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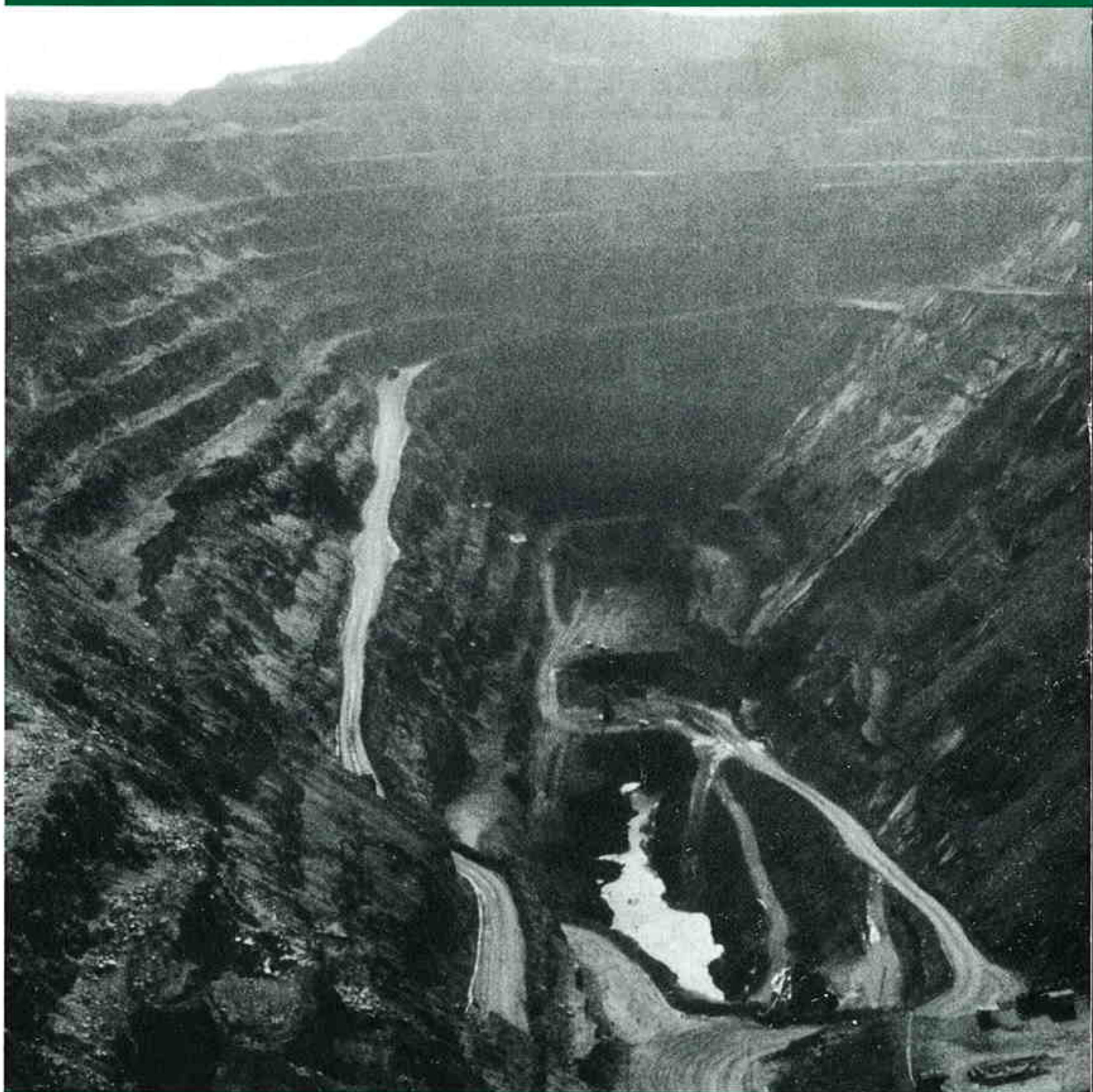

Mineral resources and the environment

Contents

Modelling slope behaviour for open-pits

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Synopsis

The options for designing slopes in complex geological conditions are reviewed. It is demonstrated that modelling—especially numerical modelling—allows a degree of analysis that cannot otherwise be achieved. Examples are given of how numerical modelling can be used to investigate fundamental aspects of rock mass behaviour; it is demonstrated that an important consequence of change in scale is that the mechanisms of failure also change. The complex failure of the footwall slope in a large open-pit mine in southern Spain is discussed. Numerical modelling with UDEC (universal distinct-element code) has allowed displacements measured in the field to be simulated realistically. The calibration of the model gives confidence that it can be used to predict the effect of future mining and to assess the effectiveness of possible remedial measures.

Open-pits need to be designed so that the mineral can be extracted economically while reserves are optimized. Slopes are generally cut as steeply as possible; minor rockfalls and bench failures are considered inevitable and are even taken as an indication that the geometrical configuration is not too conservative. Little attempt is made to stabilize temporary slopes. Care is taken, however, to avoid major failures that might disrupt operations, although solutions requiring active reinforcement, such as anchoring, are not adopted unless absolutely necessary, partly because such measures are often ineffective.[1] With such a management philosophy there is considerable onus on the geotechnical engineer to design safe slopes primarily on the basis of the relationships between the geometry of discontinuities and that of the slopes.

At many sites the general geological structure will dictate overall slope design, and the principles that must be observed to avoid sliding on or toppling due to major discontinuity sets, such as bedding, schistosity or cleavage, are well established.[2,3] Most relatively small-scale (and many large-scale) failures are geometrically quite simple and, therefore, amenable to direct mathematical analysis provided that the shear resistance of the sliding surface can be accurately determined. Piteau[4] provided a good review of those geological features which may control slope stability in open-pit mining. More complex sites are less readily dealt with and, particularly in the case of very large slopes, there may be general concern over stability even though no clearcut mechanism has been identified from investigation.

At recent conferences devoted to mining and the design of open-pits[5,6] the vast majority of contributors have discussed slope stability with reference to the relatively simple methods of analysis that were developed more than 20 years ago for well-defined, discrete failures controlled by individual discontinuities (e.g. Ashby[7]). Only a few (e.g. Rapiman[8]) have referred to the use of currently available, sophisticated geotechnical software for the modelling of slope behaviour. Stacey[9] suggested that further research is necessary in recognition of the fact that simple gravity-block analysis is not always appropriate for slope design in open-pits. It is clear that modelling, and especially numerical modelling, will play an increasingly important role in this research effort.

