OAP20

OVE ARUP & PARTNERS
CALCULATION SHEET

Job No. 12727 | Sheet No. 1.6 | Rev.

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Drg Ref.

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 hori deflection (mm)
80	600	3676	8Y25	8.0	5.2
40	600	1414	6Y20	8.0	2.6

2.0 INTRODUCTION

2.1 Summary of Calculations

In addition to normal building loads, some of the piles at this site will be subject to axial and/or lateral loads caused by ground movements. These result from the removal of overburden shortly before construction of the piles.

The likely magnitudes of these ground movements have been considered (section 3.0), and on the northern boundary of the site a bored pile wall has been installed to reduce the deformation of an adjacent building (section 4.0).

In section 6.0 it is concluded that piles in one area of the site (Zone X, see Figure 1/1, attached) must be reinforced for tension due to heave. In section 9.0 it is concluded that piles in an additional area (Zone Y) must be reinforced to withstand lateral forces due to horizontal movements in the ground. It is also considered prudent to check that piles in Zone X have at least similar reinforcement to those in Zone Y.

Piles subject to lateral loads from the structure are considered in section 8.0, and future movements of piles due to heave are considered in section 10.0. A complete summary of pile designs is presented in section 1.0.

2.2 Documents considered

1. Specification for bored piling works (etc.) of R. Travers Morgan and Partners. Appendices 2/1, 2/2 and 2/3 of the Bill of Quantities for Piling and Ancillary Works.

2. Travers Morgans' drawings as follows:

5405/LO(O)	1000	(J)
	1001	(C)
	1002	(C)
	1003	(B)
	1004	(D)
	1005	(D)
	1006	
	1007	(C)
/LO(11)	1001	(C)
	1002	(A)

2.2 cont'd

3. Architect's drawings:

   5359/L(SVR)01
   5359/LO(SVR)04

4. LMS drawing "St. Pancras Grain and Ale Stores".

5. Draft copy. Schedule of condition of party wall between the properties: Granary Site, St. Pancras Way, London NW1 and No. 8 St. Pancras Way, London NW1. Prepared by T.P. Bennett and Son.

6. Site Investigation Report By Wembley Laboratories Limited. St. Pancras Way, Camley St., NW1.
   Volume 1 - Boring records and test data 1870/JAD, June 1980.

2.3 References

Broms, B.B. (1965) Design of laterally loaded piles. Proc. ASCE, 1965, 91, SM3, 79-99.

Broms, B.B. (1972) Stability of flexible structures (Piles and pile groups). General Report, Fifth Euro. Conf. SMFE, 2, Madrid.

Burland, J.B. and Wroth, C.P. (1975). Settlement of buildings and associated damage. Review paper in Conference on Settlement of Structures, BGS, Cambridge, 1974.

Butler, F.G. (1975) Heavily overconsolidated clays. Review paper in Conference on Settlement of Structures, BGS, Cambridge, 1974.

May, J. (1975) Heave on a deep basement in the London Clay. Conf. on Settlement of Structures, BGS, Cambridge, 1974.

Simpson, B., Calabresi, G., Sommer, H. and Wallays, M. (1980). Design parameters for stiff clays. General Report in 7th ECSMFE, Vol. 5, Brighton, 1979.

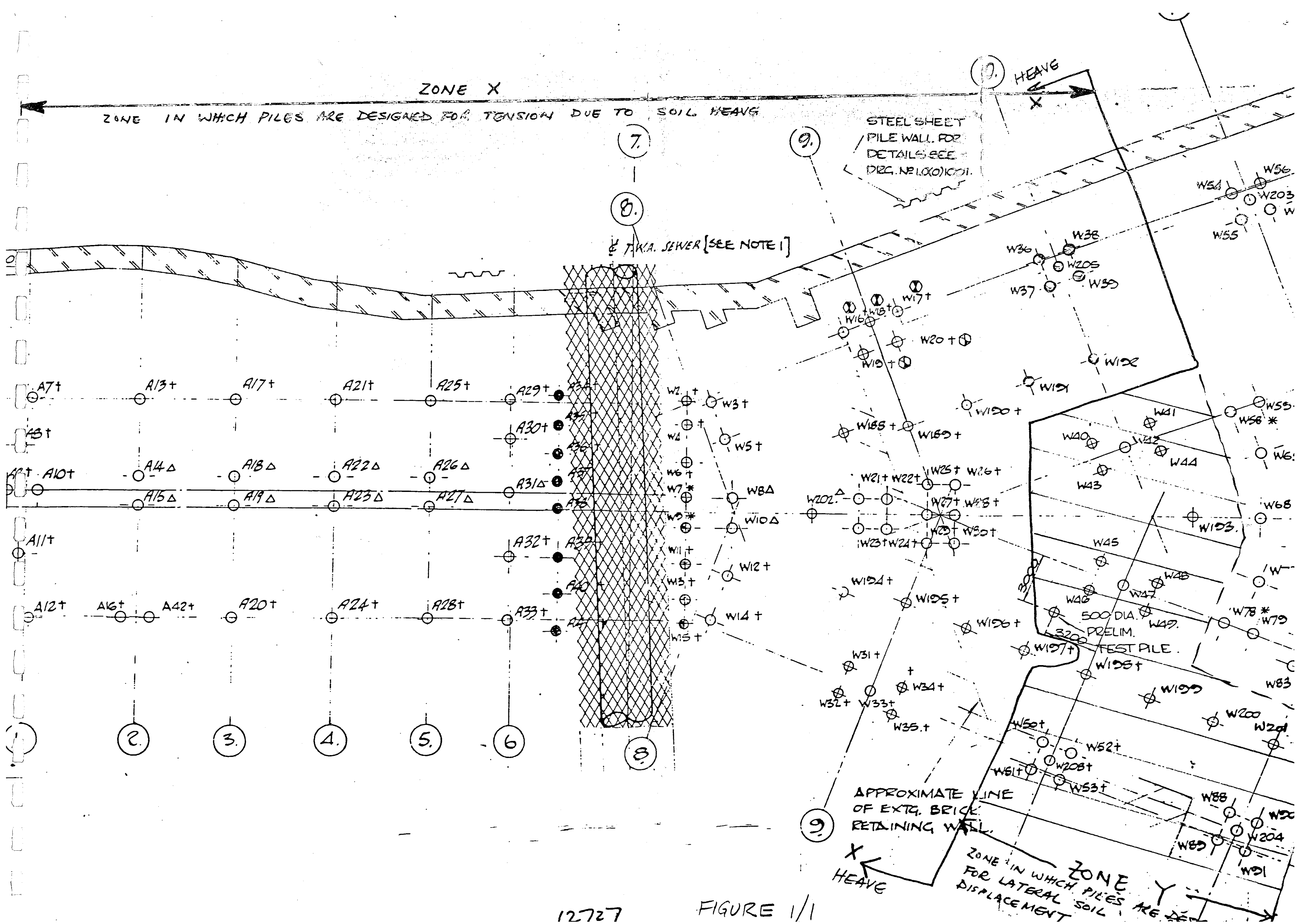

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FIGURE 1/1

3.0 ANALYSIS OF GROUND MOVEMENTS

Sheets 3.1 to 3.7

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ANALYSIS OF GROUND MOVEMENT

Analyses of movement have been carried out using the assumption of linear elasticity, as follows.

1. Program VDISP.

This program computes vertical displacement due to vertical loads on rectangular areas. The stress field due to the loads is computed assuming homogeneous elastic material. Strains are then computed from the stresses using Young's modulus and Poisson's ratio specified for layers of soil. Following Butler (1975), elastic constants were derived from the undrained shear strengths as follows:

Undrained : $E_u = 400\, c_u$ $\nu = 0.5$

Drained : $E' = 130\, c_u$ $\nu = 0.2$

The profile of c_u with depth assumed in the VDISP analyses is shown in Fig. 1.

Butler's relationships were based on measured settlements and are considered to be conservative for prediction of heave. The computed movements should therefore be regarded as an upper bound.

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2. Program SAFE

SAFE is a finite element program for analysis of 2-D problems. In order to obtain an approximate analysis for this site, the following techniques have been used.

1. An axi-symmetric analysis was used, modelling the loaded area as circular with a diameter of 90m.
2. The bored pile wall was specified to behave in plane stress, with no stiffness in the hoop direction.
3. The stiffness of the bored pile wall was made equivalent to either:
   a) the bending stiffness of the piles, or
   b) The tensile stiffness of the reinforcing steel only.

   In either case the wall was effectively very stiff in tension and very flexible in bending, relative to the stiffness of the soil.
4. The stiffness of the soil was derived using the relationships of Butler, as for VDISP. The profile of c_u with

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depth adopted for SAFE is shown in Fig. 1. As for VDISP, the computed movements should be regarded as upper bounds.

Deformation of adjacent building.

The approach adopted is as follows:

1. Confirm that despite differing assumptions the results from VDISP and SAFE are essentially similar for the adjacent building in the absence of the bored pile wall.
2. Using the results from VDISP, check which sections of the adjacent building would experience unacceptable strains in the absence of the bored pile wall.
3. Using the results from SAFE, check the bending strains in the critical sections of the building, in the absence of the bored pile wall.
4. Check the influence of walls of various lengths.

Summary of results.

For section X3 in Fig. 2, results from VDISP and SAFE are plotted in Figs.

Displacement for undrained computations	Fig. 3-5
Displacements for drained computations	Fig 6-8
Shear stresses on the clay/wall interface	Fig 9.

Computed bending moments and thrusts in the wall are attached in the Appendix

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Check movement of adjacent building using the approach by Burland and Wroth (1974)

This is to use the concept of limiting tensile strain, ε_{lim}, to assess the onset of cracking of the building. For reinforced concrete wall, the max. value of

$$\varepsilon_{lim} = 0.075\%$$

before cracks start to initiate.

The values of the limiting strain ε_{lim} on any part of the building can be obtained in the following figure.

