3. Area B - existing embankment at north end of site. In the year before the piles are loaded, swelling will occur in Area B due to removal of the existing overburden. The amount of swelling which will occur has been assessed as follows:

Butler (1975) reanalysed May's data and showed that it was reasonably consistent with his correlations of stiffness with undrained shear strength. His work indicates that about 17% of the total long-term swelling occurred within the first year after unloading.

The total long term swelling has therefore been computed using the VDISP program as in our calculations "Analysis of ground movement" dated 5/7/83. Figure 3 shows the heave expected within one year of removal of load, due to 17% of total swelling. This heave due to swelling does not include heave due to undrained deformations which occur before the piles are constructed.

In Area B the heave expected within one year generally exceeds 20mm. It is considered that this would be sufficient to mobilise the full shear strength along much of the pile/soil interface. It is therefore necessary to reinforce the piles in Area B against tension.

The calculations of tensile forces and reinforcement are attached. The calculation is based on the following assumptions:

a) Shaft friction is fully mobilised in the most adverse manner possible.
b) The shaft friction is calculated using $\alpha\ c_u$ with $\alpha = 0.7$. This high value is pessimistic in this case.
c) Cracking and extension of the piles are to be controlled. Design is based on a permissible steel stress of 250 N/mm^2.

The extent of the area in which piles are to be reinforced has been limited to the area where calculated heave within one year exceeds 10mm and is indicated on Figure 5. This will be referred to as Zone X.

4. Area C - rubble berm

In Area C an existing rubble berm is to be removed. The pressure to be removed is smaller and less extensive than is the case for Area B.

The computed heave at the ground surface for the year after removal of the load is shown in Figure 3, and does not exceed 10mm in this area. It is considered that this is unlikely to be sufficient to mobilise enough shaft friction and hence tensile force to crack the piles. If cracking does occur, however, it will consist of a small number of cracks up to about 2mm wide and these will close again when the piles are loaded. It is considered that this will not significantly affect the load-carrying characteristics of the piles. In fact, if it has any noticeable effect, it will reduce long term differential movement of the building.

4. Area C - rubble berm (contd)

In the long term the piles in Area C will be under load but, because they have factors of safety of about 2, it is likely that in swelling ground the lower portion of the pile will not be stressed by the load. It is therefore possible that this portion of the pile, which would not be needed to carry load, could be subject to tensile forces.

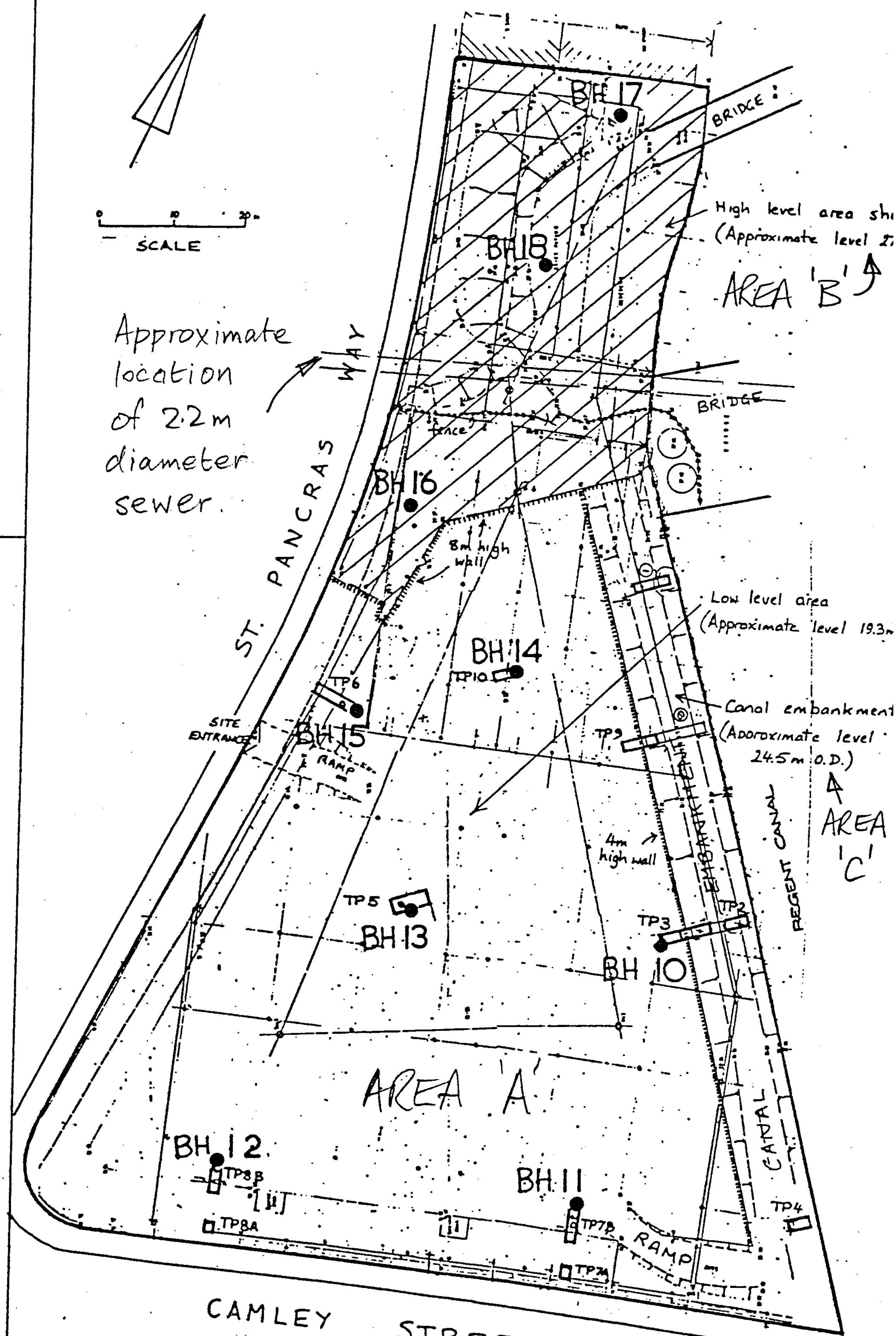
It is debatable whether this would matter, but as a check the long term extension of the ground in the lower 7m of the pile has been assessed. In this computation the working loads of the piles were applied as an equivalent uniform pressure at +7m OD. The computed long term extension of the ground between +7m OD and 0m OD is shown in Figure 4, together with the computed long term heave of the ground surface due to the nett loads.

The computed long term extension of the ground adjacent to the lower 7m of the piles does not exceed 7mm. It is considered that this is not enough to cause significant damage to the piles.

It is therefore considered that reinforcement for tension is not needed in the piles in Area C.

CONTRACT : ST.PANCRAS WAY/CAMLEY STREET,N.W.1
REPORT No.: 1870/JAD

SITE PLAN



12727
FIGURE 1

0.035 MN/m² was used.)

Construction of basement

A diagrammatic section of the basement is shown in *Fig. 4*. The basement is approximately 11 m deep over the whole site and forms a two-level car park. The sides of the excavation were retained by using diaphragm walls of reinforced concrete approximately 0.5 m thick cast in 1.5 m widths and extending into the London Clay to provide a cut-off for the ground water. These walls were incorporated in the final construction. A reinforced concrete raft, generally 1.2 m thick but thickened to 1.8 m under the central columns and

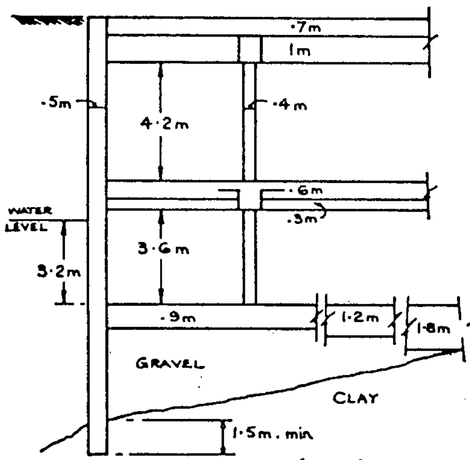


Fig. 4

lift shaft areas and reduced to 0.9 m thick near the perimeter, provides the basement floor. Intermediate and ground floors are of beam and slab construction. The plan size of the basement can be obtained from *Fig. 2* and the dead load of the basement structure is 48 kN/m². (In estimating the heave the raft was approximated to a rectangle 64.2 m × 88.8 m.)

The excavation was begun in June 1966 and finally completed in November 1967. The basement concrete was finished up to ground floor level in May 1968.

