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Developments in the analysis of footwall slopes in surface coal mining

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Abstract

Surface mining of coal, particularly in areas of mountainous topography, often involves the formation of extensive footwall slopes parallel to the strata dip. Due to structural deformation steep dips on the limbs of folds may be encountered in association with thrust faults, jointing and residual shear strength conditions. Such an environment necessitates a rigorous assessment of footwall stability in order to ensure safe and economic exploitation of the coal. This paper provides a detailed review of the factors influencing footwall slope instability in surface coal mining and the major instability mechanisms which have been recognized within the published literature. The analysis of footwalls in the design stage and the back analysis of footwall slope failures has in general been undertaken using predominantly two-dimensional limit equilibrium techniques often incorporating a simplistic elastic column analysis. The application of numerical modelling techniques to surface coal mine footwalls has received little attention. In this paper the authors attempt to illustrate the potential for investigating footwall failure mechanisms and stability using principally the distinct element method. © 1997 Elsevier Science B.V.

Keywords: Analysis of footwall slopes; Surface coal mining; Mountainous topography; Two-dimensional limit equilibrium technique

1. Introduction

Footwall slope instability is a relatively common occurrence in surface coal mines situated in areas of steeply dipping strata particularly where mountainous topography exists. These slopes are often cut parallel to dip in synclinal and anticlinal structures resulting in slope lengths which are extensive both along strike and down-dip. With mining progression down-dip, the exposed footwall slope length increases thereby resulting in potential instability problems in the thinly bedded strata. Footwalls often require careful monitoring of the slope movements using conventional survey and borehole techniques. The importance of footwall stability has long been recognized in Europe (Walton and Atkinson, 1978; Walton and Coates, 1980; Scoble, 1981; Stead and Scoble, 1983; Stead, 1984; Serra de Renobales, 1987). The factors influencing footwall slope failures in the UK have been analyzed by Scoble (1981), Cobb (1981) and

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Table 1
The factors influencing the stability of surface mine footwall slopes

Factors	Influence
Slope geometry	
Slope height	Increases driving forces. Increased stresses/tensile failure in convex slopes. Increased plastic deformation at slope toe with increasing height. Three-hinge buckling stability in rigid planar slabs increases with height. Permeability of slab may decrease with increase in slope height due to orthogonal joint closure. Increased probability of adverse structure daylighting in slope face.
Slope angle	Increased slope angle results in decrease in factor of safety. Higher the slope angle the more adverse structures may daylight. Increased rockfall problems.
Slope curvature	Increased slope sectional curvature, e.g., rolls, the greater the probability of failure, particularly buckling. More tensile failure allows basal release of blocks in roll region. Plan curvature may introduce complex footwall failures, with varying degrees of stability within one footwall.
Slope length	The greater the slope face length in the dip direction the higher the probability of buckling. The lower the slope length in the strike direction the more important the influence of 3-dimensional restraint.
Structure	
Strata dip	Varying dip results in rolls and convex slopes and thereby promotes buckling/basal release of slabs. Footwall failures may occur in slopes with strata dips as low as 17° but are more common with dips between 25 and 45°.
Joint set no.	Highly jointed toe regions may fail releasing upper slab. Increased jointing may reduce rigidity of slab and promotes complex failures.
Orientation	Joints dipping out of the slope increase the potential for bilinear slab failure, whereas joints dipping into the slope may promote ploughing instability. Sub-vertical joints control potential for three-hinge buckling.
Joint spacing	Stability decreases with reduction in spacing (increase in the slenderness ratio L/T). Euler buckling may be limited to slopes where the joint spacing parallel to slope face is less than 1 m. Slabs generally range in thickness from 0.3 to 10 m. Thicker slabs may consist of multiple slabs exhibiting bulging and chevron morphology during deformation. As the spacing of joints orthogonal to the slope decreases the potential for Euler buckling declines and mechanisms involving ploughing and/or three-hinge movement become more likely. Reduced joint spacing may also increase the permeability of the slope and reduce the build up of pore water pressures. Increased joint spacing may have the effect of increasing driving forces and the potential for higher water pressures.
Type	Numerous structures have been observed to influence instability adversely, including thrust surfaces, bedding plane shears, folding, and sedimentary features (rolls, cross-bedding). Thrust surfaces may provide the toe release for bilinear failure or be involved in ploughing. Bedding plane shear have low shear strengths (residual) and thus alow increased active forces increasing the potential for footwall translation-induced failures. Sedimentary structures may allow toe release or promote adverse slope curvature.
Material	
Lithology	The lithology of footwall slopes may include rocks which at imposed stresses behave in a brittle manner such as sandstones and limestones, in addition to more plastic and time-dependent materials including mudstones and schists. In areas of intense shearing, materials may be transitional between soil and rock in behavior, exhibiting distinct plastic behavior.
Geomechanics	There may be a wide variation in the geomechanical properties of the materials. In conventional limit equilibrium analyses the critical parameters are the shear strength properties of the discontinuities. The deformation of the slope is important and hence the deformation moduli and the joint stiffness values are significant in numerical modelling of the slope.
Water pressure	
	Water pressures may play a significant role in promoting three-hinge buckling in planar slabs and in the toe region of failures. Many failures have been reported in dry slopes. Freezing/thawing of snow has been correlated with periods of increased movement in footwall instabilities (Stacey et al., 1990).
Time	
	Failure may be rapid with little warning or slow time-dependent mechanisms (Li and Zhang, 1990). Despite the cyclic deformation observed in some slopes, little attention has been given to progressive failure mechanisms.
Man	
Blasting	A possible factor in several footwall failures (Stacey et al., 1990; Scoble, 1981).
Old workings	May promote buckling or promote increased deformation within a slope leading to a reduction in the rock mass quality.

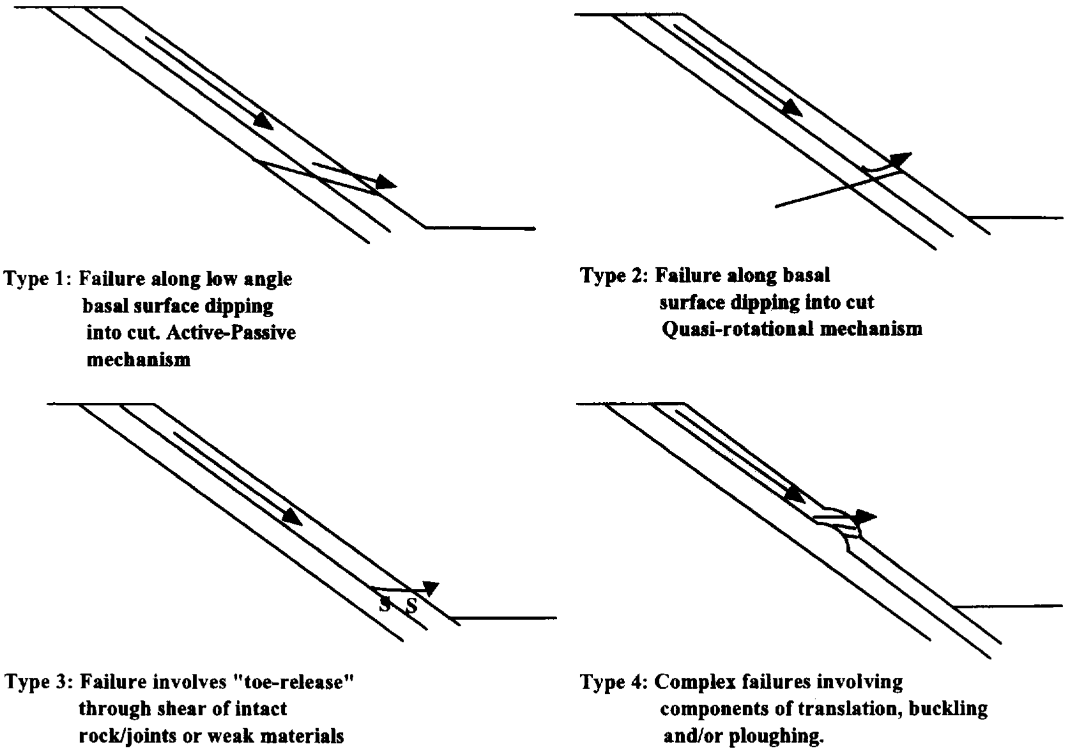

Fig. 1. Mechanisms of bilinear slab failure in footwall slopes.

Stead (1984) as part of a research project investigating rock slope stability in British surface coal mines. In North America, Brawner et al. (1971), Cavers (1981), Cavers et al. (1986), Hawley et al. (1986), Stacey et al. (1990), Dawson et al. (1993) and Zurowski (1995) have all described footwall failure mechanisms. Table 1, based on a survey of published literature on surface coal mine footwall slopes, summarizes the key factors effecting stability.