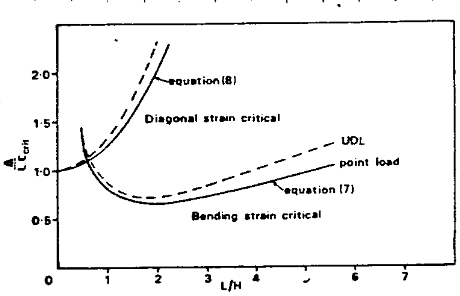

Fig. 6. Relationship between $\Delta/L\varepsilon_{crit}$ and L/H for rectangular beams deflecting due to combined bending and shear–neutral axis in the middle

Eg. Gridline X1, drained result,

$\Delta = 5mm$

$L = 20m$, Height of building, $H \doteq 10m$

$\therefore \frac{\Delta}{L} = 0.00025$

$\frac{L}{H} = \frac{20}{10} = 2.0$

Figure 1 above, for $\frac{L}{H} = 2.0$, $\frac{\Delta}{L} \cdot \frac{1}{\varepsilon_{lim}} = 0.6$

$\therefore \varepsilon_{lim} = \frac{\Delta}{L} \times \frac{1}{0.6} = \frac{0.00025}{0.6} \times 100\% = 0.042\% < 0.075\%$

(no crack)

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Using the above calculation, the rest of the gridlines then follow:

		$\frac{\Delta}{L}$	$\frac{L}{H}$	$\frac{\Delta}{L\varepsilon_{lim}}$	ε_{lim}	
GRIDLINE X1	Drained	0.00025	2.0	0.6	0.045%	VDISP results — no wall. See Fig 10, 11.
	Undrained	0.000053	2.0	0.6	0.009%	
GRIDLINE X2	Drained	0.001	2.0	0.6	0.167% *	
	Undrained	0.00025	2.0	0.6	0.042%	
GRIDLINE X3	Drained	0.0011	2.0	0.6	0.175% *	
	Undrained	0.00025	2.0	0.6	0.042%	
GRIDLINE X4	Drained	0.0009	1.4 $\left(\frac{20}{14}\right)$	0.7	0.129% *	
	Undrained	0.00025	1.4	0.7	0.036%	
GRIDLINE X5	Drained	0.00008	1.4	0.7	0.011%	
	Undrained	0.00008	1.4	0.7	0.011%	
GRIDLINE Y1	Drained	0.0012	2.0	0.6	0.195% *	
	Undrained	0.00027	2.0	0.6	0.045%	
GRIDLINE Y2	Drained	0.00019	2.0	0.6	0.032%	
	Undrained	0.000041	2.0	0.6	0.007%	
GRIDLINE Y3	Drained	0.000062	2.0	0.6	0.010%	
	Undrained	0.000027	2.0	0.6	0.005%	
GRIDLINES X2, X3	Drained 21.6m wall	0.00030	2.0	0.6	0.050%	finite element results. See Fig. 12.
X4	— " —	0.00030	1.4	0.7	0.043%	

It can be seen that if there were no bored pile wall, unacceptable strains are predicted on gridlines X2, X3, X4 and Y1. The results of the finite element analyses indicate that for gridlines X2, X3 and X4 the presence of the wall reduces the strains to acceptable values, however.

Precise analysis for gridline Y1 has not been possible. However, the finite element results indicate that the presence of the wall will reduce the peak heave to about 30 mm in the long term.
For a critical strain of 0.075%, the structure on this gridline would be able to tolerate a differential settlement Δ calculated thus:

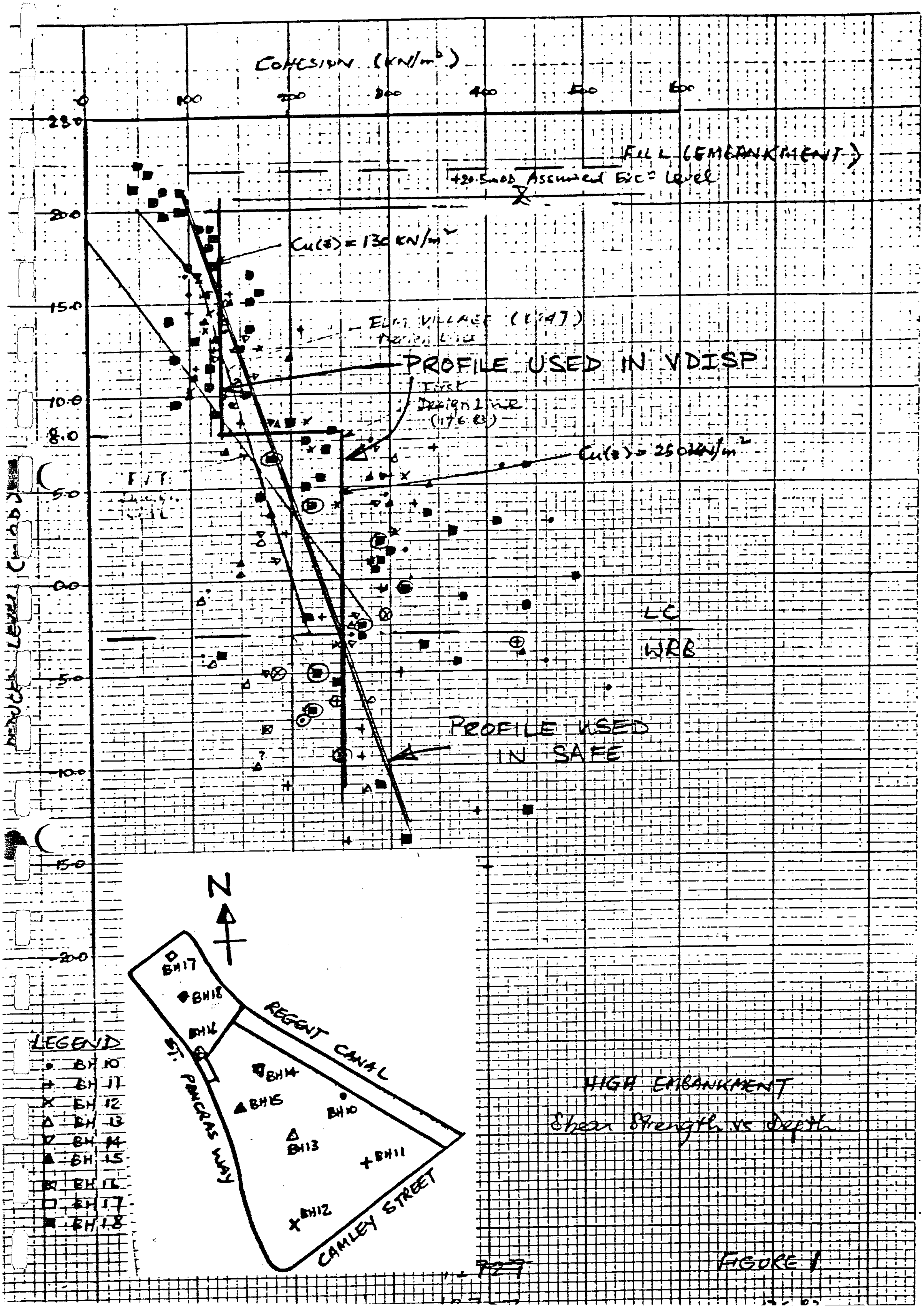
$$\varepsilon_{lim} = 0.075\%$$

$$\frac{\Delta}{L\varepsilon_{lim}} = 0.6 \qquad L = 37m$$

$$\therefore \quad \Delta = 16.7 \text{ mm.}$$

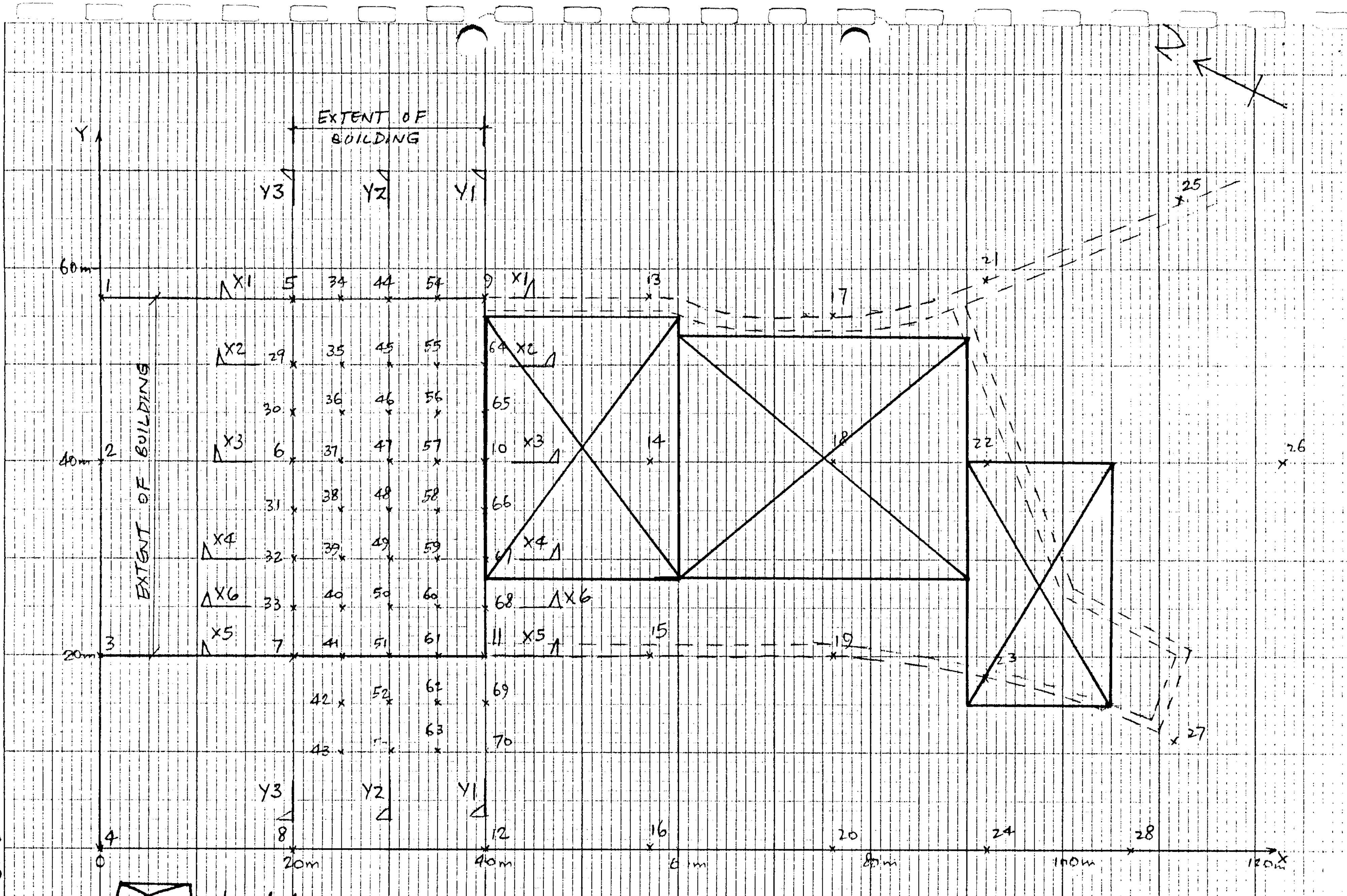
For a peak heave of 30 mm it is unlikely that this value of Δ will be exceeded.

Calculations have not been carried out for grid line X6 for which conditions are similar to X1.