Observation of heave

The measured changes in height of the various survey points (*Fig. 2*) are recorded in *Table 1* and have been plotted graphically against the square root of elapsed time in months, in *Fig. 5*. The graphs show a straight line relationship up to the last readings but show a zero error with what seems to be

Table 1

Date of reading (initial reading Sept. 1967)	Survey marks 1	2	3	4	5	6	7	8	9	10	11
	(Accumulative change in height in millimetres—upward changes positive)										
March 1968	0.8	0.7	−0.5	−0.4	−0.1	0	0.5	1.1	0.6	−0.6	0.5
July 1968	7.4	7.7	8.5	9.0	9.2	8.1	5.0	8.6	6.7	6.6	7.3
Sept. 1968	8.8	9.6	10.9	11.4	11.7	10.1	6.0	11.0	8.1	8.0	8.4
July 1969	16.8	18.1	21.1	22.5	22.2	18.7	11.4	20.2	15.7	15.4	15.0
Sept. 1969	19.4	20.5	24.0	25.8	25.5	21.2	12.9	23.3	18.0	17.3	17.0
Nov. 1969	19.7	20.6	24.1	26.2	25.7	20.7	12.6	23.5	18.0	16.8	17.0
Feb. 1970	22.2	23.7	27.4	29.6	28.4	23.9	14.7	26.7	20.3	19.7	19.8
May 1970	27.3	29.5	33.9	36.1	35.6	30.2	19.6	32.3	26.0	25.7	24.7
Aug. 1970	31.5	33.4	38.4	40.9	40.2	34.1	22.2	36.6	29.5	28.8	27.5
Dec. 1970	29.3	31.5	36.7	39.6	38.9	31.6	19.2	35.2	27.2	25.9	25.0
Mar. 1971	32.1	34.7	40.1	43.4	42.3	34.8	21.8	38.1	29.9	28.9	27.4
Aug. 1971	35.0	38.0	44.0	47.1	46.1	38.5	24.1	41.4	32.5	31.7	29.6
Oct. 1971	36.7	39.5	45.8	41.2	48.1	39.3	24.7	43.3	33.8	32.6	30.3
Feb. 1972	39.6	42.7	48.8	52.3	51.1	41.8	27.4	46.1	36.2	35.1	32.7
May 1972	41.7	45.3	51.8	55.3	53.9	44.6	29.4	48.6	38.4	37.7	34.7
Oct. 1972	44.9	48.3	55.1	58.9	57.2	47.4	31.3	51.8	40.8	39.6	36.6
Feb. 1973	46.7	50.1	57.4	60.2	59.5	49.0	32.8	53.0	42.3	41.2	38.3

measurements accurate to 0.1 mm

Points W, X, Y, Z

Initial reading July 1968	W	X	Y	Z
Change to Oct. 1971	12.2	9.1	12.2	15.2

Measurements taken on concrete slab accurate to ±5 mm only

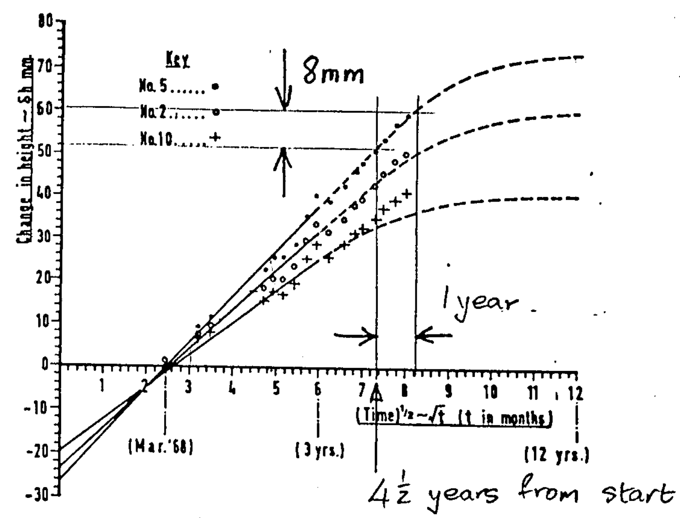


Fig. 5. Recorded heave

FIGURE 2

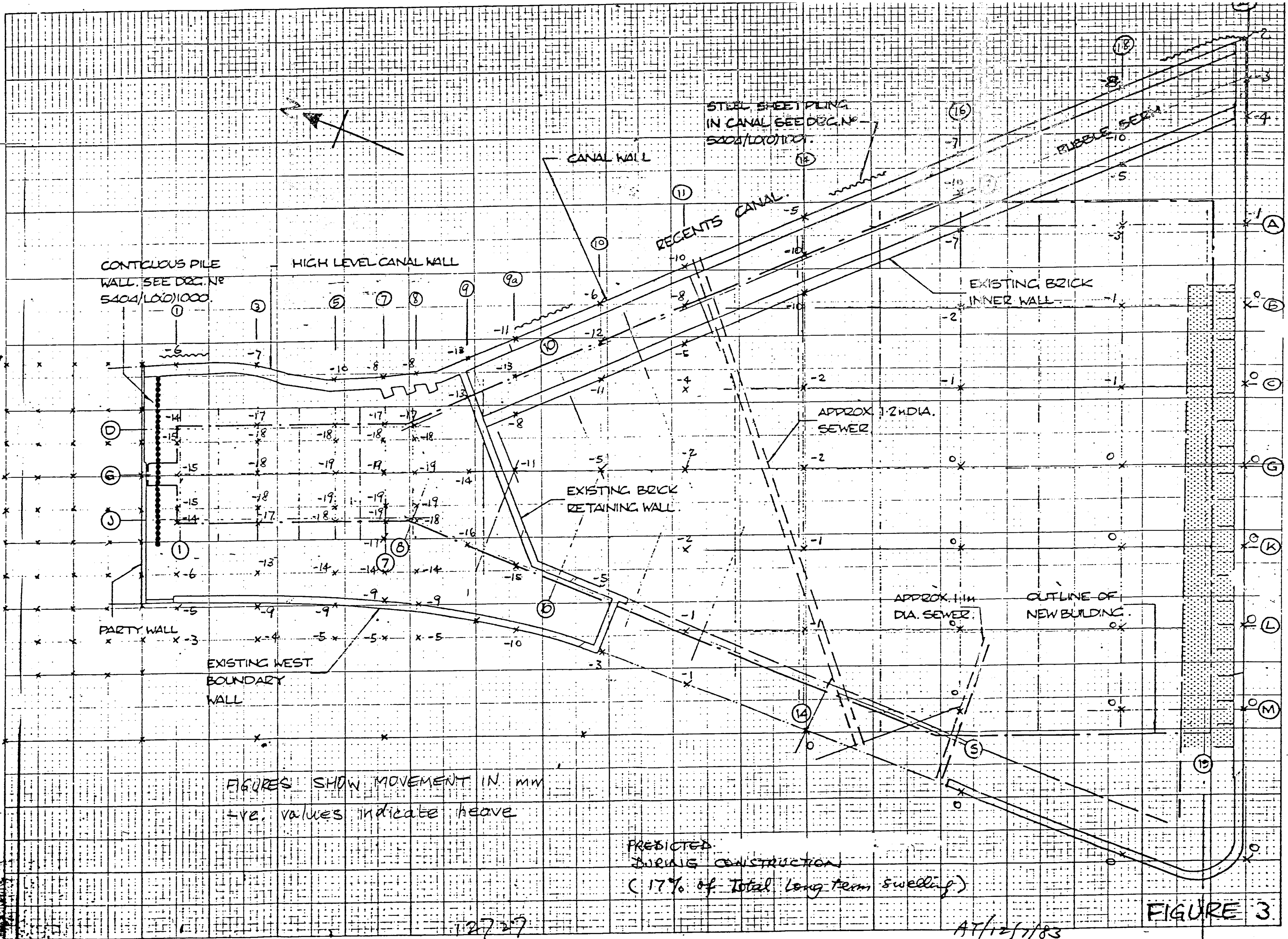
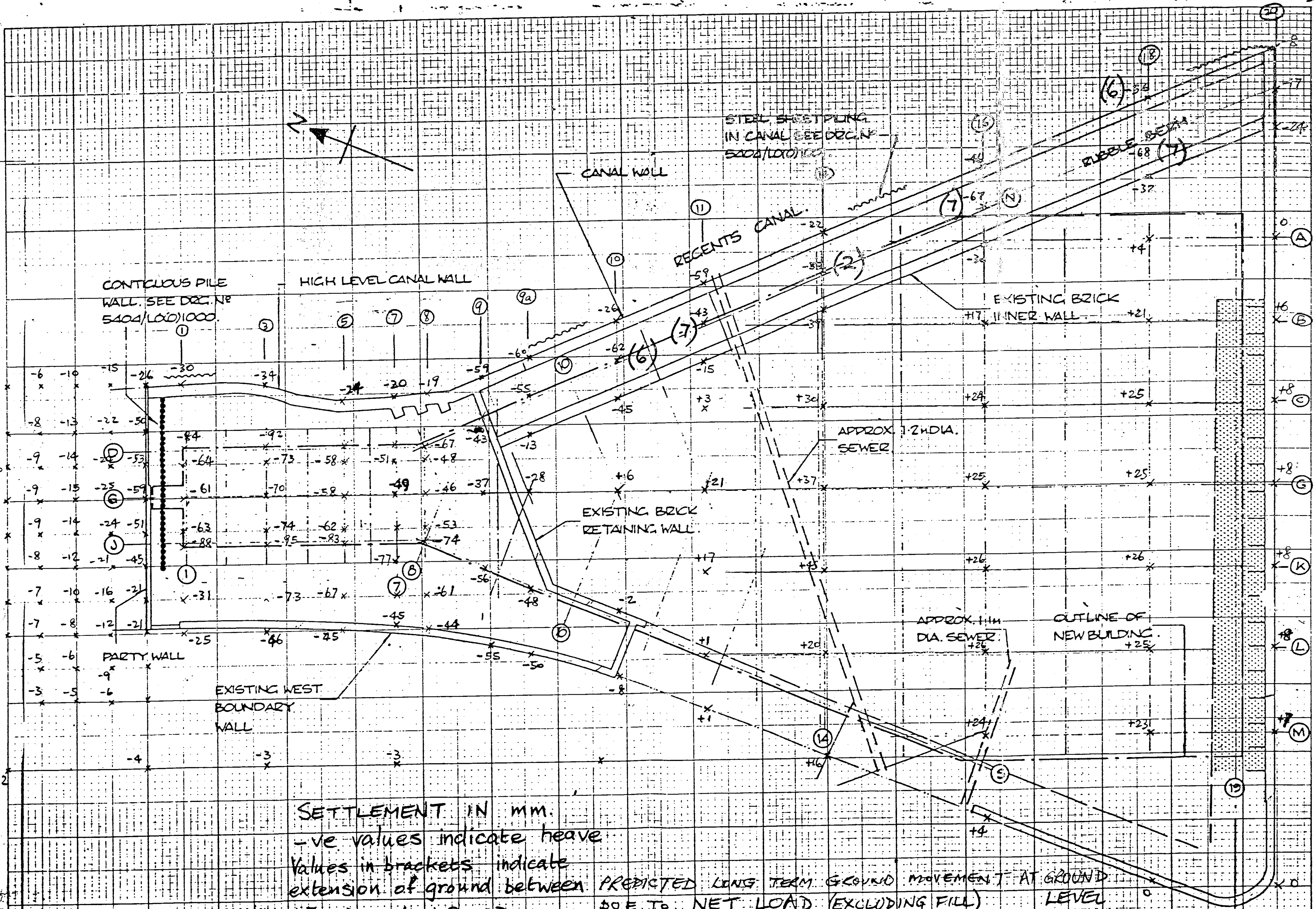


