A review of ground movements due to civil and mining engineering operations

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ABSTRACT: Man’s activity frequently causes ground movements which then may present him with problems. The most notable examples of ground movements are provided by the mining industry in the form of subsidence. Mining in the broad sense includes removal of material from the ground and that material may be solid, liquid or gas. Indeed some of the largest subsidence records have been in association with the abstraction of oil and groundwater, instances having occurred where the ground surface has been lowered by several metres over large areas. In Britain some of the most catastrophic ground movements have been attributable to the exploitation of brine in Cheshire.

The construction industry is also responsible for generating ground movements, admittedly usually on a small scale. For example, deep excavation causes a reduction in the vertical and horizontal pressure in the ground and thereby can induce heave of the base of the excavation, together with inward and vertical movements, both up and down, in the surrounding ground. Significant movements can occur at an appreciable distance from an excavation and horizontal movements can be noticeably larger than vertical movements. The most important factor which governs the magnitude of the movement is the type of ground involved. Ground movements may develop as a result of tunnelling, particularly in soft ground, and may resemble those associated with longwall mining of coal.

Induced seismicity provides a further instance of man’s action giving rise to ground movements. In this case, some of the most noteworthy examples have been provided by reservoir loading and the permeation of water into the ground. Small scale seismic events also have been associated with mining activity. They are an interesting example of the complexity of the relationships between ground movement and mining activity.

Introduction

Surface ground movements are associated with mining and some civil engineering activities such as tunnelling and the construction of deep excavations. They reflect the movements which occur as a result of removal of material, be it mineral deposit or muck. Unfortunately such ground movements can and do affect structures, services and communications adversely and, in addition, mining subsidence can lead to flooding of low-lying areas and sometimes to the sterilization of land.

Different types of ground movement have been recognized and can affect different structures in different ways. For instance, in the case of subsidence associated with longwall mining, vertical subsidence may have a serious affect on drainage systems, and associated tilting may cause serious concern in relation to railway tracks and tall slender buildings. Damage to buildings in such subsidence areas generally is caused by differential movements and compressive and tensile strains. The latter are generally more serious than compressive strains as far as structural damage is concerned. Usually, however, it is not just a simple matter of examining the reaction of a structure to a particular value of tensile or compressive strain. For example, it is quite common for a building to be subjected to compressive strains in one direction and tensile strains in another one. It also may be subjected to alternating phases of tensile and compressive ground movements so there will be a dynamic effect to consider. Thus any acceptable design for a structure situated in an active mining area must have regard to the nature, degree and periodicity of the ground movements likely to be caused by mining.

Ground movement that adversely affects the safety or function of a structure is unacceptable. However, with many buildings their appearance is also of concern and therefore significant cracking of architectural features also is unacceptable. Hence an estimation of the amount of ground movement that will adversely affect structural members and/or architectural features is required. This is influenced by many factors, including the type and size of the structure, and the properties of the materials of which it is constructed, as well as the rate and nature of the movements themselves. Because of the complexities involved critical movements have not been determined analytically. Instead, almost all criteria for tolerable subsidence or settlement have been established empirically on the basis of observations of ground movement and damage in existing buildings.

Two parameters commonly have been used for developing correlations between damage and differential settlement, namely, angular distortion and deflection (Fig. 1). Angular distortion (δ/I) is the differential movement between two points divided by the distance separating them. When related to building damage, angular distortion is commonly modified by subtracting the rigid body tilt (α) from the measured displacement. In this way the modified value is more representative of the deformed shape of the structure. The deflection ratio (Δ/l) is defined as the maximum displacement (Δ) relative to a straight line between two points divided by the distance (l) separating the points.

Investigations undertaken by the National Coal Board (now British Coal) (Anon. 1975) have revealed that typical mining damage starts to appear in conventional structures when they are subjected to effective strains of 0.5 to 1.0 mm/m and damage can be classified as negligible, slight, appreciable, severe and very
severe. However, this relationship between damage and change in length of a structure is only valid when the average ground strain produced by mining subsidence is equated by the average strain in the structure. In fact, this commonly is not the case, strain in the structure being less than it is in the ground (Geddes 1984). In addition, this assessment of damage due to subsidence takes no account of the design of the structure or of construction materials.

**Granular soils**

Irrespective of height, the maximum stable unsupported slope which can be excavated in a dry cohesionless soil is equal to its angle of repose. For dense sand this would be approximately 37° to 45° compared with about 30° for loose sand. Providing adequate support is installed during excavation the stability of steeply sloping or vertical sided excavations can be maintained, although both horizontal and vertical ground movements may occur. O'Rouke (1981) described the deflection of a supporting wall in terms of bulging and cantilevering (Fig. 2). The use of anchors near the top of the wall limits cantilevering action so that the ratio of horizontal to vertical

**Ground movements due to excavation**

Construction operations which entail either surface or sub-surface excavation are liable to bring about both vertical and horizontal deformations within the adjacent and overlying ground. Where excessive, such movements may threaten the serviceability of nearby structures and the integrity of the excavation also may be endangered. Prediction of the incidence, magnitude and style of ground movements is an important aspect of construction work involving excavation. This is particularly the case in urban areas where there are likely to be sensitive structures and services within the zone of affected ground. However, even where there are no structures to be taken into account, the effects on land drainage and the excavation itself might need to be considered.

The main causes of ground movements associated with excavation are related to reduction in horizontal and vertical stresses which lead to the release of elastic strain and, in the case of weak materials, plastic deformation. The magnitude of ground movements may be reduced by limiting these responses by supporting the ground immediately after excavation. In practice, movements still occur due to failure to support the ground quickly enough and to provide sufficiently stiff support. In addition, ground movements result from over-exca-
movement near to the edge of the excavation is correspondingly reduced from a typical value of about 1.32 for non-anchored walls to 0.6 for anchored ones.

Ground movements in the area adjacent to excavations in dense sands above the water table are generally quite small. For anchored walls lateral movements are liable to be in the range 0.18 to 0.7% of the excavation depth. The corresponding maximum settlement is 0.3% (Anon 1982). Larger movements are likely to be associated with non-anchored walls. Irvine & Smith (1983) reported that in the case of supported trenches settlements of between 0 and 3% of the depth typically occur within a zone of width one and a half to two and a half times the depth. Unless precautionary measures are taken large and unpredictable ground losses are liable to occur during excavation below the water table in loose sand or silt. Such an event is likely if water seepage pressures are sufficiently high to cause internal erosion of the deposit.

Ground losses during tunnelling occur as a result of a combination of elastic and plastic deformation into the excavation, over-excavation, deformation of the support system, re-compression of dilated ground and consolidation effects. Loose cohesionless soils may be subject to slumping in unsupported excavations. Atkinson & Potts (1977) suggested that such an event would be liable to occur if:

\[ \gamma d \sigma_T > \mu^2 + 1 \mu \]

where \( \sigma_T \) is the internal support pressure (= 100 kPa for free air), \( \gamma \) is the unit of weight of the soil, \( d \) is the diameter of the tunnel, \( \mu = (1 + \sin \phi')/(1 - \sin \phi') \) and \( \phi' \) is the effective angle of internal friction of the soil. For a loose sand \( \mu \) would equal approximately 0.49, compared with about 0.14 for a dense one.

Sources of ground loss associated with shield driven lined tunnels were described by Attewell et al. (1986). Inevitable over-excavation, soil intrusion and deformation of the shield and lining give rise to losses of between 2 and 5% of the excavated volume in granular soils above the water table. Ground losses are liable to be doubled if excavation takes place below the water table and compressed air is used to maintain the stability of the excavation. Ground losses can be difficult to predict in made ground, but a figure of 17% is suggested for household and industrial waste. Experience with tunneling in old fill comprising natural soil and rock indicates that a value of 8% might be appropriate.

Ground loss at the tunnel is transmitted through the overburden which, therefore, is deformed and it may reach the surface ultimately to produce a subsidence trough. The essential features of a typical subsidence trough are illustrated in Fig. 3. Spreading the movements over a broader area reduces their magnitude as they approach the ground surface. Furthermore, Ward & Pender (1981) pointed out that shearing action within the ground causes dilation of certain types of soil thus compensating either partly or wholly for the tunnelling ground loss. These authors suggested that for drained loose sands and dense sands at high pressure, the ratio of the settlement at the surface \( S_{\text{max}} \) to that which occurs immediately above the crown of the tunnel \( S_{\text{crown}} \) may be represented as follows:

\[ \frac{S_{\text{max}}}{S_{\text{crown}}} = 1 - 0.4 \frac{h}{d} \]

where \( h \) is the depth of the tunnel crown below the surface, \( d \) is its diameter and \( h/d > 2.5 \). In practice in unsaturated sands and gravels, ground movements attenuate fairly rapidly upwards giving little surface effect for large depth to diameter ratios.

Attewell et al. (1986) indicated that, regardless of soil type and tunnelling method, the transverse profile of the surface settlement trough follows a normal-probability or Gaussian function (see Fig. 3). The settlement \( S_{\text{max}} \) on the axis is related to the volume of the trough \( V_a \) as follows:

\[ S_{\text{max}} = \frac{V_a}{i \sqrt{2\pi}} \approx \frac{V_a}{2.5i} \]

where \( i \) is the distance from the inflection point to the axis of the trough. At this point the subsidence is 0.606 \( S_{\text{max}} \). In practice the width of the trough is approximately \( 3i \) and the forward trough extends in front of the face such that between 30 and 50% of the total settlement has occurred in front of the face. Both tensile and compressional ground strains are generated. The maximum tensile strain occurs on the flanks of the trough at a distance of \( \sqrt{3}i \) from the axis and the compressional strains reach a maximum value on the axis. Longitudinal tensile strains occur on the forward area of the settlement trough; these are subsequently replaced by compressional and then neutral strains as the face progresses.

The \( i \) parameter is a useful indication of the lateral extent of the subsidence trough. O'Reilly & New (1982) suggest that in granular soils \( i \) may be estimated as follows:

\[ i = 0.28 \text{ (Depth of tunnel axis)} - 0.1 \text{ m} \]

Compared with typical subsidence troughs for tunnels in cohesive soils, those in granular soils are deeper and have steeper sides. Departures from the ideal shape are liable to occur in non-homogeneous ground or where ground stability is not adequately controlled during tunnelling.

Attewell et al. (1986) discussed the ground strains which occur during the transference of ground losses at the tunnel to the ground surface. Building foundations and buried services are subjected to differential settlement and angular distortion. The orientation and magnitude of these strains depend on the position with respect to the tunnel and depth below the surface. Attewell et al. (1986) provided charts and equations from which it is possible to quantify the surface and subsurface strains and displacements.
Cohesive soils

Excavation in saturated plastic clays can cause upward movement of the base of the excavation accompanied by horizontal movement and settlement of the adjacent ground. Compared with most other types of soil, in cohesive materials movements continue for a protracted interval of time and achieve a greater magnitude. The style of movements are illustrated in Fig. 2 for a braced excavation in soft to medium clay. Owing to the lack of restraint of movement at the top of the wall, both bulging and cantilevering of the structure have occurred.

For anchored walls in cohesive ground lateral movements at the top of the wall tend to be quite small, in the range of 0.2 to 0.34% of the excavated depth (Anon. 1982). Such movements would be typically associated with up to 0.3% settlement near the top of the wall. The surface effects of excavation are liable to extend to a diminishing extent for distances of up to three or four times the depth. Although in the case of tied-back walls the lateral movements are reduced in magnitude, they tend to occur over a broader zone of up to five times the depth. Hurrell & Attewell (1984) suggest that trenches excavated in soft clays may cause horizontal and vertical movements of as much as 3% of the excavated depth. This compares with a figure of less than 1% for stiff clays.

