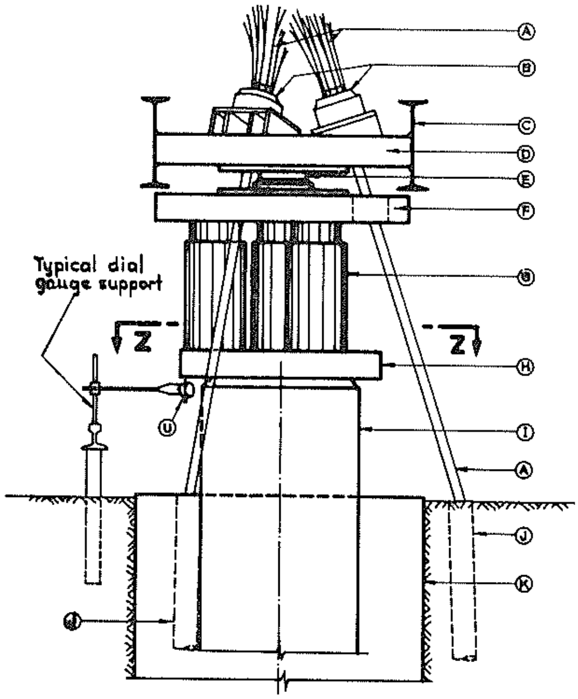


Fig. 2 : LOADING FRAME DETAILS

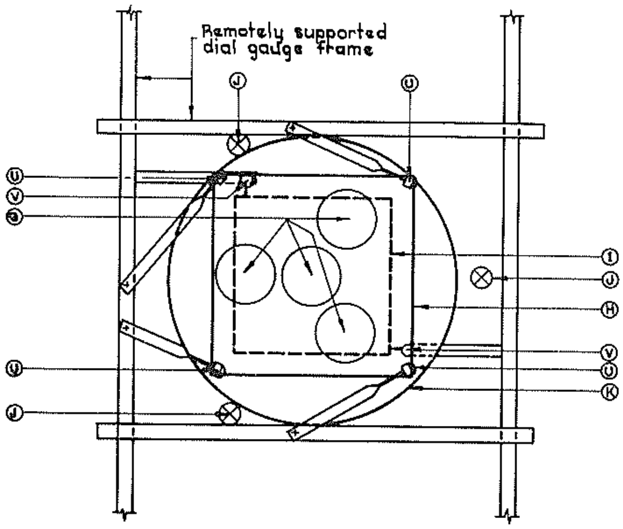


Fig. 3 : PLAN ON SECTION Z - Z

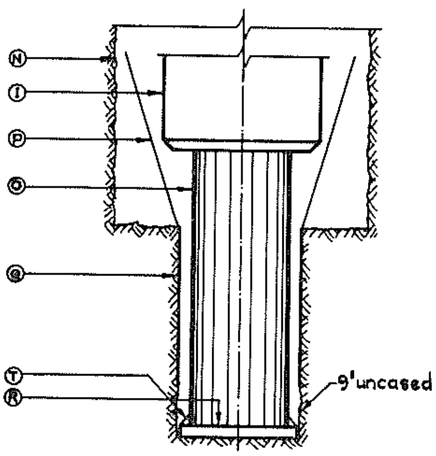


Fig.4 : PILE BASE DETAILS

KEY TO FIGS 1, 2, 3 and 4:-

| | |
|---|---|
| A | Standard Freyssinet cables (12 nos ½" strands) |
| B | Cable anchor heads |
| C | Load frame support (15" x 5" x 42 lb RSJ$^{S}$) |
| D | 42" x 42" x 5" steel plate with aligned bearing plates for cable anchor heads |
| E | Hemispherical bearing |
| F | 42" x 42" x 4½" steel plate |
| G | 4 nos 100 ton hydraulic jacks |
| H | 33" x 33" x 4½" steel plate |
| I | 26" x 26" hollow-cone, pretensioned, prestressed concrete pile |
| J | 4" diam cased anchor cable holes |
| K | 48" diam cased shaft |
| L | 46" diam cased shaft |
| M | 42" diam cased shaft |
| N | 42" diam uncased shaft |
| O | 16" diam reinforced extension cast monolithically with pile |
| P | 4 nos steel angles to guide pile into shaft |
| Q | 20" diam cased shaft |
| R | 19$\frac{5}{8}$" - diam steel plate, 2" thick |
| S | 4" diam uncased anchor cable shafts |
| T | ¼" thick fillet plates |
| U | dial gauges to record vertical movement |
| V | dial gauges to measure horizontal movement |