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Dealing with complex masses

Although failures through rock are often controlled by single discontinuities or sets that are well defined geometrically, in many other situations the potential mode of failure is far more complex, with many different discontinuities contributing greatly to the deformation mechanism yet not uniquely controlling it. This is particularly true where stresses are high relative to the strength of the rock material, when the failure of intact rock bridges becomes a possibility. Such conditions are most likely in relatively weak rock—for example, where hydrothermal alteration is present at depth. The more geologically and geometrically complex the situation, the more difficult it is to derive representative engineering parameters to use in general analysis and the less justified it is to use a simple limit-equilibrium approach.

Among the difficulties are: the impossibility of testing at a sufficiently large scale to derive representative rock mass parameters; poor theoretical or empirical understanding of the influence of scale on mass behaviour; the inherent difficulties in dealing with complex systems from first principles in a theoretical way; and the difficulties in using a back-analysis approach without oversimplifying the problem.

Three possible approaches are: to treat the rock mass as an isotropic continuum; to design by precedent; or to use modelling.

Isotropic continuum approach

When dealing with fractured rock in which clear, kinematic modes of failure controlled by well-defined discontinuity sets have not been identified a strength envelope can be derived for the mass by use of the Hoek–Brown criterion.[10] The general criterion, as discussed most recently by Hoek[11] and Hoek and co-workers,[12] takes the form

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} + s \right)^a$$

where σ_1' is major principal effective stress at failure, σ_3' is minor principal stress at failure and σ_c is uniaxial compressive strength of intact rock. m_b is a constant for the rock mass and corresponds approximately to the friction angle, ϕ, in the linear Mohr–Coulomb criterion. It varies both with rock type (m_i component) and with an index known as the geological strength index (GSI), which, in turn, is directly related to the rock mass classification ratings of Barton and co-workers[13] and Bieniawski.[14] s and a are constants that depend on the characteristics of the rock mass; s is approximately analogous to cohesive strength in the Mohr–Coulomb

Fig. 1 Complex slope failure in weak rock mass, Tsing Yi Island, Hong Kong

criterion and tends to zero for a very poor-quality rock mass, reflecting the low tensile strength of such a weak mass. For a given rock type (e.g. sandstone or granite) the parameters m_b, s and a vary with structure (classes range from widely jointed and blocky to crushed) and with the surface condition of discontinuities (conditions vary from very rough and unweathered to slickensided or clay-infilled). Generally, the parameters s and m_b reduce markedly with increasing number of discontinuity sets, increasing fracture frequency and lower shear strength of discontinuity surfaces. Within the various ratings, however, no allowance is made for the *degree of adversity* of jointing; it is taken as a basic assumption for the Hoek–Brown criterion that joint orientations are 'very favourable', as defined by Bieniawski.[14]

It is of note that whereas the Hoek–Brown criterion predicts a reducing shear strength with increasing fracture frequency for an isotropic rock mass, regularly jointed models tested by Bandis and co-workers[15] indicated the reverse to be true, the models that comprised the smallest block size showing the highest strengths. This apparent paradox appears to relate to the fact that sliding along discontinuities is, by definition, not the dominant mechanism for a Hoek–Brown mass, whereas frictional sliding was the main mode of failure in the physical models tested by Bandis and co-workers.[15] These authors suggested that where sliding does dominate, the reduced stiffness of the more closely jointed mass and the greater ability of small blocks to rotate may increase the effective roughness component of shear strength.[15]

Clearly, not only are the predicted effects of scale different for these two scenarios but the cases also represent opposite extremes of a complete spectrum of possibilities for rock mass conditions. In many situations, although the fracture network does not permit failure to be dominated by sliding along a single set of discontinuities, impersistent and adversely orientated sets of discontinuities may be present within the rock mass and sliding along these will contribute to the process of deformation and, probably, to reduced rock mass strength.[16,17] An example of a slope failure involving partial sliding along non-daylighting discontinuities as well as failure through intact rock is given in Fig. 1.

It must be emphasized, therefore, that the Hoek–Brown approach is only appropriate where the rock mass can be regarded as truly isotropic with 'very favourable' joint orientations.[11] Also, care should be taken in adopting the shear strength data from this approach for slopes as the strengths are more likely to reflect the confined rock mass conditions typical of underground situations. However, provided that the conditions are favourable, strength parameters calculated from the Hoek–Brown equations may be used in a search for potential failure surfaces using a generalized method, such as that of Sarma[18] or of Janbu.[19] Probabilistic analyses can then be carried out for a variety of potential failure surfaces through the rock mass to allow for the scatter in input data and to define the level of risk.[20] There are many commercially available geotechnical software packages (e.g. SLOPE, XSTABL) that can analyse such situations, albeit with some difficulty where the strength parameters vary non-linearly with confining stress.

An alternative approach—provided that the rock mass can be considered essentially as a continuum—is to back-analyse measured displacements to obtain rock mass parameters, as outlined by Sakurai.[21] Basically, displacements measured in the field are used in stress–strain relationships, expressed perhaps within a finite-element model, to derive representative elasticity and strength parameters. For slopes Sakurai[21] argued that a mechanical model should not be assumed prior to back-analysis but that the measured strains should instead be used to derive a unique model comprising a series of parallel layers with differing properties. Adopting such an apparently simple approach, Ono and co-workers[22] described how they were able to predict behaviour in a large slope using parameters derived by back-analysis of displacements that developed during the early stages of excavation. Sakurai and co-workers[23] used the same approach to determine geomechanical parameters for a large slope in sandstone and slate. Those parameters were then used in further analysis to calculate a factor of safety against failure. Such an approach seems particularly attractive for large open-pit slopes where faces are excavated bench by bench and where displacements are routinely measured. The increasing development and capability of geotechnical software, such as UDEC (discussed later), should allow sophisticated back-analysis of more complex geological situations than has hitherto been possible.

