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The application of the CRISP finite element program to practical retaining wall problems

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Introduction

Finite elements may be used for practical retaining wall design in one of two fundamentally different ways

(*a*) The 'what if' approach, where the sensitivity of a particular calculated value is examined so that a design can be improved in a qualitative way, or critical design parameters can be identified.
(*b*) The 'absolute' approach, where values are required to be totally realistic and accurate, used for example when attempting to predict the displacements around a diaphragm wall.

There is an increasing trend towards using finite element analyses in the second way. There is a general optimism amongst programmers that finite element analyses always yield the 'truth', but many practising civil engineers have developed an extreme mistrust of finite element output. It is certainly true that it is very easy for an inexperienced engineer to produce results which appear ridiculous to any person with real experience of the problem being solved.

When using finite element packages for retaining wall analysis there are many factors which potentially can affect a desired result. What is required is

- a knowledge of the factors, and how they influence the results
- awareness of when problems are known to have occurred
- strategies for testing finite element output, to ensure as far as possible that the results are reasonable

Retaining wall analysis is often particularly difficult because

- geometry is often complex
- both short and long term solutions are required
- there are high stiffness contrasts
- for design purposes, accurate predictions of soil displacements, wall bending moments, and prop/anchor forces are required

This Paper attempts to draw attention to the major issues involved and to highlight the potential pitfalls for the novice. Many of the examples cited in this paper have been encountered whilst using the CRISP package (Britto and Gunn, 1987) but all finite element analyses could, potentially, have similar problems.

Potential and observed problems

A number of problems and unexpected results have been reported when finite elements have been used in practical retaining wall analyses. The problems can be divided into the following categories

(*a*) Geometric modelling and discretization
(*b*) Constitutive modelling and parameter selection
(*c*) Modelling of installation, excavation and pore pressure equalisation
(*d*) Computational difficulties
(*e*) Obtaining required design output.

Geometric modelling and discretization

For all analyses (not only those involving retaining walls) the finite element method will always give an approximate solution, the accuracy of which will be influenced by the mesh used. At the outset, the analyst must decide how far away the remote boundaries should be located, how many elements (and what type) should be used, and how the element size should be varied across the mesh (grading). Each of these decisions will potentially affect the results, and the inexperienced user is usually offered little guidance.

Element-specific problems have been encountered in CRISP with bar elements used as ground anchors (Swain, 1989); it is possible to misinterpret the way in which the pre-stressing forces are applied. Interface elements to allow relative wall-soil slip and separation have been known to transmit residual tensile stresses after soil and wall have separated (Powrie and Li, 1990).

Constitutive modelling and parameter selection

Finite element packages such as CRISP generally offer the user a number of different constitutive models. These can range from simple elastic models to highly sophisticated elasto-plastic strain hardening/softening models, and the choice is closely linked with the selection of appropriate soil parameters. The issue facing the designer is, quite simply, how much of this complexity is required in order to ensure a realistic result. This section addresses some of the salient issues.

Non-homogeneity

For most soils, mean effective stress (and hence stiffness) increases with depth. Non-homogeneous stiffness models of the form $E = E_0 + m.z$ may cause problems (e.g. stress oscillations) when $E_0 = 0$. Difficulties with CRISP are known to have arisen in plate test analyses, where the "structure" is in full contact with this surface of theoretically zero stiffness (Hillier, 1992), and to a lesser extent with a retaining wall (Swain, 1989). Potts and Burland (1983), however, did not report any problems when analyzing the Bell Common Tunnel walls, with $E_0 = 0$.

For those elements at the soil surface, integration points just below the top boundary have a very low modulus E compared with integration points at the bottom boundary. Within a single element it is thus possible to have a large stiffness ratio (see below). Although there are relatively few soils which truly exhibit $E_0 = 0$, it is thought that some investigators have used a low E_0 to minimize the effect of the retained soil holding back the wall.

Anisotropy

Many natural soils exhibit anisotropy, as a result of stress history. For retaining walls embedded in such soils it would seem logical to use anisotropic elasticity as a constitutive model (e.g. Simpson et al., 1976; Creed, 1979). However, stress induced anisotropy depends on OCR, which is not itself constant but will vary (usually decreasing) with depth. So, to model stress induced anisotropy rigorously would require a fairly complex variation of E_h and/or E_v with depth. Care is also required in the selection of Poisson's ratio for anisotropic soils under conditions of undrained loading (Bishop and Hight, 1977).

Small-strain behaviour

The extent and magnitude of ground movements around retaining walls are often over-predicted using simple elastic models together with stiffness parameters measured in conventional laboratory tests. This is not only because conventional laboratory tests over-estimate strains by including bedding effects in their measured values, but also because the strain levels around walls are typically very small (see Jardine *et al.*, 1986). Two approaches are currently in use. In the first, back analysis of similar structures in similar ground conditions is used to obtain parameters (typically linear elastic cross anisotropic), and these are then used for forward prediction. In the second approach, small strain triaxial testing is carried out on samples obtained from the actual site to obtain stiffness parameters. Both approaches have potential problems associated with them. In the first approach, it is implicit that both geometry and ground conditions are sufficiently identical to allow satisfactory forward prediction, whilst the second approach assumes a simplified soil behaviour, as well as a knowledge of a suitable average operational strain level.