The failure of the base of an excavation generally will be accompanied by very large downward and lateral movements in the adjacent area. Base failure is liable to occur if the following condition is satisfied:

$$\frac{\gamma h}{c_u} > 4$$  \hspace{1cm} (5)

where $\gamma$ is the unit weight of the soil, $h$ is the depth of the excavation and $c_u$ is the undrained shear strength of the soil. Ground movements occur for ratios of less than 4 but as the value approaches this critical level, there is a corresponding increase in magnitude.

Burland et al. (1977b) reported on the ground movements which occurred during the excavation of a 29 m high unsupported face in stiff overconsolidated Oxford Clay. Movements occurred at distances of up to 60 m from the excavation and horizontal displacements of up to 200 mm with associated settlements of up to 100 mm were measured near to the face. In addition, approximately 100 mm of heave occurred in the base of the excavation. The clay adjacent to the excavation tended to move as a block into the excavation along a horizontal shear surface near the base of the face.

Research into the ground movements associated with tunnel construction (Attewell et al. 1986) indicates that a subsidence trough of the form shown in Fig. 3 is produced (see also Equation 3). Ward & Pender (1981) maintained that in cohesive ground the losses of ground at the tunnel can be related to that which occurs at the surface by the expression:

$$\frac{S_{max}}{S_{tan}} = 1 - 0.13 \frac{h}{d}$$  \hspace{1cm} (6)

where $S_{max}$ is the settlement at the surface on the axis of the subsidence trough, $S_{tan}$ is the settlement directly above the tunnel crown, $h$ is the depth of the tunnel crown and $d$ is the tunnel diameter. In fact, the fissured nature of many overconsolidated clays may cause them to dilate during undrained shear and the surface settlement would be reduced (see Equation 2). Experience with tunneling in London Clay suggests that $S_{max}/S_{tan}$ does not fall below about 0.4 for $h/d$ ratios greater than 5. In normally consolidated or lightly overconsolidated clays, there would be no volume change during undrained shearing, but drainage could lead to consolidation. In this case the multiplier in Equation 6 would be less than 0.13, especially for shallow tunnels.

The width of the subsidence trough can be defined in terms of $i$, the point of inflexion on the profile shown...
in Fig. 2. Following a review of tunnel subsidence data for a variety of ground conditions, O’Reilly & New (1982) quoted the following expression for tunnels at depths of between 3 and 34 m in cohesive soils:

\[ i = 0.43 \text{(Depth of tunnel axis)} + 1.1 \text{ m} \quad (7) \]

Attewell et al. (1986) indicated that in cohesive soils the ground loss at the tunnel depends principally on soil stiffness, immediacy of positioning support and rate of driving. For stiff clays these authors proposed that ground losses of between 0.5 and 3% of the tunnel face area with an average of 1 to 2% are typical. This value may well increase to between 2 and 5% in glacial tills and attain values between 2 and 10% in soft Recent clay deposits.

Attewell et al. (1986) pointed out that the amount of ground loss at the tunnel may be related to the tunnel depth and soil cohesion by the definition of an overload factor. This follows from work by Broms & Bennermark (1967) who suggested that for undrained conditions the failure of a plastic clay in a tunnel face will occur if the following conditions are satisfied:

\[ \sigma_l - \sigma_r + yh > 0 \quad (8) \]

In this equation \( \sigma_l \) is the value of surface loading, \( \sigma_r \) is the internal tunnel support pressure, \( y \) is the unit weight of the overburden, \( h \) is the depth of the tunnel and \( c_u \) is the undrained shear strength of the soil. For shallow tunnels the critical value of this ratio may be less than six. Obviously, very large and unpredictable ground movements take place if failure occurs but increasing amounts of ground will be intruded into the tunnel for ratios of more than about two (Romo & Diaz 1981). Attewell et al. (1986) contended that the volume of the surface settlement trough (\( V_s \)) which is a product of tunnel ground loss may be related to the ratio in Equation 8 as follows:

\[ V_s = \frac{1.33(yh - \sigma_r)}{c_u} - 1.4 \quad (9) \]

where \( 1.5 \leq (yh - \sigma_r/c_u) \leq 4 \).

A similar concept to that expressed in Equation 8 was used by Davis et al. (1980) to predict instability of an unsupported tunnel face where failure occurs if \( yd/c_u > 4 \).

**Rocks**

In rocks ground movements occur due to the elastic response of the ground to the change in stress. Such movements tend to be relatively minor, but over-exca-

vation or instability of the floor, roof, face or sides of excavations can bring about large displacements, both horizontal and vertical.

Rock mass discontinuities exert a major control over instability in rock excavations. Stratified sequences of sedimentary rocks, especially those including clay rich rocks, are prone to slippage along bedding planes. Depending on the incidence, orientation and character of these and other discontinuities, excavations in which a face is created normal to the strike tend to be stable, but where the beds dip into the excavation instability may well occur. In most cases dips of between 30° and 70° create the most critical situation. Care has to be taken in the analysis of potential instability to ensure that pore-water pressures are adequately considered and also that an appropriate angle of shearing resistance for potential slip surfaces has been used.

Faults and fracture zones can give rise to zones of instability, especially if some weathering or alteration of the rock has taken place. In addition, an unfavourable distribution of discontinuities can give rise to the release of wedges or rock into tunnels (Ward 1978) or slopes (Hoek & Bray 1981). In the case of tunnels, unless instability is controlled, displacement of parts of the rock mass may continue until stopped by bulking action, the presence of a strong beam or the establishment of a natural arch.

Ward & Pender (1981) indicated that in contrast to soils, for rock excavations elastic analyses provide a useful means of predicting the responses to stress changes. Hence the radial deformation (\( \delta r \)) due to a reduction in stress from \( p_o \) to \( \sigma_r \) in a rock mass with a given Young’s modulus, \( E \), and a Poisson’s ratio, \( \nu \), will be equal to:

\[ \delta r = \frac{1 + \nu}{E} (p_o - \sigma_r) r \quad (10) \]

where \( p_o \) is the isostatic stress and \( \sigma_r \) is the radial stress at distance \( r \) from the axis of a circular excavation. The curves in Fig. 4, from Ward (1978), show the distribution of displacements in the walls of a tunnel for cases in which the vertical and lateral stresses are equal, and in which the lateral stress is zero. In the latter case inward movements occur at the crown and invert of the tunnel and outward movements, expressed here as a proportion of the movements determined from Equation 10, occur on the spring line. Ward & Pender (1981) suggested that depending on the in situ stress conditions between 20 and 40% of the radial displacement due to the reduction of stress has occurred at the tunnel face and full relaxation is nearly complete one tunnel diameter later. The displacements are reduced if the depth to diameter ratio is less than two but not increased for greater values than this.

Displacements are dissipated in a rock mass in the manner indicated in Fig. 5 for an unlined machine excavated tunnel in mudstone (Ward et al. 1976). Significant movements are confined to within about one tunnel diameter of the excavation. The selection of suitable rock mass parameters and in situ stress conditions poses a major problem for the prediction of ground movements in rocks. Field values of elasticity and Poisson’s ratio, in particular, are liable to be less than laboratory values.

**Excavation dewatering**

The lowering of pore-water pressures which occurs as a consequence either of groundwater pumping to improve the stability of and working conditions in excava-
vations or of natural drainage of groundwater into excavations may lead to consolidation of dewatered soils. This is brought about by an increase in the effective overburden pressure amounting to about 10 kPa for each 1 m of water head reduction. Even in the case of sealed excavations reduction of pore-water pressures may occur due to drainage into disturbed ground near to the excavation.

Very large settlements have occurred where groundwater levels have undergone considerable reduction. Parts of Mexico City have settled about 6 or 7 m due to a reduction on 30 m in groundwater levels (Poland & Davis 1963). In the case of excavations the amount of consolidation and its distribution depend primarily on the thickness and consolidation properties of the materials in question and the reduction in pore-water pressures. In a stratified series of deposits in which the horizontal permeability exceeds that in the vertical direction, the lateral spread of consolidation effects will be relatively large and may well be greater than the area affected by ground movements associated with ground loss. Consolidation effects also may occur over a protracted period of time, for example, 18 months as quoted by Attewell et al. (1986) for a tunnel in organic silty clay.

The amount of consolidation settlement which occurs in dense sands and rock will be slight. However, loose sands may be subject to some consolidation, especially in the case of a fluctuating water table. On the other hand normally and overconsolidated clays, organic soils and peats are subject to consolidation effects. In the case of overconsolidated clays analyses are usually based

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**Fig. 4.** Elastic deformation due to relaxation of rock mass. (After Ward 1978).

**Fig. 5.** Vertical movements recorded adjacent to unlined tunnel in mudstone. (After Ward et al. 1976).
on the coefficient of volume compressibility \(m_v\), typical values of which are given in Table 1. The consolidation settlement \(S_{\text{con}}\) is given by:

\[ S_{\text{con}} = m_v \times \Delta p \times h \quad (11) \]

where \(h\) is the thickness of the consolidating layer and \(\Delta p\) is the increase in the effective stress (= the unit weight of water \(\times\) the reduction in groundwater level).

**Table 1. Consolidation parameters for soils (after Head 1982).**

<table>
<thead>
<tr>
<th>Type of Clay</th>
<th>Coefficient of Volume Compressibility (m_v), (\text{m}^2/\text{MN})</th>
<th>Compression Index, (C_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very organic alluvial clays and peats</td>
<td>&gt; 1.5</td>
<td></td>
</tr>
<tr>
<td>Alluvial and estuarine normally consolidated clays</td>
<td>0.3 – 1.5</td>
<td></td>
</tr>
<tr>
<td>Fluvio-glacial, lacustrine clays and weathered over-consolidated clays</td>
<td>0.1 – 0.3</td>
<td></td>
</tr>
<tr>
<td>Boulder clays and stiff or hard unweathered over-consolidated clays</td>
<td>0.05 – 0.1</td>
<td></td>
</tr>
<tr>
<td>Heavily overconsolidated clays and stiff weathered rocks</td>
<td>&lt; 0.05</td>
<td></td>
</tr>
<tr>
<td>High plasticity montmorillonitic clays</td>
<td>up to 2.6</td>
<td></td>
</tr>
<tr>
<td>Medium and low plasticity clays and silts</td>
<td>0.8 – 0.2</td>
<td></td>
</tr>
</tbody>
</table>

Analyses involving normally consolidated clays are usually carried out in terms of the compression index, \(C_c\), typical values of which are given in Table 1. In this case the consolidation settlement may be found from the following expression:

\[ S_{\text{con}} = \frac{C_c}{1 + e_0} \times h \log \left(1 + \frac{\Delta p}{\gamma h}\right) \quad (12) \]

where \(e_0\) is the original void ratio of the deposit, \(\Delta p\) is the increase in vertical stress (= unit weight of water \(\times\) drawdown of water head), \(\gamma\) is the unit weight of the soil and \(h\) is the depth of the consolidating layer. Equation 13, derived by Skempton (1944), provides an alternative means of estimating the value of the compression index for normally consolidated clays.

\[ C_c = 0.009 \text{ (liquid limit of clay, \% – 10\%)} \quad (13) \]

Head (1982) pointed out that this relationship does not apply to highly organic clays, where the liquid limit exceeds 100% or where the natural water content is greater than the liquid limit. In practice the ground is divided up into a number of layers for which values of the consolidation parameter \((m_v\) or \(C_c\)) can sensibly be assigned. The total settlement is then computed by summing the contributions for each layer.