FIGURE 3

AT/12/7/83



STEEL SHEETPILING IN CANAL SEE DRG. Nº 5404/LOO/100
CANAL WALL
REGENTS CANAL
RUBBLE BERM
CONTIGUOUS PILE WALL. SEE DRG. Nº 5404/LOO/1000.
HIGH LEVEL CANAL WALL
EXISTING BRICK INNER WALL
APPROX. 1·2m DIA. SEWER
EXISTING BRICK RETAINING WALL
PARTY WALL
EXISTING WEST BOUNDARY WALL
APPROX. 1·1m DIA. SEWER
OUTLINE OF NEW BUILDING
SETTLEMENT IN mm.
-ve values indicate heave
Values in brackets indicate extension of ground between +7mOD and ±0mOD.
PREDICTED LONG TERM GROUND MOVEMENT AT GROUND LEVEL DUE TO NET LOAD (EXCLUDING FILL)

FIGURE 4

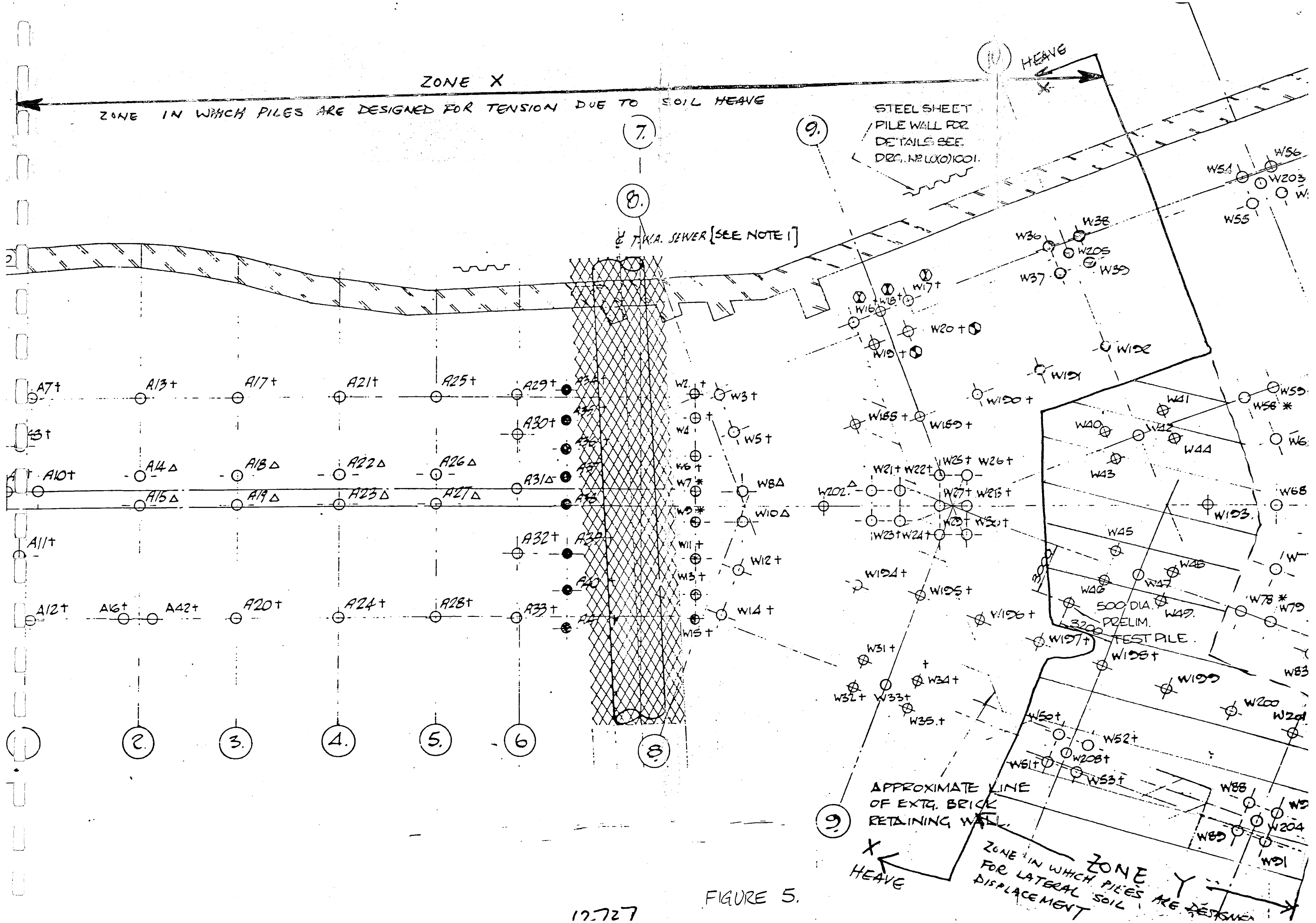


FIGURE 5.

12727

CALCULATION SHEET

12727 T4

Member / Location

Drg. Ref.

Job Title

Made by AT Date 26.8.83 Chd.

Reinforcement Design for piles within area of Heave (zone X)

The calculations as follows are divided into two parts.

Part one deals with reinforcement requirement at the top section of the piles. (pages T5 to T9)

Part two shows design charts for tension reinforcement other than top section of the piles (pages T10 and T11)

SUMMARY OF REINFORCEMENT REQUIRED AT TOP SECTION OF PILES

Pile Dia. (mm)	Type	Main Reinforcement AT PILE TOP	Shear* Reinforcement
600	Overbored and sleeved	6Y25	R8-300 c/c
600	Sleeved not subject to applied lateral load	6Y25	R8-300 c/c
600	Unsleeved not subject to applied lateral load	6Y25	R12-200 c/c
600	Unsleeved and subject to applied lateral load of 80 kN	6Y25	R8-300 c/c
600	Sleeved and subject to applied lateral load of 40 kN	6Y25	R8-300 c/c
500	Overbored and sleeved	6Y25	R8-300 c/c
500	Unsleeved not subject to applied lateral load	4Y25	R12-200 c/c
500	Unsleeved and subject to applied lateral load	6Y25	R8-300 c/c

* To the full length of piles.

CALCULATION SHEET

12727	T5

Member / Location

Drg. Ref.

Job Title: Granary Site | Made by: AT | Date: 26.8.83 | Chd.

(i) Overbored and sleeved piles within area of heave (ZONE X)

Because these piles are required to be sleeved at the upper 4m below the cut-off level, the pile section will act as a column. Hence, the reinforcement design requirement should comply with CP110, Cl. 3.5.1.1 for columns. It requires that the min steel area for the section is to be 1%.

(a) For 600 mm diameter,

Area of pile, $A_c = \frac{\pi}{4} \times 600^2 = 282743\ mm^2$

$\therefore\ A_s = 1\% \times 282743 = 2827\ mm^2$

USE 6Y25 $(A_s = 2945\ mm^2)$

(b) For 500 mm diameter,

Area of pile, $A_c = \frac{\pi}{4} \times 500^2 = 196350\ mm^2$

$\therefore\ A_s = 1\% \times 196350 = 1964\ mm^2$

require 4Y25 $(A_s = 1964\ mm^2)$

Recommend 6Y25 $(A_s = 2945\ mm^2)$
(Crack control)

Shear Reinforcement

The design of these piles are the same as the corresponding piles within ~~Zone~~ Y. Refer calculation for overbored and sleeved piles in zone Y for detail.

