## 9.0 PILES SUBJECT TO LATERAL GROUND MOVEMENT.

Sheets LD1 to LD3
and Y1 to Y16

OAP20 **OVE ARUP & PARTNERS**
CALCULATION SHEET

| Job No. | Sheet No. | Rev. |
|---|---|---|
| 12727 | LD 1/3 | |

Member / Location

Drg. Ref.

Job Title: Granary Site. Made by: DS Date: 27/5/83 Chd.

Design of piles for lateral displacement.

Computation of long term lateral movement is difficult because it depends critically on the effective Poisson's ratio and anisotropy of the clay. To obtain an approximate estimate, the following procedure has been followed:

1. Carry out finite element analysis for long term drained condition, using a circular load (90m diameter) as before, but no bored pile wall.
   ($E' = 130c_u$, $\nu = 0.2$)
2. Compare the maximum heave (176 mm) with the maximum heave beneath the building obtained using VDISP and the net loads (70 mm). The difference is due to
   a) the difference between gross and net loads
   b) The complex geometry modelled more accurately by VDISP.
3. Multiply all displacements from the finite element analysis [(1) above]

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| Job No. | Sheet No. | Rev. |
|---|---|---|
| | LD 2/3 | |

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| Job Title | Made by | Date | Chd. |
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| | | | |

by $\frac{70}{176} = 0.4$.

4. Figure LD1 attached is a plot of the lateral movements at ground level obtained in this way. Since these movements would occur as the soil swells and softens, it is unlikely to be able to sustain the high shear stresses required to give a sharp peak to the displacements. It is therefore considered that piles within about 15 m of the embankment at the north of the site should be reinforced to accept 10 mm of lateral movement of the ground relative to the pile caps. **

In addition to piles subject to heave (Zone X) piles in Zone Y will be designed for this lateral movement (Figure LD2)

5. For an applied displacement, the worst case for both bending moment and shear force occurs when the soil is as stiff as possible. Since softening of the clay is necessary to cause movement, it is considered pessimistic to assume $c_u = 100$ kPa.

** Lateral movements due to removal of the rubble bund will be relatively small and can be neglected.

CALCULATION SHEET

12727 LD3 / 3

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Job Title

Made by AT. Date 27.7.83 Chd.

6. Use a subgrade reaction model with no tension allowed on the pile shaft, the coefficient of subgrade reaction is derived with $k_h = 82\ Cu/B$ where B is the pile diameter (Broms, 1972)

7. The behaviour of the pile when subjected to lateral displacement of 10mm has been modelled using program FREW – Flexible Retaining Wall program. Three cases have been considered as below:

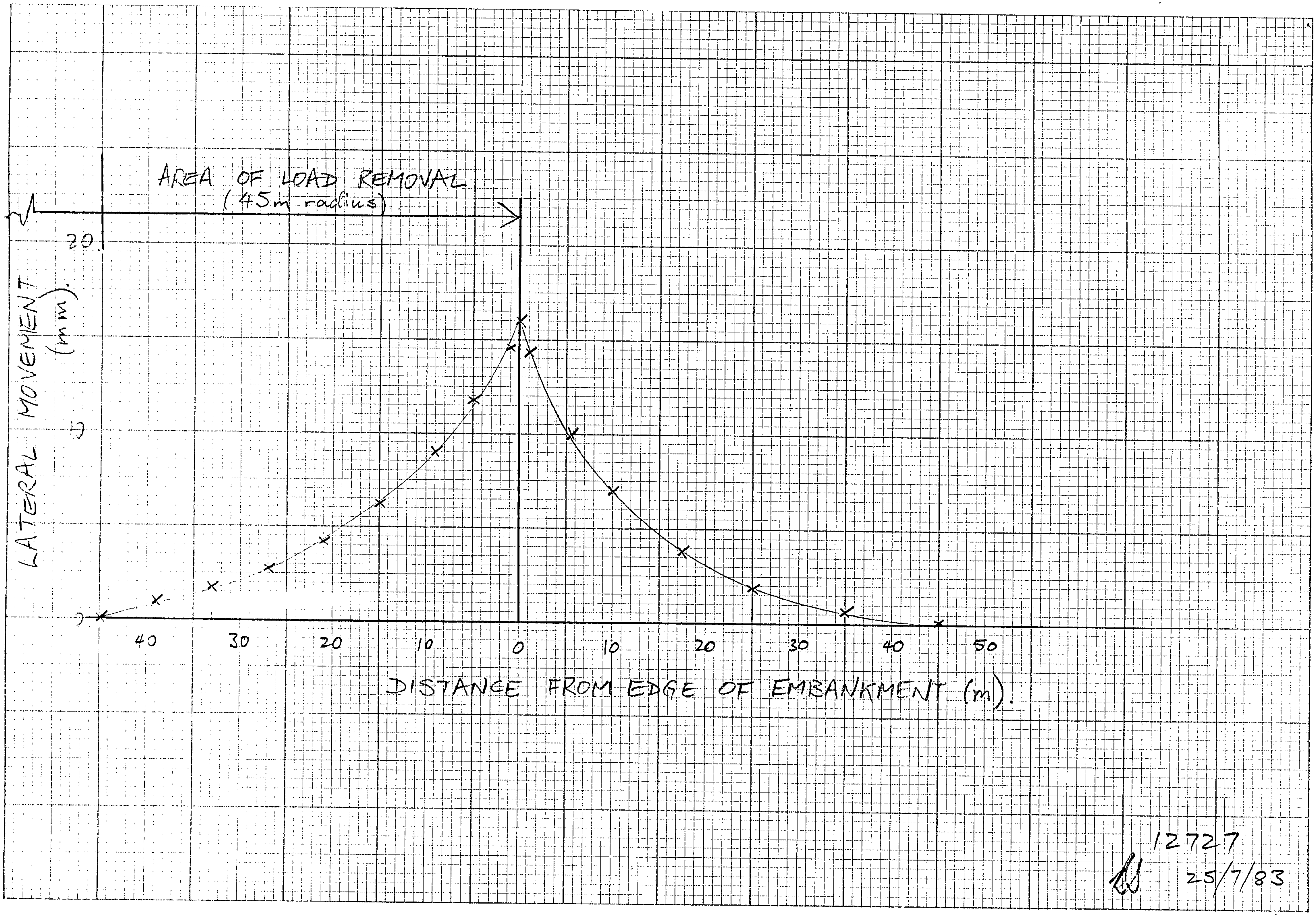
Case 1 : 1.BIL Piles unsleeved, $(EI)_p = 0.13\times10^6\ kNm^2$

