Subsidence Engineers' Handbook

NCB

National Coal Board
Mining Department 1975

Subsidence Engineers' Handbook

NCB

National Coal Board
Mining Department 1975

Foreword

In this revised issue of the Subsidence Engineers' Handbook (which replaces the 1966 edition), some sections have been rewritten with up-to-date information, and new sections have been included to cover the design of mine layouts to minimise subsidence.

It is intended as a reference book for those engineers and surveyors who are concerned with subsidence engineering and who are acquiring a specialist knowledge in the subject. Very little general explanatory material has been included – this can be found in other publications – and therefore the handbook is not suitable for background reading by non-specialist engineers or surveyors.

Like the first issue, this revised edition attempts to satisfy the need for a method of predicting subsidence and its associated phenomena of slope, strain and surface damage at any given site with accuracy and speed. To this end it presents, in concise form, relevant information from various published papers, and shows how subsidence problems may be fully resolved. It does not replace the literature on the subject which must be studied for fuller knowledge to be gained.

Contents

Section 1: Symbols, Terms and Definitions

Many of the terms in the following list are used only in research and not in everyday subsidence engineering work, but they are included because they are all closely associated with factors in everyday use.

Some strata control terms are also included to cover the liaison between the two disciplines.

Figs. 1 and 2 illustrate some of the symbols which are listed.

Other symbols, i.e. those used in the interpretation of mine plans and geological plans, will be found in the Board's Code of Surveying Practice.

Symbols	Terms	Definitions
	Abutment	The region adjoining an excavation which is normally subjected to increased stress as a consequence of the redistribution of load. This may be at the front of, or at the sides or rear of, a working face.
	Abutment pressure	The load per unit area on an abutment.
β	Angle of break	The angle of inclination of the line connecting the edge of the workings and the point of maximum lengthening.
α	Angle of dip	The apparent dip of the seam.
γ	Angle of draw (limit angle)	The angle of inclination from the horizontal of the line connecting the edge of the workings and the edge of the subsidence area.
ζ	Angle of draw (limit angle)	The angle of inclination from the vertical of the line connecting the edge of the workings and the edge of the subsidence area.
	Arch core	The relaxed strata within a pressure arch.
	Block	An area of coal worked between predetermined boundaries.
	Convergence	Movement of roof and floor towards each other after removal of mineral. The rate of convergence is measured as either: (1) the convergence for a given advance of the face; or (2) the convergence in a given time.

Symbols	Terms	Definitions
	Convergence (initial)	The amount of convergence of the roof and floor in the face directly after working and prior to stowing.
	Cover (see also *depth*)	The strata lying between the surface and the seam.
	Cover load	The dead load of the cover.
	Critical Area	That area the working of which causes the complete subsidence of one point on the surface.
	Sub-critical area:	An area of working smaller than the critical area. (When a sub-critical area is being worked, the point on the surface under examination does not undergo complete subsidence.)
	Super-critical area:	An area of working greater than a critical area. (An area on the surface undergoes complete subsidence.)
R	Critical area (radius of)	
ρ	Curvature (radius of)	The radius of curvature of any part of a subsidence profile.
h	Depth	The vertical distance between the floor of the seam being extracted and any arbitrary datum – usually the surface.
v	Displacement	The horizontal displacement of a point caused by any working.
V	Displacement (maximum)	The maximum horizontal movement of a point caused by working a sub-critical area.
V_{max}	Displacement (maximum possible)	The maximum horizontal movement of a point caused by working a critical area.
$v_x v_y$	Displacement in the direction of co-ordinate axes.	The partial displacement in the direction of the axes of a co-ordinate system, on which the calculations are based.
v_r	Displacement (spatial)	The rectilinear connection of the initial and final positions of a point moving in space. (It is determined by direction, inclination and distance.)
d	Distance	Any horizontal distance.
	Face	1. Strictly, any surface exposed by excavation for development or for the getting of mineral; 2. More generally, the supported area in the vicinity of the place at which mineral is worked.
	Face length	The distance between rib-sides or goaf-sides of a coal face.
	Face width	The distance between the line of coal face and the waste edge.
	Panel	The area of coal extracted by a longwall face.
L	Panel length	The dimension of a panel measured in the direction of the line of face advance.
w	Panel width	The transverse distance across a panel, usually equal to the face length.
	Pillar	A block of coal or mass of ore left unworked.
p	Pillar width	The shortest dimension of a rectangular pillar.
	Pressure arch	A zone of increased stress surrounding the relaxed strata which is created when any excavation is made, resulting in the redistribution of the normal strata pressures on to the abutments.
	Profile (longitudinal)	A curve depicting subsidence on a section drawn parallel to the direction of advance of an underground excavation.

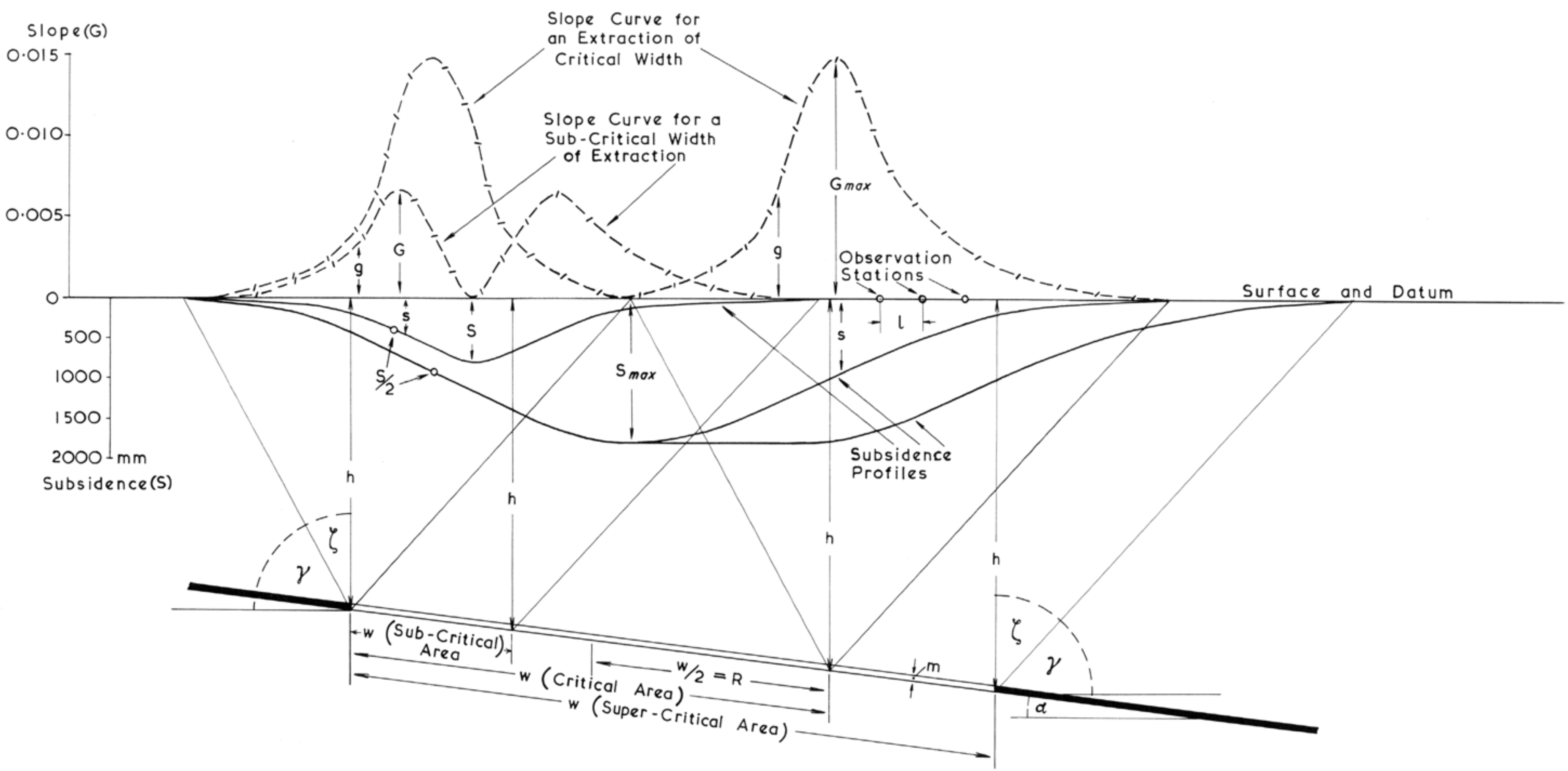


Fig. 1 Typical section through workings, illustrating standard symbols for subsidence and slope.

Symbols	Terms	Definitions
	Profile (transverse)	A curve depicting subsidence on a section drawn at right-angles to the direction of advance of an underground extraction.
g	Slope	The slope of any part of a subsidence trough. NOTE: All forms of slope are calculated as the difference in subsidence between two points close together divided by their distance apart, and expressed in parts per thousand.
G	Slope (maximum)	The maximum slope in a subsidence trough caused by working a sub-critical area.
G_{max}	Slope (maximum possible)	The maximum slope in a subsidence trough caused by working a critical area.
l	Station interval	The horizontal distance between two observation stations.
$\pm e$	Strain	Change in length per unit of length $e = \frac{\Delta l}{l}$ (Direction to be specified)
$\pm E$	Strain (maximum)	The maximum strain (lengthening or shortening) in a trough.
$\pm E_{max}$	Strain (maximum possible)	The maximum possible strain (lengthening or shortening) in a trough.
s	Subsidence	The vertical movement anywhere within a subsidence trough.
S	Subsidence (maximum)	The maximum vertical movement caused by working a sub-critical area.
S_{max}	Subsidence (maximum possible)	The maximum possible vertical movement of a point on the surface caused by working a critical area. (This corresponds, in general, to the seam thickness multiplied by the subsidence factor.)

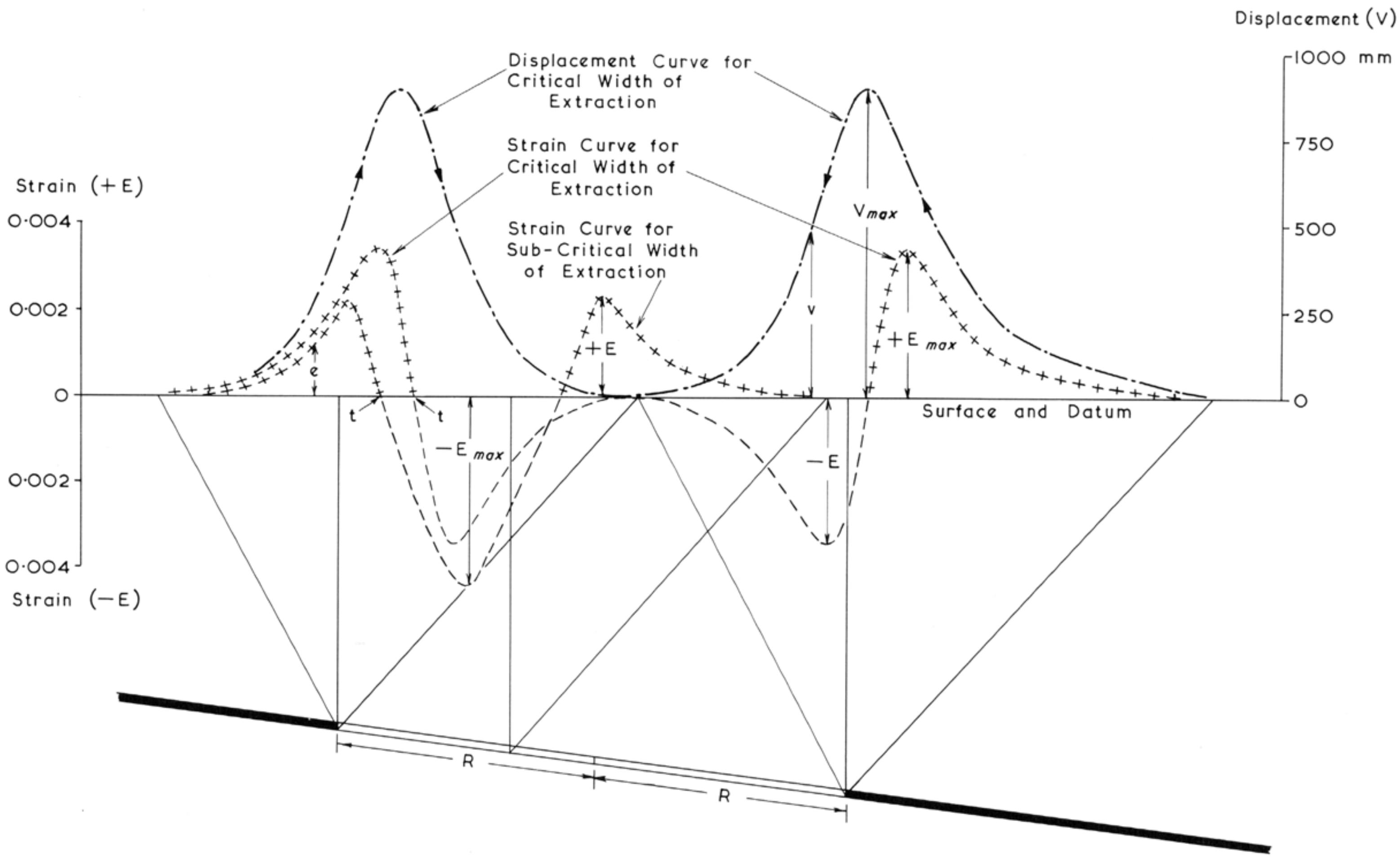
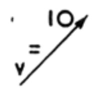
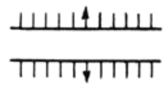
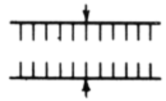


Fig. 2 Typical section through workings, illustrating standard symbols for strain and displacement.

Symbols	Terms	Definitions
$\frac{S}{2}$	Subsidence (half)	The point where subsidence has half the amplitude of that of the maximum in the profile. (This coincides with the transition point 't'.)
a	Subsidence factor	The ratio of complete subsidence to seam thickness, i.e. S_{max}/m.
	Subsidence development	The manner in which surface subsidence starts increases and finishes in relation to the position of the advancing face of an underground excavation.
	Subsidence development curve	A line on a graph depicting the changing subsidence of a surface point in relation to the position of the face of an excavation passing through the critical area of that point.
m	Thickness (of seam) extracted	
	Time-Subsidence curve	The values of subsidence of a given point on the surface plotted against time as a base.
t	Transition point	The point of transition between concave and convex curvature of a subsidence profile.
	Trough (subsidence)	The surface depression caused by subsidence.

Signs for Plans and Sections of Ground Movements

Name	Shown	Colour
Subsidence curve	———————	Black
Displacement curve		Black
Lengthening curve	+ + + + + + + +	Black with red verge
Shortening curve	- - - - - - - - - - - -	Black with blue verge
Slope curve	– –/– –/– –/– –/– –	Black
Curvature curve	– – U – – U – – U – – U – –	Black
Lines of equal subsidence	——— 200 ———	Black
Displacement directions and values		Black
Lengthening zone		Red verge
Shortening zone		Blue verge

Section 2: Prediction of Maximum Subsidence

The maximum subsidence S in any subsidence profile is related to the width w and depth of the extraction h, to both of which it is directly proportional. In cases where longwall workings are caved or strip packed it may be predicted by reference to Fig. 3 which has been compiled from numerous field observations. (Solid stowing has a moderating effect on the amount of subsidence in proportion to its efficiency; the most efficient solid stowing in Great Britain has been pneumatic stowing which can reduce the subsidence by 50%).

It is important to realise that the curves were developed from actual cases in which certain limiting conditions existed, and thus they can be used for prediction only in similar limiting conditions. These are:

(1) where the working panels extend for a distance of about 0.7 times the depth in front of and beyond the surface point where subsidence is to be predicted; i.e. the total length of the panel (L) must be about 1.4h;

(2) where the working panels have no centre gates or other zones of special packing apart from those at the main and tail gates; and

(3) where the sides of a panel are not parallel (owing to faulting, etc.) the average panel width must be developed.

Prediction of subsidence by the above method should be correct to $\pm$ 10% in the great majority of cases.

When a panel does not travel sufficiently far to cause maximum subsidence – i.e. where the extraction length is less than about 1.4h – the subsidence predicted by the above method must be reduced according to the proportion of maximum subsidence which the limited face advance permits.

This reduction is best calculated by considering the distance travelled by the face as a fraction of the depth and applying the appropriate partial subsidence curve as shown in Fig. 4 for a range of depths.

The distance travelled, as a fraction of the depth, is read off the horizontal scale in Fig. 4 and the subsidence already predicted is multiplied by the factor now read off on the vertical scale.

Example Calculation

Assume that a caved face travels a distance of only 150m in a seam 300m deep, the panel being 180m wide and the seam 1.5m thick. What is the maximum subsidence?

From Fig. 3, when $w = 180$m and $h = 300$m, then $S/m = 0.61$ $\therefore$ $S = 0.61 \times 1.5 = 0.915$m.

This is the maximum subsidence for an extraction length $L = 1.4h$ (i.e. 420m).

The correction for the limited face advance is:

$$\frac{\text{advance}}{h} = \frac{150}{300} = 0.5$$

Referring to Fig. 4, this value of 0.5 on the bottom scale gives an s/S value which is 0.51.

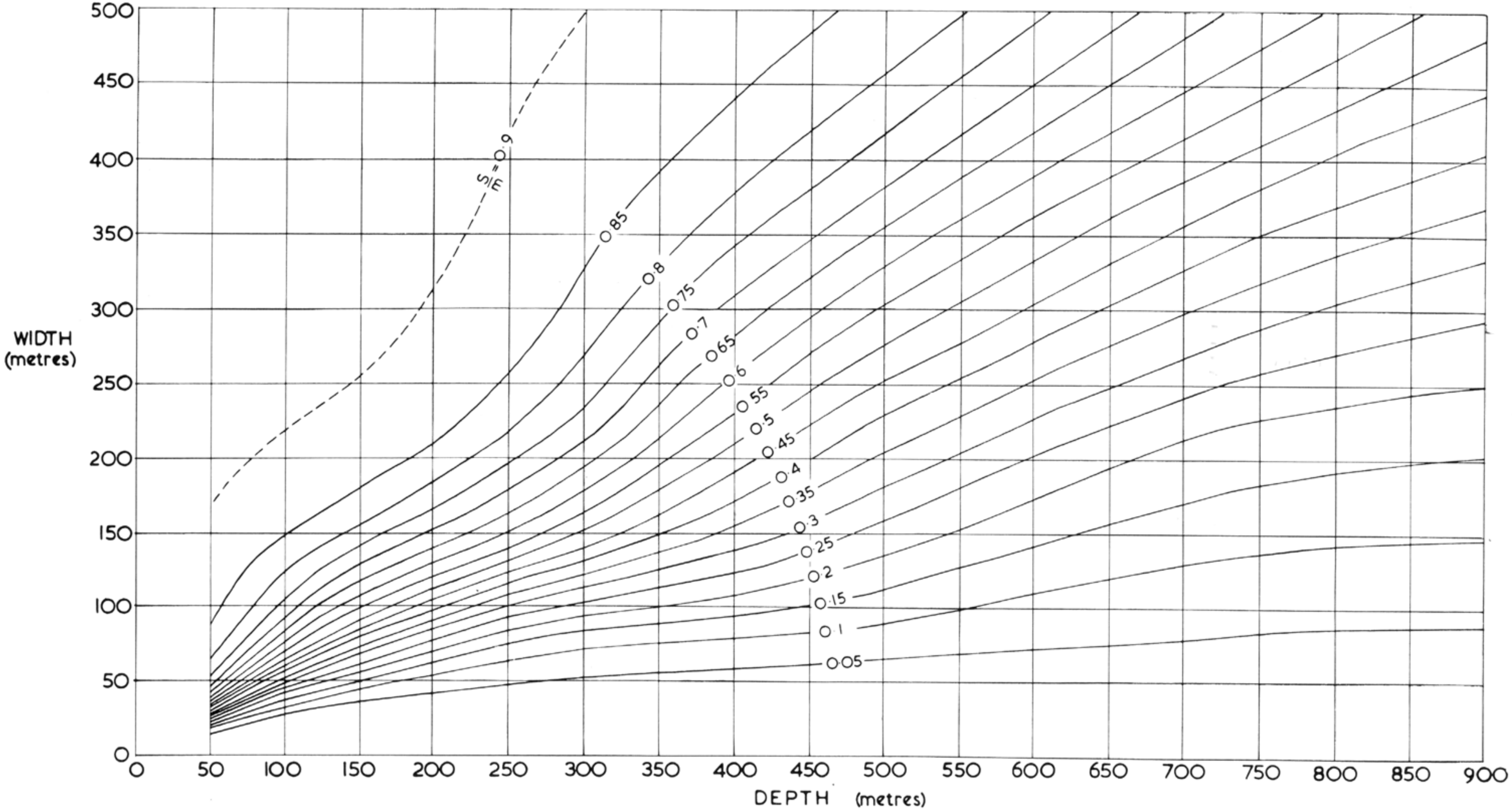


Fig. 3 Relationship of subsidence to width and depth.

Therefore, the reduced S for the limited face advance $=0.915\text{m}\times 0.51=0.467\text{m}$.

This is the maximum subsidence (S) over the centre of the rectangle of goaf.

In some panels the sides may not be parallel; as for example when one end of the face is following a fault. It is necessary in such a case to take an average width depending upon the amount of variation and upon the degree of accuracy required. If necessary the panel may be split up into several lengths, each with a different width and having an effect proportioned according to the standard development curve.

Maximum Possible Subsidence

In cases where no firm plan exists for the working of a particular seam or seams it may be necessary for the surveyor or subsidence engineer to estimate the worst possible effects at the surface for the purpose of advising on the amount of surface damage or on precautions to reduce damage.

The seam might be worked with such dimensions in the layout that the maximum possible subsidence (S_{max}) occurs so that $S_{max}=m\times a$, where a is the Subsidence Factor.

It used to be believed that the subsidence factor depended upon the degree of maximum convergence in the workings which in turn affected the amount of maximum possible subsidence. For example, with deep workings a factor of 0.9 (i.e. $S/m = 0.9$) was accepted whilst with shallow workings the factor could be as low as 0.75.

Experience has shown however that if the extracted panel is wide enough, an S/m value as high as 0.9 can be produced even by shallow workings; Fig. 3 indicates this. If we take a depth of 50 metres it is clear that for a panel width of 70 metres (i.e. a notional critical area) $S/m = 0.8$. If a width of 170 metres is taken, the value of S/m reaches 0.9, but a width of more than three times the depth is needed to cause S_{max}. If it is unlikely that such wide panels would be extracted then a lower subsidence factor must be assumed and using the probable widths of extraction the factor can be estimated from Fig. 3.

N.B. When considering workings of any w/h value in a virgin area the prediction from Fig. 3 (which was derived from cases of multi-seam working) should be reduced by a multiplying factor of 0.9.

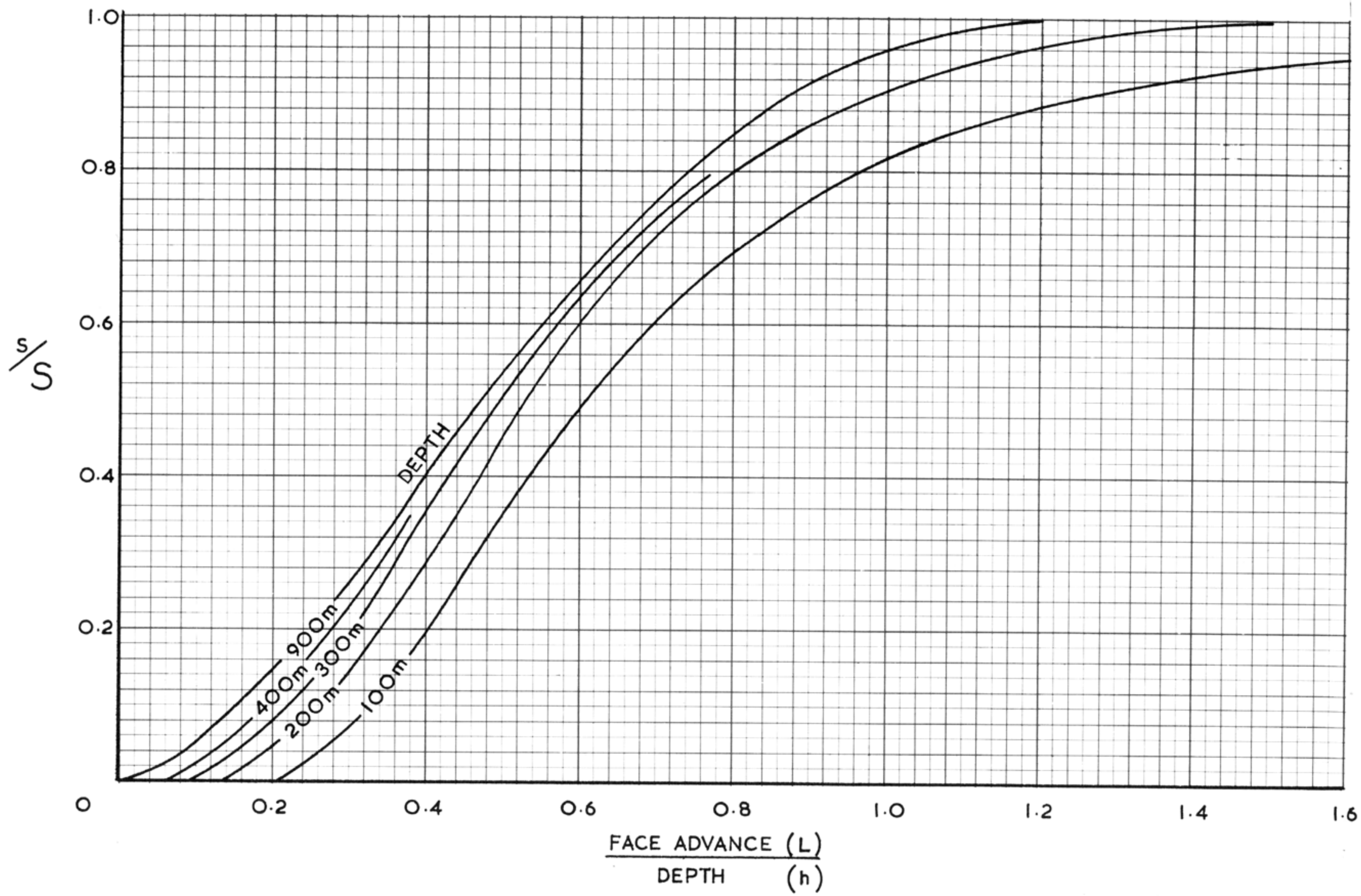


Fig. 4 Correction graph for limited face advance based on Figure 3 (accuracy ±4%).
(For cases outside the limits of this graph use the face advance as the width in Fig. 3, read off the percentage subsidence at the appropriate depth and multiply the resulting subsidence factor by 100/90.)

Section 3: Prediction of Complete Subsidence Profiles

The value of S, the maximum subsidence over a given extracted panel, as described in Section 2, is of limited importance by itself. The whole profile of subsidence must be ascertained to enable a study of the effects of mining to be made.

The shape of a subsidence profile varies with w/h, the width/depth ratio of the extraction (5)*, as has been shown by an analysis of a large number of field observations. For any particular value of w/h a curve can be plotted relating the ratio s/S at various points with d/h the distance of those points from the centre of the panel in terms of depth. A series of such graphs relating s/S to d/h for various width/depth ratios has been used to develop Fig. 5 – a family of curves or "contours" from which the subsidence values s/S for any profile can be determined. Table 1 records the values from which the "contours" are plotted and is more suitable for rapid determination of subsidence, as shown in the following example.

Example

Determine the subsidence profile for a longwall face 160 m long in a seam 1.4 m thick lying at a depth of 400 m.

First set out the known and derived parameters:

$$w = 160 \text{ metres}$$
$$m = 1.4 \text{ metres}$$
$$h = 400 \text{ metres}$$
$$w/h = 0.40$$

From the known values of w and h obtain a value of $\frac{S}{m}$ from Fig. 3 $= 0.37$.

$$\text{Hence } S = 0.37 \times 1.4 = 0.518 \text{ metres.}$$

Now construct a table as shown in Table 2 below, setting out in line 1 some assumed values of s/S (top line of Table 1). From these derive line 2 – subsidence s in metres – using the factor 0.518 for S. In line 3 set out the distance values d/h corresponding to w/h $= 0.40$ in Table 1. Thence, in line 4, derive values for the distance d in metres by multiplying the depth by the figure in line 3, e.g. $400 \times 0.90 = 360$ m.

Lines 2 and 4 are now used to plot the profile to suitable horizontal and vertical scales, $^1/_{2,500}$ being commonly suitable for the former but the vertical scale depending on the purpose of the plotting and the amount of subsidence. Fig. 6 shows the profile values from lines 2 and 4 of Table 2 plotted to scales which give a convenient section. With a sloping ground surface the section datum is drawn at the mean level of the subsided ground.

* Numbers in brackets refer to items listed in the bibliography.

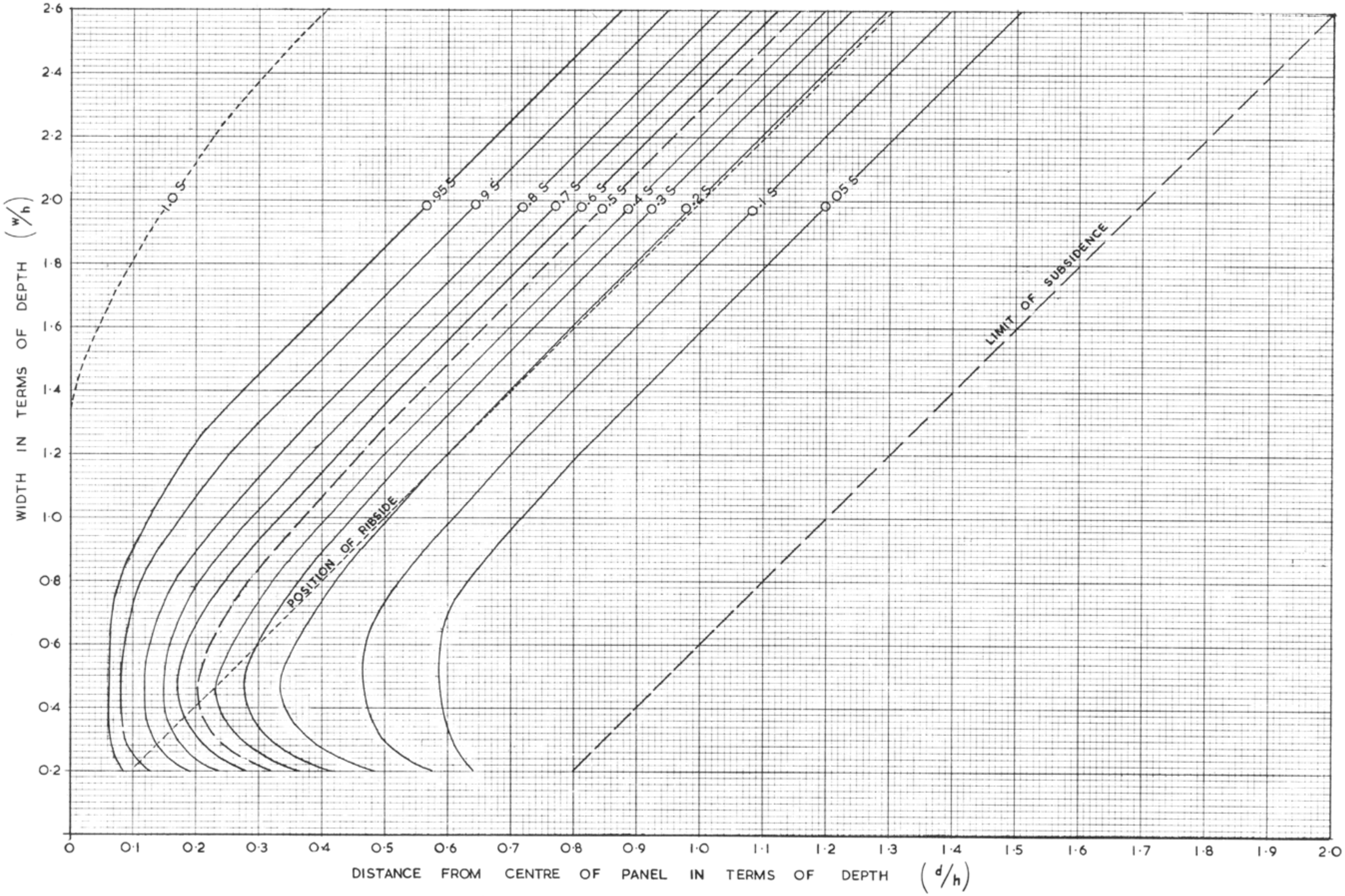


Fig. 5 Graph for predicting subsidence profiles.

TABLE 1: **Relationship between w/h and d/h for various points on a subsidence profile**

Values of s/S	0	0.05	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	0.95	1.00
w/h RATIO OF PANEL	DISTANCES FROM PANEL CENTRE IN TERMS OF DEPTH												
2.6	2.00	1.51	1.39	1.29	1.24	1.19	1.16	1.12	1.08	1.03	0.95	0.87	0.41
2.4	1.90	1.41	1.29	1.19	1.14	1.10	1.06	1.02	0.98	0.93	0.85	0.77	0.31
2.2	1.80	1.31	1.19	1.09	1.04	1.00	0.96	0.92	0.88	0.83	0.75	0.67	0.23
2.0	1.70	1.21	1.09	0.99	0.94	0.90	0.86	0.82	0.78	0.73	0.65	0.57	0.16
1.8	1.60	1.11	1.00	0.90	0.84	0.80	0.76	0.72	0.68	0.63	0.55	0.47	0.10
1.6	1.50	1.01	0.90	0.80	0.74	0.70	0.66	0.62	0.58	0.53	0.45	0.37	0.05
1.4	1.40	0.91	0.80	0.70	0.64	0.60	0.56	0.52	0.48	0.43	0.35	0.27	0.01
1.3	1.35	0.86	0.75	0.65	0.59	0.55	0.51	0.47	0.43	0.38	0.30	0.23	0
1.2	1.30	0.81	0.70	0.60	0.54	0.50	0.46	0.42	0.38	0.33	0.25	0.19	0
1.1	1.25	0.77	0.65	0.55	0.50	0.45	0.42	0.38	0.34	0.29	0.21	0.16	0
1.00	1.20	0.72	0.61	0.51	0.45	0.41	0.37	0.33	0.29	0.24	0.18	0.13	0
0.98	1.19	0.71	0.60	0.50	0.44	0.40	0.36	0.33	0.28	0.24	0.17	0.12	0
0.96	1.18	0.70	0.59	0.49	0.43	0.39	0.35	0.32	0.27	0.23	0.16	0.11	0
0.94	1.17	0.69	0.58	0.48	0.42	0.38	0.34	0.31	0.26	0.22	0.16	0.11	0
0.92	1.16	0.68	0.57	0.47	0.41	0.37	0.33	0.30	0.26	0.21	0.15	0.10	0
0.90	1.15	0.68	0.57	0.46	0.40	0.36	0.32	0.29	0.25	0.20	0.14	0.10	0
0.88	1.14	0.67	0.56	0.45	0.40	0.36	0.32	0.28	0.24	0.20	0.13	0.10	0
0.86	1.13	0.66	0.55	0.45	0.39	0.35	0.31	0.27	0.23	0.19	0.13	0.09	0
0.84	1.12	0.65	0.54	0.44	0.38	0.34	0.30	0.26	0.22	0.18	0.12	0.09	0
0.82	1.11	0.64	0.53	0.43	0.37	0.33	0.29	0.26	0.21	0.17	0.12	0.08	0
0.80	1.10	0.63	0.52	0.42	0.36	0.32	0.28	0.25	0.21	0.17	0.11	0.08	0
0.78	1.09	0.63	0.52	0.42	0.36	0.32	0.28	0.24	0.20	0.16	0.11	0.08	0
0.76	1.08	0.62	0.51	0.41	0.35	0.31	0.27	0.23	0.20	0.16	0.11	0.07	0
0.74	1.07	0.61	0.50	0.40	0.34	0.30	0.26	0.23	0.19	0.15	0.10	0.07	0
0.72	1.06	0.61	0.50	0.39	0.34	0.30	0.26	0.22	0.18	0.15	0.10	0.07	0
0.70	1.05	0.60	0.49	0.39	0.33	0.29	0.25	0.21	0.18	0.14	0.10	0.07	0
0.68	1.04	0.60	0.49	0.38	0.32	0.28	0.24	0.21	0.17	0.14	0.10	0.07	0
0.66	1.03	0.60	0.48	0.38	0.32	0.28	0.24	0.20	0.17	0.14	0.09	0.06	0
0.64	1.02	0.59	0.48	0.37	0.31	0.27	0.23	0.20	0.17	0.13	0.09	0.06	0
0.62	1.01	0.59	0.47	0.37	0.31	0.27	0.23	0.19	0.16	0.13	0.09	0.06	0
0.60	1.00	0.59	0.47	0.36	0.30	0.26	0.22	0.19	0.16	0.13	0.09	0.06	0
0.58	0.99	0.59	0.47	0.35	0.30	0.25	0.22	0.18	0.16	0.13	0.09	0.06	0
0.56	0.98	0.59	0.47	0.35	0.29	0.25	0.21	0.18	0.15	0.12	0.08	0.06	0
0.54	0.97	0.59	0.47	0.34	0.29	0.25	0.21	0.18	0.15	0.12	0.08	0.06	0
0.52	0.96	0.59	0.47	0.34	0.28	0.24	0.21	0.17	0.15	0.12	0.08	0.06	0
0.50	0.95	0.59	0.47	0.34	0.28	0.24	0.21	0.17	0.15	0.12	0.08	0.06	0
0.48	0.94	0.59	0.47	0.33	0.28	0.23	0.20	0.17	0.15	0.12	0.08	0.06	0
0.46	0.93	0.59	0.47	0.33	0.28	0.23	0.20	0.17	0.15	0.12	0.08	0.06	0
0.44	0.92	0.59	0.47	0.33	0.28	0.23	0.20	0.17	0.15	0.12	0.08	0.06	0
0.42	0.91	0.59	0.47	0.34	0.28	0.24	0.20	0.17	0.15	0.12	0.08	0.06	0
0.40	0.90	0.59	0.47	0.34	0.28	0.24	0.21	0.18	0.15	0.12	0.08	0.06	0
0.38	0.89	0.60	0.48	0.35	0.29	0.24	0.21	0.18	0.15	0.12	0.08	0.06	0
0.36	0.88	0.60	0.48	0.35	0.29	0.25	0.21	0.18	0.15	0.12	0.08	0.06	0
0.34	0.87	0.60	0.49	0.36	0.30	0.25	0.22	0.19	0.16	0.12	0.08	0.06	0
0.32	0.86	0.60	0.49	0.37	0.31	0.26	0.22	0.20	0.16	0.13	0.09	0.06	0
0.30	0.85	0.61	0.50	0.38	0.32	0.27	0.23	0.20	0.17	0.13	0.09	0.06	0
0.28	0.84	0.61	0.51	0.39	0.33	0.28	0.24	0.21	0.18	0.14	0.09	0.07	0
0.26	0.83	0.62	0.52	0.41	0.35	0.30	0.26	0.22	0.19	0.15	0.10	0.07	0
0.24	0.82	0.62	0.53	0.43	0.36	0.32	0.28	0.24	0.20	0.16	0.11	0.07	0
0.22	0.81	0.63	0.55	0.46	0.39	0.34	0.30	0.26	0.21	0.17	0.11	0.08	0
0.20	0.80	0.64	0.57	0.48	0.41	0.37	0.32	0.28	0.23	0.19	0.13	0.08	0

TABLE 2

1	**Subsidence as s/S**	0	0.05	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	0.95	1.00
2	**Subsidence in metres**	0	0.026	0.052	0.104	0.155	0.207	0.259	0.310	0.363	0.414	0.466	0.492	0.518
3	**Distance in terms of h**	0.90	0.59	0.47	0.34	0.28	0.24	0.21	0.18	0.15	0.12	0.08	0.06	0
4	**Distance in metres**	360	236	188	136	112	96	84	72	60	48	32	24	0

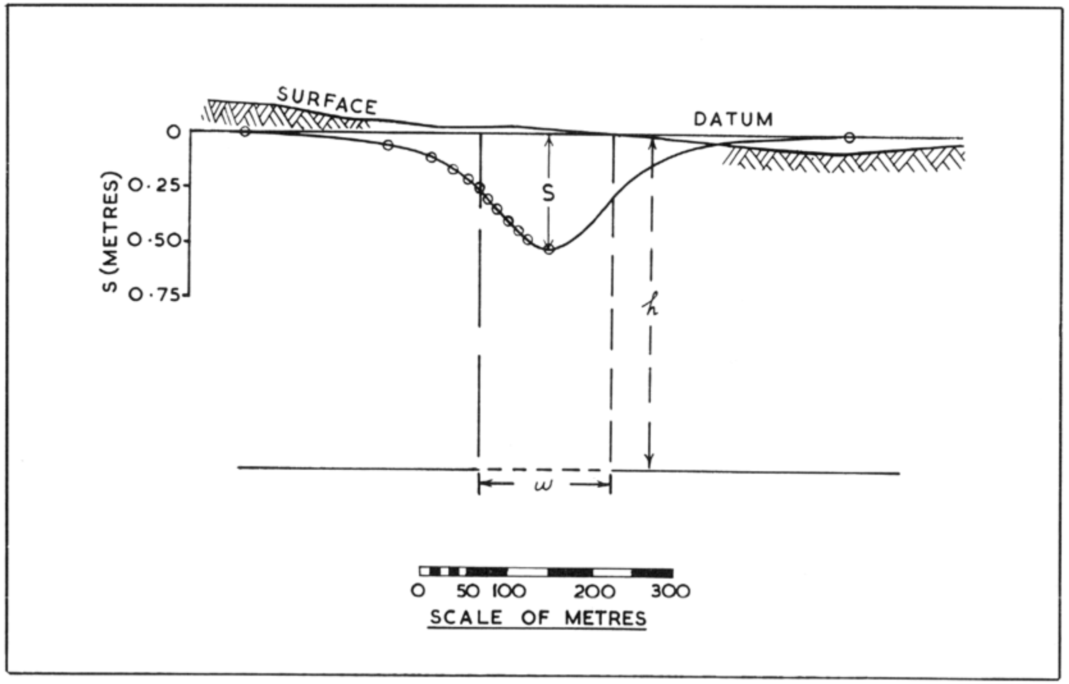


Fig. 6 Plotting of subsidence profile.

