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United Kingdom national report on braced excavations in soft ground

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SYNOPSIS: Braced excavations are extensively used in the United Kingdom for basements, underground roads and railways and many constructions of the water industry. Most of the highly populated areas of the country are underlain at shallow depth by either stiff clay or rock, though soft, or loose, soils are sometimes of engineering significance. Design is based on a variety of codes and manuals which are noted in the paper; more recent documents have adopted a limit state format but have found complete compatibility with structural codes unattainable. Much has been learned from extensive schemes of monitoring, especially with regard to assessment of ground movements.

1 GEOLOGY OF THE UNITED KINGDOM

Most major excavations in the UK (ie England, Wales, Scotland and Northern Ireland) extend into either rock or stiff, overconsolidated clays. Many of them encounter relatively little soft or loose soil (ie c_u < 40kPa or SPT blow-count N < 10), though these occur commonly as shallow surface layers and occasionally as thicker deposits. The heavily developed areas of south-eastern England are underlain by either Chalk or Tertiary deposits of stiff clays or dense sands, notably the London Clay, in which many braced excavations have been constructed.

Deposits of soft clay are generally of alluvial or estuarine origin and of thickness less than about 3m except near the mouths of major rivers. Near rivers in the cities, stiff clay or rock is sometimes overlain by sands and gravel, usually at least medium dense and up to about 10m thick.

2 USE OF BRACED EXCAVATIONS

Virtually all forms of walling have been used for braced excavations in the United Kingdom. Steel sheet piling is common in temporary works but is rare in permanent works except in water-front applications, both coastal and inland. Concrete walls are much more common for permanent works, including both slurry-trench diaphragm walls and piled walls. King post and poling board systems are uncommon in the UK.

Slurry-trench walls have been widely used. Their use in London Clay has been particularly frequent, and in recent years the Hydrofraise system has extended their use for sites underlain by dense sands and rock. Though generally treated as vertical bending elements, wall panels have also been used to act in hoop compression around a circular excavations and as a stiff horizontal arch between remote struts (Stevens et al 1977).

In recent years piled walls have often proved more economic than slurry-trench walls, mainly because of the cost of site operations for bentonite plant. These may consist of rows of contiguous piles or various forms of intersecting piles. Intersecting piles may be 'hard-hard' 'secant' piles, in which all the piles are constructed of strong concrete and may be reinforced, or 'hard-soft' 'interlocking' piles in which the primary, female piles are constructed of soft, bentonitic concrete and reinforcement is only used in the secondary, male piles. This latter technique favours the use of continuous flight auger rigs.

Open excavation, temporary propping and top-down construction are all common. Top-down construction has been favoured in urban areas in order to minimise ground movements, though recent experience in London has suggested that anchored excavations can be equally successful (St John et al 1992). There is reluctance to allow ground anchors to extend below neighbouring property.

3 CODES AND MANUALS

Several Codes of Practice and widely accepted manuals are listed in Table 1, together with a summary of the approach each document takes to stability and ultimate limit state (ULS) design calculations. Factors of safety have been given the following symbols: F_p applied to passive pressure; F_ϕ applied to $\tan\phi'$; F_{cu} applied to c_u. F_ϕ and F_{cu} are partial factors in some approaches. Load factors γ_F applied to earth or water pressures are only used in BS 8110, but draft BS 8002 and Eurocode 7 use partial factors γ_q on surcharge loading.

The existing British code of practice, CP2, was published in 1951 before the advent of embedded concrete walls. It was based on working stress design principles and is still found to be useful guide though it has never been adopted by the British Standards Institution. A limit state code has been in draft form for many years but agreement on its contents has proved to be very difficult. In the meantime, the reinforced concrete code, BS 8110, has adopted a compromise approach.

CIRIA Report 104 (Padfield and Mair 1984) is widely used, beyond its declared scope of cantilever and singly propped embedded walls in stiff clays. It summarised several methods of design

Table 1 Methods and factors of safety used for ULS design.