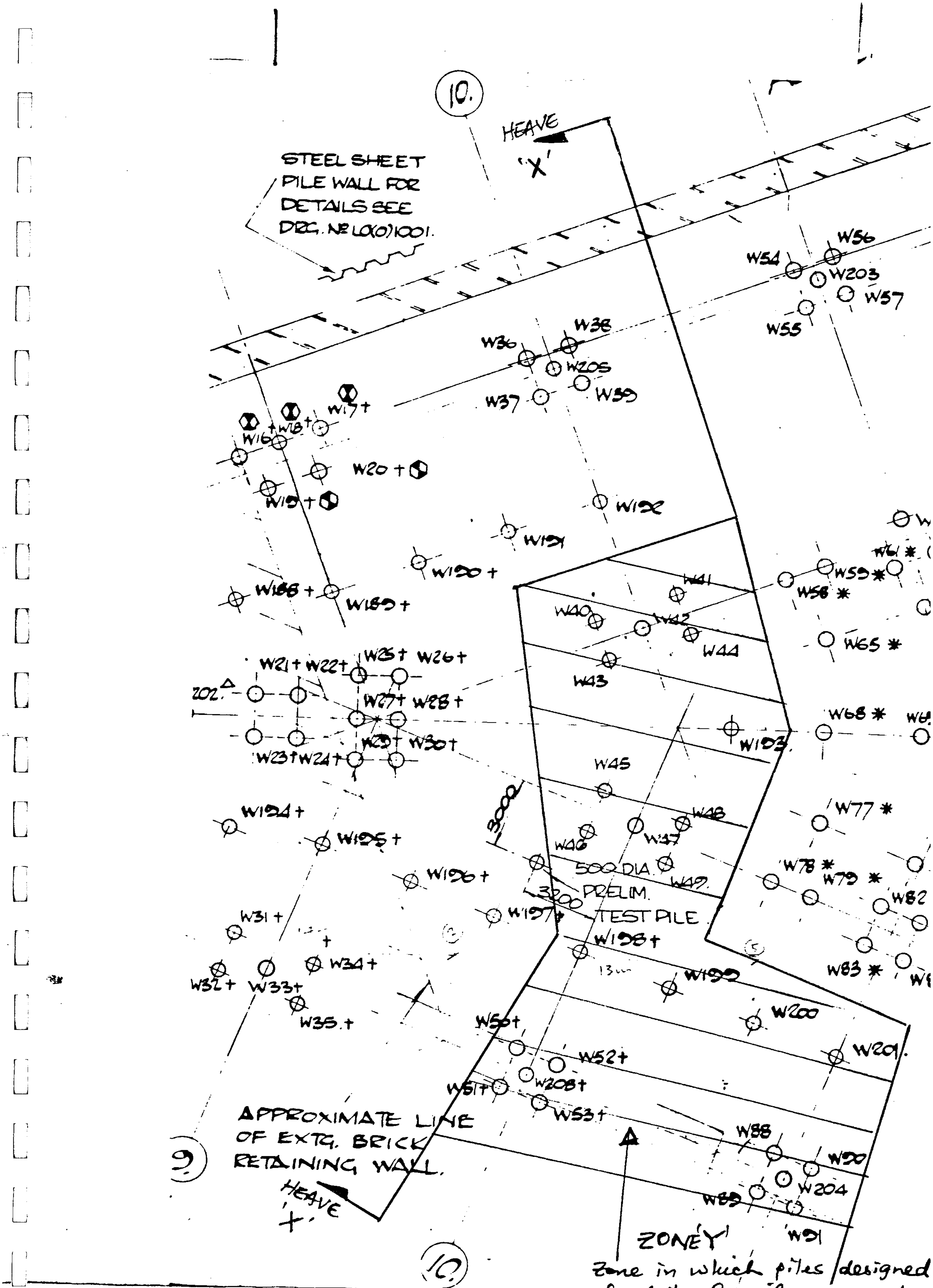
Case 2 : 4.BIL Piles overbored 4m and sleeved $(EI)_p = 0.13\times10^3\ kNm^2$

Case 3 : 5.BIL Piles overbored 4m and sleeved $(EI)_p = 0.065\times10^3\ kNm^2$

A conversion factor of 0.6 was introduced as appropriate to correct from the unit length assumed in the program to the pile diameter. The FREW results are presented in Figure LD3.