By comparing short term settlements attributed to ground losses with the longer term values which include consolidation effects, Attewell et al. (1986) suggested that less consolidation occurs in stiffer clays. Because ground loss during tunnelling is then also reduced the short term (ground loss) settlement, \(S_{\text{max}}\) is correspondingly diminished and the following relationship has been derived:

\[ S_{\text{max}} = S_{\text{con}} = 0.78 \left(S_{\text{max}} - 0.01 S_{\text{max}} \frac{\gamma h}{c_u}\right) \quad (14) \]

where \(\gamma\) is the unit weight of the soil, \(h\) is the depth of the tunnel and \(c_u\) is the undrained shear strength of the soil.

The lateral spread of dewatering settlements associated with both surface excavations and tunnels can be limited by constructing impermeable barriers around the area being dewatered. An alternative method is to re-charge aquifers in which water levels would otherwise be reduced (Bell & Cashman 1986).

**Ground movements due to mining activities**

Subsidence of the ground surface can be regarded as ground movement which takes place due to the extraction of mineral resources. It is an inevitable consequence of mining activities and reflects the movements which occur in the mined out area. As a consequence, small deformation in workings is associated with insignificant subsidence whereas total closure of workings can give rise to severe subsidence at ground level. Unfortunately subsidence can and does have serious effects on surface structures, services and communications, can be responsible for flooding, can lead to the sterilization of land or call for extensive remedial measures or special constructional design in site development.

Coal is by far the most important mineral mined in Britain and indeed mining has gone on in many coalfields in Britain for several centuries. However, the first statutory obligation to keep mine records only dates from 1850 and it was not until 1872 that the production and retention of mine plans became compulsory. Even if old records exist, they may be inaccurate. Matters may be complicated further by the fact that many old workings subsequently were built over. An appeal for the deposition of plans of old workings, launched by the Government in 1925, allowed the Department of Mines to publish a catalogue of plans of abandoned mines in 1931. Supplements to the catalogue were published annually until 1938. The catalogue and its supple-
ments are now out of print but most major libraries in mining areas retain copies. Recently, a further appeal for old plans was made by the Department of Energy.

Old abandoned coal workings occur at shallow depth beneath the surface of many urban areas in western Europe and North America and, as in Britain, the presence of coal was one of the major reasons why urban development took place in the first instance. Because many of these old workings were unrecorded they can represent a potential hazard during subsequent redevelopment.

Resume of past methods of working coal

In Britain coal mining began to be carried out on a significant scale in the thirteenth century. Drifts and adits into shallow workings were usually situated at the base of quarries and open pits or along the coal outcrops in hilly country. The workings extended as far as natural drainage and ventilation permitted.

By the fourteenth century outcrop workings had largely given way to bell pits. The shafts of bell pits rarely exceeded 12.2 m in depth and their diameter was usually about 1.3 m. They are, therefore, a feature of coalfield areas where the drift cover is thin. Extraction was carried on around the shaft until such times as roof support became impossible; another shaft was then sunk nearby. Hence, where such mining went on, the number of bell pits may be very numerous. If bell pits were backfilled, then the state of compaction of the fill is generally unsatisfactory.

Where a coal seam occurred at more than about 7 m below the surface, bell pit mining tended to be replaced by headings which radiated into the coal seam for short distances around the shaft. The pillars of coal between the headings generally represented the only type of support to the overlying strata. The layout of a mine was unplanned and simply consisted of a complex of interconnected headings. Hence the support pillars were irregular in shape and size.

Increased demand for coal in the sixteenth century led to the development of the pillar and stall method of extraction. Underground workings were shallow and not extensive, for example, they rarely penetrated more than 40 m from the shaft. Indeed, when such limits were reached, it was usually less costly to abandon a pit and sink another shaft nearby. Workings extending 200 m from the shaft were exceptional even at the end of the seventeenth century, the shaft itself usually being less than 60 m deep (Bell 1979).

In early mining the remnant pillars were rather haphazard in size and arrangement, but with time mining became more systematic and pillars of more or less uniform shape (generally square or rectangular in outline) were formed by driving intersecting roadways in the seam. Also there was a general tendency for the size of stalls to increase. Regional differences in mine layouts tended to reflect preferences in working method or the influence of the geological setting (for example, steeply dipping seams). Several variations of the pillar and stall method were devised, for example, Staffordshire squarework was developed to work the Ten Yards Coal seam, and in the Sheffield area ribs of coal were generally left to support the roof rather than pillars.

With the development in the eighteenth century of steam power and the use of coal to smelt iron, the demand for coal accelerated. Longwall working evolved more or less at the same time, probably first originating in the Shropshire coalfield. This method involves total extraction, panels being developed from an initial drivage within the seam. The support at the working face is continuously being moved as the face advances leaving the roof from which support has been withdrawn to collapse.

Pillar and stall workings and subsidence

The pillar and stall method, or variations of it, is used in many parts of the world to mine coal and extensively for other stratified deposits. Pillars have to sustain the redistributed weight of the overburden which means that they and the rocks immediately above and below are subjected to added compression. In the case of coal, although its intrinsic strength varies, the important factor concerning pillars is that their ultimate behaviour is a function of seam thickness to pillar width, the depth below ground and the size of the extraction area. The mode of failure also involves the character of the roof and floor rocks. The height to width ratio of the stall also is significant. In addition the greatest stress occurs at the edges of pillars, between pillar and roof or between pillar and floor. Pillars in the centre of the mined out area are subjected to greater stress than those at the periphery. Individual pillars in dipping seams tend to be less stable than those in horizontal seams since the overburden produces a shear force on the pillar.

Pillars often experience local failures whilst mining is taking place. If a pillar is highly jointed, then its margin may fail and fall away under relatively low stress. Such action reduces and ultimately removes the constraint from the core thereby subjecting it to increasing stress. This can lead to pillar failure. Slow deterioration and failure of pillars may take place years after mining operations have ceased, although observations in coal mines at shallow depth, along with the resistance of coal to weathering suggests that this is a relatively uncommon feature at depths less than 30 m. However, old workings affect the pattern of groundwater drainage which, in turn, may influence pillar deterioration. On the other hand, the small pillars of earlier workings in coal may be crushed out once the overburden exceeds 50 or 60 m (Piggott & Eynon 1978). Old pillars in coal mines at shallow depth have occasionally failed near faults and they may fail if they are subjected to the effects of subsequent longwall mining.

Collapse of one pillar can bring about collapse in others as increased loads are placed on those remaining. Yielding of a large number of pillars can bring about a shallow broad subsidence over a large surface area. Marino & Gamble (1986) referred to such bowl-shaped depressions as sags. The ground surface in a sag dis-
places radially inwards towards the area of maximum subsidence. The inward radial movement generates a tangential compressive strain.

Sag movements depend on the mine layout and geology, as well as the topographic conditions at the surface. They tend to develop rather suddenly, the major initial movements lasting, in some instances, for about a week, with subsequent displacements occurring over varying periods of time. The initial movements can produce a relatively steep-sided bowl-shaped area. Sag profiles which develop slowly are shallower. In fact the shape of the profile can vary appreciably and because it varies with mine layout and geological conditions, it can be difficult to predict accurately. Normally the greater the maximum subsidence, the greater the likelihood of variation in the profile. Maximum profile slopes and curvatures frequently increase with increasing subsidence. Marino & Gamble (1986) found that the magnitude of surface tensile and compressive strains ranged from slight to severe.

Trough-like surface subsidences resulting from the failure of large pillars in other rocks which have been mined (for example, limestone) frequently are bounded by relatively steep sides which may be fissured. The floors of such troughs often are irregular, reflecting the remains of collapsed or crushed pillars.

The potential for pillar failure should not be ignored, particular warning signs being strong roofs and floors allied with high extraction rates and moderately to steeply dipping seams. Prediction of subsidence as a result of pillar failure requires accurate data regarding the layout of the mine. Such information frequently does not exist in the case of abandoned mines. On the other hand when accurate mine plans are available or in the case of a working mine, the method outlined by Goodman et al. (1980) may be used to evaluate collapse potential. Basically the method involves calculating vertical stress based on the tributary area load concept which assumes that each pillar supports a column of rock with an area bounded by stall centres and a height equal to the depth from the surface and comparing this with the strength of the pillar. Previously a review of the various methods of determining pillar strength and pillar failure had been provided by Hustrulid (1976) and Bell (1978). In addition, a recent survey of underground excavation failure mechanisms has been given by Hoek & Brown (1980).

A programme to assess the stability of pillars in limestone mines has been outlined by Agapito (1986). Two types of measurements were made. First, measurements of internal strain in drillholes were obtained for the calculation of stresses. Secondly, measurements of rock mass deformation between roof and floor and within the roof and floor were taken. The objectives of the programme included determination of the stress field, in order to evaluate mine loading background and its possible effects on roof stability and pillar orientation. A second objective considered pillar stress distributions and average loads to help assess mine stability and pillar strengths. In addition, pillar and roof deformations were investigated to help define movement patterns in the roof, pillars and floor, such data being required to determine mine stability and roof strengths. The finite element method of computer modelling was used to evaluate pillar stability. Lastly, an assessment of Young's modulus was required for use in numerical modelling.

Squeezes or crushes sometimes occur in a mine as a result of the pillars being punched into either the roof or floor beds. In such cases the roof or floor may have been weakened or altered by the action of water or weathering. Once again surface subsidence adopts a trough-like or basin form and minor strain and tilt problems occur around the periphery of the basin thereby produced (Bruhn et al. 1981). Briggs (1929) reported that when pillars are forced into a yielding pavement subsidence may restart. This could take place many years after mining had ceased and could damage property. He quoted an example from Wallsend, in NE England, where the surface subsidence amounted to 1.2 m. Such action becomes more significant with increasing depth or high extraction ratio since these impose greater loads upon the pillars. Although such subsidence is often slow it may be quickened by the ingress of water into the workings.

Even if pillars are relatively stable the surface may be affected by void migration. This can take place within a few months of or a very long period of years after mining and in British coal mines it is a much more serious problem than pillar collapse. Void migration develops when roof rock falls into the worked out areas (Fig. 6). When this occurs the material involved in the fall bulks, so that migration is eventually arrested, although the bulked material never completely fills the voids. Nevertheless, the process can, at shallow depth, continue upwards to the ground surface leading to the sudden appearance of a crownhole. These are typically steep-sided. Site investigations frequently reveal partially choked voids in abandoned pillar and stall workings in coal. For example, the open upper part of the void may be less than a metre in height, the next being occupied by waste which may have suffered some amount of decomposition (Thorburn & Reid 1978). The infill is frequently chevron-shaped, especially when controlled by discontinuities and generally loose or poorly compacted.

The factors which influence whether or not void migration will take place include the width of the unsupported span; the height of the workings; the nature of the cover rocks, particularly their shear strength and the incidence and geometry of discontinuities; the thickness and dip of the seam; the depth of overburden; and the groundwater regime. According to Wardell & Eynon (1968) the maximum width of a self-supporting span (s) can be derived from the following expression:

\[ s = \sqrt{2 \frac{t \sigma_t}{}} \]  

where \( t \) is the thickness of the seam and \( \sigma_t \) the ultimate tensile strength of the roof rock beam (this should make due allowance for discontinuities).

