THE GEOTECHNICAL SOCIETY OF IRELAND

EMBEDDED RETAINING WALL DESIGN IN DUBLIN

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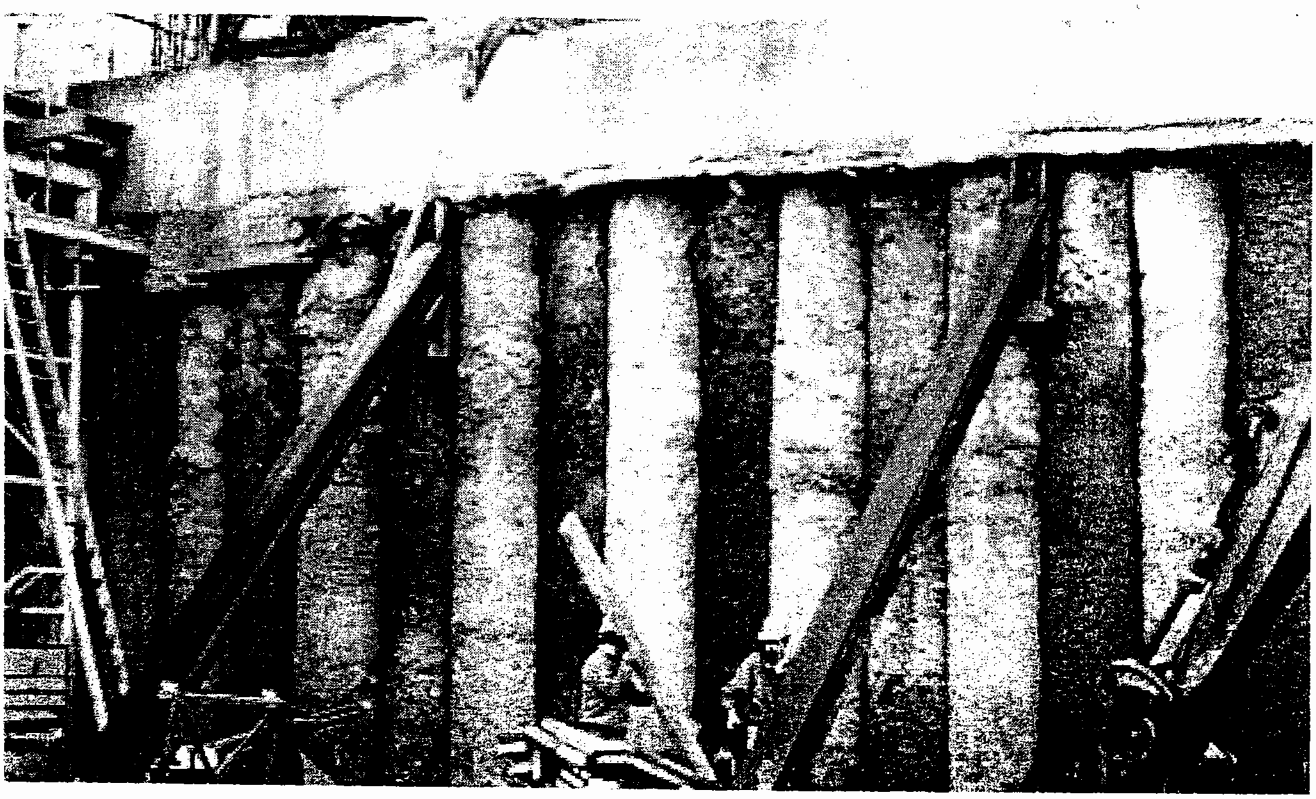

Abstract

In the employment of Ove Arup & Partners Ireland the author has designed a number of both temporary and permanent embedded retaining walls in Dublin in recent years. This paper describes the geology of Dublin and gives typical engineering soil parameters used. The generic design processes undertaken are then described with particular focus on the design elements carried out on computer. A two stage approach to the computerised design is usually adopted. Firstly a program which employs classical earth pressure theory is used to calculate the depth of embedment required to provide limit state equilibrium. This design, duly factored, is then input into a program which uses an iterative procedure to distribute out of balance forces based on the relative stiffness of each element. Typical output from each of the programs is presented and the advantages and limitations of the procedure are discussed. Finally, a case history is given for the basement retaining wall recently constructed for the new Jervis Shopping Centre in the centre of Dublin.