The use of numerical modelling in the analysis of underground mine and civil engineering excavations has, with the availability of increased desktop computing power, become routine practice. In contrast, surface mine design continues to be undertaken using predominantly deterministic and to a lesser extent probabilistic limit equilibrium techniques. Without doubt, these methods have generally proven successful. In some situations, however, where the failure mechanism is complex and uncertain the use of numerical modelling techniques provide an additional tool which can easily be incorporated in routine mine design. In this paper, the authors demonstrate the potential application of numerical modelling techniques to the analysis of surface coal mine footwall slopes.

2. Mechanisms of surface coal mine footwall instability

The predominant footwall failure modes encountered in western Canadian footwalls have been summarized by Hawley et al. (1986), Cavers et al. (1986) and Dawson et al. (1993). In European surface coal mining practice Walton and Atkinson (1978), Walton and Coates (1980), and Serra de Renobales (1987) have similarly discussed footwall failures. The major footwall failure mechanisms recognized include bilinear slab, ploughing, buckling, step-path, planar and old workings collapse. Details of relevant limit equilibrium methods of analysis are presented in Hawley et al. (1986) and Cavers (1981). Minor and secondary footwall

Fig. 2. Plate showing footwall failure at a Western Canadian coal mine.

failure modes such as circular, wedge, rockfall, and secondary toppling have also been documented. This paper will concentrate on the major failure mechanisms which are to a large extent characteristic of footwall slopes.

2.1. Bilinear slab failures

Bilinear slab failures form the most common mechanism in UK footwall failures and have also been described frequently in the North American literature. Some of the earliest work on bilinear slab failures was undertaken by Brawner et al.

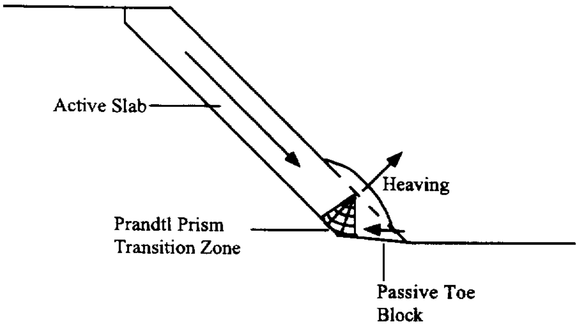

Fig. 3. Location of the Prandtl prism in a bilinear footwall failure.

(1971) in the East Kootenay region of British Columbia, Canada. He recognized and proposed methods of analysis for bilinear slab failures involving bedding and low angle thrust planes. Stability charts were derived for estimating the allowable footwall slope height as a function of bedding angle, bed thickness, cross-over joint orientation and friction angle. Brawner's analyses assumed that the slab was homogeneous, isotropic, and behaved in a rigid translational manner. Fig. 1, based both on a survey of UK footwall failures and a review of the relevant literature, illustrates variations of the bilinear slab failure mechanism. The bilinear failure surface usually comprises an upper failure surface either along bedding or along a major structural plane of weakness. The lower failure plane or cross-over surface may be subdivided according to its orientation into: (1) *Type 1*, low angle daylighting thrust planes, joints or bedding planes; (2) *Type 2*, low angle planes dipping away from the excavation; (3) *Type 3*, intact rock shear/jointing at the slope toe; (4) *Type 4*, mixed bilinear/buckling failure modes.

Type 1 bilinear slab failures are probably the most common variety of failure mechanism involving an active–passive wedge failure mechanism. These failures have been analyzed using biplanar

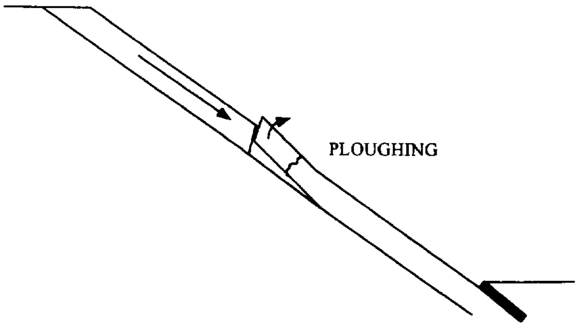

Fig. 4. Mechanisms of ploughing slab failure in footwall slopes.

active–passive wedge limit equilibrium techniques developed by Sultan and Seed (1967), Sarma (1973, 1979), Mochalov (1975), Stimpson and Robinson (1982) and Coulthard (1979). Conventional multiplanar limit equilibrium slope analysis methods, such as Janbu (1973), Morganstern and Price (1968) and the Generalized Method of Slices (Fredlund and Krahn, 1977), are also often used. In general, sensitivity analysis approaches have been adopted to investigate the effects of varying slope height, slope angle, groundwater and shear strength parameters on stability and to determine support requirements or remedial measures that might be necessary. Three-dimensional analyses of footwall slopes using the method of columns have been undertaken using the CLARA program, (Hungr, 1992) and may indicate a 30–40% increase in the factor of safety relative to conventional two-dimensional analyses.

Zurowski (1995) describes a footwall translational failure at the Line Creek Mine in British Columbia, Canada. This failure involved a 7.5–9 m thick sandstone slab up to 300 m in height. The dip of the footwall varied from 10° near the toe up to 50° owing to a pronounced bedding roll. Fig. 2 shows the footwall failure which was progressive in nature, occurring over a period of 3 days. Minor rockfalls preceded the main slab failure by 2 days. Post-failure geotechnical investigations in the area of the roll indicated that the failure surface involved a combination of bedding and thrust faulting. In the case of high slopes, sufficient driving forces may be mobilized in order to enable failure either by sliding along a cross-over surface up dip and/or to promote failure by intact rock shear at the toe of the slope (Types 2 and 3). This may be particularly important where the upper failure surface is along a low shear strength clay seam and high groundwater pressures exist. Results of preliminary numerical modelling also indicate that mixed failure modes may exist in high slopes exhibiting rolls where the slab fails by a combination of buckling and sliding. It is possible that the development of a buckle or bulging in a footwall due to translation of the main slab may result in tensile/shear fractures and the formation of a cross-over surface. The Type 4 bilinear failure may represent such a complex failure process. Dawson et al. (1993), after undertaking limit equilibrium back-analyses of the Upper East Pit footwall failure, Smoky River Coal, Grande Cache, Alberta, suggested that a bilinear slab failure was probably the critical failure mechanism. They noted however that the actual failure

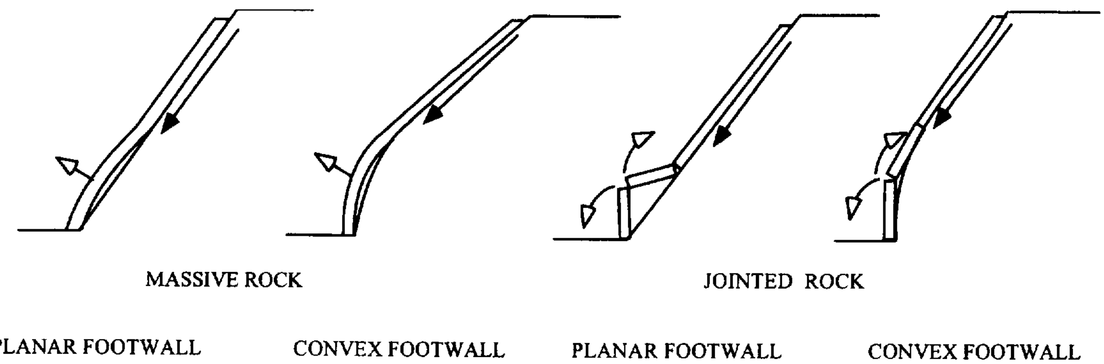

Fig. 5. Mechanisms of buckling slab failure in footwall slopes.

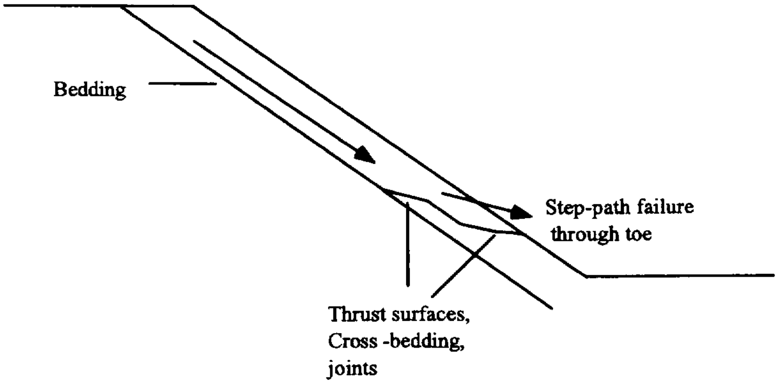

Fig. 6. Step-path failure in footwall slopes.

mechanics were very complex involving surface slabbing due to ploughing and a complex development of shear zones near the slope toe. The possibility of a Prandtl-wedge (Kvapl and Clewes, 1979) forming in a footwall slope has been considered during investigations of UK footwall failures. In this mechanism a zone in between the footwall slab and the toe block (above the potential cross-over surface) is assumed to undergo a bearing-type failure. This zone referred to as the Prandtl-wedge, transmits the driving forces of the upper footwall slab to the toe block, Fig. 3. Bulging in the footwall slope area above the Prandtl zone may precede failure.