HIGH EMBANKMENT
Shear Strength vs Depth

FIGURE 1



DEFINITION OF INPUT DATA FOR ELASTIC ANALYSIS USING VDISP

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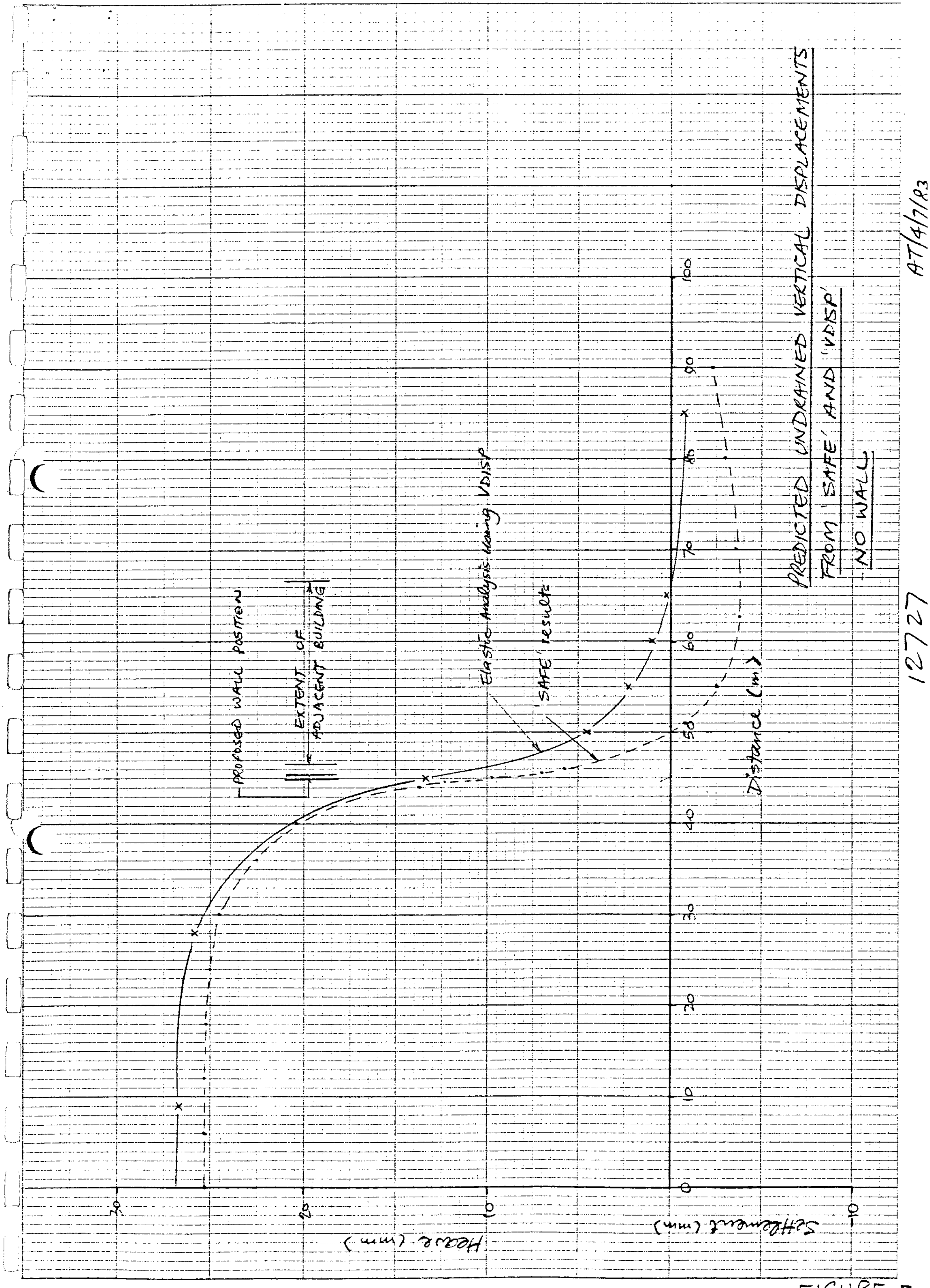
FIGURE 2


PREDICTED UNDRAINED VERTICAL DISPLACEMENTS
FROM 'SAFE' AND 'VDISP'
- NO WALL
AT/4/7/83
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PROPOSED WALL POSITION
EXTENT OF ADJACENT BUILDING
Elastic analysis using VDISP
'SAFE' results
Distance (m)
0
10
20
30
40
50
60
70
80
90
100
Settlement (mm)
Heave (mm)
20
10
10

