Ferry Building- Soil Investigation

Brickell, Moss & Partners

56259 9 September 1981

Preliminary Soils Investigation
Ferry Building Strengthening
85 Quay Street, Auckland

Report to Messrs. Wargon Chapman & Gurley
Consulting Engineers

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9 September 1981 56259

Messrs. Wargon, Chapman & Gurley,
Consulting Engineers,
40 College Hill,
AUCKLAND

Attention: Mr C.R. Gurley

Dear Sirs,

Re: PRELIMINARY SOILS INVESTIGATION,
FERRY BUILDING STRENGTHENING,
QUAY STREET, AUCKLAND.
For THE AUCKLAND HARBOUR BOARD.

Introduction

This report presents the results of our preliminary soils investigation, undertaken in conjunction with the proposal to strengthen the Auckland Harbour Board's Ferry Building. The Building is adjacent to Quay Street, Auckland, and bounded by the Waitemata Harbour, Ferry Wharf and Queens Wharf, as shown on the attached Plate 1, Site Plan.

BLC 22/9/81

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Some subsurface information on this area had been obtained during our 1967 foundation investigation for Stage 1 of the Downtown Project (opposite the Building on the south side of Quay Street). That information was supplemented with research by Mr. Gurley and the results, including details of the existing foundations of the Ferry Building, are shown on Drawing SK 835, Sheet 1, dated Nov. 1980, and described in a Wargon, Chapman & Gurley report, dated December 1980.

The Building site is located on reclamation filling, placed in about 1905. The filling is retained along the north, east and part of the western sides of the site by a mass concrete seawall, as shown on the Site Plan.

The Building is apparently supported partly on concrete piles (driven down through the reclamation filling to the underlying rock), and partly on the seawall. The seawall was to be constructed directly on and keyed into the underlying rock. It was thought to be backed by a wedge of sandstone lumps (as shown on drawing SK 835), but whether this rock fill is present and if so how it relates to the piled construction is not known.

An earlier stage of reclamation filling was thought to be contained by a rock-filled timber breastwork, adjacent to the southern side of the Building. There may also be the remnants of a weighbridge along the eastern half of this southern side of the building, under the Quay Street footpath.

The purposes of our study were to provide some information on the reclamation filling underlying the Building, to check the condition of the mass concrete sea wall and its foundation, to confirm the depth to the underlying rock, and to provide preliminary geotechnical engineering design criteria for the proposed strengthening project.

Investigation and Testing

Subsurface conditions were explored by drilling three borings at the locations shown on the site plan. These positions were chosen to provide a cross-section across the site, and to correspond with accessible locations.

One of our engineers supervised the drilling on a full-time basis, logged the soils encountered, and obtained samples for examination and possible testing. Notes on the drilling and a summary of the abbreviations used on the boring logs are included on Plate 2, Notes and Abbreviations for Logs. Logs of the soils and rock encountered in the borings are presented on Plates 3-A to 3-C, Boring Logs. The soils are described in accordance with the Unified Method of Soil Classification, which is summarised on Plate 4, Method of Soil Classification.

Standard penetration tests were considered to be the most effective method to indicate the in-situ engineering properties of the reclamation filling materials. Where applicable, these tests were carried out at close intervals of depth in the borings, and the results are shown on the logs. The field values, shown on the logs, have also been corrected to allow for overburden and submergence values, with results as follows:

Boring	Depth	Field SPT	N, corrected
1	6.6	3	4
1	6.8	5	6
3	5.5	10	14
3	6.8	12	16
3	8.4	2	2

It was considered unnecessary to carry out extensive laboratory testing of the soil samples, due to the inherent variability of the reclamation materials. Particle size tests have been carried out on the sandy soils, from below the upper reclamation filling, and results are summarised on Plate 5, Grading Tests. Atterburg limit tests on two of these samples showed the fines to be non-plastic. Classification tests (liquid limit, plastic limit, and percent fines passing 75um sieve) were also carried out on a sample of soft clay from under the sandy soils in Boring 1, and results are shown on the boring log.