Point of Inflection

The point of inflection of a subsidence profile is that point where the convex curve ends and the concave curve begins. The subsidence at this point is coincident with the point of half-maximum subsidence S/2 (which is not necessarily $S_{max}/2$).

It is the migration of the inflection point inwards with increasing w/h values that causes profiles to be of different shapes. For example, when w/h = 0.25 or less, the profile has transition points well outside the ribsides of the extraction so that curvature is very small. At w/h = 0.4 the transition points coincide with the ribsides. When w/h equals 1.2 or more, the transition points are well inside the goaf area (as much as 0.14h) and curvature is quite pronounced. Fig. 7 shows some half profiles for other w/h values.

The exact positions of the points of zero subsidence and maximum subsidence are difficult to determine. This is because movement at these points, as determined by precise levelling, is very small and may be obscured by movement of the observation stations and the ground in which they are fixed caused by changes in temperature, rainfall, etc.

Seam Inclination

All the preceding details relate to level seams. In the case of inclined seams the limit angles must be varied according to the graph in Fig. 8, which extends notionally the effect of dip beyond field experience to date. Note that limit angles must be measured from the floor of the workings, which, of course, is where levels are taken underground.

Fig. 9 shows the profile determined above, and plotted at Fig. 6, now replotted with reference to the (dipping) seam, the extraction and the surface.

Surface Slope due to Subsidence

The degree to which any surface site may be expected to tilt as a result of subsidence is calculated from the subsidence profile. As shown in Table 3, tabulate the points on the subsidence profile, with reference to their distances from the panel centre (Col. 2), and the subsidence at these points (Col. 3). By subtraction, obtain the difference in subsidence (Col. 4) and the distance between stations (Col. 5), sometimes called the bay length (*l*). The subsidence difference divided by the distance between stations gives the slope (Col. 6).

The site will thus be subjected to slopes of 25 in 10,000 (1 in 400) over part of its length and 20 in 10,000 (1 in 500) over the other part.

A working may not always follow a projected extraction plan since geological and other influences may prevent the plan being carried out; therefore it is best, in predicting the subsidence effect on a new structure, to allow for the maximum slope of the seam or seams which might be extracted. The maximum slope which any extraction will cause is, on average, $2.75S_{max}/h$. This allows for the panel width varying up to the critical width, as it might well do for any structure,

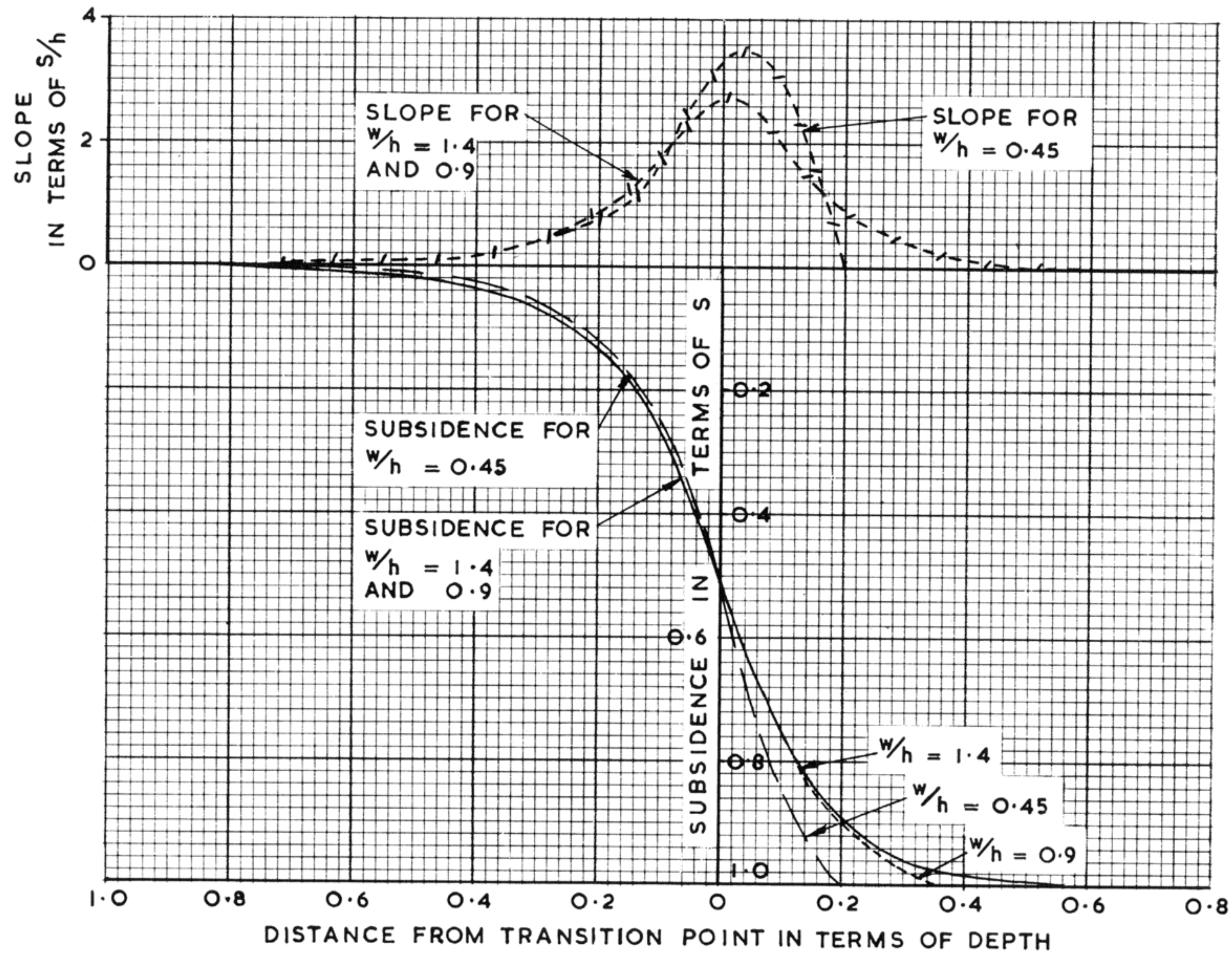


Fig. 7 Typical half profiles and slope curves.

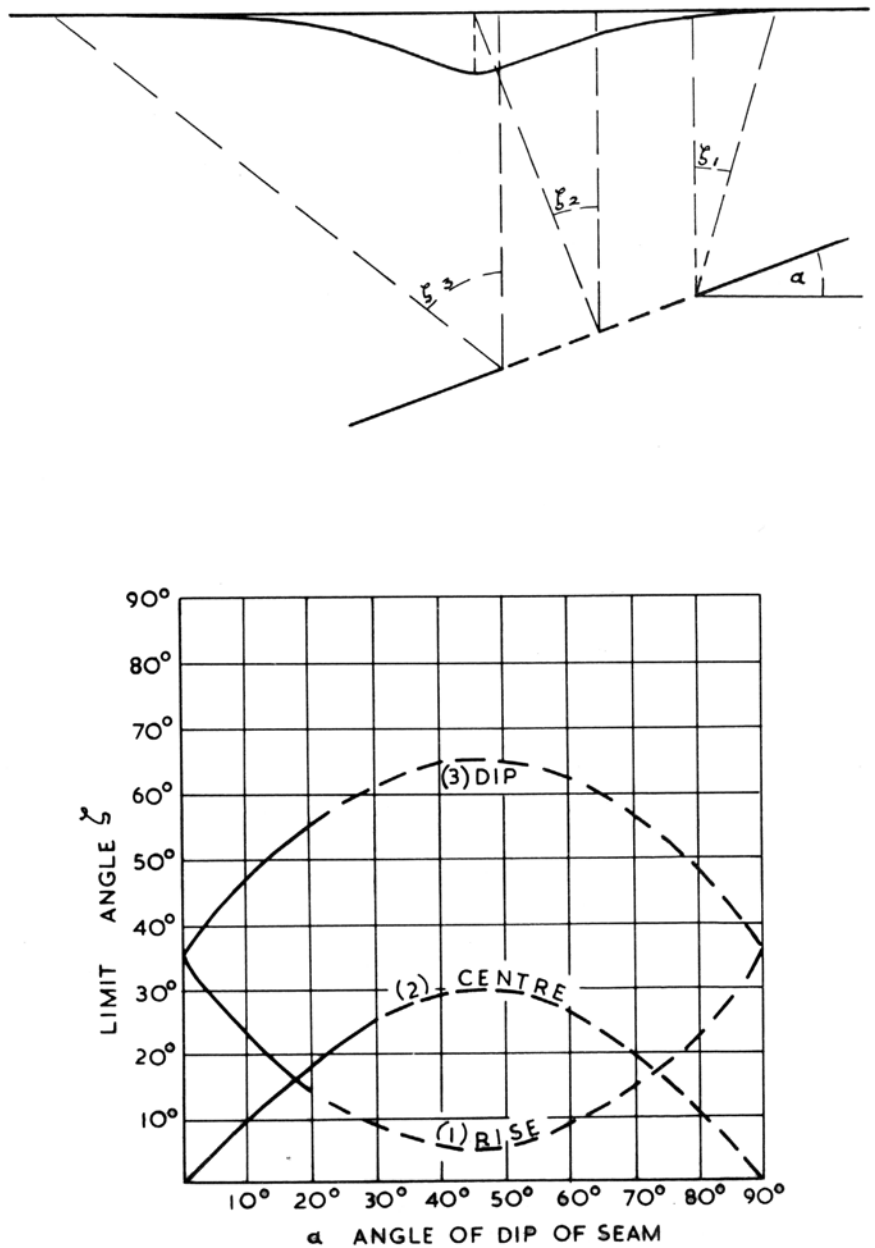


Fig. 8 Effect of seam inclination on limit angles.

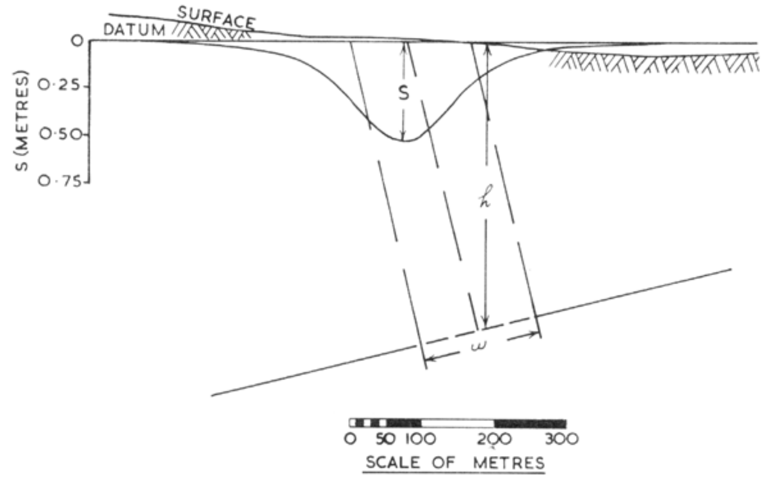


Fig. 9 Profile plotting with inclined seam.

TABLE 3

1	2	3	4	5	6
Point	Distance from panel centre (metres)	Subsidence (metres)	Subsidence difference	Distance between stations (metres)	Slope
1	92	0.196			
			0.049	20	0.0025
2	112	0.147			
			0.040	20	0.0020
3	132	0.107			

even if two or three adjacent panels have to be extracted to achieve the critical width. In such a case S_{max} would not be as high as 0.9m, as the extraction would be with simple panels without centre gates. (The allowance to be made for the interruption of the overall profile by gate and packing zones, etc., is dealt with later in this Section.)

On Fig. 7 the corresponding slope curves have been added to the subsidence profiles. Although at w/h = 0.45 the steepest slope in terms of S/h is observed, the *actual* slope is less than in the case of a wider panel because the value of S is smaller.

Estimation of Subsidence by Computer

During the past few years a computer system has been developed jointly by the Board's Western Area Subsidence Branch and Computer Services.

In general, descriptions of the mining areas using a system of co-ordinates are supplied as data to the computer system along with other relevant information such as seam thickness, depth of mining, and amount and direction of seam gradient. From this information, plans to any required scale are produced by the computer, showing contours of subsidence resulting from the mining area and/or subsidence profiles along lines across the mining area. The system is also designed to give results from one or more seams taken either individually or in various combinations.

The method of estimation used in the system is of the annular zone area type used on a wide scale in most Continental European coalfields, but in this case adjusted to give the same result as that which, under standard conditions, would be produced by the systems described in this handbook.

Copies of the appropriate data input sheets and information about the system may be obtained from the Head of Mining and Technical Section, Compower Ltd., Coal House, Cannock.

Differential Slope or Curvature

A site like that in the example on page 16, which has two different slopes, is really undergoing curvature and the two halves of the site are, in the calculations, tangents to the curve. This information is important to civil engineers or architects concerned with future damage to an existing building or with the design of a proposed structure. The curvature may be given as differential slope (from Table 3 it would be 0.0005, or the radius of curvature (ρ) may be calculated thus:–

$$\rho = \frac{(\text{bay length})^2}{\text{second differential of subsidence}}$$

$$\rho = \frac{20\text{m} \times 20\text{m}}{0.009\text{m}} = 44{,}444\text{ m}$$

Hump Prediction

The subsidence profiles analysed to obtain the values and relationships already dealt with were deliberately chosen as having no centre gate or other zone of packing or support in the goaf. Where deep seams are concerned, critical areas are large and two or three panels may be needed to cover the critical area; in these conditions several gate roads and packs occur close together in the goaf. Over each of these support zones a hump – or at least a flattening – occurs in the subsidence profile. (As regards centre gates even in sub-critical widths of panel, these are often encountered because in deeper seams even a wide panel can still be sub-critical.) The maximum subsidence is often reduced as a result of the centre-gate packs and Fig. 10 indicates both an actual profile as observed from levelling and an estimate of what the profile would have been with an empty goaf from side to side.

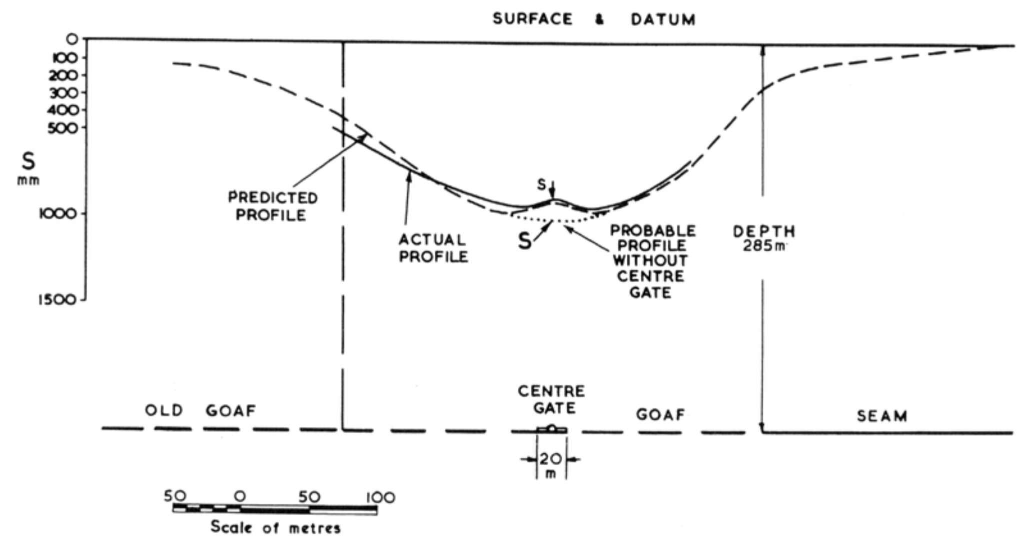


Fig. 10 Subsidence profile – Case 60.

A number of such cases have been examined and the graph in Fig. 11 shows the relationship between s at the top of the hump and S as it would have been over a "simple" panel. S is thus actually the predicted subsidence. In the graph, s/S is plotted against the width/depth ratio of the support zone; one curve is the mean through the centre gate cases and the other is the mean through the cases where a pillar of solid coal was left – as often happens between consecutive panels.

Theoretically, as we have seen, the effect of solid stowing (such as we can consider the centre gate and its good support packs to be) can about halve the subsidence, and the value of s in Fig. 11 varies from about 0.4S with a pack zone approaching critical width to 1.0S where the pack zone is too narrow to have any effect at the surface. Similarly a solid coal pillar of critical width will reduce s to zero but have no effect where it is too narrow.

In plotting values for the pillar curve in Fig. 11, the width of solid coal pillars was taken in determining the values of the width/depth ratio. Thus, in reading values of width/depth ratio for a pillar from this curve, only the width of pillar should be considered – despite the fact that the hump effect is produced by the combined width of the pillar and the gate road and associated packs adjacent to it.

From Fig. 11 any condition can be read off and s predicted. The profile is then plotted by sketching in the curve. In the case of a simple panel and pillar extraction where the pillars are wide enough to remain stable (502 and 503) the profiles over adjacent panels can be plotted overlapping and the subsidence at the overlaps added together to produce the final predicted profile. Where pillars are so narrow that they might collapse and cause additional subsidence the method of superposition should not be used.

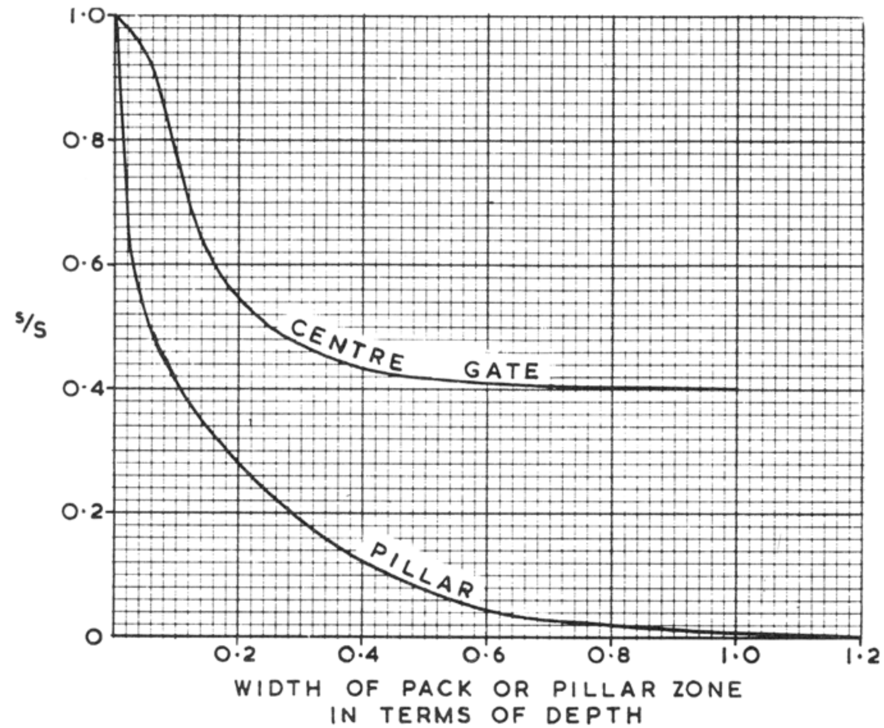


Fig. 11 Subsidence over centre gates and solid coal pillars.

Section 4: Horizontal Strains

In subsidence engineering terminology, "strain" (e) is the change in length over a piece of ground or structure expressed either as a dimension over the whole length or as a fraction of the unit of length. Thus we may express a strain of 0.01 metre in 10 metres as one part per thousand, or one millimetre per metre, or simply as 0.001. The direction is always specified, extensions and compressions being indicated by a + and — sign respectively.

In the same way that "contours" joining the points of equal subsidence can be drawn on a graph (see Fig. 5), a graph can be made showing lines joining the points where the horizontal strains are similar at various width/depth ratios. Fig. 12 shows such a diagram drawn from an analysis of all the available strain profiles where the strains were measured in the field. Table 4 shows the distances in quickly readable form. In both the graph and the table the distances are measured from the panel centre.

Extension and Compression

The zone of maximum extension (+E) and the zone of maximum compression (—E) in relation to the transition point are of particular importance.

The former coincides with the position of the ribside where the width/depth ratio is greater than 1.35, but lies outside the rib when the width/depth ratio is smaller.

The maximum compression occurs at the panel centre in the narrower panels (w less than 0.42 h), but with a greater width/depth ratio the profile develops two compression zones. Fig. 12 shows only half a profile. Fig. 13 illustrates the shape of the strain profile at the different states: when w=0.42 h, when the single compression zone has a greater intensity than the extension; when w=0.9 h, when a hump occurs in the compression curve; and when w=1.5 h, when two separate compression zones occur. The middle diagram encompasses a range of width/depth ratios from about w=0.5 h to about w=1.4 h. Fig. 14 shows how the compression on the hump (which we may call - e) compares with the compression at its maximum amount (—E). The proportion —e/—E varies from unity with a small width/depth ratio to zero with super-critical widths of extraction, and the curve on the graph can be used to predict the shape of the compression part of the strain profile. The proportion —e/—E is also reflected in the turning in of the compression contours on Fig. 12, and a profile plotted across these contours for sub-critical width/depth ratios will produce the same shape as is indicated by the graph line in Fig. 14.

Relationship Between +E and —E

The next matter to be decided in predicting a strain profile is the proportion of extension to compression for the given width/depth ratio. Fig .15 shows this, and the two strain curves indicate that the intensity of +E can be about a quarter as big again as that of —E when critical widths of panel are involved.

It also appears that over very narrow panels the —E intensity can be much greater than that of +E and not merely twice as great, as had been believed for

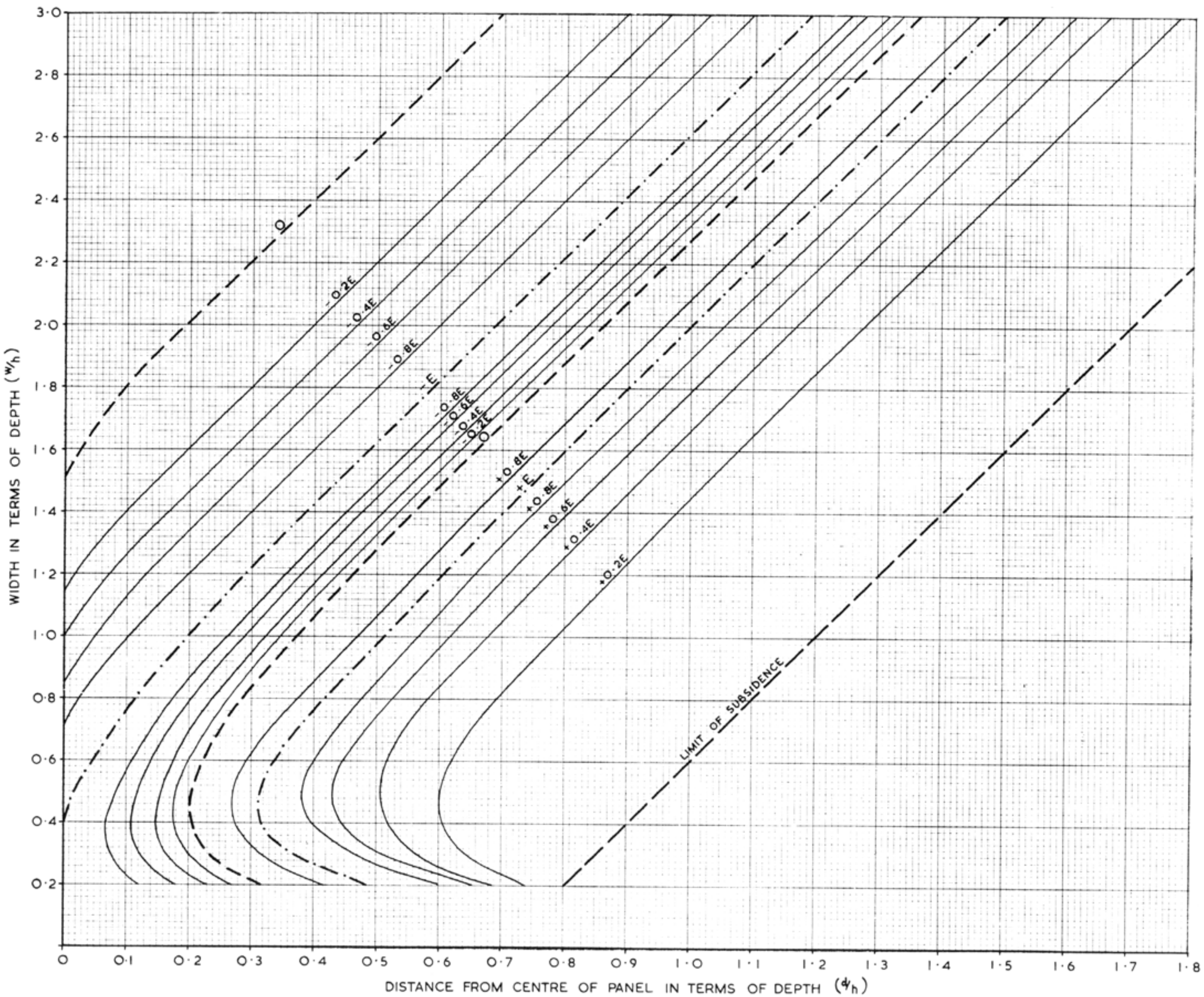


Fig. 12 Graph for predicting strain profiles.

TABLE 4 **Relationship for various strain values in a subsidence profile**

				Extension (+E)								Compression (—E)						
Values of e/E	**0**	**0.20**	**0.40**	**0.60**	**0.80**	**1.00**	**0.80**	**0**	**0.20**	**0.40**	**0.60**	**0.80**	**1.00**	**0.80**	**0.60**	**0.40**	**0.20**	**0**
w/h RATIO OF PANEL	DISTANCES FROM PANEL CENTRE IN TERMS OF DEPTH																	
3.0	2.2	1.78	1.67	1.61	1.56	1.50	1.46	1.36	1.34	1.31	1.28	1.25	1.19	1.10	1.03	0.96	0.90	0.70
2.6	2.0	1.58	1.47	1.41	1.36	1.30	1.26	1.16	1.14	1.11	1.08	1.05	0.99	0.90	0.83	0.77	0.70	0.50
2.2	1.8	1.38	1.27	1.21	1.15	1.10	1.06	0.96	0.94	0.91	0.88	0.85	0.79	0.70	0.63	0.57	0.50	0.30
2.0	1.7	1.28	1.17	1.11	1.05	1.00	0.96	0.86	0.84	0.81	0.78	0.75	0.69	0.60	0.53	0.47	0.40	0.20
1.8	1.6	1.17	1.07	1.01	0.95	0.90	0.86	0.76	0.73	0.71	0.68	0.65	0.59	0.50	0.43	0.37	0.30	0.10
1.6	1.5	1.08	0.97	0.91	0.85	0.80	0.76	0.66	0.63	0.61	0.58	0.55	0.49	0.40	0.33	0.27	0.20	0.03
1.4	1.4	0.98	0.87	0.81	0.75	0.70	0.66	0.56	0.53	0.51	0.48	0.45	0.39	0.30	0.23	0.17	0.10	0
1.3	1.35	0.93	0.82	0.76	0.70	0.65	0.61	0.51	0.49	0.46	0.43	0.40	0.34	0.25	0.18	0.12	0.05	0
1.2	1.3	0.88	0.77	0.71	0.66	0.61	0.56	0.46	0.44	0.41	0.38	0.35	0.29	0.20	0.13	0.07	0.02	0
1.1	1.25	0.83	0.72	0.66	0.61	0.56	0.52	0.42	0.39	0.37	0.33	0.31	0.24	0.15	0.09	0.03	0	0
1.0	1.2	0.79	0.68	0.62	0.57	0.51	0.47	0.37	0.35	0.32	0.29	0.26	0.20	0.10	0.05	0	0	0
0.98	1.19	0.78	0.67	0.61	0.56	0.50	0.46	0.36	0.34	0.31	0.28	0.25	0.19	0.09	0.04	0	0	0
0.96	1.18	0.77	0.66	0.60	0.55	0.49	0.45	0.35	0.33	0.30	0.27	0.24	0.18	0.09	0.04	0	0	0
0.94	1.17	0.76	0.65	0.59	0.54	0.48	0.44	0.35	0.32	0.30	0.26	0.23	0.17	0.08	0.03	0	0	0
0.92	1.16	0.75	0.64	0.58	0.53	0.47	0.43	0.34	0.31	0.29	0.25	0.22	0.16	0.07	0.02	0	0	0
0.90	1.15	0.74	0.63	0.57	0.52	0.46	0.42	0.33	0.30	0.28	0.24	0.21	0.15	0.06	0.02	0	0	0
0.88	1.14	0.73	0.62	0.56	0.51	0.46	0.41	0.32	0.29	0.27	0.24	0.21	0.15	0.05	0.01	0	0	0
0.86	1.13	0.72	0.61	0.55	0.50	0.45	0.40	0.31	0.29	0.26	0.23	0.20	0.14	0.05	0	0	0	0
0.84	1.12	0.71	0.60	0.54	0.49	0.44	0.39	0.30	0.28	0.25	0.22	0.19	0.13	0.04	0	0	0	0
0.82	1.11	0.70	0.59	0.53	0.48	0.43	0.38	0.29	0.27	0.25	0.21	0.18	0.12	0.03	0	0	0	0
0.80	1.10	0.69	0.58	0.53	0.48	0.42	0.37	0.29	0.26	0.24	0.20	0.17	0.11	0.02	0	0	0	0
0.78	1.09	0.68	0.57	0.52	0.47	0.41	0.36	0.28	0.26	0.23	0.20	0.17	0.11	0.02	0	0	0	0
0.76	1.08	0.67	0.57	0.51	0.46	0.40	0.36	0.27	0.25	0.22	0.19	0.16	0.10	0.01	0	0	0	0
0.74	1.07	0.67	0.56	0.50	0.45	0.39	0.35	0.26	0.24	0.22	0.18	0.15	0.09	0.01	0	0	0	0
.072	1.06	0.66	0.55	0.49	0.44	0.38	0.34	0.26	0.24	0.21	0.17	0.15	0.09	0	0	0	0	0
0.70	1.05	0.65	0.54	0.48	0.44	0.37	0.33	0.25	0.23	0.20	0.17	0.14	0.08	0	0	0	0	0
0.68	1.04	0.64	0.54	0.47	0.43	0.37	0.32	0.24	0.22	0.20	0.16	0.13	0.07	0	0	0	0	0
0.66	1.03	0.64	0.53	0.47	0.42	0.36	0.31	0.24	0.22	0.19	0.16	0.13	0.07	0	0	0	0	0
0.64	1.02	0.63	0.53	0.46	0.41	0.35	0.31	0.23	0.21	0.19	0.15	0.12	0.06	0	0	0	0	0
0.62	1.01	0.63	0.52	0.45	0.41	0.34	0.30	0.23	0.21	0.18	0.15	0.12	0.05	0	0	0	0	0
0.60	1.00	0.62	0.52	0.45	0.40	0.34	0.29	0.22	0.20	0.18	0.14	0.11	0.05	0	0	0	0	0
0.58	0.99	0.62	0.51	0.44	0.39	0.33	0.29	0.22	0.19	0.17	0.14	0.10	0.04	0	0	0	0	0
0.56	0.98	0.61	0.51	0.44	0.39	0.33	0.28	0.22	0.19	0.17	9.13	0.10	0.03	0	0	0	0	0
0.54	0.97	0.61	0.51	0.43	0.39	0.32	0.28	0.21	0.19	0.16	0.13	0.09	0.03	0	0	0	0	0
0.52	0.96	0.60	0.51	0.43	0.38	0.32	0.27	0.21	0.18	0.16	0.12	0.09	0.02	0	0	0	0	0
0.50	0.95	0.60	0.51	0.43	0.38	0.32	0.27	0.21	0.18	0.16	0.12	0.08	0.02	0	0	0	0	0
0.48	0.94	0.60	0.51	0.43	0.38	0.31	0.27	0.20	0.18	0.15	0.12	0.08	0.01	0	0	0	0	0
0.46	0.93	0.60	0.51	0.43	0.38	0.31	0.27	0.20	0.18	0.15	0.11	0.08	0.01	0	0	0	0	0
0.44	0.92	0.60	0.51	0.43	0.39	0.31	0.27	0.20	0.18	0.15	0.11	0.07	0.01	0	0	0	0	0
0.42	0.91	0.60	0.51	0.44	0.39	0.31	0.27	0.20	0.18	0.15	0.11	0.07	0	0	0	0	0	0
0.40	0.90	0.61	0.52	0.45	0.40	0.32	0.28	0.21	0.18	0.15	0.11	0.07	0	0	0	0	0	0
0.38	0.89	0.61	0.53	0.45	0.41	0.32	0.28	0.21	0.18	0.15	0.11	0.07	0	0	0	0	0	0
0.36	0.88	0.62	0.53	0.46	0.42	0.33	0.29	0.21	0.18	0.15	0.11	0.07	0	0	0	0	0	0
0.34	0.87	0.62	0.54	0.48	0.43	0.34	0.30	0.22	0.19	0.15	0.11	0.07	0	0	0	0	0	0
0.32	0.86	0.63	0.55	0.49	0.45	0.35	0.30	0.22	0.19	0.16	0.12	0.97	0	0	0	0	0	0
0.30	0.85	0.65	0.57	0.51	0.47	0.37	0.32	0.23	0.20	0.16	0.12	0.08	0	0	0	0	0	0
0.28	0.84	0.66	0.58	0.54	0.49	0.39	0.33	0.24	0.21	0.17	0.13	0.08	0	0	0	0	0	0
0.26	0.83	0.68	0.60	0.57	0.51	0.41	0.35	0.26	0.22	0.18	0.14	0.09	0	0	0	0	0	0
0.24	0.82	0.70	0.63	0.60	0.54	0.44	0.37	0.28	0.23	0.20	0.15	0.10	0	0	0	0	0	0
0.22	0.81	0.72	0.66	0.63	0.58	0.47	0.39	0.30	0.25	0.21	0.16	0.11	0	0	0	0	0	0
0.20	0.80	0.74	0.69	0.66	0.61	0.49	0.42	0.32	0.27	0.23	0.18	0.12	0	0	0	0	0	0

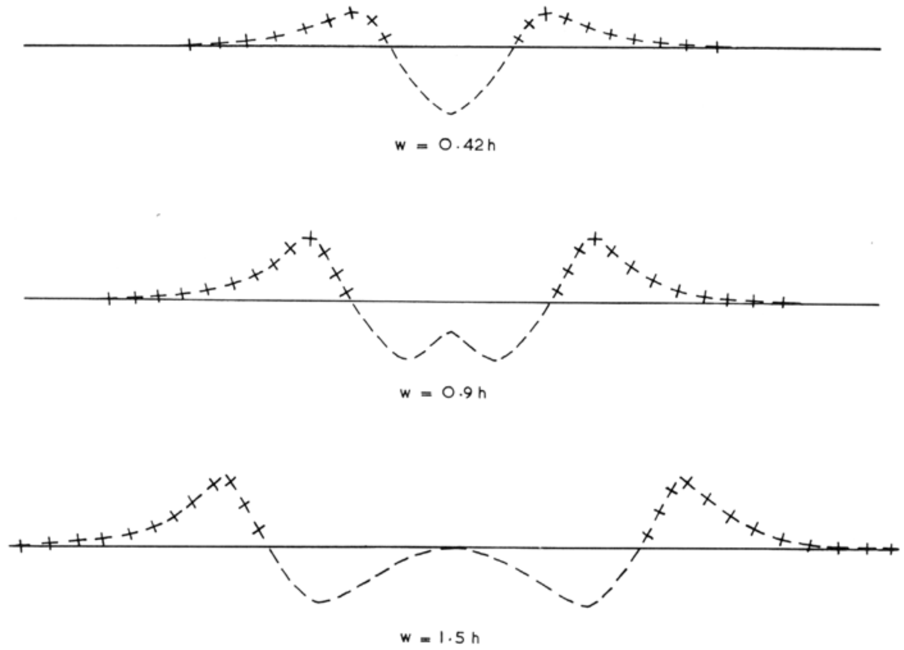


Fig. 13 Three principal types of strain profile.