Design by precedent

Again, provided that the mass might be regarded as isotropic, an alternative approach for dealing with complex or very large rock masses is to design by precedent, stability being expressed empirically (by observation) with respect to simple relationships, such as slope height versus slope angle.[2,24] One of the problems with this approach, however, is that the situation may be overgeneralized and the conclusions may be questionable owing to a lack of sufficient data or only valid within very severe constraints (the discussion by Leroueil and Tavernas[25] on the difficulties in interpreting individual cases is relevant). This conclusion is supported by the results of a major study of the stability of weathered slopes in Hong Kong (CHASE) that was conducted in the early 1980s using empirical methods.[26] The aim of the exercise was to identify key factors associated with slope failure that could then be used as inputs to the design process or at least for checking. The study stemmed partly from perceived difficulties in determining the shear strength of severely weathered rock at either the small, intact rock scale or the mass scale and the anomalous survival of some slopes despite calculated factors of safety of less than unity. More than 200 failed and stable slopes were described in great detail, involving teams of geologists, geotechnical engineers, site investigation contractors and surveyors in fieldwork for more than six months. More than 400 items of information were collected for each slope and multivariate statistical analysis was used in the attempt to discriminate between failed and stable slopes. The results of the research showed that, despite great care and effort, no simple rules could be determined that would allow instability to be predicted with any confidence.[27] One conclusion following the experience of the CHASE study, which was later confirmed by detailed studies of several major landslides, is that failures are commonly the result of site-specific geological features that cannot be dealt with realistically using an empirical approach.[28]

Modelling

Where the geology is complex such that potential slope instability may involve several mechanisms, such as sliding on some discontinuities, toppling and, perhaps, failure of intact rock, all acting within the same failure, analysis by traditional methods is particularly difficult and it is in this type of situation that modelling may play a key role. Modelling is also particularly useful for investigating the development of failure mechanisms with time (progressive failure), an aspect that normal kinematic analysis of the factor of safety cannot hope to address. Over recent years numerical modelling, in particular, has become a cheap and viable option for site-specific problems. In the present contribution modelling approaches will be discussed that can provide insight into potential modes of failure. Examples will be given of how measured deformations can be explained through modelling with some degree of confidence that the envisaged mechanisms are correct.

Principles of modelling

Models can be useful both for assessing conditions at particular sites and as research tools to investigate some of the fundamental unknowns.

Starfield and Cundall[29] reviewed the use of models for geotechnical studies and offered several important comments and guidelines, some of which can be summarized as follows: models are simplifications; models should be designed to answer specific questions; a few simple models may be better than one complex model; and the approach should be that of a detective rather than a mathematician.

Despite this good advice to keep things simple, models are being increasingly used—and with remarkable success—to simulate some very complex situations. For example, Barton *et al.*[30] reported almost perfect agreement between measured displacements and those predicted by numerical modelling of a cavern with a span of 62 m in rock of only fair quality.

Physical modelling

Physical models have been used for many years in structural geology and rock mechanics to simulate rock mass behaviour. Construction can be from a range of materials, from the carefully scaled sand, oil and water glass used by Griggs[31] to the brick models of Fumagalli.[32] Physical models can also be used to simulate particular aspects of a problem, such as the shear behaviour of rock discontinuities.[15,33] Many authors have considered the problems of scaling from the field situation (prototype) to the laboratory, and several have attempted to provide a fully dimensionless scaling of the rock mass.[34,35,36] It seems, however, that it is not possible to scale all aspects of a model consistently with the prototype at the same time and it is therefore necessary to concentrate on scaling particular relevant parameters according to some 'similar performance criterion' that is selected on the basis of the nature of the problem that is being modelled.[37,38] Most

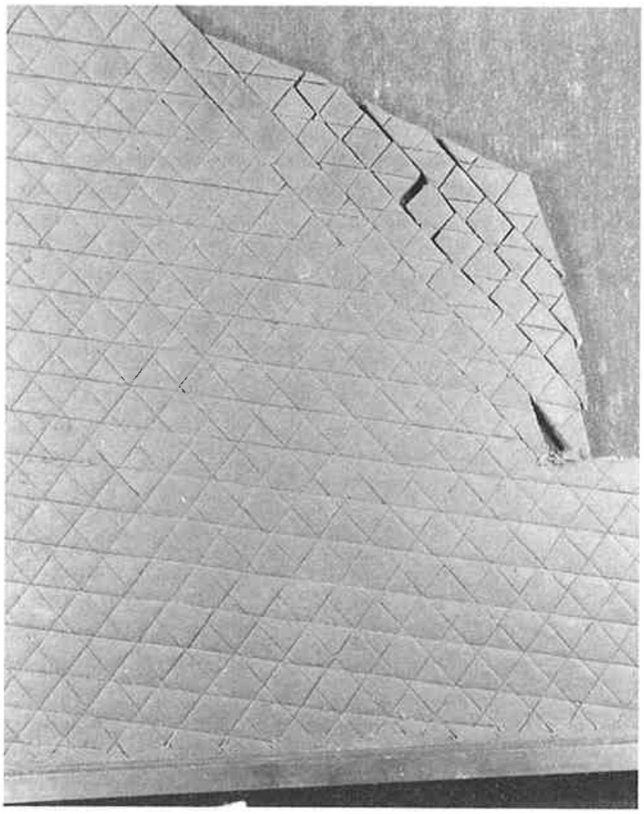


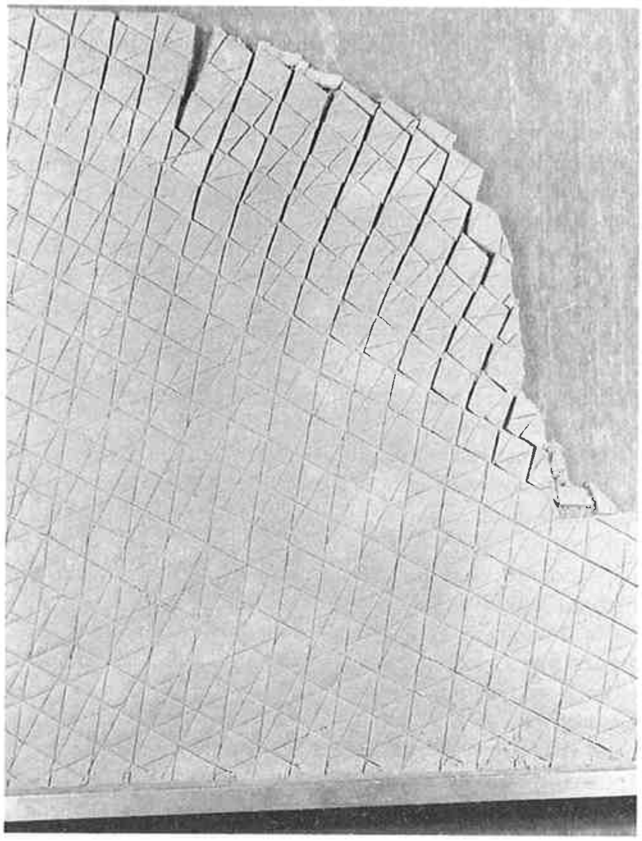
Fig. 2 Base-friction model of slope failing by sliding on daylighting discontinuities

modellers try to scale tensile or compressive strength while assuming that frictional characteristics are the same for both rock and model. This assumption may limit the quantitative nature of any conclusions.