Document	Factors of safety used to derive wall penetration	Bending moments used in ULS calculation of wall strength	Factors of safety for strut loads
British Code CP2 (1951)	$F_p \approx 2$, applied in practice to effective stress or c_u.	Working stress design with active pressures.	Working stress design with active pressures.
CIRIA Report 104 (1984)	Various - see Table 2	Moments from <u>unfactored</u> soil strengths x1.5 for ULS design.	F=2 plus increase for upper props.
BS 8110 (1985) Concrete code	Not applicable	<u>All</u> working earth and water pressures (from CP2) $x\gamma_F$=1.4.	<u>All</u> working earth and water pressures (from CP2) $x\gamma_F$=1.4.
BS 8002 (DRAFT) British Standard	F_ϕ=1.2, F_{cu}=1.5 plus mandatory 'unplanned excavation'. γ_q=1.5 on surcharge.	Bending moments from wall length calcs used as design (factored) values to BS8110	
Hong Kong Geoguide 1 (1982)	Strutted walls: F_p=2 or F_ϕ=1.5 to 2.0, F_{cu}=2 to 3 Cantilevers: F_ϕ=3 (!)	To be taken from structural codes	Terzaghi and Peck with structural codes
Hong Kong Geoguide 1 (DRAFT)	F_ϕ = 1.2 F_{cu} = 2.0	Bending moments from wall length calcs used as design (factored) values.	
Eurocode 7 Chapter 8 (DRAFT)	F_ϕ=1.25, F_{cu}=1.5 plus recommended 'unplanned excavation' γ_q=1.1 on variable loads.	Bending moments from wall length calcs used as design (factored) values EC2.	Strut forces from wall length calcs used as design (factored') values to limit state code EC2.

and the factors of safety adopted are shown in Table 2. It was emphasised that the justification for these values was unclear and an attempt was made to ensure compatible results from the various schemes of factoring.

The Hong Kong Geoguide 1 (GCO 1982) is a useful update of CP2 and a re-draft based on limit state and partial factor methods is to be published soon. The new draft has much in common with Eurocode 7 which will probably influence other codes in the coming years and is therefore included in this report.

In practice, British engineers make use of the advice and design approaches in all these documents, despite their mutual inconsistency. Rigid adherence to any one approach is usually avoided since all have weaknesses.

4 COMMON DESIGN PROCEDURES

Initial design of most walls is performed using simple diagrams of active and passive earth pressure. Finite element computations are sometimes used on large projects, and semi finite element analyses, described below, are more common. These more detailed computations may lead to adjustments of the initial design. The semi-empirical Terzaghi-Peck envelope is also used occasionally, especially in the design of steel sheet pile walls.

4.1 Wall geometry and ultimate strength

The geometry of the wall and strutting system is usually established using diagrams of active and passive earth pressure. These are modified by factors of safety but usually no account is taken of redistribution of earth pressure due to deformation. For multiply propped excavations, a separate calculation is performed for each dig and strut level during the excavation, representing the higher struts by imposed forces. Table 2, based on Padfield and Mair (1984), shows values of factors of safety for the two main calculation methods used to determine the geometry of the wall.

The adoption of a factor of safety F_p=2 on passive pressure was encouraged by the old code of practice CP2 and is still common. The factor is generally applied to passive effective stress to determine the wall geometry and the bending moments derived from this calculation are not used. Instead, the wall is re-analysed using a unit factor of safety F=1 and the bending moments derived from this calculation are adopted as 'working' moments; for ultimate limit

Table 2 Recommendations of Padfield and Mair (1984).