If a competent rock beam is to span an opening, then
FIG. 6. Working in the Top Brockwell seam (0.6 m in thickness) at Sproats opencast site, Northumberland. Void migration is terminated against a bed of flaggy sandstone. (Courtesy of Roy Taylor).

it should be equal to twice the span width in order to allow for arching to develop. Chimney-type collapses can occur to abnormally high levels of migration in massive strata in which the joints diverge downwards. Walton & Cobb (1984) mentioned that three times the width of the stall appeared to be an upper limit to the extent of most void migrations in abandoned coal mines and 1.5 times the span was another frequently found factor. However, because of the difficulty of obtaining stall dimensions in abandoned workings, the height of void migration normally is determined from the thickness of mineral extracted and the difference in densities between the roof material in situ and when collapsed.

Depth of cover should not include superficial deposits or made-ground since low bulking factors are characteristic of these materials. Weak superficial deposits may flow into voids which have reached rockhead, thereby forming features which may vary from a gentle dishing of the surface to inverted cone-like depressions of large diameter.

The height of void migration can be determined, in general, by using the following expression:

$$D_c = t \left[ \frac{\rho - \rho_1}{\rho} \left( 1 - \frac{\rho_1}{\rho} \right) \right]$$

where: $t =$ thickness of the seam, $\rho =$ bulk density of the roof rocks, $\rho_1 =$ bulk density of collapsed roof materials, and $D_c =$ height of migration.

Alternatively the simpler expression:

$$D_c = \frac{h}{\sqrt{V - 1}}$$

can be used, where $h =$ the height of the working and $V =$ the volume occupied by the collapsed strata divided by the volume it occupied in the roof.

However, if as Piggott & Eynon (1978) conceived, the void adopts various geometrical forms such as conical, wedge and rectangular collapses, then different expressions must be used to calculate void migration. They showed that for a particular width of mine opening ($B$), the height of collapse or migration ($D_c$) is a function of the original height of the mine opening ($h$) and the bulking factor ($b_f$) of the overlying strata. They derived the following simple expressions for obtaining the height of void migration for the three geometrical forms mentioned (see also Fig. 7):

(i) Conical collapse

Volume of intact beds ($V_o$) = $\frac{nB^2}{4} \times \frac{D_c}{3}$

Total volume of collapse zone

$$V_c = V_o + \frac{nB^2}{4} \times h$$

But $b_f = \frac{V_c - V_o}{V_o}$

Hence $b_f = \frac{3h}{4}$ or $D_c = \frac{3h}{b_f}$
(ii) Wedge collapse

\[ V_0 = \frac{BDf}{2} \]  

where \( f \) = length of collapsed roadway

and \( V_c = V_0 + Blh \)  

Hence \( b_t = \frac{2h}{4} \) or \( D_c = \frac{2h}{b_t} \)  

(iii) Rectangular collapse

\[ V_c = Blh \]  

\[ V_0 = V_c + Blh \]  

Hence \( b_t = \frac{h}{D_c} \) or \( D_c = \frac{h}{b_t} \)  

Subsidence due to longwall working of coal

Longwall mining involves the total extraction of coal. Roof support is necessary and is moved as the coal is extracted thereby allowing the unsupported roof rock to collapse into the mined out void. The subsidence which occurs over a completely mined out area in a flat seam is trough-shaped and extends outwards beyond the limits of mining in all directions. In trough subsidence the resulting stratal and surface ground movements are regarded as largely contemporaneous with mining, producing more or less direct effects on any surface development (Brauner 1973a). Trough-shaped subsidence profiles develop tilt between adjacent points which have subsided different amounts and curvature results from adjacent sections which are tilted by differing amounts.

Maximum ground tilts are developed above the limits of the area of extraction and may be cumulative if more than one seam is worked up to a common boundary. Where movements occur, points at the surface subside downwards and are displaced horizontally inwards towards the axis of the excavation (Fig. 8). Differential

The maximum height of void migration is directly proportional to the thickness of seam mined and inversely proportional to the change in volume of the collapsed material. It would appear that the height of collapse is independent of the width of the excavation although it is a limiting factor. In other words the larger the span, the more likely collapse is to occur. The maximum height of migration in exceptional cases might extend to ten times the height of the original roadway, however, it generally is three to five times the roadway; height. If a competent bed occurs in the roof rocks, which is thicker than 1.75 times the span width, it will arrest the collapse.

 Exceptions, of course, do occur and void migration in excess of 20 times the seam thickness has been recorded. The self-choking process may not be fulfilled if dipping seams are affected by copious quantities of water which can redistribute the fallen material. Indeed Carter et al. (1981) attributed the development of super voids to such a process. Roof collapse voids could develop in this fashion which are actually larger than the original stalls by engulfing one or two surrounding pillars. Migration of super voids up to rock head then produces large scale subsidence at ground level.

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horizontal displacements result in a zone of apparent extension on the convex part of the subsidence profile (over the edges of the excavation) whilst a zone of compression develops on the concave section over the excavation itself. Differential subsidence can cause substantial damage, the tensile strains thereby generated usually being the most effective in this respect (Fig. 9).

In addition laminated rocks may suffer bed separation. The fracture zone defined by the extent of dilation extends at least half the face width above seam level.

Comparatively slight deviations in the subsidence profile are accompanied by appreciable variations in strain. In fact ground movement is three dimensional and movements of the vertical and two horizontal components may occur simultaneously.

As mentioned, removal of roof support in longwall mining is followed by collapse of those rocks which were immediately above the coal since they are subjected to bending and tensile stresses. These broken rocks offer partial support to the superincumbent roof layers. Nevertheless stresses in the rock mass remaining in place are significantly increased, and the resultant fracture and associated dilation mean that the rock strength is reduced from a peak to a residual value with loss in load bearing capacity and the redistribution of stress.
and, secondly, because at greater depths several work-
ing evidenced by the area of influence is completely worked out. Consequently the time which elapses before subsidence is complete varies according to circumstances.

Residual subsidence takes place at the same time as instantaneous subsidence and may continue after the latter events for periods of up to two years. The mag-

itude of residual subsidence is proportional to the rate of subsidence of the surface and is related to the mechanical properties of the rocks above the seam. For instance, strong rocks produce more residual subsidence than weaker ones. Residual subsidence rarely exceeds 10% of total subsidence if the face is stopped within the critical width, but falls to 2 to 3% if the face has passed the critical width. Very occasionally values greater than 10% have been recorded.

Geological factors and subsidence due to longwall mining

Ground movements induced at the surface by mining activities are influenced by variations in the ground conditions, especially by the near-surface rocks and superficial deposits. However, the reactions of surface deposits to ground movements are usually difficult to predict reliably. Indeed it has been suggested that 25% of all cases of mining subsidence undergo some measure of abnormal ground movement which, at least in part, is attributable to the near-surface strata.

In concealed coalfields the strata overlying the Coal Measures often influence the basic movements developed by subsidence. In fact, abnormal subsidence behaviour and inconsistent movements are much more common in concealed than exposed coalfields. The occurrence of abnormally thick beds of sandstone can modify stratal movement due to mining. Such beds may resist deflection, in which case stratal separation occurs and the effective movements at the surface are appreciably less than would otherwise be expected. The differences in behaviour disappear when the extraction becomes wide enough for the sandstone to collapse, when subsidence behaviour reverts to normal.

Whittacker & Breeds (1977) found a predominant jointing pattern in the Permo-Triassic strata overlying the Coal Measures in Nottinghamshire. They showed that in both the Bunter Sandstone and Permian Lime-

stone there was a tendency to produce less subsidence than that predicted for width-depth ratios exceeding 1.0. This, they suggested, was due to the two formations behaving as block-jointed media in which the individual blocks do not return exactly to their former positions after compressional ground strain has ceased. Obviously the friction between the blocks plays an important role. On the other hand rock type does not appear to have a marked influence on subsidence when the width-depth ratio is less than 1.0.

Higher values of maximum tensional ground strain due to subsidence were found to occur in the Bunter Sandstone than in the Permian Limestone, or indeed the Coal Measures strata. The values of tensile strain in the Bunter Sandstone also were much more irregular.

By contrast, there was no recognizable difference in the maximum compressional strains recorded. The type of rock did not appear to have a notable influence on the maximum ground slope resulting from subsidence. The relationship between subsidence over the ribside and width-depth ratio in the Bunter Sandstone again suggests that block behaviour occurs at the surface. Furthermore the maximum tensile strain appears to be displaced further away from the extraction area, that is, towards the area of less constraint on a line of surface blocks. Hence there is a greater likelihood of discontinuities opening further from the compressional zone rather than nearer to it. This does not appear to be valid for width-depth ratios less than 0.4, which suggests that magnitudes of strain need to be sufficiently large to promote surface block behaviour. However, in this situation it appears that the discontinuities in the Permian Limestone do not necessarily influence subsidence behaviour as much as those in the Bunter Sandstone and that the limestone tends to behave normally.

Rigid inclusions in stratal sequences above coal seams being mined, such as sandstone lenses in marls may be forced upwards by the associated compressive forces.

Drift deposits are often sufficiently flexible to obscure the effects of movements at rockhead. In particular, thick deposits of till tend to obscure tensile effects. On the other hand superficial deposits may allow move-

ments to affect larger areas than otherwise.

The necessary readjustment in weak strata to subsi-
dence effects can usually be accommodated by small movements along joints. However, as the strength of the surface rock and the joint spacing increases so the movement tends to become concentrated at fewer points so that in massive limestones and sandstones movements may be restricted to master joints. Well developed joints or fissures in such rocks concentrate differential dis-
placement. Tensile and compressive strains many times the basic values, have been observed at such discontinuities. For example, joints may gape anything up to a metre in width at the surface. According to Shadbolt (1978) it is quite common for the total lateral movement caused by a given working to concentrate in such a manner. In such instances no strain is measurable on either side of the discontinuity concerned.

Faults also tend to be locations where subsidence movement is concentrated thereby causing abnormal deformation of the surface. Unfortunately their exact location at the surface is not always easy to determine. What is more many coalfield areas are heavily faulted. The extent to which faults influence and modify subsi-
dence movements cannot be quantified accurately.

Whilst subsidence damage to structures located close to or on the surface outcrop of a fault can be very severe, in any particular instance the areal extent of such damage is limited, often being confined to within a few metres of the outcrop. Also many faults have not reacted adversely when subjected to subsidence (Hel-
lewell 1988).

Faults tend to act as boundaries controlling the extent of the subsidence trough. When workings terminate against a fault plane which has an angle of hade larger
than the angle of draw, then the subsidence profile extends to the surface with associated permanent strains probably accompanied by severe differential subsidence. Indeed a subsidence step may occur at the outcrop of the fault. Where the width of the extraction is large enough the surface subsidence at a point vertically above the intersection of the fault and the workings is approximately one half that of the maximum subsidence for that working. When the hade of the fault is less than the angle of draw, the fault again determines the extent of the subsidence trough, which in this case is less than that normally expected.

By contrast, a fault can have a beneficial effect by accommodating most of the subsidence movement and thereby reducing the amount of damage which normally would be experienced over a wider area. The presence of a fault does not increase the amount of subsidence on the side of the fault nearer the workings compared with normal conditions but subsidence on the opposite side is considerably reduced (by upwards of 50%).