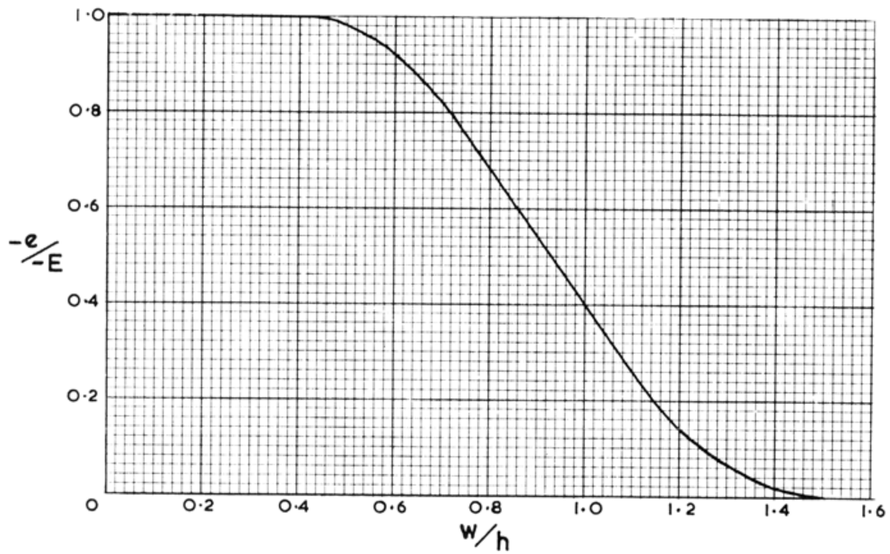


Fig. 14 Compression at panel centre compared with maximum compression.

many years. It is when a panel width is about 0.43 h that the compression is double the extension. The scatter of the results of field measurements on a variety of observation stations and in variable ground has obviously masked the truth, and the percentage error possible from predictions made with Fig. 15 appear to be quite large. We shall see later that this error is not so great because a more accurate prediction of strain is possible from the curvature of the subsidence profile than from the measured ground strains. Strain prediction from curvature has shown the mean values of the measured ground strains to be better than the scatter would indicate.

Strain-Subsidence-Depth Relationships

So far we have discussed the relative values of extension and compression and the positions of various points on the strain profiles; but strain obviously has to be related to subsidence and seam depth.

Strain is proportional to subsidence and inversely proportional to depth, so that the maximum strain (E) over an extraction is proportional to S/h, S being the maximum subsidence over the extraction.

The prediction of +E has been conveniently done since 1957 (16) by the formula +E=0.75 S/h for panel widths up to about 0.7 h. We now find that this was only an average value and, furthermore, that the width/depth ratio does not impose a limit to the *method*, as was believed to be the case in 1957.

Average values are found to be as shown in Fig. 15 (which also shows the curve for slope values). Table 5 gives a few values from this graph for ready comparison:—

TABLE 5

w/h	+E	−E	G
0.24	0.6 S/h	2.2 S/h	2.55 S/h
0.3	0.7 S/h	2.0 S/h	2.95 S/h
0.5	0.8 S/h	1.35 S/h	3.35 S/h
0.8	0.65 S/h	0.7 S/h	2.8 S/h
1.4	0.65 S/h	0.51 S/h	2.75 S/h

These values, or the values read off the graph for other width/depth ratios, can be used to obtain maximum G, +E or −E for a given simple profile. The subsidence profile may be one which has been predicted or one which has been plotted from actual levels. The quickest method is first to calculate S/h and then to multiply the quotient by the appropriate factor from Fig. 15. It is interesting, and may be useful, to note that +E approximately equals G/4.2.

The factor values are, however, the mean from simple cases – i.e. panels having no centre gate or other cause of interruption in the straight-forward

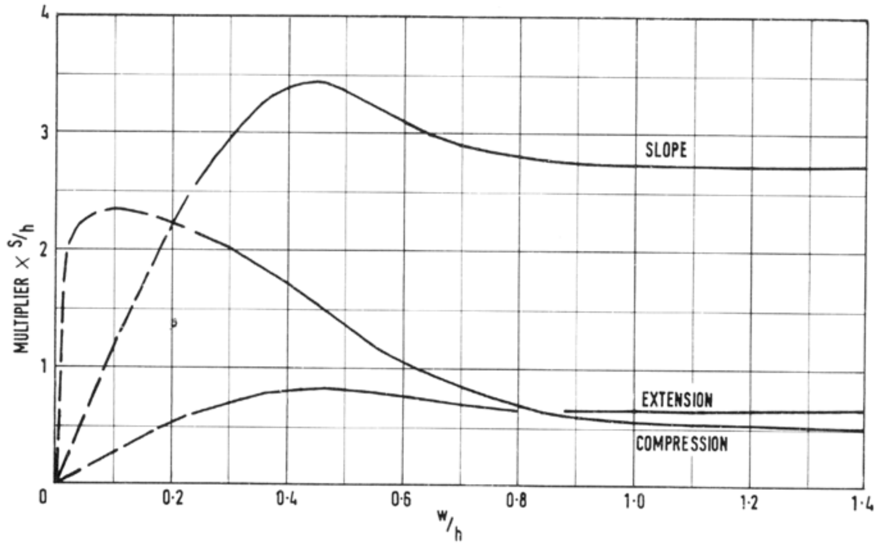


Fig. 15 Graph for predicting maximum slope and strain for various width/depth ratios of panel.

subsidence profile. When a hump occurs in the profile the value of S is less than the maximum possible for the width of panel concerned and factors in Fig. 15 would not apply.

The values of g, +e and −e at any point on the subsidence trough could be predicted from the above values only if a complete profile were plotted and the amount at the appropriate point on the profile read off the scale. This method is valid but again only for simple cases.

Effect of Sloping Surfaces

All the foregoing concerns the surface effects of mining when the surface is level or approximately so. With a sloping surface, however, calculated ground strains need correcting.

The basic components causing horizontal strain between two points on a subsidence profile are differential subsidence (ds) and differential displacement (dv), in combination. Providing ds is small and the ground more or less level, the situation (Fig. 16) obtains where

$$\frac{dv}{l} \simeq \frac{dv^{1}}{l}$$

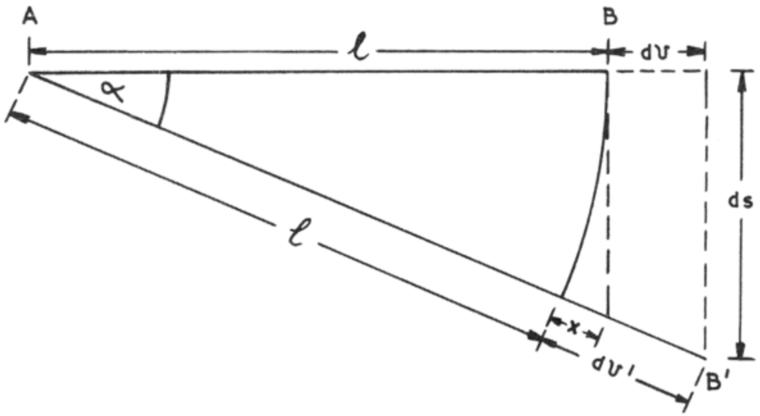


Fig. 16

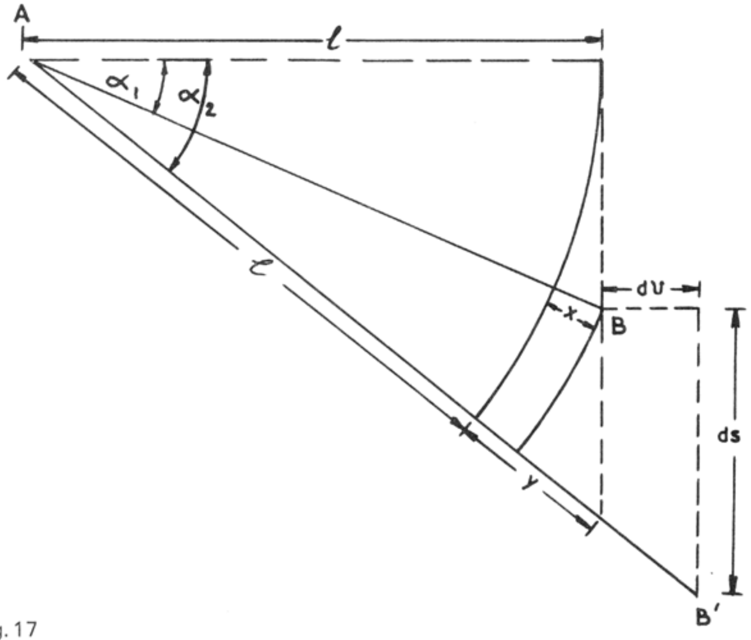


Fig. 17

A surface gradient as small as 1 in 40, however, will affect the amount of horizontal strain when, for example, 76 mm of differential subsidence occurs over a bay length of, say, 6 m.

From Fig. 16 it is seen that:–

$$\frac{dv}{l} = \frac{dv^1 - x}{l+x}$$

$$x = (l+x) - l$$

But $l + x = l \sec\alpha$

$$\therefore x = l \sec\alpha - l$$

$$\text{and strain adjustment} = \frac{l \sec\alpha - l}{l}$$

$$= \sec\alpha - 1$$

Referring to Fig. 17 the original bay A-B is now on sloping ground, and if it is subjected to differential displacement dv, and differential subsidence ds then:–

$$x = l \sec\alpha_1 - l$$

$$\text{and } y = l \sec\alpha_2 - l$$

where α_1 = original gradient,
and α_2 = new gradient.

Since AB was originally on a gradient of α_1, the increase in length becomes

$$y - x = (l \sec\alpha_2 - l) - (l \sec\alpha_1 - l)$$

$$= l \sec\alpha_2 - l \sec\alpha_1$$

$$\text{and strain adjustment} = \frac{l \sec\alpha_2 - l \sec\alpha_1}{AB}$$

But $AB = l \sec\alpha_1$

$\therefore$ strain adjustment $= \dfrac{\sec\alpha_2}{\sec\alpha_1} - 1$ when the gradient has been increased.

In the process of predicting strains the strain adjustment arrived at above must be added to tensile strain and subtracted from compressive strain.

When strains are measured in the field and need to be corrected to horizontal equivalents, the adjustment must be subtracted from the tensile strain and added to compressive strain.

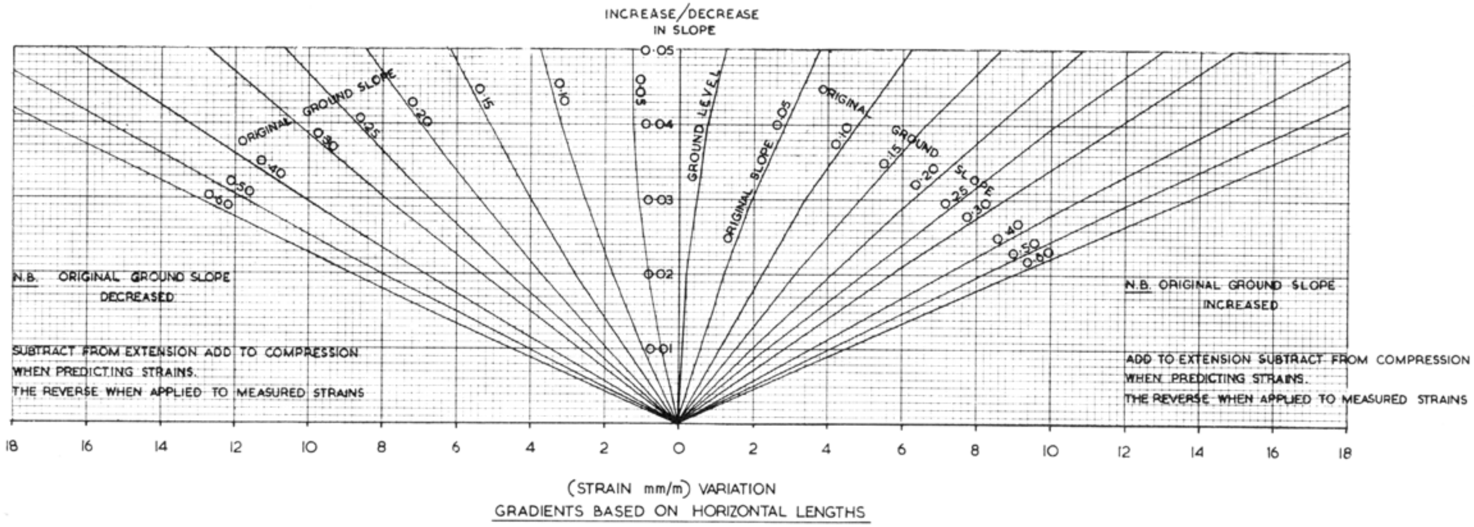
When the gradient of the bay is decreased, on the other hand, then:–

$$\text{the strain adjustment} = 1 - \frac{\sec\alpha_2}{\sec\alpha_1}$$

and the corrections are the opposite to those when the gradient has been increased.

To save time and labour, the upper nomogram in Fig. 18 may be used for those cases where the bay length has been measured or computed horizontally.

The lower nomogram in Fig. 18 should be used for those cases where the gradients are based on slope lengths.



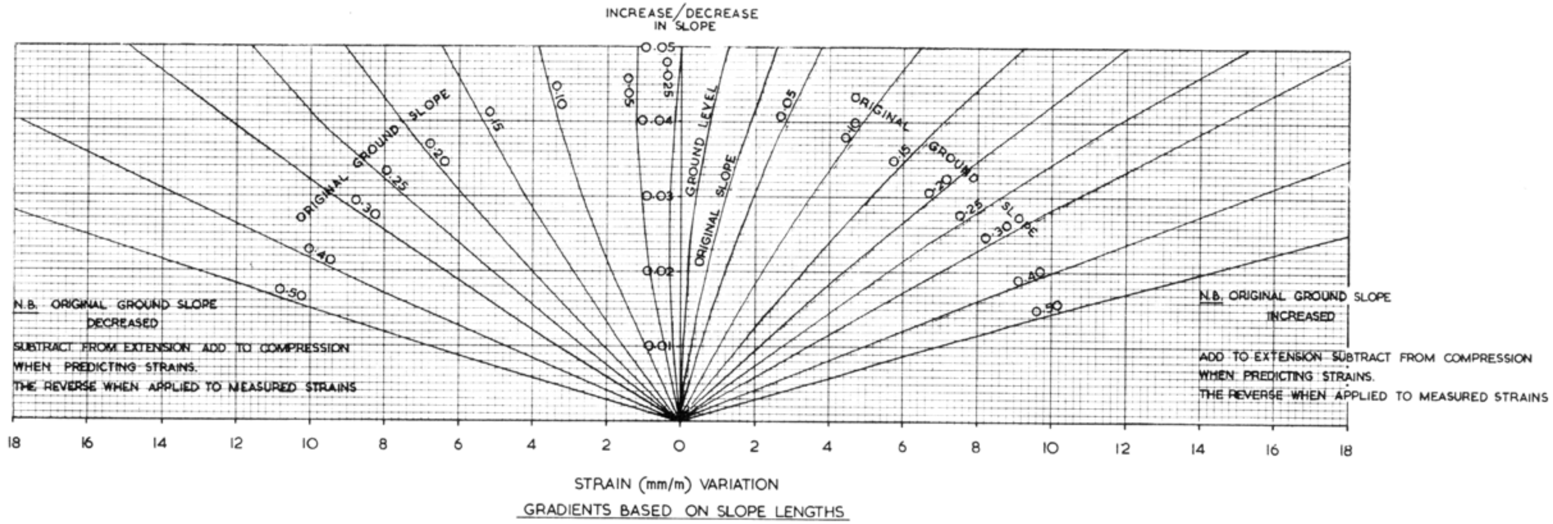


Fig. 18 Nomogram for calculating variation in strain due to ground slope.

With regard to the problem of plotting a strain profile relating to a sloping surface it is, at present, regarded as sufficiently accurate to plot the profile as for level ground, with a depth equal to the mean depth of the extraction, and then to project the profile on to the sloping ground proportionally, as indicated in Fig. 19.

It will be noted that although *slight* surface gradients do not justify these special corrections, the effect of subsidence on hillsides can be to substantially vary the strain which would be calculated for level ground and in such conditions the corrections cannot be ignored.

Effect of Dipping Seams

It is important to note that where the *surface* is level but the seam is dipping, then by the application of the correction the tensile strain on the dip side of a panel is increased and on the rise side decreased, a state of affairs which is commonly found in field measurement. The *exact* relationship between this phenomenon and the strain corrections for sloping surface has still to be investigated.

Table 6 gives corrections based on empirical results rounded off to cause the addition of the dipside and rise-side strains to equal the sum of two normal maxima for a level seam.

TABLE 6 **Effect of Seam Gradient on Tensile Strain**

GRADIENT	Proportion of Normal Tensile Strain	
	Rise	Dip
1 in 1.5	0.29	1.71
1 in 2.0	0.35	1.65
1 in 2.5	0.41	1.59
1 in 3.0	0.46	1.54
1 in 3.5	0.51	1.49
1 in 4.0	0.56	1.44
1 in 4.5	0.60	1.40
1 in 5.0	0.64	1.36
1 in 5.5	0.69	1.31
1 in 6.0	0.73	1.27
1 in 6.5	0.76	1.24
1 in 7.0	0.80	1.20
1 in 7.5	0.83	1.17
1 in 8.0	0.85	1.15
1 in 8.5	0.87	1.13
1 in 9.0	0.89	1.11
1 in 9.5	0.91	1.09
1 in 10	0.92	1.08

NOTE: Rise and dip maxima always add up to twice the maximum tension.

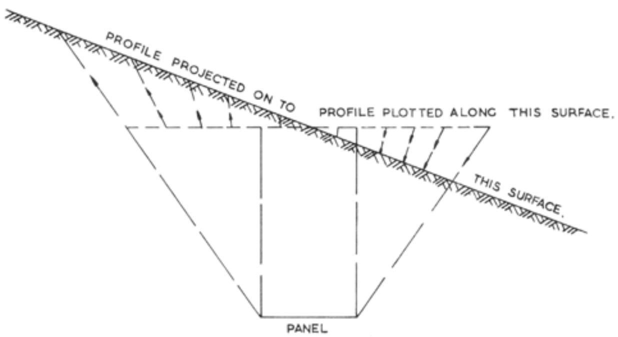


Fig. 19

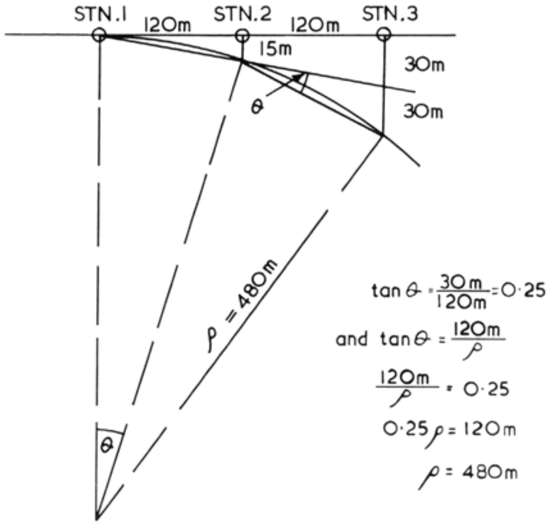


Fig. 20 Curvature and radius of curvature.

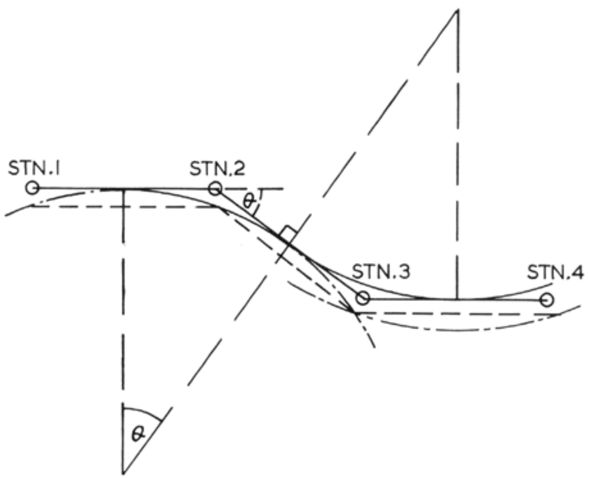


Fig. 21 Use of tangential measuring bays instead of chords.

If, for example, a seam were dipping at 1 in 4 and the calculated strain for a level seam were 1 mm/m, the corrected strains would be 0.56 mm/m on the rise side and 1.44 mm/m on the dip side.

Whilst this correction is not exact it provides a useful guide within the gradient range 1 in 10 to 1 in 1.5.

It is essential in calculating the strain to use the *mean* depth and the *plan* width of the panel.

Curvature

The prediction of strain from curvature is a useful device which can be applied to any part of any profile. It is convenient at this juncture to consider the nature of curvature. First we may recall that on page 20 a formula was given for calculating the radius of curvature of a circular part of a subsidence profile.

To calculate the radius from first principles we should consider Fig. 20 in which the amounts of subsidence are grossly exaggerated. Angle θ represents the differential slope and has the same value as the angle subtended at the centre of the circle by the chord. In order to find ρ, the radius of curvature, we can consider the triangle to be a right-angle triangle as the chord length is very small compared to the radius.

The use of chords to fix a subsidence profile is not good because a curve cannot be drawn through the chord intersections which have a smooth transition when it changes direction. Tangents are used instead and, as shown in Fig. 21, a smooth curve results because there is a common tangent at the point of inflection. The difference in radius between the chord and the tangent is insignificant. (The length, in either case, is called a "bay" length; this term is also used to denote the distance between observation stations where field research is carried out.)

The smaller the radius of curvature the greater is the strain; so strain is proportional to $1/\rho$. Since θ is a very small angle, if we consider it in radians and call the bay length l then $\rho = l/\theta$.

Thus strain is proportional to θ/l. Table 7 shows how θ and θ/l are arrived at, but it is necessary first to examine the significance of l the bay length.

Bay Lengths

It is common to find that a profile being examined across a given extracted panel has only part of its length measured, and owing to the interruption of the line caused by buildings or other surface features there are merely a few spot levels at irregular distances being observed over part of the affected surface. Uneven bay lengths due to this or other causes can be a considerable difficulty because the strain, either measured or calculated, is altered from the normal.

If a bay length is too long, the effect may be that of measuring along part of a smaller radius curve and part of a larger radius curve as in Fig. 22. Ideally the length should just cover the curve of smallest radius in the profile, but for practical purposes a useful length is found to be 0.05 h, i.e. one twentieth of the depth.

Fig. 23 shows how the apparent value of the strain alters with different bay lengths. The changes in length over two adjacent bays were added together and divided by the combined length of the two bays when a strain was obtained which was smaller than that over one bay. The same process over three bays produced an even smaller strain and over four bays a strain even smaller still. Fig. 23 indicates how the amount of strain may well increase with even shorter bay lengths than 0.05 h. But there is a practical limit to the reduction of the distance both in the field, where the number of observations would become too great and expensive, and in the office, where the calculation of strain could be taken to the length of false accuracy. Further research is needed in the question of bay length, and for the time being 0.05 h is recommended both for field observations and for computing strains.

If an observation line has been established with bays of unequal length it is best to carry out the calculation of strain at any station by reference to the shortest of the bays adjoining that station as in Fig. 24.

Having fixed the value of l, Table 7 can now be constructed and θ/l computed as the example in the Table shows.

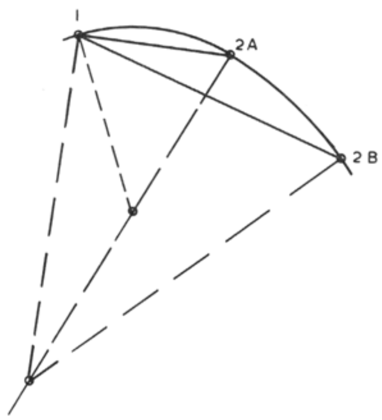


Fig. 22 Excessive bay length masking maximum curvature.

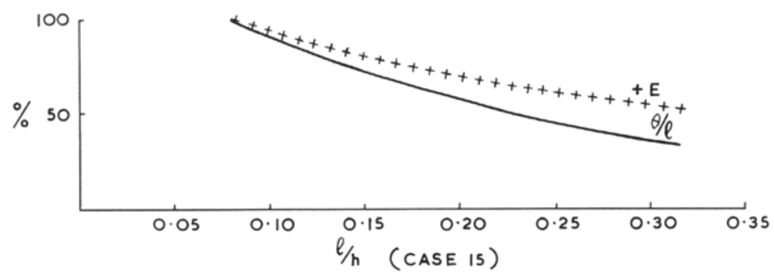
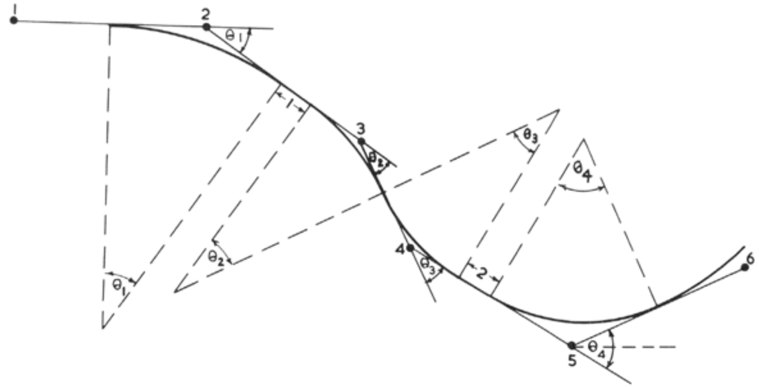


Fig. 23 Effect of change of bay length (l) on strain (+E) and θ/l.



N.B. θ_2 = (SLOPE 3 to 4) − (SLOPE 2 to 3)

θ_4 = (SLOPE 5 to 6) + (SLOPE 4 to 5)

SMALL STRAIGHTS 1 AND 2 DUE TO UNEQUAL BAY LENGTHS

Fig. 24 Effect of unequal bay lengths.

TABLE 7 **Subsidence Profile Data**

Case No. 15
Line 5

Depth 241 m
Surface: Trias

1	2	3	4	5	6	7	8	9	10	11
Station	**Subsidence** m	**Differential Subsidence** m	**Length** m	**Length** l m	**Slope**	**Differential Slope** θ	θ/l	**Predicted Strain**	**Measured Strain**	**Notes**
9	0.037									
		0.012	18.267		0.000657				+ 0.0003	
10	0.049			18.267		0.000492	0.000027	+ 0.0008		
		0.021	18.273		0.001149				+ 0.0007	
11	0.070			18.273		0.001857	0.000102	+ 0.0016		
		0.055	18.297		0.003006				+ 0.0012	
12	0.125			18.297		0.003002	0.000164	+ 0.0020		
		0.110	18.309		0.006008				+ 0.0021	
13	0.235			18.294		0.003995	0.000218	+ 0.0023		
		0.183	18.294		0.010003				+ 0.0005	
14	0.418			18.245		0.001671	0.000092	+ 0.0015		
		0.213	18.245		0.011674				− 0.0011	
15	0.631			18.245		0.003027	0.000166	− 0.0020		
		0.158	18.273		0.008647				− 0.0022	
16	0.789			18.273		0.004114	0.000225	− 0.0023		
		0.083	18.309		0.004533				− 0.0022	
17	0.872			17.959		0.003030	0.000169	− 0.0020		
		0.027	17.959		0.001503				− 0.0015	
18	0.899									

Computing and Plotting Strain

The mean values of the strains proportional to θ/l have been ascertained within reasonable limits of accuracy, and are shown on the prediction graph in Fig. 25.

Having entered all the strains in column 9 of Table 7 the measured strains (such as those on a control bay or bays) should be entered in column 10 and a check made for reasonable agreement.

Before plotting the strain profile, if this is required, it is essential to note the sign of the strain, i.e. whether it is positive (extension) or negative (compression). This is particularly important in the case of an irregular, undulating subsidence profile, which should be referred to. The *convex* curve from the limit of subsidence to the transition point gives the plus sign to the strains, and the *concave* curve along the bottom of the trough gives the minus sign for the compression.

It is important to note that the computed strains are plotted at the observation stations and not midway between them as would be the case with measured strains.

Fig. 26 is an example of a complex profile and shows how the computed strains compare with those measured in the field. The first part of the computed strain curve, stations 10–17, is in fact the example in Table 7.

Sometimes a profile of subsidence is available in which the stations are too far apart so that the curve is really a series of long straight lines joining the plotted subsidence points. Such a "curve" can be smoothed out by plotting on a large scale and drawing a curve which passes through the points instead of straight lines. The datum line of the profile can then be divided into suitably small bays – e.g. 0.05 h – and the subsidence read off at each station according to the subsidence scale. The strains can then be computed in the manner described above. If strains were measured on the over-long bays they would present a false picture and the computed strains should be substituted.

The strain on any particular small part of a profile can be computed when it is necessary to examine the probable effects of working on a special structure, and for this purpose it is necessary to predict only a small part of the subsidence profile – two stations more than the distance over which the strain is to be computed.

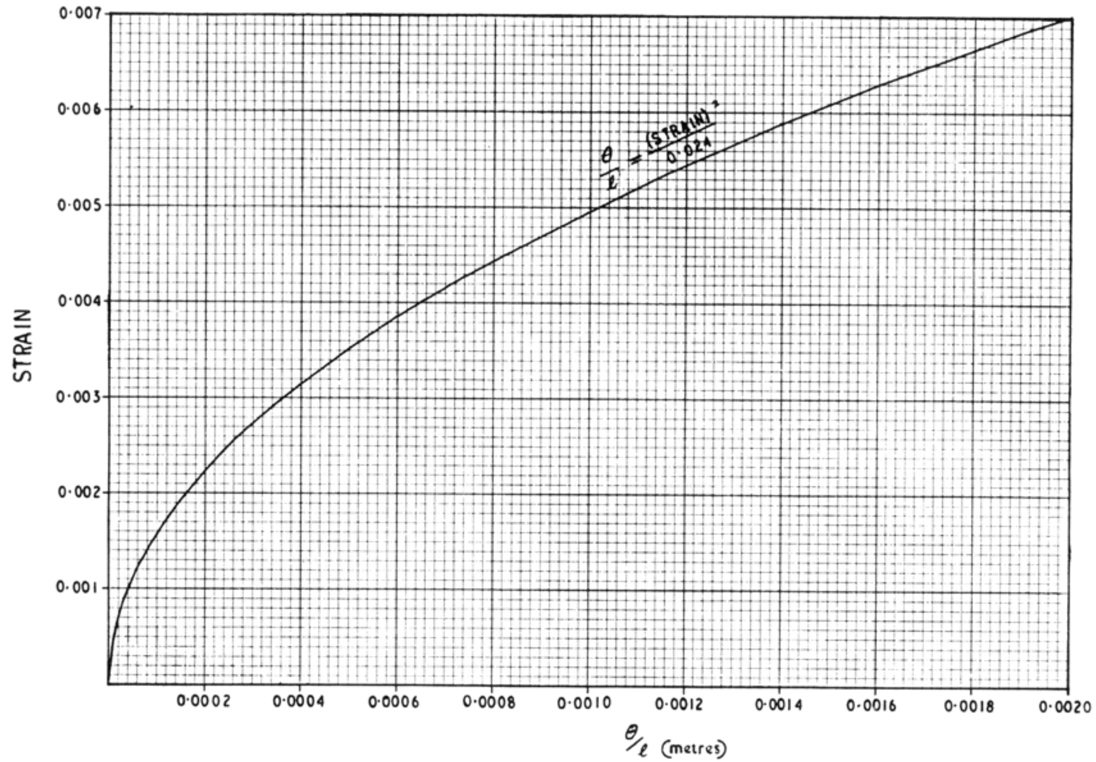


Fig. 25 Relationship between strain and θ//metres.

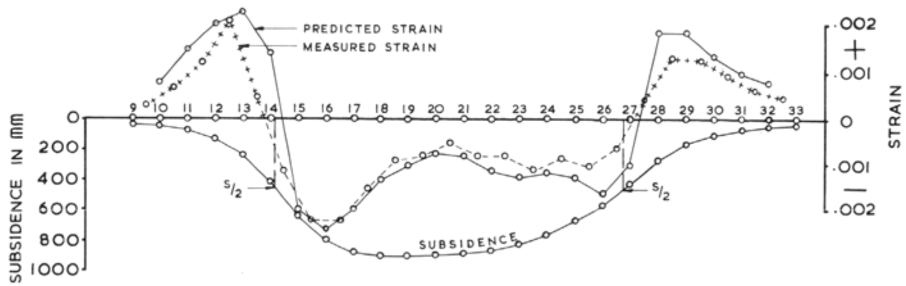


Fig. 26 Comparison of predicted strain with measured strain.

Section 5: The Relationship of Subsidence to the Position of the Excavation and to Time

Subsidence Development

Fig. 27 shows the development of subsidence at a surface point as a coal face passes below and beyond it. The curve is a typical development curve and represents the average shape and position of a number of such curves plotted from data obtained from different coalfields.

The measurable subsidence is seen to start when the face is within about three-quarters of the depth (0.75h) from the surface point P, and reaches about 15 per cent of the maximum possible subsidence when the face is below the point. ("Below" is a simplification of the true position, which is – a point in the seam lying on a line drawn normal to the seam through the surface point.)

When the face has advanced a distance equal to about 0.7h beyond the surface point, and either passes further beyond that point or stops, active subsidence of the point is complete. This distance is somewhat variable, as the last 2 or 3 per cent of the subsidence is as difficult to determine as the first 2 or 3 per cent, and this leads to the inevitable difficulty of fixing the limit distance both in a development curve and on the flanks of an excavation. Within a distance whose accuracy depends upon the width of the error-band of the levelling, the active subsidence is almost fully developed after a coal face has travelled a distance about equal to the draw beyond the surface point in question.

In addition to the active subsidence there is a time-dependent element in subsidence development which has been investigated in the field by both automatic photographic methods and by conventional survey methods. As Fig. 28 indicates, active and time-dependent subsidence occur together until the active subsidence ceases, as above, and thereafter only time-dependent subsidence occurs. This is called residual subsidence and the amount varies according to the distance behind the face and in front of it. Those points which were subsiding fastest have the most residual subsidence; other points have progressively less towards the limit of draw, both forwards and backwards. Fig. 29 shows a longitudinal profile (line of advance) in which the maximum residual is, typically, about 9 per cent of the total subsidence and that this maximum residual occurs, again typically, at the point of half maximum subsidence.

There are some more remote parts of the profile where the subsidence ceases abruptly and finally after the face has stopped within the critical area of that part of the surface. Virtually no time lag occurs between the time of a face stopping for a holiday period and the subsidence development almost stopping

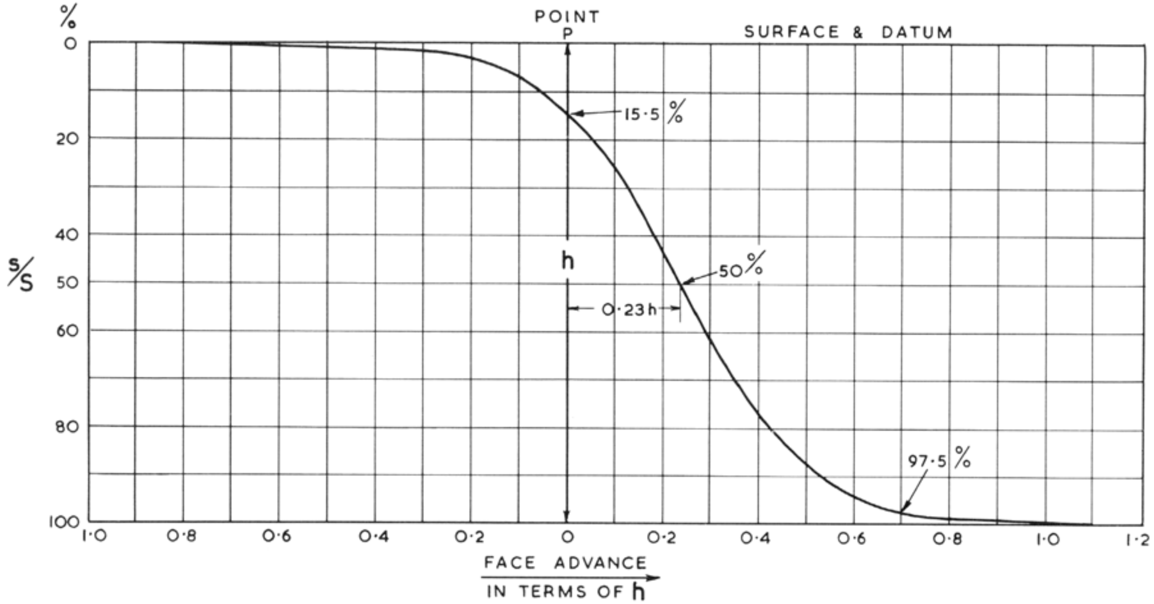


Fig. 27 Typical subsidence development curve.