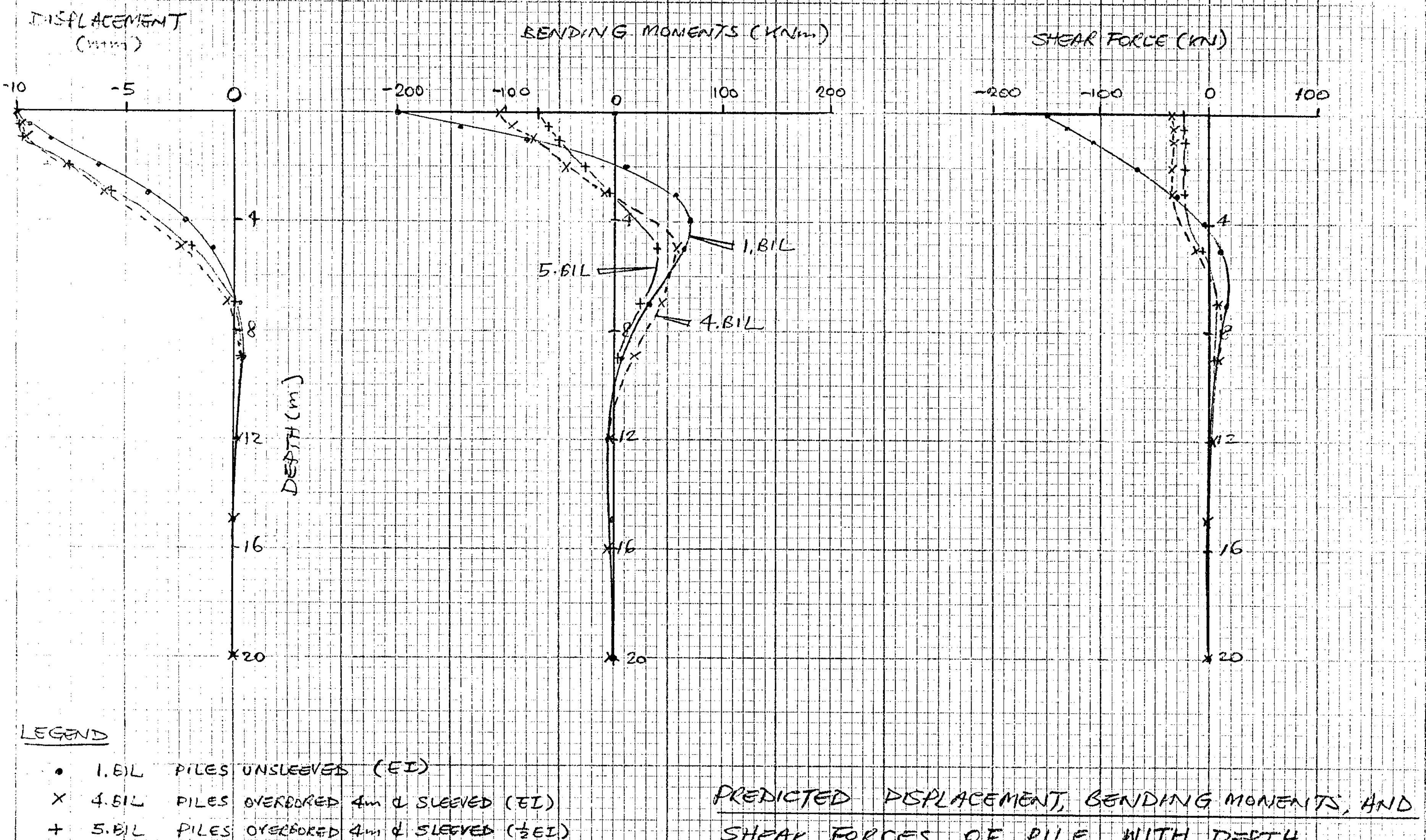
8. The enclosed calculations consider steel reinforcement design of piles both unsleeved and overbored and sleeved. Design of unsleeved piles in Zone Y is based on bending moments and shear forces from 1.BIL result whereas overbored and sleeved piles are from 4.BIL results, both cases being the worst condition.


AREA OF LOAD REMOVAL
(45m radius)
LATERAL MOVEMENT (mm).
20
0
40
30
20
10
0
10
20
30
40
50
DISTANCE FROM EDGE OF EMBANKMENT (m).
12727
25/7/83




12727

FIGURE LD2




PREDICTED DISPLACEMENT, BENDING MOMENTS, AND SHEAR FORCES OF PILE WITH DEPTH

(FREW RESULTS)

FIGURE 1 D.3

CALCULATION SHEET

Piles subject to Lateral Soil Displacement

12727 | Y1/16

Member / Location

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

The following set of calculations presents the reinforcement design of piles subject to lateral soil displacement (zone Y). It is concluded that 1% of steel is required to the piles.

It is considered that piles within area of heave (zone X) should be reinforced for the possibility of lateral soil displacement. Therefore, this requires a minimum amount of 1% steel in those piles.

CALCULATION SHEET
Reinforcement Design of Piles in ZONE Y

12727 | Y2/15
Member / Location
Drg. Ref.

Job Title Granary Site
Made by AT. Date 23.8.83. Chd.

Reinforcement Design of Piles in ZONE Y (Figure LD2)

Within this zone, the soil is assumed to move horizontally by an amount of 10mm, thus exerting a lateral force on all the 25 piles in this area. The piles are divided into the following types:

(i) 600mm diameter piles

1. unsleeved

W40, 41, 42, 43, 44, 45, 46, 47, 48, 49,
W88, 89, 90, 91, 204

2. Overbored & sleeved (sleeved length = 4m below cut-off level)

W50, 51, 52, 53, 208.

(ii) 500mm diameter piles

3. Unsleeved

W 199, 200, 201

4. Overbored & sleeved (sleeved length = 4m below cut-off level)

W198

A computer program FREW was run to calculate the shear forces and bending moments experienced by the piles with soil stiffness $E_s = 4000 KN/m^2$, $\nu = 0.2$.