Linearity

The overall pattern of displacements predicted using a simple linear model (e.g. the settlement profile behind a wall) may be incorrect (Burland and Hancock, 1977). The principal reason for this has been shown to be the non-linear nature of most natural soils (Simpson *et al.*,1979), even at relatively small strains (Jardine *et al*, 1986, 1991). As with other codes (for example, ICFEP) a non-linear model based on a power law has been devised (Gunn *et al*, 1992) and used successfully within CRISP. It would be perfectly possible to implement an empirical stiffness-strain model of the form proposed by Jardine *et al* (1986). Many different testing strategies have been used to obtain the required parameters for such analyses, and they are typically relatively complex (see, for example, Jardine *et al.*, 1991). Simpler, bi-linear, elastic models have been developed for CRISP (e.g. Leach, 1985), and work is under way to develop kinematic models to mimic the changes of stiffness typically observed when soils are subjected to changes in stress path direction.

Yielding

All soils exhibit non-recoverable (i.e. plastic) behaviour above certain levels of stress or strain. For effective stress analysis, the Mohr-Coulomb yield criterion appears to be well established and is widely used in CRISP and other finite element programs. For total stress formulations, the equivalent Tresca yield criterion may be used (e.g. Jardine *et al*, 1986).

Closely allied to the selection of the yield criterion is the specification of in-situ stresses. As observed by Burland (1978), there is little point in carrying out a sophisticated non-linear elasto-plastic analysis if the initial stresses are incorrect. In this respect the at-rest earth pressure coefficient K_0 can be particularly difficult to measure or estimate over the full depth of interest.

Normality

When the stress state of an element of soil reaches (and remains on) the yield surface of an elasto-plastic material, plastic straining will occur. The relative components of plastic volumetric and plastic shear strain are determined by the plastic potential. If the flow rule is associated and normality applies, then the yield surface and plastic potential are coincident, and the plastic strain vector is normal to the yield envelope. This assumption is notorious for giving excessive dilation or, where drainage is prevented, negative excess pore pressures at yield/failure (for example, see Simpson *et al.*, 1979).

Associated flow is considerably easier to implement and use in a finite element package than non-associated flow, and is therefore commonly used. But such strong dilation (i.e angle of dilation = ϕ') is unrealistic, and can lead to over-enhanced strength and stiffness around the wall (Ponnampalam, 1990; Powrie and Li, 1990). There is little consensus on the most suitable angle of dilation ($\phi'/2$ is used by some) or what aspects of retaining walls are most affected. Potts and Burland (1983) studied it and concluded that wall movements were largely unaffected. It seems likely that earth pressure distributions would be more sensitive.

Modelling of installation, excavation and pore pressure equalisation

When analyzing the installation of a diaphragm-type wall with a finite element program there are three different strategies that one might adopt

(*a*) Begin the analysis with the wall already installed (the so-called "wished-in-place" wall),
(*b*) Begin with original undisturbed ground and then swap the relevant soil and concrete elements in the same time increment, or
(*c*) Begin with original undisturbed ground and then simulate the excavation of soil under bentonite slurry support, the placing of wet concrete via tremie pipe, and the subsequent hardening of the concrete.

Higgins *et al* (1989) compare strategies (*a*) and (*c*) in a re-analysis of the Bell Common tunnel, using undrained analysis in the short term and imposed pore pressure changes to model the long term. Gunn *et al* (1992) give further details of the numerical aspects of strategy (*c*), but using a coupled-consolidation approach. One significant anomaly which has been observed when wall installation is modelled is the spatial and temporal oscillation of effective stress and pore pressure in the lateral direction. This was first reported for CRISP by Kutmen (1986) and subsequently by Ponnampalam (1990). Similar oscillations were observed by Naylor (1974).

In contrast to most other finite element codes, users of CRISP are able to model the whole sequence of installation, excavation and pore pressure equalisation in one continuous analysis, using the coupled consolidation (Biot) approach. Whilst this is a convenient and elegant approach, the choice of time step is important if numerical problems are to be avoided (Woods, 1986). The method by which excavation is modelled can also be a problem - incorrect formulations lead to solutions which even for elastic cases are increment dependent (Ishihara, 1970; Gunn, 1982; Brown and Booker, 1985).

Computational difficulties

Disparate stiffnesses

There is a limit to the difference in stiffness that can be tolerated between (adjacent) elements in a mesh, before computational problems arise. This has implications for all soil-structure interaction problems where soil and concrete (or steel) are being modelled. The problem is well known and is caused by adding numbers of different orders of magnitude in a finite precision computer. The net result is equilibrium error in the analysis and possible ill-conditioning.

Equilibrium errors due to disparate stiffnesses have been reported by Woods and Contreras (1988), Vaziri (1988), and Hillier (1992) for rigid steel plates in contact with soil. Exactly the same problem can be expected to occur in the analysis of a thin steel sheet pile wall. These types of problem are associated both with the level of precision (i.e. single or double) invoked by the code, and also by the precision used by the computer's c.p.u. - they can therefore vary from machine to machine, even when using identical code.