Subsurface Conditions

In making an assessment of subsurface conditions from a few borings, there is always a risk of undetected variations. In this case it is particularly important to remember that only three borings have been drilled, in part of an area where the reclamation methods and materials are known to have been extremely variable. However, the results obtained are generally consistent with available background data and the site geology, and are therefore described here as a basis for the conclusions of this preliminary report.

The site is blanketed with non-compacted fill, comprising soft to firm sandy clays, sandy silts and silty sands to a depth of approximately 5 metres. The fill is underlain by loose to moderately dense silty sands and some soft clays. Competent dark grey sandstones and siltstones of the Waitemata series rocks are present as bedrock below a depth of about 8.6 metres.

Boring 1 encountered cobble-sized basalt boulders, down to a depth of 5.5 metres below the footpath surface. The boulders are in turn underlain by the loose sands and soft clays. These boulders may be part of the rock-filled timber breastwork which defined the limit of the earlier stage of reclamation filling.

The seawall bounding the east, north and part of the west sides of the site is comprised of hard competent concrete, which contains some basalt cobbles. The bottom 400 mm of wall concrete appears to have been affected by seawater, as it has become whitened and somewhat softer than the unaffected wall above (such that it can be indented by a fingernail). Hard siltstone of the Waitemata series rocks was encountered immediately below the base of the wall, at 9.3 m depth.

Groundwater was encountered in the borings, and the water surface appeared to follow with tidal sea levels, but with a lag which resulted in water level differences which we noted as ranging up to about one metre.

18" sq. = 2¼ ft²

$\frac{110+}{2.25} = 49+/\text{ft}^2$

Previous Earthquakes

Over the past 100 years, the shocks from more than 50 earthquakes are recorded as having been felt in Auckland. This represents an average of one every two years. The closest recorded epicentre to Auckland of a significant earthquake was off Port Waikato in 1891. This earthquake had a Modified Mercalli intensity in Auckland of about VII.

Since the Ferry Building was constructed, in about 1908, available information indicates that the maximum seismic shock experienced in Auckland would have been equivalent to a Modified Mercalli intensity of about V, arising from the Buller earthquake on 16 June 1929. A similar intensity of seismic shock may have been experienced during other earthquakes, for example one centred off Taranaki in 1949, but detailed information is not available. However, it does seem certain that the Ferry Building has not yet been subject to an earthquake of intensity equivalent to that being assumed for the design of the proposed strengthening.

There is evidence of some minor structural distress to the building, specifically as cracking in the archway along the north side of the building and in cross-walls at higher levels. Our inspection indicates that this could only be explained by a spreading of the building at ground level, such as by an outward movement of the top of the seawall, rather than by any foundation settlement. Perhaps this movement occurred during an earlier earthquake.

Piled Foundations

The foundation piles, which are 0.45m-square precast-concrete driven piles, apparently had a design working load of about 110 tonnes. This is equivalent to an end-bearing pressure of about 5400 kPa. We have no detailed information on the construction of the piles, so, for purposes of this report, we have assumed that they were driven to near refusal on the underlying weathered Waitemata series bedrock. The piled foundations appear to have

2¼ft^2

49t/f

$$\frac{200t}{2.25 \times 144} = 0.6t/in^2$$

= 1380 lb/in²

= 90t/ft²

9000 kPa =

performed satisfactorily under the normal vertical loadings imposed by the existing building. This observation is consistent with our opinion that an end bearing pressure of 5400 kPa, under the normal working load condition, is reasonable for such piles driven to near refusal on the underlying bedrock.

As indicated, it is proposed to strengthen the Ferry Building so that its structure has better seismic resistance. This strengthening of the structure will of itself involve some increase in the normal vertical loadings on the foundation piles. We would expect that an increase in the end-bearing pressure of up to 10 percent, that is up to 6000 kPa, should only cause minor additional deformation.