Hence,
Recommend R8 - 300 mm c/c

CALCULATION SHEET

12727	T6

Member / Location

Drg. Ref.

Job Title

Made by AT Date 8/83 Chd.

(ii) Sleeved piles within area of Heave (Zone X) not subjected to applied lateral load

i.e Piles nos A37 and A38. (600mm diameter)

Use 1% of steel reinforcement as recommended in the reinforcement calculation in zone Y.

i.e. $\frac{100 A_s}{A_c} = 1\%$

or USE 6Y25 ($A_s = 2945 mm^2$)

Shear Reinforcement

Nominal reinforcement required.

i.e. R8 - 300mm c/c

(iii) Unsleeved Piles with area of Heave (zone X) not subject to applied lateral loads

(a) 600 mm diameter piles

Pile nos. W36, 37, 38, 39 and 205.

The reinforcement design for these piles has been included in the calculation under "Piles subject to additional tensile forces within heave area".

required steel is 4Y25 ($A_s = 1600\ mm^2$)

Recommended steel is 6Y25 ($A_s = 2945\ mm^2$)

(b) 500 mm diameter piles

Pile nos. W191, W192

Use calculated steel reinforcement as in zone Y.

i.e 4Y25 ($A_s = 1964 mm^2$)

Shear Reinforcement

Same as calculated reinforcement in zone Y.

500 mm diameter R12 − 200 mm c/c

600 mm diameter R12 − 200 mm c/c

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CALCULATION SHEET

12727	T 8

Member / Location

Drg. Ref.

Job Title

Made by AT Date 8/83 Chd.

(iv) <u>Unsleeved Piles within area of Heave (Zone X)</u>
<u>and also subject to applied lateral load of 80KN</u>

<u>600 mm diameter</u>

Piles Nos: A14, A15, A18, A19, A22, A23, A26, A27, A31, W8, W10, and W202

Refer calculation on "piles subject to lateral loads."

Use $\frac{100A_s}{A_c} = 1.0\%$

i.e. <u>USE 6Y25</u> ($A_s = 2954 mm^2$)

(V) <u>Sleeved piles within area of Heave (zone X)</u>
<u>and also subject to applied lateral load of 40 kN</u>

<u>600 mm diameter</u>

Pile nos. W7 and W9.

Refer calculation on "piles subject to lateral loads"

Applied lateral load = 40 kN.

Required $\frac{100 A_s}{A_c} = 0.825\%$

<u>USE 6Y25</u> ($A_s = 2945 \text{ mm}^2$)

<u>Shear Reinforcement</u>

<u>USE R8 - 300 mm c/c</u>

CALCULATION SHEET

Job No.	Sheet No.	Rev.
12727	T10	

Member / Location

Drg. Ref.

Job Title

Made by AT | Date 8/83 | Chd.

PART TWO

DESIGN CHARTS FOR REINFORCEMENT TO PILES SUBJECT TO TENSION OTHER THAN TOP SECTION OF PILE

PILE DIAMETERS : 500mm and 600mm.

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CALCULATION SHEET

Maximum tensile force on piles in embankment area

12727	T11
Member / Location	
Drg. Ref.	
Made by AT.	Date 7.7.83	Chd.

Job Title Granary Site

Calculation of Maximum tensile force on piles in embankment area (600mm pile)

The removal of 6 to 7m of embankment will cause clay to heave, thus causing tension to the piles in the embankment area.

Assuming the clay is fully mobilised on the whole of the pile perimeter,

length of mobilised clay surface = πD

Now, max. shaft force per pile

$= \alpha \times \pi D \times \int_0^y C_{uy}\,dy$

Take $\alpha = 0.7$ (conservative)

$C_{uy} = 70 + 11y$ KPa

$D = 600$mm

$l = 20.3$m (typical pile with cut-off level at 20.3m, toe level at 0.0m)

$\therefore T = 0.7 \times \pi \times 0.6 \times \int_0^{20.3} (70 + 11y)\,dy$

$= 1.32\,(70y + 5.5y^2)_0^{20.3}$

$= 1.32 \times 3687.5$

$= \underline{4866 \text{ KN}}$

Length of pile in clay (m)	Max. calculated tensile force, T exerted by clay,* (KN)
5	644
10	1650
15	3020
20	4752
25	6848

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CALCULATION SHEET

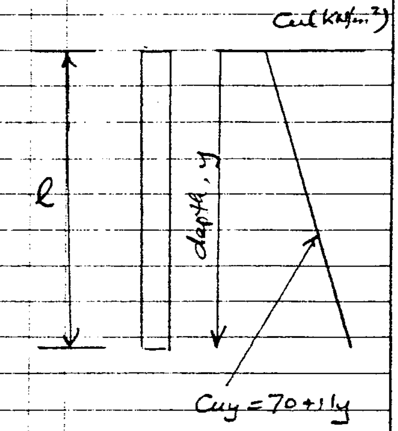
Job No	Sheet No	Rev.
12727	T12	

Member / Location

Drg Ref

Job Title	Made by	Date	Chd
Granary Site	AT	10/8/83	

<u>Check on reinforcement for 500 mm diameter</u>
<u>piles within the heave area</u>



pile diameter, D = 500 mm.

Max. shaft force on each pile

$= \alpha \times \pi D \times \int_0^y C_{uy}\, dy$

The following calculation is similar to those in Section T of Appendix G.

Take $\alpha = 0.7$

$C_{uy} = (70 + 11y)$ kN/m²

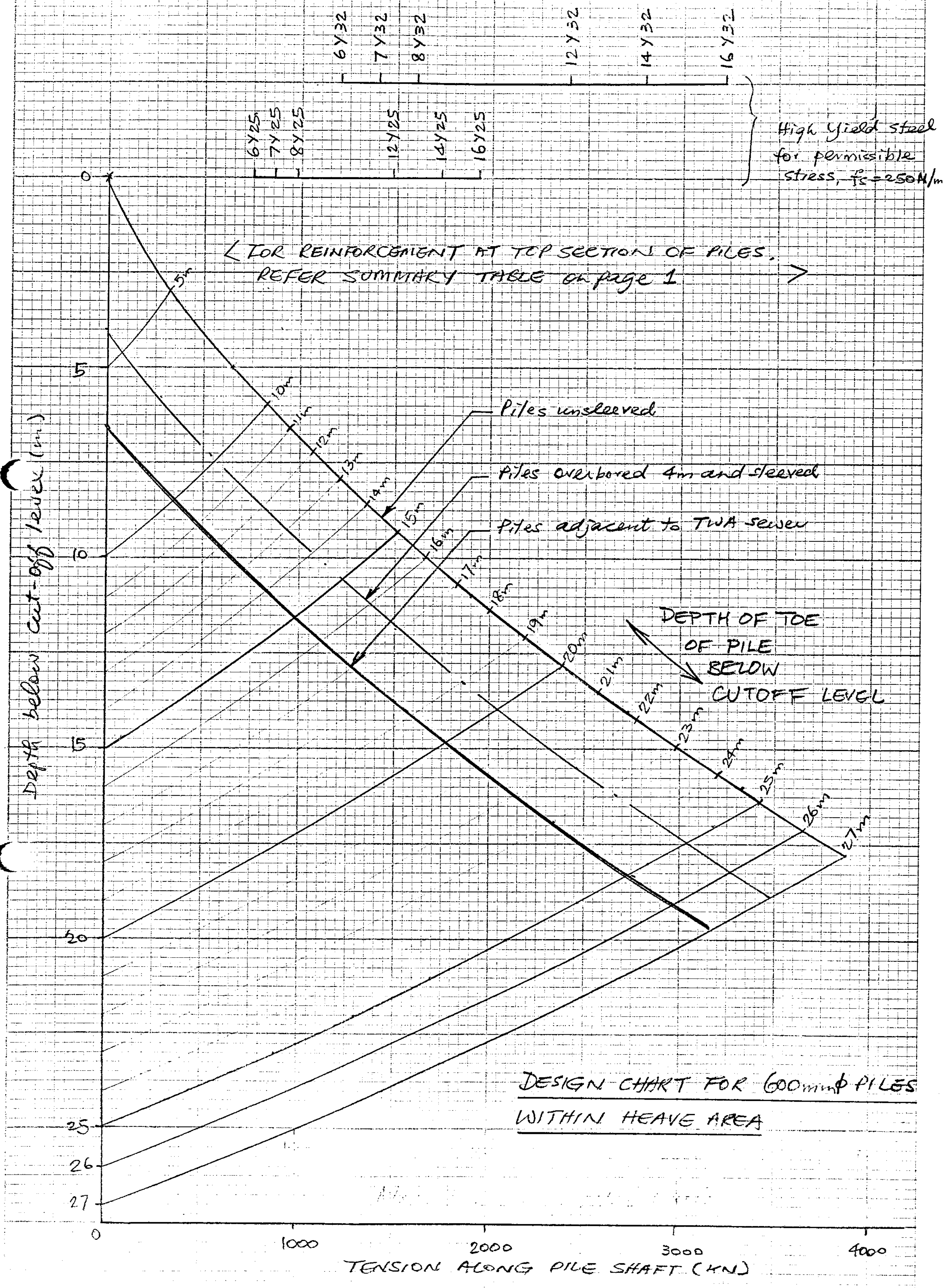
$\therefore$ Max shaft force, $T = 0.7 \times \pi \times 0.5 \int_0^l (70 + 11y)\, dy$

$= 1.1 \times (70 l + 5.5 l^2)$

$= (77 + 6.05 l)\, l$ kN.