FIGURE 3

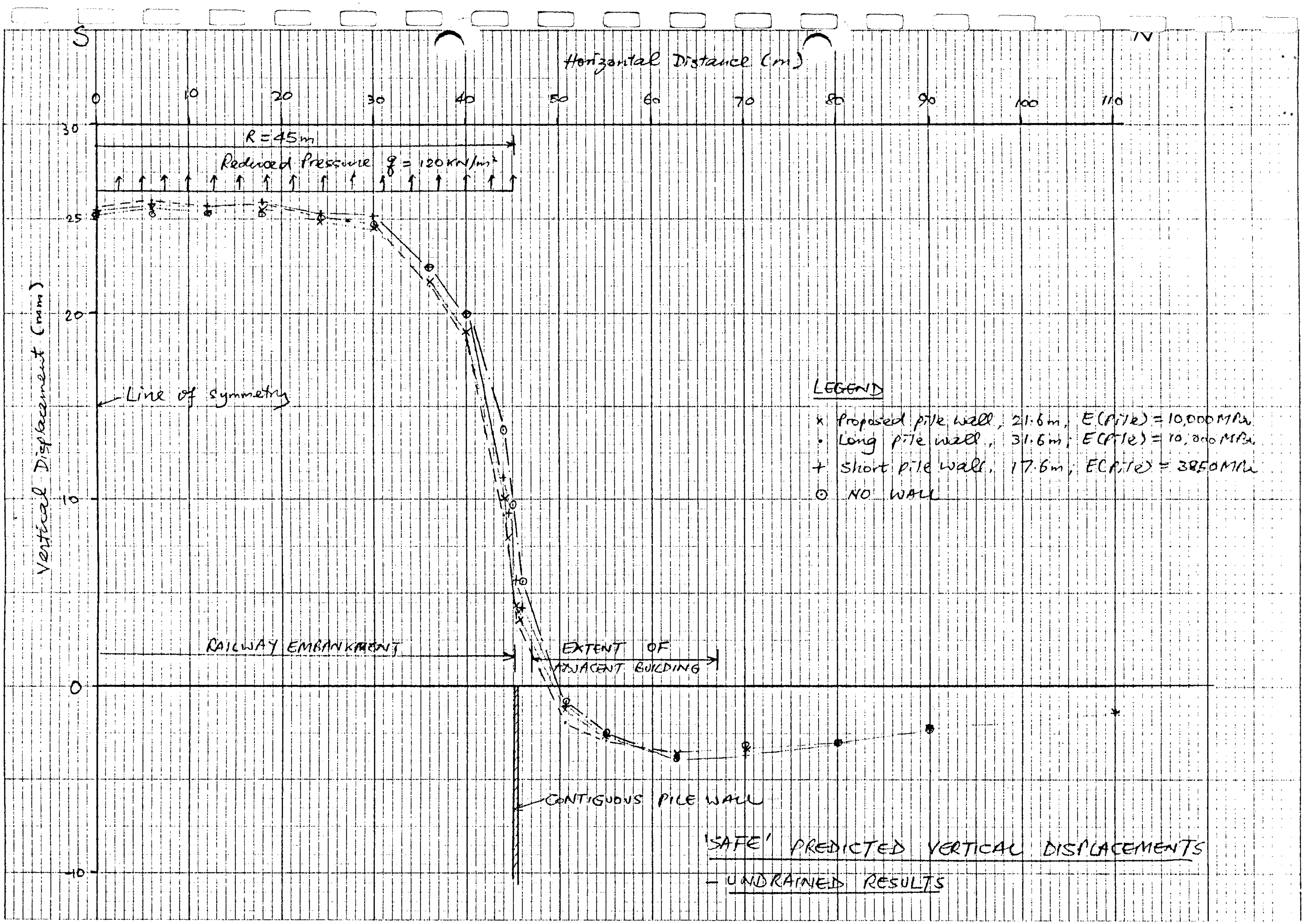

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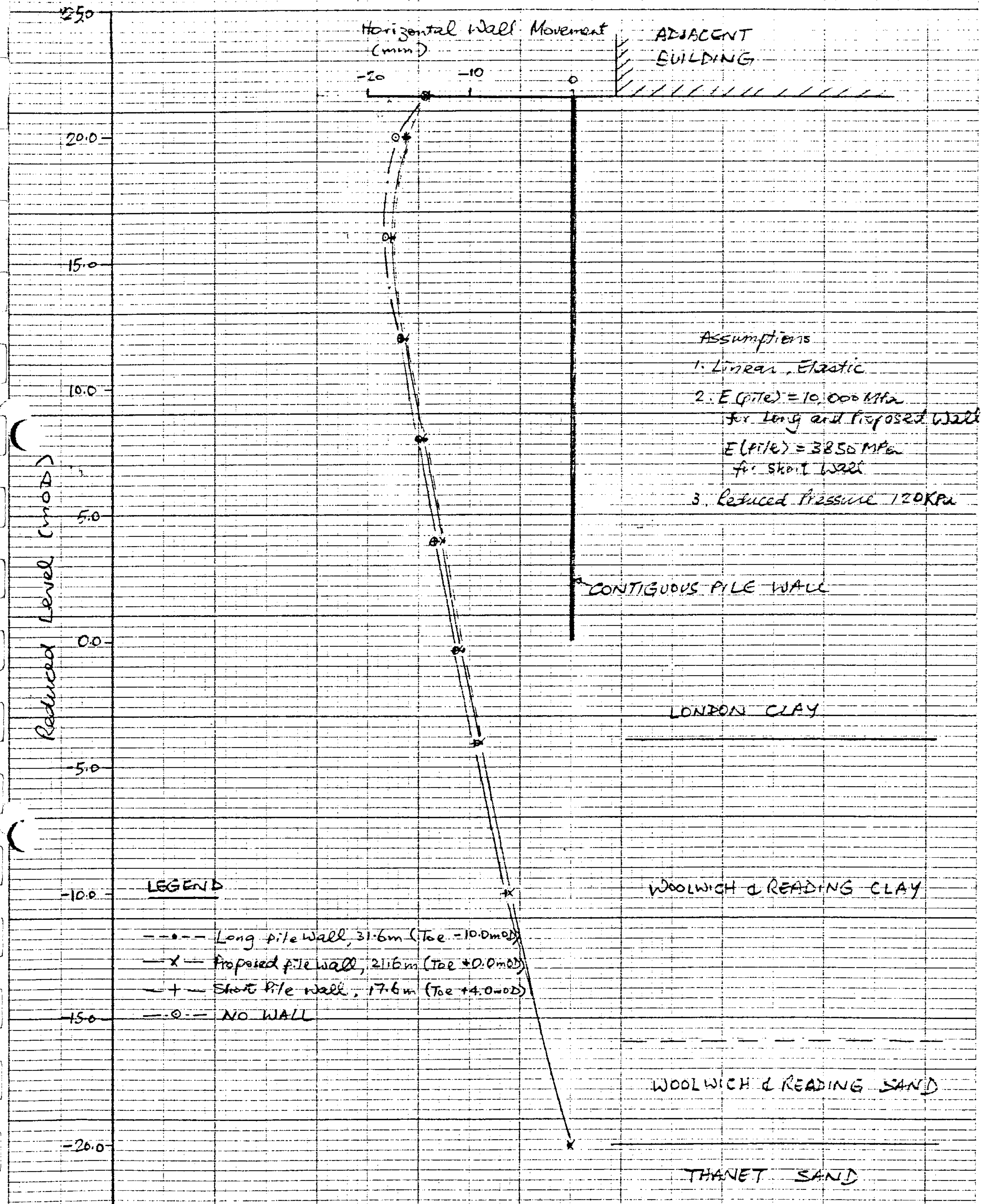
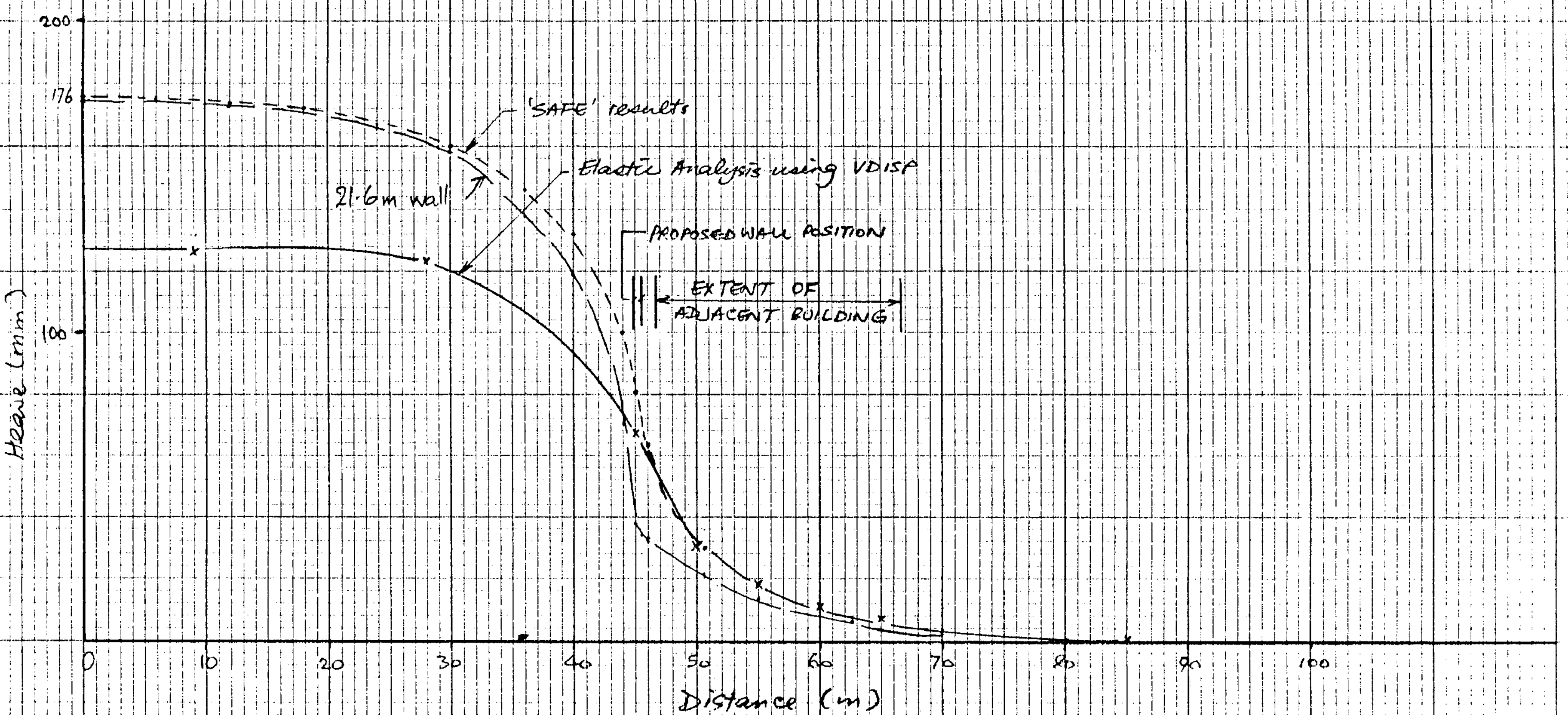
FIGURE 4



IMMEDIATE HORIZONTAL WALL MOVEMENTS
'SAFE' UNDRAINED RESULTS



PREDICTED DRAINED VERTICAL DISPLACEMENTS
FROM 'SAFE' AND 'VDISP'
NO WALL AND 21·6 m WALL.

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AT/4/7/8?