With the added effects of the strengthening, and including allowance for seismic overturning, the maximum working load on a pile is expected to increase to about 200 tonnes. This is equivalent to an end-bearing pressure of about 9700 kPa. We are concerned that under this loading condition the piles may be overloaded. We would prefer to keep the maximum end-bearing pressure, for the dead plus live plus seismic working load condition, down to less than 9000 kPa.

We must point out that we cannot substantiate these allowable bearing pressure figures, other than by our judgement based on the performance of the building to date, information on the Waitemata Series bedrock obtained from the borings, and results of a full-scale pile loading test carried out some years ago for the adjacent Downtown project.

The net uplift capacity of a pile under the seismic working load condition may be assumed to be 5 tonnes. Note that in assessing the gross uplift capacity, which would include the added weight of the pile itself, the weight of the pile must be reduced to allow for bouyancy effects.

Seismic Liquefaction

When a loose saturated sand is subjected to ground vibrations, it tends to densify and decrease in volume. In the case of a fine sand, and particularly when silty fines are present, the water between the particles cannot drain so the reduction in volume causes an increase in porewater pressure. If the porewater pressure increases to the point at which it equals or exceeds the overburden pressure, then the effective stress between the particles becomes zero, the sand loses its inter-granular strength completely, and the soil adopts a liquified state analagous to that of a quicksand.

The grading and relative density of the silty sand at this site are such that we consider liquefaction must be considered a real possiblity under severe earthquake conditions. Based on the formulae included in a paper entitled "A Simplified Procedure for Evaluating Soil Liquefaction Potential", by Seed H.B. and Idriss I.M., we estimate that the onset of liquefaction could correspond with a peak acceleration for ground movement of between about 0.07g and 0.12g, based on a minimum of 20 stress cycles, which is equivalent to an earthquake intensity of about VI or VII on the Modified Mercalli scale. For the design peak acceleration of 0.15g, which is equivalent to an earthquake intensity of about VII or VIII on the Modified Mercalli scale, it should therefore be assumed that liquefaction would occur, of all such material below the groundwater level at the time.

We have considered the possibility of injection grouting to avoid this risk of liquefaction. However, the grading of the silty sands is too fine for this to be feasible, even with the use of chemical grouts.

Whilst the subsoils are confined by the seawall, any lateral spreading due to liquefaction cannot occur. However, there would be an increase in relative density of the silty sands which could cause a surface subsidence. If for example a 3-metre-thick layer was subject to an increase in relative density of 20%, then the resulting subsidence would be on the order of 500 mm. However, if surface venting (sand boils) of the liquefied material were to occur, the subsidence could in the extreme be equal to the total thickness of the layer prior to liquefaction; that is, three metres in the example given. For this site, we consider this latter possibility to be most unlikely.

Seawall Foundation

The one boring drilled through the seawall indicates that it is founded on sound bedrock. A softer zone of concrete in the bottom of the wall is probably due to the effects of seawater, or the result of tremie placement of the concrete, or both. For assessing stability of the wall, we consider that the toe bearing pressure (for unfactored working loads) should not exceed 2000 kPa, and the maximum angle of shearing resistance at the rock-concrete interface beneath the wall for the loading condition which includes lateral seismic effects should not exceed 40 degrees (with no additional allowance for any passive resistance which might be developed by the key).

The seawall is relatively rigid, and must be considered as unyielding if it is to fulfill its role of supporting the outer wall of the Ferry Building. It is therefore appropriate to use the "at-rest" coefficient for lateral pressure, Ko, which for this material may be taken as 0.60. This is applied to the effective vertical pressure; that is, to the gross vertical soil pressure reduced by the buoyancy effects of water pressure below the groundwater level.

In addition, the wall is subject to hydrostatic pressure due to any difference in water levels, from one side of the wall to the other, resulting from any lag in response to tidal variations of sea level - for design purposes this maximum difference may be taken as 1 metre.

Lateral Effects of Seismic Loading

Under the seismic loading condition there will also be a lateral dynamic loading on the seawall due to acceleration of the soil mass. For preliminary design purposes, we estimate that this may be taken as a uniform lateral pressure of 25 kPa acting on the full height of the wall.