The following results are used for the reinforcement design:

Run 1 : 1.BIL 600mm unsleeved piles $(EI)_{pile} = 0.13 \times 10^6 KNm^2$

Run 2 : 4.BIL 600mm overbored & sleeved piles $(EI)_{pile} = 0.13 \times 10^6 KNm^2$

Run 3 : 8.BIL 500mm unsleeved piles $(EI)_{pile} = 0.031 \times 10^6 KNm^2$

Run 4 : 9.BIL 500mm overbored & sleeved piles $(EI)_{pile} = 0.031 \times 10^6 KNm^2$

ii) Reinforcement for unsleeved piles – 600mm

Max. axial load = 1700 KN

FREW 1.6H result :

Max bending moment = 198.9 KNm per pile

Max. shear force = 148 KN per pile

For ultimate limit state, use factor 1.4

Main reinforcement

Design $M = 198.9 \times 1.4 = 278.46$ KNm

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

CP110, Part 3, Figure 108,

$f_{cu} = 25\ N/mm^2$, $f_y = 410\ N/mm^2$

$$\frac{h_s}{h} = \frac{450}{600} = 0.75 \text{ (say 0.8)}$$

$$\therefore \frac{M}{h^3} = \frac{278.46 \times 10^6}{600^3} = 1.29\ N/mm^2$$

$$\frac{N}{h^2} = \frac{1530 \times 10^3}{600^2} = 6.61\ N/mm^2$$

$$\therefore \frac{100 A_s}{A_c} = 0.9\%$$

Since $A_s = \frac{\pi}{4} \times 600^2 = 282743\ mm^2$

$\therefore A_s = 0.9\% \times 282743 = 2545\ mm^2$

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

$$\frac{100 A_s}{A_c} = \frac{2945}{282743} = 1.04\%$$

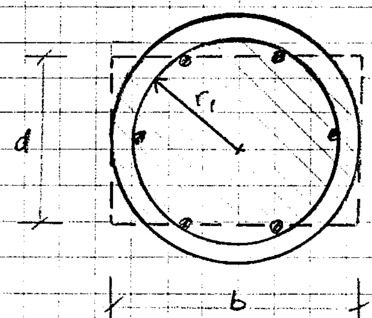
Shear Reinforcement

Design the shear reinforcement of the pile section as if in beams according to CP110, Cl. 3.3.6 where shear stress

$$v = \frac{V}{bd} \qquad (1)$$

where V is ultimate shear force
b is width of rectangular section
d is effective depth

Find equivalent rectangular pile section.




Idealize the circular pile section to be the same as an equivalent rectangular section with $b \times d$

where b is the diameter of the pile
d is the depth of pile such that the top & bottom surfaces are taken to be at compression and tensile reinforcement.

with $d = 2r_1 \cos 30^\circ$ (assuming 6 main bars are used)

Shear Reinforcement (cont'd)

FREW 1.BIL result,
max. shear force = 148 KN

Ultimate shear force = $148 \times 1.4 = 207.2$ KN

Pile section: $b = 600$ mm

$2r_1 = 600 - 2 \times 75 = 450$ mm (cover = 75mm)

$\therefore d = 2r_1 \cos 30°$
$= 450 \cos 30°$
$= 390$ mm

From eqn ①,

$$v = \frac{V}{bd} = \frac{207.2 \times 10^3}{600 \times 390} = 0.89 \ N/mm^2$$

CP110, Table 6, $f_{cu} = 25 \ N/mm^2$

max shear stress, $v_{ult} = 3.75 \ N/mm^2 > v$

$\therefore$ O.K.

CP110, Table 5, $f_{cu} = 25 \ N/mm^2$

Take $\frac{1}{2} \times$ total main reinforcement are in tension;

$$\therefore \frac{100 A_s}{A_c} = \frac{1}{2} \times 1\% = 0.5\%$$

Table 5, $\therefore v_c = 0.5 \ N/mm^2$

$\therefore v > v_c$ require shear reinforcement

Shear reinforcement

$$\frac{A_{sv}}{S_v} \geq \frac{b(v - v_c)}{0.87 f_{yv}}$$

where $A_{sv}$, $S_v$ are area and spacing of shear reinforcement
$f_{yv}$ is yield stress of shear reinforcement

$f_{yv} = 250 \ N/mm^2$ for mild steel

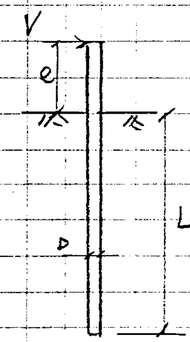
$$\therefore \left(\frac{A_{sv}}{S_v}\right)_r \geq \frac{600(0.89-0.5)}{0.87 \times 250} = 1.08 \text{ mm}$$

USE R12 - 200 mm c/c

$$\frac{A_{sv}}{S_v}(\text{actual}) = \frac{2 \times \pi \times 6^2}{200} = 1.13 \text{ mm} > \left(\frac{A_{sv}}{S_v}\right)_r$$

<u>R12 - 200 mm c/c</u> O.K.

<u>Length of cage</u>




Use method suggested by Broms (1965)
"Design of laterally loaded piles"
Proc. ASCE, Vol. 91, SM3 pp. 79-99.

Assume :

① Pile unrestrained at top

② Fill is 3m thick, i.e. $e = 3$m.

③ Average undrained shear strength $c_u = 100$ kN/m²

④ Soil strength in working condition, $c_w = c_u \div 3$

⑤ Total length of cage $= (L + e)$ below cut-off level of pile

Ultimate applied shear force, $V = 207.2$ kN

$$\frac{V}{c_w D^2} = \frac{3 \times 207.2}{100 \times 0.6^2} = 17.3$$

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

Figure 3, $\frac{L}{D} = 10.8$

$$\therefore L = 10.8 \times 0.6 \doteqdot 6.5\text{m}$$

$\therefore$ Total length of cage below cut-off level

$= 6.5 + 3 = \underline{9.5\text{m}}$ (say $\underline{10\text{m}}$)

i2) Reinforcement for overbored & sleeved piles – 600mm

Max. axial load = 1600 KN

FREW 4.81L results

Max. bending moment = 105.9 KNm per pile

Max shear force = 33.4 KN per pile

Main reinforcement

Design $M = 105.9 \times 1.4 = 148.26$ KNm

Design axial load, $N = 1600 \times 1.4 = 2240$ KN

where factor 1.4 for ULS design.

$$\frac{M}{h^3} = \frac{148.26}{600^3} \times 10^6 = 0.69 \ N/mm^2$$

$$\frac{N}{h^2} = \frac{2240}{600^2} \times 10^3 = 6.22 \ N/mm^2$$

CP110, Part 3, Figure 108,

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

$\therefore$ nominal reinforcement required.