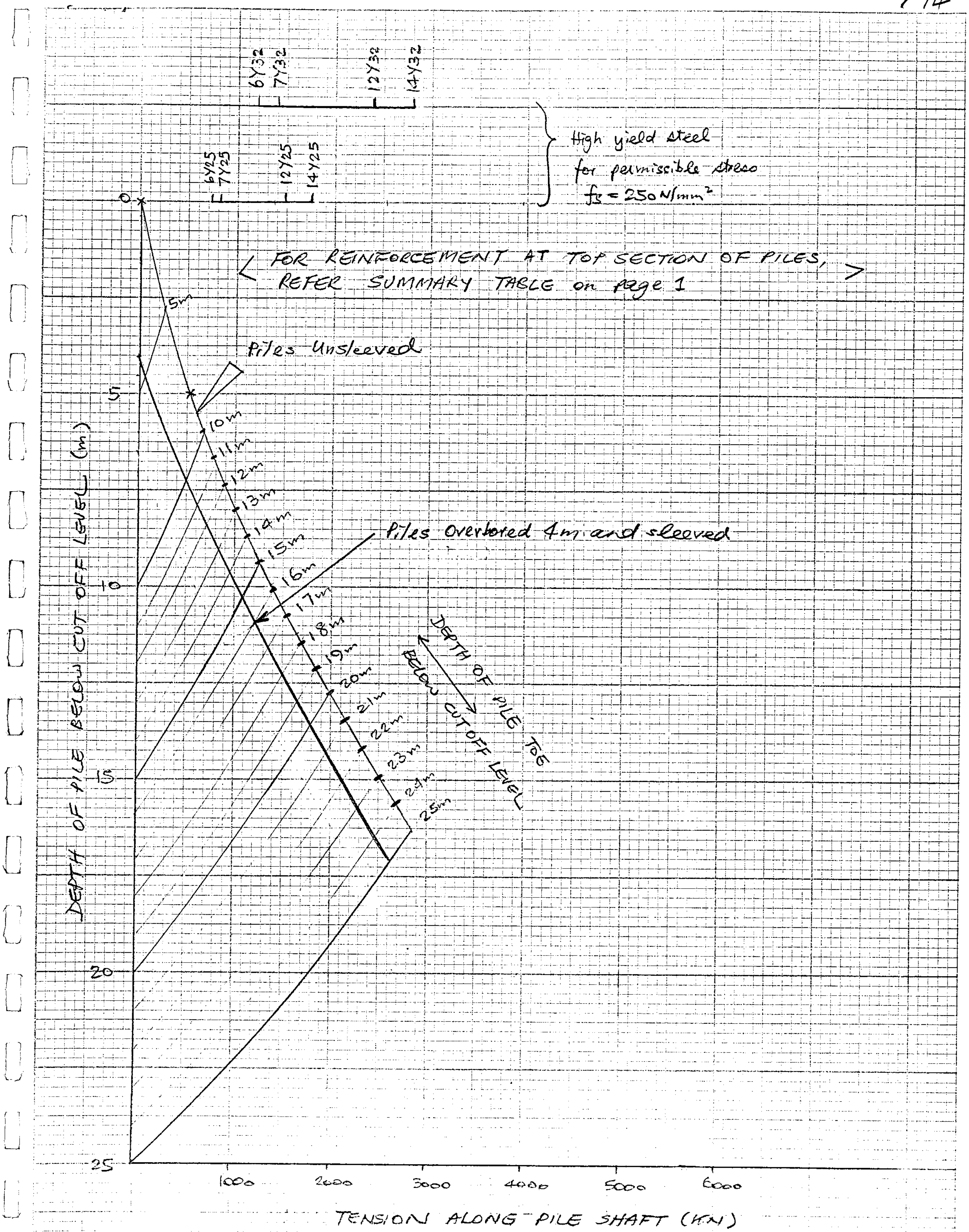
Length of pile (m)	Max shaft force, T (kN)	Max tension on pile (kN) ($\frac{1}{2}$ x max. shaft force)
0	0	0
5	536	268.0
10	1375	687.5
15	2516	1258.0
20	3960	1980.0
25	5706	2853.0

See Design Chart.



DESIGN CHART FOR 600mmϕ PILES WITHIN HEAVE AREA

AT/11/7/83



DESIGN CHART FOR 500mm∅ PILES WITHIN HEAVE AREA

18727

AT/10/8/83

CALCULATION SHEET
Piles subject to additional tensile forces within heave area

12727 TF1/4

Member / Location

Drg.Ref.

Job Title Granary Site

Made by A.T. Date 23.8.83 Chd.

PILES SUBJECT TO ADDITIONAL TENSILE FORCE

It was noted in the correspondence on 12 August, 1983 from R Traver's Magaw that there will be uplift pressure exerted to the underside of the beams and pile caps as the clay heave to piles along gridlines D/1 – 8 and N/8 – 10. It is anticipated that these tensile forces to piles will be short term and will be relieved by the building weight as the construction progresses.

The piles affected included (see attached letter)

Gridline	Pile Nos.	Short term max. tensile force (kN)
D/1 – 8	A7, A13, A21, A25, A29, A30, A31, A35 (8 Nos)	600
N/8 – 10	W2, W3, W4, W16 – W20, W36 – W39, W205 (13 Nos)	400

Method of calculating reinforcement

Because of the existence of tensile forces at the top of these piles, additional reinforcement is required to resist these forces and to prevent excessive cracking to the piles.

It is noted that the above piles will also experience tensile forces on the pile shaft as they are within area of heave. Except for piles W36 – W39 & W205, all of these piles are overbored and sleeved.

Design of steel reinforcement for piles overbored and sleeved at the upper 4m (6.5m for piles adjacent to TWA sewer) within the heave area has been made on 21 July, 83, with

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CALCULATION SHEET

Job No.	Part No.	Rev.
12727	TF1/4	

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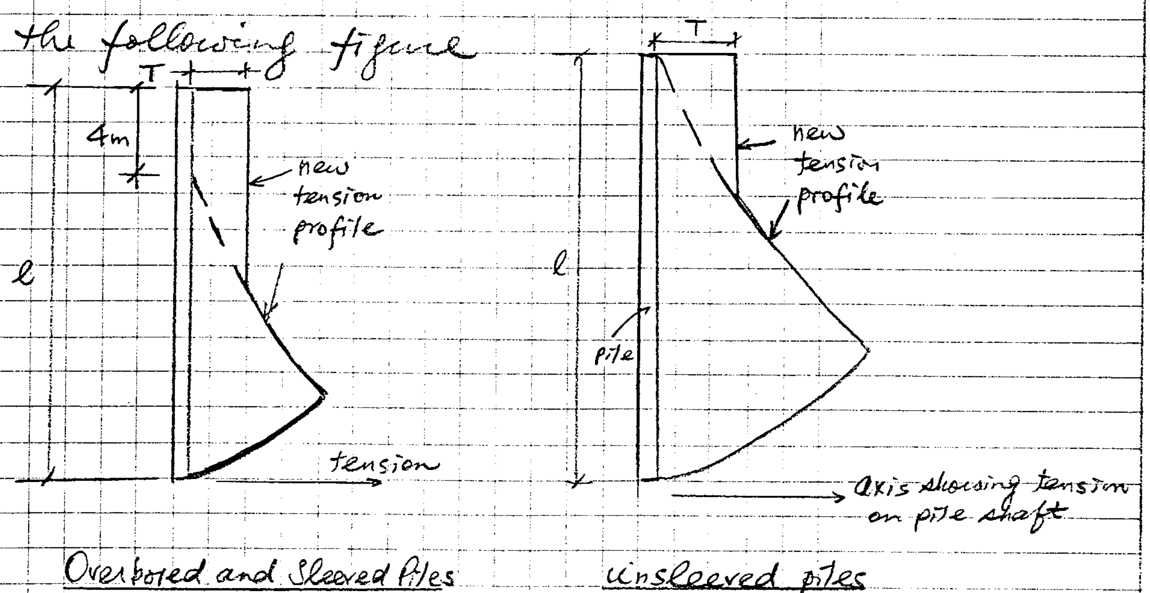
Job Title

Made by AT. Date 23.8.83. Chd.

the recommended steel reinforcement required to be 0.5%. Subsequently, it is noted that the sleeved section would act as a column. Hence, the minimum amount of area of steel is 1% according to CP110, Cl. 3.5.1.1.