FIGURE 6

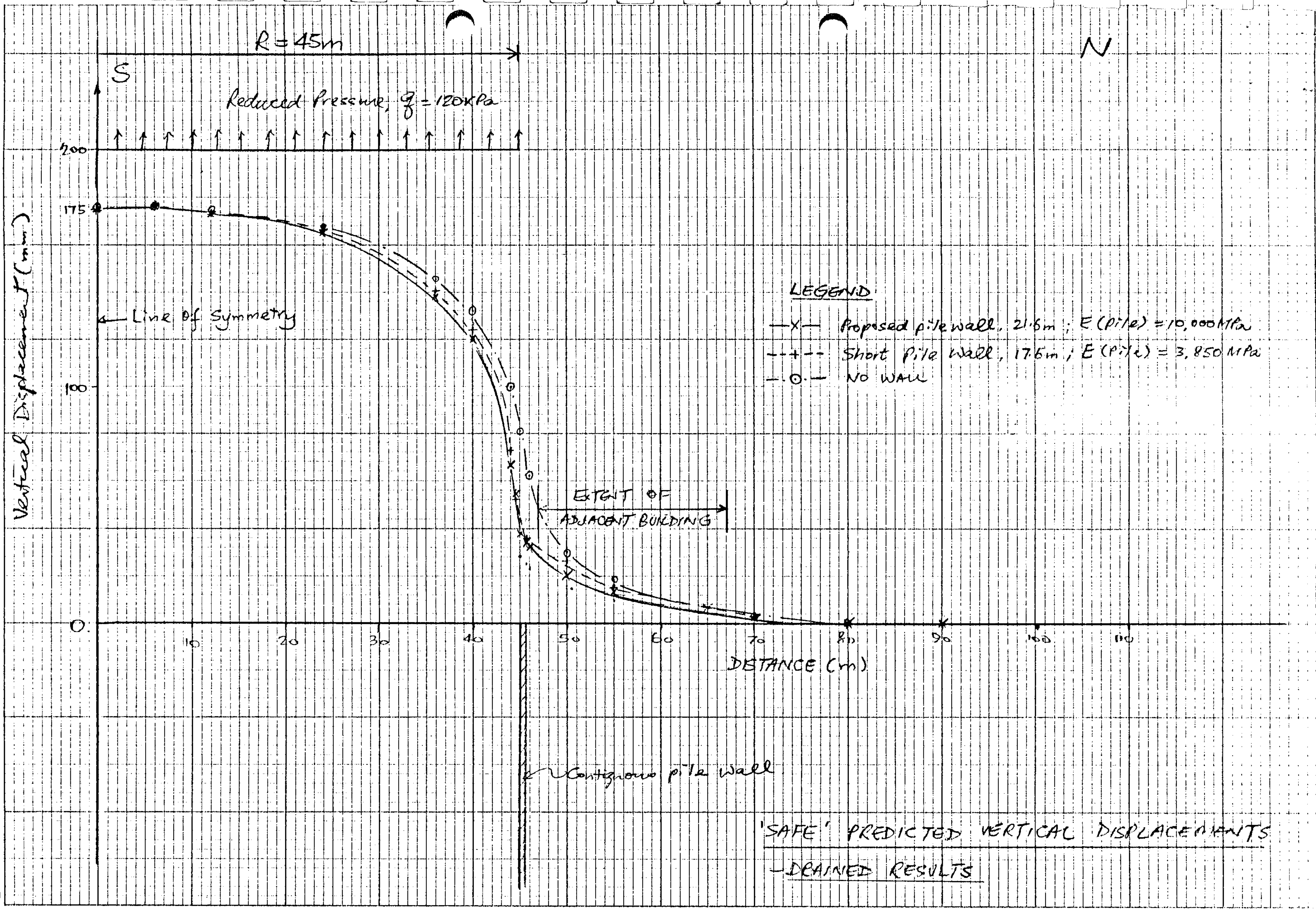

FIGURE 7

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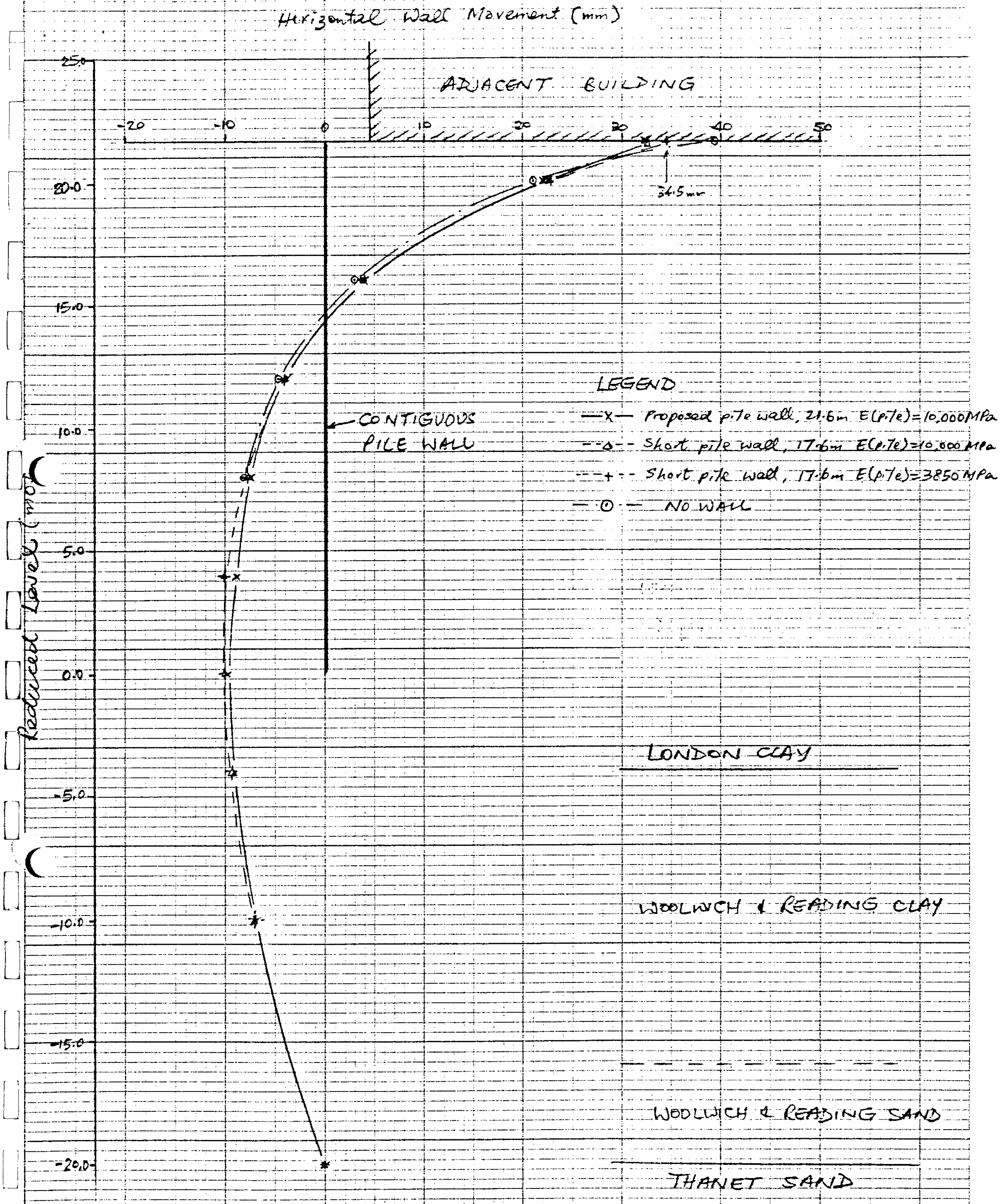
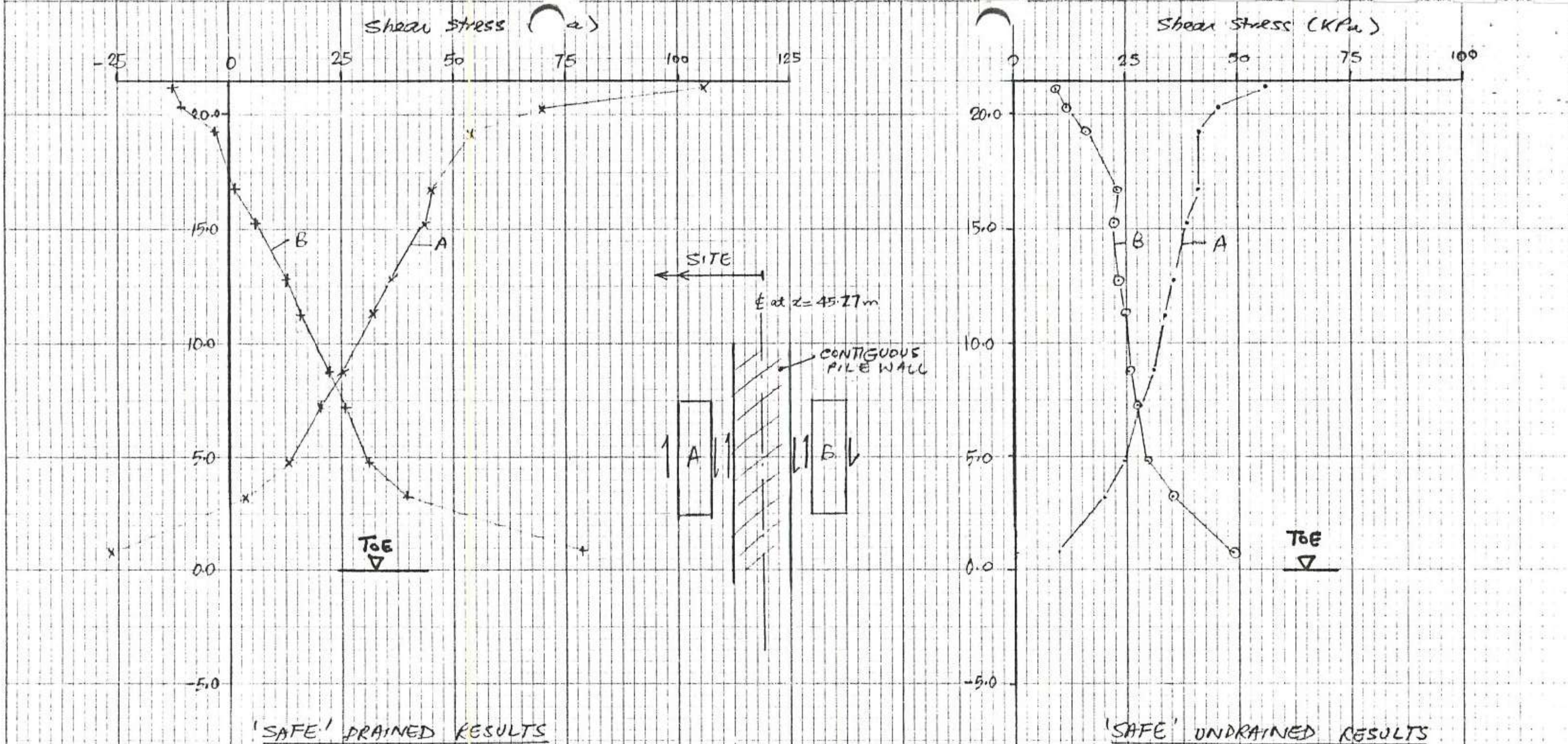
AT 27/6/83



COMPARISON OF LONG TERM HORIZONTAL WALL MOVEMENTS

'SAFE' DRAINED RESULTS



LEGEND

× • } Element A at point x = 44.789m

+ ⊙ } Element B at point x = 45.637m

Sign Convention : +ve clockwise

SHEAR STRESS DISTRIBUTION AT WALL SURFACE

WALL LENGTH = 21.6m

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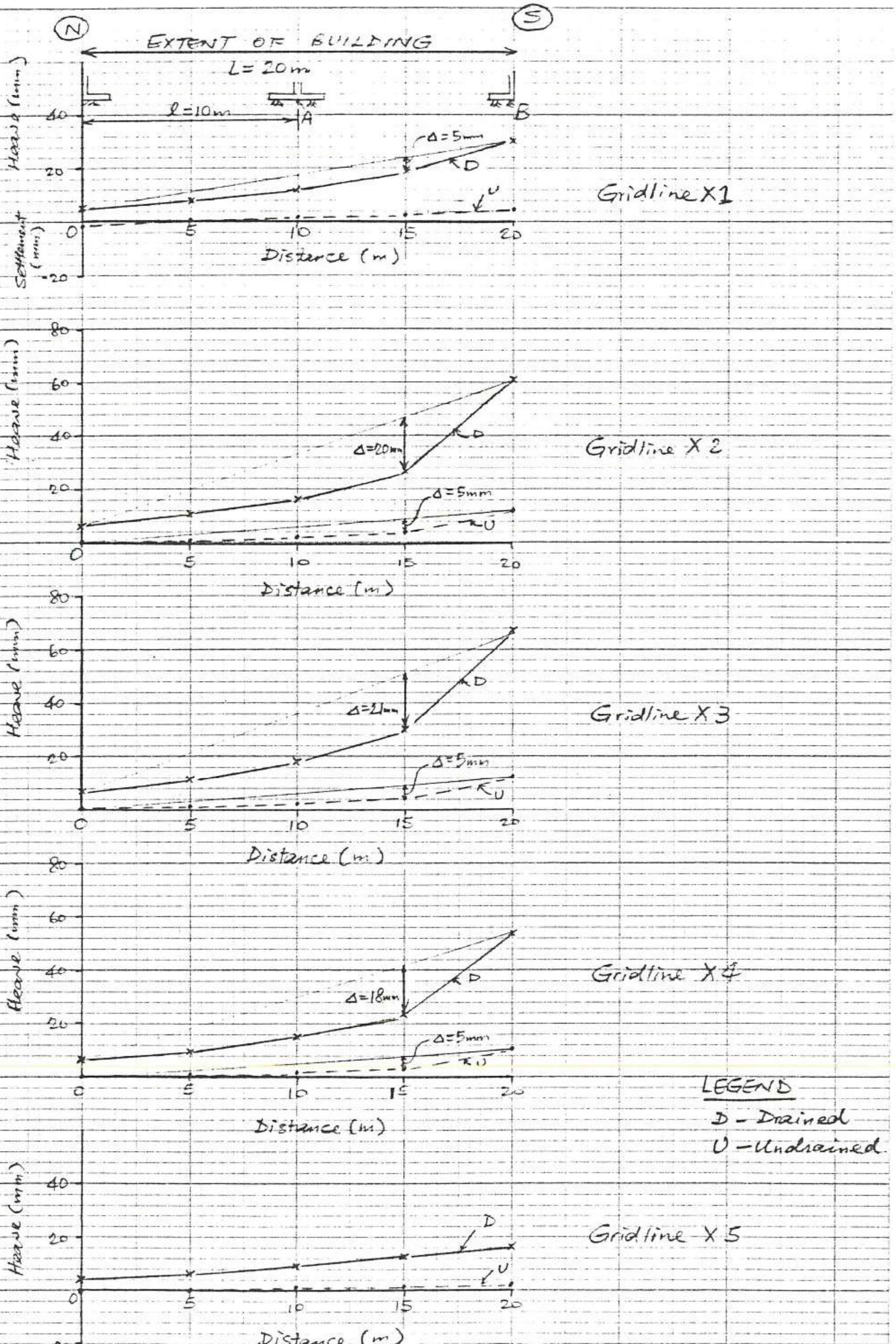
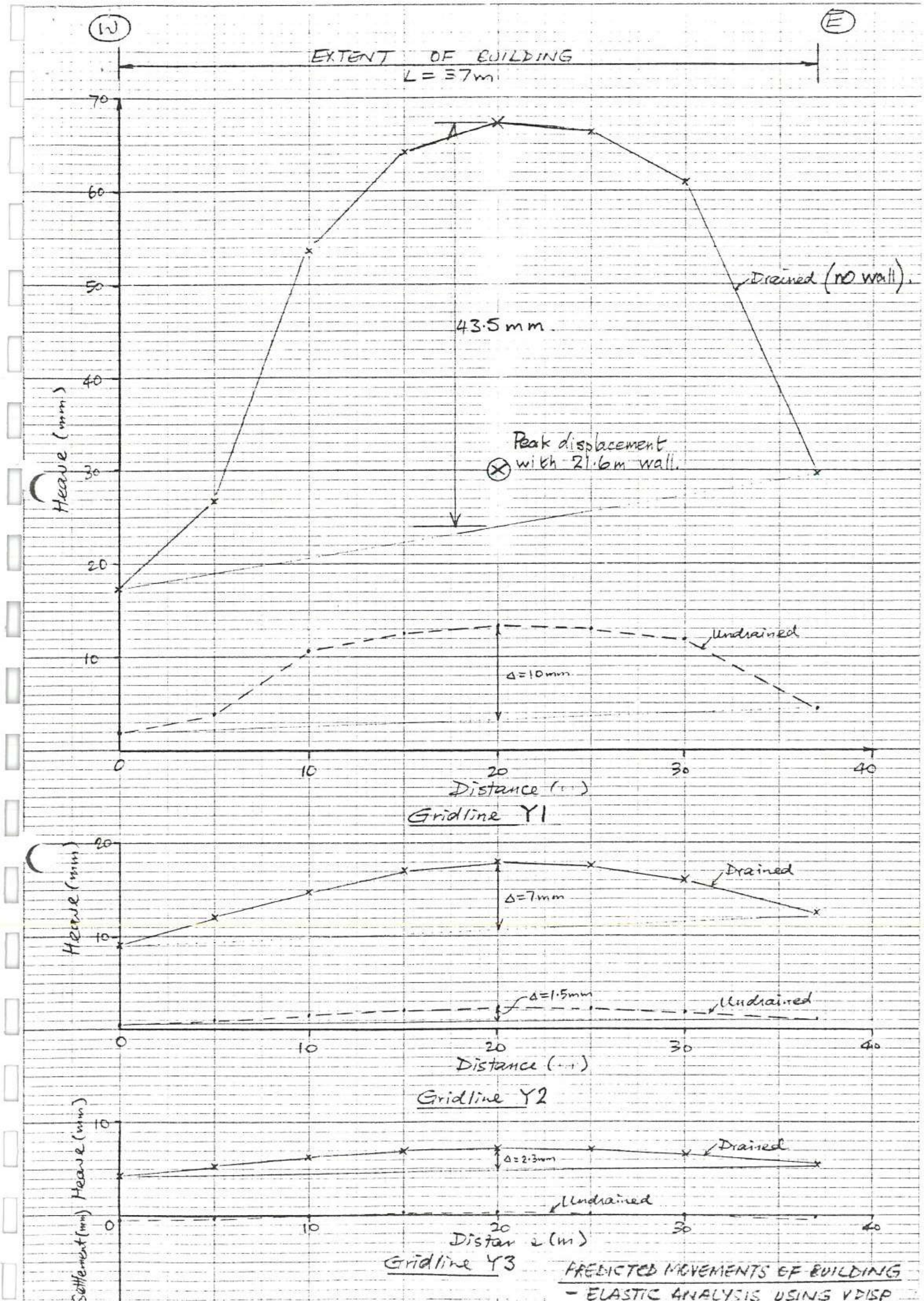
FIGURE 9.