One of the most useful physical modelling methods for the relatively low-stress conditions of the majority of slopes is the base-friction system, in which a model is constructed from a weak material, perhaps bound with plaster, on a flat bed that is coated with frictional material, such as sandpaper. When the base is moved relative to the model the frictional drag simulates the effects of gravity in a fairly realistic way. The theory of base-friction modelling was discussed in detail by Bray and Goodman[39] and the development of the technique has been reviewed by Al-Harthi and Hencher.[38] Model materials can be recyclable, which means that the method is easy, rapid and cheap and allows the influence of the various inevitable uncertainties at a site to be systematically explored without major difficulty. An example of a base-friction model of a slope is presented in Fig. 2. Comparative tests of base-friction and numerical models can show good correlation[38,40]—at least in a qualitative sense—as will be illustrated in the following section.

Numerical modelling

Numerical methods are increasingly being used to model rock masses and have been reviewed in the context of underground excavation by Hoek and co-workers.[41] Models are used in a general way to explore site-specific behaviour or may be applied to investigate fundamental aspects of behaviour. Continuum methods, such as finite-element, finite-difference and boundary-element, may have a role in open-pit slope design but are not designed specifically for anisotropic, blocky rock masses although they have been used in this way.[42] Over the last decade methods that allow the modelling of masses consisting of discrete blocks have reached an advanced stage of development. For example,



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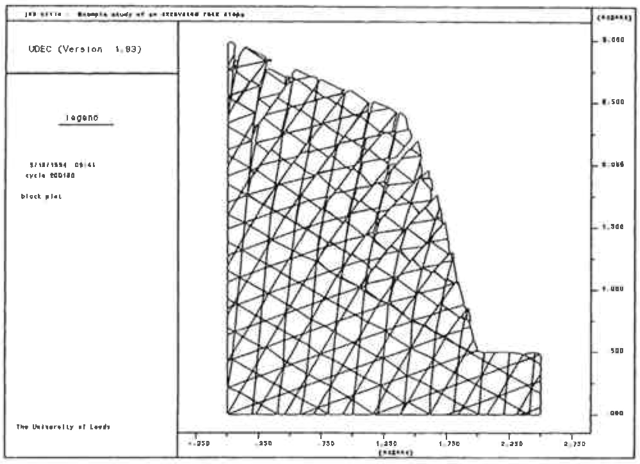
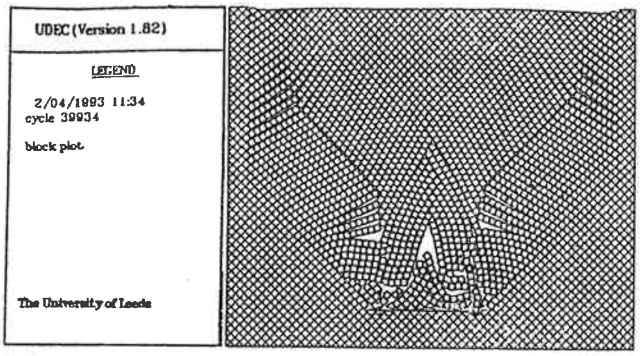


(*b*)

Fig. 3 (*a*) Base-friction and (*b*) UDEC models of slope with four sets of discontinuities

the discontinuous deformation analysis (DDA) method of Shi[43,44] has attracted much recent attention and is the subject of continued development (e.g. by Amadei and co-workers[45]).

The universal distinct-element code (UDEC), originally described by Cundall,[46] has now been developed into a very powerful code that is capable of dealing with intensely fractured and complex masses in two or three dimensions and is widely used for modelling the behaviour of both underground excavations and slopes.[47] The code solves numerical procedures that involve the equations of motion of particles or blocks rather than the continuum. Discontinuities are generated in the model either individually through reference to a grid-point system or as sets defined by such



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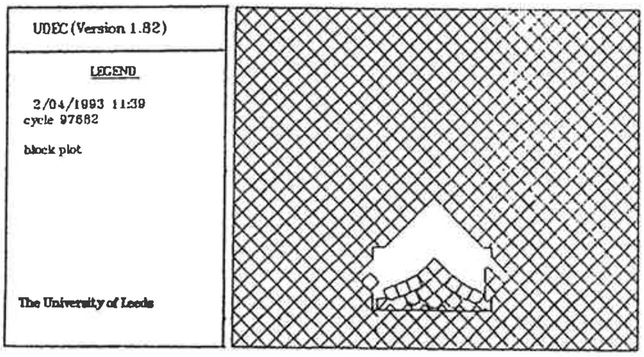

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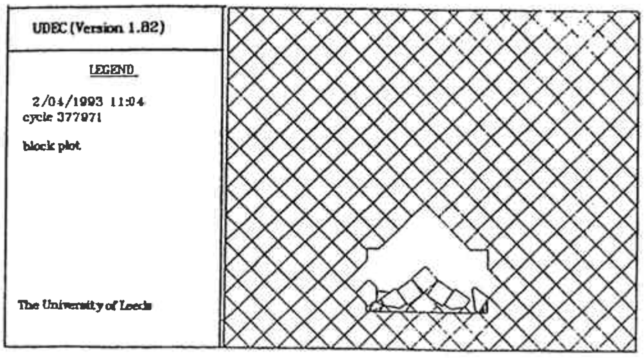
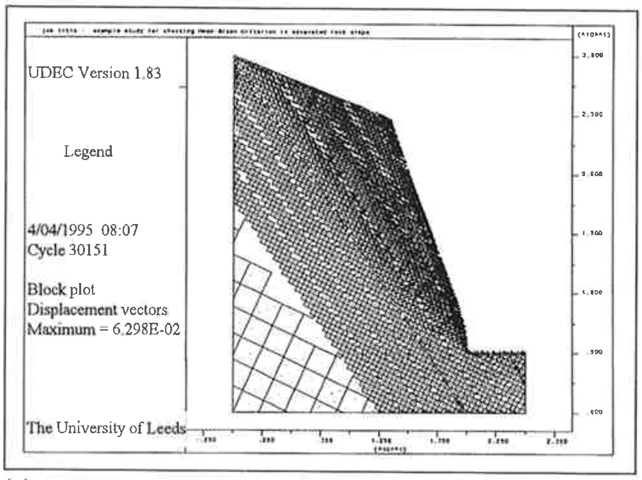


(*c*)