Faults are most likely to react adversely when their hade is less than 30°, when they have a simple form and the material occupying the fault zone does not offer high frictional resistance to movement. For example, subsidence steps tend to occur when faults represent single, sharp stratal breaks (Lee 1966). By contrast if a fault consists of a relatively wide shatter zone, then the surface subsidence effects are usually less pronounced but more predictable in terms of location and amount. The thickness and nature of the unconsolidated deposits above bedrock can influence the magnitude of such features.

The location of the workings in relation to the fault and method of mining determines the position and nature of the anomalies. The most notable steps occur when the coal is worked beneath the hade of a fault, faces in other positions being much less likely to cause differential movement. Steps are usually down towards the goaf but if old workings exist, then steps may occasionally occur away from the face. Furthermore a single working of small width-depth ratio approaching a fault at right angles is less likely to cause a step than a large width-depth working parallel to the fault. A fault step is much more likely to develop when the fault has been affected by previous workings in shallow seams than it is for a single working in a virgin area. Workings on the upthrown side of a fault are more likely to cause stepping than similar workings on the downthrow side.

Once differential movement has occurred, further mining in the area can cause renewed movement, which at times may be out of all proportion to the thickness and extent of the coal extracted. As successive workings increase the effect on the fault plane, so the chances of steps developing are increased.

In concealed coalfields where, for example, a fault passes through Permo-Triassic strata severe fissures or fractures can occur at the surface. The fissures are generally parallel to the line of the fault but can occur up to 300 m away from it.

### Prediction of subsidence due to longwall mining

An important feature of subsidence due to longwall mining is its high degree of predictability. Usually movements parallel and perpendicular to the direction of face advance are predicted. Although adequate for many purposes such treatment does not consider the three dimensional nature of ground movement. For instance, recent observations have shown that individual points move on approximately helical paths, that the pitch and radii change from point to point and that the direction of rotation is different on opposite sides of the subsidence basin.

Methods of subsidence prediction can be grouped into three fundamental categories. First, empirical methods attempt to fit subsidence functions to field measurements. Secondly, theoretical methods of prediction derive subsidence functions from elastic theory and rock mechanics. Thirdly, semi-empirical methods develop subsidence functions based on theory but which are related to field data by the use of constants and correlation coefficients.

Empirical methods of subsidence prediction such as those used by the National Coal Board, now British Coal, have been developed by continuous study and analysis of survey data from British coalfields (Anon. 1975). Since they do not take into account the topography, the nature of the strata and geological structure involved or how the rock masses are likely to deform, Voight & Pariseau (1970) emphasized that such empirical relationships can be applied only under conditions similar to those of the original observations. Nevertheless these methods are continually being refined so that they can yield more accurate results. In fact they allow the amount of subsidence due to longwall mining in Britain to be predicted usually within ± 10%.

Burton (1978, 1981, 1985) developed an empirical three dimensional model of subsidence prediction which can be represented graphically. In this way a series of average subsidence and strain profile shapes which are related to width-depth ratios, can be portrayed as a single graph. Burton showed that surface points move forwards in the line of advance. By fixing one point and then calculating the displacements of another point in relation to this, a series of differential movements is obtained. Accordingly strains can be calculated along the line between the two points under consideration. In other words strain can be calculated in any direction with respect to face advance. This is important because the value of strain, or change in surface length, is direction related, that is, a zone of tension or compression, which is related to a line normal to the ribs, has no meaning for a line running in another direction. Hence a strain contour map must be related to a surface direction.

The theoretical methods of subsidence assessment assume that the stratal displacement behaves according to one of the constitutive equations of continuum mechanics over most of its range. The continuum theories were developed from the analysis of a displace-
ment discontinuity produced by a slit in an infinite elastic half-space. Analytical procedures were subsequently developed for three types of subsurface excavations based on elastic ground conditions, that is, non-closure, partial closure and complete closure. Further work extended the closed form solution to transversely isotropic ground conditions in both two and three dimensions.

Berry (1978) reviewed various theoretical methods which have been developed for predicting mining subsidence. He concluded that the ground is not sufficiently homogeneous to be treated as an elastic medium but nevertheless accepted that elastic models simulate most ground deformations with reasonable accuracy. The disadvantage is that for any particular problem estimation of the various elastic constants for massive layered rocks is difficult.

Numerical models permit quantitative analysis of subsidence problems and are not subject to the same restrictive assumptions required for the closed form analytical solutions. Finite element modelling frequently has been applied to subsidence problems since it can accommodate non-homogeneous media, non-linear material behaviour and complicated mine geometries. For example, Dahl (1972) analysed mining subsidence by using two and three dimensional finite element models assuming both elastic and elastoplastic rock behaviour. Although the finite element method has the advantage of allowing the elastic constants to differ for each element so that appropriate constants can be used for each layer, because of the involvement of considerable depths below the surface and either side of the excavation zone, it perhaps is of questionable value in terms of estimating subsidence.

Alternatively, finite difference models can be used for the large strain, non-linear phenomena associated with subsidence development. Other elastic approaches employing numerical techniques include boundary element methods. Tsur-Lavie et al. (1985) presented a boundary element method for subsidence evaluation based on a fundamental solution of stresses and deformation around a single rectangular indentation at the boundary of an infinite elastic half plane. The solution can be used for both compressible and incompressible material. The model was used for the analysis of ground subsidence as a function of the span and height of the longwall opening, assuming various values of Poisson’s ratio.

After reviewing the literature Aston et al. (1987) concluded that no analytical method currently exists which can predict surface subsidence reliably. In a comparison with results derived from empirical and semi-empirical methods, they found that analytical methods yielded smaller vertical displacements by almost an order of magnitude. As pointed out, most analytical models assume elastic bending, whereas additional mechanisms are involved in subsidence. It is therefore necessary to identify and understand each of the mechanisms involved, as well as their role in the overall process, in order to improve subsidence prediction.

Most viable methods of subsidence prediction fall into the category of semi-empirical methods since fitting field data to theory often results in good correlations between predicted and actual subsidence. There are two principal methods of semi-empirical prediction, namely, the profile functions and the influence functions methods (Brauner 1973b, Hood et al. 1983). The profile function method basically consists of deriving a function which describes a subsidence trough. The equation produced is normally for one half of the subsidence profile and is expressed in terms of maximum subsidence and the location of the points of the profile. Provided the input data are available, the profile function method provides an accurate technique for the prediction of subsidence where the mine geometry is relatively simple as in longwall mining.

The influence functions approach to prediction of subsidence is based upon the principle of superposition. The subsidence trough is regarded as a combination of many infinitesimal troughs formed by a number of infinitesimal extraction elements. There are two types of influence function, namely, the zone area or circle method and complementary influence functions. Both are suited to computer analysis.

Marr (1975) described the zone area or circle method of subsidence prediction in which surface subsidence is estimated by constructing a number of concentric zones around a surface point, the radius of the outer zone being equal to the radius of the area of influence, which can vary with the geological conditions. The subsidence at the surface point is obtained from the summation of the proportions of coal which are extracted in each zone, multiplied by its particular subsidence factor. In practice it has been found that three, five or seven zones are suitable for most estimations. The method permits estimation of subsidence which develops when panels of irregular shape are mined. Ren et al. (1987) compared the results obtained by using the zone area method of prediction with those derived by the National Coal Board method (Anon. 1975) and found that they were similar for both subsidence and displacement.

The fundamental concept of complementary influence functions is that the separate influence functions describing the response of mined and unmined zones act together to produce the subsidence (Sutherland & Munson 1984). Each influence function is defined by the response of a limit element, that is, an unmined element for the coal left in place and a mined element for the void created by extraction. The amount of subsidence is predicted by appropriately summing these elements over the entire seam, it being the sum of the influence of both the mined and unmined response. Hence, complementary influence functions can be used to compute subsidence above mines with complex geometry (that is, for pillar and stall as well as longwall workings). This method, however, tends to overestimate the subsidence directly over the ribside.

Subsidence associated with mining other deposits than coal

In addition to coal, a number of other materials have
been extracted from Coal Measures. Fireclays and ganusters have been mined for refractory purposes. These workings are widespread but gannister is of more importance in the Lower than in the Middle Coal Measures, whereas fireclays are important in both, particularly the latter. Fireclays have been worked by pillar and stall methods in many areas, notably in the Leeds district, south Staffordshire, Lancashire and west Lothian. Clay ironstone workings in Coal Measures shales date back to Roman times. These ironstones were frequently worked by means of bell pits (Fig. 10). Sandstones were mined, particularly for flagstones; for example, the Elland Flags were worked in the Bradford-Halifax area.

The most important metal ore mined in Britain was iron. The sedimentary iron ores of the Jurassic system were worked in several areas, chiefly in Northamptonshire, Lincolnshire and in the Cleveland Hills. The mines were worked by the pillar and stall method. Collapses have occurred in the Northamptonshire field and at Santon Mine, Scunthorpe, small depressions probably due to void migration occurred over the shallower part of the mine. The hematite deposits of Cumberland were worked by total extraction methods which gave rise to subsidence (Fig. 11). For example, subsidence due to extraction of ore from the Hodbarrow Mine, Millom, was responsible by 1922 for the formation of four large depressions each about 400 m wide and 15 m deep.

Many other metals have been worked in Britain, notably lead and zinc in the Pennine orefield and the Lake District, and tin and copper in Cornwall. When these occur as vein deposits the workings follow the vein and are consequently narrow. In addition the host rock is invariably competent. Such workings do not as a rule produce notable subsidence problems although associated abandoned mine shafts do constitute a problem.

An example of the effects of subsidence produced by the extensive metalliferous mining, namely gold, can be taken from South Africa. Gold has been mined from quartzite reefs in the Johannesburg area since the nineteenth century (Stacey 1986). The workings take the form of extensive narrow tabular stopes with occasional pillars left to support the roof. These pillars were frequently robbed when a mine neared the end of its working life. In common with all old areas of mining activity, the records of early workings are poor and inaccurate, if indeed they exist.

Subsidence has been associated more frequently with workings at shallow depth. Nonetheless, with time, subsidence affecting the surface could occur due to movements in workings at greater depths. In fact surface subsidence has occurred in the Johannesburg area where workings are located at depths exceeding 240 m. Four mechanisms have been identified leading to surface subsidence. Three of these types of subsidence are associated with the hanging wall, that is, with its collapse, with cantilevering of the hanging wall or with its deformation. The remaining type of subsidence is attributed to block movements at fault or dyke contacts and gives rise to a step at the surface.

Gangue minerals such as fluorspar, calcite and barytes

![Fig. 10 Bell pits in the German Ironstone Band exposed at Sproats opencast site, Northumberland. The bell pits were generally 4.6 to 6.1 m in diameter at their base. (Courtesy of Roy Taylor).](http://egsp.lyellcollection.org/Downloaded from http://egsp.lyellcollection.org/)
have been and still are worked in the Pennine orefield. As far as subsidence is concerned very much the same applies to them as it does to their associated metal ores. Some subsidence due to the fluorspar working has occurred in north Derbyshire but no property has been damaged, as it affected farmland.