PREDICTED MOVEMENTS OF ADJACENT BUILDING — ELASTIC ANALYSIS USING VDISP



PREDICTED MOVEMENTS OF BUILDING
– ELASTIC ANALYSIS USING VDISP

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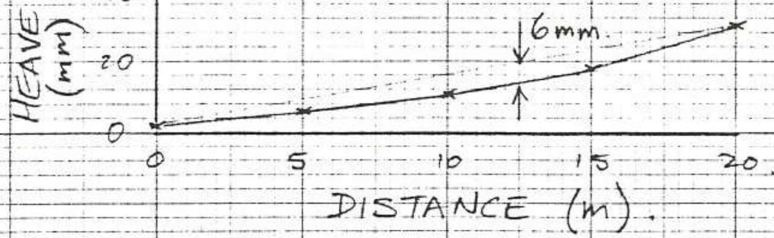
FIGURE 1



Gridlines X2, X3, X4

DRAINED COMPUTATION
21.6m LONG WALL

PREDICTED MOVEMENT OF
ADJACENT BUILDING
– ANALYSIS USING SAFE

FIGURE 12.

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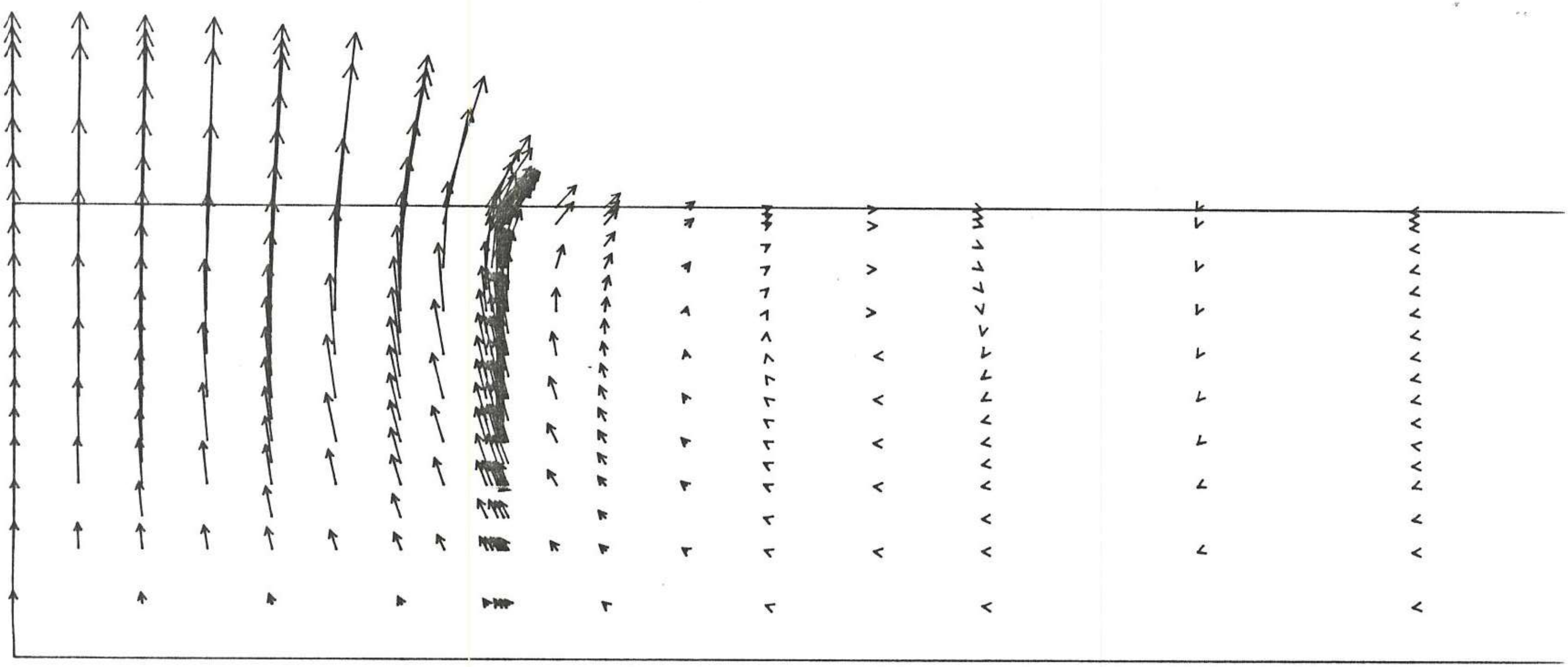
APPENDIX.