Method		For moderately conservative parameters (c', ϕ', or c_u)		For worst credible parameters (c' = 0, ϕ')	
		Temporary	Permanent	Temporary	Permanent
Strength factor method	Eff. stress F_ϕ	ϕ'=20° 1.2 $\phi'\geq$30° 1.1	ϕ'=20° 1.5 $\phi'\geq$30° 1.2	1.0	1.2
	Total stress F_{cu}	1.5		-	-
Factor on passive F_p	Effective stress	ϕ'=20° 1.2 $\phi'\geq$30° 1.5	ϕ'=20° 1.5 $\phi'\geq$30° 2.0	1.0	ϕ'=20° 1.2 $\phi'\geq$30° 2.0
	Total stress	2.0	-	-	-

state design to structural codes they are multiplied by 1.4 or 1.5. A similar approach is applied to undrained soils, but retained (active) soil is not usually treated as undrained. Simpson (1992) has argued that the use of $F_p=2$ leads to walls which are unnecessarily long, but not strong enough to take advantage of their length.

The use of a strength factor F_ϕ or F_{cu} is sometimes adopted, especially by geotechnical engineers. Current draft versions of a British code of practice (BS 8002), Eurocode 7 and the Hong Kong Geoguide all recommend this method in various forms. These documents do not follow the recommendation of Padfield and Mair to use a higher value for F_ϕ for lower angles of shearing resistance. They propose that the bending moments derived from the calculations may be used directly as ultimate limit state design values for structural design (ie no further load factor is applied), and in this respect differ from the approach generally adopted in structural codes where partial load factors are applied to earth pressures. Proposed values of F_ϕ applied to $\tan\phi'$ are 1.2 or 1.25 and an allowance for the 'unplanned' removal of a small amount of the passive soil is recommended in some cases. This approach generally leads to shorter, stronger walls in clays; in soils with high angles of shearing resistance the wall penetration required may be greater than derived with $F_p=2$.

4.2 Serviceability

Padfield and Mair note that, in stiff clays, the design methods they propose generally provide for serviceability requirements without further strengthening, though the distribution of reinforcement must be checked to limit crack widths. The drafts of Eurocode 7 and BS 8002 require explicit consideration of the earth pressures likely to prevail in the serviceability limit state, and note that these may be greater than at collapse. BS 8002 proposes that these earth pressures may be assessed by considering (a) the initial (at rest) pressures, following deposition or compaction of the soil, and (b) the ability of the wall and restraining soil to withstand high pressures in the retained soil. Similar considerations led to the adoption in BS 8110 of the factor 1.4 on all earth pressures, both adverse and beneficial to wall stability, in ultimate limit state design 'in order to satisfy the requirements of safety and serviceability without undue calculation'.

4.3 Strut loads

A cautious approach to the evaluation of strut loads is adopted in most current designs. Padfield and Mair recommend that forces should be calculated from unfactored active/passive diagrams, and a factor of safety of 2 should be applied having first increased the loads calculated on the top two props. The draft limit state codes use strength factors F_ϕ but no further factor on strut loads; this will probably lead to lower design prop forces. ICE (1992) included discussion on the significance of changes of prop force due to temperature effects.

For steel sheet pile walls, and occasionally for concrete walls, the envelope of strut forces developed by Terzaghi and Peck is used. More commonly, results are taken from computer programs of the type discussed below.

4.4 Numerical analysis

In many cases, no calculations beyond those described above are undertaken. However, where there is a possible economic advantage in a refined design, or where ground movements are critical, finite element or semi finite element computations are common. These often examine the 'working' or 'serviceability' state but are sometimes used to investigate ultimate limit state conditions, leading to modification of the values of forces and moments adopted for structural design.

'Semi finite element' programs use beam elements to represent the wall and model the soil using either a system of springs (a subgrade reaction model) or an elastic-plastic continuum (a boundary element model) as described by Pappin et al (1986) for the FREW program. These programs are simple to use and can model the entire sequence of excavation and strutting. Active and passive limits are specified directly but only a very simple elastic-plastic model of the soil is available and pore pressure changes are not identified. In comparison with finite element computations, these programs often compute similar bending moments but specification of appropriate coefficients of subgrade reaction for a springs model is difficult, if not impossible. The FREW program was developed to overcome this problem and to allow more correctly for vertical transfer of stress in the soil; some applications are described in ICE (1992). Simpson (1993) has pointed out that semi finite element programs do not allow for the effects of vertical shear on the plane of the wall and that this leads to an over-prediction of displacement in some cases.