1.0 INTRODUCTION

In recent years there has been significant economic growth in Ireland. The resultant upturn in the construction industry has seen a number of large developments (shopping centres, hotels etc.) incorporating single and double level basements around Dublin's city centre. In order to maximise space within these basements the building line is often very close to adjoining properties. There has therefore been a requirement for a number of engineered temporary and permanent retaining walls to retain adjoining properties and facilitate basement construction.

2.0 GEOLOGY OF DUBLIN

2.1 General

The geology of Dublin city is that of carboniferous limestone overlain by glacial deposits of boulder clays and gravels. Human activities, generally the reclamation of esturine and flood plain areas, have generally lead to a further covering of fill material.

2.2 Solid Geology

The solid geology is generally known as the "Dublin Formation" (locally known as "Calp") and is composed of interbedded layers of calcisiltite, calcarenite and occasional layers of calcareous shales. The rock is generally thin to medium bedded and has closely spaced discontinuities. The limestone unit is usually slightly weathered and strong. The shale interbeds, however, can range from slightly to completely weathered and from moderately strong to weak.

2.3 Glacial Drift

The glacial drift material is generally found to be Boulder Clay with inter-stratified layers of sands and gravels.

The boulder clay is a lodgement till which has been subjected to high vertical stresses during glaciation leading to generally highly overconsolidated deposits (Hanrahan 1977). The boulder clay is subdivided into the brown and black boulder clays. The brown is the weathering product of the underlying black, with a higher void ratio, higher moisture content and lower strength and stiffness. The thickness of the brown boulder clay is limited typically to 3 m.

The brown boulder clay is generally found to be firm to stiff while the black boulder clay is generally very stiff. Farrell and Wall (1990) have reported respective average moduli of 100MN/m^2 and 82MN/m^2 for the undrained and drained stiffness of Dublin black boulder clay, assessed from plate bearing tests.

2.4 Made Ground

Made ground is the uppermost layer of the ground sequence within Dublin city. The materials generally encountered, in varying proportions, are clays, gravels, builders rubble, domestic refuse and industrial waste.

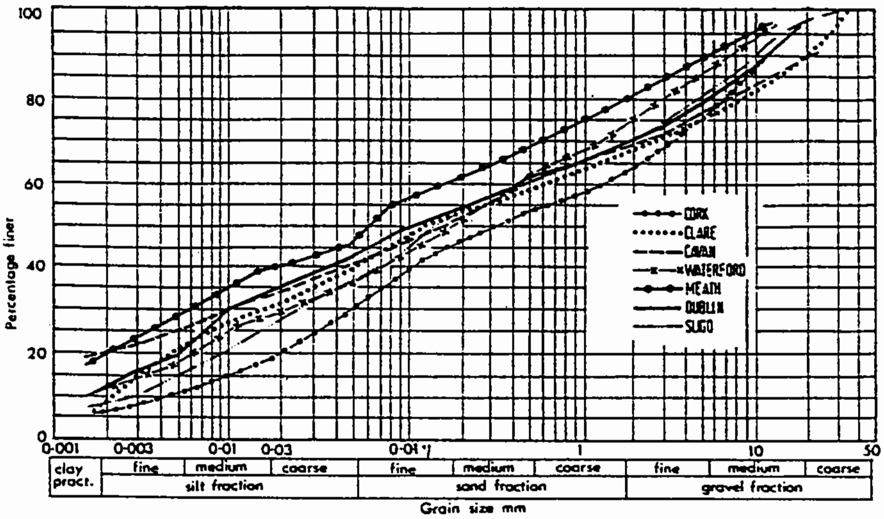

Figure 1: Selected grading Curves of Boulder Clay (Hanrahan 1977)

2.5 Soil Parameters

Typical engineering soil parameters for each of the above strata are given in Table 1 (the rock has been modelled as a very dense gravel).