But CP110, Cl 3.5.1.1 states min steel area for column to be 1% (Note: top 4m of pile acted as column)

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

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

i.e USE 6 Y 25 ($A_s = 2945 \ mm^2$)

CALCULATION SHEET

12727 Y8 / 16

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 25.8.83 Chd.

## Shear Reinforcement

FREW 4.B1L results,

max shear force = 33.4 KN

Ultimate shear force = 33.4 × 1.4 = 46.76 KN

Pile section, $b = 600$ mm, $d = 390$ mm (see page Y4)

$$v = \frac{V}{bd} = \frac{46.76 \times 10^3}{600 \times 390}$$

$$= 0.2\ N/mm^2$$

CP110, Table 5, $f_{cu} = 25\ N/mm^2$

for $\frac{100 A_s}{A_c} = \frac{1}{2} \times 1\% = 0.5\%$

$v_c = 0.5\ N/mm^2$

i.e. $v < v_c$

nominal reinforcement required

CP110, Cl 3.11.4.3,

| | | |
|---|---|---|
| | | Y 25mm |
| Min size of links = | $\frac{1}{4}$ × smallest size of main bars | 6.2mm |
| Min spacing of links: | 12 × smallest size of main bars | 300mm |

∴ **USE R8 - 300mm c/c**

## Length of Cage

Figure and assumptions as Case (ii) on page 5,

ultimate applied shear force, $V = 33.4 \times 1.4 = 46.76$ KN

$C_u = 33\ KN/m^2$, $D = 0.6$m, $e = 4$m (sleeved length = 4m)

$$\therefore \frac{V}{C_u D^2} = \frac{46.76}{33 \times 0.6^2} = 3.9 \text{ (say 4)}, \quad \frac{e}{D} = \frac{4}{0.6} = 6.7$$

CALCULATION SHEET

12727 | Y9/16

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 25.8.83. Chd.

Figure 3, $\frac{L}{D} = 5.5$

$\therefore L = 5.5 \times 0.6 = 3.3m$

$\therefore$ Total length of cage below cut-off level

$= 4 + 3.3 = 7.3m$ (say 8m)

CALCULATION SHEET

12727 | Y10/15

Member / Location

Drg. Ref.

Job Title | Made by AT. | Date 25.8.83 | Chd.

ii3) Reinforcement for 500mm unsleeved piles

Max. axial load, N = 550 KN

FREW 8.BIL results where $(EI)_{pile} = 0.031 \times 10^6$ KNm²

Max. bending moment, M = 117.8 KNm per pile

Design $M = 117.8 \times 1.4 = 164.92$ KNm

Design $N = 550 \times 1.4 = 770$ KN

$$\frac{M}{h^3} = \frac{164.92}{500^3} \times 10^6 = 1.32\ N/mm^2$$

$$\frac{N}{h^2} = \frac{770 \times 10^3}{500^2} = 3.08\ N/mm^2$$

$h = 500$ mm, $h_s = 500 - 2 \times 75 = 350$ mm

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

$f_{cu} = 25\ N/mm^2$, $f_y = 410\ N/mm^2$

CP110, Part 3, Figure 109,

$$\therefore \frac{100A_s}{A_c} \doteqdot 0.85\%$$

$$A_c = \frac{\pi}{4} \times 500^2 = 196350\ mm^2$$

$$\therefore A_s = 0.85\% \times 196350 = 1669\ mm^2$$

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

Actual $\frac{100A_s}{A_c} = \frac{1964}{196350} \times 100\% = 1.0\%$

CALCULATION SHEET

12727 Y11 /16

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 25.8.83. Chd.

## Shear Reinforcement

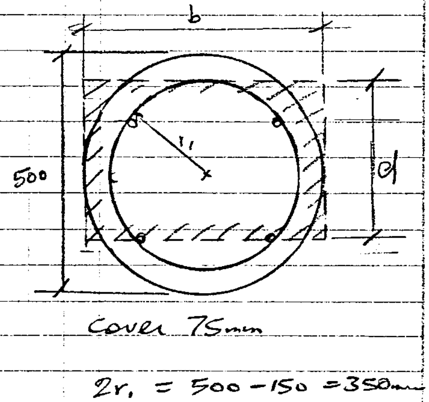
FREW 8.B1L results

Max. shear force = 94.9 KN

Ultimate applied shear force, $V = 94.9 \times 1.4 = 132.86$ KN

Pile Section, 500 mm diameter

Equivalent rectangular section,




$b = 500$ mm

$d = r_1(\cos 45° + 1)$

$= 175(0.7071 + 1)$

$= 298.5$ mm

Shear stress,

$$v = \frac{V}{bd} = \frac{132.86 \times 10^3}{500 \times 298.5} = 0.89 \text{ N/mm}^2$$

CP110, Table 5, $f_{cu} = 25$ N/mm²

for $\frac{100 A_s}{A_c} = \frac{1}{2} \times 1\% = 0.5\%$

$\therefore v_c = 0.5$ N/mm²

$\Rightarrow v > v_c$ require shear reinforcement

$$\left(\frac{A_{sv}}{S_v}\right)_r = \frac{b(v - v_c)}{0.87 f_{yv}} = \frac{500 \times (0.89 - 0.5)}{0.87 \times 250} = 0.90 \text{ mm}$$

USE R12 - 200 mm c/c

Where Actual $\left(\frac{A_{sv}}{S_v}\right) = \frac{2 \times \pi \times 6^2}{200} = 1.13\text{mm} > \left(\frac{A_{sv}}{S_v}\right)_r$

$\therefore$ R12 - 200mm c/c O.K.