Finite element programs have been used extensively for fundamental studies, prediction of ground movements and as an aid to design of the walls and strutting systems. Many studies of wall behaviour in London Clay have been carried out, including early linear elastic computations, linear elastic-plastic models (eg Hubbard et al 1984) and Cam-clay models (Powrie and Li 1991). Over the last 15 years, it has become recognised that many soils, including London Clay, exhibit much higher stiffness at small strain than they do when larger strains develop. A range of numerical models has been developed to reproduce this effect, including Jardine et al (1986) and the BRICK model of Simpson (1992). Numerical studies have been used to guide the development of design methods, and specific predictions have been used in design on many projects.

4.5 Ground movements

The magnitude and distribution of ground movements is generally considered on the basis of recorded case histories, often using collations such as Clough and O'Rourke (1990). Thompson (1991) has collected case histories of ground movement caused by construction of the wall prior to excavation itself. When finite element programs are used, they give a complete computation of ground movements, and much of the recent development of stress-strain models has been stimulated by the need to improve predictions of both magnitude and distribution of deformation.

The semi finite element programs provide an indication of wall movement but do not compute the distribution of ground movement behind the wall. It is often assumed that the displacement of the wall may be reflected back to the ground surface as proposed by Nicholson (1987), for example. Generally, it is found that deformations in stiff clays may extend to a distance equivalent to about 3 times the excavation depth, whilst deformations are more localised in soft clays and granular materials. It is recognised that in soft clays settlement caused by dewatering may be serious unless well controlled.

5 INSTRUMENTATION AND MONITORING OF STRUCTURES

Major braced excavations are often instrumented, partly in order to protect the interests of the client against spurious claims for damage and partly for research purposes. Design methods, especially in relation to ground movements, have depended very heavily on the availability of measurements from previous excavations. As a minimum, optical surveys of the tops of walls are frequently made.

Many projects include measurement of the displacement of walls using either inclinometers or, more recently, electrolevels (Lings et al 1991). Several such projects are described in ICE (1989), together with comments on installation techniques and reliability. In some cases extensometers are also installed to measure vertical displacements within the ground. General experience has been that magnetic extensometers are robust and reliable and inclinometers give adequate results when cast into concrete walls but are less reliable when used in boreholes. Electrolevels, though more expensive to install, are more reliable and much more economical to read.

Measurement of strut loads is comparatively rare, but valuable records have been made by St John (1975) in London and Lings et al (1991) in the Gault Clay at Cambridge; recent work has highlighted the effects of temperature and other secondary effects on strut forces (ICE 1992). Successful measurement of pore pressure changes during excavation is also fairly unusual; Lings et al found that pore pressures beneath an excavation rapidly reached equilibrium whilst the swelling of the clay is continuing long afterwards.

Ground movements are influenced by the initial stresses in the ground before excavation. Many attempts have been made to measure these or to estimate them. This concern has led to the development of the spade cell (Symons and Carder 1989), a push-in device for measurement of in situ horizontal stress, and of the filter paper method (Chandler and Gutierrez 1986) for derivation of in situ stress by laboratory techniques. Symons and Carder measured horizontal stresses in the passive wedge in front of a cantilever embedded wall in London Clay.

Settlements of buildings are often measured using optical levelling. Other measurements are less routine, but include surveys by theodolite, crack monitoring, and tilt measurement (ICE 1989,1992).

6 IMPORTANT CASE HISTORIES

Many case histories have been noted above and others are easily accessible in the proceedings of conferences (ICE 1989,1992). Formative early work was carried out by Skempton and Ward (1953) in the soft alluvial clays of the Thames estuary. Particularly important and complete case histories have included St John (1975), Tedd et al (1984), Ryalls and Stevens (1989) and Lings et al (1992).

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