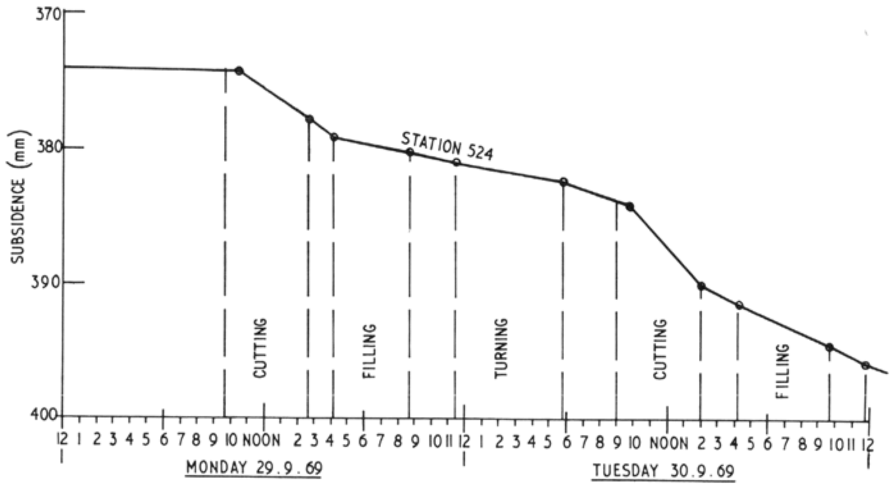


Fig. 28 Active and time-dependent subsidence.

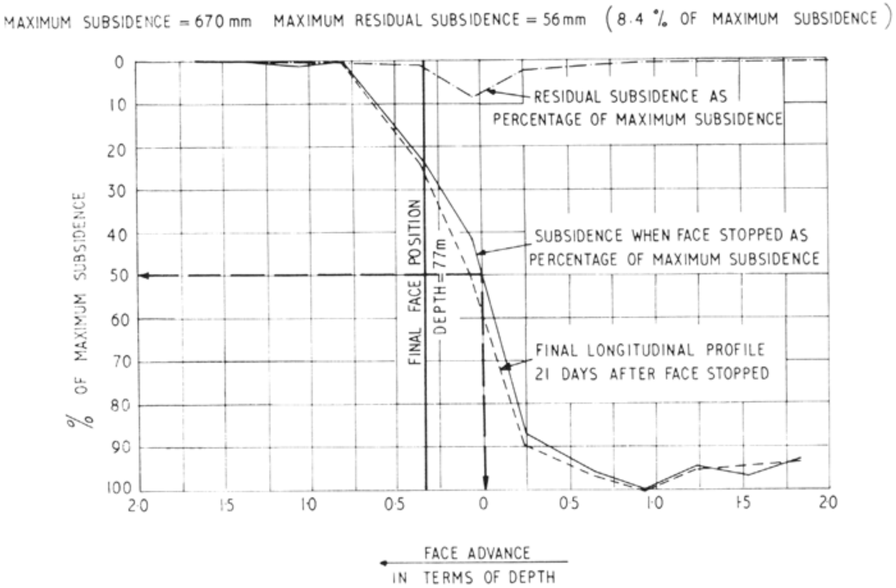


Fig. 29 Residual subsidence profile.

or suddenly slowing down as is illustrated in Fig. 30. The subsidence occurring during the holiday period was time dependent subsidence.

Subsidence begins at the surface as soon as a sufficient distance has been travelled by a face power loader for the strata at the seam horizon to begin to relax. Thus there is no "time factor" in the transition of the movement to the surface which is almost instantaneous. In cyclic mining only time-dependent subsidence occurs between cutting shifts.

The existence of residual subsidence and the wide interpretations of the time taken for it to occur are often referred to in claims against the Board. These should be treated with great reserve. It is sometimes claimed that damage occurred several months (or even years) after the workings passed beyond the critical area of the subject of the claim and that the residual subsidence was "the last straw that broke the camel's back". Bearing in mind the usually negligible amount of movement possibly attributable to residual subsidence, it is more likely that the damage – if it is mining damage – occurred during working and has either just been uncovered or has been ignored until a convenient time for repairs.

This is not to say that investigation does not need to be carried out regarding the reported cases of delayed subsidence. For example, open gate-roads could close after a lapse of time and, in shallow workings, could theoretically cause delayed subsidence – perhaps of measurable amount. It is significant that the time taken for the residual subsidence to occur does not normally exceed the time taken for the face to travel a further distance equal to half the radius of the critical area; it is often less.

On the other hand, unusually strong strata in the overburden can delay the residual subsidence (and reduce the amount of total subsidence) but such cases are the exception. Fig. 31 shows the difference between the behaviour of entirely Coal Measures strata and Coal Measures strata with thick beds of stronger rock. There can be more maximum residual subsidence with the latter and there is certainly more residual subsidence all along the subsidence profile.

Pillar and Stall Workings

None of the foregoing remarks applies to pillar and stall working. Here the development of subsidence may be different in every case, since there are such a great variety of pillar sizes and depths.

Pillars may fail after years have elapsed, the amount of movement depending on the heading space available into which they can crush. Or they might be forced through a soft floor such as fireclay, especially if this should become wet, the result being to lower the surface just as effectively as though the pillars had crushed and spread. Where the floor is soft the limiting factors are the thickness of the soft floor stratum and the space available in the headings into which the floor can be forced. With properly designed pillars the roof subsidence may not reach the surface in measurable amounts. (Mine layout design is referred to in Section 10.)

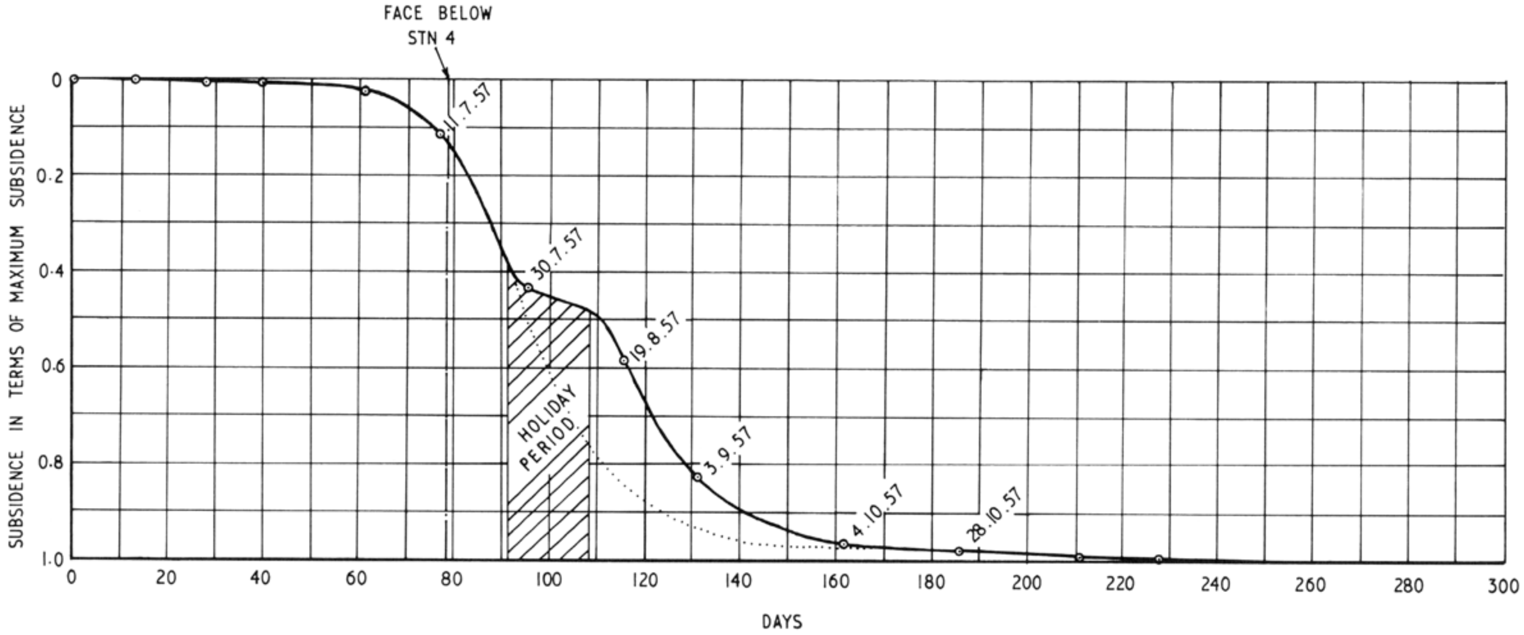


Fig. 30 National Case No. 9—time-subsidence curve.

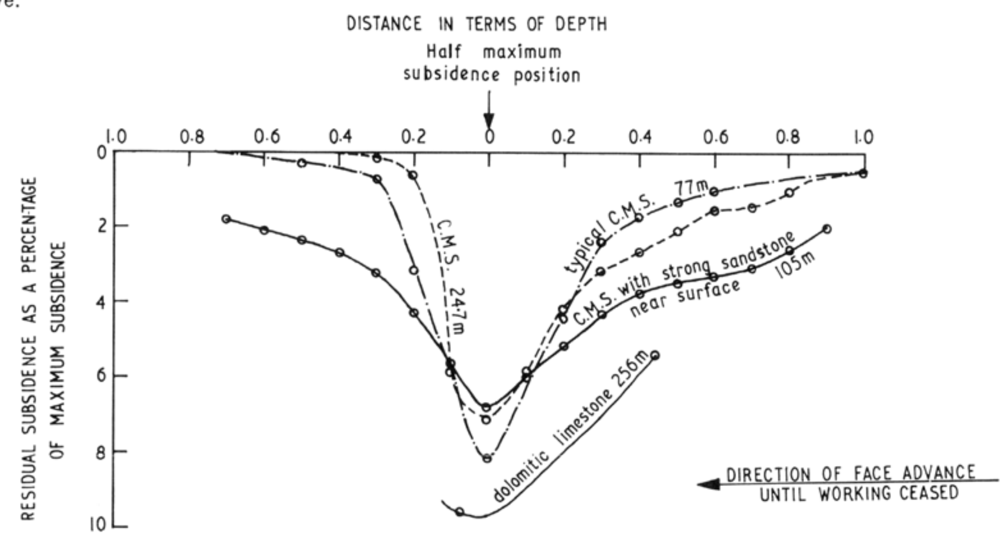


Fig. 31 Comparison of residual subsidence profiles.

Goaf Treatment

The type of goaf treatment does not affect the development of subsidence to any measurable extent. A paper dealing with alternative goaf treatments (18) has demonstrated that the subsidence develops only according to the dimensions of the excavation (depth, width and advance, etc.) and irrespective of whether it is caved, strip-packed or solid stowed either by pneumatic or mechanical means. Fig. 32 reproduces the graph demonstrating this similarity of behaviour.

Time/Subsidence Relationship

Permanent repairs should not normally be carried out if a damaged property is due to be undermined again in another seam or by another face in the same seam lying within the critical area. First-aid repairs are often satisfactory for short periods, but discretion is necessary when long intervals are likely to occur between successive excavations or when the property is going to be in an unstable state over a long period.

If the stability of a building is affected and the site is required for development, an estimate should be made of the earliest date when building could safely commence. This should be done by plotting a development curve or a time/subsidence curve and usually ignoring any possible residual subsidence period, which in any case would probably be covered by the site preparation period.

The nomogram in Fig. 33 may be used for this estimation of total time for a single face working. Starting at the depth line, the limit angle is next chosen and this gives the diameter of the critical area (2R). The value for the rate of advance of the face is next intersected to give the total time on the right-hand half of the base line. In the example the broken line shows that for a seam 450 m deep and an angle of draw of 35°, the width of the critical area is 630 m. With a face advance of 1400 m per year the time taken to work the panel is about 6 months.

Research Work

The field investigation needed to throw further light upon the cessation of subsidence is primarily the continuance of levelling of the most accurate possible quality to indicate a period of no movement so that the beginning of that period can be accepted as the end of the subsidence development curve. This involves levelling for as long as is reasonably possible after the coalface has passed beyond the supposed critical area, although the levels need only be taken about twice a year.

It is important that development curves should be plotted *in every coalfield* in order to be able to demonstrate the behaviour of the ground locally and not to have to rely on the principle proved in other parts of the country.

Fig. 34 shows the start and finish of the subsidence in Case No. 9* and indicates a useful method of determining the final amount of subsidence.

A mean line was drawn between the plotted points in order to reduce to a

* See Appendix 1.

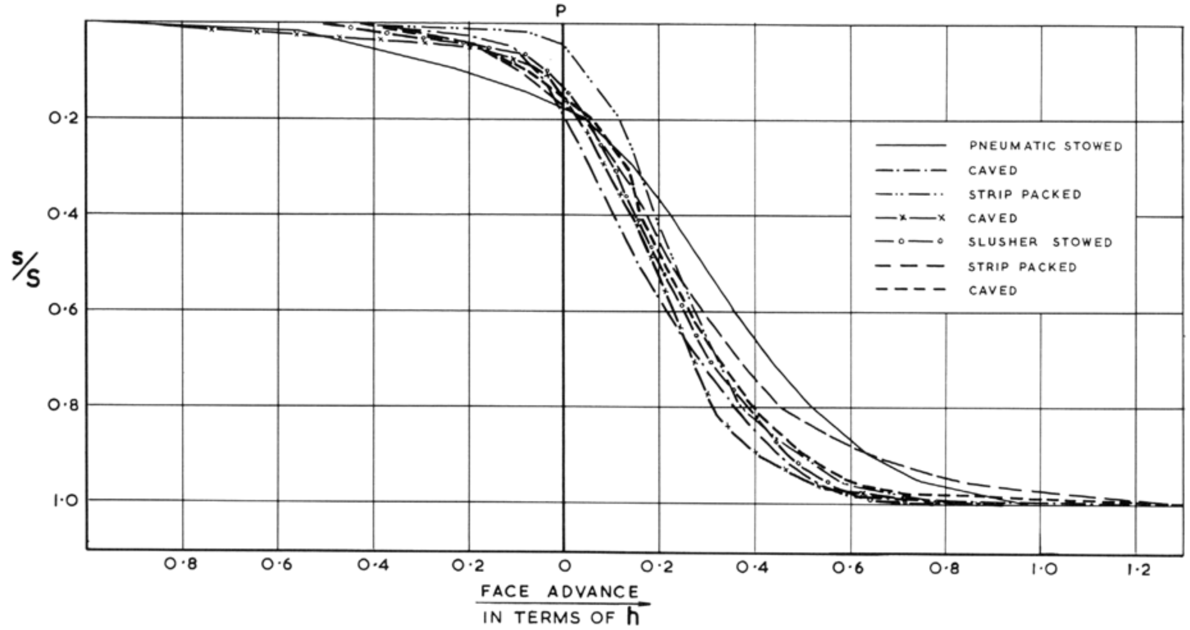


Fig. 32 Subsidence development with various goaf treatments.

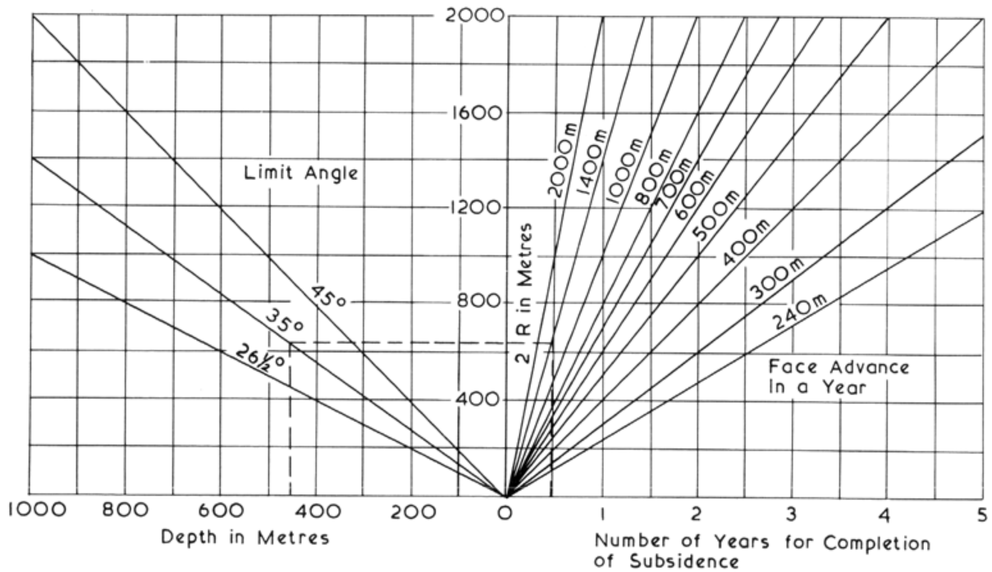


Fig. 33 Nomogram for estimating duration of subsidence.

smooth curve the zig-zag line occasioned by levelling error and natural causes. The levelling off of the mean line shows the points of commencement and of cessation of subsidence (which was plotted in millimetres) and the amount corresponding to 100% of the final subsidence can be read off.

The error band of ±5 mm is shown to indicate the scale of the plotting and represents an acceptable error in this case although the mean curve is probably closer to the true maximum than 5 mm. It is advisable in all cases, to select an acceptable error in this way.

When plotting the development curve it is important to measure the distance of any position of the face from the surface station with proper regard for any inclination the seam might have in the line of the face advance. This is done by drawing a line from the station to the seam at right angles to the seam and measuring from the intersection of this line with the floor of the seam.

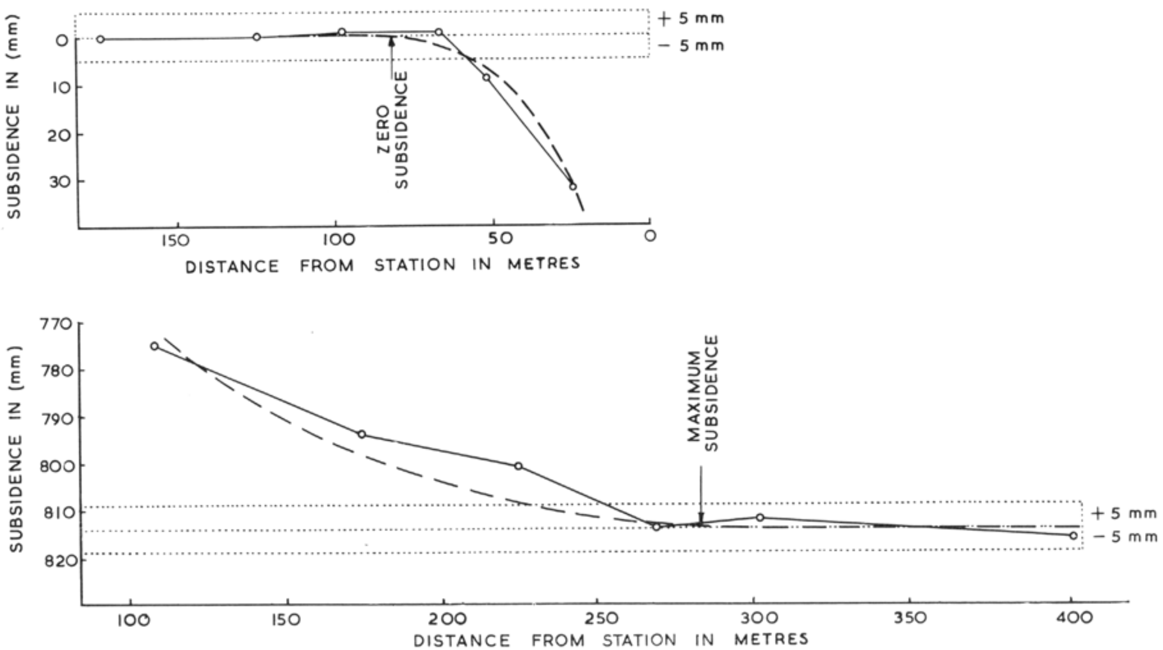


Fig. 34 Start and finish of subsidence development (Case 9).

Section 6: Relating Ground Movement to Surface Damage

Damage Due to Subsidence

The vertical component of ground movement causes changes in gradient in the ground which can adversely affect, for example, drainage, tall buildings and machinery in factories.

The calculation of tilt was dealt with in previous sections of the handbook and it will be recalled that the *maximum* tilt over a given extraction occurs for only a relatively short distance. Accordingly the whole of a small building or a machine may be tilted by the maximum amount, whereas a sewer or drainage pipeline may be severely tilted for only a small part of its length, which often means that the pipe continues to function to the extent that it is affected by the overall tilting. It is usually convenient to consider sewers in lengths between manholes and to calculate the existing gradient from the invert levels at the manholes.

Having observed or calculated the tilt due to mining subsidence and noted whether it is the same way as the flow of the drain or counter to it, the resultant gradient is calculated as follows:—

If a drain falls at 1 in 90 and the subsidence-induced gradient is in the same direction at 1 in 420, then the total gradient is best calculated by reducing the gradients to percentages and adding or subtracting them thus:—

$$\frac{1}{90} = 1.111\% \text{ and } \frac{1}{420} = 0.238\%$$

Adding these together, $1.111\% + 0.238\% = 1.349\%$

Converting back to a gradient $\frac{100}{1.349} = 1 \text{ in } 74.1$

If the induced gradient were a counter gradient then the result would be

$$1.111\% - 0.238\% = 0.873\%$$

Whence $\frac{100}{873} = 1 \text{ in } 114$

It would be for a public health engineer to decide whether at 1 in 74 a pipe would have too much scouring action or whether at 1 in 114 it would have a flow fast enough for it to be self-cleansing.

Any adverse condition may be only temporary, as more coal may well be worked which would restore the original gradient. Even backfalls over short distances can be tolerated as a temporary matter, usually when deep manholes can, by being surcharged, even out the overall gradient.

The surcharging of open streams similarly depends on the height of the banks or the height to which they can economically be raised to provide suitable freeboard.

Another drainage problem is the overflowing of filter beds at sewage disposal works, and of water reservoirs, etc. The amount of free-board in such installations should be compared with the subsidence tilting to predict any overflowing.

Apart from subsidence causing ponding and flooding, the abrupt troughs formed by incomplete extraction of shallow workings (i.e. with coal pillars left between panels) can cause a switchback effect which cannot be tolerated on railways or motorways.

The tilting of factory machines and plant can be classed as damage when the degree of tilting renders the equipment inoperable and cannot be rectified by 'built-in' methods. The latter are, in fact, quite common and consist of either screw feet which can be adjusted to re-level the machine or instrument, or holding-down bolts which can be loosened for the insertion of wedges or shims. It has been observed that only rarely are machines checked frequently for level; many are not touched after installation. This means that fine changes in gradient are tolerated or go unnoticed.

The following example is useful. The maximum gradient from working a seam is 2.75 S/h (see 'Slope' in Section 3 and also Fig. 15); in a 1 m seam 600 m deep this amounts to about:

$$2.75 \times \frac{1 \times 0.9}{600} = 0.004 = 1 \text{ in } 250$$

It may well be that a tilt of 10mm would not affect the operation of, say, a lathe or boring machine 2.5m long but could cause overflowing of a tank or the malfunction of a drip-feed unless re-levelling devices were built in.

Tall buildings and chimney stacks also require careful consideration when predicting the results of undermining. A tilt similar to that mentioned above (1 in 250) is unlikely to cause any reaction in the structure due to redistribution of weight. A chimney whose height is 40 m would be 0.16 m out of vertical, which is probably tolerable as a temporary matter and is certainly within the safety margin for stability. With the same angle of tilt an eleven storey block of flats, having a height of about 27.5 m would be 0.11 m out of vertical; this is unlikely to throw such an undue weight upon one side of the foundation as to cause an unstable condition due to subsoil penetration, but could affect the working of lifts. A tilt of this order is undesirable as a permanent feature, although such tower blocks are not generally built to an accuracy of less than two or three centimetres from the vertical.

Damage Due to Horizontal Strains

Horizontal extension and compression are the causes of the most commonly seen type of subsidence damage. Extension is characterised by the pulled open joints in brickwork or the fracturing of masonry, and compression by the

Fig. 35 Typical damage due to extension.

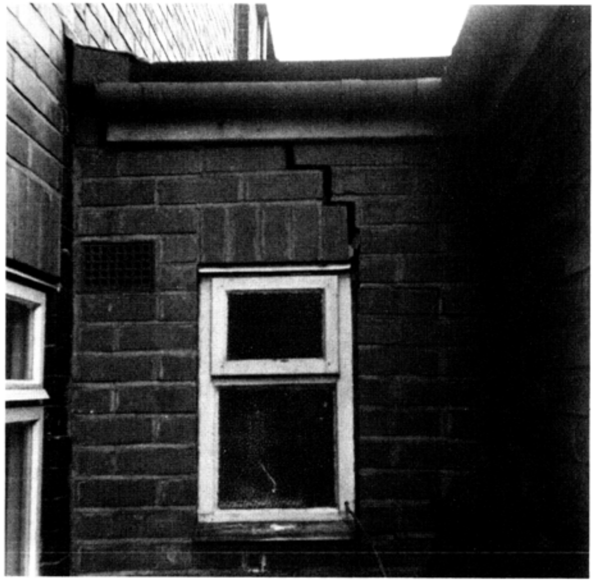


Fig. 36 Tensile strain following line of least resistance.

squeezing-in of voids such as doors and windows and the horizontal movement of brickwork where a continuous length has been thrust above or below some restrained part of the structure. Figures 35 to 40 show a variety of types and degrees of subsidence damage in surface structures.

The degree of damage is so dependent upon the nature of the structure (the materials, shape, age and design) that only approximate rules can be formulated for predicting the severity of damage. The main factors are the intensity of strain and the size of the structure to which the strain is applied. For example, a pair of semi-detached houses of overall length 14 m, when subjected to compression of 0.001 (i.e. 1 mm/m) will shorten by $14 \times 0.001 = 0.014$ m. As this amount will be mainly absorbed by distribution over the numerous voids in the brickwork, and to some extent by the brickwork itself, damage will be very slight or negligible.

On the other hand a terrace of houses 60 m long will suffer, with the same degree of strain, a shortening of 0.06 m which might well show at some weak point such as a change in roof level.

As a result of experience gained over many years and from the compilation of many such photographic examples a damage scale classification has been devised (see Table 8) which lists five accepted grades of subsidence damage. The factors of strain and building length used in compling the classification give only a general guide in the prediction of damage intensity. Accurate prediction also depends upon an expertise difficult to reduce to quantitative terms and which can only be acquired from a wide experience with buildings of various age and type of construction. Fig. 41 reproduces a graph which may be usefully employed in conjunction with Table 8.

Examples of damage where the ground strain was actually measured are to be seen in Figs. 42 to 45. The latter is also an example of the effect of the outcropping of a large fault which concentrated compression in the building.

In addition to the cases where the strain was *measured* there are several recorded examples of damage where a reliable *estimate* of the strain can be compared with the photographic record. For example, Fig. 46 shows the plan and section of workings in two seams which caused compressive damage to a railway bridge, while Fig. 47 is a photograph of the bridge which suffered appreciable damage from about 2 mm/m compression.

Fig. 48 shows the tensile fracturing of a wall where the maximum strain was about 4.5 mm/m; this example demonstrates the importance of the length of the structure and an important principle which must be observed in predicting damage. The seam, in this case, was extracted at a depth of 152 m and it follows that the whole tensile strain zone was only about 122 m long. The average strain over this length was probably of the order of 1.5 mm/m (about a third of the maximum) giving a total lengthening of $122 \times 0.0015 = 0.183$ m. An examination of the typical strain profile shows that over a distance equal to a tenth of the

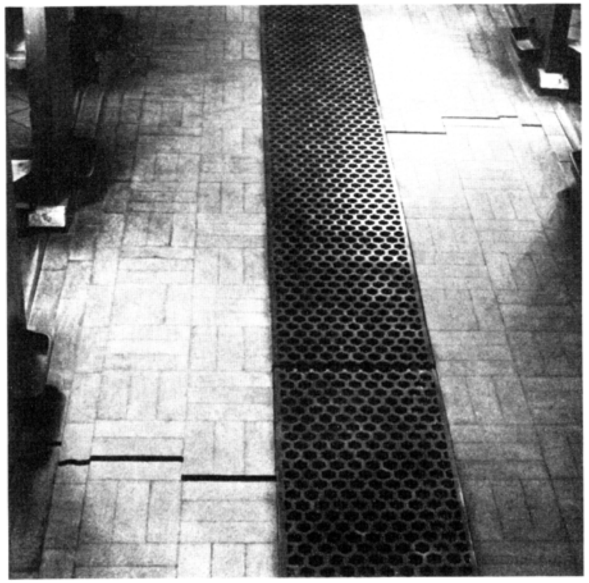


Fig. 37 Extension in a church floor.

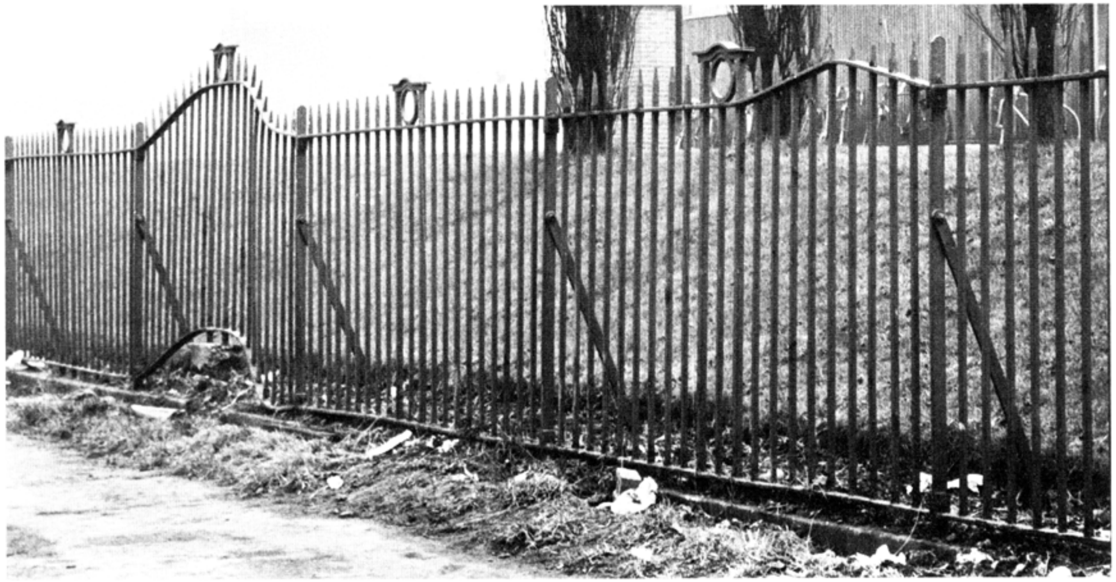


Fig. 38 Horizontal compression in iron railings.

Fig. 39 Severe compressive damage.

Fig. 40 Fissure in Bunter Pebble Beds.

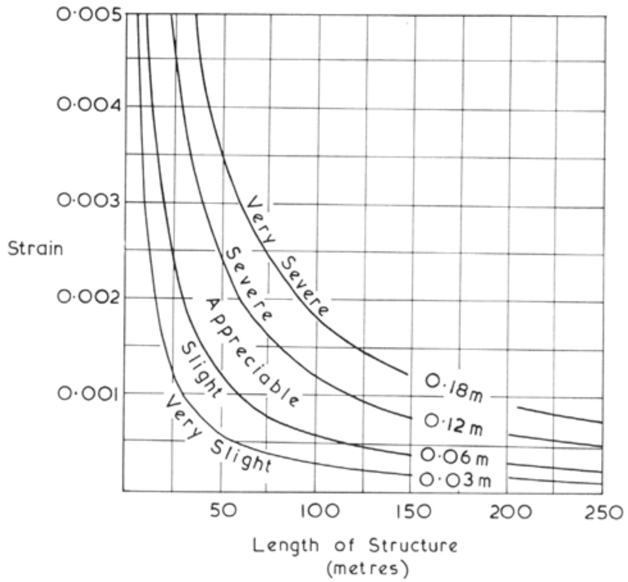


Fig. 41 Relationship of damage to length of structure and horizontal ground strain.

TABLE 8 **National Coal Board Classification of Subsidence Damage**

Change of Length of Structure	Class of Damage	Description of Typical Damage
Up to 0.03 m	1. Very slight or negligible	Hair cracks in plaster Perhaps isolated slight fracture in the building, not visible on outside.
0.03 m – 0.06 m	2. Slight	Several slight fractures showing inside the building. Doors and windows may stick slightly. Repairs to decoration probably necessary.
0.06 m – 0.12 m	3. Appreciable	Slight fracture showing on outside of building (or one main fracture). Doors and windows sticking; service pipes may fracture.
0.12 m – 0.18 m	4. Severe	Service pipes disrupted. Open fractures requiring rebonding and allowing weather into the structure. Window and door frames distorted; floors sloping noticeably; walls leaning or bulging noticeably. Some loss of bearing in beams. If compressive damage, overlapping of roof joints and lifting of brickwork with open horizontal fractures.
More than 0.18 m	5. Very severe	As above, but worse, and requiring partial or complete rebuilding. Roof and floor beams lose bearing and need shoring up. Windows broken with distortion. Severe slopes on floors. If compressive damage, severe buckling and bulging of the roof and walls.

Fig. 42 Severe damage from a tensile strain of 4.3 mm/m in a block of houses 26m long. The fracture widens appreciably towards the top. This damage was the result of extracting a seam 1.7m thick at a depth of only 137 metres.

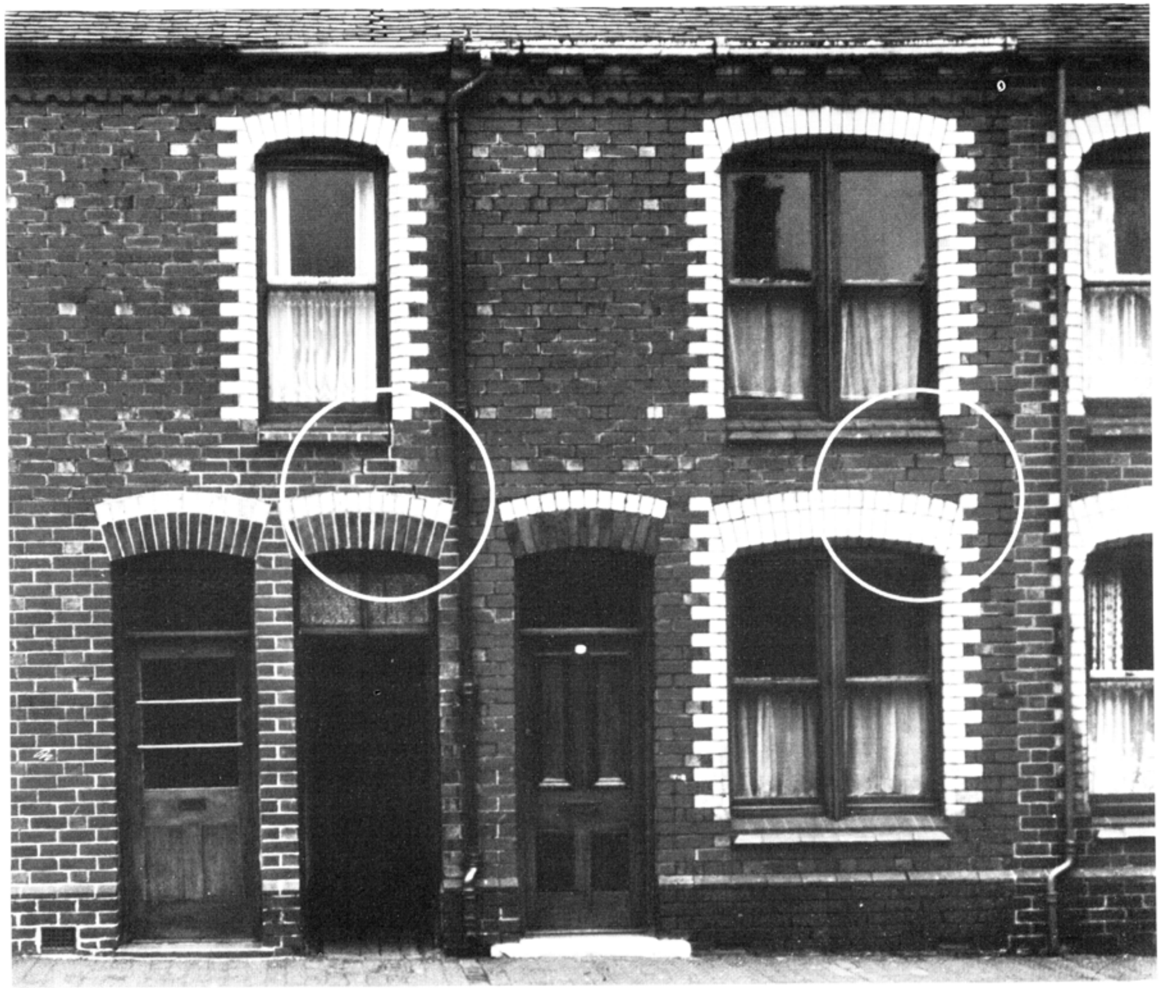


Fig. 43 Extension of 2.8 mm/m in terrace house block 46m long ; damage slight to appreciable.

Fig. 44 Slight damage caused by extension of 0.6 mm/m in a retaining wall 64m long.