Insufficient published data exists on footwall slope failures to allow a rigorous statistical analysis to be undertaken. A limited analysis of data collected on UK bilinear slab failures, by Stead (1984), showed the failure height to range from 15 to 92 m with the majority being less than 40 m. UK bilinear slab failures were found to retrogress behind the slope crest to normally less than 10 m with a characteristic range of 3–14 m. The overall slope angle varies between 19 and 40° with a calculated mean of 31°. The length of the bilinear slab instabilities ranges from 42 to 250 m with a mean of approximately 130 m. The recorded strata dip is a critical parameter in the assessment of slab failures and in UK slope failures this ranges from 10 to 45° with a mean of 32° into the excavation. Failure in all UK cases involved low shear strength horizons comprised of mudstone strata, bedding plane shears (claybands) and basal cross-over surfaces.

2.2. Ploughing

Ploughing footwall failures were described by Walton and Coates (1980) who undertook parametric studies using base friction models. Failure occurs due to sliding along the footwall bedding planes with toe release by a ploughing action along steeply inward-dipping joints sub parallel to the slope. The passive toe block is lifted and rotated

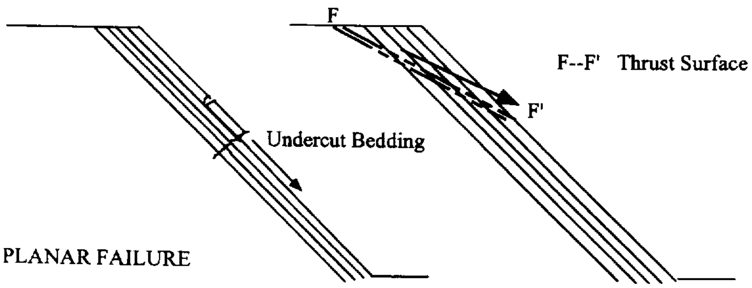

Fig. 7. Planar failure mechanisms in footwall slopes.

out of the footwall due to movement of the active or driving footwall slab (Fig. 4). Ploughing failure mechanisms have been discussed in North American practice (Dawson et al., 1993), and limit equilibrium methods of analysis presented (Hawley et al., 1986). However, Cavers et al. (1986) previously commented on the lack of observation of this type of failure in western Canada but note that this mechanism could be a problem high up on a footwall face where a large amount of movement could occur. At the toe of the footwall they speculated that an active slab moving by ploughing may lock up due to kinematic restraints placed on the ploughing toe block. Dawson et al. (1993), after a comprehensive evaluation of footwall failure mechanics in western Canadian mountain coal mines, note the occurrence of ploughing failures and suggest that in general such failures are limited to slabs less than 5 m thick, the critical plough height being 40–60% of the total slope height. Serra de Renobales (1987) refers to the occurrence of footwall failures exhibiting a ploughing mechanism in opencast bituminous coal mines in the Central Asturian Carboniferous Basin in Spain. This ploughing mode of footwall failure is particularly amenable to analysis using the distinct element method as will be discussed later. It is suggested that this mechanism may be an underestimated component of complex translational footwall failures.

2.3. Buckling

Buckling failures have been reported in slopes formed parallel to steeply dipping strata both in natural slopes and surface coal mines. Fig. 5 illustrates varieties of buckling instability recognized in planar and curved footwall slopes. Scoble (1981) described a major buckling failure at the Westfield Open pit coal mine which occurred in 1974 resulting in a fatality. Limit equilibrium methods of analysis of footwall buckling failures have been described by Walton and Coates (1980), Cavers (1981), Cavers et al. (1986), Serra de Renobales (1987) and Choquet et al. (1993). These methods commonly involve a combination of limit equilibrium and Euler column theory. The driving or disturbing forces for the given footwall slope geometry and geology are calculated assuming representative shear strength properties for the bedding planes which forms a potential sliding surface. To obtain a factor of safety for the footwall, the driving forces are compared with a calculated critical buckling load. The critical buckling load determination is based on the geometry of the footwall slab (length and thickness), and assumptions regarding the elastic modulus of the slab and its end conditions (usually pinned). The relevance of this method to natural slopes has been questioned on several occasions. Varying rock mass conditions and, in particular, jointing require a more sophisticated analysis.

Cavers and co-workers (Cavers, 1981; Cavers et al., 1986) describe two methods of analysis of buckling failure involving planar or curved discontinuity-bounded slabs. The principle of three-hinge buckling was proposed using force and moment equilibrium equations to assess the stability of a slab cut by three orthogonal cross-joints. The stability of each of the two blocks involved is

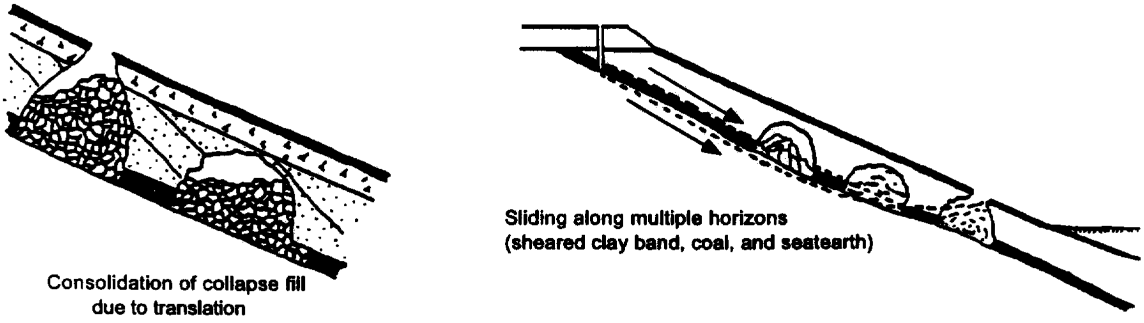

Fig. 8. Footwall failure involving collapse of old workings, after Walton and Coates (1980).

determined and the required interblock force for equilibrium calculated. The individual blocks are assumed to behave in a rigid manner. Cavers (1981) notes that the upper block must yield to provide space for block rotation. The equilibrium equations therefore require modification to reflect crushing at the block corners. The three-hinge analysis limit equilibrium computer program, CURVBUK, has been used to investigate the stability of convex footwall slopes by Cavers et al. (1986) and Choquet et al. (1993). Soukatchoff et al. (1991) and Choquet et al. (1993) describe the analysis of the strata buckling mechanism at the Grande-Baume coal mine in France. At this mine a 250 m high footwall with an adverse bedding dip of 40° was planned. They analyzed the slope using a combination of physical modelling (friction table) and three-hinge buckling limit equilibrium analysis. A series of design charts applicable to the stability analysis of slopes against three-hinge buckling were produced. These charts, derived from CURVBUK analyses illustrate the critical slope height and length for given bed thickness, dip and assumed friction angle. Cavers (1981) suggests that failure by elastic buckling of beds according to Euler theory is rare unless very thin beds exist. Dawson et al. (1993) state that pure flexural Euler buckling does not appear to be a realistic failure mechanism and go further to emphasize that this method should not be used to predict maximum stable footwall slope height. Euler type buckling may have some relevance in the weathering of exfoliated rocks such as in the Sierra Nevada Range, California, where buckling

Table 2
Key input parameters and property values used in both the finite difference and distinct element numerical modelling studies

Model parameter	Input property
Intact material	
Density, ρ	2100–2400 kg m^{-3}
Bulk stiffness, K	8–13 GPa
Shear stiffness, G	4–8 GPa
or	
Young's modulus, E	10–20 GPa
Poisson's ratio, v	0.25–0.30
Cohesion, c	5–10 MPa
Intact friction angle, ϕ	15–30°
Tensile strength, t	0.5–2 MPa
Discontinuities	
Joint normal stiffness, jkn	10 GPa/m
Joint shear stiffness, jks	1 GPa/m
Joint friction angle, jfric	10–20°
Joint cohesion, jcoh	0.01–0.1 MPa

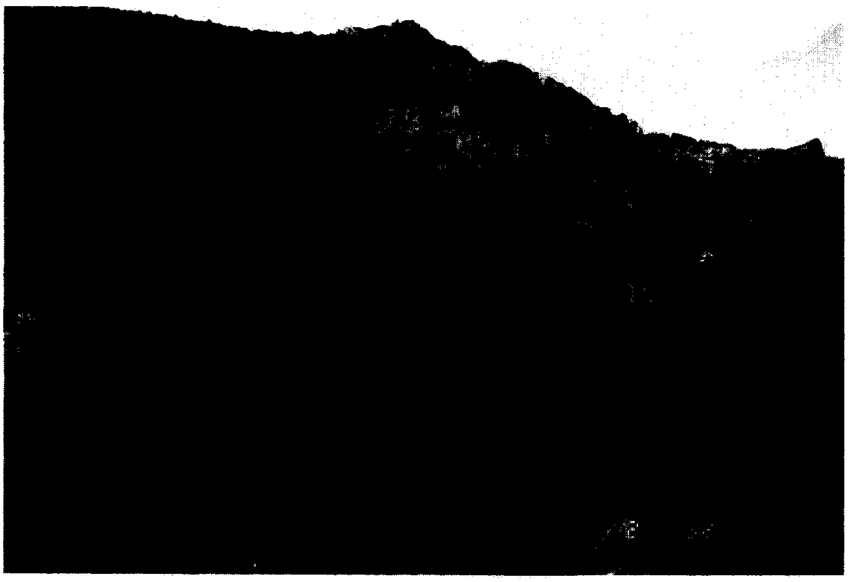

Fig. 9. Plate showing importance of jointing in footwall failures.

of thin plates of granite occurs due to high in situ residual stresses (Watters and Inghram, 1983). Watters and Roberts (1995) describe in situ stress studies on such buckled rocks using acoustic emission techniques and the Kaiser effect.