Figs. 2, 3 and 4

TEST PILE DETAILS

plumbing down the central inspection core of the pile. A loading frame of the type detailed in figures 2 and 3 was installed between the load transfer pile and the cable anchorages. The four hydraulic jacks and concentric hemispherical seating were positioned by precise surveying. The jacks used were idential and had been specifically calibrated for this project. They were coupled to a common manifold. During erection, the load frame was supported by large greased timbers. A heavy timber frame (not illustrated) was erected to resist the horizontal reaction of the steeply-inclined anchors and to prevent contact between the cables and pile head assembly. The exposed sections of the anchor cables which extended through the load frame were sheathed as a safety precaution. Rigid, remotely-supported horizontal steel rails were erected in a square around the top of the test pile as a datum for determining its movement under test. Four dial gauges were installed to measure vertical deflections and two to determine lateral movements of the pile.

VII.-LOADING SEQUENCE

The sequence of applied loading, determined after a review of selected published data relating to similar types of test (references 1 and 2), were planned to provide information on:-

(a) Settlement performance of an individual pile subjected to pressures equivalent to those imposed by future gravity loads;

(b) Yield-point bearing pressure of the deposits underlying the pile; and

(c) Ultimate bearing pressure intensity of the supporting foundation mass

The weight of the transfer pile and loading frame were calculated to impose a base pressure of $5\frac{1}{2}$ TSF, or approximately 4 TSF in excess of the natural overburden pressure at the test depth. This initial net pressure has been neglected in all test results; thus "applied pressure" equals the actual load applied to the head of the pile divided by its base-end bearing area.

The initial stage of testing comprised a maintained load test to overcome seating problems and provide load-settlement data for future piles subjected to normal gravity loads. The applied pressure was increased in 4 TSF increments at 24 hour intervals. The pile was loaded to a maximum applied pressure of