Mining of the Wenlock Limestone (Silurian) in the West Midlands dates back to the mid-eighteenth century, the abandoned old pillar and stall workings extending over some 1.6 km². However, the risk of subsidence damage to property in the area affected is low (Braithwaite & Cole 1986). For example, prior to 1978 about 100 surface disturbances had been noted, affecting around 50,000 m². Such a figure represents approximately 5% of the area underlain by limestone mines. The situation has been complicated in some areas by subsidence due to mining in Coal Measures above the limestone, it being difficult, if not impossible, to distinguish between subsidence due to workings in one or the other formation. Nevertheless, the collapse of Cow Pasture Mine in 1978 led to an extensive investigation being undertaken to assess the subsidence risk and to recommend remedial measures. The investigation identified two types of subsidence: firstly, crownholes have appeared at the surface above shallow workings and result from roof collapse and void migration. Secondly, general trough-like subsidence has occurred as a consequence of pillar failure. However, the limestone pillars usually are strong enough to carry the overburden load. It is the upward extension of the pillars, as a result of roof collapse, into the overlying shales which gives rise to the problem in that it is the shale ‘pillars’ which collapse. Failure of one pillar throws a greater load onto those about it which, in turn, may cause their failure. The collapse in Cow Pasture Mine affected a surface area of 200 m by 300 m and subsidence at the centre of the trough amounted to some 1.2 m.

Limestone has been, and is, mined in several other areas of Britain. Other limestones mined include the Hopton Wood Limestone (Carboniferous) which is worked by the pillar and stall method at Middleton-by-Wirksworth, Derbyshire. Although this is a strong com-
petent limestone, and the extraction ratio is about 50%, subsidence (Fig. 12) through a few metres over an appreciable area, and in the form of crownholes at the surface, has occurred. The Bath Stone (Great Oolite) has been worked in Wiltshire and Somerset, for instance, and within the Box-Corsham district of Wiltshire there are 96.6 km$^2$ of pillar and stall workings. Formerly, the extraction ratio reached 85%, but it has been reduced to around 65%. Numerous subsidences have occurred and were due to either pillar failure or void migration.

Shallow mine workings occur in the Chalk, particularly in East Anglia and Kent. Some of those in East Anglia were made by Palaeolithic man in his quest for flint. They are usually similar to bell pits, they may be about 3-6 m in height, and have small tunnels, following the flints, running from the main chamber. A dramatic collapse recently occurred in Norwich with a double-decker bus falling into old workings (Fig. 13).

A notable example of a series of subsidences in chalk occurred in Bury St. Edmunds in July 1967. The subsidence took the form of crownholes and affected Jacqueline Close (a street of terraced houses completed in September 1966) with the result that the houses were eventually declared unsafe and therefore abandoned. As far as the workings are concerned the pavement is usually between 15 and 18 m below the surface and the galleries 2 to 3.5 m high. Although the ground was potentially unstable, in that crownholes were likely to occur, the process was accelerated by site development which led to an increase in subsurface drainage.

Some of the deneholes which occur in Kent are believed to have been excavated in the Chalk during the stone and bronze ages. A shaft, from 10 to 20 or so metres in depth, generally leads to two chambers (as many as six have been noted). Numerous deneholes have been found in the Bexley area and one shaft may be within 30 m of another. The excavation from one shaft apparently does not break into those from adjacent shafts, presumably to minimise the risk of roof collapse. Many of these deneholes have six chambers, three radiating from each side of the base shaft. This is referred to as a double trefoil pattern. These deneholes are of later age than the ones previously mentioned and in some instances a primitive pillar and stall pattern evolved from the double trefoil method of excavation. Their purpose was the extraction of chalk.

Gypsum, which occurs in the Permian and Triassic systems, has been, and is, mined in various parts of Britain, for example, in Cumbria, Durham, Yorkshire, Nottinghamshire, Staffordshire and Sussex. The pillar and stall method of extraction was, and is, used. Most of the beds which were worked do not exceed 5 m in thickness and the extraction ratio is generally less than
GROUND MOVEMENTS DUE TO ENGINEERING OPERATIONS

60%, hopefully to minimize or avoid subsidence. Even so some notable subsidences have occurred, for instance, in Cumbria. Fortunately they only affected agricultural land.

Mining of salt in Cheshire was more important throughout the eighteenth century than brine pumping as far as production was concerned. But because of poor mining practices, particularly very high extraction ratios and flooding in wet rockhead areas, collapse was a frequent occurrence. The collapse of a mine produced what was called a ‘rock pit hole’ at the surface (Fig. 14). This was a large crater which quickly became flooded. Obviously the abandonment of a mine meant an added danger to its neighbours. Mines were very common in some areas, notably between Northwich station and Marstonhall mine where both the top and bottom beds were honeycombed with mines. The last catastrophic subsidence occurred in 1928 with the collapse of the Adelaide mine. This brought an end to mining in the Northwich area. Today the only salt mine still in operation is the Meadowbank mine at Winsford. The pillar and stall workings are in dry rockhead and the extraction ratio varies between 60 and 65%.

An interesting example of past mining which was not realised to be so extensive occurs in the Upper Greensand near Mersham, Surrey. Here the rock was worked for hearthstone by a crude pillar and stall method, the extraction ratio varying between 65 and 75%. The workings occur around and beneath the M23-M25 interchange. Old workings in the Basal Permian Sands at Castleford and Pontefract, and associated surface movements, have been described by Baldwin & Newton (1988).
Subsidence due to the abstraction of fluids; a note on prediction

The removal of fluids from sediments reduces the pore pressures and as a consequence the effective pressures are increased. This in turn leads to consolidation, the degree of which depends on the compressibility of the material involved. The net result is surface subsidence. For instance, over-abstraction of water from the Chalk over a period of 150 years caused subsidence in excess of 0.3 m in some areas of London (Fig. 15). In 1820 the artesian head in the Chalk was approximately +9.1 m AOD, but by 1936 this had declined in some places to −90 m AOD. The decline in artesian head was accompanied by under-drainage in the London Clay. Between 1865 and 1931 subsidence averaged between 60 and 180 mm throughout much of London.

A detailed description of subsidence associated with the withdrawal of groundwater, oil and gas, and brines is given by Bell (1988) in this volume and so will not be repeated here.

Obviously it is necessary to determine the amount of subsidence which is likely to occur as a result of the withdrawal of fluids from the ground, as well as to estimate the rate at which it may occur. Unfortunately a large number of prediction methods have been developed, some of which are relatively simple whilst others are complex. One of the probable reasons for this is that stratal sequences are different in different areas where subsidence has occurred and consequently different models have been devised. Nonetheless a number of steps can be taken in order to evaluate the subsidence likely to occur due to the abstraction of fluids from the ground. These include defining the in situ hydraulic conditions; computing the reduction in pore pressure due to removal of a given quantity of fluid; conversion of the reduction in pore pressure to an equivalent increase in effective stress; and estimating the amount of consolidation likely to take place in the formation affected from consolidation data and the increased effective load. In addition to depth of burial, the ratio between maximum subsidence and reservoir consolidation also should take account of the lateral extent of the reservoir in that small reservoirs which are deeply buried do not give rise to noticeable subsidence, even if undergoing considerable consolidation whilst, by contrast, extremely large reservoirs may develop significant subsidence. The problem therefore is three-dimensional rather than one of simple vertical consolidation. Subsidence prediction in relation to the rate of groundwater abstraction therefore should involve relating the consolidation model to a two- or three-dimensional hydrological model based on the groundwater flow equation. Variations in the hydraulic head in both time and space in response to groundwater abstraction are obtained from the hydrological model. These values then can be used to derive the time dependent consolidation curve at any point in the system. This provides an indication of the amount of subsidence likely to occur.

Like mining subsidence prediction methods, prediction methods associated with subsidence due to fluid abstraction also can be grouped into empirical, semi-empirical and theoretical categories. Empirical methods involve the extrapolation of available information such as the amount of subsidence, the amount of consolidation or the decline in fluid level being plotted against time in order to determine future trends (Figueroa Vega & Yamamoto 1984).

The semi-empirical approach also depends on the relationship between subsidence and related phenomena. For instance, Castle et al. (1969) found that subsidence in six oil fields in the United States varied more or less linearly with net production of liquid but the correlation between the decline in reservoir pressure and subsidence was poor. The most likely explanation of this poor correlation is that pressure decline as measured at individual producing wells probably is not representative of the average decline over a field. The ratio of subsidence to head decline represents the ratio between these two factors in coarse-grained permeable beds of consolidating aquifer systems over a common interval of time (Poland 1984). This ratio reflects the change in thickness per unit change in effective stress and can be used to predict a lower limit for the amount of subsidence in response to a given increase in virgin stress (stress exceeding the past maximum). If pore pressures in the consolidating aquitards reach equilibrium with those in adjacent aquifers, then consolidation ceases and the subsidence-head decline ratio represents a true measure of the virgin compressibility of the system. Until equilibrium of pore pressure is attained, the ratio of subsidence to head decline is a transient value. Contours of the ratio of subsidence to head decline for a given period of time can be plotted.

![Fig. 15 Lines of equal subsidence (1865-1930) due to abstraction of water from the Chalk beneath London: the contours are in tenths of a foot. (After Wilson & Grace 1942).](http://egsp.lyellcollection.org/Downloaded from)
on a map and indicate the amount of head decline required to produce a particular magnitude of subsidence throughout the area concerned.

As remarked above, the abstraction of fluid from the ground reduces the pore pressure which leads to a transfer of load to the granular skeleton and to its subsequent reduction in volume. However, in trying to develop an explanation of the phenomenon one encounters the problems associated with multi-phase systems representing solids, liquids and gases, the properties of which must be inferred from statistical averages or from representative tests. The materials concerned include in their mechanical properties the combined behaviour of their individual components (that is, elasticity and plasticity of solids; viscosity of liquids; compressibility of gases; decay of organic matter; attraction and repulsion of ionic charges etc.). Hence the mechanical properties are anisotropic, as well as dependent on stress history and time. Accordingly such material is difficult, if not impossible, to deal with in a theoretical model of subsidence. Resort has to be made to simplifying assumptions in order to develop any type of model of subsidence prediction, in particular, the properties of the ground must be idealized. Other simplifications may include the assumption that strata are horizontal; that flow in aquifers is horizontal whilst it is vertical in aquitards; and that subsidence primarily is due to consolidation of aquitards. Obviously the greater the number of simplifications which are incorporated in a model, the more restricted its use becomes.

Subsidence above water, oil or gas reservoirs frequently is treated as a mechanical problem involving the elastic behaviour of the reservoir and is analysed with the aid of a mathematical model. Unfortunately the amount of data available frequently is limited which means that, as remarked above, certain assumptions have to be made before the model can be applied. For example, Geertsma (1973) simulated the subsidence above the Groningen gas field by assuming a homogeneous and isotropic semi-infinite porous medium. The reservoir was assigned an idealized shape, that is, a horizontal circular cylinder of limited thickness. The ratio between maximum subsidence and reservoir consolidation was governed by the ratio between depth of burial and the lateral extent of the reservoir. In the same year Gambolati & Freeze (1973) developed a mathematical model to simulate subsidence due to groundwater withdrawal at Venice. First, with the aid of the finite element technique, the regional drawdowns in hydraulic head were determined in a two-dimensional vertical cross section in radial coordinates, using an idealized ten-layer representation of the geological conditions. Then the values of the hydraulic head calculated for the aquifers were used as time dependent boundary conditions in a set of one-dimensional vertical consolidation models solved by the finite difference technique and applied to a more refined representation of each aquitard. Subsequently Gambolati et al. (1986) developed a method of analysis of subsidence due to the withdrawal of oil and gas from a reservoir overlain by layered anisotropic soils by using the finite element technique. They assumed a disc-shaped reservoir of uniform thickness, which underwent elastic deformation. The model consisted of layered anisotropic soil units which were characterized by five elastic constants. Alternating sands and clays were assumed to occur above the reservoir, their compressibility progressively decreasing with depth. They found that for a given geometry, depth of burial and fluid pressure decline, that subsidence was basically related to the compressibility of the oil-gas bearing strata and of the adjacent overlying/underlying clays. The depth at which a rigid basement occurred beneath a reservoir appeared to have only a limited influence on ground movement.