Corbyn (1978) presented an analysis for determination of stress distribution in a laminar rock at the point of failure. This analysis was applied to the design and back-analysis of rock slopes cut parallel to bedding. The effect of "knots" of curved laminae within the slope were used in the development of the failure theory. Yuan and Sijing (1991) describe a method of elastic analysis for flexural deformation of rock slopes with application to Chinese hydropower stations. Most analyses of buckling failure conducted to date assume a two-dimensional slab, although Watters and Inghram (1983) and Nilsen (1987) used elasticity theory to analyze a buckling plate. The majority of buckling or flexural analyses performed to date have assumed an elastic deformation prior to buckling of the slab or plate. Serra de Renobales (1987), however, describes strata buckling in a footwall slope in Spain where the importance of jointing and, in the case of extensive slopes, "plastification" at the foot of the slope, were recognized. Creep during buckling failure has also been reported in China by Li and Zhang (1990). It is suggested that in extensive footwall slopes plastic and viscoplastic mechanisms may be involved prior to failure.

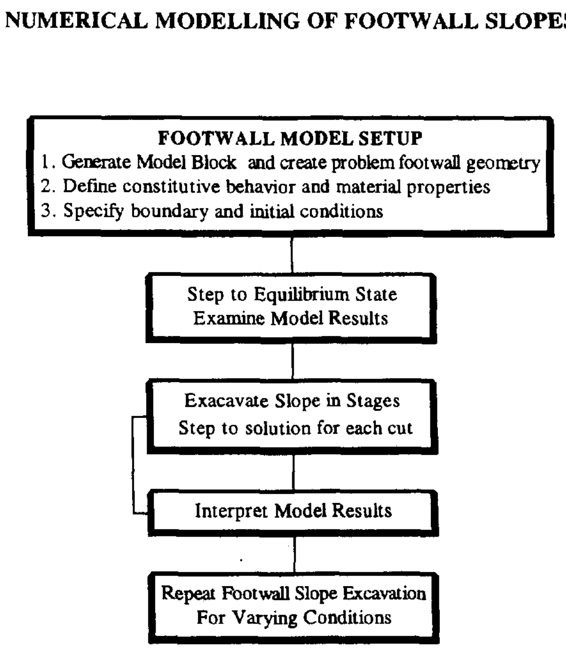

Fig. 10. Methodology adopted in numerical modeling of footwall slopes.

2.4. Step-path

The concept of step-path failure mechanisms has been discussed within the literature for several decades (Jennings, 1970). This type of failure mechanism may involve sliding on varying joint set orientations, separation along sub-vertical joints and intact fracture through rock bridges (Fig. 6). Several major open pit mines have used probabilistic limit equilibrium, step-path analyses in the design process (Read and Lye, 1983). Step-path failures have rarely been reported in the literature in the failure of surface coal mine footwalls, although in an examination of footwall instabilities in western Canadian mines Hawley et al. (1986) note their occurrence. Similarly in a survey of footwall failures in UK surface coal mines jointing frequently controlled the release mechanism at the toe of footwall failures. Distinction in practice between bilinear slab, ploughing and step-path failures may be difficult and this failure mechanism is undoubtedly more important than commonly recognized.

2.5. Planar failure

As most footwalls are formed sub-parallel to bedding, planar failures should be a rare occurrence. A study of UK surface coal mine rock slope failures indicated that planar failures were indeed rarely reported in footwalls. Hawley et al. (1986) and Cavers et al. (1986) comment on planar failures in footwalls where failure occurs due to the daylighting of a fault, along bedding offset by faulting or where a bench is excavated undercutting the slope (Fig. 7). In Canada, such planar failures are reported by Cavers et al. (1986) to be the most common failure mode. This type of failure mechanism is conveniently analyzed using available limit equilibrium techniques. The three-dimensional influence of lateral release surfaces can be investigated using the method of columns limit equilibrium analyses such as CLARA (Hungr, 1992). Such analyses have been used in the stability assessment of western Canadian footwalls. Distinct element models or block theory (Goodman and Shi, 1985) may be useful in predicting key-blocks within the footwall. Windsor and Thompson (1994) developed a block theory program, SAFEX, which may be appropriate in some cases for combined limit equilibrium-block theory analyses of footwall slopes.

2.6. Old workings collapse

Collapse of old workings within footwall slopes was a significant factor in several reported UK footwall failures (Stead, 1984; Walton and Coates, 1980). An example of a footwall with extensive collapse-related instability is shown in Fig. 8. Failure in such slopes is a complex process involving components of both translational sliding and collapse. Limit equilibrium analyses have been modified to assess critical footwall lengths with failure involving buckling of the bedding forming the roof of the mine openings. Indirect effects of old workings on footwall stability may include the reductions in rock mass quality due to mine-induced movements, and changes in groundwater flow and pressures. Modelling of the effects of old workings, void migration and collapse-induced movements are amenable to analysis using distinct element techniques.

3. Numerical modelling of surface coal mine slopes

The application of numerical modelling techniques to surface coal mining has in general received very little attention, the majority of slope stability investigations involving limit equilibrium analyses. Limit equilibrium methods, however, are quite limited especially with respect to the degree of complexity that can be incorporated into the analysis both in terms of problem geometry and material behavior. They allow a static analysis of footwall slabs movement along discontinuities but fail to simulate the intact rock deformation which precedes slope failure. Numerical modelling techniques provide a means to solve more complex problems involving both sliding along joints and intact rock deformation. The application of numerical techniques to open pit coal mining have mostly involved the use of the finite element method with the assumption of a continuum and simplistic constitutive criteria. The major areas investigated have included spoil pile stability and the effects of subsidence and old workings on surface coal mine slope deformation. More recently, distinct element techniques which allow for the analysis of a discon-

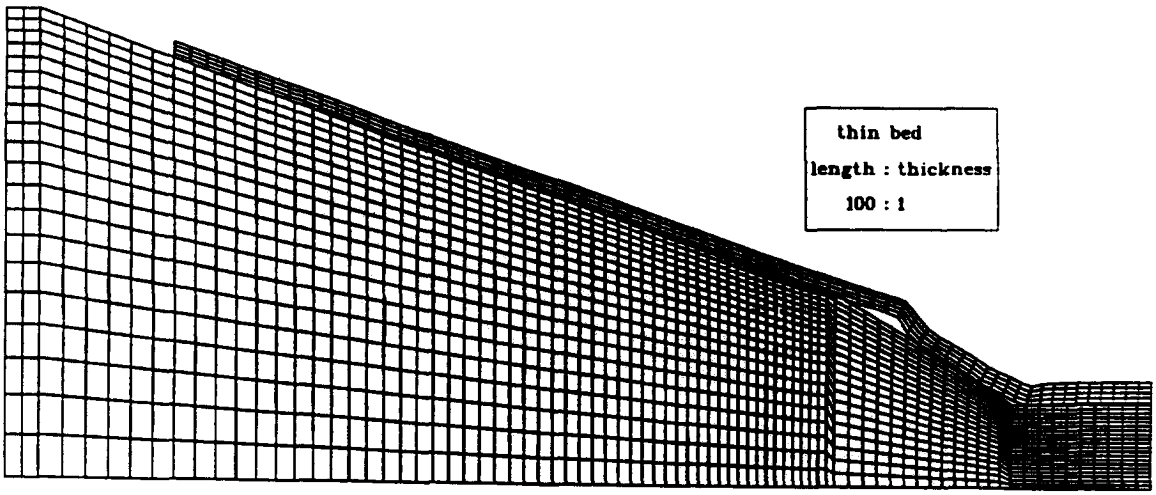

Fig. 11. Deformed mesh of a buckling failure using the finite difference program FLAC (slope height 460 m).