Effective stress method

Most geotechnical finite element codes (including CRISP) use the effective stress method (Naylor, 1974) for both drained and undrained loading. The bulk modulus of the pore fluid K_W, is set either to zero (= drained) or to an arbitrarily large number (= undrained). It is generally believed that the actual magnitude of K_W used for undrained analysis is unimportant, provided it is much greater than the bulk modulus of the soil skeleton. However, Ponnampalam (1990) has shown that horizontal stress distributions resulting from wall installation can be very erratic when K_W, is large. The problem is analogous to that of selecting a value of Poisson's ratio close (but not exactly equal) to 0.5 for an undrained analysis in terms of total stresses.

If the value of K_W, is too low, then the behaviour modelled is similar to a partially saturated soil ($B < 1$), with some load being taken in effective stress even in the short term. Clearly K_W must be neither too high nor too low, relative to the drained modulus of the soil, but it is not clear that there will always be a satisfactory range between these limits, for all combinations of geometry and soil properties.

Horizontal stress distributions

Soil elements closest to the wall on the excavated side (just below final dig level) have been known to exhibit high horizontal effective stresses, after removal of elements to simulate excavation. It appears to be possible to obtain stress states outside the specified failure envelope near soil surfaces where the soil has undergone swelling. This phenomenon has been observed with CRISP by Clarke and Wroth (1984), Ponnampalam (1990), and Powrie and Li (1990). It has also been observed by Rodrigues (1975) and Creed (1979) for other finite element codes.

On the retained side, tensile stresses may develop behind the top of the wall as it attempts to move away from the soil. According to Powrie and Li (1990), these are particularly pronounced for elastic-perfectly plastic models and for high soil stiffness, but may be reduced by the use of interface elements (or setting by $E_0 = 0$, as noted above).

Stress oscillations have already been mentioned, and whilst these oscillations are clearly unrealistic, their cause remains uncertain.

Solution scheme

CRISP uses an incremental (tangent stiffness) solution scheme. For elastic perfectly plastic models, corrections are applied to elements that have yielded in order to bring the stress state back to the yield surface. For critical state based models no such corrections would be carried out as they would be considerably more complex. It is therefore relatively easy to produce an invalid analysis unless increment size is sufficiently small. For a single stage of loading on a Mohr-Coulomb material, about 50 increments are typically required. In contrast, other codes (for example, ICFEP) use different solution strategies, and can provide adequate solutions with only a few increments per loading step.

Obtaining required design output

Wall and ground deformations

There is no problem in obtaining displacements as they are the primary unknowns solved directly by the finite element method. As discussed above, however, there are a host of influences which will affect the reliability of both the magnitude and pattern of displacements.

Stress distributions

Stress distributions may be required for comparison with classical design methods, or perhaps with in-situ measurements. However, they should not be used as data for bending moment calculations, and may be unreliable as a basis for simple hand calculations of equilibrium. Unexpected patterns can occur on both the retained and excavated sides (see above), potentially confusing the inexperienced analyst. It must be remembered that the finite element method only ensures equilibrium of nodal forces, and that in general there will neither be local equilibrium within an element nor equilibrium of stresses across element boundaries. Creed (1979) describes a method of inferring approximate pressure distributions from nodal loads.

Wall bending moments

The calculation of wall bending moments may be based on

(*a*) transverse stress distributions in the wall elements
(*b*) horizontal soil pressures acting externally on the wall elements
(*c*) nodal forces acting between wall elements

(*d*) nodal forces from the soil acting externally on the wall elements

The first two methods are intuitively reasonable and also quite convenient as they make use of stresses calculated at integration points in the relevant elements. However, as noted earlier, stress distributions can be far from reliable; furthermore, method (*b*) multiplies (possibly) inaccurate stresses by an increasing lever arm as one moves down the wall. Comparisons of these two methods have shown significant discrepancies (Swain, 1989; Gunn and Ponnampalam, 1990; Powrie and Li, 1990).

Methods (*c*) and (*d*), on the other hand, make use of nodal forces which can be expected to be rather more reliable. As these forces are not normally output to the user, some program modifications may be necessary.

Prop and anchor forces

Temporary and permanent props are frequently modelled by solid 2D elements, and load distributions can be readily obtained from compressive stresses at integration points. Anchors may be modelled by bar elements, and once installed it is relatively easy for the program to calculate the excess force over the initial (pre-stress) value. The total force at any stage is thus readily computed.

Nodal forces may be used to make simple (hand) checks that the output satisfies force equilibrium; for example, to detect equilibrium errors due to inadequate precision.

Conclusions

Finite element analysis of retaining walls is potentially particularly problematic; perhaps more so than any other geotechnical application of finite elements. The most serious problems are choice of constitutive model and associated soil parameters, the modelling of installation and excavation, numerical irregularities, and the derivation of design output. Finite elements cannot be used "blindly" for retaining wall analysis. Validation against real problems is required, and in this respect there is a great need for more published case histories and guidelines for the inexperienced user.

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