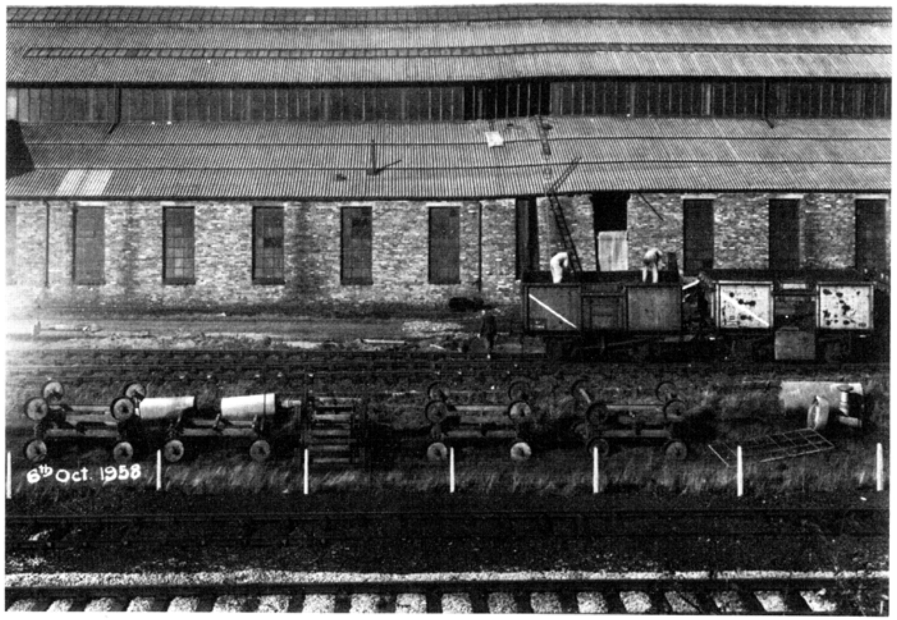
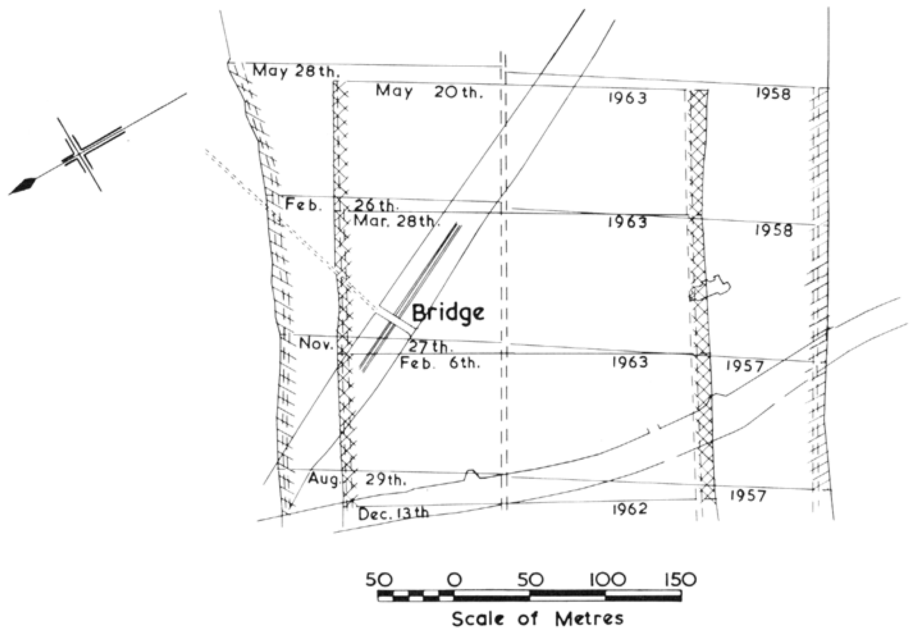


Fig. 45 Severe compression of 3.3 mm/m over 27.4m at subsidence ramp due to a fault outcrop. Average compression of 1.5 mm/m over a length of 110m. Note the partial closure of the window space at the position of the subsidence ramp.



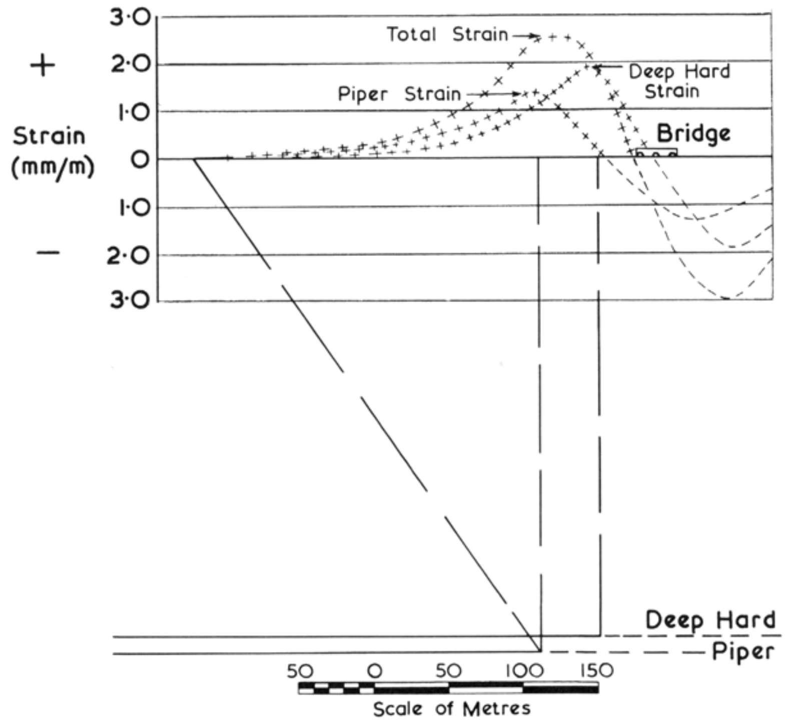


Fig. 46 Effect on a railway bridge of extracting two seams.

Fig. 47 Damage to the railway bridge shown in Figure 46 (South face, May 10, 1963).

Fig. 48 Fracturing of a wall due to an average extension of 1.5 mm/m.

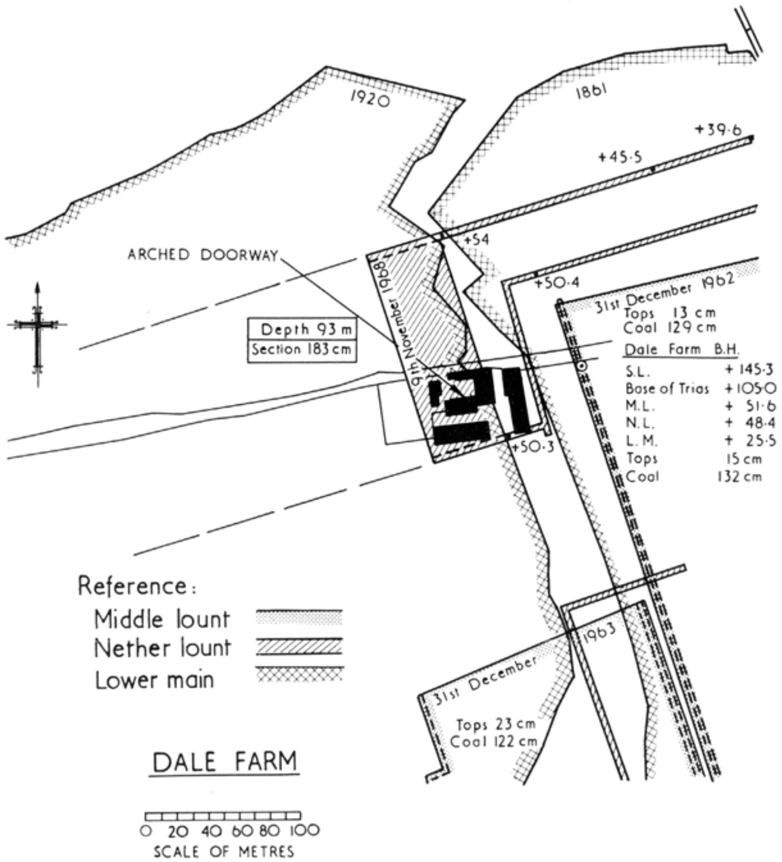


Fig. 49 Shallow workings causing intense compression.

depth spanning the maximum strain point the strain averages 0.9 of the maximum. Thus we have here a 15 m length of wall suffering a strain of $0.0045 \times 0.9 = 0.00405$, i.e. a change of length of about 0.06 m. The damage shown in the photograph is about equivalent to this amount of lengthening, but there were numerous smaller cracks along the wall to bring the total lengthening to roughly the calculated 0.18 m.

In a long structure it is essential to consider the extent to which the structure covers the strain zone and not simply to multiply the whole length by the maximum strain. Some buildings may lie half in extension and half in compression and the severity of the damage will often depend on the extent to which the building is anchored to the ground, i.e. whether it is tied all along its length or merely at each end like a bridge. In the latter the abutments may move away from or toward each other but the damage to the decking will often be reduced by the built-in provision at its ends for thermal expansion. Each abutment must be regarded as a separate structure when predicting the damage to the whole bridge.

Fig. 49 is the plan of shallow workings which caused very intense compression —about 9 mm/m at the site of a farm building which reacted by absorbing most of the cumulative shortening of the site in one doorway as shown in the photograph, Fig. 50. The interior damage in this case included the loss of bearing of a roof truss through the complete displacement of the supporting pier as shown in Fig. 51. In the same case the manner in which the ground moved undeı the building with a slip plane at about ground level is exemplified by Fig. 52 which shows a rising water main moved from its original position in the corner of the building.

It is difficult to predict if and where slip planes will occur in old buildings. However, the introduction of an artificially created slip plane in the correct position can be beneficial as a structural precaution in a new building (see Section 8). The subsoil in some cases may be soft enough to be compressed against the foundations under stress from mining so that stress is not transmitted to the superstructure. Fig. 53 shows ground which flowed under one end of a railway sleeper which was obviously held tightly by the chair and clip.

The effect of compressive strain of the order of 2 mm/m from deep workings is usually slight on a domestic building but the photograph in Fig. 54 shows the cumulative effect on an airfield runway. It also shows the disruptive effect of concrete used in a mining area. This wartime construction consisted of approximately 230 mm of mass concrete laid in panels about 6.1 m by 3 m with a macadam running surface. The propensity of the slabs to lift at the pour joints is well illustrated.

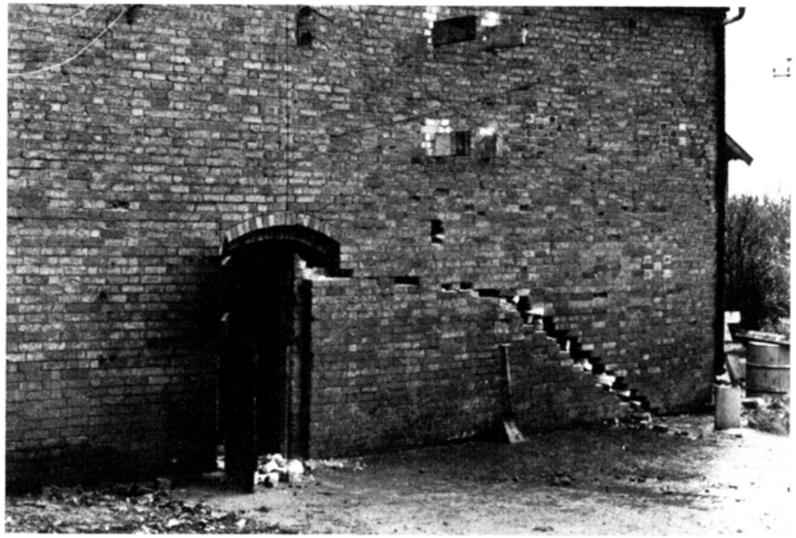


Fig. 50 Damage caused by severe compression due to shallow workings (Fig. 49).

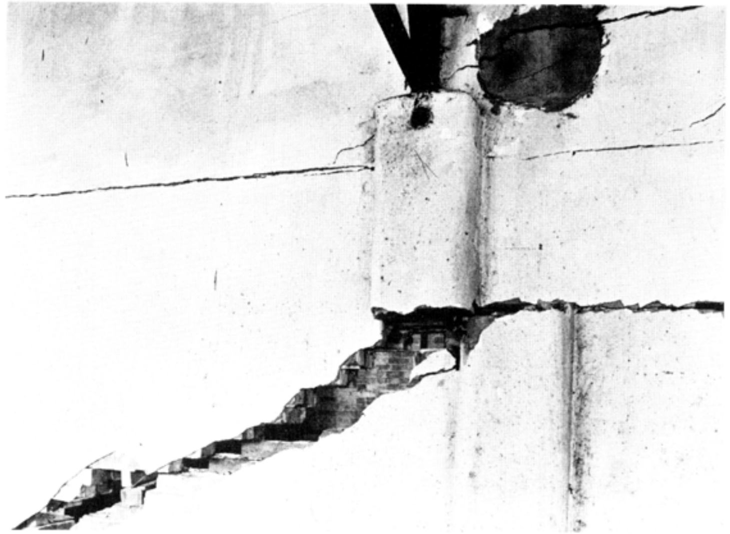


Fig. 51 Displacement of roof-supporting pier.

Fig. 52 Movement of floor relative to wall in zone of severe compression.

Fig. 53 Ground flow around a railway sleeper.

THE CHARACTERISTICS OF DAMAGE

It is useful to list the main types of structure and to consider briefly the ways in which each may be affected by ground movement, both in order to aid recognition of damage due to mining (as distinct from that due to other causes) and to ensure that no eventuality is overlooked when a programme of coal extraction is being planned below a built-up area or below a specific structure.

Buildings

The basic effects of both tilting and of horizontal strain have already been referred to, but it is as well to repeat that compression occurs over ground which has been given concave curvature and results in crushing, overthrusting and horizontal openings in brickwork, whereas convex ground causes extension with fractures tapering from the ground upwards.

Preliminary warning may be given by the appearance of cracks, the binding of doors and windows and the loosening or heaving of floor tiles. With both types of strain the cracks tend to develop first at ground level and to be most evident at voids such as doors and windows. The transmission of the damage upward from the foundation into the super-structure depends upon the nature of the structure, but tensile strain may cause opposite walls to move bodily apart with a corresponding loss of bearing surface in floor and roof joists.

Severe compression can cause the feet of opposing walls to move closer whilst the tops of the walls are strutted apart by roofs, but the converse is less likely with tensile strain.

It is important to know whether a structure is in a tension zone or a compression zone so that non-mining damage may be readily detected. For example opposite walls of a building which both lean outwards in a tension zone cannot have been tilted that way by mining but are most likely to be suffering from roof-spread, i.e. a thrusting outwards of the tops of the walls by the roof timbers, being incorrectly cross-tied, settling downwards and outwards.

Walls which have moved along the line of the damp-proof course are suffering from compression. The latter has closed up to some extent the voids below the damp-proof course which has a lower coefficient of friction than the brickwork above which tends to remain its original length. Quite commonly the effect of door openings is to allow the walls to compress more at ground floor level so that door voids close and windows lozenge.

The concavity of ground depends upon the depth of the workings and amount of subsidence, but it is obvious that except with very shallow extractions the concavity is not noticeable within a structure and local depressions in floors are then due to other causes as are sagging roof lines. There are frequently, however, unexpected aspects of damage which may be due to twisting motions or uneven foundations and are difficult to recognise unless the probable ground movement has first been calculated.

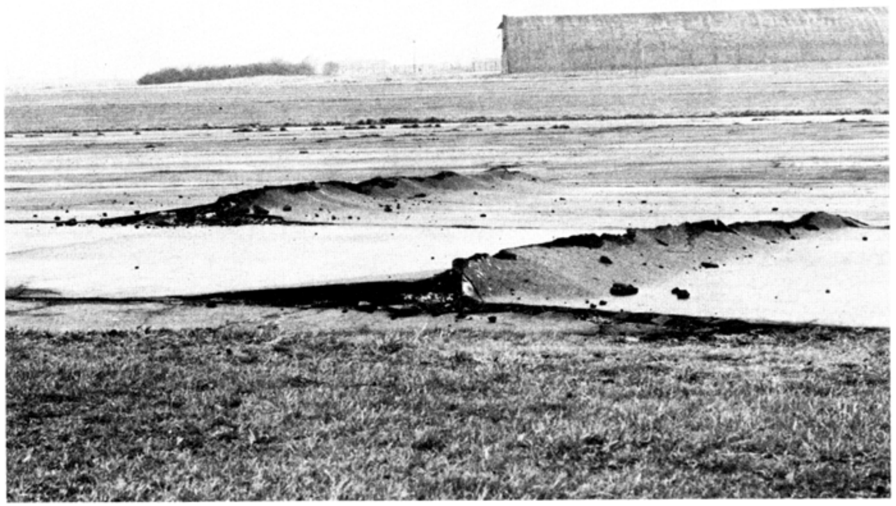


Fig. 54 Effect of 2 mm/m compression on an airfield concrete runway, having a macadam running surface.

Bridges

The supports or piers of a bridge can move toward or away from each other according to the type of horizontal strain, but they can also tilt in different directions or twist in the horizontal plane with complex and often serious effects upon the decking or arches.

In long bridges where there are a number of spans the bridge length may be greater than the subsidence wave and adjacent pairs of supports will suffer more relative movement than the net overall strain from one end of the bridge to the other.

The appearance of subsidence damage on bridge arches is often a complex of cracks and the precise originating movements are difficult to visualise. Compressive damage often appears as crushing and spalling off of brick facings and of horizontal joint openings. Fig. 47 is a good example of damage from compression.

Extension can cause the bridge supports to move away from each other to such an extent that the arch loses shape by flattening. But many arches unaffected by subsidence show open joints and flattening due to loss of tightness in the keystones. Perished mortar is one cause of this.

Roads

A highway is deemed to consist of carriageway, footpaths, verges, kerbs, stormwater drainage, fences, cycletracks and all ancillary services (i.e. sewers, gas mains, water mains, electricity cables and telephone cables) in so far as they affect the highway.

All these parts can be affected by subsidence the damage in the main comprising:

(a) distortion of horizontal and vertical alignment;
(b) fracturing, leading to deterioration of foundations;
(c) corrugations on the running surface;
(d) damage and displacement of kerbs, channels, flagging and fences;
(e) disruption of drainage;
(f) consequential damage due, for example, to action of water from fractured mains.

In the case of motorways in particular, severe local changes in gradient can be a source of danger to high-speed cars, while adversely affected drainage causing water to lie on a carriageway would also be dangerous.

Sewerage

The effects of subsidence on sewers has been referred to in considering the purely vertical movement, but there is also the possibility of cracked and broken pipes, drain joints and cracked manholes. Sewage disposal works, in addition to suffering the effects of tilting, may suffer cracking of walls and floors. Damage to pump wells leads to infiltration of water, increasing the demand on the pumps, and to misalignment of the pump shafts.

Water Supply

Water mains are affected in much the same way as the pipelines considered under "Sewerage". Breakages of mains usually occur at the points where ferrules have been tapped into the mains for service pipes.

Reservoirs are subject to the obvious hazards due to tilting and may easily be cracked to the extent that loss of water becomes serious.

The subsidence of *canals* necessitates systematic raising of the retaining banks and their waterproofing with clay puddle and in extreme cases the dumping of clay into the bottom to eliminate excessive depth.

Bridges over canals need to be raised when head clearance becomes too small.

Locks can be affected both by the subsidence in the way of loss of freeboard and by horizontal strains which can crack walls and spillways.

Culverts below canals can become submerged and watercourses can overflow.

Gas Supply

Gas mains are affected in much the same way as pipes for other services already mentioned and plant and buildings at gas works can be damaged by horizontal strain.

The effects of tilting on gasholders, when severe, can be detrimental both in the risk of losing the water seal and in the excessive binding of rollers on guide columns.

Electricity Supply

Underground cable systems vary as to method of laying, and damage from subsidence thus varies also. Extension usually shows at cable joints where discontinuity results and compression can cause the conductor to earth by contact with the lead sleeve over the joint in the sheathing. The lead sleeve itself can also break. Moisture may enter through cracked sheathing and cause breakdown and oil-filled or gas-compression type cables can leak.

On high-voltage cables the faults usually become apparent by causing protective-gear to operate, but with low-voltage cables open circuits may not be apparent for a considerable time.

Overhead lines. Long spans (240 m to 300 m) are employed with steel towers (pylons) and it is possible for only one tower to be affected. The result may be reduced electrical clearances due to tilting and reduced ground clearance due to subsidence. With very shallow workings tilting may be severe and also the individual leg foundations of the tower may suffer uneven subsidence which will distort the tower.

Section 7: Pseudo Mining Damage

There is a wealth of experience which needs to be gained by subsidence engineers and estates managers and which may be usefully employed in distinguishing subsidence damage from that caused by a variety of phenomena. Many claims against the National Coal Board have, of necessity, to be repudiated as being more properly attributable to one or more of these other causes. In some cases there is a mixture of both mining damage and other damage and the total claim has to be reduced accordingly, although proof of the other cause may not always be readily available. It may in some cases seem ungenerous not to repair all the damage as though it were subsidence damage, but if this is done the cause is not removed and the defect will recur. Thus the Board will be asked to repair again for unsatisfactory work and will in this way acquire a continuing liability for defects which were not their responsibility in the first place.

Fig. 55 Distortion of chimney stack caused by sulphate attack.
(Note the horizontal cracks corresponding to brickwork joints.)

Sulphate Attack

Perhaps the greatest single cause of non-mining damage is sulphate attack due to the interaction between Portland cement and soluble sulphate salts in moist conditions (sulphate of sodium, magnesium, and calcium, combining with the alumina in the cement; similarly with alumina in many hydraulic limes).

Most bricks contain soluble salts and today mortar usually contains Portland cement, so that all brickwork in this country is potentially liable to sulphate attack, but the attack is slow and requires the presence of water so that it takes place only where the brickwork is wet for long periods. Consequently such damage is most commonly found where the water can reach all sides as in parapets, copings, garden walls, retaining walls, and chimneys.

The effect of this attack is that the mortar expands, showing first horizontal cracks in the joints, and the face of the mortar may fall off. The expansion causes deformation of the brickwork, often in a way very similar to the compression movement due to mining. Later the mortar deteriorates and may become merely a wet powder. Sulphate attack is also often seen in brickwork which is covered by a dense rendering, where water penetrates at some faulty places and then cannot dry out through the rendering. The normal shrinkage cracks are first of all exaggerated and later horizontal cracks form along the lines of the mortar joints.

A chimney can be deformed by sulphate attack as shown in Fig. 55. Examples of this are also common with the use of slow combustion domestic boilers. The flow of the flue gases is much less than with an open range and the gases cool to below dew point before they leave the chimney. Sulphur compounds from the flue gases dissolve in the moisture condensing on the flue wall and attack the brickwork.

Concrete ground floors can also be damaged, either from natural sulphate bearing clays beneath or from hardcore filling which contains sulphate. A common filling containing sulphate is incompletely burnt colliery shale, which is often used in mining areas. This defect should be looked for in fairly new buildings.

Shrinkable Clay

Damage similar to that caused by subsidence also results from the movement of firm shrinkable clay. To measure the vertical movements the Building Research Station buried a series of plates 0.3 m, 0.6 m, 0.9 m and 1.2 m below the surface of the site on shrinkable boulder clay at their Research Station at Garston. They recorded vertical movements from 30 mm at 0.3 m below the surface to as little as 5 mm 1.2 m below the surface. The drying shrinkage is considerably increased by the proximity of fast-growing trees such as poplar, elm and willow. The seasonal movements caused by normal climatic drying are considerably reduced under paved areas; on the other hand the drying effect of tree roots is generally increased under paved areas because the paving shelters the clay.

The drying effect of tree roots is illustrated in Fig. 56. Of a row of survey stations sited alongside a large hedge, most subsided during or after a dry spell, the maximum subsidence being about 20 mm. They rose again with the subsequent increase in rainfall (shown by a thick black line as it was measured at the Area Office concerned) and Station 3 which was sited near a large tree subsided more than 76 mm during the drought and subsequently regained much of that fall. Station 10 actually showed a slight rise during the drought but this station was in a tarmac footpath. The broken straight line across the centre of the diagram refers to an observation line established in the Millstone Grit off the coalfield which suffered neither mining subsidence nor climatic changes.

Another example is shown in Fig. 57 where two ground stations in clay soil fluctuated in level by up to 25 mm with rainfall changes.

Unlike mining movement where horizontal strains cause the trouble, the damage caused to buildings by clay shrinkage is due to differential vertical settlement. The building itself protects the clay directly beneath it and the drying of the clay near the external walls causes them to settle. In settling, the outer walls pivot on the lower structure and tilt outwards, frequently pulling the first floor and the roof outwards also. In winter the clay will swell and cause cracks to close partially, but not entirely. Further shrinkage of the clay recurs in the next dry season, resulting in a tendency for the fractured portion of the house to settle a little more each year. In taking levels to explain the damage to a vicarage in Coalville a rise of 32 mm was recorded in the surface of the ground between October 1959 and February 1960.

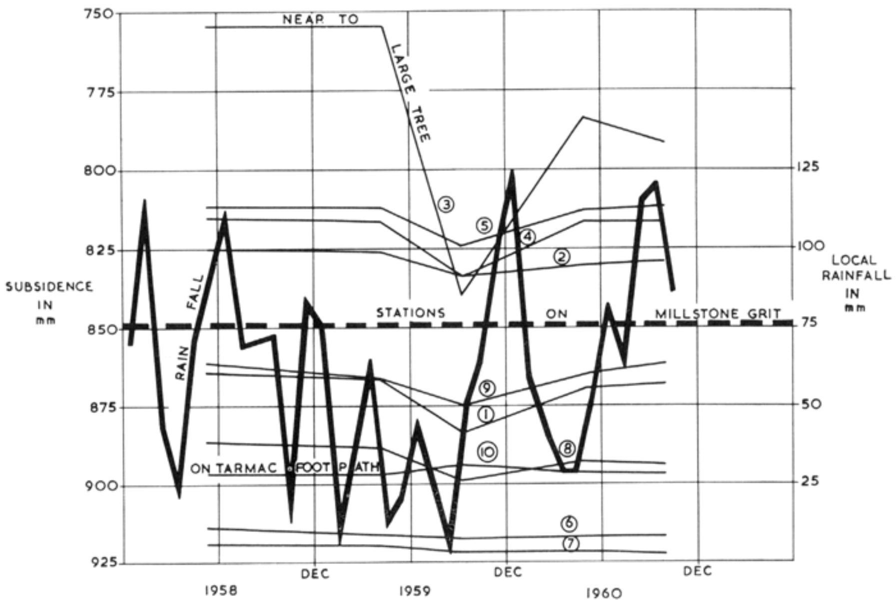


Fig. 56 Varying effect of rainfall changes on height of survey stations (numbered 1 to 10) alongside a hedge and tree.

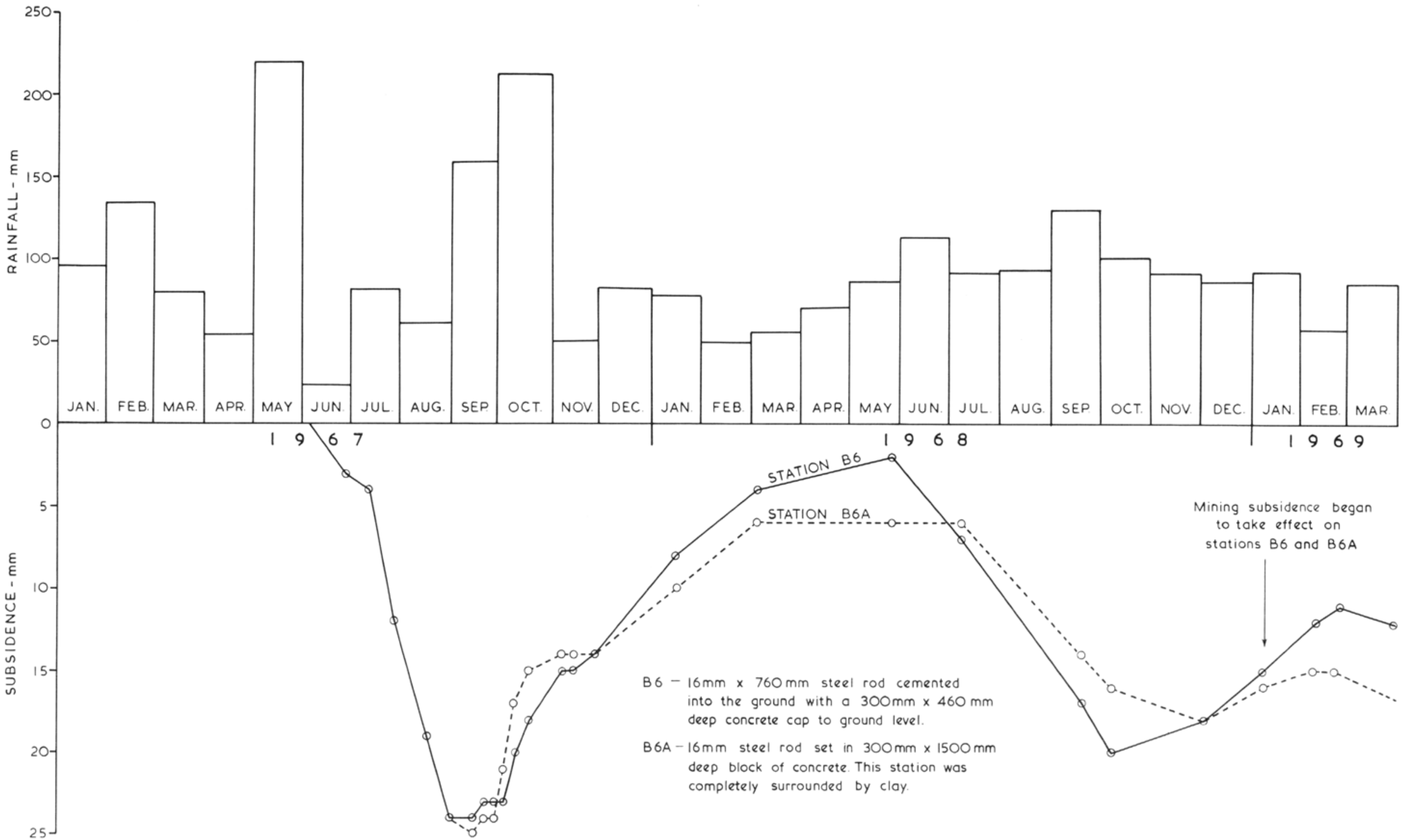


Fig. 57 Effect of rainfall changes on survey stations in clay.

Differential Settlement

An example of the effects of really wet clay, which softens and loses bearing capacity, is seen in Fig. 58 in which the marked curvature of the building in St. James's Park, London, is obviously due to the site being on the edge of a lake.

Fig. 59 shows the section through a house in South Wales where the annexe tilted away from the main part of the house because the clay subsoil had been softened by water from defective roof drainage to the extent that the foot of the end gable wall was sinking through the clay. The clay, when analysed, contained 25% of water. The levels on the house floor show a situation impossible in a mining subsidence trough – that is a change of gradient from level to 1 in 45.5 in the width of a house.

Differential settlement also occurs on made-up ground, particularly where a building has its foundations partly on made-up ground and partly on virgin ground. Similar damage is more localised where a building is partly on old foundations which have finished their initial settlement and partly on new foundations on the virgin ground. Fig. 60 shows a fracture due to uneven settlement of the foundations of a house well away from a coalfield.

Rust Damage

Corrosion of iron and steel embedded or enclosed in masonry can also produce cracking, as in Fig. 61. The same kind of damage is caused by iron dowels in stonework and by hooks built into piers to hang gates, cramps in copings, etc.

Glass window panes crack in steel casements when rust in the rebates expands and presses on the edges of the glass. Where cracks appear in fixed lights of a building subjected to movement it is difficult to deny that the movement is not the cause, but if the panes crack in an opening light which is still free to open, then it is probable that all the other cracked glass in the building is due to rusting.

Thermal Effects

Clay floor tiles can become loose on a concrete sub-floor, due to differences in shrinkage and thermal movement between the tiles and the concrete beneath. Sometimes the tiles form an arch and stay in that position, but at other times the arch breaks and the tiles fall loosely. In a new building this usually occurs within the first nine months, but following heavy frost it could happen in any building up to fifty years.

Where the mining movement has been slight it is worthwhile to remember that a concrete roof 15 m long will move 12 mm in Britain due to differences in temperature between summer and winter. Fig. 62 shows the effect of frost in a roadside.

Roof Spread

Roofs can spread owing to faulty construction, e.g. inadequate cross tying and in old buildings owing to timber decay, causing cracking in external walls which are pushed outwards at the top. Usually one wall has moved in a direction contrary to the effects of mining.

Fig. 58 Subsidence due to wet clay.

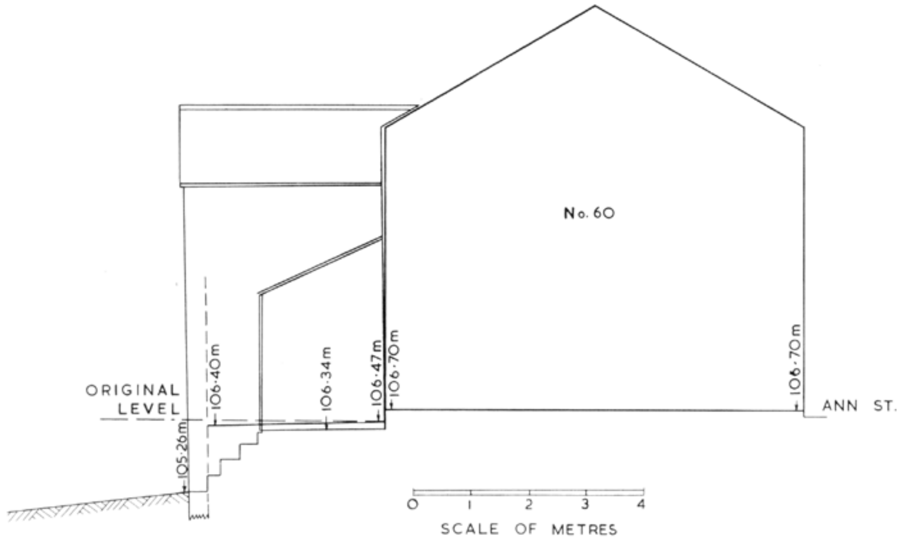


Fig. 59 Annexe of house tilted through penetration of wet clay.

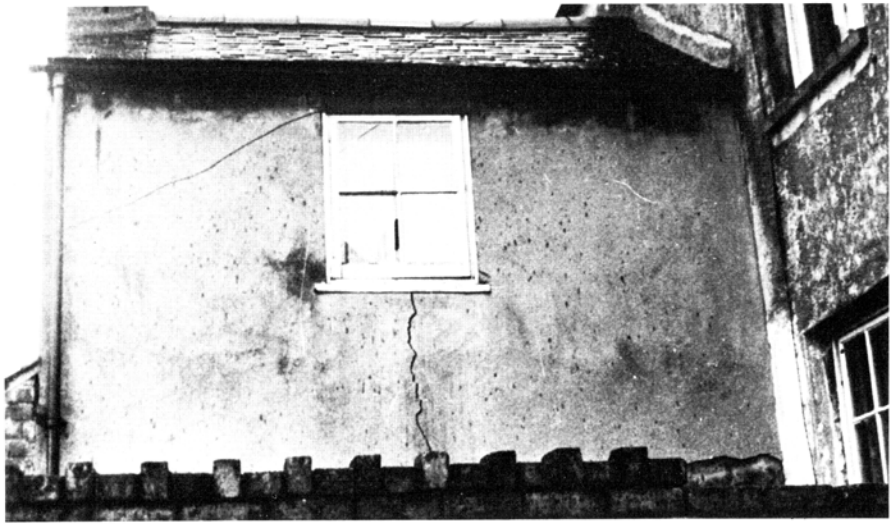


Fig. 60 Crack due to uneven foundation settlement in a house off the South Wales Coalfield (Cardiff).

There are, of course, innumerable defects commonly occurring in building, the causes in some cases being fairly plain, such as frost damage, a collapsed drain and sagging roof timbers, but in others rather obscure. A general characteristic of pseudo-mining damage is uneven lowering over relatively short lengths of structure.

Figs. 55 and 61 originally appeared in National Building Studies Bulletin No. 9 *Some Common Defects in Brickwork*.
Acknowledgement is made to Her Majesty's Stationery Office for permission to reproduce these illustrations.

Fig. 61 Cracking of brick pier owing to corrosion of partially embedded steel.

Fig. 62 Frost damage lifting stone sets in gutter.

Section 8: Structural Precautions (General)

The subsidence engineer is called upon from time to time to advise on the principles involved in the design of new structures and in the adaptation of existing structures with the object of reducing the damage that might be caused by subsidence. In this Section the design principles in new structures are considered first, because ideal or special methods of construction can indicate the best modifications to adopt in existing structures.

NEW STRUCTURES

Location

That it is advisable to avoid erecting any structure within several yards of the known position of the outcrop of any fault almost goes without saying. However, it is possible for a fault situated near a building to have a beneficial effect, since it can act as a useful safety valve for ground strain – the strains tending to be concentrated at the weak zone of the fault plane.

Foundations

Buildings erected in subsidence areas should be designed, so far as possible, to be either completely flexible or completely rigid. Partial strengthening of foundations or superstructures may merely increase the damage caused by ground movement. Foundations anchored to the subsoil are, through being deeply entrenched, bound to move – either to shorten or to lengthen – and to take up some curvature when the ground does so. This applies even to reinforced foundations, unless they are so deep as to form a beam or cantilever.

Buildings do not always change length exactly as the ground does, one reason being that they normally have shallow foundations against which layers of the more plastic type of subsoils (such as some clays) tend to compress and even bulge upwards. Also, old foundations which are fairly loose and compressible will sometimes fail to transmit movement upwards. For a new building to be able to resist horizontal ground movement it should be so designed that the superstructure is capable of sliding over its foundation on a slippery membrane. A flexible superstructure will accommodate ground curvature; a rigid one will ride above the curved ground and can, if necessary, be jacked to restore it to a level attitude if the ground remains permanently tilted or curved. Some system-built or industrialised housing units have considerable rigidity. Tall blocks of flats can be built with adequate rigidity so that they tilt en bloc; also, having small base dimensions and great stiffness of foundation, they can ride out strain and curvature, and can be jacked level if necessary.

The arrangements to permit these large buildings to ride out ground movements are incorporated at relatively small extra cost. But such precautions as sliding layers become a considerable extra proportional cost in the case of small houses and are often not justifiable. In an estate of some hundreds of houses, the cost of repairs to the relatively small number which may be damaged through coal being worked in the vicinity may be much less than the cost of incorporating special arrangements in every house.