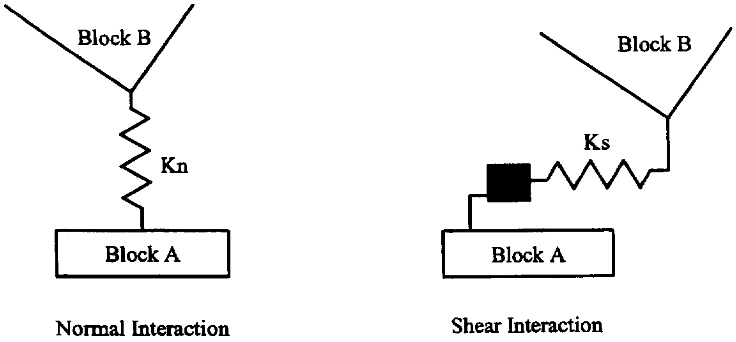

Fig. 12. Representation of joint normal and shear stiffness in the distinct element code, UDEC.

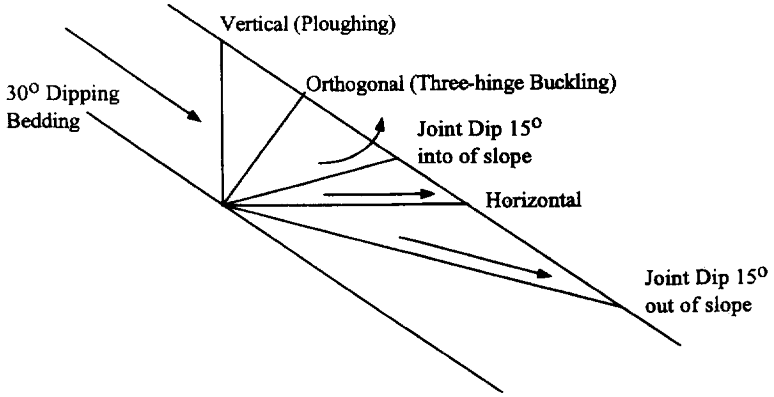

Fig. 13. Variation in orientation of footwall discontinuity sets modeled using distinct element code, UDEC.

tinuous medium have been incorporated to model the deformation and interaction of joints as well as the intact material itself. The analysis presented in this paper deals primarily with the use of the distinct element technique, using the UDEC code (Itasca, 1993b), to model the inter-block joint controlled sliding and rotation of footwall blocks in a thinly bedded slope. This paper also briefly touches on the continuum modelling approach, using the finite difference code FLAC (Itasca, 1993a), to model the deformation of a buckling slab through the use of a number of different constitutive criteria.

3.1. Construction of numerical models of footwall slopes

The relatively simple geometry of surface coal mine footwall slopes makes the construction of a model comparatively straightforward. The degree of complexity that needs to be incorporated in the model dictates the method to be used. It is possible to use two-dimensional finite difference techniques to analyze the stability of a footwall slab which is either massive or cut by discrete planes of weakness. In this case, the bedding plane forming the base of the upper slab and the weakness plane forming the base of the toe block are formed by interfaces along which joint normal and shear stiffness are assumed, in addition to the Mohr Coulomb shear strength parameters. The intact deformation of the slab is modeled by assuming representative values for the elastic constants, intact cohesion, angle of internal friction and tensile strength. These properties can be simply derived by conventional uniaxial compression (with axial and lateral strain measurement) and indirect tensile testing. It is possible to analyze

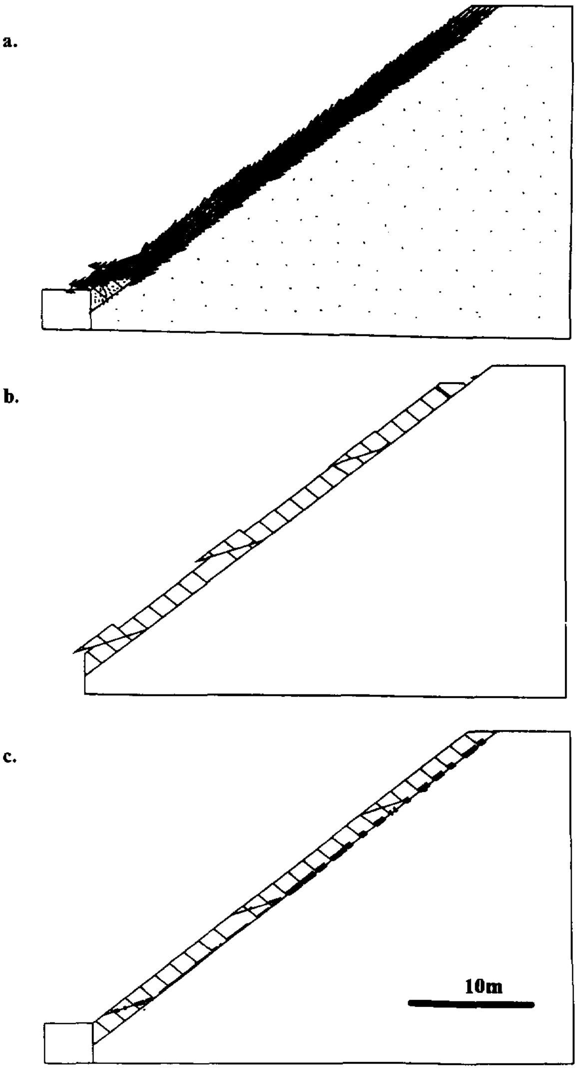

Fig. 14. Distinct element analysis of a UK bilinear footwall instability, planar slope, assuming gravitational stresses. a. Displacements; b. block movement; c. shear.

slabs of varying thickness and to simulate the process of excavation and lengthening of the exposed footwall until a critical height is reached.

Where the footwall slope is known to be well jointed, Fig. 9, then a numerical method which conveniently allows the modelling of joint-bounded block deformations such as the distinct element method must be utilized. In this case additional data is required on discontinuity orientation, spacing and continuity. Determination of material input parameters also becomes more difficult since properties have to be applied to both the discontinuities and the intact material. Table 2 contains a listing of the input parameters required for a Mohr Coulomb constitutive model as well as the typical property values used in both the finite difference and the distinct element footwall slope models (these properties vary depending on the lithology represented in the model, i.e., sandstone, shale, etc.). In terms of sensitivity, the modelling of thin slab footwall failures are highly sensitive to such properties as the joint shear stiffness, joint friction angle and the joint cohesion. These properties dictate the amount of relative movement allowed along the block interfaces. The deformation properties of the intact material itself also plays a large role in the failure mechanism, portraying more realistic footwall movements than those given through the use of rigid blocks. It is also possible using numerical modelling to investigate the effect of in situ stresses, groundwater pressures and excavation sequencing. The simplified flowchart shown in Fig. 10 illustrates the main stages in the construction and execution of a numerical model of a surface coal mine footwall slope. It is apparent that numerous combinations of the input factors will affect the actual potential for failure. An exhaustive numerical modelling study of the factors influencing footwall failure is beyond the scope of the present paper. In the analyses illustrated in this paper the slopes are assumed dry. In practice considerable effort, such as the installation of drainage measures is often exerted to ensure that instability is not promoted by high pore water pressures. The results shown in the following sections are primarily intended to illustrate the ability of the methods to simulate observed failure mechanisms in surface coal mines. Assumptions of water tables have been avoided although both finite difference and distinct element techniques can be used to investigate the effects of water pressures on instability. Similarly, provision exists for the selection of "history" points on the footwall model at which it is possible to estimate changes in displacement and stress. To warrant the use of such facilities, the model must be constrained by available deformation and groundwater data.

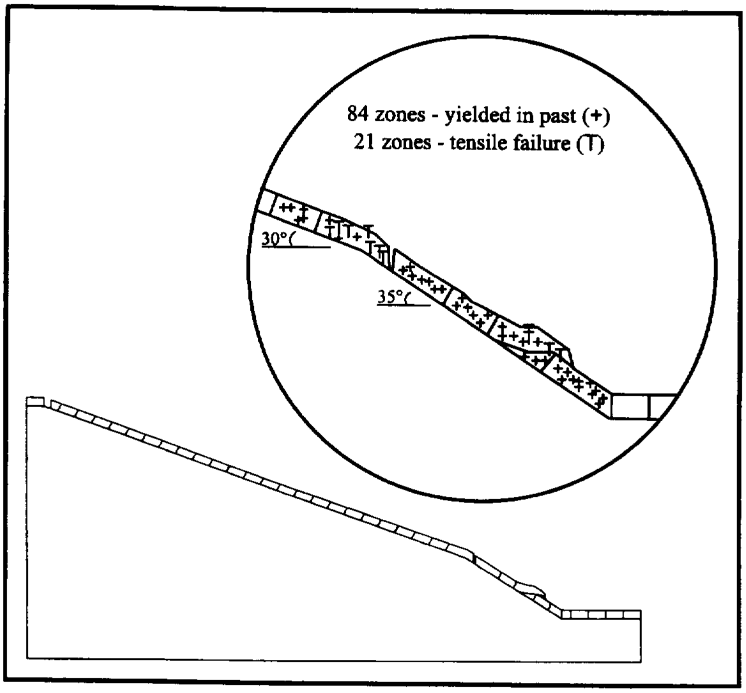

Fig. 15. Distinct element analyses of a bilinear failure, curved slope, gravitational stresses (slope height 460 m).