For a further discussion of subsidence prediction see Saxena (1979) and Johnson et al. (1986).

**Investigation of ground movements**

Ground movements are usually only acceptable if they are less than those predicted, and within tolerable limits. Ground movements which occur during and following construction can be anticipated from the ground investigation and sample testing programme for the site and allowed for in the foundation and structural design. In the case of many small and lightweight structures no attempt will be made to monitor constructional movements. However, for larger and more expensive structures, the anticipated ground movements will be investigated by the installation of a range of instruments designed to measure any movements or changes in pore water pressures. This may be particularly important where a new structure is built close to existing ones as, for example, in the construction of underground urban railway systems (Shirlaw et al. 1988).

The situation with regard to the investigation of ground movements associated with mining is rather different. Such ground movements may be of two types; those associated with the gradual deterioration or sudden collapse of old mine workings where the extent of the workings may be only partially known, and those relating to current mining which can be largely predicted and controlled and where the resultant damage is allowed for in the costing of the mining. In the case of old workings, the investigation work involves the locating of the extent and size of the workings, the determination of their condition and the monitoring of either roof falls or the effects of remedial measures on controlling ground movements (Bell 1986, Culshaw & Waltham 1987). For modern mining, monitoring of the associated ground movements is necessary to ensure that subsidence is of the magnitude anticipated.

**Instrumentation for monitoring ground movements**

Hanna (1985) has described instrumentation systems for the monitoring of deformation of the ground in some detail. However, despite the increasing sophistication of the various instruments employed, it remains true that, in general, success in monitoring is most likely if
the instrumentation has suitable resolution, is reliable in use, is rugged enough for its intended purpose and is properly installed. Various types of instrument are used to measure accurately settlements and heaves, lateral movements, positional changes and dimensional changes. The principal types of instrumentation described by Green (1974) for monitoring ground movements included optical survey, fluid settlement gauges, rod and wire extensometers, inclinometers and electronic tiltmeters.

Optical surveying methods are used for levelling, triangulation, trilateration and alignment. The use of electronic distance measurement or laser equipment, in particular, provides accurate results (Penman & Charles 1974). The design and construction of reference points is particularly important so that stable stations are installed outside the area of movement and that those within it are not damaged or unduly affected by environmental factors.

Fluid settlement gauges use an air/water or mercury column to transmit pressure over a distance to enable the elevation of a point to be measured relative to a benchmark and remotely from it. They are installed where it is not practicable to use a vertical reference rod, for example, beneath a road. Green (1974) pointed out that the main difficulties associated with this type of gauge relate to problems of de-airing, variations in temperature, the effect of flow rate variation, the vulnerability of pressure transducers and mercury column breakdown. Difficulties also arise where large level differences occur between the measuring tip and the recorder.

Borehole extensometers record the vertical displacements of a series of anchor points located at increasing depth within a borehole. Some can be used in horizontal and inclined boreholes as well as vertical ones. The accuracy of extensometers is affected by environmental factors, and instruments, particularly surface access points, may be damaged easily during or after construction. Other potential problems described by Green (1974) include the avoidance of anchor slip for borehole installations, especially in soft soils, the possible inaccuracy of marked electrical cables and the lack of convenience when using steel tapes.

The attitude of a borehole can be measured by an inclinometer. Such results can be affected by a number of factors. For example, the precision of the pendulum system is largely controlled by the principle of tilt measurement used and by the level of maintenance and calibration (Hanna 1985). Other problems include spiralling of the casing resulting in the grooves at depth having different orientation than at the surface, lack of repeatability of the reading position, sensitivity of the instrument to climatic changes, and operator error.

Electrical tiltmeters (or electrolytic liquid levels) are used to measure the inclination to the horizontal of the ground (Cooke & Price 1974). From the results, precise measurements of settlement, shear movement, foundation rotation or distortion, low frequency ground movements or structural sway can be made (Sherwood & Currey 1974). Errors in the use of this type of instrument have been described by Forrester (1974) as those relating to the positioning of the instrument, to the contamination by dirt of the contact between the instrument and the measuring station, and to the nature of the electro-level instrument itself, and the associated electronic circuitry.

**Monitoring of movements caused by construction**

Monitoring of surface movement can be done by conventional surveying techniques. However, Burland et al. (1977b) indicated that stable reference points must be set up at a distance of at least three times the depth of an excavation away from its perimeter when monitoring movements.

Precise results also can be obtained by using close-up photographs taken from ground stations which are then measured in a stereocomparator. Movements may be revealed by examination of a sequence of photographs taken at suitable intervals of time (Burland et al. 1977b).

Movements which take place within the ground also should be investigated. Subsurface movements can be recorded using the settlement gauges, extensometers and inclinometers described above. Magnet extensometers have been widely used for measuring vertical movements outside, and also beneath, excavations. They also have been installed in horizontal boreholes drilled into the faces of excavations.

Burland et al. (1972) noted that magnet-extensometers were suited for monitoring heave at various depths beneath excavations. A problem which is sometimes associated with the excavation of deep basements is that of base failure due to hydraulic uplift. In such situations magnet extensometers have been used in conjunction with piezometers to monitor the stability of the floor.

Pore-water pressure measurements are used to check assumptions made in effective stress analyses and to monitor the effectiveness of drainage and dewatering schemes. The different types of piezometer used for these measurements have been described by Vaughan (1974). The simplest type comprises a standpipe installed in a borehole. If the minimum head recorded is less than 8 m below ground level ‘closed system’ piezometers, connected to mercury manometers are normally used. Pressure transducers are necessary where greater heads have to be measured, especially in low-permeability ground. These instruments respond to pressure changes acting on a flexible diaphragm by recording diaphragm deflections which are then converted into pressure values. Only very small quantities of water are required to produce full-scale deflection and such piezometers are therefore especially helpful when an almost instantaneous response is wanted.

Settlement surveys of the surrounding buildings, using precise levelling, were carried out throughout the construction period for the underground car park at the House of Commons (Burland & Hancock 1977). Any changes in the verticality of the Clock Tower were recorded by an autoplumb. Inclinometers were used to monitor horizontal deflections of the diaphragm retaining wall about the excavation. The heave was measured...
at various depths during the excavation period, two borehole extensometers having been positioned near the centre of the excavation before operations commenced. A number of standpipe and pneumatic piezometers were installed in boreholes, and were used along with the extensometers to check that hydraulic uplift did not develop.

**Investigations in areas of mining subsidence**

There are four elements in the investigation of subsidence areas:

(i) the desk study
(ii) indirect subsurface exploration
(iii) direct subsurface exploration
(iv) monitoring of movement

The desk study includes a survey of appropriate maps, documents, records and literature. Geological maps indicate the probable positions where coal seams outcrop and these, together with Ordnance Survey maps, may show the positions of old shafts and adits. All geological and Ordnance Survey maps of the area in question, going back to the first editions, should be looked at. British Coal and the Abandoned Mines Record Office represent primary sources of information relating to past mining activity in Britain. Other sources include county record offices, public record offices, museums, libraries, private collections and the British Geological Survey. Where records of past mining are available, they must be treated with some reserve as they are frequently inaccurate and incomplete. This is particularly the case for records dated before 1872.

The use of remote sensing imagery for the detection of surface features caused by subsidence is more or less restricted to rural areas and scale is a critical factor. The resolution necessary for the detection of the relatively small subsidence features (1.5 - 3 m across) is provided by aerial photographs with scales between 1:30 000 and 1:10 000 (Russell et al. 1979). False colour may be more effective than black and white photographs since it reveals subtle changes in vegetation related to subsidence. Scales larger than 1:10 000 may prove useful for evaluating structural damage to buildings.

Except in special circumstances, the common methods of geophysical exploration have not proved very successful in detecting and revealing the layout of shallow abandoned mine workings. Seismic refraction has not been used particularly often in searching for voids created by previous mining in shallow coal seams since such voids are generally too small to be detected by this method. What is more, the detection of subsurface voids becomes even more unlikely if they are situated at a depth greater than three times their diameter or if their diameter is less than several geophone spacings (which are commonly not less than 1 m each). The depth limitation is due to the attenuation of waves in the rock overlying the void.

After extensive field work, Maxwell (1975) concluded that, except for workings with a depth of cover of less than 5 m, it was unlikely that resistivity profiling would detect the presence of dry pillar and stall workings. Moreover, if the coal seam possessed an infinite resistance, workings could not be detected. This conclusion was supported by Burton & Maton (1975), who indicated that where the depth of a cavity is equal to its diameter the maximum disturbance in the resistivity profile is only about 10% of the background level. As noise frequently exceeds this value, cavities at depths greater than twice their average dimension are usually not recorded.

According to McDowell (1981) terrain conductivity meters have several advantages over conventional electrical resistivity equipment when used for locating small near-surface anomalous features.

Burton & Maton (1975) maintained that voids in shallow abandoned mine workings were generally too small and located at depths too great to be detected by normal magnetic or gravity surveys. Subsequently, however, McDowell (1981) referred to the fluxgate magnetic field gradiometer that permits surveys of shallow depths to be carried out. Quantitative analysis of the depth, size and shape of an anomaly generally can be made more readily for near-surface features using the gradiometer than from total field measurements obtained with a proton magnetometer. On the other hand, a proton magnetometer can more easily detect larger and deeper features and yields results which are more suitable for contouring. A microgravity method is capable of locating medium-size cavities which are near the surface and larger ones at greater depth.

Recently, a number of indirect techniques have been developed which have had some success in subsurface exploration. Of these techniques, ground-probing radar probably has had the most success. Indeed, according to Benson (1979) ground-probing radar is capable of detecting small subsurface cavities directly.

Most of the geophysical methods have a down-the-hole counterpart which can be used to log a hole. Unfortunately these methods generally suffer the disadvantage of limited penetration. In inter-borehole acoustic scanning an electric sparker, designed for use in a liquid-filled drillhole, produces a highly repetitive pulse. This signal is received by a hydrophone array in an adjacent drillhole, similarly occupied by liquid (McCann et al. 1975). The method can be used to detect subsurface cavities if the cavity concerned is directly in line between two boreholes and has at least one-tenth of the borehole separation as its smallest dimension. Air-filled cavities are more readily detectable than those filled with water.

Down-the-hole radar probes also have been used to detect underground voids and according to Rubin & Fowler (1978) they can be used from drillhole to drillhole.

Most of the geophysical methods used for the detection of cavities have been summarised by McCann et al. (1987) in relation to natural cavities.

The only way to prove the existence of a cavity from the surface is by exploratory drilling or probing, the locations of holes being influenced by data obtained from the desk study and any indirect exploration...
methods. However, it must be admitted that exploratory drilling is frequently unable to establish the layout of old mine workings although some site investigations have been relatively successful. For example, Carter et al. (1981) described a site investigation at Bathgate where almost half the drillholes encountered voids at seam level.