In the case of liquefaction, the combined static and dynamic lateral pressures due to the soil and groundwater would be substituted by the dynamic (tank-full) effects of a heavy fluid with density equal to 1900 kg/m3. The upper surface of the fluid may be assumed to be at the groundwater surface level at the time of liquefaction, and the effective vertical soil pressure at this groundwater level should be added to the fluid pressures. Because the liquefied soil loses all shear strength, the overlying unliquefied zone has no resistance to lateral excitation. Thus, the mass of the soil above the groundwater surface level should be added to the mass of the building.

The development of any lateral resistance due to passive pressure on the piles and foundation beams would require movement. However, movement cannot occur, because of the direct connection of the building to the relatively rigid seawall, without corresponding deformation of the building itself. Accordingly, the lateral seismic force from the building should also be transmitted to the top of the seawall during south to north earthquake loading. If the building did deform, then the proportion of lateral seismic loading taken through passive pressure would still in turn be transferred, through the upper soil, to the face of the seawall.

The same problem occurs in reverse with north to south earthquake loading, and may be the more severe condition as it would develop tension across the base of the building. This may be the explanation for the minor cracking observed along the north side of the building. Perhaps the foundation grillage could even yield or separate from the wall due to tension across the base of the building, and in this case the seismic loading would then need to be resisted by passive pressure against the upper (non-liquefied) zone of reclamation filling, and any breastworks, out into Quay Street. For purpose of design calculations this may be assessed using a passive co-efficient for lateral pressure of 2.0, but with no additional allowance for friction under the base of the pile caps.

Another potential problem under seismic loading is that the foundation piles could be subject to high shearing forces at the interface between the upper reclamation fill and the lower zone of silty fine sand, which may be subject to liquefaction. If the piles are brittle they may fail in shear or bending at this interface.

General

This study indicates that there are potential problems of pile capacity, soil liquefaction and seawall stability, which will need to be resolved in any seismic strengthening of the Ferry Building. The general scope of these problems has been identified by these studies, and preliminary geotechnical design criteria are provided here as a basis for structural engineering feasibility studies.

If the project proceeds to final design, then we consider that further site investigation, involving more drilling and testing, will be required. Further data and analyses will also be required to enable the review and amplification of the preliminary design criteria.