3.2. Finite difference modelling

The finite difference program FLAC (Itasca, 1993a) can be used to analyze potential failure mechanisms in footwall slopes. Eberhardt and Stead (1993) describe the analysis of continuous footwall slabs using finite difference methods. The excavation of the slope was simulated by removing elements representing overburden in specified lifts. In this manner the length of the continuous footwall slab was gradually increased. The models were also constructed so that the bed thickness and dip could be varied to investigate the effect on stability. The finite difference program, FLAC, offers a wide choice of constitutive models with which to represent the footwall slab. An initial investigation involved the use of an elastic constitutive criterion in an attempt to simulate Euler buckling. The results of this investigation indicated that this form of buckling deformation is not significant in typical planar footwall slope geometries without the assumption of external forces (water or in situ stress). When the geometry of the slope changes, such as at the location of a roll within the footwall, tensile stresses develop. The magnitude of elastic buckling deformations is still minimal and failure requires tensile fracture followed by sliding. To model buckling on continuous footwall slabs more realistically, an elasto-plastic Mohr-Coulomb criterion was assumed. Buckling in planar slopes was still found to be insignificant without the assumption of external forces. With the assumption of a minor roll in the footwall involving a change in dip from 30 to 35°, failure was found to occur at the point where the bedding steepens. Fig. 11 shows the results of a finite difference model for a footwall slope with a roll undergoing buckling failure. Failure initiates once again by tension at the roll in the footwall slope with the slope failing in a translational mode. Predicted buckling deformations are, as would be expected, more significant using an elasto-plastic criterion. Although the finite difference program, FLAC, is capable of modelling discrete discontinuities using "interfaces" along which joint normal and shear stiffness are assumed, modelling of jointed footwall strata is more appropriately undertaken using discontinuum techniques such as

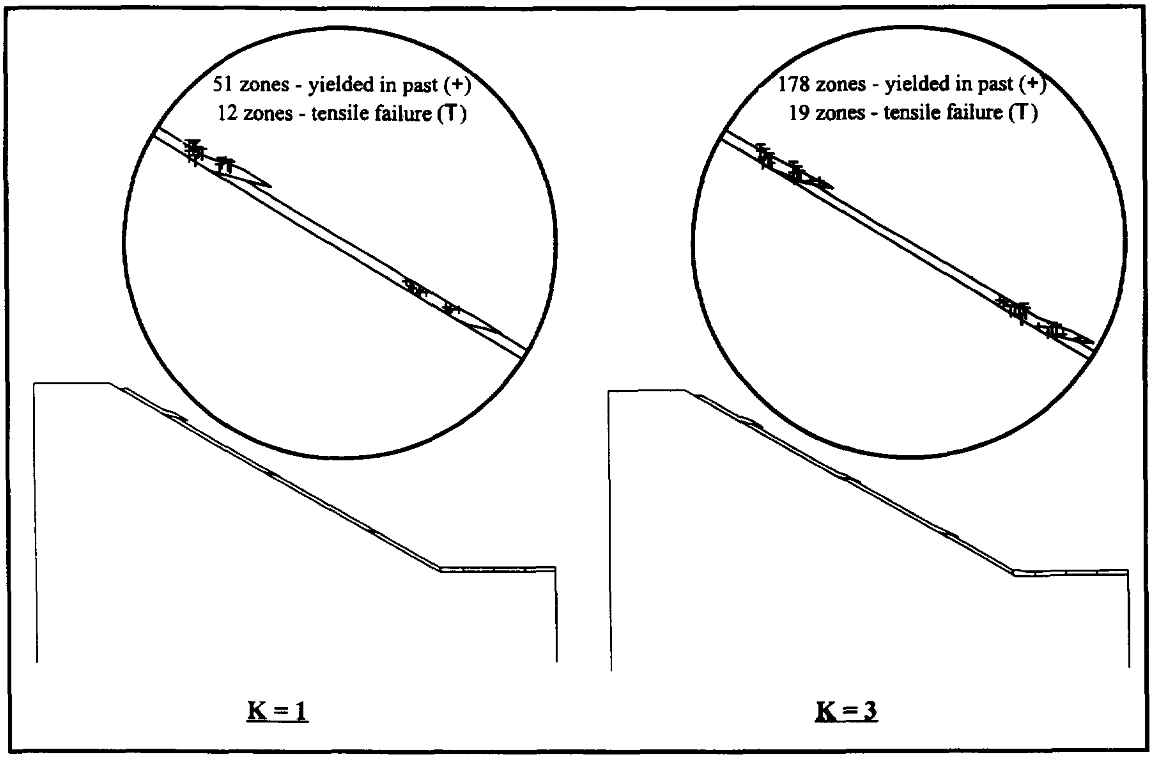

Fig. 16. Distinct element analysis of a bilinear instability, planar footwall slope, assuming (i) $K=1$ and (ii) $K=3$. Slope height 250 m, slope angle 30°.

the distinct element method. An advantage of the FLAC program is however the ability to model time-dependent deformation in footwall slopes.

3.3. Distinct element modelling

The distinct element method developed by Cundal (1971), has significant potential in the analysis of surface coal mine slopes. This technique allows the simulation of jointed slopes using discrete blocks. The blocks may be rigid or discretized into a finite difference mesh thereby allowing deformation. Various constitutive criteria may be adopted to analyze movement along the discontinuities and deformation of the blocks. Deformation along joints is represented by spring-slider systems representing the normal and joint shear stiffness, Fig. 12. The program UDEC (Itasca, 1993b), was used in the distinct element analyses illustrated in this paper. Using this code it has been possible to reproduce all the major failure mechanisms observed in surface coal mine footwall slopes.

Distinct element models have been constructed using typical footwall slope geometries both with and without bedding rolls. The distinct element analysis is also able to simulate a footwall parallel to bedding with varying joint set orientations and spacing. The effect of jointing in the analysis of footwall slope instability is often not fully appreciated. Analyses have been conducted assuming a specified bedding dip and varying orientations for the dominant joint set. Fig. 13 shows the variation in joint orientations modeled, with respect to the footwall slope. The orientation of the dominant joint set has a critical effect on the mechanism by which the slope fails. In our analyses we have investigated both planar and curved footwall slopes. Slope failure, as has been recognized by earlier workers, is without doubt more probable

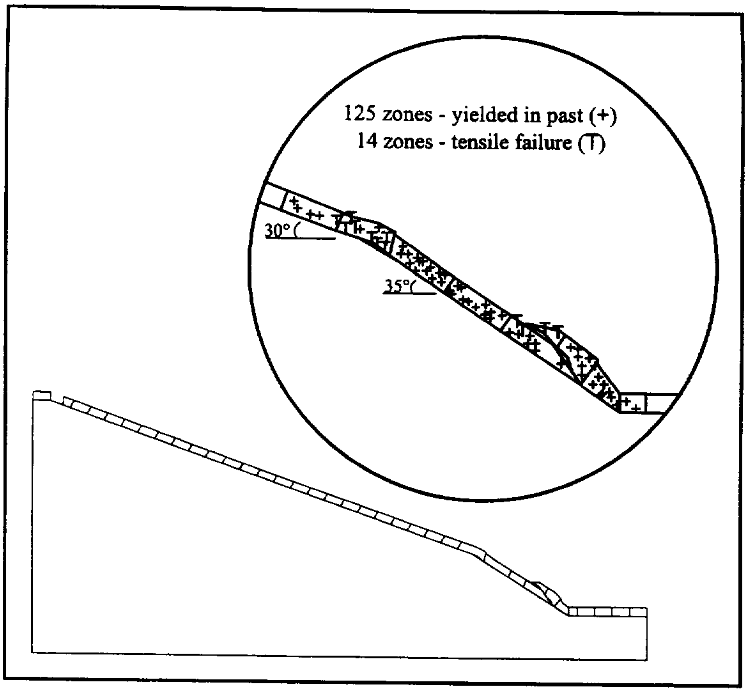

Fig. 17. Distinct element model of a ploughing instability, curved footwall slope, assuming gravitational stresses, slope height 460 m.

in slopes exhibiting curvature. Steepening of the slope allows more structures to daylight into the pit. Most operating mines with footwall slopes occur in areas of significant topography and where tectonic forces have been responsible for mountain building processes. To date, current methods of slope analysis have largely ignored the effects of high horizontal stresses within slopes. Initial numerical modelling analyses undertaken by Eberhardt and Stead (1993) assumed, as is the conventional practice, gravitational stresses within the model slope. In these conservative analyses it was found that curved footwall slopes were significantly more susceptible to bilinear, ploughing and buckling failure than planar slopes. In the majority of cases, planar slopes appeared stable unless extremely low shear strength parameters were assumed. The effects of high horizontal stresses in quarry floors in producing "pop-ups" has been recognized by many authors (Franklin and Hungr, 1978; Quiggley et al., 1978; Roorda, 1995). Preliminary distinct element modelling indicates that high horizontal to vertical stress ratio, *K*, noticeably increase both the potential for instability in footwall slopes and the probability of pop-up phenomena in surface mine floors. Bilinear, ploughing and buckling deformations are all exacerbated by increasing horizontal stresses orthogonal to the strike of the slope.