Drilling to prove the existence of old mine workings is frequently done by open holes, which allows relatively quick probe drilling. The stratal sequence should be established by taking cores in at least three drillholes. If a grid pattern of drillholes is used some irregularity should be introduced to avoid holes coinciding with pillar positions. Furthermore the grid axes should not run along the dip and strike directions of the coal seam(s) involved.

Below-surface workings may be examined by using borehole cameras or closed-circuit television, information being recorded photographically or on videotape. However, their use in flooded old workings has not proved very satisfactory. Occasionally smoke tests or dyes have been used to aid the exploration of subsurface cavities.

Access to shallow abandoned coal mines is rare. Consequently, if old workings which have remained open are to be explored directly, access must be gained either by driving a heading from the outcrop of the seam, if this is close at hand, or by sinking a shaft to the coal seam. When such workings are entered, strict safety precautions must be adhered to because of the possible dangers resulting from their partially collapsed or partially flooded condition or from the presence of noxious or explosive gases.

The monitoring of ground movement due to mining subsidence utilizes many of the instruments used for monitoring ground movement in relation to construction (Longworth 1988).

Measurement of bedding separation due to subsidence movements can be carried out either from holes drilled from the surface or drilled underground. The amount of such separation in the roof rocks above a mine can be assessed by a multiple point borehole extensometer.

The wireline method of monitoring strata movement makes use of electronic logging devices. In the bullet perforation technique, as described by Peng (1978), a bullet perforator is lowered down a drillhole, usually uncased, to given positions and bullets containing radioactive material are electronically fired into the wall. This allows any movement in the strata to be monitored.

Peng (1978) also described the time domain reflectometry method. In this technique a drillhole is sunk to the coal seam in question and a cable is grouted in place throughout the length of the hole. Movement of the roof strata may break or create faults in the cable. When an electric pulse is sent down the cable, it is partly reflected back to the source by any faults. The time delay between the initial signal and the arrival of the reflected signal, multiplied by the signal velocity, indicates the depth at which the 'fault' occurs.

Another method of monitoring movement involves grouting a weight in a given position in a drillhole. The weight is attached by a wire, via a pulley, to a counterweight at the surface. Movement of the rock to which the weight is anchored is recorded by measuring the movement of the counterweight at the surface if several anchors are installed.

A review of the various types of extensometers which have been used to measure strain developed due to ground movements has been provided by O'Rourke et al. (1979). For example, a full profile borehole extensometer with electromagnetic torpedo, housed in a probe, provides a means of measuring subsurface vertical deformation. The probe detects the positions of wire rings around flexible non-magnetic borehole casings which deform with ground movements. Depth is measured by the length of cable played out. A tape extensometer can be used to measure the surface strain and tilt of the ground, as well as mine level convergence.

Subsurface horizontal deformation can be determined by a full profile borehole inclinometer with servoaccelerometer torpedo. Horizontal displacements are recorded as the inclinometer is raised up the drilling casing. Tilt is measured along two orthogonal axes in the plane perpendicular to the inclinometer.

### Induced seismicity and underground movements

Ground movements are inextricably linked with the build-up of stresses in, and the consequent straining of, rock masses. A rock mass usually consists of both solid and fluid components, and the local stress patterns developed in the mass may be a consequence of the removal or influx of either, or both, of these phases. The transition of the fluid phase into the gaseous phase may additionally complicate the problem. The size of the ground movement associated with solid, liquid or gas removal or influx may be macroscopic, and hence easily noticed at the surface, or microscopic in which case only precise instrumentation of the area will detect the change in conditions. All such ground movements will tend to be accompanied by the release of energy in the form of elastic (seismic) waves. This induced seismicity radiates away from the source of the disturbance and may be monitored using microseismic or acoustic emission techniques, or, where the energy released is large, conventional seismic detection methods.

The release of stored strain energy as elastic waves can be investigated in two ways:

(i) by the passive monitoring of a rock mass in an attempt to predict stress build up and hence either avoid unnecessary dangers or provide a record of the stress–strain behaviour (for example, in areas of active or disused mines);

(ii) by the monitoring of the released elastic energy to provide accurate location information on the nature of the induced deformation produced by...
the active stressing, in addition to information on the in situ stress regime.

Within this framework, induced seismicity has many specific applications to engineering geological situations; these have been reviewed in detail by McCann (1988).

Mining activities

These include the monitoring and prediction of in situ stress conditions, together with failure, in both dormant and active mining situations. Styles & Emsley (1988) reported an excellent correlation between microseismic responses and the patterns of daily extraction/levels of activity at Cynheidre Colliery, South Wales. Redmayne (1988) described the monitoring of mining induced seismicity in British coalfields using the British Geological Survey National Seismograph Network. This latter activity involved earth tremors with magnitudes up to 2.8 ML and was felt strongly locally. Further documentation of the relationship between longwall coal extraction and seismicity was given by Kusznir et al. (1980); the sources of the earth tremors moved in unison with the working face and had two possible mechanisms identified from the character of the received seismic signals. The larger events, with magnitudes of approximately 3.0 ML, were suggested as being generated by implosional source mechanism suggesting that they were active coal face. The smaller tremors, however, had an implosional source mechanism suggesting that they were generated by strata collapse. In contrast to this coal environment, in the gold mines of South Africa rock bursts, involving the sudden release of substantial amounts of strain energy, are a major problem (Salamon 1983).

Similarly, though less frequent and perhaps less well defined events have occurred in British coal mines; Styles & Emsley (1988) described outbursts of gas and coal in South Wales anthracite mines. Disused mines, such as the limestone workings in the West Midlands, often cause engineering problems through deterioration with time following abandonment. Recent studies of this area (McCann 1988) have shown that substantial ground movements do occur, often with the associated development of surface features. Seismicity generated in connection with this particular area, however, seems to be confined to the impact of material, derived from roof falls, with the mine floor, rather than with rock failure in the roof. This is attributed by McCann (1988) to the clay infilling of joints inhibiting the development and propagation of the elastic waves.

Fluid pumping

This includes sub-surface fluid storage and extraction, the disposal of waste fluids through forced pumping and hydrofracturing studies. The connection between man’s handling of pore-fluids and the onset of additional seismic events was first noted in 1965 in the area around Denver, Colorado, USA. The disposal of chemically-contaminated water into a well drilled to a depth of 3671 m initiated additional earth tremors. Evans (1966) noted a direct correlation between the periods of earthquake activity and the periods of pumping. This accidental observation prompted a controlled experiment at an oilfield near Rangely, Colorado. Here water was pumped into a reservoir to maintain the pore pressures. In a lengthy experimental study involving several different pumping phases, a moderate degree of correlation was obtained between the number of earthquakes and the pumping phases. More important than this, however, was the suggestion that there was a threshold pore fluid pressure which was required for the earthquake activity to rise above background levels (Bolt 1988). Further evidence of the connection between fluid pumping/pressure and seismic response was obtained during the depressurization of an oil storage cavern, located in a salt dome near Freeport, Texas (Albright & Pearson 1984). A small change in the internal pressure of the cavern led to a significant increase (19) in locatable microearthquakes which peaked after five days. This suggested to the authors that the pressure change might have been sufficient to cause failure in the rock salt near the cavern walls.

All of these cases demonstrate the use of seismic monitoring to evaluate the secondary effects of fluid pumping. Conversely, the pumping of fluids may be used to initiate and propagate fractures in a rock mass, the subsequent seismicity associated with this activity then being used to monitor and locate the strain deformation. Perhaps the most successful documented case is that of the Camborne School of Mines Geothermal Energy Project (UK). Here, a hot dry rock geothermal reservoir has been created at a depth of 2 km in the Carnmenellis granite by fluid injection. Microseismic monitoring is used to locate the source of the emission (Baria et al. in press), and fault plane analysis is used to evaluate the in situ stress conditions (Green et al. 1988).

These results suggest that the fluid acts in two distinct ways in triggering the seismic emissions. Firstly, it may behave as a lubricant, once it has crept into microcracks and fissures. Secondly, it can increase the pore fluid pressure, reducing the shear strength of the rock; this may trigger the release of energy in pre-strained rocks.

Reservoirs

The loading effected by filling a large volume at the ground surface with water is not considered to be sufficient alone to cause a seismic response, except where the rock mass supporting the load is already in a critical state (McCann 1988). Additional factors may be important, particularly where there is some seepage of water into the underlying ground, and the mechanisms contributing to the ground movements may be similar to those involved in fluid pumping situations.

Whilst the first observation of an association between the filling of a reservoir and induced seismicity dates back to the Lake Mead case history (Carder 1945), one
of the most documented examples is that of Koyna Reservoir in India, where the environment is characterized by wrench faulting close to the Indian continental divide (Snow 1982). Following the first filling of Koyna Reservoir during 1962, small felt earthquakes were common. The first tremor to cause concern for safety was of magnitude 5.8 (Richter Scale) in September 1967 and was felt up to 120 km away. This was followed, three months later, by an earthquake of magnitude 7.0. This was accompanied by considerable destruction of local settlements, and cracked the concrete gravity dam from face to face below the waterline. The earthquake was judged to be very shallow with an epicentre some 10 km north of the dam. Snow (1982) stated that no other comparable structure has been so nearly destroyed by man-made earthquakes as Koyna Dam.

Evidence suggests that Koyna was already in a critical stress state prior to the construction of the dam (Snow 1982). Consequently the conditions were approaching those necessary for a seismic response to be initiated. It is likely that in most cases of reservoir induced seismicity the original ground conditions must be close to the critical stress state for the impounding of the dam to provide the necessary triggering mechanism.

**Thermal stress**

The proposed disposal or storage of high level radioactive waste materials has led recently to studies of the geotechnical properties of possible repository formations. One such field study by Majer et al. (1984) investigated the mechanical behaviour of hard crystalline rock subject to heating conditions similar to those predicted for a concentration of high level radioactive waste. They noted that the heating caused the rock to be locally stressed, resulting in a series of discrete microseismic events occurring in a previously quiet zone. Microseismicity associated with high level nuclear waste disposal is, therefore, a potential problem for the future, and one that has received little consideration to date.

**Discussion**

The release of energy as seismic waves or acoustic emission represents the final stages in the sequence of events associated with ground movements. Essentially there is a build up of stress within the rock mass on either side of some discontinuity. The rock mass initially behaves as an elastic body and strains occur. Eventually, the rock mass, which resembles a spring and stores the elastic energy associated with the straining, reaches its yield point and gives way, releasing the stored energy. The release of energy may be fairly discrete or it may be composed of a series of foreshocks and aftershocks, either side of the main event; this will depend on the size of the source volume, the characteristics of the formation, and the nature of the local stress regime. The build up of stress leading to a critical state may be due to many factors, as may the modification of the local stress regime which effectively induces seismicity.

Within this framework, however, the role of water may be particularly important.

The application of microseismic/seismic monitoring to ground movements covers a wide range of situations from its use as a simple monitor of the *in situ* stress distribution to the location and evaluation of actual deformations at depth. In all cases it is important to have a record of the background seismic signature of a region prior to the main investigation (which was unfortunately not so for the Denver earthquakes or the Rangely experiment) and a clear understanding of the local geology and expected stress regime. Where fluids are involved, the time for the dissipation of a substantial pressure pulse may be considerable and the effects may lag well behind the initial event. Care should be taken, therefore, to avoid rapid changes in the overall stress regime of the rock mass.

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