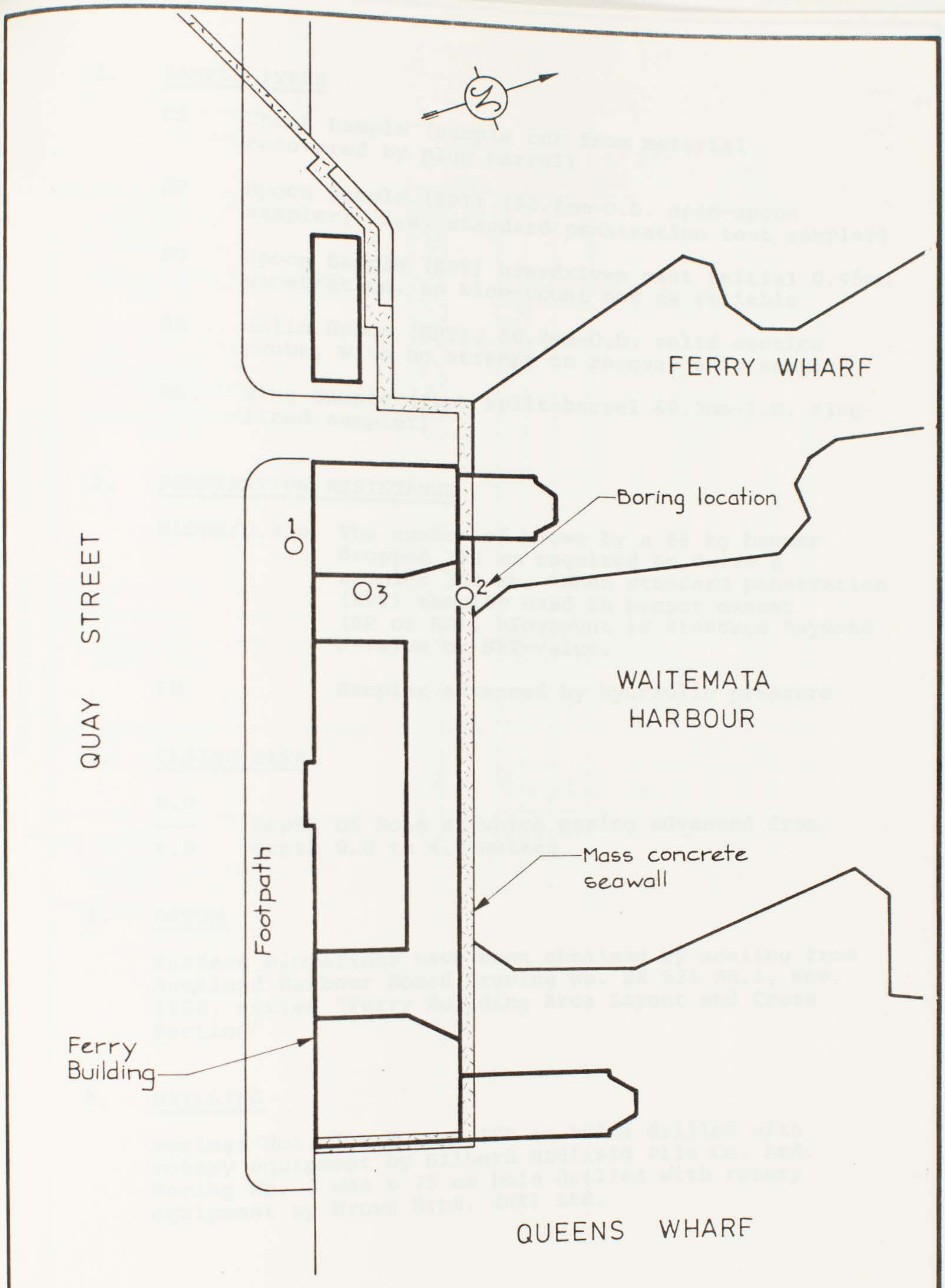
The following plates are attached and complete this report.

Plate 1	Site Plan
Plate 2	Notes and Abbreviations for Logs
Plates 3-A to 3-C	Boring Logs
Plate 4	Method of Soil Classification
Plate 5	Grading Curves

Yours faithfully,
p.p. BRICKELL, MOSS & PARTNERS

David E. Hollands

David E. Hollands



SITE PLAN

Scale 1:500

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PLATE 1

1. SAMPLE TYPES

CS	Chunk Sample (sample cut from material recovered by plug barrel)
SP	Spoon Sample (SPT) (50.8mm-O.D. open-spoon sampler; i.e. standard penetration test sampler)
SO	Spoon Sample (SPT) overdriven past initial 0.45mm penetration, so blow-count not as reliable
SS	Solid Spoon (SPT), 50.8mm-O.D. solid section probe, with no attempt to recover soil sample
RS	Ring Sample (from split-barrel 60.3mm-I.D. ring-lined sampler)

2. PENETRATION RESISTANCE

BLOWS/0.3 m	The number of blows by a 64 kg hammer dropped 760 mm required to drive a sampler 300 mm. When standard penetration (SPT) sampler used in proper manner (SP or SS), blowcount is standard Raymond N-value or SPT-value.
PH	Sampler advanced by hydraulic pressure

3. CASING DATA

$\frac{0.0}{4.0}$	Depth of hole at which casing advanced from depth 0.0 to 4.0 metres

4. DATUM

Surface elevations have been obtained by scaling from Auckland Harbour Board Drawing No. SK 835 Sh.1, Nov. 1980, titled "Ferry Building Area Layout and Cross Section"

5. DRILLING

Borings Nos. 1 & 2 were 100 mm holes drilled with rotary equipment by Gilberd Hadfield Pile Co. Ltd. Boring No. 3 was a 75 mm hole drilled with rotary equipment by Brown Bros. (NZ) Ltd.