Table 1: Typical Soil Parameters

Stratum	γ (kN/m^3)	$\varnothing'$	c' (kN/m^2)	C_U (kN/m^2)
Fill	17	30	0	0
Brown Boulder Clay	20	35	0	100
Black Boulder Clay	20	37	0	250
Gravel	20	38	0	0
Weathered Rock	22	42	0	0
Intact Rock	22	45	0	0

The boulder clay in general consists of sand, gravel and occasional cobbles and boulders embedded in a very stiff matrix of low plasticity silt and clay. Boulder clays generally exhibit an almost straight line grading as shown in Figure 1 (extracted from Hanrahan 1977).

Glacial gravels and sands occur commonly as water bearing lenses and pockets of variable lateral and vertical extent (several centimetres to several meters) within the boulder clay. The grading of these materials can vary greatly from place to place but they are generally found to be dense to very dense.

3.0 RETAINING WALL DESIGN

3.1 General

A two stage approach to retaining wall design is usually adopted with both stages being carried out using separate computer programs. The two stages are:

(i) determine the depth of embedment required to provide "factored" overall wall stability.

(ii) calculate wall displacements, bending moments and shear forces and prop forces in the serviceability condition.

The two programs used are STAWAL (STAbility of retaining WALls) and FREW (Flexible REtaining Walls) both of which are part of the OASYS GEO suite of programs (OASYS is the software group within OVE ARUP & PARTNERS).

3.2 STAWAL

The computer program STAWAL performs calculations using conventional Rankine active and passive earth pressure theory. It uses an ultimate limit state (ULS) approach to calculate the depth of embedment required. Therefore the safety factor can either be applied by reducing the strength of each soil layer (Eurocode 7 - Part 1, Table 2.1) or by increasing the ULS depth of embedment (CIRIA Report 104, Table 5). Usually the former approach is adopted.

All data required by STAWAL is requested by the program, through a series of menus, as discussed below.

The user is asked to specify the collapse mechanism as being either "Fixed Earth" or "Free Earth" for cantilever or singly propped walls respectively (CIRIA Report 104). The post excavation stratigraphy is input by means of specifying the level of the top of each strata on both the active and passive side.

The soil properties (bulk unit weight, earth pressure coefficients - Ka, Kp, Kac, Kpc - and the cohesion at the top and bottom of the soil layer) are then entered. The user must separately adjust Ka and Kp for wall friction as appropriate. A hydrostatic or piezometric water profile can be specified and surcharges (uniformly distributed or strip) and horizontal struts (with forces) can be applied to either side of the wall.

Effective active and passive pressures are calculated by the program down the front and back of the wall using the following equations:

$$P_a' = K_a \sigma_v' - K_{ac} c$$
$$P_p' = K_p \sigma_v' + K_{pc} c$$

where

K_a and K_p = horizontal coefficients of active and passive pressure.

K_{ac} and K_{pc} = cohesive coefficients of active and passive pressure.
c = effective cohesion or undrained strength as appropriate.
σ_v' = vertical effective stress.

The soil on the active side is not allowed to go into tension and a minimum value of zero is set for P_a'.

The program then determines the penetration required to prevent instability. Bending moments, shear forces (and the strut force for free earth conditions) are also calculated at the limit equilibrium condition. Results can be output in either tabular or graphical form.

Typical graphical output from STAWAL is presented in Figure 2.

For further details of STAWAL the reader is referred to the program's user manual.

The method selected by the author is called the "Safe Flexibility Method" in which the soil is modelled as an elastic-plastic solid (SAFE is the OASYS geotechnical finite element program). The soil stiffness matrices are developed from prestored finite element stiffness matrices which are scaled to the parameters of the given problem. The program analyses the behaviour for each stage of the construction sequence, e.g. excavation, dewatering, insertion of strut.

The user is asked to define the geometry of the problem but does not have to develop a finite element mesh. The stiffness of the structural elements along with any surcharge or strut are also entered. Soil parameters (horizontal stiffness, bulk unit weight, Ko, Ka, Kp, Kr, and c) and groundwater conditions (hydrostatic or piezometric) are further entered.