Fig. 4 Effect of changing block size on failure mechanism in regularly jointed rocks for an underground excavation. (*a*) Block width ≈ $h/10$, (*b*) block width ≈ $h/5$ and (*c*) block width ≈ $h/3$, where h is height of opening. (After Al-Harthi and Hencher[40])

parameters as dip angle, trace length and spacing. Although UDEC requires that fractures cannot terminate within intact rock, this restriction can be overcome by generating 'fictitious' joint segments with parameters chosen so that they behave as intact rock.[48] Constitutive laws are used to define the engineering behaviour of both the intact rock blocks and the discontinuities between rocks, with several options available according to the perceived nature of the rock. For example, blocks may be taken as either rigid or deformable through the use of finite-strain elements within the blocks. Rock joints are represented as contact surfaces between the edges of blocks and the shear behaviour may be defined by various models, including that of Barton and Bandis (Bandis and co-workers[49]). Fluid pressures within the discontinuities can be included. Although highly developed, such methods are not always straightforward to use. In particular, the decision on which parameters to employ in the model is not trivial and difficulties are often



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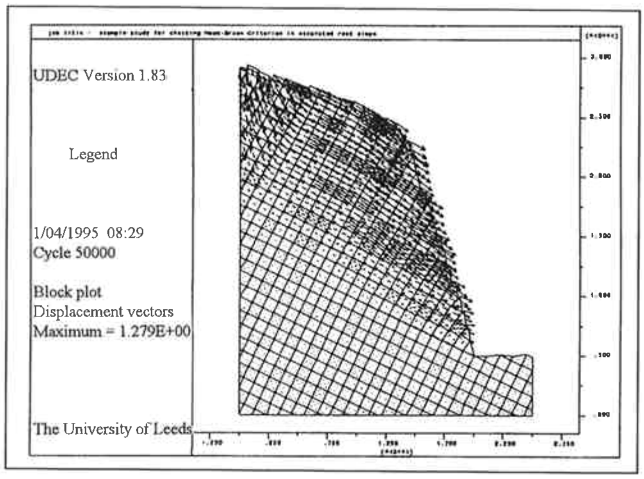

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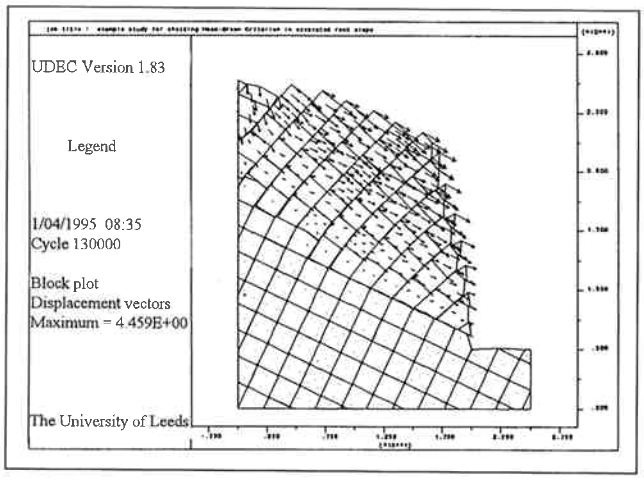

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Fig. 5 Effect of changing block size on failure mechanism in rock slope. (*a*) Block width ≈ $h/50$, (*b*) block width ≈ $h/20$ and (*c*) block width ≈ $h/10$, where h is height of steepest section of slope

encountered owing to numerical problems (contact overlap of blocks) and such aspects as trying to simulate the various stages of excavation realistically. Such difficulties can be frustrating, particularly as each run may require considerable computing time.

UDEC, like its sister continuum code FLAC (fast Lagrangian analysis of continua), is a time-stepping program. The gradual development of failure mechanisms can therefore be monitored as the joints and intact rock react to the changing distribution of forces according to the constitutive relationships that have been adopted for the problem. An example of a slope analysis using both base-friction modelling and UDEC is presented in Fig. 3. The rock mass under consideration contains four sets of discontinuities. The models are in agreement in demonstrating the dominating effect of toppling despite the potential for sliding along daylighting joints.

An opportunity that has been opened up by the availability of distinct-element codes such as UDEC is the possibility of exploring fundamental aspects of rock mass behaviour that are not amenable to analysis in other ways. As an example, the use of UDEC models to demonstrate the effect of changing block size on the stability of an underground opening within a regularly jointed rock mass is illustrated in Fig. 4. The results show that the effects of scale are not simply a matter of a change of mass strength or deformability but that changes in mechanism also result. Similarly, the effects of variations in block size on mechanisms of slope failure are illustrated in Fig. 5. In each case the orientation, continuity and joint shear strength parameters are constant. For the slope with the smallest block size (Fig. 5(*a*)) a shallow translational slide develops along a failure surface that bounds many blocks (essentially a Hoek–Brown mass failure). The slope comprising blocks of intermediate size (Fig. 5(*b*)) fails by sliding on the daylighting discontinuity set (which is present in all models). At the largest block size (Fig. 5(*c*)) toppling becomes the dominant mechanism.

Case study: Aznalcóllar open-pit mine, southern Spain

General geotechnical situation

Aznalcóllar mine is an open-pit about 240 m deep (from the +100-m level to the −140-m level) that is located approximately 40 km northwest of Seville in southern Spain. The stratiform orebody is a pyrite-rich, argillaceous unit within Palaeozoic slates, phyllites and occasional felsite and rhyolite units, all of which strike essentially east–west with a northward dip, although boundaries are often difficult to distinguish. These rocks are overlain by approximately 10 m of mainly stiff brown and red clays of Miocene age.

The north-facing footwall at the mine has a history of instability despite an overall slope angle of only about 33°, whereas the hanging-wall, which dips at between 40 and 45°, is generally stable apart from minor surficial toppling. A view of the unstable footwall slope is given in Fig. 6.

The Palaeozoic rocks are generally slightly weathered, with staining on joint surfaces. Uniaxial compressive strength is typically in the range 20–50 MPa. The degree of weathering increases towards the contact with the orebody, adjacent to the contact with the Miocene deposits, and also to the west of the open-pit, where a major fault cuts through the sequence. In these zones the rock is more discoloured and much weaker, with compressive strengths typically below 20 MPa. In the hanging-wall the Palaeozoic rock units dip to the north at 35–55° and in the footwall zone at 45–60°. The units vary in thickness and many are lensoid both downdip and along strike, possibly reflecting unconformities or transitional changes in lithology. The slates and phyllites show a strong cleavage that strikes east–west, roughly parallel to the lithological units. In the hanging-wall cleavage dips to the north, typically at angles between 40 and 60°. Throughout the footwall the dip of the cleavage ranges from 45 to 70°, although locally dips can be as low as 40° and elsewhere near-vertical. The spacing of joints parallel to cleavage ranges from 500 to 2000 mm and persistence can be greater than 80 m along strike. The surface roughness of cleavage in the slate is not significant, although on a larger scale the cleavage surfaces undulate and are frequently stepped where kink structures pass through the slaty units. The phyllite cleavage tends to have a rougher surface texture with pitting to a depth of 2–3 mm, although the surfaces are mineralized and still smooth to the touch.