Where the expenditure is justifiable, however, as is sometimes the case with houses and is usually the case with longer buildings (such as schools, blocks of flats or maisonettes, etc.) the underside of the foundation slab should be smooth and free from projections and should be placed on a layer of polythene sheeting. This should be laid in turn upon a friable layer such as 150 mm or more of sand. The tensile strength of the slab should be sufficient to resist a force equal to the product of half the weight of the structure and the co-efficient of friction at the underside of the slab. This co-efficient is usually taken as 0.66, and the tensile strength of mild steel reinforcement as 207 MN/m^2. In the case of a rigid superstructure of small plan dimensions it may be possible to make the slab thick and rigid, but otherwise it should only be thick enough to carry the imposed load and should be reinforced centrally in order to withstand horizontal stresses. Such a slab is laid at ground level so that little or no end or side thrust is encountered from the surrounding ground.

Where ordinary strip foundations are considered adequate, these should be similarly laid on sand or its equivalent; if the strip foundations are laid in trenches, some space should be left at the ends and these gaps filled with compressible material. If a basement is provided to a building the backfilling of the excavation should be carried out with compressible material, or alternatively an "area" should be left round the basement walls.

Superstructures

Where a traditional building is to be built without special foundations or stiffening, all parts of the superstructure should be as flexible as possible. Lime mortar should be used in brickwork, damp-proof courses should be of the bitumastic type; window frames should be of wood with oversize rebates, or if steel frames are used they should be mounted in wood. Fibre-board ceilings should be used instead of plaster, and drywork generally is beneficial including on walls. Arches of brick or stone should be avoided. Small buildings should be kept separate from one another by the avoidance of connecting wing walls, outbuildings or concrete drives. Drives and service roads should be of macadam or other flexible material.

Drainage

Drainage should be by flexible pipelines with telescopic joints as outlined in Section 9.

Drains should be laid to greater falls than normal, to allow for the counter gradients which may accompany subsidence. The maximum slope which can be induced in a variety of working conditions may be read off the graph in Fig. 63. These curves indicate the steepest slopes in the subsidence profiles induced by such workings.

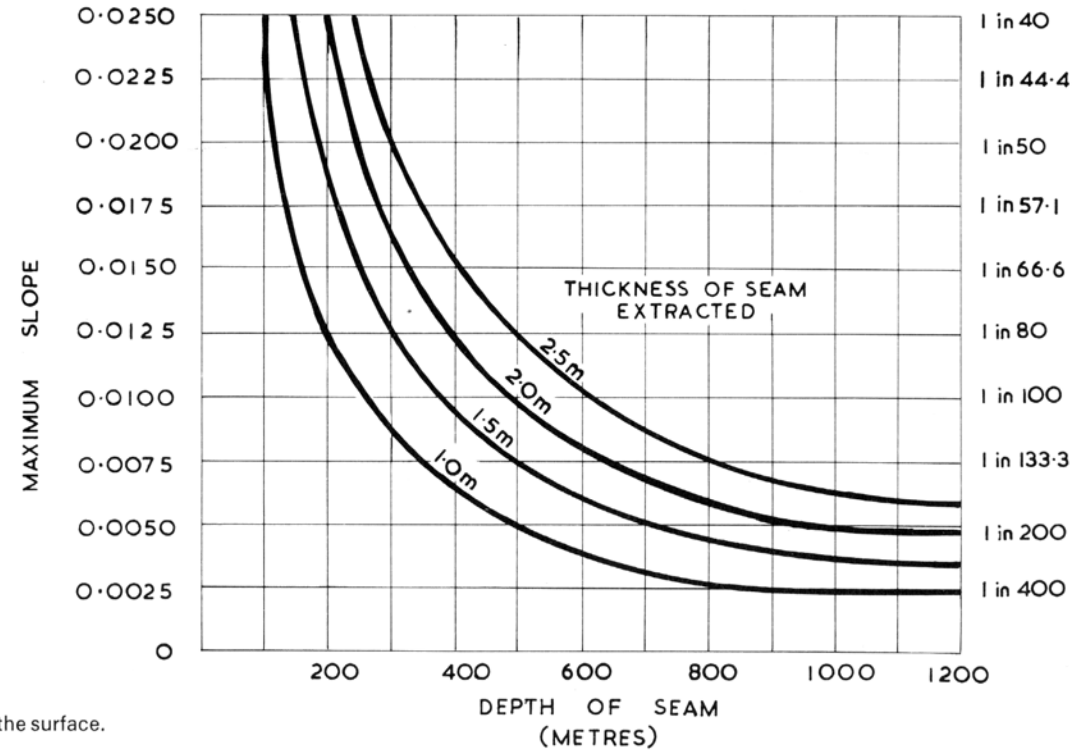


Fig. 63 Effect of depth on the maximum slope induced at the surface.

Provision of Gaps

In determining the intensity of strain to which a structure will be subjected, the total length of a structure is important. It may be that a short building will be completely within the zone of maximum strain; a longer building may occupy ground which is only in part subject to the maximum strain and is in other parts being strained to a lesser degree. To assess the overall damage the average strain over the length of the structure (building, pipeline, bridge deck, etc.) should be ascertained. The average strain must also be predicted before making allowance for extension or compression to be incorporated in the design.

Fig. 64 has been so designed that the total change of length can be read off directly on the right-hand horizontal scale for any length of structure (if necessary by interpolation) and for any of the given seam depths. The graph allows for the structure being sited in the worst possible position, i.e. straddling the maximum strain zone over the rib-side of an extracted panel of at least critical width depth ratio.

In the example given in Fig. 64 (dotted line), a building 30 m long is regarded as straddling the peak of the extension zone over an extraction of critical width in a seam 1.3 m thick and 150 m deep. The total change of length of 172 mm would cause damage to the structure involving (if it were tensile strain) open joints and fractures to the total width of 172 mm. Conversely 172 mm would be the total width of compression gaps to be incorporated in the design of a new building. In effect the principle involved is that of dividing long structures into much smaller units, the size of the gaps between adjacent units being estimated in this way.

Since curvature also occurs in the ground, the gap must be wider to allow for the degree to which the adjacent walls lean together at the top. Fig. 65 gives a ready estimate of the size of gap necessary to absorb the effect of curvature which must be added to the normal compression gap. Use of the graph is demonstrated by the path traced by the dashed line. Beginning with the seam thickness on the left-hand horizontal axis, a projection is made to cut in turn the graphs for depth (h), l/h, and height of structure, to give finally the length of gap resulting from curvature at the intersection with the lower vertical axis. The example given is the same as that in Fig. 64 of a shallow working (150 m deep), which naturally causes both the curvature and strain to be heavy, and if the building were 10 m high and 30 m long and had a structural joint at its mid length to allow for compression it would need a gap of 140 mm. This must be added to the 172 mm horizontal change of length, giving a total gap of 312 mm. The problem of weather covering for this sort of gap is rarely very great and a simple metal strip having slots for the fixing pins is usually sufficient. In structures where provision for thermal expansion has to be made, the size of the gaps can be modified to cope with strain, curvature and thermal expansion.

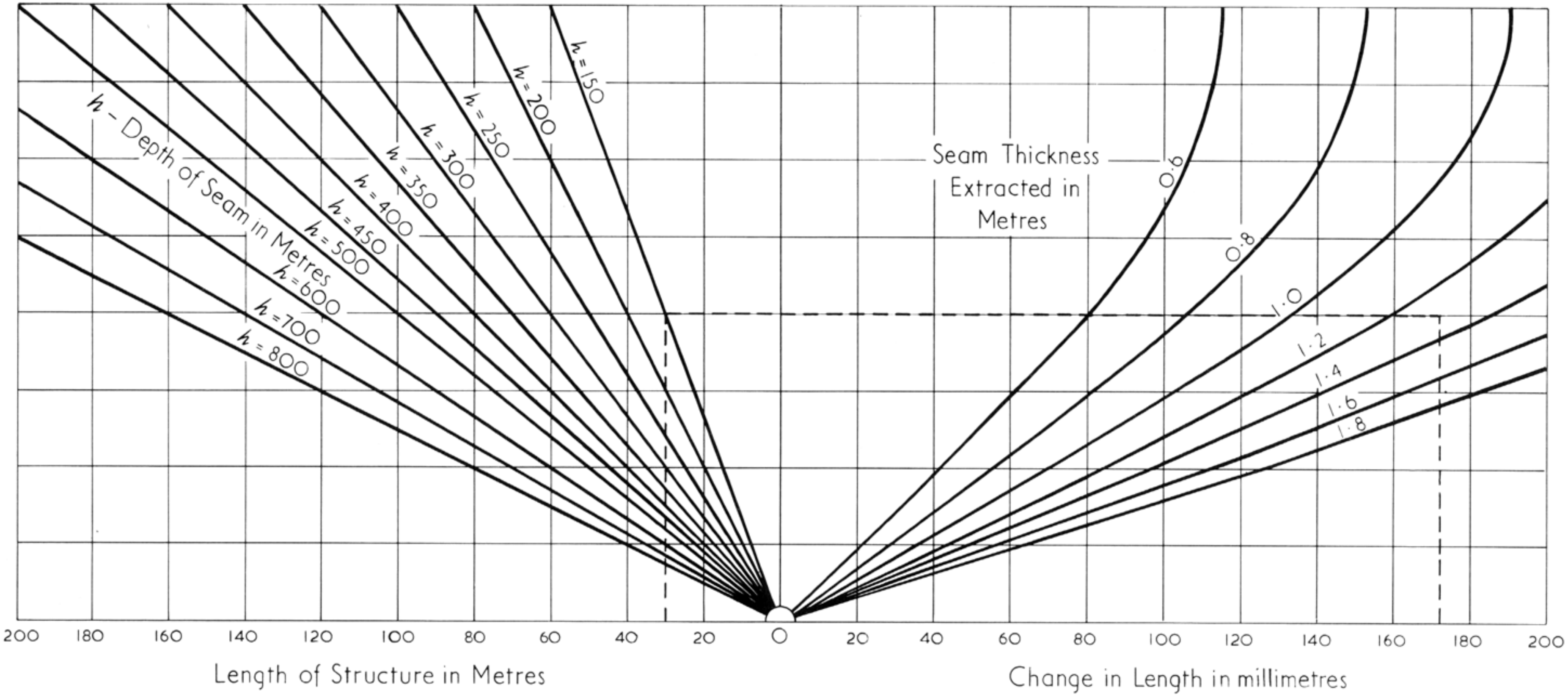
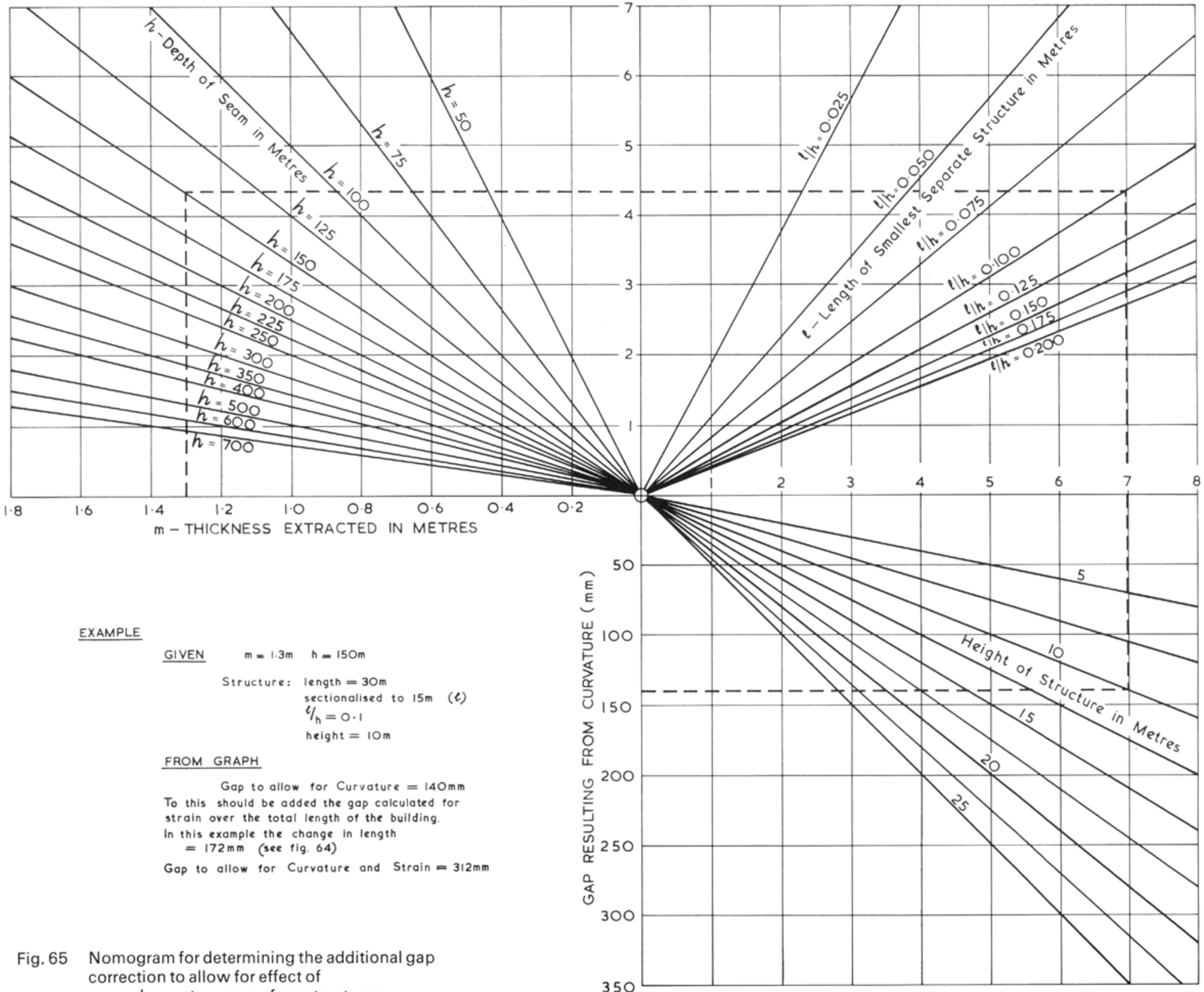


Fig. 64 Nomogram for the determination of change of length in a surface structure, for various depths and thicknesses of extraction.



EXAMPLE

GIVEN   m = 1·3m   h = 150m

Structure: length = 30m
sectionalised to 15m (ℓ)
ℓ/h = 0·1
height = 10m

FROM GRAPH

Gap to allow for Curvature = 140mm
To this should be added the gap calculated for strain over the total length of the building.
In this example the change in length = 172mm (see fig. 64)
Gap to allow for Curvature and Strain = 312mm

Fig. 65 Nomogram for determining the additional gap correction to allow for effect of ground curvature on surface structures.

Industrialised Building

While the main precaution against subsidence damage so far as domestic scale building is concerned is the restriction in size, there are obvious safeguards such as those outlined above with regard to traditional building – e.g. the use of soft black mortar and slipping bituminous damp-proof courses.

There are today over a hundred types of industrialised building systems (prefabricated buildings) for dwelling houses alone, including both rigid and flexible methods of construction. But very few are either sufficiently rigid or sufficiently flexible to be recommended for mining areas. It is not possible to list the acceptable systems at present because of the large numbers involved and the difficulty of examining and reporting on them. It is thus advisable for each system proposed for a mining area to be studied when the proposals are made before giving the Board's agreement.

In general, many of the wooden-framed houses are flexible – or could be so with some attention to the method of applying cladding – and some of the steel-framed houses are rigid, especially where the cladding itself is rigid. Those types which consist of precast concrete slabs are not likely to withstand any movement, without suffering damage, and certain prefabricated wooden components are fitted to such close tolerances that quite small ground movement would cause distortion.

The systems are, in general, devised without considering mining conditions, the main consideration in design being economy and speed of erection. For mining areas a very careful selection of available types should be made.

While it is beyond the scope of this handbook to catalogue all known structures specially designed for subsidence areas, one or two examples may serve to illustrate the principles already outlined and act as a guide to architects and structural civil engineers.

Flexible Structures

The systems incorporated in the buildings erected by C.L.A.S.P.* were among the first to be based on a proper study of ground movement and the advantages of bulk purchase and economical and rapid erection have led to the building of many schools, offices, etc. in subsidence and non-subsidence areas, both in this country and on the Continent.

The foundation is of the type already described (i.e. a thin concrete raft on a friable base) and the superstructure is a lightweight steel frame, pin-jointed for flexibility and with spring-loaded diagonal braces against wind pressure. (Rigid braces are provided in non-subsidence areas.) All cladding and internal finishes are of such a type (concrete, tile or enamelled steel sheet, etc.) and are so hung that movement can take place without distortion. Full attention is paid to such details as stairs (which are hinged), windows and services. Fig. 66 shows the spring bracing on the steel frame and Figs. 67 to 70 show examples of finished

*Consortium of Local Authorities Special Programme.

Fig. 66 Spring loaded braces in CLASP frame.

Fig. 68 Flexible building constructed for the fire and ambulance station, Sutton-in-Ashfield. (Courtesy—Brockhouse Steel Structures Ltd.).

Fig. 67 Tuxford technical school (CLASP).

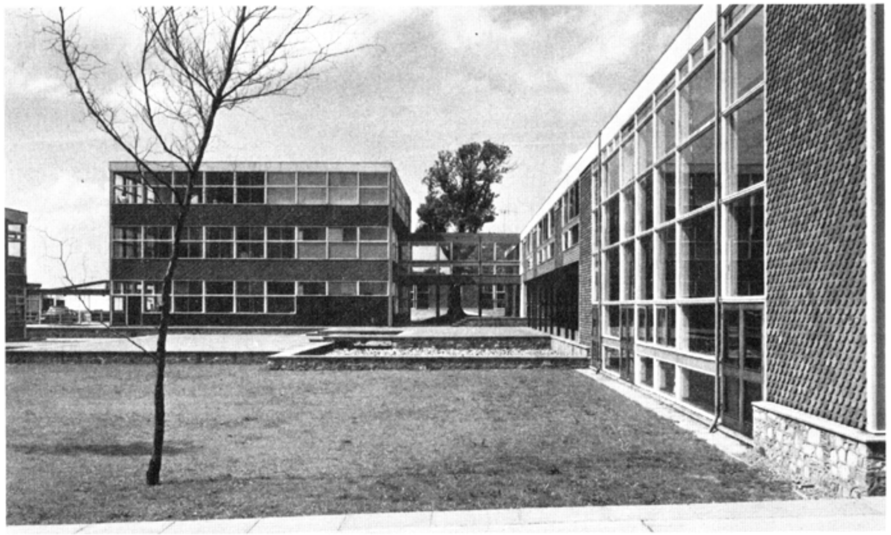


Fig. 69 Typical CLASP building of flexible construction (School at Kirkby, Nottinghamshire—Architect Henry Swain).

C.L.A.S.P. structures. The manufacturers of the steelwork (Brockhouse) now make it available to architects who are not members of the Consortium.

Semi-detached houses are small enough to escape serious damage except in areas of severe subsidence, but some flexibility can easily be incorporated in those cases where rigidity has not been attempted. For example, materials like fibre board should be used instead of plaster for ceilings and partitions, and damp-proof courses should be of normal bitumastic materials. Walls should not be reinforced.

Rigid Structures

For isolation of the foundations from the subsoil, the same provisions should be made as with flexible structures; but the foundations themselves should be strongly reinforced concrete rafts or beams capable of supporting the superstructure over ground subject to both change of length and curvature. Fig. 71 shows a cellular raft, an example copied from types designed and used in the West Riding County Council's buildings in subsidence areas. The superstructure should also be as rigid as possible, damp-proof courses, for example, being of Staffordshire blue bricks which will obviate sliding.

The principle of keeping buildings to independent units of small plan area is most important and allows higher construction, even – with suitable rigidity – to the extent of 15 or 16-storey blocks of flats. It is desirable that such tower blocks be capable of being jacked to new levels should permanent tilting occur and that they should be built only over areas of deep mining, where the maximum gradients which mining can produce are small. In one such mining area a considerable number of 13-storey blocks have been built and higher blocks are being built elsewhere (see Fig. 72). These tower blocks have plan dimensions of about 25 m × 16 m and are supported on raft foundations about 2 m thick, suitably reinforced and imposing a maximum increased load of 22 tonnes per square metre. The excavations below the raft are carefully inspected at formation level, and if the stratum is weak it is excavated and back filled with a 1:10 mix of mass concrete. The mass concrete is brought to formation level by a 50-mm thick protective layer of high quality concrete, which in turn is covered by a layer of polythene sheeting. A 150-mm thick layer of compacted sand is laid on top of the polythene and a further layer of polythene sheeting is laid on the sand; then another layer of blinding concrete 100-mm thick is laid, upon which the raft is constructed. The soffit of the raft is horizontal with no projections. The working space immediately around the raft is filled with ash or some other suitable crushable material to minimise the effect of horizontal stresses upon the structure. The finished surface immediately around the block is not paved with flag-stones but is finished as a strip of garden or paved with macadam. The lift walls are constructed with an additional 75-mm clearance on all sides over the normal requirement of the lift engineers. Main piped services have flexible joints where they enter the building and observation sockets are built-in just above ground

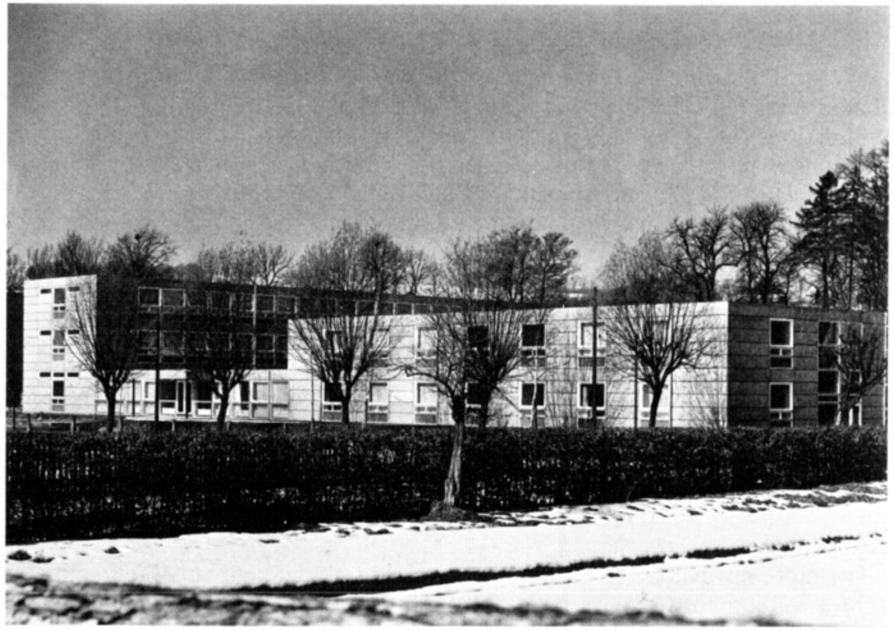


Fig. 70 CLASP school at Büren, Germany.

level so that levels and measurements may be taken to record subsidence, tilting, and horizontal movement.

There is little point in trying to site buildings with any particular orientation with regard to the mine workings, unless the workings are to be made in the very near future and their planning is definite. There is not enough difference between the maximum slope in a transverse profile and that in a longitudinal or travelling profile materially to affect the design of a structure. Further it is not always possible for the siting of structures on a particular part of an expected profile to be successful as the orientation of coal faces is subject to variation through geological and other influences. The design should allow for the maximum *normal* movements predictable for the seams which remain to be worked.

Dams and Reservoirs

Although there is little or no scope for the incorporation of precautions in the construction of a dam to counter the effects of subsidence, it would be reasonable to suggest that in order that a support pillar may be limited in size to protect the dam itself, other precautions should be taken. Take-off pipes and mains should be made flexible by the use of telescopic joints (see Section 9) and the freeboard of reservoirs and surrounding drainage channels should leave room for some subsidence.

In the case of tanks constructed in mining districts, there are alternative methods of allowing for movement, including the separation of large tanks into smaller units with flexible connections. An example is given in Fig. 73 of a 2¼ million litres rigid tank built on three concrete pads and capable of being jacked level if necessary. The tank was designed for erection in South Wales by Thomas and Morgan and Partners, Consultant Engineers, Pontypridd.

The essential design principles are clearly illustrated in this example and include the following:–

1. Foundations are as small and stiff as possible to reduce and cope with bending, extension and compression of the ground during subsidence.
2. Ground pressures are kept as high as possible to enable pressure redistribution to take place under the tank.
3. The structure sits on a sliding layer and on granular material to reduce friction.
4. The conical shape is more rigid than a cylindrical shape.
5. Three jacking pockets are provided.

Twelve of these tanks are planned to be constructed in the Rhondda Valley and will be subjected to total subsidence of up to 7 m (over 20 years) the shallowest seam being 353 m deep.

The more normal type of service reservoir should be kept small, i.e. units of from 1 million litres to 2¼ million litres capacity, with flexible jointed pipe connections between units. Construction is generally in reinforced concrete, being kept watertight by copper or rubber seals.

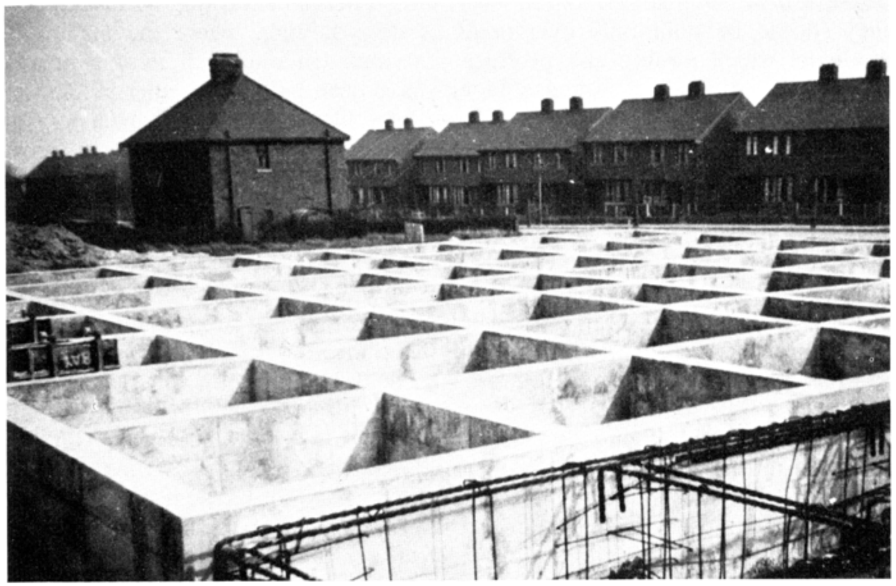


Fig. 71 Cellular raft for rigid structure.

Fig. 72 High-rise flats of rigid construction.

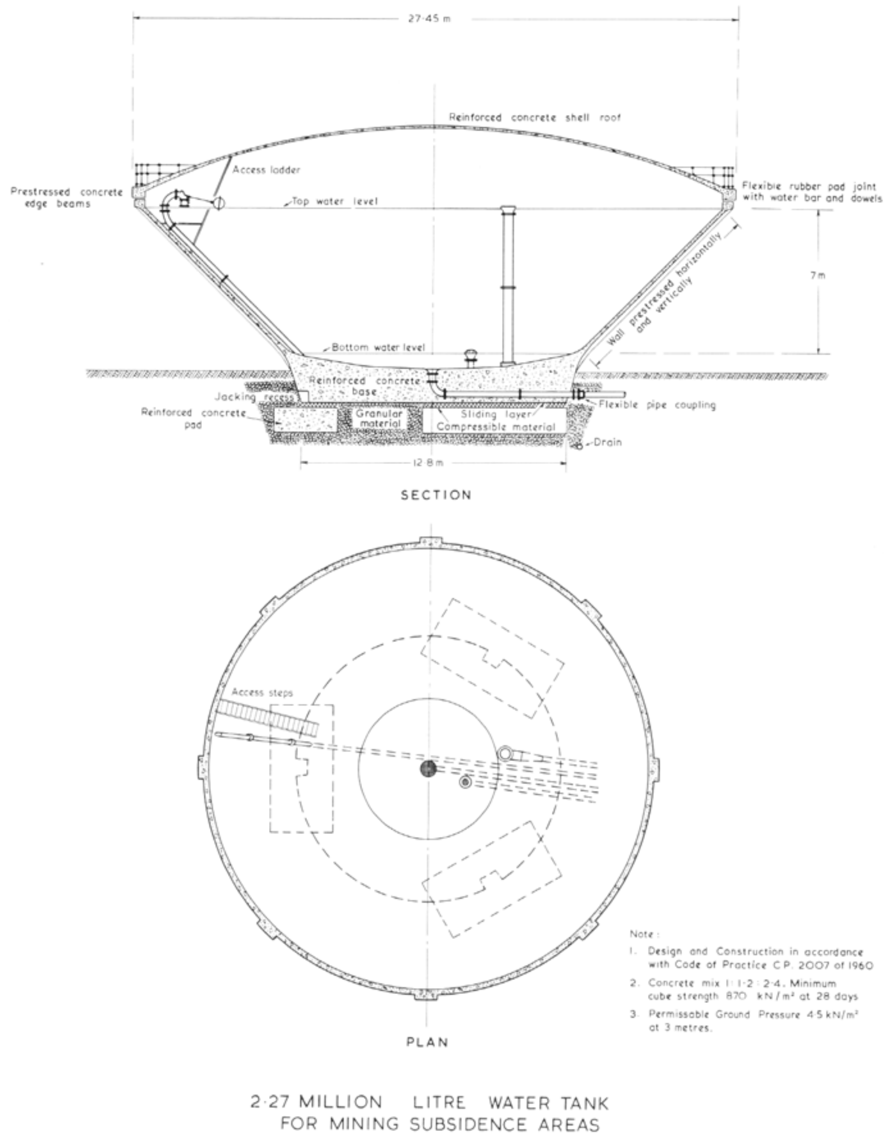


Fig. 73 $2\frac{1}{4}$ million litre water tank for mining subsidence areas.

On a larger scale two reservoirs of 9 million litres each have been built in Nottinghamshire to a design calculated to withstand subsidence strains in the order of 4 mm/m in ground which is already naturally fissured. A paper by Lackington and Robinson, (406), describes both the principles concerned and the details of construction. Fig. 74 shows the details of a typical joint in the floor of one of the reservoirs which are 48.8 m square with floor slabs varying in size from 5.7 m to 6.1 m square. It is important to note that such reservoirs need cost no more than traditionally built rigid service reservoirs.

Sewers and Sewage Disposal Works

Information on the laying of pipelines and provision of flexible connections is contained in Section 9. The choice of material for sewer pipes is not a matter for the subsidence engineer, but attention is drawn in Section 9 to the need for avoiding the bedding in concrete of pipes which might not be strong enough to stand the loads imposed, unless the concrete is discontinued at the joints in order to allow them to move with the ground.

Manholes should be of reinforced concrete, rather than brickwork, and have flexible connections with the pipes.

Sewage disposal works should not be designed as large monolithic structures protected entirely by support pillars since they will be left higher than the subsiding countryside, necessitating pumping. To allow for adequate fall from the effluent outfall to the receiving stream, it might be advisable in some cases to allow extra fall in the design.

A system in which sedimentation tanks are cleared by mechanical scraping or other facilities is preferable to one in which separation is effected by channels which depend upon gradient. Weirs should be adjustable, and small adjustable circular filters should be used rather than large rectangular ones.

Bridges

A bridge in a mining area should either be rigidly constructed to resist ground movement or so articulated that it can accommodate movement without damage. If the direction of workings can be so orientated as to move in line with the bridge, so that only two-dimensional movements have to be considered, precautions could be quite simple as in the following examples:–

Short Span Bridges (*up to* 30 *m*) should be of a simply supported type. To allow for longitudinal and vertical movement the deck is fixed at one end and free at the other where an expansion joint is used. This allows the length of the bridge to vary and limits the failure of the road surface to one place (see Fig. 75). Jacking pockets may be provided to allow subsequent relevelling.

Multispan Bridges. If there is a series of short spans, as above, piers have to be hinged top and bottom to allow for tilting and change of length. Tilting or rocker bearings are required at each pier. Jacking pockets may be provided.

Fig. 74 Flexible joint in an articulated service reservoir for a mining area (after Lackington and Robinson).

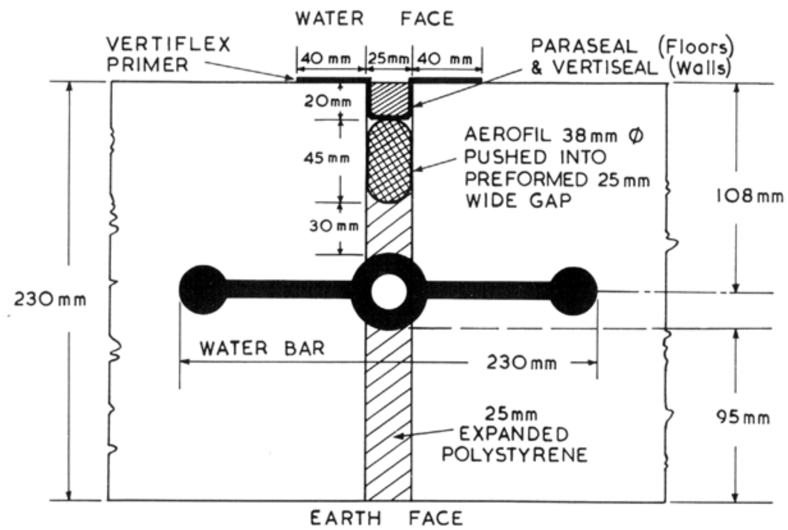
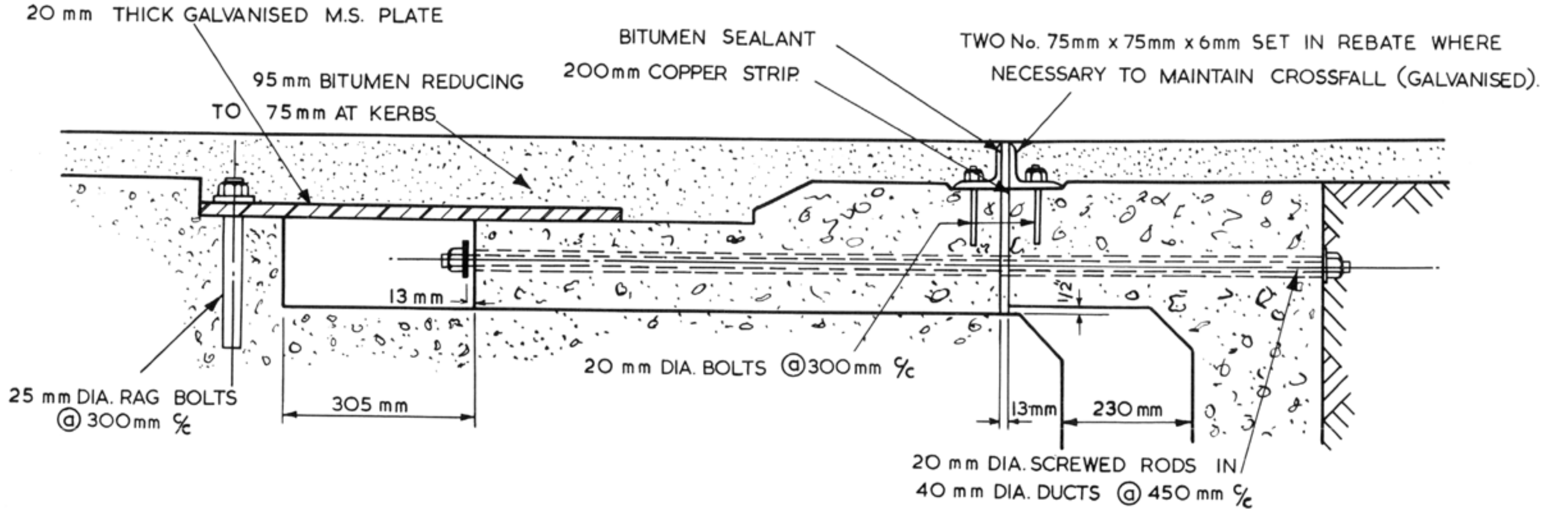


Fig. 75 Combined expansion joint—subsidence trench for bridge deck.



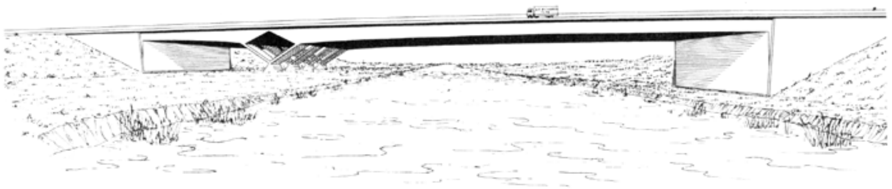


Fig. 76 General view of hinged bridge structure.

Long Span Bridges are, for economic reasons, generally not of the simply supported type. Examples of suitable types are cantilever and suspended span construction, the Wichert Truss and the three pinned arch.

Usually, however, the direction of approach of workings to a bridge cannot always be anticipated and three-dimensional movement must generally be allowed for. This can be achieved by supporting the bridge on three points only, the movements being as in the two-dimensional movement mentioned above with the addition of the transverse tilting and strains which will produce torsion in the deck. (Figs. 76 and 77.)

EXISTING STRUCTURES

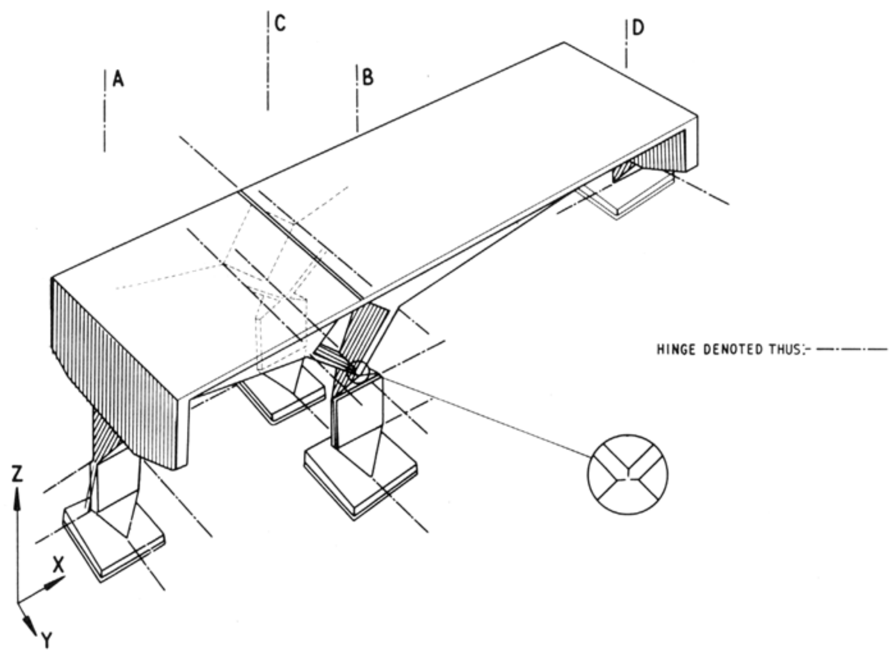


Fig. 77 Diagram of hinged bridge structure.