Since the piles have been designed to experience tension on the pile shaft; a design chart has been produced to calculate steel reinforcement of them. The additional tension at the top section can be shown in the following figure



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CALCULATION SHEET

Job No.	Sheet No.	Rev.
12727	TF 3/4	

Member / Location

Drg. Ref.

Job Title

Made by A.T. Date 23.8.83 Chd.

Calculation of steel reinforcement with additional tensile force

(i) Short term tensile force = 600 KN

Piles along Gridline D/1-8.

For steel reinforcement to resist tension,

$$T = A_s f_s$$

Where T = tensile force,

A_s = area of steel reinforcement

$f_s = 250\ N/mm^2$ allowable stress to control cracking in concrete.

$$\therefore A_s = \frac{T}{f_s} = \frac{600 \times 10^3}{250} = 2400\ mm^2$$

USE 6Y25 $(A_s = 2945\ mm^2)$

$$\text{Actual } \frac{100 A_s}{A_c} = \frac{2945}{\frac{\pi}{4} \times 600^2} = 1.04\%$$

The amount of steel complies with CP110, Cl 3.5.1.1.

(ii) Short term tensile force = 400 KN

Tension $T = A_s f_s$

$$\therefore A_s = \frac{T}{f_s} = \frac{400 \times 10^3}{250} = 1600\ mm^2$$

$$\text{or } \frac{100 A_s}{A_c}(\text{required}) = \frac{1600 \times 100}{\frac{\pi}{4} \times 600^2} = 0.57\% \Rightarrow A_s = 1612\ mm^2$$

(a) Overbored and sleeved piles

This is less than 1% required by CP110, Cl. 3.5.1.1

$\therefore$ USE 6Y25 $(A_s = 2945\ mm^2)$

(b) Unsleeved Piles (W36-W39, W208) require 6Y20 $(A_s = 1885\ mm^2)$

Recommend 6Y25 $(A_s = 2945\ mm^2)$

It is considered not necessary to design steel reinforcement to the above piles for combined tension and bending moment due to lateral soil displacement for the short term because:

(1) The tension and bending moment will be unlikely to occur at the same time during the construction of the building (Refer paragraph 2 of attached letter)

(2) If they occur at the same time, the crack sections of the pile will reduce the pile stiffness, hence bending moments will be significantly reduced.

SUMMARY OF STEEL REINFORCEMENT SUBJECT TO TENSILE FORCE

Gridline	Pile Nos.	As (mm^2)	Main Reinforcement	Shear Reinforcement
D/1-8	A7, A13, A21, A25, A29-31, A35	2945	6Y25	Nominal Reinforcement Required R8-300c/c.
N/8-10	W2-W4, W16-W20, W36+B9, W205	2945 1600	6Y25 +6Y25	

+ Recommended value, see calculation for detail.

1510 TF5

R TRAVERS MORGAN & PARTNERS

Consulting Engineers

136 LONG ACRE LONDON WC2E 9AE
TELEPHONE 01-836 5474 TELEX 8812307

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A GOLDSTEIN CBE BSc(Eng) ACGI DIC FEng FICE FIStructE FCIT FIHE
R L WILSON BSc(Eng) FICE FIHE
K C WHITE BSc(Eng) MICE FIStructE FIHE
C J HOLLAND BSc DipTP MICE MRTPI MCIT
L D TURZYNSKI FICE FIStructE FIHE
B G HORNE BSc(Eng) ACGI MICE MIStructE
M P CROCKER DipCE MICE FIPHE

Consultant
G MOULD BSc(Eng) FICE FIStructE

Messrs. T.P. Bennett & Son,
262, High Holborn,
London, WC1V 7DU.

Our ref: S.5404/WG/ 12th August, 1983.

Dear Sirs,

NWMLO, Granary Site
Pile Design - Short Term Tensile Forces

Further to our letter of 10th August referring to the design of piles, Mr. Holland of Expanded Piling requested to know the reason for the requirements in paragraph 4(c). These requirements call for a minimum of 6 Y25 bars to resist possible heave forces at the top of the piles.

We explained that we anticipate that along grid lines D/1-8 and N/8-10 small uplift pressures will occur on the underside of the beams and pile caps as the clay heaves. These uplift forces will develop tension forces in the piles in the short term until sufficient building weight is added to the piles. We anticipate that 50% of the tension force will be relieved by building weight within 4 months after completion of the pile caps and 100% within 9 months.

We estimate the maximum tension forces due to this effect will be 600 kN for piles A7, 13, 21, 25, 29, 30, 31 and 35 and 400 kN for piles W2, 3, 4, 16, 17, 18, 19, 20, 36, 37, 38, 39 and 205.

In our letter of 10th August we proposed the provision of a minimum of 6 Y25 bars to resist these forces. As Expanded Piling are responsible for the design of the piles, Mr. Holland wished to know if they should allow for these short term tensile forces in their design or confine themselves to providing a minimum of 6 Y25 bars. We would emphasize Expanded Piling's responsibility and recommend that they should be advised to consider these short term tensile forces and alter their design if they consider it necessary. // OAP

Yours faithfully,
for R. TRAVERS MORGAN & PARTNERS

A. H. D[illegible]

Copy to :
Expanded Piling Co. Ltd. ✓
Messrs. Cyril Sweett & Partners
E.R.

THE EXPANDED PILING CO. LTD.
15 AUG 1983
CATERHAM 40418

Associates
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8.0 PILES SUBJECT TO LATERAL LOADS

Sheets L1 to L16

CALCULATION SHEET

PILES SUBJECT TO APPLIED LATERAL LOAD

12727 | L1 / 16

Member / Location

Drg. Ref.

Job Title: GRANARY SITE | Made by: AT | Date: 28.8.83. | Chd.

PILES SUBJECT TO APPLIED LATERAL LOAD

Some piles as shown below have been designed to resist lateral load of 40KN and 80KN due to wind loading to the building.
(Refer Contract Drawing No. LO(0)1000/J)

PILE NO.	DESIGN HORIZONTAL LOAD (KN)	
A14	80	PILES WITHIN HEAVE AREA (ZONE X)
A15	80	
A18	80	
A19	80	
A22	80	
A23	80	
A26	80	
A27	80	
A31	80	
W8	80	
W10	80	
W202	80	
W7	40	
W9	40	
W75	80	PILES OUTSIDE HEAVE AREA (EXCL. ZONE Y)
W76	80	
W151	80	
W152	80	
W153	80	
W154	80	
W58	40	
W59	40	
W60	40	
W61	40	
W62	40	
W63	40	
W64	40	
W65	40	
W66	40	
W67	40	
W68	40	
W69	40	

(Continue next page)

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12727	L2/16

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Made by A.T. Date 28.8.83.

Pile No.	Design horizontal load (kN)	
W70	40	
W71	40	
W72	40	
W73	40	
W74	40	
W77	40	
W78	40	
W79	40	
W80	40	
W81	40	
W82	40	
W83	40	PILES
W84	40	OUTSIDE
W85	40	HEAVE
W95	40	AREA
W96	40	
W97	40	
W98	40	
W102	40	
W103	40	
W126	40	
W127	40	
W128	40	
W129	40	
W130	40	
W131	40	
W132	40	
W133	40	
W137	40	
W138	40	
W139	40	
W142	40	
W143	40	
W144	40	

CALCULATION SHEET

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12727	L3/16

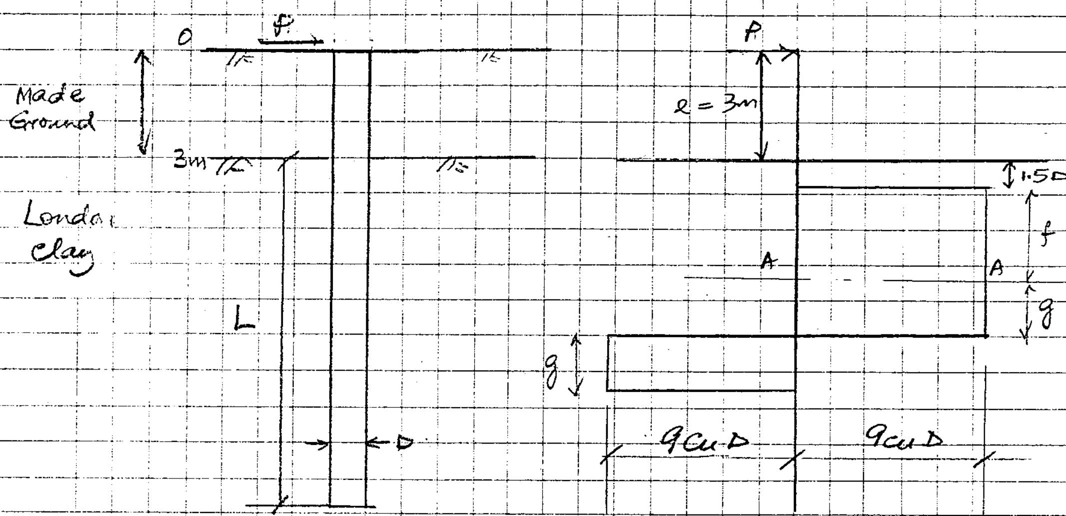
Member / Location

Drg. Ref.