In order to illustrate the application of the numerical models using both conventional gravitational stresses and horizontal to vertical stress ratio, *K*, greater than unity, analyses using both assumptions are presented herein.

3.3.1. Bilinear models

Bilinear failure modes have been analyzed in both planar and curved footwall slopes. As a mine is deepened, the potential for toeing-out along basal shear surfaces can be investigated. The mine designer can hence obtain an indication of the critical height of the footwall slope at which instability problems might occur. Fig. 14 shows a distinct element analysis of a UK surface mine footwall with potential toe block release along low shear strength thrust planes. Modelling indicates sliding of the slabs occurs progressively as the slope is deepened. Fig. 15 shows an analysis of a

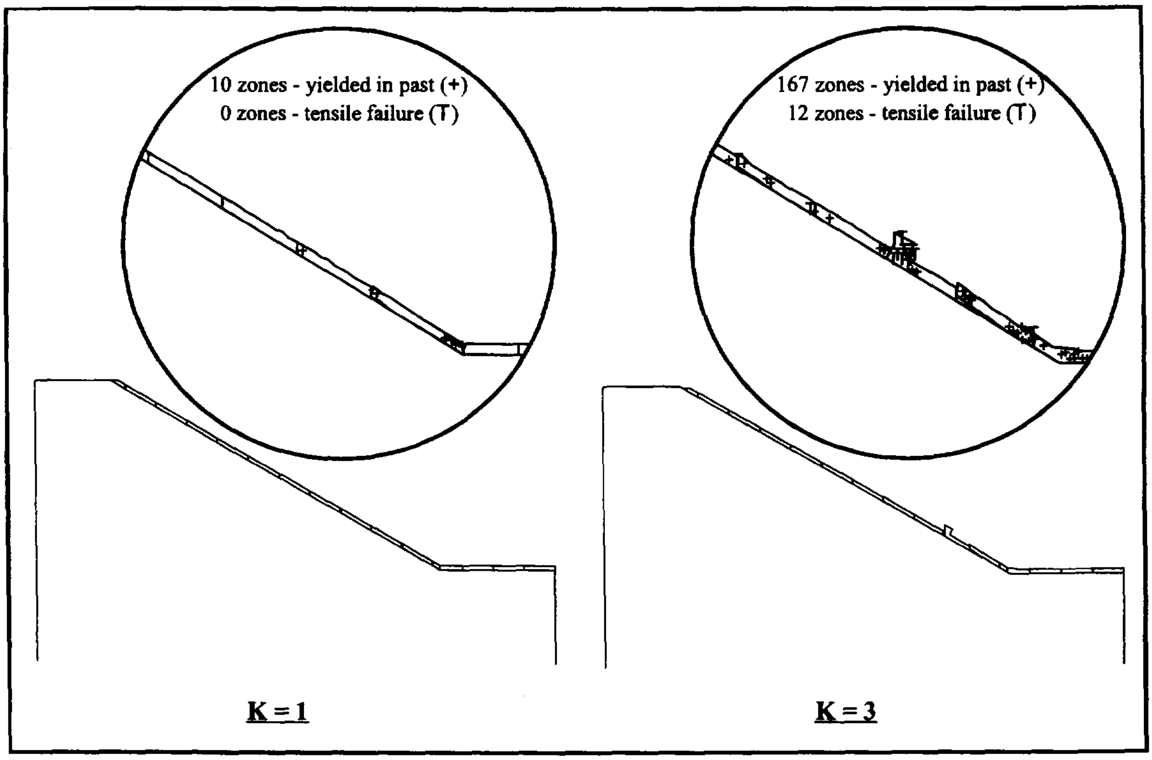

Fig. 18. Distinct element model of a ploughing instability assuming (i) $K=1$ and (ii) $K=3$. Slope height 250 m, slope angle 30°.

typical western Canadian footwall with a distinct roll in bedding. The distinct element model indicates zones of plastic and tensile failure and the development of a bilinear footwall with failure occurring by separation at the roll and toeing out along a sub-horizontal thrust. Distinct element modelling allows the influence of variations in shear strength along bedding and joints to be investigated in addition to the effect of additional disturbing forces such as groundwater or in situ horizontal stresses.

Distinct element modelling results show that an increase in the horizontal to vertical stress ratio, K, from 1 to 3 results in a significant deterioration in stability of the footwall. Fig. 16 shows a distinct element analysis of a planar footwall slope assuming bilinear failure with toeing out along a joint dipping 15° out of the slope. The additional horizontal stresses acting out of the slope can be seen to result in a significant increase in instability.

The presence of joints either orthogonal or sub vertical to bedding can radically effect the potential instability mechanism. Limit equilibrium analyses assume rigid block sliding along translational failure surfaces. Distinct element analyses allow the engineer to incorporate the effects of both normal and shear stiffness on joints. In addition the effects of deformation of the material itself, which may be significant in weak coal measures rocks, need to be considered. Block interaction including the potential for ploughing and buckling can be simulated. Post-processing of the distinct element model results allows zones of plastic failure, tensile failure, joint separation and shear to be highlighted. The mechanism of block movement can be illustrated using displacement contours and vectors.

3.3.2. Ploughing

Ploughing analyses have been undertaken both in the case of curved and planar footwall slopes. Fig. 17 shows a distinct element model of a foot-

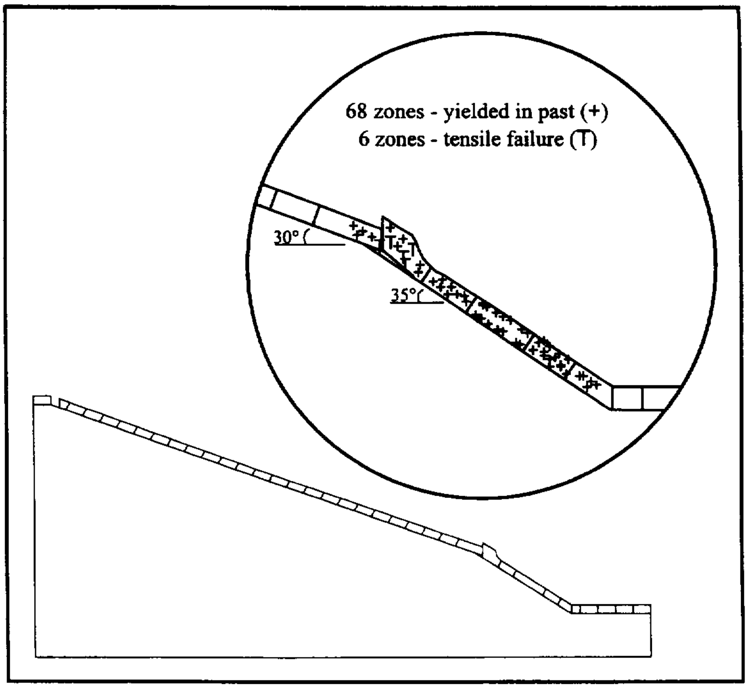

Fig. 19. Distinct element analysis of a buckling failure, curved footwall, assuming gravitational stresses; slope height 460 m.

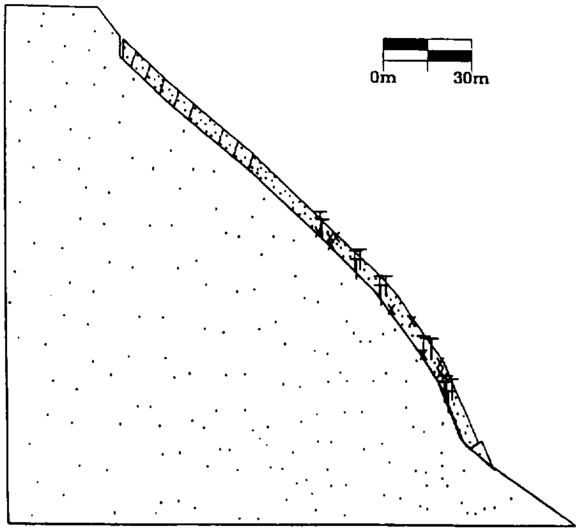

Fig. 20. Distinct element analysis of the Westfield buckling failure of 1971.

wall slope with a roll undergoing ploughing failure. The main discontinuity set is oriented perpendicular to bedding. Above the toe of the footwall a high angle discontinuity is assumed to dip into the slope. Modelling shows that this plane, which could represent a thrust or joint facilitates a ploughing failure mechanism. An analysis of a planar slope dipping at 35° with sub-vertical joints is shown in Fig. 18. The main joint set is oriented sub-vertically. Analysis of this slope with a horizontal to vertical stress ratio, K, of unity did not indicate any significant failure. A horizontal to vertical stress ratio of 3 indicates a ploughing instability with associated zones of tension and yield.