In addition to joints parallel to cleavage, several other joint sets are present throughout the pit, as listed in Table 1.

Table 1 Cleavage and main joint sets at Aznalcóllar, with typical orientations

Set	Dip, degrees	Dip direction, degrees
Cleavage	55	008
D_1	80	276
D_2	78	120
D_3	32	213
D_4	75	228
D_5	55	143

The continuity of joint set D_1 typically ranges from 600 to 2000 mm and of other sets from 500 to 6000 mm. The roughness of all joint sets is similar, with local asperities on the surfaces and slight undulation on a larger scale. The typical spacing of all joints is from 500 to 2000 mm.

A recent photograph of the footwall slopes with the partially backfilled slope (that modelled) to the rear is presented as Fig. 7. Attention is drawn to the continuous cleavage surfaces in the foreground.

Hydrogeological studies of the area have established that there are at least two phreatic surfaces, one associated with the Miocene cover and the second with the Palaeozoic rock mass. The depth to the phreatic surface within the Palaeozoic (slates/phyllites) has been measured at between 30 and 70 m below ground surface, yet it can show a rapid response to heavy rain. During and immediately after one storm with 121 mm of rainfall two piezometers indicated rises in water level of 7–8 m.

History of movements

There has been a history of footwall-slope movements at Aznalcóllar mine from as early as 1976.[50,51] Initially, these were single-bench failures in the slate, slabs of rock failing on relatively shallow cleavage planes or a combination of planes producing a wedge failure. Several multi-bench failures have occurred subsequently and movements are continuing. These have been monitored by surface measurement, photogrammetry and deep inclinometers. Several movements have been associated with periods of intense rainfall, but, conversely, significant displacements have occurred during dry periods.

The first multi-bench failure occurred in the footwall slope in October, 1979. During that month a total of 245 mm of rain fell, the main movement occurring after one particularly heavy rainstorm. Crest-line cracks, approximately 50 m in length, appeared at the +85-m level. The pit bottom at this stage was at a level of +12 m, but the toe of the main footwall slope was only at the +40-m level.

In January, 1983, a major part of the footwall slope moved. A crest-line crack approximately 330 m long was created as a result of the failure. At this time the open-pit had developed to the −33-m level, but the main part of the

Fig. 6 General view of Aznalcóllar mine, southern Spain. Partially backfilled footwall slope to right is that modelled in present work

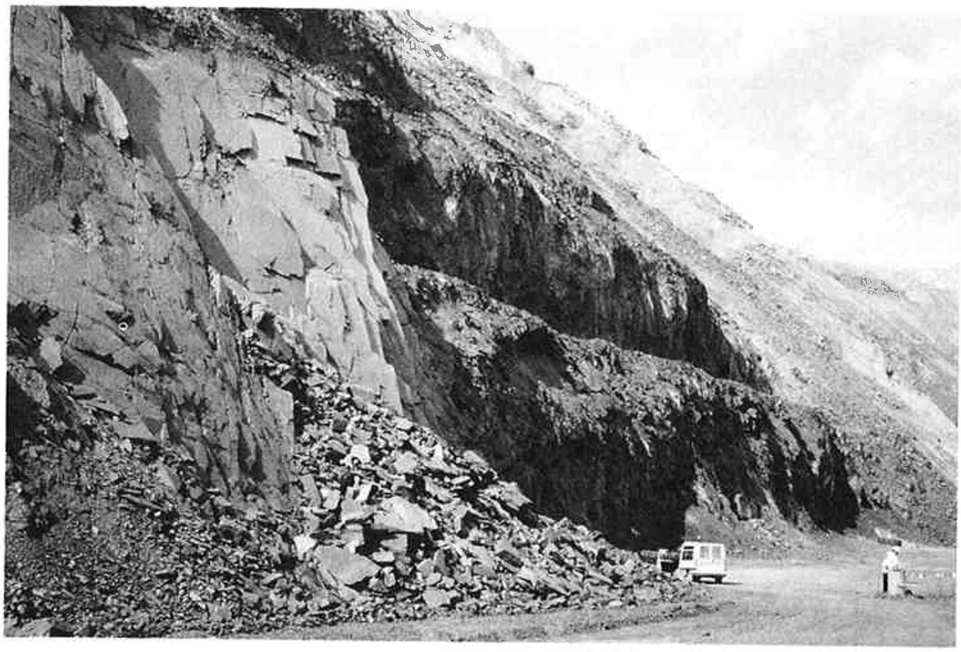

Fig. 7 Footwall slopes at Aznalcóllar with steeply dipping cleavage surfaces in foreground. Unstable slope, now partially backfilled, is to rear

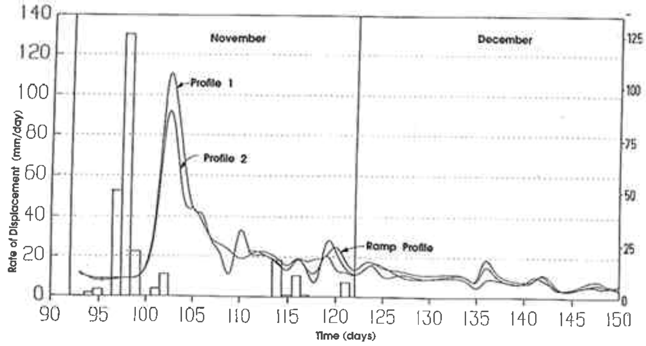

Fig. 8 Acceleration of displacement in footwall slope at Aznalcóllar following intense rainfall (indicated by bars) during November, 1988. Maximum daily diplacements given for monitoring points on three separate profiles (1, 2 and ramp)

footwall slope was at the +5-m level. The failure occurred during a dry spell; records show that no rain fell in January, 1983, and only 29 mm in December, 1982.

In December, 1987, the greater part of the footwall slope moved during and after a period of continuous and heavy rainfall (309 mm in 15 days).

In November, 1988, an area similar to that affected in December, 1987, moved after heavy rainfall of 200 mm over a three-day period. A plot of the rate of displacement with time for this event is presented as Fig. 8.