CALCULATION SHEET

12727 Y12/16

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 25.8.83 Chd.

<u>Length of cage</u>

Figure and assumptions same as case (i) on page 5,

ultimate applied shear force, $V = 132.86KN$.

$C_w = 33 KN/m^2$
$D = 0.5m$
$\ell = 3m$ (thickness of fill)

$\therefore \frac{V}{C_w D^2} = \frac{132.86}{33 \times 0.5^2} = 16.1$

$\frac{\ell}{D} = \frac{3}{0.5} = 6$

Figure 3, $\therefore \frac{L}{D} = 10.5$

$\therefore L = 10.5 \times 0.5 = 5.25m$

$\therefore$ Total length of cage below cut-off level

$= L + \ell = 5.25 + 3 = 8.25m$ (say <u>8.0m</u>)

Case (ii 4) Reinforcement for 500mm overbored & sleeved piles

Max. axial load, N = 550 KN

FROM 9.BIL results where $(EI)_{pile} = 0.031 \times 10^5$ KNm$^2$

Max bending moment, M = 41.8 KNm per pile

Design M = 41.8 × 1.4 = 58.52 KNm

Design N = 550 × 1.4 = 770 KN

$$\frac{M}{h^3} = \frac{58.52 \times 10^6}{500^3} = 0.47 \text{ N/mm}^2$$

$$\frac{N}{h^2} = \frac{770 \times 10^3}{500^2} = 3.08 \text{ N/mm}^2$$

CP110, Part 3, Figure 109, $f_{cu} = 25$ N/mm$^2$, $f_y = 410$ N/mm$^2$

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

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

nominal reinforcement required

BUT, because top 4m has been sleeved, it acts as a column, CP110, Cl. 3.5.1.1 states min steel area for column to be 1%

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

$A_c = 196350$ mm$^2$

$\therefore A_s = 1964$ mm$^2$

USE 4 Y 25 ($A_s = 1964$ mm$^2$)

$\therefore$ Actual $100 \frac{A_s}{A_c} = 1.0\%$

CALCULATION SHEET

12727 Y14 /18

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 25.8.83 Chd.

Shear Reinforcement

FREW 9.BK results

Max. shear force = 14.0 KN

ultimate applied shear force, V = 14 × 1.4 = 19.6 KN

Pile section

Equivalent rectangular section, (see page 10)

$b = 500mm$
$d = 298.5mm$

shear stress $v = \frac{V}{bd} = \frac{19.6 \times 10^3}{500 \times 298.5} = 0.13 \ N/mm^2$

CP110, Table 5, $f_{cu} = 25 \ N/mm^2$

for $\frac{100A_s}{A_c} = \frac{1}{2} \times 1.0\% = 0.5\%$

$v_c = 0.5 \ N/mm^2$

i.e $v < v_c$

nominal reinforcement required.

CP110, Cl 3.11.4.3,

Since main reinforcement is 4Y25

Shear reinforcement of R8 - 300mm c/c is adequate

Length of Cage

Figure and assumptions same as case (ii) on page 5,

ultimate applied shear force V = 19.6 KN

$C_w = 33 \ KN/m^2$, $D = 0.5m$, $e = 4m$ (sleeved length = 4m)

$\therefore \frac{V}{C_w D^2} = \frac{19.6}{33 \times 0.5^2} = 2.38$

$\frac{e}{D} = \frac{4}{0.5} = 8$

CALCULATION SHEET

12727 Y/5 /15

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 25.8.83. Chd.

Figure 3, $\therefore \frac{L}{D} = 6$

$\therefore L = 6 \times 0.5 = 3m$

Total length of cage below cut-off level

$= L + e = 3 + 4 = 7m$ (use $\underline{8m}$)

12727 Y16 / 16

Member / Location

Drg. Ref.

Job Title

Made by AT. Date 25.8.83. Chd.

SUMMARY OF REINFORCEMENT DESIGN FOR ZONE T PILES

| Pile Dia | Type | Main Reinforcement | Shear Reinforcement | Cage Length below cut-off level |
|---|---|---|---|---|
| 600 | unsleeved | 6Y25 | R12 - 200mm c/c | 10m |
| 600 | overbored & sleeved | 6Y25 | R8 - 300mm c/c | 8m |
| 500 | unsleeved | 4Y25 | R12 - 200mm c/c | 8m |
| 500 | overbored & sleeved | 4Y25 | R8 - 300mm c/c | 8m |