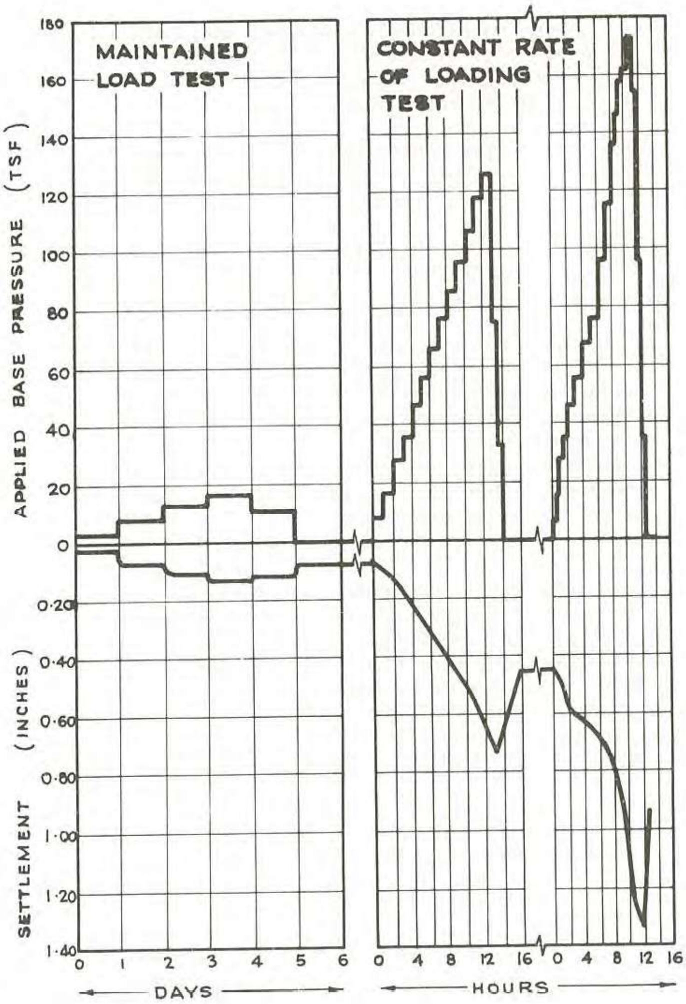


Fig. 5 : LOADING - SETTLEMENT HISTOGRAM

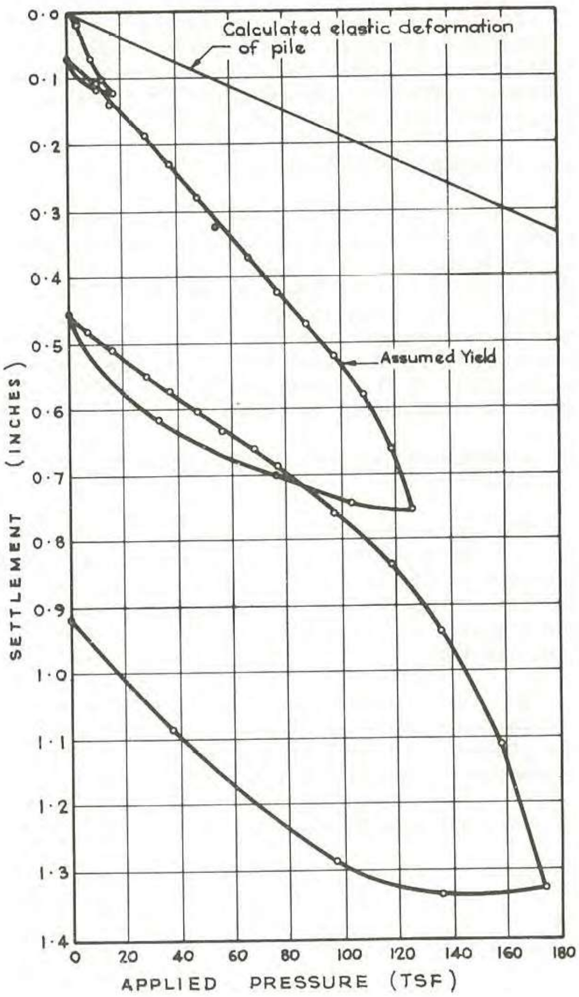


Fig. 6 : PRESSURE VS SETTLEMENT

16 TSF and then unloaded in two 24 hour cycles. During each increment of loading, deflection readings were taken at intervals identical to those of a normal consolidation test to determine the time-rate characteristics under load.

The second stage of loading, undertaken after a three-day break, consisted of a constant rate of loading test to assess the yield point bearing intensity. The applied pressure was increased in 10 TSF increments at 1 hour intervals until the plot of applied pressure against deflection and the rate of deflection in each loading cycle indicated that the yield point of the bearing material had been attained. The maximum applied pressure for this phase of testing was 130 TSF and the pile was unloaded in three, half-hour stages.

The third stage of testing was commenced after a further two day break. This test, to determine the ultimate bearing capacity of the foundation deposits, repeated the procedures of the constant rate of loading test. The applied pressure was increased in 10 TSF increments each hour to 80 TSF and thereafter in 20 TSF, hourly increments. The maximum applied pressure attained was 175 TSF at which point the pile yielded continuously under constant loading. The pile was then unloaded in four, half-hour intervals.

The results of the three loading stages are illustrated graphically in figures 5 and 6. The indicated settlements are the average of the four vertical deflection measurements recorded by dial gauges at each corner of the pile.

## VIII.-ANCHOR CABLE TESTS

Each of the three grouted cable anchors was subjected to loading tests subsequent to the completion of the pile test. An axial tensile load was applied individually to each cable by hydraulic jacks bearing against a surface grillage. Each cable was loaded to 170 tons or approximately 80 percent of the ultimate tensile strength of the cable. At these loads, no yielding of the anchor could be detected and the tests were abandoned for reasons of safety.

## IX.-CORRELATION OF THEORETICAL AND ACTUAL PILE CAPACITIES

The calculated ultimate capacity of the pile used in the load test was equal to an end-bearing pressure of 181 TSF. This value agrees remarkably (or fortuitously) closely with the measured ultimate bearing pressure of between 170 and 180 TSF under test loading.

The above capacity was based on shear test values available at the time of planning the load test. The ultimate strength parameters adopted for the predominantly sandstone basal deposits were:-

apparent cohesion $c_u = 2150$ PSF

apparent angle of internal friction $\phi_u = 40^\circ$ Direct Shear.