1. Computed bending moments and thrusts in the wall (Tension = negative thrust).

2. Vector diagrams of computed displacements for wall 21·6m long.

DRAINED DISPLACEMENTS $\times \frac{1}{5}$



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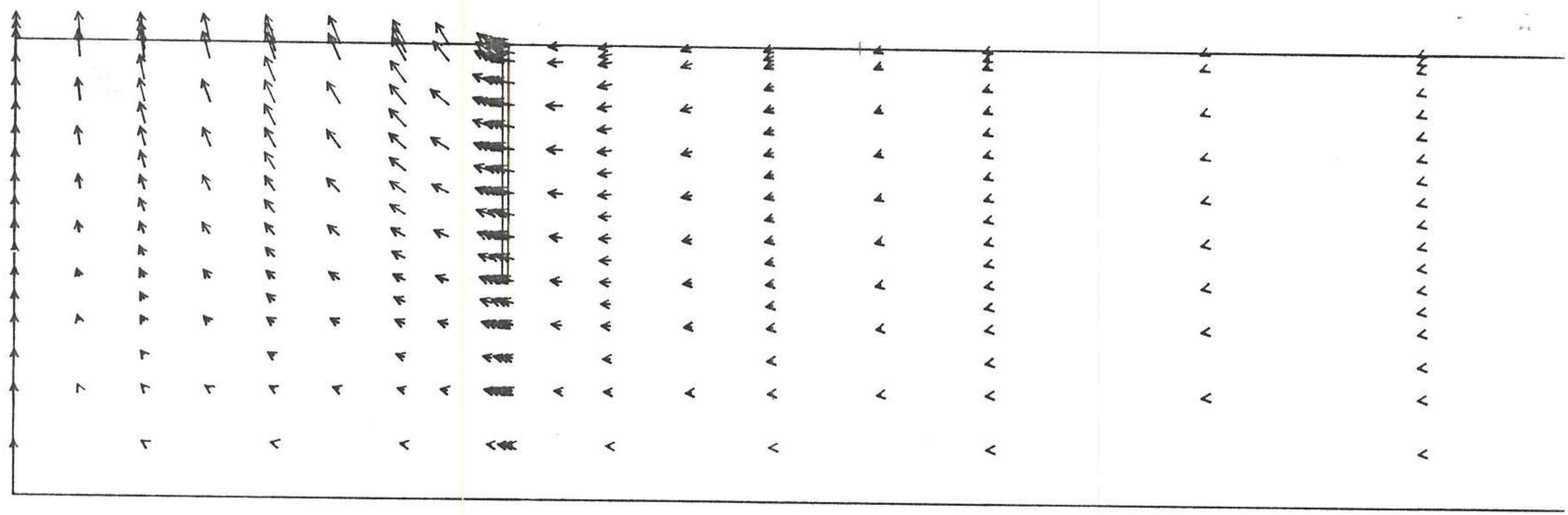
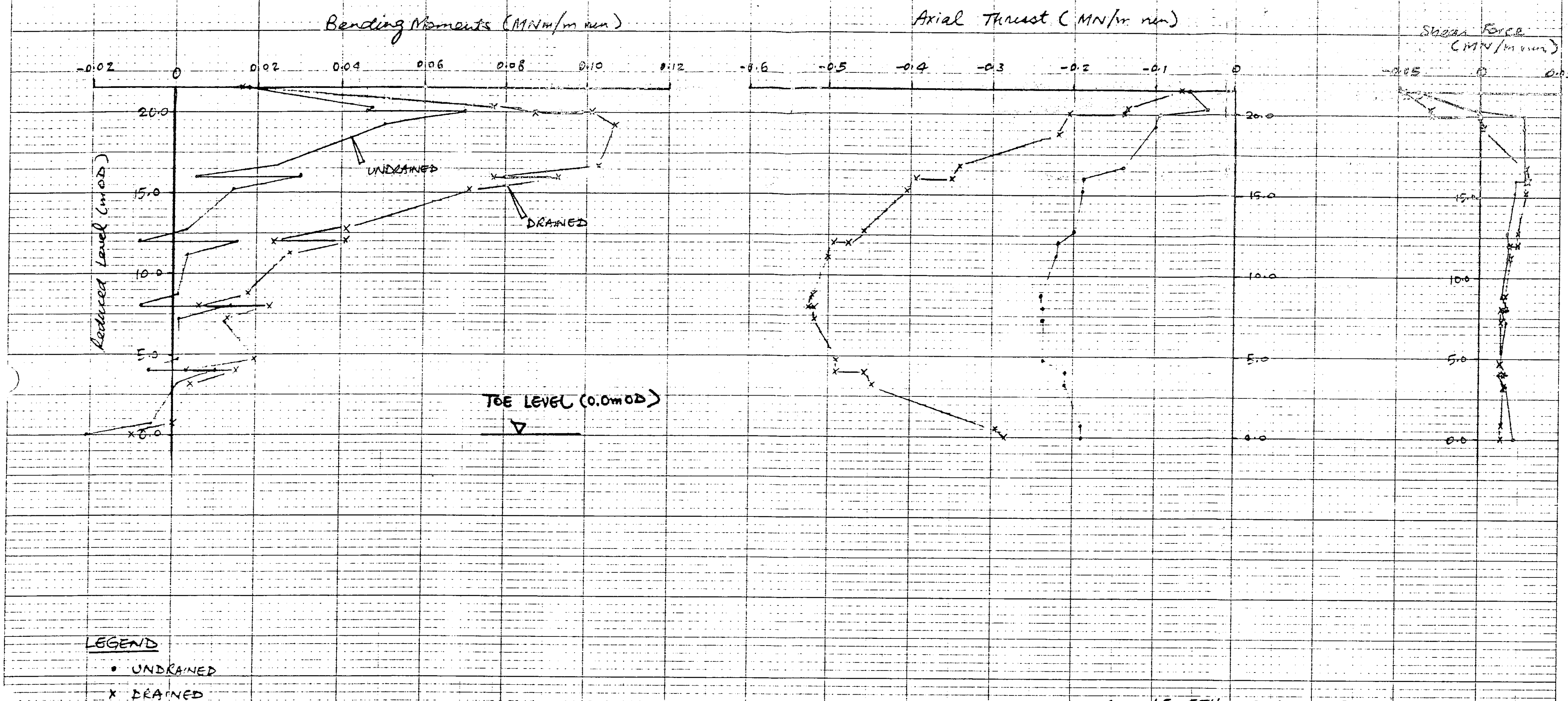
UNDRAINED DISPLACEMENTS × $\frac{1}{5}$



12727 UNDRAINED D



WALL LENGTH = 21.6m   E(PILE) = 10,000MPa.

PREDICTED BENDING MOMENTS, AXIAL THRUST, SHEAR FORCE OF CONTIGUOUS PILE WALL

– PROPOSED PILE WALL

FIGURE 3

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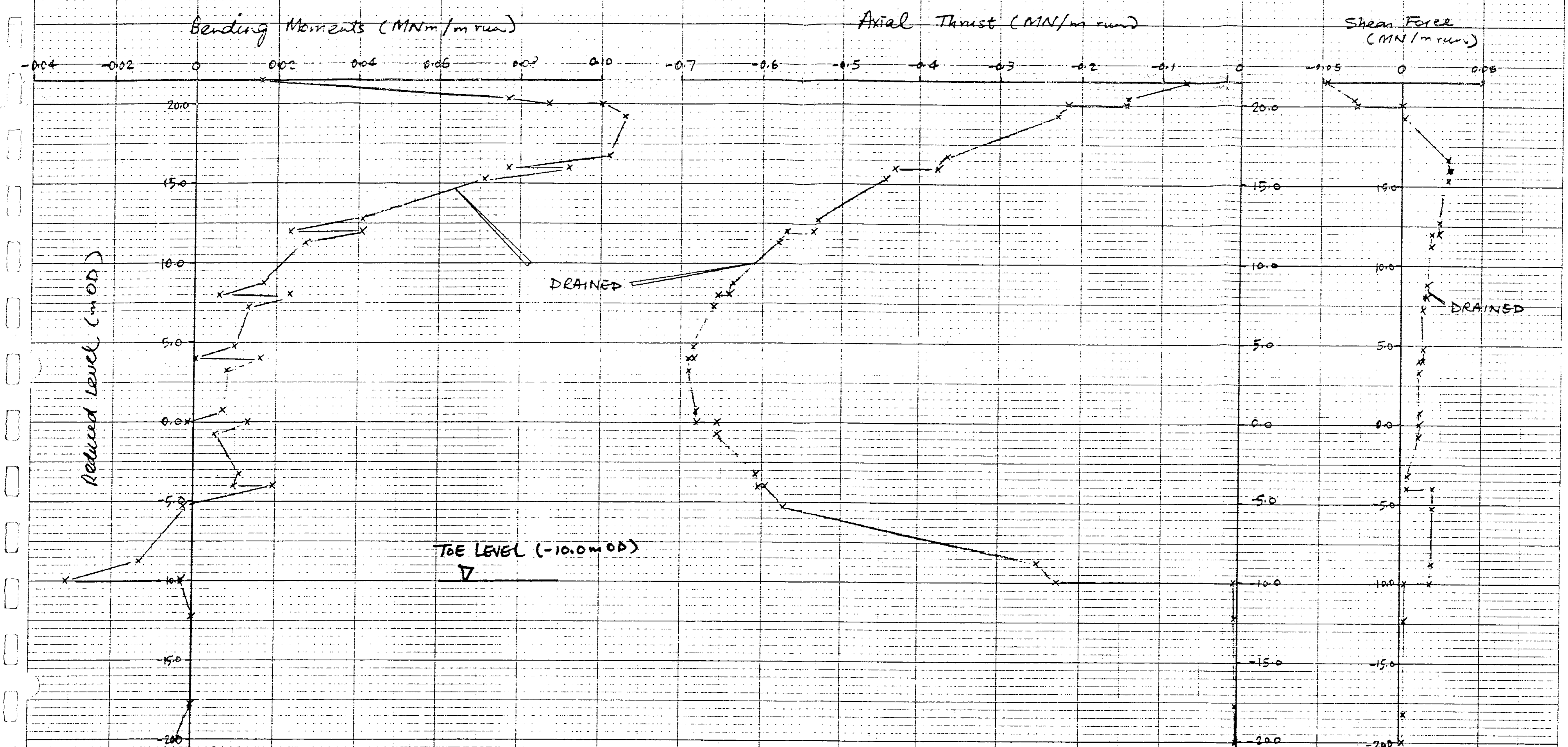

WALL LENGTH = 31.6m E(PILE) = 10,000 MPa

<u>PREDICTED BENDING MOMENTS, AXIAL THRUST, SHEAR FORCE OF CONTIGUOUS PILE WALL</u>

<u>– LONG PILE WALL</u>

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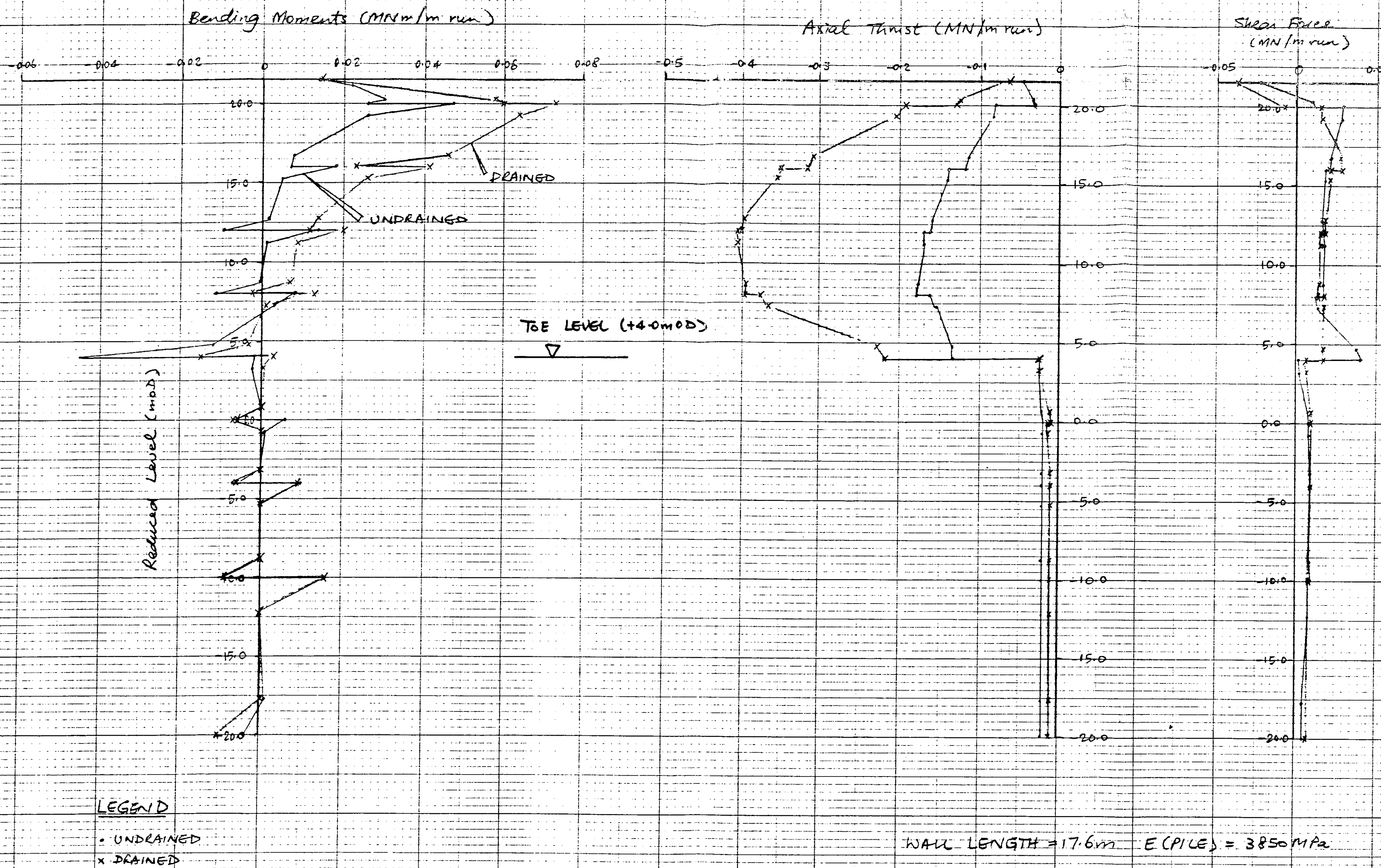
AT/3/7/83



WALL LENGTH = 17.6m E (PILE) = 3850 MPa

PREDICTED BENDING MOMENTS, AXIAL THRUST, SHEAR FORCE OF CONTIGUOUS PILE WALL
— SHORT PILE WALL

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4.0 DESIGN OF BORED PILE WALL

Sheets 4.0 to 4.8

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Bored pile wall — design assumptions.