NOTES AND ABBREVIATIONS FOR LOGS

SURFACE ELEVATION 5.2 DATUM AHB
GROUNDWATER DEPTH 1.43* DATE 6.5.81
FELD * Water level appears to correspond with tide level
DRILLING DATE(S) 4 - 5.6.81

BORING Nº. 1

DRILLING METHOD	CASING DEPTH	STRATIGRAPHY	ELEV / DEPTH metres	SAMPLE DATA TYPE	SAMPLE LOST DISTURBED	BLOWS PER 0.3 METRES	WATER/DENS. DRY DENSITY Mg/m³	NATURAL WATER CONTENT %	CLASSIFICATION LIQUID LIMIT	PLASTICITY INDEX	% FINES (<75 µm)	STRENGTH DATA TYPE OF TEST	PARAMETERS Su kPa c' kPa ϕ' degrees	OTHER
Plug Barrel	0	PAVEMENT (bitumen + base course)	4.8 0.4											
		Brown SILTY SAND (SM) and SANDY CLAY (CL) (moderately dense FILL, with pieces of weathered mudstone & siltstone)		CS										
			3.2 2.0	SP		1								
Rotary Plug		Dark grey BASALT COBBLES (hard, vesicular, inferred size 100mm to 300mm) (with some brown SCORIA, hard)	-0.3 5.5											
Plug Barrel		Dark grey ORGANIC SILTY SAND (SM) (loose, with pieces of wood, organic, some OL)		CS				29.7			43			
				CS				26.1			36			
				RS		PH	1500	29.1			34			
				SS		3								
			-2.1 7.3	SS		5								
		Grey SANDY CLAY (CH) (soft, organic material present)		SP		0			60	35	78			
				SO		9								
	0		-3.3 8.5											
		Grey SILTSTONE (very stiff, moderately weathered Waitemata Series)	8.7	SP		50								

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PLATE 3A

SURFACE ELEVATION 5.0 DATUM AHB

GROUNDWATER DEPTH * DATE

FIELD * Not recorded

DRILLING DATE (S) 5-6-7-8.5.81

BORING Nº. 2

DRILLING METHOD	CASING DEPTH	STRATIGRAPHY	LOG	ELEV / DEPTH metres	SAMPLE DATA: TYPE	SAMPLE DATA: SAMPLE LOST DISTURBED	SAMPLE DATA: BLOWS PER 0.3 METRES	WATER/DENS: DRY DENSITY Mg/m³	WATER/DENS: NATURAL WATER CONTENT %	CLASSIFICATION: LIQUID LIMIT	CLASSIFICATION: PLASTICITY INDEX	CLASSIFICATION: % FINES (<75 µm)	STRENGTH DATA: TYPE OF TEST	STRENGTH DATA: PARAMETERS Su kPa c' kPa ϕ' degrees	OTHER
Tricone Bit	0	Grey CONCRETE (hard, homogeneous; concrete seawall) CONCRETE WALL													
Diamond Coring		(with grey BASALT COBBLES, hard, vesicular, size 100mm to 300mm contained within concrete)			CORE SAMPLE RETAINED										
		White CONCRETE (firm to stiff, affected by seawater)		-3.9 8.9 -4.3											
Plug Barrel	0	Grey SILTSTONE (very stiff, moderately weathered Waitemata Series)		9.3 -4.7 9.70	SP		50								

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SURFACE ELEVATION 5.4 DATUM A.H.B.
GROUNDWATER DEPTH 3.65* DATE 27.5.81
FIELD *Water level appears to correspond with tide level
DRILLING DATE(S) 27.5.81

BORING Nº. 3

DRILLING METHOD: Plug Barrel (0 – 9.3)
CASING DEPTH: 0 … 0

STRATIGRAPHY	ELEV / DEPTH metres
Concrete PAVEMENT	5.2 0.2
Brown SANDY CLAY (CL) and SANDY SILT (MI) (soft - firm, FILL, pieces of brick, organic material) (grades grey, with brown layers) (small pieces of broken mudstone) (pieces of brick) (small pieces of sandstone and siltstone, shells, piece of scoria) F I L L	0.0 5.4
Grey SILTY SAND (SM) (loose) (bands of firm - stiff silt) (grades dark grey, moderately dense) (very loose, with shells)	-3.3 8.7
Yellow CLAYSTONE (hard)	9.1
Grey SILTSTONE (very stiff, moderately weathered Waitemata Series)	9.3