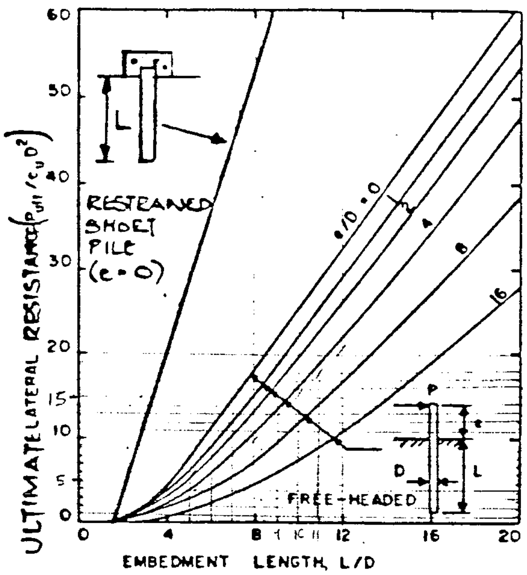



FIG. 3 ULTIMATE LATERAL RESISTANCE FOR COHESIVE SOILS RELATED TO EMBEDMENT LENGTH

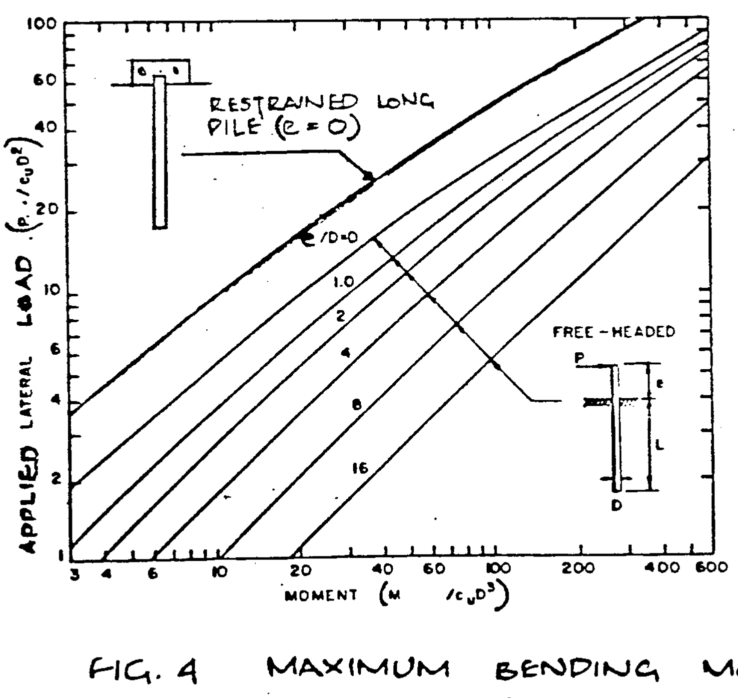



FIG. 4 MAXIMUM BENDING MOMENT IN PILE IN COHESIVE SOIL

Taken from BROMS (1965) "Design of Laterally Loaded Piles" PROC. ASCE, Vol. 91, SM3.

Granary site. 12727

## 10.0 RELATIVE MOVEMENTS OF PILES DUE TO HEAVE

Sheets PM1 to PM6.

OAP20

**OVE ARUP & PARTNERS**
CALCULATION SHEET

Job No. 12727 | Sheet No. PM1/6 | Rev.

Member / Location

Drg. Ref.

Job Title: GRANARY SITE | Made by: BS | Date: 26.8.83 | Chd.

Relative movement of piles in the area of heave.

The following steps have been taken:

1. Compute ground movements at level +6 m OD which will take place in the long term after installation of piles.

2. Estimate settlement gradient caused by differences of length of adjacent piles, assuming that the "45° rule" of clause 8.01.6 of the specification is applied.

Conclusions.

1. The attached figure PM1 shows the computed movement at +6 m OD. This is considered to be representative of the global movements of the piles. In areas of settlement, no allowance is made for the effect of removal of the Granary Warehouse, and settlements may therefore be significantly over-predicted. Nevertheless, the gradient of settlement/heave with plan distance nowhere exceeds 1/800

2 The attached calculations indicate that local gradients of settlement caused by variation of pile length will not exceed 1/1000 provided that the 45° rule is applied.

3. Thus, the worst computed gradient of settlement between adjacent piles is

$$\frac{1}{800} + \frac{1}{1000} = 1/440.$$

4. The assumed toe levels of the piles are indicated on the attached figure PM3.

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**OVE ARUP & PARTNERS**
CALCULATION SHEET

| Job No. | Sheet No. | Rev. |
|---|---|---|
| 12727 | PM 3/6 | |

Member / Location

Drg. Ref.

Job Title

Made by BB  Date 8/83  Chd.

Relative vertical movement of piles due to varying lengths in the area of heave.

The following steps have been followed using output from VDISP runs:

HEUNS — undrained heave (no imposed load)

LODRS — long term (drained) heave for net loading.

1. For 7 typical points in the area of heave the relative displacements of levels +6 m OD and +11 m OD were extracted.
2. The difference $\Delta$ between the long term heave ($\delta_d$) and the short term heave ($\delta_u$) was obtained for each level. This difference $\Delta$ (mm) is considered to correspond to the long term movement of piles for which $\frac{2}{3}$ toe depth is at +6 and +11 m OD.
3. The gradient of $\Delta$ with depth was found.
4. From this it was shown that even for short piles at these high levels the computed gradient of heave

with respect to horizontal position would be less than 1/500 provided the gradient between the toes of the piles is less than 1.07 m (vertical) to 1 m (horizontal).

Thus calculated differential settlements lie within 1/500 provided the "45° rule" of specification clause 8.01.6 is applied.