During the first half of 1992 the background rate of slope movement was approximately 2–4 mm/day; this started to increase in mid-July, the main slope movement occurring between 22 and 24 July, when a peak displacement rate of 66 mm/h was recorded. No rain fell between 23 June and 29 August of that year.

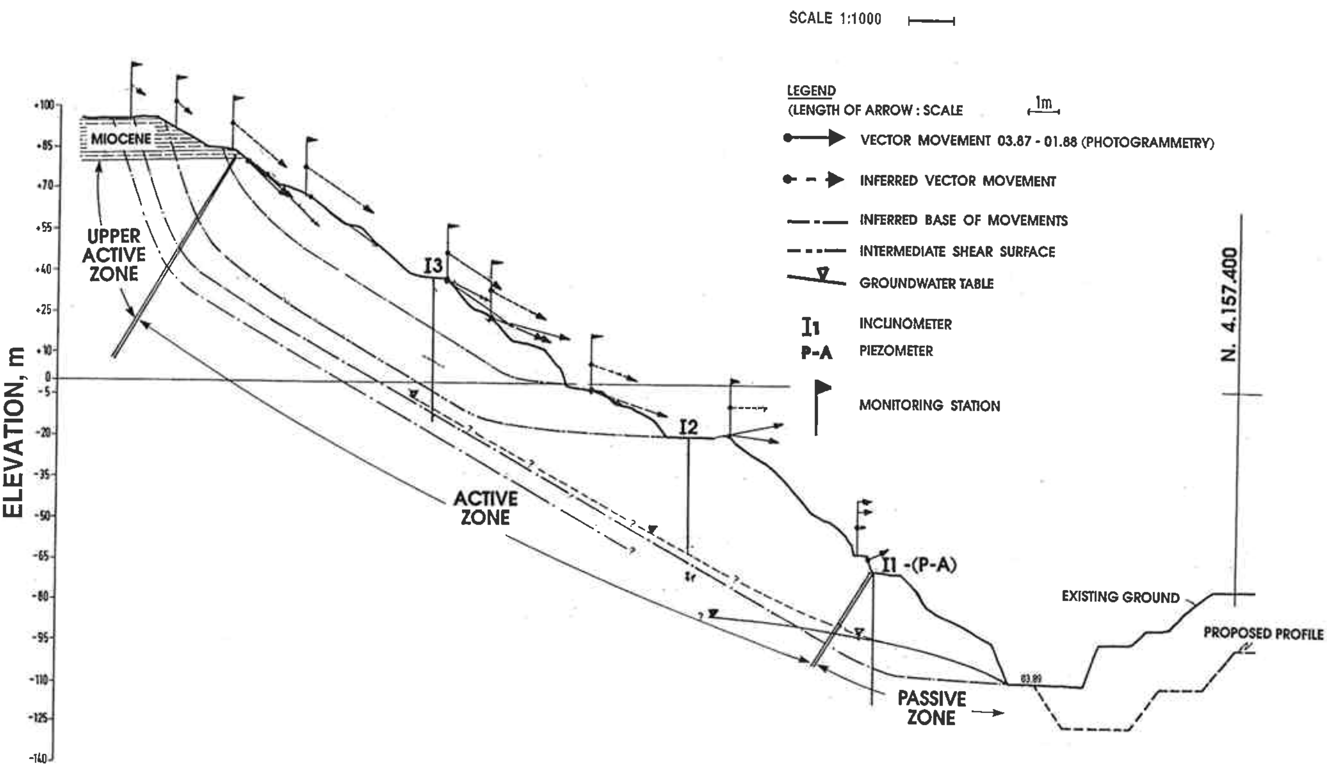

Fig. 9 Schematic cross-section of footwall slope at Aznalcóllar showing inferred stability conditions

Failure mechanism

A schematic cross-section of the pit showing geotechnical conditions and observed displacements is presented as Fig. 9. In the upper parts of the slope displacement vectors are, essentially, downwards and parallel to the slope. In the lower third of the slope displacements are out of the slope with an upwards component. The main groundwater level lies deep within the Palaeozoic sequence and generally below the main part of the sliding mass, but a shallower, transient surface probably develops in response to storm events. However, horizontal drains, typically 30–50 m in length, installed in the upper levels of the slope have been mostly dry, although some drains closer to the toe do carry water. The failure mechanism is believed to be primarily translational along a surface that is sub-parallel to the slope, but with a flatter surface towards the base providing a passive toe that is moving sub-horizontally into the pit. The slope can be considered as comprising three zones, as described below.

Upper active zone

The top of the active failure zone is governed by the tension cracks that extend along the crest-line of the pit. Field observations indicate that the tension cracks dip at about 70–90° into the pit and are closely related to the dominant discontinuities that run parallel to the cleavage.

Active zone

The main part of the failure surface is aligned sub-parallel with the slope. The inclinometer data indicate the presence of a number of failure surfaces at depths of less than 50 m below the slope. These are considered to represent failure zones that have developed progressively as the pit was deepened, the first significant surface being established in 1983. No significant displacement gradient with depth is noted from the inclinometer data and movement has been primarily by planar translation.

Passive zone

The lowest part of the failure surface is flatter than the active zone, providing a mechanism for the surface to exit at the toe of the slope. Displacement gradients were recorded in the inclinometers that have been installed in the lowest section of the slope. These indicate that there has been some rotation of the rock mass above the failure surface.

Although the tension cracks in the upper part of the slope appear to be related to the cleavage direction, neither the active nor the passive zone can be related to any specific geological structural controls. Clearly, the major discontinuity sets are not daylighting in the footwall slope and, therefore, kinematic sliding on lithological boundaries, cleavage or joints is not expected to be a controlling factor although the possibility of adverse, unsampled discontinuities cannot be ruled out.

The influence of water pressure on stability is not clear. The first major movement occurred after heavy rainfall and others since then have also been associated with heavy rain. However, some subsequent major displacements have occurred during dry periods. The implication is that the slope is generally close to limiting equilibrium, which may be disturbed by heavy rain, by changes in the geometry of the pit or simply owing to the progressive development of internal failure mechanisms.

The slope is difficult to analyse conventionally using a limit-equilibrium approach because of the lack of apparent structural control and the difficulty of determining representative mass-strength parameters, particularly where the anisotropy of the mass is a significant but not a controlling factor. In these circumstances numerical modelling offers an especially attractive alternative approach for investigation of the nature of the failure and the factors that control behaviour.