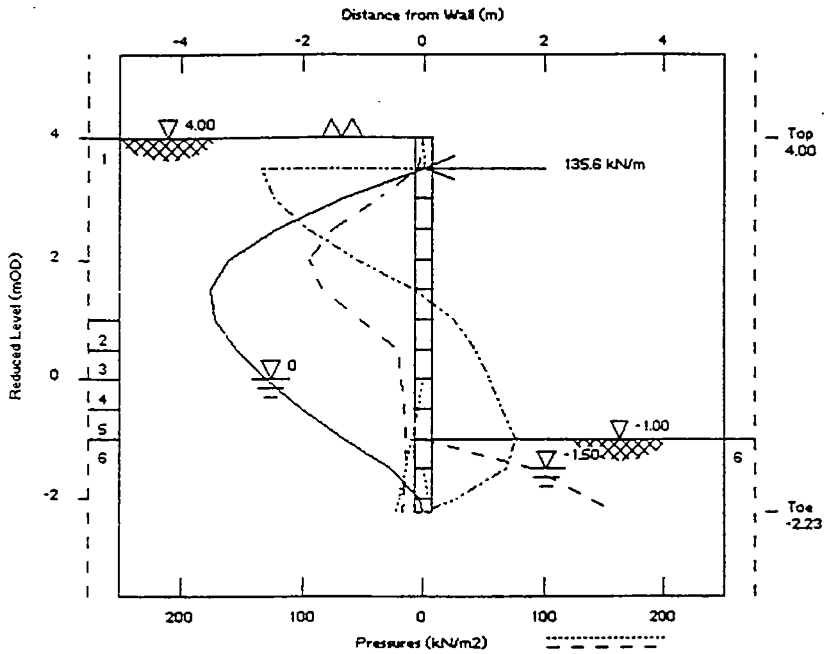

Figure 2: Typical STAWALL Output

3.3 FREW

The computer program FREW analyses soil structure interaction problems of flexible retaining walls. The wall is represented as a series of nodal points and three stiffness matrices relating nodal forces to displacements are developed. One represents the wall in bending and the others represent the soil on either side of the wall.

The program has three separate methods to model the soil behaviour of which the user must choose one.

The program calculates the initial force imbalance, due to a construction operation, at each node and determines wall displacements and soil stresses using the stiffness matrices. An iterative procedure is used to redistribute all out of balance forces to within an acceptable tolerance. Beginning at the "at-rest" condition, soil stresses are limited to within the active and passive limits.

The program gives results for earth pressures, displacements, shear forces and bending moments of the wall and strut forces. Results can be output in either tabular or graphical form.

Typical graphical output from FREW is presented in Figure 3.

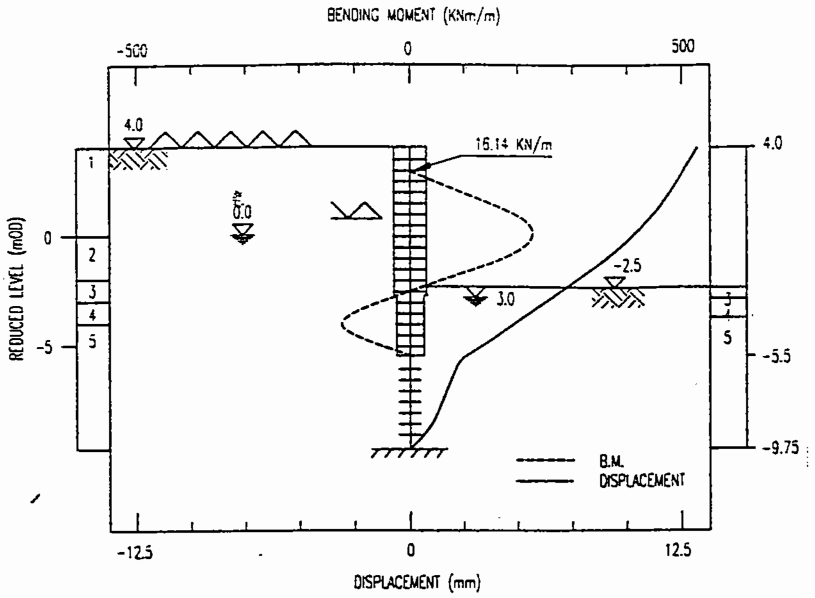

Figure 3: Typical FREW Output

Full details of the assumptions and analysis methods are given by Pappin *et al* (1986).