Modifications may be made to existing structures before ground movement occurs if it is believed that the amount of damage avoided will justify the expense or if there are other circumstances which make it imperative to minimise the damage without weighing the cost too closely. Such circumstances may, for example, include: schools and hospitals, where alternative accommodation may not be available if repairs become necessary of such a degree that the building has to be evacuated; or churches, houses, etc. which are scheduled as ancient monuments; or buildings of historic or architectural importance. In all cases where the fabric is of such a construction or age that even moderate movement might involve danger, it may be necessary to provide temporary supports and/or strengthening before mining takes place. (This can usually be taken to mean before the first approaching coal face is within about half-depth range of the structure.)

Examples of forms designed to preserve the shape of arches when the building undergoes tensile strain are shown in Figs. 78 and 79. Several other types of support have been used, but these particular examples illustrate the neatness with which the precautions can be carried out.

The use of the traditional tie rod is only justified if the predicted extension of the ground is sufficiently severe to make it likely that roof trusses will be pulled off their bearing when the supporting walls move apart. Injudicious use of tie rods can lead to needless disfigurement of buildings by the rods and star plates themselves and by concentrating damage at the points where the plates tend to pull through the walls. In some cases extra bearing has been given to roof trusses by means of temporary corbels.

The effects of compression may be reduced by removing part of the rigid paving, or even the superstructure, to provide room for compression. This can be done by cutting out slots in floors and paved areas joining adjacent buildings, or at intervals over large areas of flooring to provide adequate compression space. Concrete floors are difficult subjects, but wood block floors and brick and

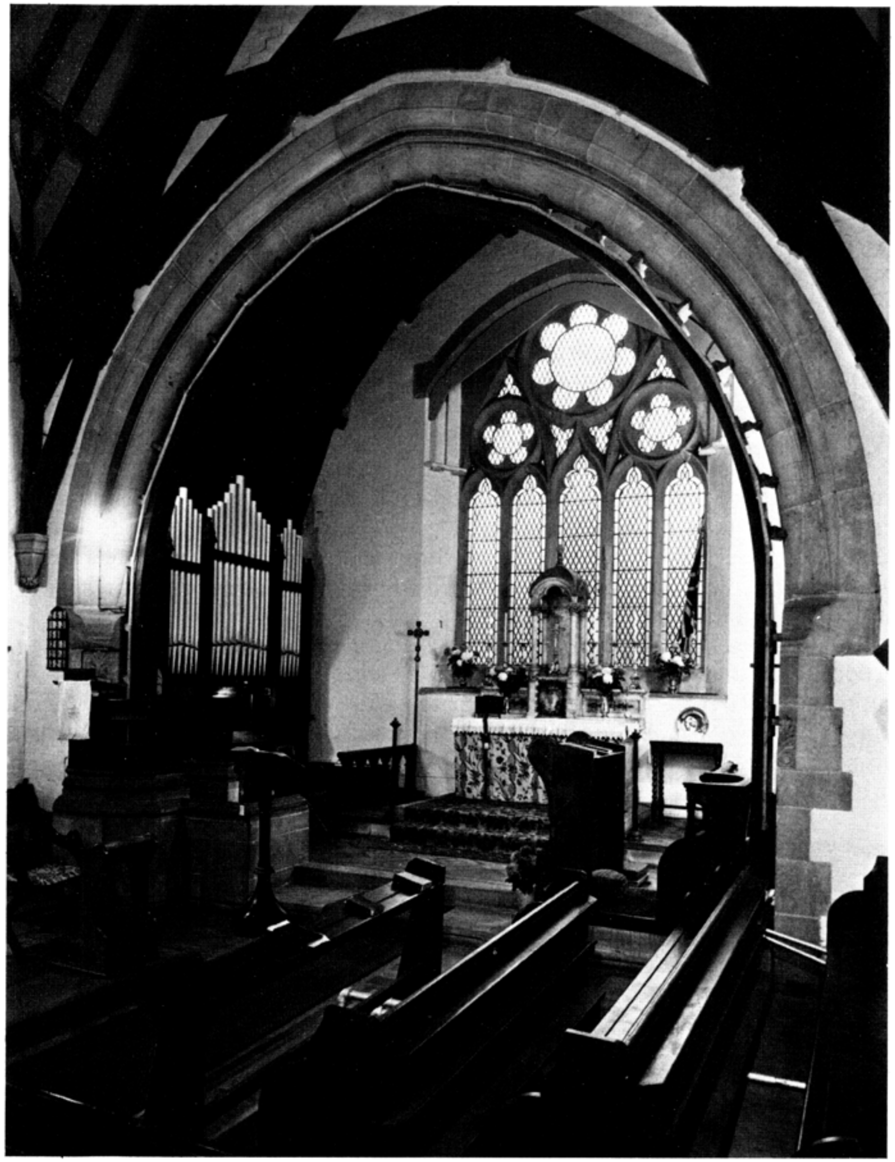


Fig. 78 Unobtrusive support for chancel arch of a church.

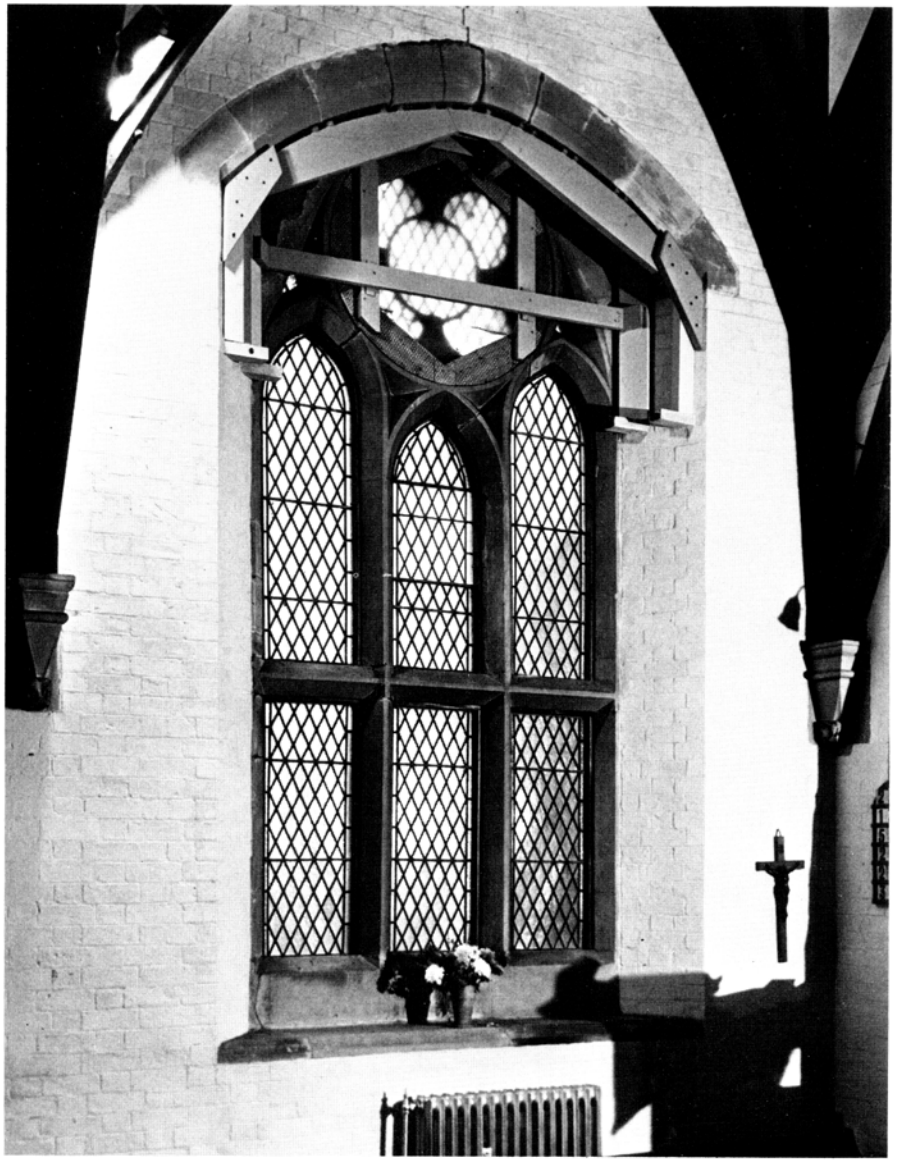


Fig. 79 Church window arch support.

stone paving are easily dealt with. In some cases it has been thought a reasonable precaution to remove and store stained glass windows; partitions, screens and ornaments can generally be freed from walls and floor and independently supported. In extreme cases part of a house in a long terrace has been cut out, or even a complete house removed, to allow for compression space.

Service pipelines can be modified by inserting telescopic and flexible joints and the passage of pipes through walls can be made free by inserting a slippery or plastic filling or cover fillet when making good the hole cut out. Where there is no flexibility between a service pipe and a structure – e.g. a water tank or gas holder – a great deal of damage can be avoided by providing a flexible connection.

Trenching

The excavation of trenches around buildings has been successful in reducing the damage due to horizontal strains, particularly compression. Trenches taken out as near to the structure as possible and to a depth just below the underside of the foundations, absorb strain by permitting the ground to thrust into the gap.

The space has, in fact, to be filled with material which is strong enough to support the sides of the trench, but softer than the surrounding sub-soil so that crushing can take place. The material generally used today is graded boiler clinker not less than 25 mm size. It is a common procedure to excavate a trench around a building using a machine which can both remove the soil and shovel the compressible fill into the trench.

The trenches may only need to be dug alongside certain walls of the building, according to the direction of expected severe strain, and may be finished off by capping with paving slabs for use as a footpath or by cultivating as a flower border, or lawn. Fig. 80 and 81 illustrate a trench before and during filling. The cost is about £5 per metre run for a trench at an average depth of about 1.2 m in average ground conditions.

Fig. 80 Mechanically excavated trench to absorb ground movement.

Fig. 81 Trench being filled with boiler clinker.

Section 9: Structural Precautions – Pipelines and Joints

In this section attention is drawn to the need for flexibility in pipelines and to the way in which special pipe joints provide both angular and telescopic movement. Details of available joints for typical pipes of various sizes are included, with notes of the amounts of movement which they provide.

Pipe Movement

Pipelines buried in the ground are assumed to move as and when the ground moves owing to the friction between the pipes (particularly at joints where there are projecting flanges) and the ground. Thus when ground undergoes curvature and horizontal change of length, a pipeline may facture unless it is designed to deflect and telescope at its joints to suit the new vertical alignment and change of length. The horizontal strain is assumed to be distributed evenly over a number of joints if the anchorage through friction is complete and if the ground itself stretches or compresses uniformly – which is more or less the case unless either a natural or a man-made break has at some time been made.

Anchorage

If a pipeline is laid above ground the strain may be uniformly imposed at the support points but the movement may not be equally distributed over the joints. This is because some of the joints, being less tight than others, will be pulled out too far, causing the pipeline to fail. Thus, in pipelines laid above the ground (or in tunnels) an anchor point should be provided at every joint to ensure that the ground strain is evenly distributed along the various joints.

An alternative method is to provide an adequate harness restricting the movement of the joint to its maximum travel, short of pulling out. Fig. 82 shows such a harness and Fig. 83 indicates the same manufacturer's recommendation regarding the support and anchorage of a pipe fitted with the harness. A large-diameter pipe which may frequently be full of water may have to be provided with additional guide supports because of the shear force which would operate at the joints. Fig. 84 shows a large diameter buried pipeline (the Derwent water pipeline) being fitted with a flexible coupling.

When pipes are laid in trenches it is difficult to make the bottom of the trench a perfectly straight line, and thus it is not always possible to achieve a uniform contact with the underside of the pipeline. It is possible, in these circumstances, for only some of the pipes to be firmly bedded and for these to support part of the whole of the length of adjacent pipes. It is important that the pipes and joints possess sufficient shear strength to provide this support.

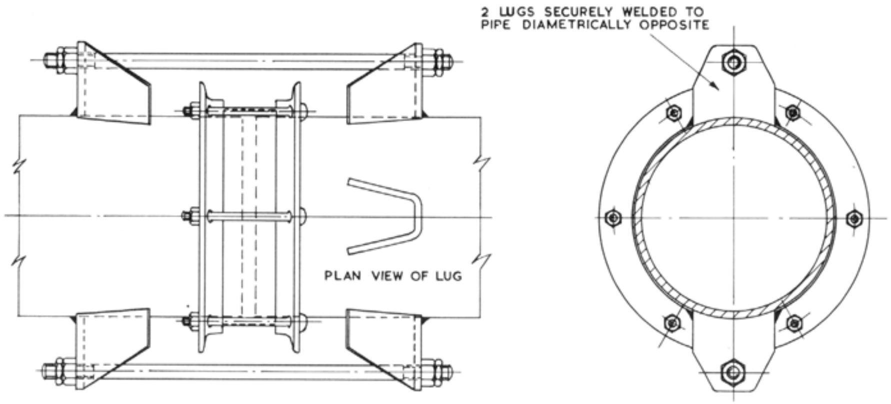
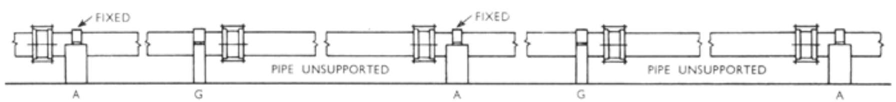


Fig. 82 Viking Johnson coupling harness.



A. ANCHOR SUPPORT. Sufficiently strong to prevent creeping of the pipe. The anchor points at the ends of straight runs must be strong enough to prevent end separation at the maximum internal pressure.

G. GUIDE SUPPORT. Permitting longitudinal movement.

Note 1. Alternate pipes are unsupported and free to take lateral movement.

Note 2. Coupling sleeves can be supplied with centre register, without centre register or with a removable centre register in the form of a screwed plug.

Fig. 83 Method of supporting and anchoring pipes fitted with Viking Johnson couplings.

Fig. 84 Insertion of flexible coupling into large-diameter pipeline.

Pipe Laying

It is obviously pointless to lay pipes with flexible joints and to bed them continuously in concrete. With ground movement the concrete will break and will do so at random points, which may cause strain to be concentrated at individual joints instead of being distributed evenly over the strain zone. At the same time it is important that an even bedding should be provided in order to avoid excessive shear stress.

A suitable method (for loadings up to 1.9 × the three-edge crushing strength of the pipe) is to use granular material which will distribute the loading of the pipe, its contents and the backfill, evenly, and at the same time avoid concentrations of the horizontal movements due to mining. This method is recommended in National Building Studies Special Report No. 32 (408).

Fig. 85 illustrates the approved method of supporting a pipeline and backfilling the trench which is equally suitable for subsidence and non-subsidence areas. For greater loadings it is better to use concrete rather than gravel fill, but in mining areas to allow flexibility at the joints by means of discontinuity in the concrete.

Telescoping

In all pipe ranges in subsiding ground the total predicted strain must be provided for by an adequate number of joints at which the total possible movement is sufficient to equal the change in ground length. It is essential that the spigots should be marked, or a gauge used, so that they are not pushed right home when laid – an action which would remove any capacity to withstand compression. It is safe to assume that under extension a sleeve joint will draw out on one side only, since the spigots on both sides will not have exactly the same friction. Thus the capacity of a sleeve joint to take up extension would be only half the capacity for compression because once one spigot was forced home the other would start to move. (This does not apply to a harnessed sleeve joint, which has the capability of full travel in either direction). If, on setting, both spigots were pushed in for two-thirds of the length of the halve-sleeve the amount of extension which the joint could take up would equal the amount of compression. Spigot and socket joints should, of course, be laid with the spigots half-way in. The characteristics of available pipe joints should be assessed on these assumptions.

Example

A pipeline may be provided with sleeve joints as in the examples in Figs. 86 and 87 giving a freedom of extension of, say, 20 mm per joint. Assuming that the ground in which it is proposed to lay this pipeline is expected to suffer tensile strain of the order of 4 mm/m maximum over a length of, say, 30 m, then five 6 m lengths would suffice, which would have joints totalling 100 mm of safe movement. However, a 30 m long pipeline extended by 4 mm/m is longer by 120 mm, so that it would be necessary to specify the next smaller size, say 5 m lengths, of which six tubes would be needed giving six joints with a total freedom of movement of 120 mm.

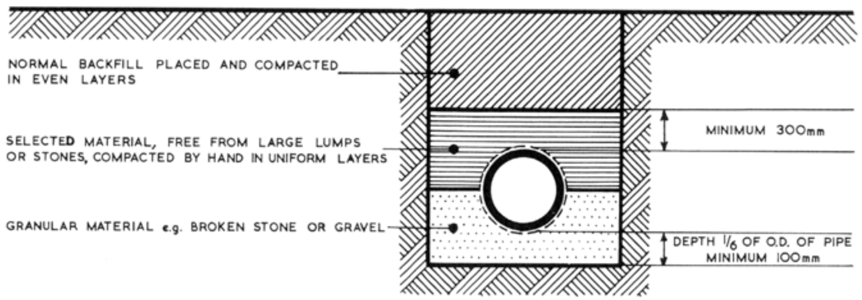


Fig. 85 A recommended method of laying pipes in trench. (Courtesy Concrete Pipe Association.)

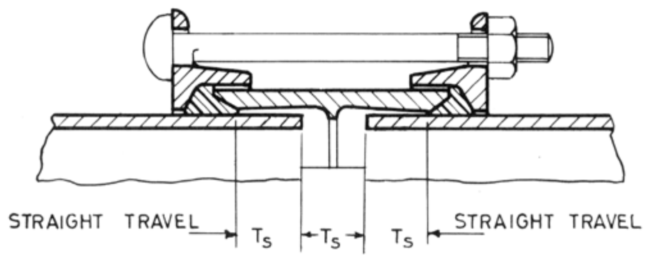


Fig. 86 Typical sleeve joint (iron pipes).

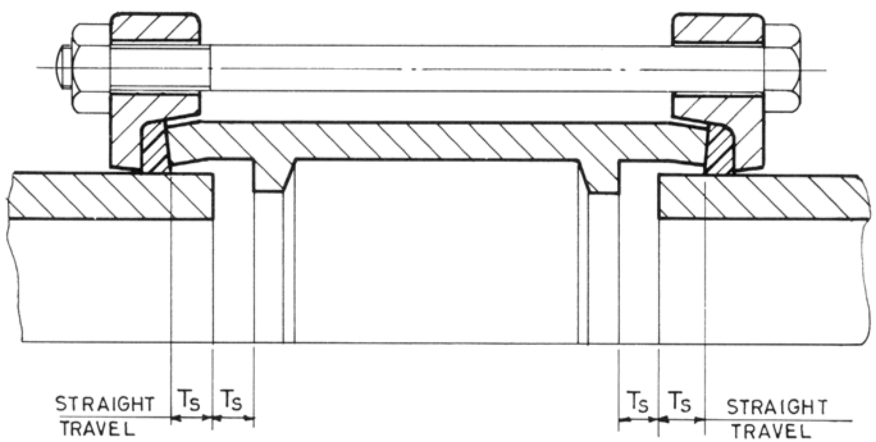


Fig. 87 Sleeve joint for asbestos cement pipes.

It must be remembered that thermal expansion has also to be absorbed by these telescopic joints; thus the amount of flexibility available to cope with mining strain is correspondingly reduced.

Figs. 88 and 89 show two types of flexible joint available for vitrified clay pipes. Fig. 90 shows a clamp joint for polythene pipes. This joint has no telescopic movement but the pipes themselves can bend freely. Suitable telescopic joints are also available for pitch fibre pipes and for unplasticised pvc pipes.

Performance Requirements

Pipe joints should have minimum performances which comply with a draft Part 2 of British Standards 65 and 540* which details the requirements of joints "having regard to the provision of sufficient flexibility, telescopic travel, durability and corrosion resistance, together with the ability to maintain water tightness and resistance to possible root penetration under various forms of misalignment". There are three criteria: deflection, straight draw and line displacement:–

Deflection

Deflection is defined as the distance from the extended centre line of one pipe to the centre line of the other pipe at its free end.

When tested at a constant pressure of 6 m head of water the joints shall allow the deflections shown in Table 9.

TABLE 9

Nominal Bore	Deflection per metre of pipe length
mm	mm
75 – 150 inclusive	45
225 – 300 inclusive	25
375 – 450 inclusive	15

Straight Draw

When tested at a constant pressure of 6 m head of water the joints shall allow for a total straight draw of 10 mm. In practice this should be considered as $\pm$ 5 mm since the movement might be either tensile or compressive and the spigots will, when properly set, be half-way in.

Line Displacement

Joints which allow line displacement when tested at a constant pressure of 6 m head of water shall allow for line displacement of 6 mm.

The minimum straight draw allowed by the flexible joints described above is quite sufficient for changes of length associated with normal mining subsidence today.

*Specifications for clay, drain and sewer pipes including surface water pipes and fittings.

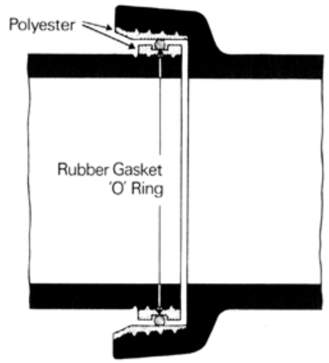


Fig. 88 Flexible joint for vitrified clay pipes.

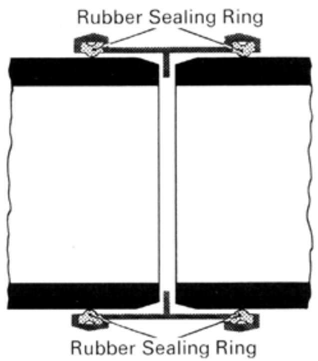


Fig. 89 Plastics joint for clay pipes.

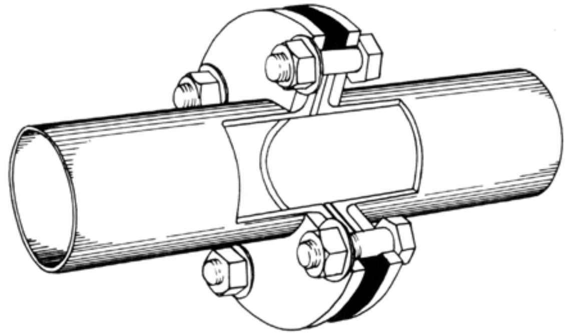


Fig. 90 Clamp joint for polythene pipes.

The draw of $\pm$ 5 mm referred to amounts, in a pipe 1 m long, to 5 mm/m and that is equivalent to a fairly severe subsidence strain.

In the case of a pipeline where the calculated strain exceeds 5 mm/m a shorter length of tube should be used so that the joints are closer together.

Ordinary differential settlement, due for example to made ground compacting relevant to an adjacent building, may cause some local telescoping. This, together with the easier joint making, makes a case for using flexible joints even in non-mining areas.

Effects of Deflection upon Straight Draw

Angular deflection due to curvature of a pipeline causes some loss of straight draw because the tilting of one pipe-end relative to its neighbour takes up some of the telescoping allowance as illustrated in Fig. 91. Curvature of the ground due to mining subsidence is usually so slight as to cause negligible deflection between adjacent pipes, but a pipeline can be laid to a horizontal curve and differential ground settlement may induce locally a vertical curve which reduces the straight draw. For example, a pipe of nominal bore 300 mm will have a straight draw of 5 mm/m to comply with the draft British Standards. The same specifications permit an angular deflection of 25 mm/m and this deflection will take up 3.7 mm out of the 5 mm straight draw also specified.

Example

To take another example, if a local settlement of 35 mm occurred at one end of a 1 m length of pipe, the angle of deflection would be 2°, that is 0.035 radians. Supposing the pipe to have an external radius of 65 mm, the loss of straight travel will amount to $65 \times 0.035 = 2.275$ mm. This leaves only 2.7 mm straight travel on each joint so that an extra joint would be required to restore the 5 mm/m capacity to the pipeline. This could be done by using two shorter lengths.

Types of Pipe Failure

The various types and causes of fractures in pipes have been identified by N. W. B. Clarke (402), and those which could also be caused by mining subsidence are shown in Fig. 92 and listed in Table 10.

Shear fractures and beam fractures are mostly caused by incorrect laying but the other types of fracture can be avoided by providing enough flexible joints.

It is important in pipe laying that a travel gauge should be used or the spigot marked so that it is not pushed more than half-way into the socket.

Welded steel pipelines

Welded pipes are laid when a smooth internal surface is required, such as when a joint groove could cause a build-up of sediment or when high pressure is applied as in natural gas pipelines.

The welded joints are rigorously tested and the steel pipelines, even those of quite large diameter, have considerable flexibility. The ability of a buried steel

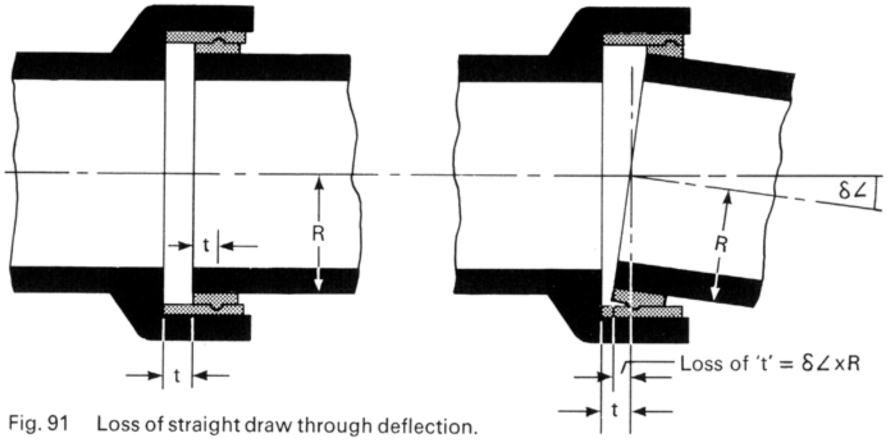


Fig. 91 Loss of straight draw through deflection.

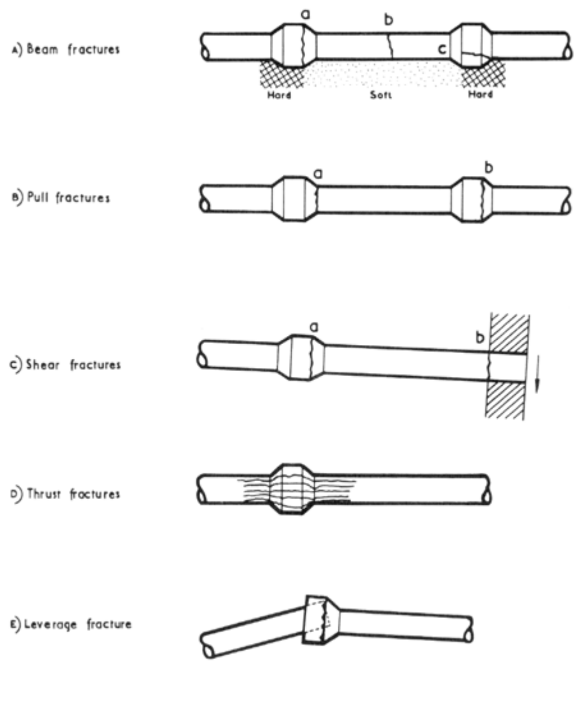


Fig. 92 Possible types of pipe failure.

pipeline to withstand horizontal change of length, particularly compression, depends upon both the number of bends or changes of direction of the line and on a less than complete friction between the pipeline and the backfill.

If a welded joint fails in an area subject to subsidence it should be examined for quality of weld and any indication of the type of strain, which should then be compared with the known ground behaviour due to mining.

TABLE 10 **Types and Causes of Pipe Fracture**

FIG. 92	Type of failure	Cause	Prevention
A	Beam Fractures	Uneven resistance of foundations or soil movement, or differential settlement. Caused by mining in extreme cases of shallow workings.	Flexible joints and uniform hardness of foundation.
B	Pull Fractures	Thermal or drying shrinkage of pipe, or site concrete, drying shrinkage of clay soil. Extension of ground through mining.	Flexible telescopic joints and gaps in site concrete at pipe joints.
C	Shear Fractures	Differential settlement of wall relative to pipe or vice versa. Can be caused by fissuring in subsided ground.	Flexible joints at a and b and making ab not more than 1 m.
D	Thrust Fractures	Restrained thermal or moisture expansion of pipe or compression due to subsidence.	Flexible telescopic joints. Spigot end not laid hard up in socket.
E	Leverage Fractures	Excessive angular displacements. Extreme cases of differential subsidence.	Flexible joints. Avoidance of excessive slew when laying.

Section 10: Design of Mining Layouts to Minimise Subsidence and Damage

Systematic partial extraction layouts incorporating longwall extraction with substantial pillars left between extracted panels have been successful in many places where subsidence has had to be minimised to avoid high repair costs, difficult drainage problems, or adverse public opinion. It has been a valuable compromise between complete sterilisation and complete surface support. Up to about 70% extraction is possible under built-up or even heavily industrialised areas.

Much has been written on the subject, but basically the principle depends upon the small subsidence and gentle curvature of surface subsidence profiles over panels extracted at a small width/depth ratio (less than h/3 but commonly h/4). With deep seams quite long faces are suitable, although in very deep seams the face length may be limited for reasons of underground environment. For example in a seam 900 m deep faces may be preferred at a length of 180 m which is only one fifth of the depth of cover.

In the case of shallower seams experience had led to high productivity from shorter faces which have travelled more quickly. A series of such panels, separated by pillars of suitable size causes an overall subsidence, due to the addition of the overlapping adjacent subsidence profiles, which is up to 2.2 times the maximum subsidence from a single panel. The law of superposition can be applied because the pillars are far wider than the minimum necessary for stability so that no additional subsidence from pillar failure need be allowed for. The main function of the pillars is to separate the individual panels by the correct distance so that the summation of the overlaps produces a flat profile virtually free from humps and from horizontal strain.

Fig. 93 illustrates the plan of a typical layout and Fig. 94 the section through the seam and the surface showing the total profile. Every such scheme should be plotted by summating the individual panel profiles to check the total effect, using the graph in Fig. 3 to determine the maximum subsidence for one panel. Fig. 5 or Table 1 is used to plot the whole profile over one panel.

Firstly, however, in order to obtain a close indication of a suitable layout the nomogram in Fig. 95, or 96 or 97 may be used. The conversion of the maximum subsidence over one panel to the total for a series of panels (enough to cover a critical area) is facilitated by the nomograms which also permit the effects of different combinations of panel width (w/h) and pillar width (p/h) to be compared.

The first step in designing a partial extraction layout is to decide upon the maximum subsidence and strain to which the structure or surface area can safely be subjected. This requires a knowledge of the area concerned which may have to be supplemented by liaison with the relevant local authority or statutory authority, etc.

It may only be feasible in a drainage area to lower the surface by a few inches without incurring the cost of pumping in perpetuity and this could also apply to

Fig. 93 Typical layout for longwall partial extraction under a residential area.

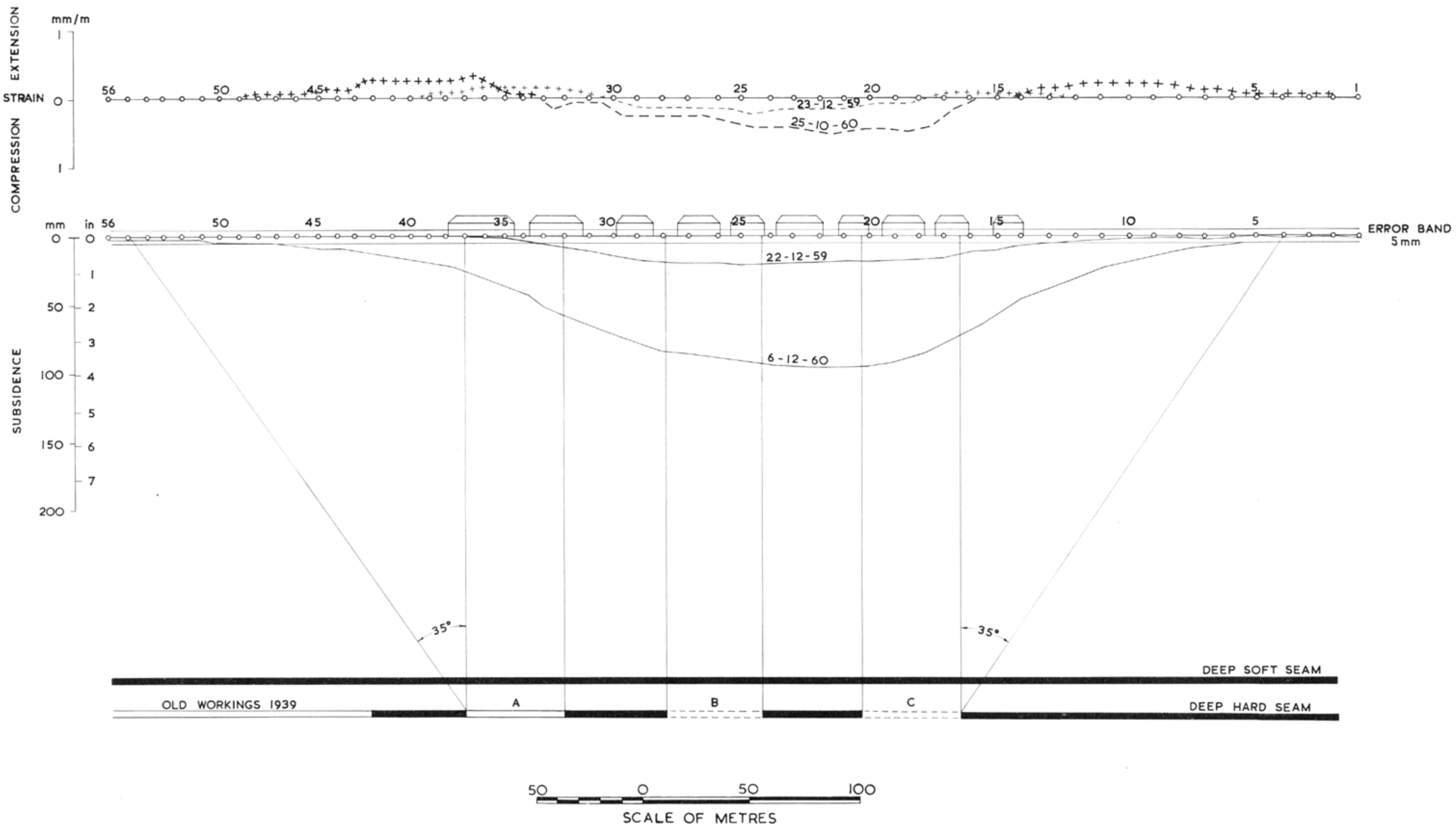


Fig. 94 Section across panels and pillars illustrated in Fig. 93.

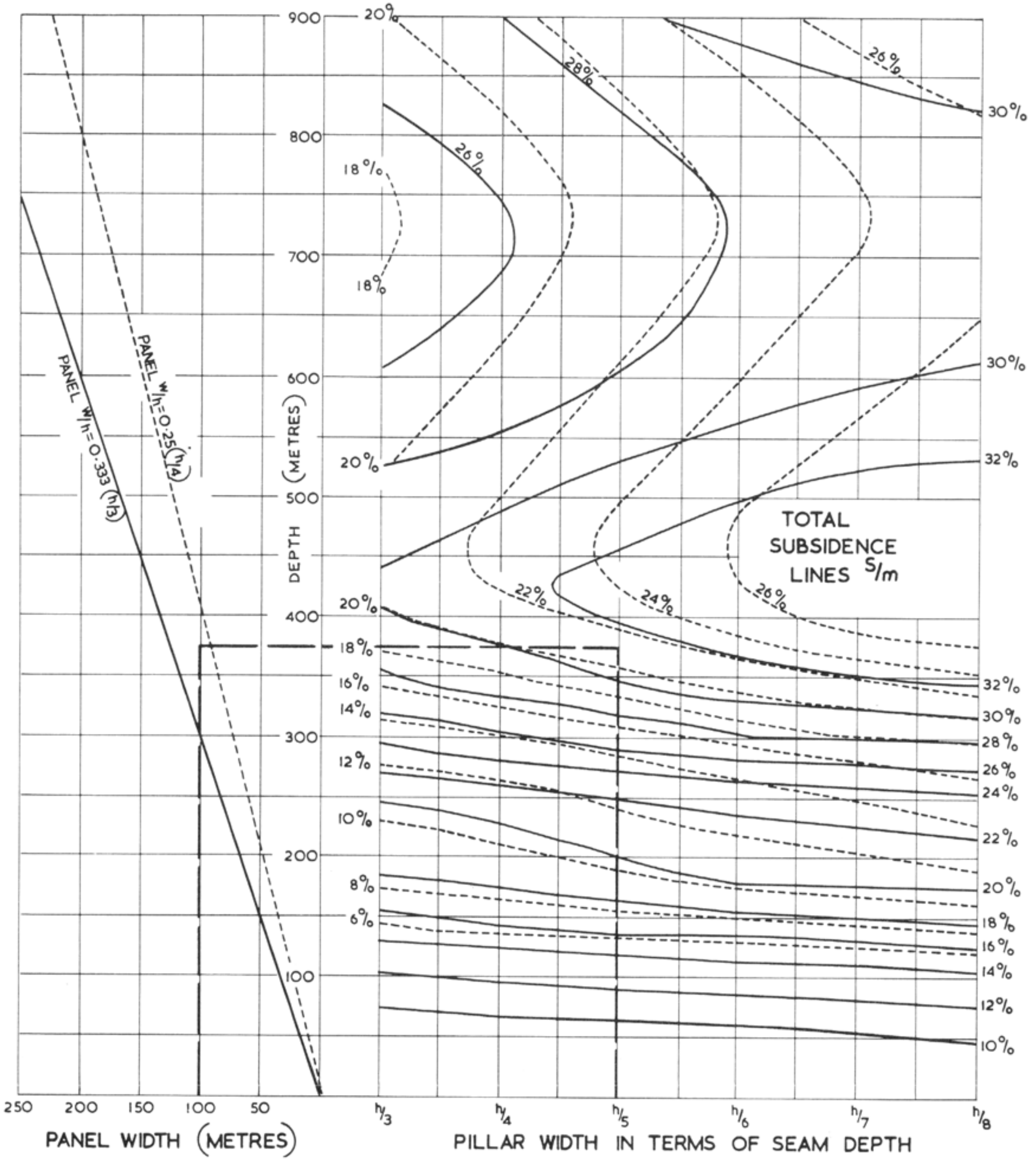


Fig. 95 Partial extraction layout nomogram—for width/depth ratios 0.25 to 0.33.