This calculation is very conservative since application of the "45° rule" requires that no piles in the area of heave have toe levels above +2 m OD. Thus highest "$\frac{2}{3}$ depth" level is +8 m OD. The profile of computed displacement with depth is attached for the VDISP location with the most severe value of $\frac{d\Delta}{dz}$. This indicates that between +8 m OD and 0 m OD the gradient $\frac{d\Delta}{dz}$ is 1.6 mm/m. Thus if the 45° rule is applied the maximum gradient of settlement will be 1.6/1.5 mm/m

i.e. 1.07 mm settlement per 1.0 m in plan (1/940)

(say 1/1000).

| VDISP point | Level +6 $\delta_u$ | Level +6 $\delta_d$ | Level +6 $\Delta$ | Level +11 $\delta_u$ | Level +11 $\delta_d$ | Level +11 $\Delta$ | $d\Delta/dy$ |
|---|---|---|---|---|---|---|---|
| 150 | 14·2 | 24·7 | 10·5 | 19·2 | 43·6 | 24·4 | 2·8 |
| 14 | 13·6 | 23·4 | 9·8 | 18·6 | 42·07 | 23·5 | 2·8 |
| 22 | 10·8 | 10·2 | -0·6 | 13·8 | 23·5 | 9·7 | 2·1 |
| 148 | 12·2 | 21·6 | 9·6 | 16·4 | 38·0 | 21·6 | 2·4 |
| 133 | 8·6 | 12·8 | 4·2 | 12·2 | 26·1 | 13·9 | 1·9 |
| 144 | 12·4 | 22·4 | 10·0 | 17·0 | 39·5 | 22·5 | 2·5 |
| 166 | 13·2 | 20·2 | 7·0 | 17·8 | 37·9 | 20·1 | 2·6 |
| 76 | 9·3 | 7·6 | -1·7 | 11·6 | 18·3 | 6·7 | 1·7 |

} worst $\frac{d\Delta}{dy}$

Require $\frac{d\Delta}{dx} < \frac{1}{500} = 2$ mm/m. (x – hori. axis, z – vert. axis)

$\therefore \frac{dy}{dx} < \frac{2}{2\cdot 8} = 0\cdot 71$ m/m.

ie. gradient between $\frac{2}{3}$ points of adjacent piles ≯ 0·71 m (vertical) per m (horizontal)

∴ gradient between bases of adjacent piles ≯ 0·71 × 1·5

= 1·07 m/m.

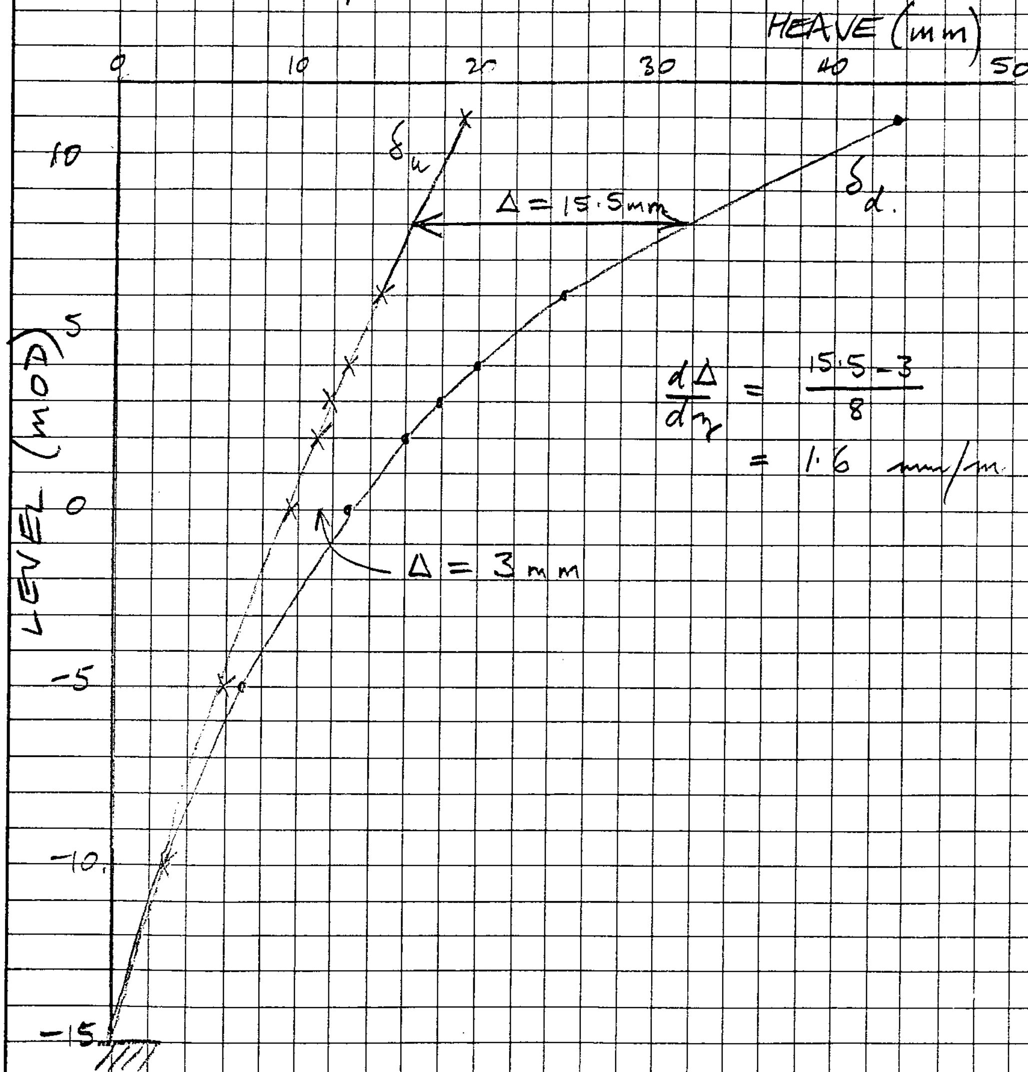
This is slightly less critical than the 45° rule in the specification, clause 8.01.6

OAP20

**OVE ARUP & PARTNERS**
CALCULATION SHEET

| | |
|---|---|
| Job No. | 12727 |
| Sheet No. | PM6/6 |
| Rev. | |
| Member / Location | |
| Drg. Ref. | |
| Made by | RS |
| Date | 8/83 |
| Chd. | |
| Job Title | |

VDISP point 150




$$\frac{d\Delta}{dy} = \frac{15.5-3}{8}$$

$= 1.6$ mm/m