4.0 COMMENT

4.1 General

The above two stage approach has been used for the design of a number of both temporary and permanent retaining walls in Dublin in recent years. Many of these retaining walls have been for single and double level basements to multi-storey car parks, hotels, shopping centres and the like.

4.2 Comment on Geology

The geology of Dublin, as discussed in Section 2.0, provides favourable conditions for such basement constructions.

The boulder clays are remarkably consistent and their undrained strength can be relied on in the short term. In fact, many steep slopes and cuttings can be seen in the boulder clays around Dublin which have been standing for many years.

However, drained triaxial testing of both undisturbed and remoulded samples of the brown and black boulder clays (Farrell and Wall 1990; Ove Arup & Partners Ireland 1996) have shown that neither of these materials has an effective cohesion intercept. This has the effect that long term analyses, with c'=0, predict displacements that may not be realised for many years. Long term maximum displacements of propped walls tend to be in the range 15 to 20mm which generally does not cause much concern. However, as the boulder clays are modelled essentially as a medium dense gravel ($\varnothing'$=35 to 37°, c'=0) and coupled with the fact that these clays are overconsolidated (with initial high lateral stresses) the displacements predicted for cantilever walls tend to be quite high, 25 to 50mm. However few walls are required to cantilever in the long term.

Two of the main considerations when deciding on the type and depth of a basement retaining wall in Dublin ARE the presence or absence of gravels and the depth to bedrock.

Gravels in Dublin are nearly always found to be waterbearing and their presence will dictate that a watertight wall be constructed. Depth to bedrock can be anywhere between 5m and 20m+ below ground level. Where rockhead is high basements are rarely constructed too deep due to the expense of penetrating the rock with a retaining structure and of general rock excavation.

Therefore the three main retaining wall types which are generally constructed in Dublin are as follows:

(i) Sheet piled walls in gravels or boulder clays, for temporary retention, where rock levels are low enough to allow sufficient wall depth below excavation level.

(ii) Contiguous piled walls, 300 to 750mm diameter, where gravels are absent.

(iii) Hard - soft secant piles walls, 450 to 900mm diameter, where gravels are present above relatively shallow rock and/or there is a requirement for a stiff wall to retain adjoining properties.

4.3 Comment on Design Method

The two stage design method discussed provides a coherent approach for the design of flexible retaining walls.

The walls are first designed to be ultimately stable, with an appropriate factor of safety, within the theoretical horizontal stress limits of the particular soils. Further, wall deflections, shear forces and bending moments are calculated taking into account the at-rest soil pressures and the soil and wall stiffnesses (all of which have a significant effect on wall movements).

The method has the advantage that, as the finite element mesh does not have to be assembled by the user a design can be carried out reasonably quickly by and experienced designer. Further, FREW has been proven to give reliable results in a variety of ground conditions (Wallace *et al* 1992).

The method has the disadvantages that only simple plane strain conditions can be modelled and no assessment is made of ground movements resulting from wall displacements.

5.0 CASE HISTORY - JERVIS SHOPPING CENTRE

5.1 General

The Jervis St. Shopping Centre was constructed between July 1995 and November 1996 by Pierse Contracting Ltd. The development included a basement area with a maximum depth of approximately 7.5m. Therefore a basement retaining structure was required and the contract for this was let on a design and construct basis by the project's consultant engineers Design & Management (D & M). Murphy International limited (MIL) were awarded this contract and they appointed Ove Arup & Partners Ireland as their design engineers.

5.2 Site Location

The site is located in the centre of Dublin city and occupies an area of approximately 0.9ha. It is bounded to the north by Mary St., to the west by Jervis St., to the south by Upper Abbey St. and to the east by the existing Marks and Spencer Store. The site, which is approximately 250m north of the River Liffey, was formerly occupied by the Jervis St. Hospital.

5.3 Geology

The ground conditions at the site, as revealed by conventional shell & auger drilling and rotary coring of the rock, are presented in Table 2.

Table 2: Ground Conditions

Stratum	Thickness (m)
-	Range / Average
Made Ground	2.6 - 2.7 / 3.2
Alluvium	0.0 - 0.6 / 0.1
Gravels	1.7 - 3.0 / 2.2
Black Boulder Clay	0.0 - 1.6 / 0.8
Weathered Rock	0.1 - 1.9 / 0.5
Intact Rock	7.5m proven

Groundwater was generally encountered at 3.0m to 3.5m below ground level in the gravel layer.