Job Title

Made by AT   Date 12.7.83.   Chd.

Lateral Load



Soil Reaction

L = length of penetration, m.
D = Diameter of pile, m (0.6m)
Cu = undrained shear strength, KPa
P = applied lateral load.

Assumptions

1. Thickness of made ground is 3m outside leave area
2. The pile is unrestrained at the top
3. Use Cu = 100 KPa as average soil strength
4. For calculation of ultimate lateral load and bending moment, reduce the soil strength by a factor of 3.

Design Method

Use method by B.Broms (1965) in
"Design of Laterally Loaded Piles"
Proc ASCE, Vol. 91, SM3, pp.77-99.

CALCULATION SHEET

12727 L4 /16

Member / Location

Org. Ref.

Job Title

Made by AT. Date 28.8.83 Chd.

(A) Reinforcement Design for Piles within area of Heave (ZONE X)

Three categories to be considered:

1. 600 mm diameter piles subject to applied lateral load 80 KN
2. 600 mm diameter piles subject to applied lateral load 40 KN
3. 500 mm diameter piles subject to applied lateral load 80 KN.

Assume eccentricity, e = 1m, for analysis of piles within area of heave.

A.1. 600 mm diameter piles subject to applied lateral load 80 KN

Applied lateral load, P = 80 KN

Undrained shear strength = 100 KN/m²

At working condition, apply a factor of 3,

i.e. $C_w = \frac{C_u}{3} = 33 KN/m^2$

Diameter of pile = 0.6 m

$\therefore \frac{P}{C_w D^2} = \frac{80}{33 \times 0.6^2} = 6.7$

Eccentricity, e = 1 m

$\therefore \frac{e}{D} = \frac{1.0}{0.6} = 1.67$

$\therefore$ Figure 4, $\frac{M}{C_w D^3} = 24$

$\Rightarrow M = 24 \times C_w D^3$
$= 24 \times 33 \times 0.6^3$
$= \underline{171.1 \text{ KNm}}$

$\therefore$ Maximum bending moment is 171.1 KNm

This value of bending moment will be used as ultimate bending moment for the reinforcement design of piles.

CALCULATION SHEET

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Drg.Ref.

Job Title

Made by AT. Date 28.8.83 Chd.

A.1. Reinforcement
(Cont'd)

Maximum axial load = 1700 kN

Design axial load = (1700 × 1.4) (Safety factor = 1.4 to ultimate limit state)
N = 2380 kN

Design bending moment, M = 171.1 kNm

$$\therefore \frac{M}{h^3} = \frac{171.1 \times 10^6}{600^3} \quad (h = 600\text{mm, dia of pile})$$

$$= 0.79 \text{ N/mm}^2$$

$$\frac{N}{h^2} = \frac{2380 \times 10^3}{600^2}$$

$$= 6.61 \text{ N/mm}^2$$

Diameter of cage, h_s = 600 - 2×75 cover = 75mm.
= 450 mm

$$\therefore \frac{h_s}{h} = \frac{450}{600} \doteq 0.8$$

CP110, Part 3, Figure 108, $f_{cu} = 25\text{N/mm}^2$, $f_y = 410\text{N/mm}^2$

$$\therefore \frac{100 A_s}{A_c} = 0.2\%$$

$$A_s = 0.2\% \times \frac{\pi}{4} \times 600^2$$
$$= 565 \text{ mm}^2$$

Because of the possibility of lateral soil movement within embankment area, it is recommended that all the piles should be reinforced with minimum 1% steel. (Also refer calculation for piles in zone Y).

$$\therefore \frac{100 A_s}{A_c} = 1\% \Rightarrow A_s = 1\% \times \frac{\pi}{4} \times 600^2 = 2827 \text{mm}^2$$

$\therefore$ USE 6Y25 ($A_s = 2945\text{mm}^2$)

The above steel reinforcement apply for the following piles : A14, A15, A18, A19, A22, A25, A26 & A27 (8 No.)

CALCULATION SHEET

12727	L6/16

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 28.8.83. Chd.

A.1. (Cont'd) Reinforcement for Pile No. A31

Applied axial load = 900KN

Design axial load, $N = 900 \times 1.4 = 1260KN$

Design bending moment, $M = 171.1 KNm$ (previous calc.)

$$\frac{M}{h^3} = \frac{171.1 \times 10^6}{600^3} = 0.79 N/mm^2$$

$$\frac{N}{h^2} = \frac{1260 \times 10^3}{600^2} = 3.5 N/mm^2$$

CP110, Part 3, Figure 108,

$$\therefore \frac{100 A_s}{A_c} = 0\%$$

As calculation above, steel area is required to be 1%

$$\frac{100 A_s}{A_c} = 1\% \Rightarrow A_s = 2827 mm^2$$

$\therefore$ USE 6Y25 ($A_s = 2945 mm^2$)

CALCULATION SHEET

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Job Title

Made by AT. Date 28.8.83 Chd.

A. 2. 600 mm diameter piles subject to applied lateral load 40KN

Applied lateral load, $P = 40KN$

Soil strength, $C_u = \frac{100}{3} = 33\ KN/m^2$

eccentricity, $e = 3m$

$D = 0.6\ m$

$\therefore \frac{P}{C_u D^2} = \frac{40}{33 \times 0.6^2} = 3.34$

$\frac{e}{D} = \frac{1.0}{0.6} = 1.67$

Figure 4, $\frac{M}{C_u D^3} \doteqdot 11$

$\therefore M = 11 \times 33 \times 0.6^3$

$= \underline{78.4\ KNm}$

$\therefore$ Maximum bending moment $= 78.4\ KNm$

Reinforcement

Applied axial load $= 1700\ KN$

Design axial load, $N = 1700 \times 1.4 = 2380KN$

Design bending moment, $M = 78.4\ KNm$

$\frac{M}{h^3} = \frac{78.4 \times 10^6}{600^3} = 0.36\ N/mm^2$

$\frac{N}{h^2} = \frac{2380 \times 10^3}{600^2} = 6.61\ N/mm^2$

CP110, Part 3, Figure 108,

$\therefore \frac{100 A_s}{A_c} = 0\%$

Recommend to use 1% of reinforcement to be consistent with reinforcement calculation for piles in zone Y.

$\therefore A_s = 1.0\% \times \frac{\pi}{4} \times 600^2$

$= 2827\ mm^2$

USE 6Y25 ($A_s = 2945\ mm^2$)

A.3. 500mm diameter piles (W202) subject to applied lateral load 80KN

Applied lateral load = 80KN

$C_u = \frac{100}{3} = 33 KN/m^2$
$e = 3m$
$D = 0.5m$

$\frac{P}{C_u D^2} = \frac{80}{33 \times 0.5^2} = 9.7$

$\frac{e}{D} = \frac{1.0}{0.5} = 1.67$

Figure 4, $\frac{M}{C_u D^3} = 37$

$\therefore M = 37 \times 33 \times 0.5^3$
$= 152.6 KNm$

$\therefore$ Maximum bending moment is 152.6 KNm

Reinforcement

Applied axial load = 500 KN

Design axial load, $N = 500 \times 1.4 = 700KN$

Design bending moment, $M = 152.6KNm$

$\frac{M}{h^3} = \frac{152.6 \times 10^6}{500^3} = 1.22 N/mm^2$ ($h = 500mm$ dia. of pile)

$\frac{N}{h^2} = \frac{700 \times 10^3}{500^2} = 2.8 N/mm^2$

$h_s = (500 - 2 \times 75) = 350 mm$

$\frac{h_s}{h} = \frac{350}{500} = 0.7$

CP110, Part 2, Figure 109,
$\therefore 100 \frac{A_s}{A_c} = 0.8\% \Rightarrow A_s = 0.8\% \times \frac{\pi}{4} \times 500^2 = 1571 mm^2$

But require min. $\frac{100 A_s}{A_c} = 1\%$ for possibility of lateral soil movement.

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Made by AT. Date 28.8.83 Chd.

From p. L8/16,

$$\therefore \frac{100 A_s}{A_c} = 1\%$$

$$A_s = 1\% \times \frac{\pi}{4} \times 500^2 = 1963 \text{mm}^2$$

Require 4Y25 $(A_s = 1964 \text{mm}^2)$

To control cracking,

Recommend to use 6Y25 $(A_s = 2945 \text{mm}^2)$

CALCULATION SHEET

12727 L10/16

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 28.8.83. Chd.

(B) Reinforcement Design for Piles outside the Heave area

Only two categories are considered:

4. 600 mm diameter piles subject to applied lateral load 80 kN

5. 600 mm diameter piles subject to applied lateral load 40 kN.