SAMPLE DATA			WATER/ DENS		CLASSIFICATION			STRENGTH DATA		OTHER
TYPE	SAMPLE LOST DISTURBED	BLOWS PER 0.3 METRES	DRY DENSITY Mg/m^3	NATURAL WATER CONTENT %	LIQUID LIMIT	PLASTICITY INDEX	% FINES (<75 µm)	TYPE OF TEST	PARAMETERS Su kPa c' kPa ϕ' degrees	
SP		2								
CS										
SP		3								
SO		5								
CS										
SP		8								
SO		5								
CS										
SP		10								
CS				32.1	NP		45			
SP		12		28.1			30			
CS				41.5	NP		52			
SP		2		30.8			42			
SP		50								

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

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PLATE 3 C

COARSE GRAINED SOILS (less than 50 percent fines)	GRAVELS		GW	Well graded gravels or sand-gravel mixtures, less than 5% fines.
			GP	Poorly graded gravels or gravel-sand mixtures, less than 5% fines.
			GM	Silty gravel, gravel-sand-silt mixtures.
			GC	Clayey gravel, gravel-sand-clay mixtures.
	SANDS		SW	Well graded sands or gravelly sands less than 5% fines.
			SP	Poorly graded sands or gravelly sands, less than 5% fines.
			SM	Silty sand, sand-silt mixtures.
			SC	Clayey sand, sand-clay mixtures.
FINE GRAINED SOILS (more than 50 percent fines)	SILTS & CLAYS	L L < 50	ML	Inorganic silts, rock flour, sandy or clayey silts of low plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low to medium plasticity.
			MI	Inorganic silts or clayey silts of medium plasticity.
		L L > 50	MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of high plasticity, organic silty clay, organic silt.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

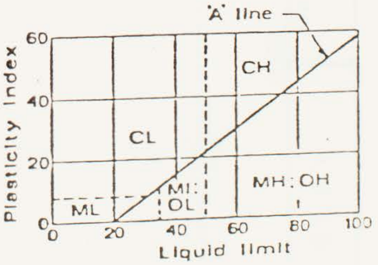
CLASSIFICATION		EQUIVALENT SIEVE SIZE	
		B S.	A S
COBBLES		8 in - 3 in	200mm - 75mm
GRAVEL.		3 in. - $\frac{3}{16}$ in	75mm - 4.75mm
	coarse	3 in - $\frac{3}{4}$ in	75mm - 19mm
	fine	$\frac{3}{4}$ in - $\frac{3}{16}$ in	19mm - 4.75mm
SAND.		$\frac{3}{16}$ in - No 200	4.75mm - 75 μm
	coarse	$\frac{3}{16}$ in - No 7	4.75mm - 2.36mm
	medium	No.7 - No 36	2.36mm - 425 μm
	fine	No.36 - No 200	425 μm - 75 μm
FINES, silt & clay		below No.200	below 75 μm

COHESIONLESS SOILS

RELATIVE DENSITY	'N' (SPT) VALUE. blows/ft.
VERY LOOSE	0 to 4
LOOSE	4 to 10
MODERATELY DENSE	10 to 30
DENSE	30 to 50
VERY DENSE	Above 50

COHESIVE SOILS

CONSISTENCY	UNDRAINED SHEAR STRENGTH.	
	p.s.f.	kPa
VERY SOFT	0 to 250	0 to 12.5
SOFT	250 to 500	12.5 to 25
FIRM	500 to 1000	25 to 50
STIFF	1000 to 2000	50 to 100
VERY STIFF	Above 2000	Above 100