The soil parameters used by Ove Arup & Partners Ireland in the retaining wall design are given in Table 3 and were based on the results of the site investigation, published data and experience of the Dublin soils.

Where:
E_h is the horizontal soil stiffness
Ko is the coefficient of earth pressure at rest
f_b is the ultimate end bearing value
f_s is the ultimate skin friction value

Notes: (i) The low value of f_b = 6000kN/m^2 was a non negligence insurance stipulation. Values of 12000kN/m^2 and greater are regularly used by the author and (ii) the gravels at this site were uncharacteristically loose.

5.4 Proposed Development

The basement dig depth around the perimeter of the site varied between approximately 5.5m along Upper Abbey Street to approximately 7.5m along the Mary St. side. The maximum dig depth for foundations was about 9.7m below ground level which was about 6.5m below the water table.

Several sensitive structures (including a Marks & Spencer store and two facades listed for preservation) were located immediately adjacent to the perimeter of the site and this necessitated a stiff wall to limit wall deflections and associated ground loss. These structures also imposed a significant surcharge loading on the wall of up to 700kN per metre run.

The wall also had to accommodate significant structural loads in the form of isolated columns and load bearing walls.

5.5 Retaining Wall Solution

The two retaining wall solutions which were considered were:

(i) A sheet piled wall with a subsequent reinforced concrete wall constructed against its inner face.

(ii) A permanent secant piled wall.

A trial was carried out to investigate the feasibility of installing the sheet piled wall. The trial involved the construction of a sheet pile cofferdam of approximate dimensions 5m x 5m x 5.5m deep using Larssen 25W sheet piles. The sheet piles were driven to "refusal" on the top of the weathered rock. It was found, however, that an adequate water seal could not be achieved between the bottom of the sheet piles and the top of the weathered rock. This was due to the very uneven nature of the top of the rock. Although the flow of water could have been controlled by pumping there would possibly have been an associated ground loss outside the wall due to the washing of fines out of the gravel. It was therefore decided to adopt the secant pile wall solution.

5.6 Pile Size and Configuration

The chosen pile size and configuration is shown in Figure 4. The wall comprises of interlocking male and female piles. The piles are 900mm diameter in the overburden, reducing to 810mm diameter in the rock. The overlap between the piles is 130mm resulting in the centre to centre distance, male pile to male pile, of 1.54m. A 900mm diameter pile was chosen over, say, a 600mm diameter, due to the need to provide a very rigid wall to limit displacements. Also as the construction program was very tight the fact that a 900mm diameter secant wall could be constructed much quicker than a 600mm diameter wall was also of major benefit.

The male piles were constructed using C35 concrete and reinforced with 8 No. T32 reinforcing bars. The female piles were constructed of C7 concrete and were unreinforced.

The female piles were designed to have a toe level of at least 0.5m

Table 3: Soil Parameters at the Jervis Shopping Centre Site

Material	E_h (kN/m2)	γ (kN/m^3)	Ko	Ø'	c' (kN/m^2)	SPT "N"	f_b (kN/m2)	f_s (kN/m2)
Made Ground	30,000	18	0.5	30	0	13	-	-
Alluvium	5,000	17	1.0	(28*)	20	-	-	-
Gravels	35,000	20	1.0	33	0	21	-	-
Black Boulder Clay	100,000	20	1.3	(36*)	270	45	-	-
Weathered Rock	100,000	22	1.0	42	0	-	-	-
Intact Rock	300,000	24	1.0	45	0	-	6000	2000

* Effective stress soil parameters used in STAWAL

below the proposed excavation level to ensure an adequate water cut-off.