B.4. 600 mm diameter piles subject to applied lateral load 80 kN

$\therefore P = 80\,kN$

$C_u = \frac{100}{3} = 33\,kN/m^2$

$D = 0.6\,m$

$e = 3\,m$ (Thickness of fill)

$\therefore \frac{P}{C_u D^2} = \frac{80}{33 \times 0.6^2} = 6.7$

$\frac{e}{D} = \frac{3}{0.6} = 5$

Figure 4, $\frac{M}{C_u D^3} = 43$

$\therefore M = 43 \times 33 \times 0.6^3$
$= 306.5\,kNm$

Reinforcement

Maximum axial load = 1700 kN

Design axial load, $N = 1700 \times 1.4 = 2380\,kN$

Design bending moment, $M = 306.5\,kNm$ (from Calc. Section A1)

$\therefore \frac{M}{h^3} = 1.42\,N/mm^2$

$\frac{N}{h^2} = 6.61\,N/mm^2$

CP110, Part 3, Figure 108,

$\therefore \frac{100 A_s}{A_c} = 1.3\%$ $\Rightarrow A_s = 1.3\% \times \frac{\pi}{4} \times 600^2 = 3676\,mm^2$

Require 8Y25 ($A_s = 3927\,mm^2$)

OAP20

OVE ARUP & PARTNERS
CALCULATION SHEET

Job No.	Sheet No.	Rev.
12727	L11/16.	

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Drg. Ref.

Job Title: Granary | Made by: LL | Date: 8/83 | Chd.

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CALCULATION SHEET

12727	L12/16

Member / Location

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Job Title

Made by AT. Date 28.8.83. Chd.

B.5. 600 mm diameter piles subject to applied lateral load 40 KN

$\therefore P = 40\ KN$

$C_w = \frac{100}{3} = 33\ KN/m^2$

$D = 0.6\ m$

$e = 3\ m$

$\therefore \frac{P}{C_w D^2} = 3.34$

$\frac{e}{D} = 5.0$

Figure 4, $\frac{M}{C_w D^3} = 22$

$\therefore M = 22 \times 33 \times 0.6^3$
$= \underline{156.8\ KNm}$

Reinforcement

Max. axial load = 1700 KN

Design axial load, N = 1700 × 1.4 = 2380 KN

Design bending moment, M = 156.8 KNm

$$\frac{M}{h^3} = \frac{156.8 \times 10^6}{600^3} = 0.73\ N/mm^2$$

$$\frac{N}{h^2} = \frac{2380 \times 10^3}{600^2} = 6.61\ N/mm^2$$

CP110, Part 3, Figure 108,

$\therefore \frac{100 A_s}{A_c} = 0\%$

Since min. of area of reinforcement required = 0.5%.
(Specification, Cl. 16.04)

$\therefore \frac{100 A_s}{A_c} = 0.5\% \Rightarrow A_s = 0.5\% \times \frac{\pi}{4} \times 600^2 = 1414\ mm^2$

USE 6Y20 $(A_s = 1885\ mm^2)$

Find minimum length of steel cage

eccentricity, $e = 3m$
diameter, $D = 0.6m$
$C_u = 33 KN/m^2$

For $L = 3m$ where L = length of penetration into good clay.
i.e. this case, length below cut-off level.

that is, total length of cage below cut-off level

$= L + e = 3 + 3 = \underline{6m}$

$$\frac{L}{D} = \frac{3}{0.6} = 5$$

$$\frac{e}{D} = \frac{3}{0.6} = 5$$

Figure 3, $\therefore \frac{P_{ult}}{C_u D^2} \doteq 3.3$

$\therefore P_{ult} = 3.3 \times 33 \times 0.6^2$
$= 39.2 KN$ < smallest applied lateral load of 40 KN

$\underline{L = 3m \text{ is inadequate}}$.

For $L = 5m$

Total cage length below cut-off level $= 5 + 3 = \underline{8m}$

$$\frac{L}{D} = \frac{5}{0.6} = 8.3 \quad , \quad \frac{e}{D} = \frac{3}{0.6} = 5$$

Figure 3, $\therefore \frac{P_{ult}}{C_u D^2} = 11$

$\therefore P_{ult} = 11 \times 33 \times 0.6^2 = \underline{131 KN}$

This value is equivalent to a factor of $(\frac{131}{80} =) 1.6$ to the largest applied load.

$\therefore L = 5m$ is adequate.

$\therefore$ Total cage length required is 8m below cut-off level

CALCULATION SHEET

12727 | L14/16

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 28.8.83. Chd.

Deflection of laterally loaded piles at ground surface

(Pile Diameter 600mm)

Using Broms' theory,

$$\beta = \sqrt[4]{\frac{K_h D}{4(EI)_p}}$$

where k_h = coefficient of hori. subgrade reaction

D = diameter of pile = 600 mm

$(EI)_p$ = stiffness of pile section.

Take $E_p = 20000\ MN/m^2$

$$I_p = \frac{\pi D^4}{64}$$

$$\therefore (EI)_p = 20{,}000 \times \frac{\pi}{64} \times 0.6^4 = 127.234\ MNm^2$$

Since $k_h = 67 \frac{C_u}{D}$ (Broms, 1972)

Take $C_u = 70$ kPa at top of London clay

$$\therefore k_h = 67 \times \frac{70}{0.6} = 7816\ kN/m^3$$

$$\therefore \beta = \sqrt[4]{\frac{7816 \times 0.6}{4 \times 127.234}} = 0.31$$

For pile length = 21.3m (pile no. A26)

$$\therefore \beta L = 0.31 \times 21.3 = 6.6 > 1.5$$

$\therefore$ long pile

For restrained long pile, deflection at ground (eg. pile cap)

$$y_o = \frac{H\beta}{K_h D}$$

For H=40kN, $= \frac{40 \times 0.31}{7816 \times 0.6} = \underline{2.6\ mm}$

For H=80kN, $y_o = 2.6 \times 2 = \underline{5.2\ mm}$

CALCULATION SHEET

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Job title

Made by AT. Date 28.8.83. Chd.

SUMMARY OF PILES SUBJECT TO APPLIED LATERAL LOAD

(A) Piles within area of Heave (zone X)

Applied Lateral Load (KN)	Pile Diameter (mm)	Pile No.	Required Steel area (mm)	Main Reinforcement
80	600	W8, W10 A14, A15, A18, A19, A22, A23, A26 & A27	2827	6Y25
	600	A31	2827	6Y25
	500	W202	1963	*6Y25
40	600	W7, W9	2827	6Y25

* See calculation for recommended main reinforcement.

(B) Piles outside area of Heave (ie NOT IN ZONES X OR Y)

Applied Lateral Load (KN)	Pile Diameter (mm)	Required Steel area (mm^2)	Main Reinforcement	Cage Length (m)	Calc. hor. deflection (mm)
80	600	3676	8Y25	8.0	5.2
40	600	1414	6Y20	8.0	2.6

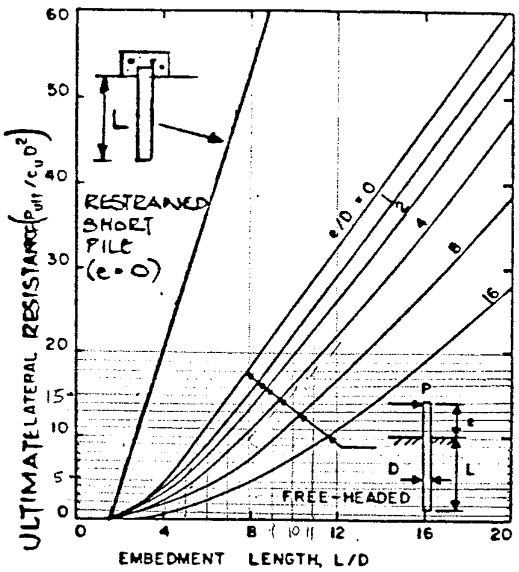

FIG. 3 ULTIMATE LATERAL RESISTANCE FOR COHESIVE SOILS RELATED TO EMBEDMENT LENGTH

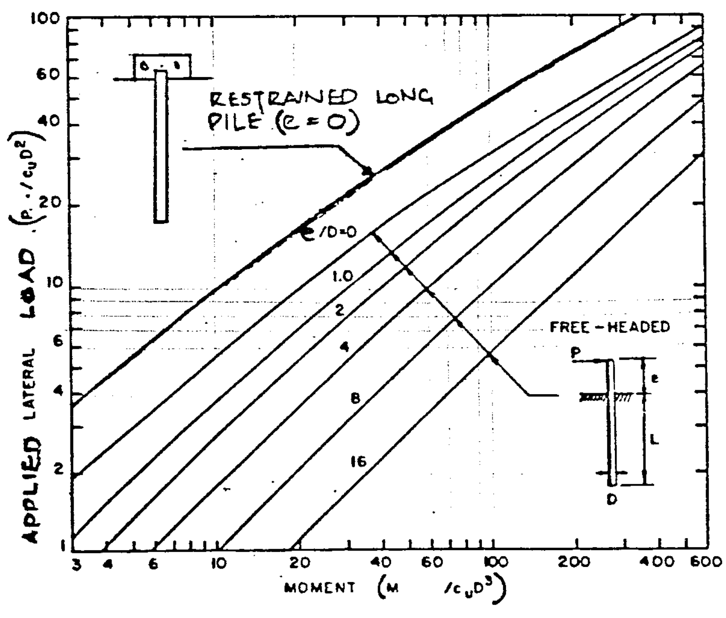

FIG. 4 MAXIMUM BENDING MOMENT IN PILE IN COHESIVE SOIL