NOTE Finite element analyses have shown that the principal function of the wall is as a vertical anchor. Bending moments are fairly small.

1. Calculate the maximum tension that could be transmitted to the wall by the soil by shearing on the front face. Provide steel to accept this tension at a stress of 250 N/mm², in order to limit cracking.
2. Check, using finite elements, that the tension actually mobilized is likely to be much less than this.
3. Check, using finite elements, that the wall is long enough to act as an anchor and so restrain ground movements.
4. Check that the steel provided in (1) above can accept the bending moments and shears computed by finite elements (with large reserve).
5. Check that movements to the adjacent building will be tolerable.

Design of Steel Reinforcement to Contiguous Pile Wall

1. Piles

Diameter = 600mm
Length = 21.6m

2. Material Properties

Concrete : f_{cu} = 25 N/mm² Class 2 sulphate resistance. (S.I. Report)

Steel : High Yield with f_y = 410 N/mm²

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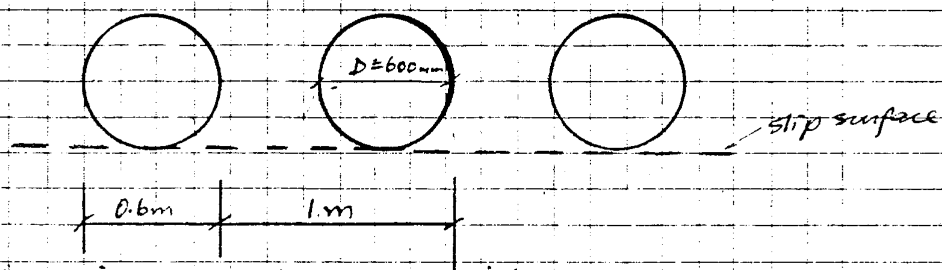
3. Calculation of Max. tensile force on Pile Wall

Consider intact soil strength, when clay swells, shear may develop along extreme surface of wall as shown below:-



Assuming (i) the piles are spaced at 1m centre to centre,
(ii) the rear faces of the piles do not shear

Length of clay surface mobilised during shear = 1m per pile

Now, max. tension per 1m of clay

(3.1) $= \left[1.0 \times \alpha \int_0^y C_{uy}\,dy\right] \times \frac{1}{2}$ ($\frac{1}{2}$ is being taken for one face only)

Take $\alpha = 1$ (for intact clay)

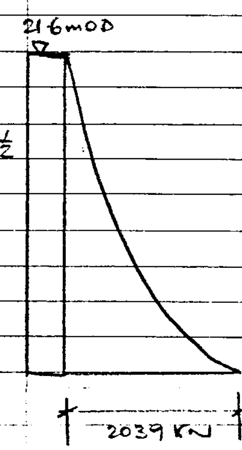
$C_u = 70 + 11y$ KPa

$\therefore$ Max. tension per pile $= 1.0 \times 1 \int_0^{21.6} (70 + 11y)\,dy \times \frac{1}{2}$

$= 1.0\,(70y + 5.5y^2)_0^{21.6} \times \frac{1}{2}$

$= 1.0 \times 4078 \times \frac{1}{2}$

$= \underline{2039}$ KN



4.0. Force Envelope for Design

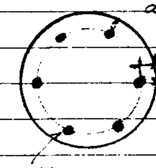
It is considered that the function of the pile wall is still effective with 1 pile per metre ie. 400mm spacing between piles. Figure 1 shows the plot of maximum axial forces experienced by the pile wall from the soil heave. The axial thrust of wall has been computed by 'SAFE' program and plotted on the Figure. It is noted that the 'SAFE' result lies outside the required limit bounded by the line with FOS=2 and $\alpha = 0.45$. In this case, the wall will experience tensile force at the toe level which has not accounted for. The excess axial tensile force is eliminated by shortening the pile wall in the analysis. This leads to slightly increased wall movements both vertically and horizontally. Figure 2 shows the plot of the short pile wall. It is noted that the computed 'SAFE' axial tension is acceptable. Hence, the design envelope is drawn with max tensile force 2039 kN in Figure 2.

5. <u>Max. permissible stress of steel</u>

Since the piles will be in tension, cracks may be formed and the control of crack widths is important.

From CP110, Appendix A,

$$\text{Crack width} = \frac{3 a_{cr} \varepsilon_m}{1 + 2\left[\frac{a_{cr} - C_{min}}{h - x}\right]}$$



For 75mm cover to all steel,

$\therefore C_{min} = 75mm$

Consider point A, $a_{cr} = 75$ mm $= C_{min}$

$$\therefore \frac{a_{cr} - C_{min}}{h - x} = 0$$

$\therefore$ Crack width (directly over a bar)

$= 3 a_{cr} \varepsilon_m$

$\not> 0.004\ C_{min}$

$\therefore 3 \varepsilon_m \not> 0.004$

or $\varepsilon_m \not> 0.00133 = \underline{0.133\%}$.

$\therefore$ Tensile strain of steel reinforcement should not be greater than 0.133%.

Take $E_{st} = 200\ KN/mm^2$

$\therefore f_s$ (steel) $= E_{st} \times \varepsilon_m$

$= 200 \times 10^3 \times 0.133\%$

$= 266\ N/mm^2$ $\therefore$ USE $\underline{f_s = 250\ N/mm^2}$

In CP110 design,

design ULS (ultimate limit state) steel strength $= \frac{410}{1.15} = 357\ N/mm^2$

This gives equivalent ULS load factor $= \frac{357}{250} = \underline{\underline{1.4}}$

6.0 Design of steel reinforcement

Ignore concrete stiffness in tension,

(a) Figure 2, design max. axial tensile force

$= 2039\ KN$

Max. permissible steel stress, $f_s = 250\ N/mm^2$ (Section 5)

∴ Steel strength $= f_s \times A_{st}$

Where A_{st} is area of steel reinforcement

$f_s \times A_{st} = 2039 \times 10^3\ N$

$\therefore A_{st}(\text{req'd}) = \frac{2039 \times 10^3}{250}\ mm^2 = 8156\ mm^2$

Cross-sectional area of pile

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

∴ USE 6Y32 ($A_s = 4825\ mm^2$)
6Y32 ($A_s = 4825\ mm^2$)

total $A_s = 9650\ mm^2$ > A_{st} (req'd)
(3.41%)

(b) Other Sections

USE OF 6Y32 for all other sections
$A_s = 4825\ mm^2$

7. Check on Steel reinforcement requirement using 'SAFE' results

From Finite Element 'SAFE' results,

(a) Undrained case, Max predicted tensile force = 243 kN/m run

Total predicted elongation of wall = (4.4 − 3.7) = 0.7mm

Allow elongation of wall up to 5mm

$\therefore$ Tensile strain $= \frac{5}{20{,}000} = 0.00025$

$= 0.025\%$

(take pile length to be 20m)

$E = 200{,}000\ kN/mm^2$

i.e. $f_{st} \leq 0.00025 \times 200{,}000 = 50\ N/mm^2 < 250\ N/mm^2$

O.K. (section 5)

$\therefore$ For 1m wide wall, ignore concrete stiffness,

Area of steel

$$A_{st} = \frac{\text{Tensile force}}{f_{st}} = \frac{243000}{50} = 4860\ mm^2$$

For 600mm (pile diameter)

$$A_{st} = 4860 \times 0.6 = 2916\ mm^2$$

But area of pile, $A_c = 282743\ mm^2$

$$\therefore 100 \frac{A_{st}}{A_c} = \frac{2916}{282743} = 1.03\%$$

(b) Drained case,

Max. predicted tensile force = 525 kN/m run

Total predicted elongation of wall ('SAFE') = (35.7 − 34.2) = 1.5mm

BUT allow elongation of wall up to 10mm

$\therefore$ Tensile strain $= \frac{10}{20{,}000} = 0.0005$

i.e. $f_{st} \leq 0.0005 \times 200{,}000 = 100\ N/mm^2 < 250\ N/mm^2$

O.K.

∴ Area of steel required

$$A_{st} = \frac{525000}{100} \times 0.6 = 3150$$

$$\therefore 100\frac{A_{st}}{A_c} = \frac{3150}{282743} = 1.1\%$$

∴ % of steel in each pile is 1.1% ($A_{st} = 3150 mm^2$ per pile)

(c) Drained case for short pile

Pile wall length = 17.6 m.

Max. tensile force ≑ 410 kN/m run

Total predicted elongation of wall = (436 − 41.0)

= 2.6 mm

But allow elongation of wall up to 10mm

$$\therefore \text{Tensile strain} = \frac{10}{17600} = 0.00057$$

$$\text{ie } f_{st} \le 0.00057 \times 200{,}000 = 114\ N/mm^2 < 250\ N/mm^2$$

OK.

8. Check Stress due to Bending Moments to wall

Figure 3, $M = 0.107\ MNm/m$ (drained)

$= 107 \times 10^6\ Nmm/m$

CP110, Figure 108

$\therefore$ stress $= \frac{M}{d^3}$ ~~where d = diameter of pile~~

$= \frac{107 \times 10^6}{600^3}$

$= 0.5\ N/mm^2$

CP110, Figure 108 $\therefore \frac{100 A_s}{A_c} = 0.5\%$

$\therefore A_s = 1414\ mm^2 < 4825\ mm^2$ (6Y32)

O.K.

9. Check Shear Stress

Figure 3, shear force diagram,

$V = +0.047\ MN/m$ (drained)

$= +0.047 \times 0.6 \times 10^6\ N/pile$

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

$\therefore$ Stress $= \frac{-0.047 \times 0.6 \times 10^6}{282743} = +0.1\ N/mm^2$

CP110, Table 5,

For $f_{cu} = 25\ N/mm^2$ & $\frac{100 A_s}{A_c} = 1.8\%$

$\therefore$ Ult shear stress $= 0.75\ N/mm^2$

$\therefore$ O.K since small shear stress.

This has a factor of safety against shear of 7.5 //