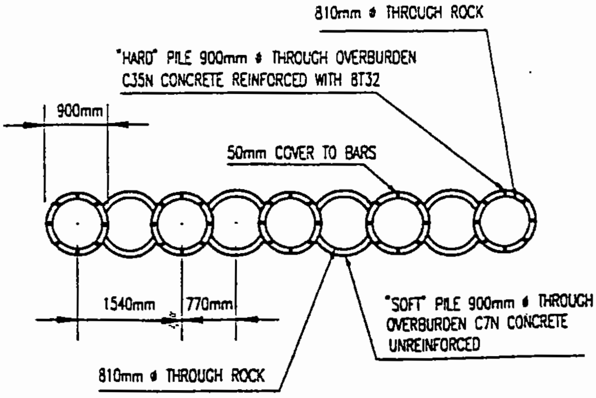

Figure 4: Pile Confiruration

The male piles need a greater toe-in for structural reasons, which will be discussed below.

5.7 Retaining Wall Design

As mentioned above the piles had to accommodate significant vertical loads and so in this instance the first design calculation was to determine the depth of embedment required into intact rock to safely carry these loads. A value of 0.88m penetration was determined where the maximum pile load of 3,300kN existed.. It was decided that, as this embedment is directly below formation level and that there may be a danger of over-excavation, this depth should be increased to 1.5m in the heavily loaded areas. In areas where the column loads were significantly less and the wall was not surcharged by adjacent buildings the depth of embedment was fixed at 0.75m into intact rock.

An analysis was subsequently carried out to investigate the depths of embedment required to provide overall stability with full excavation and surcharge loading included. This analysis was carried out with the computer programme STAWAL using adequately factored effective stress soil parameters, as discussed in Section 3.2. The analysis concluded that the minimum depths set above were more than sufficient to provide overall stability.

As discussed the computer program FREW was used to model the wall system in service. The FREW model is set up so as to represent the various construction stages carried out on site. The construction stages used in this case were:

Stage O: Set up the problem.
Stage 1: Apply the surcharge loading and partially excavate leaving a berm.
Stage 2: Install temporary props and excavate to formation.
Stage 3: Install permanent props (floor slabs) and remove the temporary props.

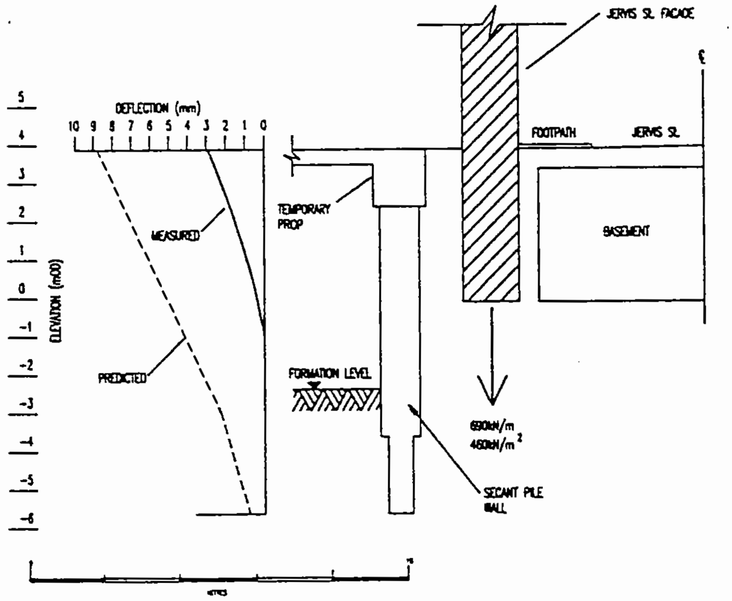

Figure 5: Inclinometer Readings

A total of eight design cases were analysed to model the various surcharge conditions and excavation depths around the perimeter of the site.

The critical data which is output by FREW is wall bending moments, wall displacements and prop forces.

Calculated wall bending moments were within the capacity of the wall (maximum ultimate value of 850kNm per male pile) while universal column sections (maximum 305 x 305 UC x 97 kg/m) at 6m centre to centre were sufficient as the temporary propping element (maximum prop force of 185kN/m service)

The predicted maximum wall displacements at the top of the piles varied from 7.0mm, where the wall was surcharged only by street loading and where the maximum excavation depth was approximately 6.25m, to 17.0mm where the wall was loaded by the adjacent Marks & Spencer building's foundation and the depth of excavation was approximately 7.25m.