Numerical modelling of slope behaviour

Numerical modelling of the unstable, north-facing footwall slope at Aznalcóllar was performed with UDEC to explore the influence of various assumptions with respect to geomechanical properties and approaches to modelling.[52] The models of McCullough[52] were revised and rerun for the purposes of the present contribution.

The basic model expressed the two-dimensional cross-sectional geometry of the footwall slope. The model was composed of blocks whose outlines were defined by the geometry of the main set of northerly dipping but non-daylighting cleavage-parallel joints, which are thought to be the main control on behaviour, together with a set of steeply dipping release joints. Horizontal stresses were assumed to be equal to vertical (gravitational) stresses. Intact rock was modelled as elastic–plastic with Mohr–Coulomb failure criteria. Discontinuities were similarly modelled as elastic–plastic, with shear strengths defined by Mohr–Coulomb criteria. In the absence of site-specific data reference was made to the work of Brown and co-workers[53] regarding possible friction angles for the predominantly slaty rock. These authors indicated that the friction angle for relatively smooth-textured discontinuities through slate can range from about 20° in wet conditions to 29° when dry. In this case, where the influence of infiltration on stability has clearly been significant on some occasions but apparently not on others, models were run using both values. Similarly, to explore the influence of pore pressures runs were carried out with the slope dry (but using low and high friction angles) and fully saturated. The parameters that were employed in the various models are listed in Table 2.

Table 2 Properties assigned to intact rock and discontinuities in UDEC models

Parameter	Value
Intact rock	
Density, kg/m^3	2649
Young's modulus, GPa	7.08
Poisson's ratio	0.208
Friction angle, degrees	40
Dilatation angle, degrees	10
Cohesion, MPa	1.0
Tensile strength, MPa	1.0
Discontinuities	
Friction angle, degrees	20 (wet), 29 (dry)
Dilatation angle, degrees	3
Cohesion, MPa	0
Tensile strength, Mpa	0
Normal stiffness, GPa/m	10.0
Shear stiffness, GPa/m	5.0

The results presented in Fig. 10 are for a dry slope but using the lower-bound friction angle for discontinuities (i.e. surfaces wet but without pore pressure). Fig. 10(*a*) shows the whole slope; Fig. 10(*b*) and (*c*) show details of behaviour in the middle zone and towards the toe of the slope. The zone of rock that is indicated as undergoing significant displacements in Fig. 10(*a*) (darker area) agrees quite well with the observed extent and depth of failure in the field. Local displacement vectors also correspond quite well to field observations. Other models (dry, high ϕ; wet, low ϕ) indicated essentially the same mechanisms but different maximum displacements. The broad correlation between field observations and model behaviour allows some confidence that mechanisms are being dealt with correctly. The model therefore provides a basis for exploring the sensitivity of the slope to changing conditions (continued mining or a change of geometry or drainage).

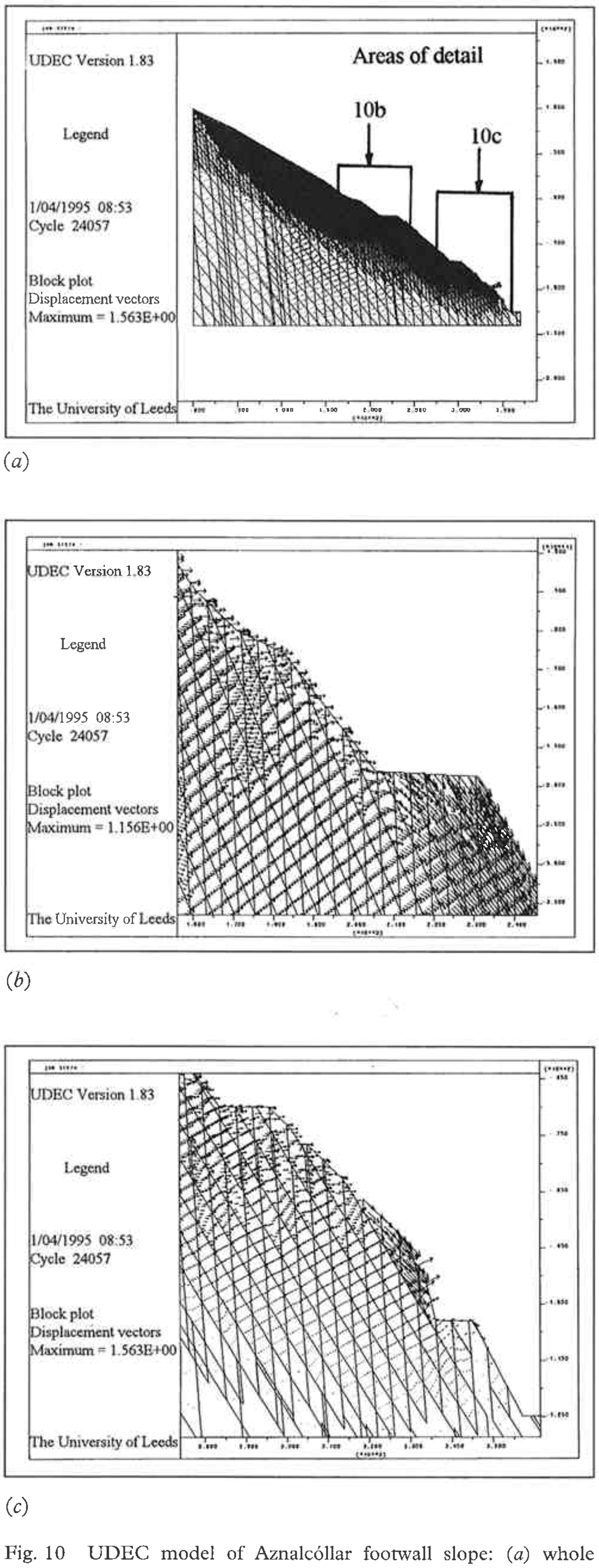

Fig. 10 UDEC model of Aznalcóllar footwall slope: (*a*) whole slope, maximum displacement, 1.563 m; (*b*) detail of mid-section; (*c*) detail of toe zone

Conclusions

Until recently it has been impossible to analyse complex geological conditions realistically; the approach has necessarily been essentially empirical, with design by precedent an attractive option. The development of sophisticated software and the availability of cheap, powerful computing facilities mean that the numerical analysis of complex conditions is a real possibility for many sites. Definition of a realistic geotechnical model and the assessment of representative engineering parameters remain as problematic as ever, however, and the codes themselves are not yet as user-friendly as might be hoped. Nevertheless, they represent a real advance in the options open to engineers. They also offer the possibility for scientific study of some of the basic unknowns in rock engineering.

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