## X.- PILE DESIGN

In view of the confirming evidence provided by the load test, design capacities for bored piles at the site were evaluated by the Terzaghi "General Shear Failure" formula. It was assumed in analyses that piles may be underlain by weaker siltstones of the Waitemata series. The pile capacities were determined solely on the basis of end-bearing support and no allowance was made for frictional sidewall support on pile shafts by the soils above the base. Accordingly, and with the agreement of the local authority, drilled piles were proportioned to impose the following maximum end-bearing pressures:-

| | |
|---|---|
| Real loads: (dead plus permanently-applied live loads) | 15 TSF |
| Design gravity loads: (dead plus code live loads) | 20 TSF |
| Total design loads: (including seismic or wind loads) | 24 TSF |

These pressures were assessed to include an absolute minimum factor of safety of 2 with respect to failure under total design loading conditions. The equivalent theoretical factor of safety for real loads is in excess of 2.5. If the effects of sidewall friction were included, these factors would be considerably higher. The factor of safety for the sandstone deposits, as indicated by the pile load test results, is in excess of 5.

For contractural purposes, the founding depths of piles were selected directly from the correlated boring log data. Factors which governed the selection of pile depths included:-

(a) The random occurrence of weaker siltstones within the critical influence zone of two-base diameters of depth below piles; and

(b) Adequate depths below the surface of the Waitemata series deposits in which to found and bell the bases of bored piles.

In order to confirm selected founding depths, proof borings were drilled at approximately 50-foot centres in building areas ahead of piling construction. These borings were continuously cored to depths of at least two base diameters below the tentatively selected founding elevations. If siltstone deposits having equivalent triaxial deviator strengths of less than 50 pounds per square inch were encountered within this zone, the piles were deepened accordingly.

The total settlements of the multi-storey structures founded in this manner were conservatively estimated to be on the maximum order of 1 inch. Differential settlements between adjacent columns were not expected to exceed ½ inch. Approximately 80 percent of any settlement which did occur was expected to take place during construction on the initial application of permanent loads.

## XI.- CONCLUSIONS

(a) The decision to undertake this load test was vindicated by the satisfactory results and correlation achieved, notwithstanding the relatively high cost involved. The subsequent construction savings derived by the developers are estimated to be in excess of twenty times the cost of conducting the test.

(b) The success of any loading test, as confirmed by this project, depends more critically on preliminary planning than on any other single related factor.

## XII.-ACKNOWLEDGEMENTS

This paper has been presented with the permission and encouragement of the developers, Mainline-Dillingham-Fletcher.

The author is indebted to his colleagues, D.E.Hollands and K.H.Gillespie for their helpful criticism during its preparation.

## REFERENCES

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2. A Performance Investigation of Pile Driving Hammers and Piles, Michigan State Highway Commission, 1965.

## APPENDIX

### LABORATORY STRENGTH TESTS

The shear strengths of the Waitemata series sandstones and siltstones were determined by three laboratory testing methods; unconfined compression, triaxial shear and direct shear.

Samples of the Waitemata series deposits were recovered as a continuous series of 3-foot long cores in an NX triple-tube core barrel. Although few cores were lost, the samples tended to break along horizontal bedding planes into 1 - to 12-inch lengths. All cores tested were of consistent diameter and required no preparation other than end trimming to the required test length. The samples were tested at their field moisture contents within 24 hours of recovery.

(a) Unconfined Compression Tests

All unconfined compression tests were performed at nominal confinement pressures of 10 PSI. A summary of the results of these tests is given in Table 1 below.

TABLE 1

UNCONFINED COMPRESSION TESTS

| | Siltstone | Sandstone |
|---|---|---|
| Number of Tests | 54 | 75 |
| Minimum Strength | 19 PSI | 12 PSI |
| Maximum Strength | 285 PSI | 383 PSI |
| Average Strength | 113 PSI | 106 PSI |
| Median Strength | 102 PSI | 95 PSI |

(b) Triaxial and Direct Shear Tests

Eleven quick, undrained triaxial shear tests were performed on selected cores of the Waitemata series deposits. In addition, 19 strain-controlled direct shear tests were carried out under quick, undrained conditions and surcharge pressures approximately equal to or greater than the natural overburden pressures.

(c) Strength Parameters

For the purpose of pile design, the strength parameters of the supporting foundation deposits were assessed from the combined results of the above strength tests. The apparent angles of internal friction were determined from a combination of the results of the triaxial and direct shear tests. These values are given in Section III of the main text.