An inclinometer was installed in a male pile adjacent to the Jervis St. Hospital facade in order to measure lateral wall movement. The actual versus predicted movements (from FREW) are shown in Figure 5. It can be seen that the maximum recorded movement was 3mm compared to about 9mm predicted. However it can be seen that FREW accurately predicted the shape of the wall deformation.
One of the main reasons for this fairly large discrepancy is the fact that both the made ground and the gravel layer were modelled in FREW

as cohesionless materials. The fill material, although extremely variable across the site, contained a reasonable amount of clay. The gravel was completely dewatered by pumping and negative pore pressures may have developed in this layer. In any event both the made ground and the gravel stood almost vertical in excavations around the site. Presumably the materials behind the wall were similar and therefore would not have loaded the wall to anything near the design loads of fully drained materials. Although boulder clays are modelled as undrained in the short term it is not thought prudent to ever treat real gravels this way. However the drained approach to the fill material may need some reconsideration in light of the above.

5.8 Retaining Wall Construction

The piles were formed using a bored cast *in-situ* method. A total of approximately 300 male and 300 female piles were constructed to complete the perimeter wall. The pile lengths varied between 6.5m and 11.9m.

Excavation was initially carried out along the line of the secant pile wall in order to remove obstructions. As the on-plan position of the secant piles was important, each pile was located using a guide frame. A 900mm diameter open ended steel casing was then rotated into the ground and the overburden materials removed using a Soilmec R10 boring rig fitted with a 900mm diameter auger. The procedure of advancing the casing and augering the soils continued until the casing reached the surface of the rock and all of the overburden was removed from inside the casing.

The rock was removed to the required depth in an unlined hole. A rotary percussive Ingersoll-Rand cluster drill comprising five 250mm diameter drill bits mounted on a Soilmec RT3 rotating shaft and carried by a 40T Hitachi crane was used for this purpose. The overall diameter of the rock socket formed was 810mm.

The reinforcement cages were placed in the male piles and the piles were concreted using a tremie pipe. The temporary casing was then removed. Work was carried out only on every second pile at any one time to minimise disturbance to the fresh concrete.

6.0 SUMMARY

The geology of Dublin, consisting essentially of glacial boulder clays over limestone bedrock, present favourable conditions for the construction of single and double level basements. Where waterbearing gravels are present the task is made more arduous as the retaining wall must be watertight.

Retaining walls have been designed in two separate stages using separate computer programs. The walls are first designed to ensure overall stability with an adequate factor of safety. This is done by a program (STAWAL) which uses conventional earth pressure theory. Wall bending moments and shear forces, strut forces and soil stresses are then calculated in the service condition by a program (FREW) which takes into account the at-rest soil pressures and the relative stiffness of each soil strata and structural element.

The design methods above as applied to the Jervis Shopping Centre have been discussed. Predicted wall displacements were significantly greater than those observed on site. It is thought that the main reason for this is the apparent undrained behaviour of the made ground and the gravel layer.

REFERENCES

1. Hanrahan, E.T. (1977) "Irish Glacial Till: Origin and Characteristics" An Foras Forbartha RC164

2. Farrell, E. and Wall, D. (1990) "The Soils of Dublin", IEI Transactions 1990 - 1991.

3. European Committee For Standardization (1994) "Eurocode 7: Geotechnical Design - Part 1: General Rules"

4. CIRIA (1984) "Design of Retaining Walls Embedded in Stiff Clays, Report 104"

5. Oasys (1991) "STAWAL User Manual"

6. Oasys (1991) "FREW User Manual"

7. Pappin, J.W., Simpson, B., Felton, P.J. and Raison, C. (1986) "Numerical Analysis of Flexible Retaining Walls", Sym. on Computer Applications in Geotech. Eng., The Midland Geotech. Soc., April.

8. Ove Arup & Partners Ireland - Unpublished Data

9. Wallace, J.C., Ho, C.E. and Long, M.M. (1992) "Retaining Wall Behaviour for a deep Basement in Singapore Marine Clay", Proc. International Conf. Retaining Structures, Cambridge, July, pp195 - 204, Thomas Telford.

10. Dougan, I., Long, M.M.and Byrne, J.J., (1996) "The Geotechnical Aspects Of The Deep Basement For The Jervis Street Shopping Centre", IEI Transactions 1996 - 1997.