3.3.3. Buckling

The importance of slope curvature in promoting buckling failures has been emphasized by Cavers (1981). Distinct element analyses have been conducted to investigate buckling failure in both planar and curved slopes with and without jointing orthogonal to bedding. Potential footwall instability has been investigated assuming either elastic or elasto-plastic constitutive criteria. Buckling of elastic slabs or unjointed columns does not appear to be a likely failure mechanism unless very thin bed

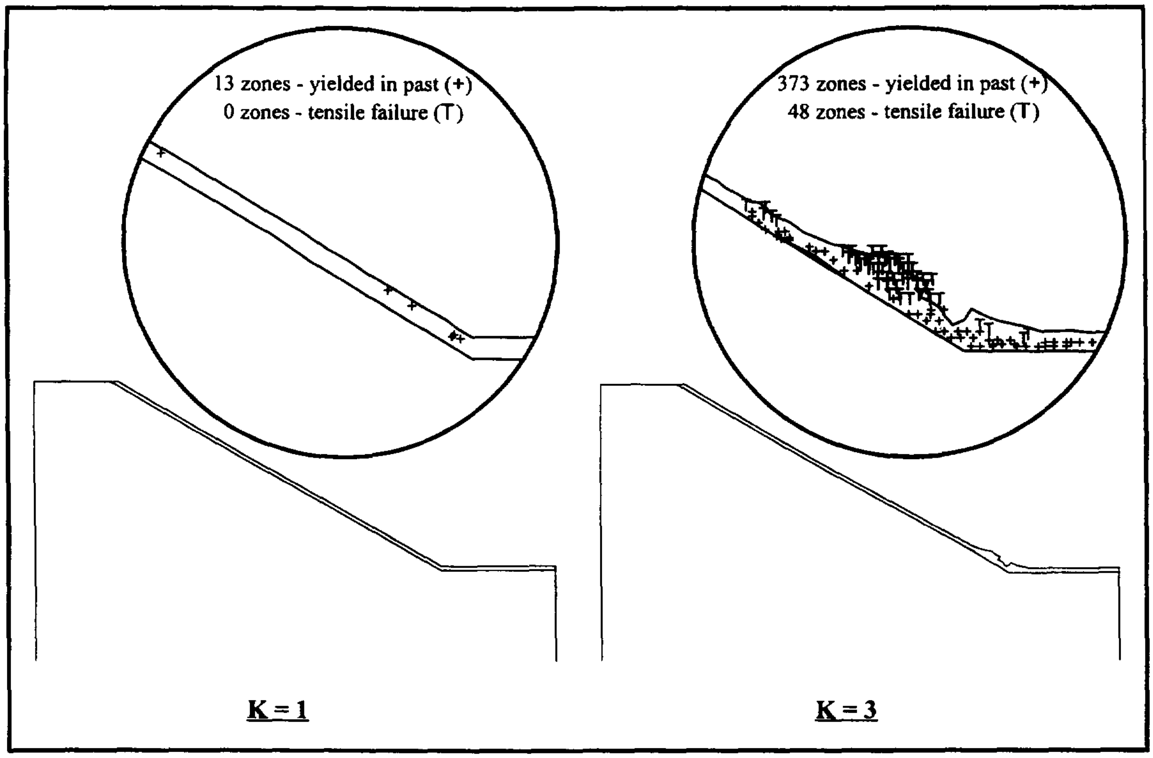

Fig. 21. Distinct element buckling analysis of an unjointed slope assuming (i) $K=1$ and (ii) $K=3$. Slope height 250 m, slope angle 30°.

thicknesses are assumed. This is in agreement with the observations of Cavers (1981) and Dawson et al. (1993). It would appear from the results of modelling that the conventional application of Euler theory/limit equilibrium analysis may be inappropriate. Potential buckling failure in footwall slopes is strongly influenced by: (1) the deformation characteristics of the footwall; (2) the assumed external forces (horizontal stresses or groundwater pressures); (3) the slope profile in section and plan; (4) the jointing orientation and spacing.

The assumption of an elastoplastic criteria allows greater deformations and potential failure of the footwall. In addition, shear strength/shear stiffness assumptions determine the amount of displacement of the footwall slab and hence the probability of buckling. Analyses of curved footwall slopes with jointing orthogonal to bedding show the potential for buckling at the point of maximum slope curvature, the roll (Fig. 19). Where slopes are convex, buckling failure appears to be far more probable with the development of tensile stresses eventually allowing fracture and release of sections of bedding above the roll. An example of a major buckling failure was the Westfield open pit wall failure in Scotland in 1971. This failure occurred in a convex footwall comprising interbedded sandstones, shales, siltstone and ash (Scoble, 1981). Fig. 20 shows a distinct element analysis of this failure and clearly indicates zones of tension and incipient buckling within the slope. In the analyses of planar footwall for buckling failure it was found that this mode was difficult to reproduce unless high external forces and a massive slab were assumed. With the assumption of orthogonal joints in the planar footwall slab it was found that much of the movement appeared to be absorbed by joint closure. The closer the spacing of the joints the more movement of the slab can be absorbed. If a massive slab deforms elastoplastically then buckling is indicated above the slope toe by significant tensile and plastic failure (Fig. 21). As with other footwall failure mechanisms, the potential for instability increases with higher horizontal stresses. In a planar slope it thus appears, that a three-hinge model may not always represent the most appropriate failure mechanism.

In the buckling analyses presented in this paper the slopes are considered dry. The input of groundwater pressures reduces the effective shear strength along potential sliding surfaces and hence the stability of the slope. In most cases it does not change the failure mechanisms. It has been suggested that high water pressures may promote buckling failure (Cavers, 1981), and this is the subject of on-going research by the authors using numerical modelling. Groundwater observations within footwall slopes are, however, often severely limited and insufficient data exists with which to conduct site specific analyses. The very nature of footwall slope failures with instability being confined to relatively thin jointed slabs parallel to the footwall indicates that there exists a high potential for drainage. It might appear initially that water pressures would be less significant in this type of failure mechanism than with deep-seated failure; however, given the lower weight of materials involved, lower groundwater pressures might have a more significant effect.

The orientation of the principal stress directions with reference to the slope trend is important. While high stresses orthogonal to the slope may promote instability, high stresses parallel to the slope will increase the confining stresses and the resisting forces available on any potential lateral release surfaces. The effects of high horizontal stresses parallel to the slope requires further study using three-dimensional distinct element codes. It is important however that such analyses be constrained by high quality input data. Although three-dimensional modelling is of significant interest in aiding our understanding of footwall slope failure mechanisms, the data, time and costs involved would seem to preclude their use in routine design at present. The plan curvature has been shown to be a critical factor in rock slope stability assessment in UK surface coal mines (Stead, 1984). Slopes that are convex in plan tend to be more unstable. This is a function of both the stress distribution and the easier kinematic release of failure blocks. Most footwall slopes are linear and hence release surfaces in the form of discontinuities must be present and extremely persistent. Observations on footwall slopes indicate three-dimensional release via joint planes may be a critical factor in the progressive failure. Failure of keyblocks frequently appears to allow the opening of proximal discontinuities and the kinematic release of adjacent blocks. The propagation of failures with an apparently zigzag joint controlled lateral release may be evident (Fig. 9). The results of the two-dimensional distinct element modelling presented in this paper emphasize the importance of jointing in the determination of the potential failure mechanisms. Field observation would seem to indicate the importance of discontinuities in both dimensions.

4. Conclusions

This paper describes the potential application of numerical modelling methods in the investigation of surface coal mine footwall stability. To date, routine design of footwalls has been undertaken using limit equilibrium based techniques. The work conducted by the authors suggests that numerical modelling, particularly using distinct element codes, allows the mine engineer to gain an improved appreciation of the effects of jointing and slope curvature on the critical footwall slope length. With the recent advances in desktop computing power the authors believe that two-dimensional distinct element modelling provides a highly attractive addition to present footwall design technology. The analysis of any footwall slope must however be site dependent and a detailed knowledge of the geological structure is important. It is essential that the orientation, spacing, persistence and properties of joint sets be known, as they may be critical in allowing the kinematic release of failures.

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