METHOD OF SOIL CLASSIFICATION

(UNIFIED CLASSIFICATION SYSTEM)

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PLATE 4

GRADING CURVES

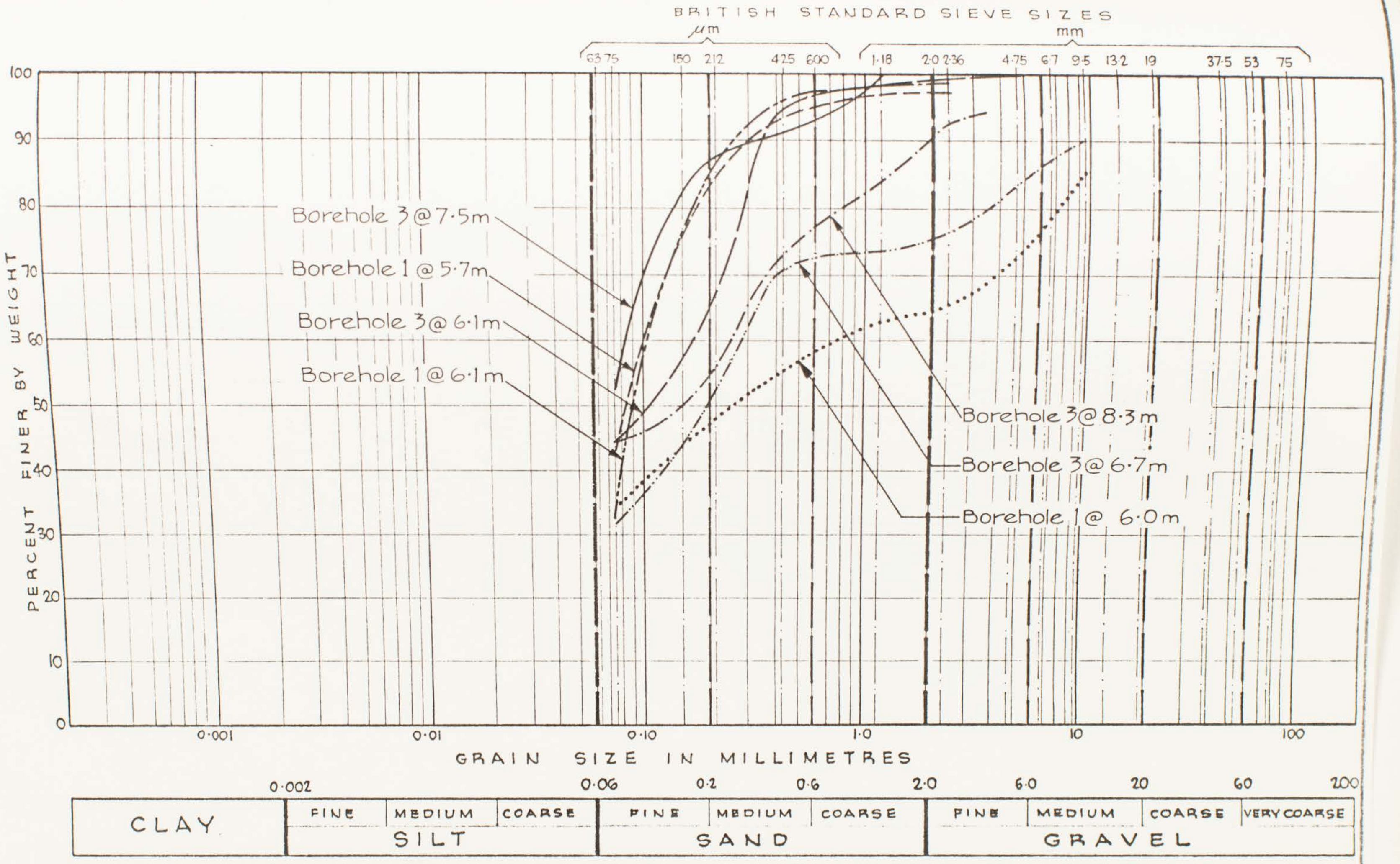

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PLATE 5