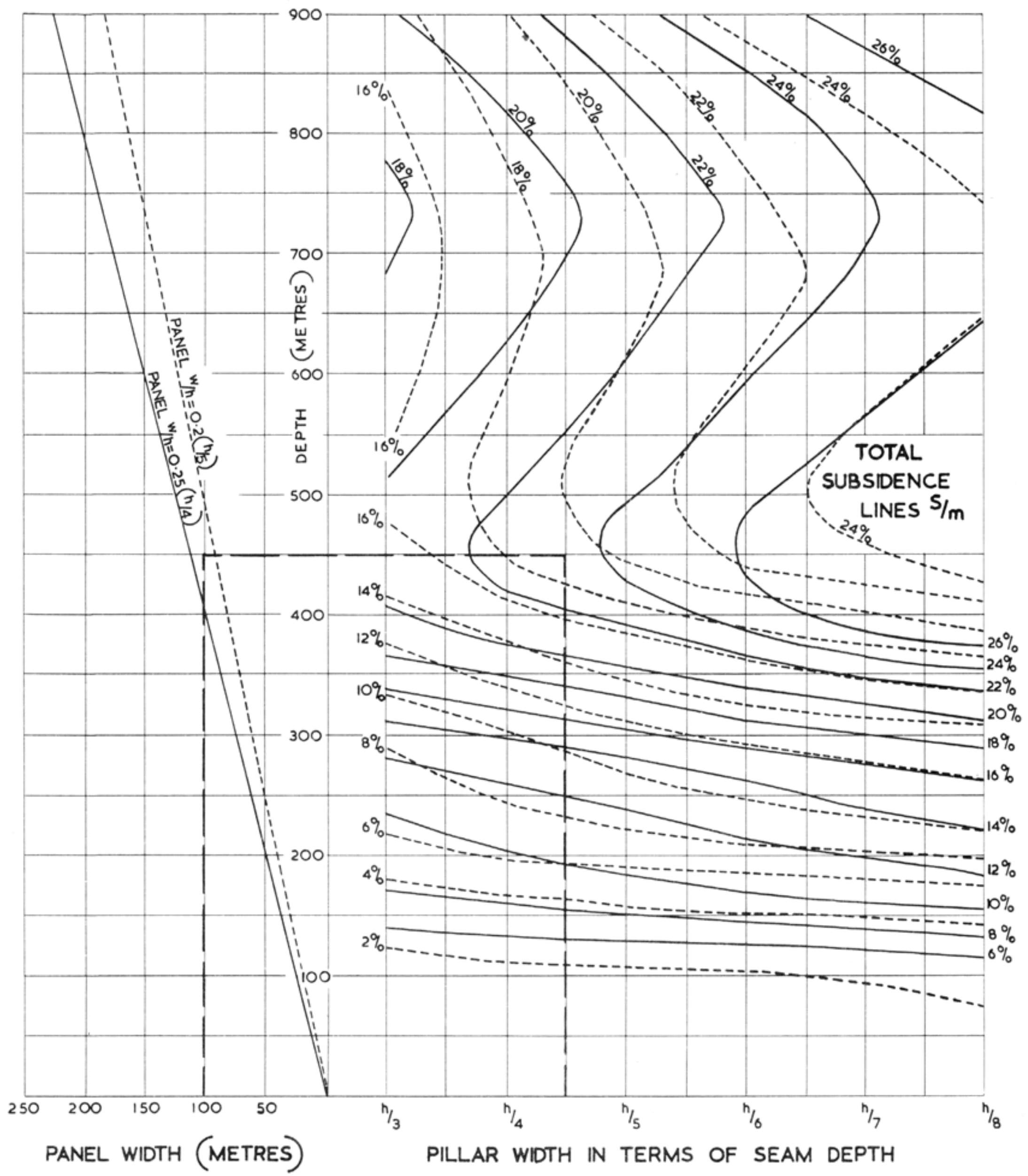


Fig. 96 Partial extraction layout nomogram—for width/depth ratios 0.20 to 0.25.

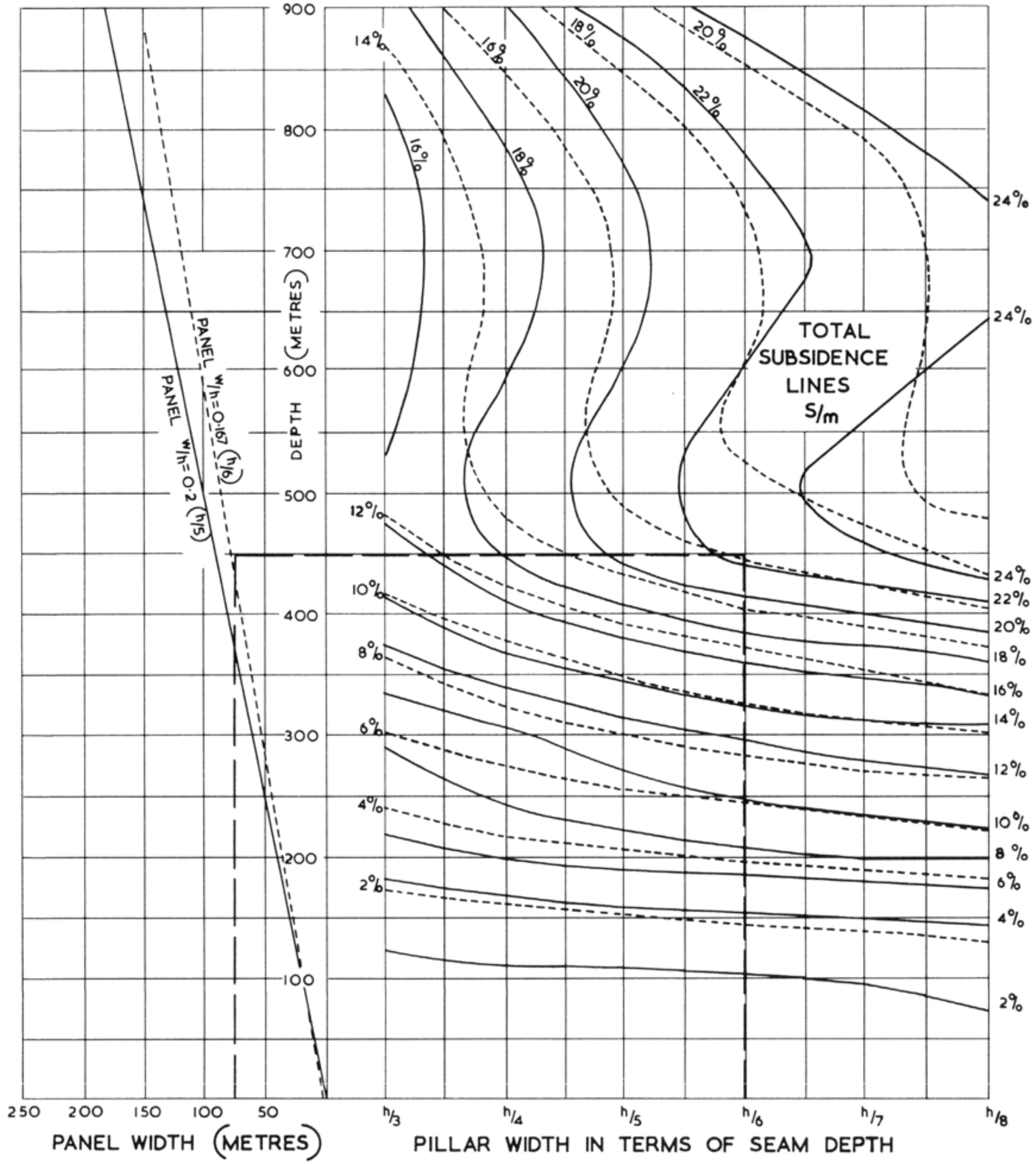


Fig. 97 Partial extraction layout nomogram—for width/depth ratios 0.167 to 0.20.

sewage disposal schemes. A built-up area may consist mainly of small houses which could withstand one or two millimetres per metre strain without undue damage. A large factory may, on the contrary, contain machinery which could not be tilted by more than, say, 1 in 2,000 without going out of production.

In each of these types of case the subsidence, strain and tilt limits can be calculated as in Sections 2, 3 and 4 of this Handbook from a knowledge of the seam depth and thickness and a subsidence of x% of the seam thickness is noted.

The nomograms 95, 96 and 97 can be used either forwards or backwards. Supposing the seam lies at a depth of 450 metres and a desirable face length for production purposes is chosen as 100 metres, then on the left hand side of Fig. 96 for example, the w/h ratio of the panel would be 0.2125 or w=h/4.5 i.e. a point midway between the solid line of h/4 and the broken line of h/5.

If a horizontal line is now followed to the righthand side, it is seen that the (total) percentage subsidence depends upon the pillar size selected.

If we select a pillar size of h/4.5, giving 50% extraction, we reach a point between the 18% and 20% broken lines (actual interpolation 19.0%) and between the 22% and 24% solid lines (actual interpolation 23.4%). The interpolated value between 19.0% and 23.4% is 21.2%. If this amount of subsidence is excessive then we can choose instead either a narrower panel or a wider pillar. If the former, and, for example w=h/5 exactly, we must ignore the solid percentage subsidence lines, and our subsidence drops to 19.0%. If we were, at the same time, to take a wider pillar, say h/3, then the total subsidence is 15%, i.e. between the broken 14% and broken 16% lines.

The percentage extraction is now reduced to about 37% and it is worth turning to Fig. 97 and considering a 75 metre panel, i.e. h/6 and a 50% extraction layout with h/6 pillars, giving 16% subsidence.

If the seam in this case had an extracted thickness of 2.0 metres, then $S=2.0\times 0.16=0.32$ metres.

If required face lengths necessitate panels with w/h ratios such as h/3.75, the method is the same as before, but the final interpolation, between the values of the broken and solid subsidence percentage lines must be in the same proportion as the w/h ratio of the panel is between the w/h ratio of the panels shown by solid and broken lines on the lefthand side of the graph. For example, Panel h/3.75 Pillar h/5, h=375 m and using Fig. 95, the value read between the solid subsidence percentage lines is 31.2% and between the broken lines 21.0% interpolated value 28.6%.

Strata Control Pillars

The surveyor or subsidence engineer may frequently be required to advise on pillars which have an underground support function in places where there is no surface damage problem. That subject is beyond the scope of this handbook and the reader should refer to the work of A. H. Wilson (502). For convenience

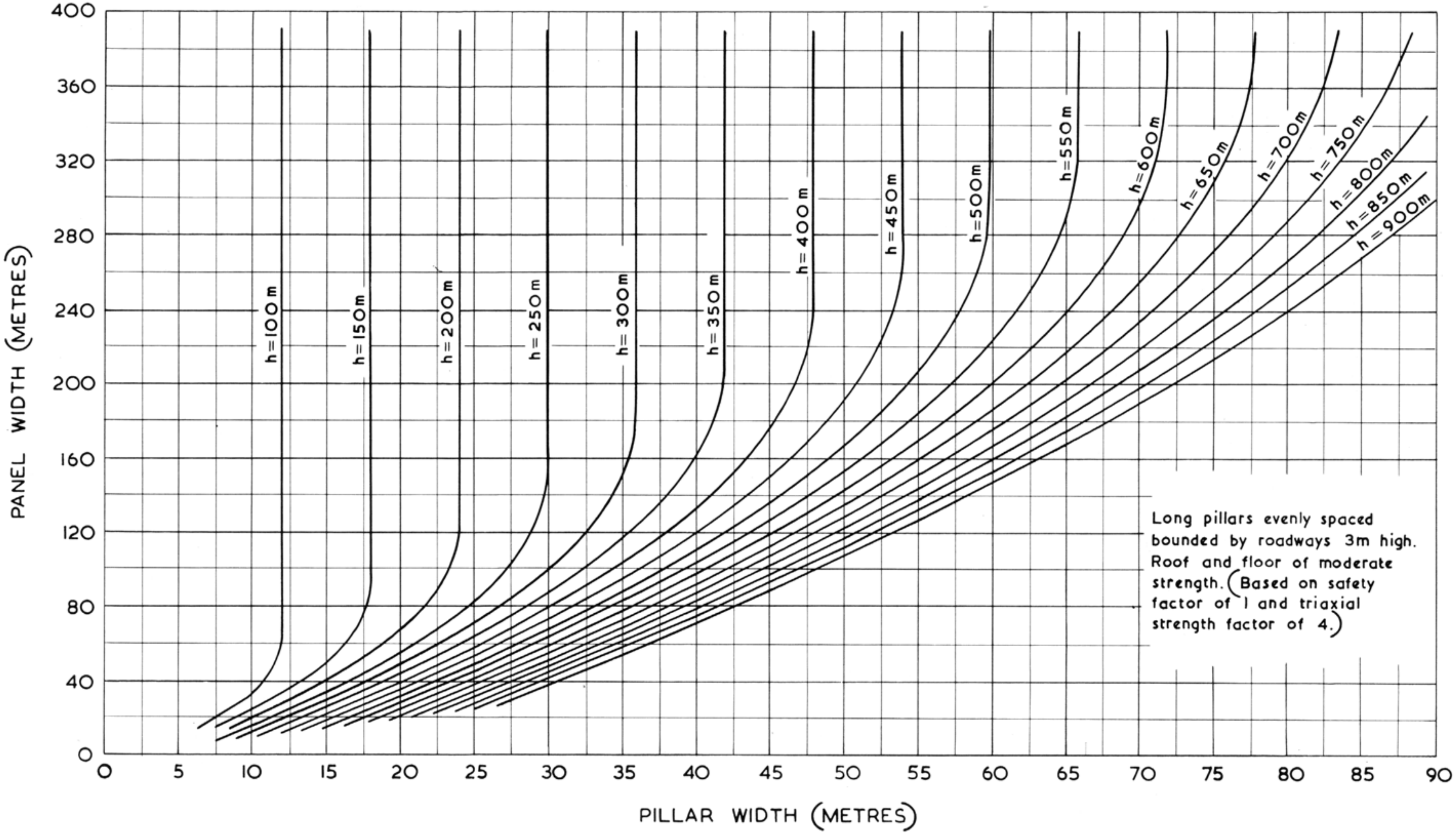


Fig. 98 Relationship of width of panels and stable pillars in longwall workings.

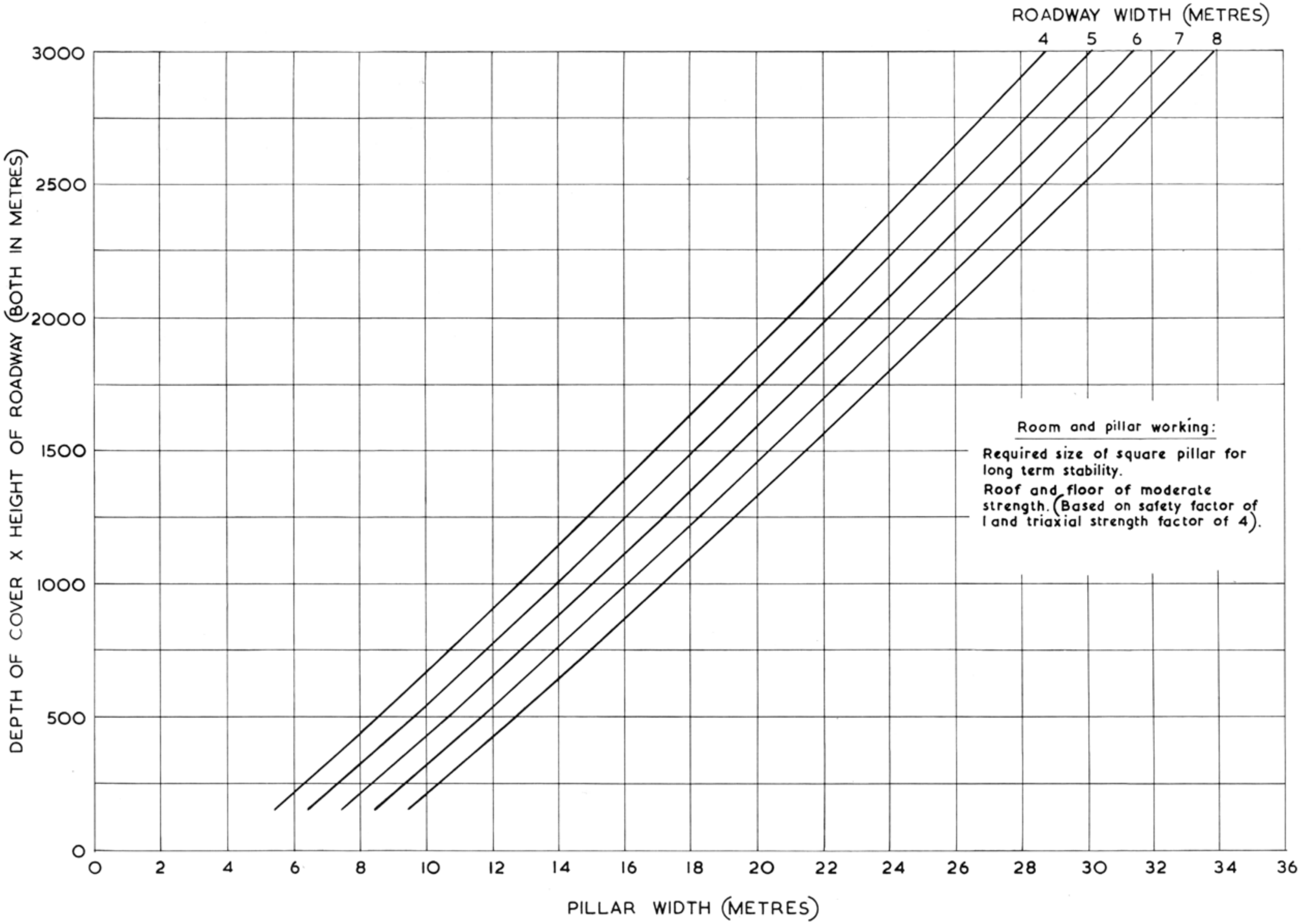


Fig. 99 Pillar sizes in room and pillar workings.

of reference, however, the nomograms relating to minimum stable pillar dimensions are reproduced in Figs. 98 and 99 and the following main points, based on observation, are important:—

a) that a stable pillar consists of a non-yielding core surrounded by a yielding marginal zone $\hat{y}$ equal in width to 0.005 m.h. metres;
b) that because of the lateral constraint afforded by the yielding zone the core of the pillar may be loaded up to four times the cover load:
c) that the stress in a pillar increases linearly from 0 at the pillar edge to $\hat{\sigma}$ at $\hat{y}$;
d) the stress in the waste increases linearly from 0 at the waste edge to a maximum at 0.3 h from the waste edge:

where $\hat{y}$ = width of yield zone in metres;
m = height of extraction in metres;
h = depth of cover in metres;
$\hat{\sigma}$ = maximum permissible stress in the core of a pillar in MN/m^2; and

where the vertical pressure gradient (i.e. the rate of increase of pressure with depth) in the U.K.

= 1.1 lb/in^2 per foot of depth;
= 0.025 MN/m^2 per metre of depth

Fig. 98 applies to long pillars with 3 m high roads, moderately strong roof and floor and a safety factor of 1*. The graph enables the appropriate pillar width to be read off for a given depth and a given width of panel.

A "stable" pillar has a width equal to 0.12 h which is in fact a pillar which would remain stable indefinitely no matter what width of panel were to be extracted on each side of it. The maximum effective total width of panel is in fact assumed to be 0.6 h (0.3 h each side) since full cover load is applied at 0.3 h from the waste edge. Thus the curves on the graph become vertical lines when the stable pillar width is reached.

Fig. 99 shows the safe size of square pillars in pillar and stall workings for four different widths of heading surrounding the pillar.

**Note:* Any departure from moderate strength might necessitate a safety factor greater than 1 being used, and such cases should be examined on their merits.

APPENDIX 1

Case Histories of Observation Lines

Case No.	Division	Area	Colliery	Observation Line	Seam	Direction of Line	Panel Width ft.	Mean Depth ft.	Width Depth Ratio	Goaf Treatment	Remarks
1	E.M.	5	Denby Hall	7 (Peasehill Road, Ripley)	Blackshale (22's panel)	Transverse	(mean) 632	830	0.76	Strip Packing	Face length varies. One Ribside one Goafside.
2	E.M.	5	Denby Hall	3 (Upper Hartshay)	Blackshale	Transverse and Longitudinal	variable	416	>4	Strip Packing 45–50%	Panel width reduces 180′ past observation line.
3	Kent	–	Chislet	Supperton Farm	No. 7 (1 panel)	Transverse and Longitudinal	420–260	1,690	0.25–0.15		
4(a)	E.M.	6	Wollaton	1 Wollaton Park	Piper (2E)	Transverse	(least) 642	726	0.88	Strip packed 48%	Width varies (around pillar line)
4(b)	E.M.	6	Wollaton	2 Wollaton Park	Piper (3E)	Transverse	1,119	756	1.48	Strip 45%	Centre gate. Combined effect of 4(a) and 4(b) observed.
5	E.M.	5	Moorgreen	No. 2	Low Main (11's)	Transverse	(mean) 480	1,190	0.37	Strip packed	Width varies.
6	Yorks.	5	Wharncliffe Silkstone	Westwood Main Rd.	Flockton	Transverse	330	330	1.0	Slusher stowed	Centre gate. One ribside one goaf side.
7	Yorks.	5	Wharncliffe Silkstone	Moon Lane, Tankersley	Flockton	Transverse	330	375	0.8	Striped Packed (40%)	Two ribsides.
8	Yorks.	5	Wombwell Main	Aldham House Farm	Winter	Transverse and Longitudinal	420	333	1.26	40% Packing	One ribside, one goafside.
9	E.M.	4	Newstead	Nos. 1 & 2 Newstead Station	High Main (S. 1s)	Transverse and Longitudinal	765	665	1.15	2 yd. packs 10 yd. wastes	One ribside.
10	E.M.	4	Newstead	No. 3 (Vernon Farm)	High Main (17s)	Diagonal	890	825	1.08	Caved	One ribside.
11	E.M.	4	Newstead	No. 4	High Main (15s and 17s)	Transverse	(15s) 880	771	1.14	Caved	Hump over pillar between two panels.
							(17s) 890	804	1.11	Caved	
12	E.M.	4	Newstead	Nos. 5/6	High Main (15s)	Transverse and Longitudinal	(mean) 790	770	1.03		Width varies. Centre gate. One ribside, one goafside.
13	Yorks.	6	Bullcliffe Wood	Bullcliffe Farm	Top Haigh Moor	Transverse and Longitudinal	354	259	1.37	Not known	Face finished beneath line.
14	E.M.	4	Shirland	Oakerthorpe	Deep Soft (18s)	Longitudinal	(mean) 315	103	3.06	Strip packed (35%)	Face finished beneath line.
15	E.M.	5	Moorgreen	No. 5 (Long Lane)	2nd Waterloo (2s panel)	Transverse and Longitudinal	920	790	1.16	Strip packing	Centre gate.
16	E.M.	4	Newstead	No. 5	High Main (17s)	Transverse	890	787	1.13	Caved	Hump over pillar between 15s and 17s.
17	W.M.	4	Haunchwood	C. 1/A	Bench	Diagonal	790	1,308	0.60	Not known	Centre gate.
18	W.M.	4	Ansley Hall	Line 22–32	Ryder	Diagonal	430	1,269	0.33	Not known	No strain measurement. Large station interval.
19	W.M.	4	North Warwick	Stipers Hill	7 ft.	Transverse	1,000 (approx.)	600	1.67	Not known	Old case. Width varies. No strain measurement. Large station interval.
20	W.M.	4	North Warwick	Stipers Hill	Deep Ryder	Transverse	870	690	1.26	Not known	Old case. Width varies. No strain measurement. Large station interval. Effect added to case 19.
21(a)	E.M.	7	Rawdon	Bath Yards	Stockings. (3 panels)	Transverse	860	883	0.97	Pneumatically stowed excellent	15′ heading extractions leaving 2′ 3″ fenders.
21(b)	E.M.	7	Rawdon	Bath Yard 4	Stockings (4 panels)	Transverse	1,200	894	1.34	Pneumatically stowed excellent	15′ Heading extractions leaving 2′ 3″ fenders.
22	E.M.	7	Rawdon	Bath Yard 2	Stockings (2 panels)	Transverse	610	882	0.69	Pneumatically stowed excellent	15′ Heading extractions leaving 2′ 3″ fenders.
23	E.M.	7	Rawdon	Bath Yard 1	Stockings (1 Panel)	Transverse and short Longitudinal	345	876	0.39	Pneumatically stowed excellent	15′ Heading extractions leaving 2′ 3″ fenders.

Case No.	Division	Area	Colliery	Observation Line	Seam	Direction of Line	Panel Width ft.	Mean Depth ft.	Width Depth Ratio	Goaf Treatment	Remarks
24(a)	E.M.	7	Granville	Reservoir	Kilburn (1 panel)	Transverse and short longitudinal (2 stations)	420	1,267	0.33	Strip packing good (6 yd. x 6 yd.)	Interim report on case 24(b).
24(b)	E.M.	7	Granville	Reservoir	Kilburn (2 panels)	Transverse and short longitudinal (2 stations)	875	1,272	0.71	Strip packing good (6 yd. x 6 yd.)	Panel width increasing.
24(c)	E.M.	7	Granville	Donington	Kilburn	Transverse and short longitudinal (2 stations)	1,055	1,304	0.67	Strip packing good (6 yd. x 6 yd.)	Panel width increasing. Short face advance after line.
25	E.M.	7	Snibston	Coalville	Lower Main	Transverse (diagonal)	196	830	0.24	Pneumatic stowing	First panel of 50% Partial extraction scheme. Effect separable from aggregate effect.
26	E.M.	7	Snibston	Coalville	Lower Main	Transverse	196	812	0.24	Pneumatic stowing	Second panel of above. Some residual subsidence from 1st panel.
27	E.M.	7	Rawdon	Sweethill Lodge	Stockings (8s)	Transverse and 3 stations longitudinal	(mean) 500	1,225	0.41	Strip packing (good)	
28	E.M.	7	Rawdon	Sweethill Lodge	Stockings	Transverse and longitudinal (3 stations)	1,100	1,225	0.90	Strip packing (good)	Additional panel worked alongside above case.
29	E.M.	3	Thoresby	Perlethorpe Church	Top Hard (51s)	Transverse and some point levels	660	1,975	0.33	Strip packing	
30	E.M.	3	Sherwood	Intake Farm School	Deep Soft 3s Panel	Transverse	591	1,562	0.38	Strip packed 47% good	
31	E.M.	3	Sherwood	Intake Farm School	Deep Soft 5s Panel	Transverse	607	1,314	0.46	Strip packed 38% good	
31(a)	E.M.	3	Sherwood	Intake Farm School	Deep Soft 8s Panel	Transverse and longitudinal	540	1,510	0.36		8s Panel is alongside 3s.
32	York.	6	Bullcliffe	Hoylands Farm	Barnsley	Transverse and longitudinal	320	80	4.02	Not known	
33	York.	3	Wath Main	Hollowgate Avenue	Melton Field	Diagonal	420	354	1.19		Concentration of packing in panel centre.
34(a)	W.M.	2	Yew Tree Drift	Yew Tree Drift		Transverse/ Longitudinal	500	148	3.38	Not known	Very old case – no strain measurements.
34(b)	W.M.	2	Yew Tree Drift	Yew Tree Drift		Transverse	(mean) 145	at rib 691	2.10	Not known	Width varies – short advance passed line – old case.
35(a)	W.M.	4	Coventry	Line E – F	Thick Coal	Longitudinal	2,350 (approx.)	2,350	1.00	Strip packing	Incomplete record. Short face advance.
35(b)	W.M.	4	Coventry	Line E – F	Thick Coal	Longitudinal	910	2,130	0.43	Strip packing	Old case incomplete record.
36	W.M.	4	Coventry	Line G – H	Thick Coal	Transverse	900	2,155	0.42	Strip packing	Old case no strain measurements.
37	W.M.	4	Coventry	Corley Church	Thick Coal	Longitudinal	(min.) 1,700	2,200 (approx.)	0.77 (min.)	Strip packing	Old case $\frac{\text{Advance}}{\text{Depth}} = 0.76$
38	E.M.	5	Moorgreen	No. 5 Long Lane	2nd Waterloo Panel 3	Transverse	690	870	0.79	Caved	Hump effect through coal left between panels.

Case No.	Division	Area	Colliery	Observation Line	Seam	Direction of Line	Panel Width ft.	Mean Depth ft.	Width Depth Ratio	Goaf Treatment	Remarks
39	E.M.	5	Moorgreen	No. 5 Long Lane	2nd Waterloo Panel 4	Transverse	782	911	0.86	Strip	
39(a)	E.M.	5	Moorgreen	No. 5 Long Lane	2nd Waterloo Panel 5	Transverse	708	956	0.74	Caved – Part strip packed	Not final figures.
40	E.M.	4	Newstead	Abbeyfields Farm	High Main South 4s	Short transverse longitudinal	771	689	1.12	Caved	Plumbings made on brick pillars. Face finished beneath line.
41	E.M.	1	Ireland			Transverse	460	530	0.87		Incomplete record.
42	E.M.	1	Parkhouse		Tupton	Transverse	610	103	5.92		No strain available. Incomplete record.
43	W.M.	1	Florence	9	Moss	Transverse	960	2,448	0.39	Strip packing	
44	W.M.	1	Foxfield	7 (Dilhorne)	Dilhorne	Diagonal	(mean) 600	540	1.11	Strip 58%	Width varies.
45	W.M.	1	Stafford	19	Whinghay	Transverse/ Longitudinal	900	1,371	0.66	Strip packing	See Case 47. Face changes direction.
46	W.M.	1	Stafford	Sideway Canal	Whinghay	Diagonal	930	1,266	0.74	Strip packing	
47	W.M.	1	Stafford	18	Whinghay	Transverse/ Longitudinal	900	1,320	0.68	Strip packing	See case 45. Face changes direction.
48	W.M.	1	Hemheath	2	Cannel Row	Transverse	(mean) 600	1,770	0.34	Strip 51%	Width varies.
49	W.M.	1	Hemheath	1	Great Row	Transverse	810	1,347	0.601	Strip 48%	Width varies.
50	W.M.	1	Norton	1	Bullhurst	Transverse	670	810	0.83	Strip packed 56.5%	Face changes direction.
51	W.M.	1	Chatterley Whitfield	4	Bellinger	Transverse and longitudinal	555	438	1.27	Strip packed 62.5%	
52	W.M.	1	Wolstanton	9	Peacock	Transverse	480	1,851	0.26	Dummy Gates 67%	Width varies.
53	W.M.	1	Wolstanton	27	Peacock	Transverse	660	1,830	0.36	Strip packing	
54	W.M.	1	Victoria	3 and 3A Brownlees	Diamond	Transverse and longitudinal	750	1,260	0.60	Strip packing	
55	E.M.	6	Radford	No. 1 Wollaton Park	Tupton (31s)	Transverse short advance line	834	828	1.01	Strip packing 45%	Centre gate.
55(a)	E.M.	6	Wollaton	No. 3 Wollaton Park	Deep Hard (25s)	Transverse short advance line	(mean) 1,074	750	1.43	Strip 49%	Short face advance past the line.
56	E.M.	5	Selston	12 High Park	Blackshale 102°	Transverse short advance line	646	1,343	0.48	Caving	
57	E.M.	7	Stanhope	Ashley Road East	Kilburn	Transverse	487	420	1.16	Pneumatic (fair)	
58	E.M.	4	Newstead	3	High Main 19s	Transverse	790 (approx.)	833	0.95	Caved	Width varies.
59	E.M.	6	Bestwood	No. 1 Mansfield Turnpike	High Main 5s Panel	Transverse	1,044	936	1.12	Sfrip packing 45%	First set of lines before Sherwood Lodge Lines. See cases 156/157.
60	E.M.	6	Bestwood	No. 1 Mansfield Turnpike	High Main 7s Panel	Transverse	1,020	936	1.09	Strip packing 45%	
62	E.M.	5	Moorgreen	2 (Moorgreen – Watnall)	Low Main 12s	Transverse	555	1,159	0.48	Strip packing	
63	E.M.	5	Moorgreen	2 (Moorgreen – Watnall)	Low Main 10s	Transverse	468	1,145	0.41	Strip 3 yards packs 7 yards wastes	
64	E.M.	5	Moorgreen	2 (Moorgreen – Watnall)	Low Main 18s	Transverse	490	1,162	0.42	Strip packing	
64(a)	E.M.	5	Moorgreen	2 (Moorgreen – Watnall)	Low Main 19s	Transverse	725	1,160	0.62		145 ft. pillar between 19s and 10s Panels (Case 63).

Case No.	Division	Area	Colliery	Observation Line	Seam	Direction of Line	Panel Width ft.	Mean Depth ft.	Width Depth Ratio	Goaf Treatment	Remarks
65	E.M.	5	Coassall	4 Strelley	Piper	Transverse/ Advance	1,800	640	2.81	Some strip packing some caved	Harmonic extraction.
66	E.M.	5	Moorgreen		Piper	Longitudinal	1,500	690	2.17		
67	Sc.	1	Mary Lees		Yard	Transverse	750	450	1.67		
68(a)	N. & D.	1	Weetslade		Low Main 40s and 41s	Transverse and longitudinal	580	908	0.64	Caved	
68(b)	N. & D.	1	Weetslade		Low Main 40s 41s 44s	Transverse and longitudinal	830	908	0.91	Caved	
69											
70											
71	Sc.	6	Manor Powis	Cambus (2–43)	Upper Hirst	Transverse	520	490 (approx.)	1.06		No strains measured.
72	Sc.	6	Manor Powis	Cambus (71–93)	Upper Hirst	Transverse	480	492	0.98		No strains measured.
73	E.M.	5	Woodside	8 Heanor, B's (Sunningdale Avenue)	Ashgate and Mickley Bs panel	Transverse	768	1,151	0.67	Strip packing	Some coal pillars left in the Ashgate Seam.
74	Yorks.	5	Skiers Spring	Stead Lane	Swallow Wood	Transverse and longitudinal	372	132	2.82	Slusher Stowed	
75	Yorks.	5				Diagonal	336	692	0.50		
76	E.M.	5	Denby Hall	11 Alfreton Hosiery, Factory, Ripley	Blackshale 31s	Transverse short advance	422	625	0.67	45 to 50% Strip packing	
77	W.M.	4	Coventry	X – Y	Thick Coal	Transverse	(mean) 800	2,077	0.38		Width varies. Incomplete record.
78	W.M.	4	Coventry	G – H	Thick Coal	Transverse	900	1,942	0.46		
79	W.M.	2	Cannock Wood	Cannock Wood	Yard	Transverse/ Longitudinal	900	375	2.40	Strip 47%	
80	W.M.	4	Arley	Spring Hill	Rider	Transverse	390	1,161	0.34	Strip	
81	W.M.	4	Haunchwood	Church Lane	9 ft. (Ds)	Transverse/ Longitudinal	1,180	1,390	0.85	Strip	
82	N.W.	1	Bradford	A – B (502–270)	Roger	Tramsverse	845	2,225	0.38	Strip	Width varies.
83	N.W.	1	Bradford	188–202	Roger	Transverse	910	2,080	0.44	Strip	
84	N.W.	1	Bradford	83–96	Roger	Transverse	930	2,035	0.46	Strip	
85	E.M.	7	Whitwick	Broomsleys Road	Nether Lount	Transverse/ Longitudinal	220	915	0.24	Strip – good	
86	W.M.	1	Foxfield	9 (Brookhouses)	Dilhorne	Transverse/ Longitudinal	530	325	1.63	Strip 58%	Width varies.
87	W.M.	1	Foxfield	12 (Brookhouses)	Dilhorne	Transverse	470	280	1.70	Strip	Affected by coal pillars.
88	W.M.	1	Foxfield	7 (Dilhorne)	Alecs	Diagonal	460	625	0.74	Strip	
89(a)	W.M.	1	Wolstanton	28/29	Cannel Row	Transverse/ Longitudinal	(mean) 460	1,805	0.25	Strip	Width varies.
89(b)	W.M.	1	Wolstanton	28/29	Cannel Row	Transverse/ Longitundinal	(mean) 890	1,830	0.49	Strip	Panel worked alongside the one in Case 89(a).
90	W.M.	1	Norton	2 (West)	Bowling Alley	Transverse	(mean) 660	1,105	0.60	Strip	Width varies.

Case No.	Division	Area	Colliery	Observation Line	Seam	Direction of Line	Panel Width ft.	Mean Depth ft.	Width Depth Ratio	Goaf Treatment	Remarks
91	W.M.	1	Norton	2 (West)	Bowling Alley	Transverse	390	880	0.44	Strip	
92	W.M.	1	Glasshouse	5	Peacock	Transverse	180	150	1.20	Strip	
93	W.M.	2	Hamstead	6	Thick Coal	Transverse/ Longitundinal	700	2,546	0.27	Strip packing 35%	
94											
95	W.M.	2	Baggeridge	4	Thick Coal	Transverse	450	2,236	0.20	Strip packs 31%	
96	N. & D.	1	Hylton	Hylton Site (63–49)	Maudlin	Transverse	680	1,375	0.50	Caved	See Cases 97 – 148 Records in Box 3/2.
97	Durham	1	Hylton	Hylton Site (12–40)	Maudlin	Transverse	680	1,320	0.52	Caved	See Cases 96–148 Records in Box 3/2.
98	W.M.	1	Deep Pit	32 (Hanley)	Yard/Ragman	Transverse	410	2,609	0.16	Strip 48%	
99	W.M.	1	Deep Pit	30 (Hanley)	Yard/Ragman	Transverse	410	2,653	0.16	Strip	
100	W.M.	1	Holditch	6/7 (Chesterton)	Cannel Row	Transverse	(mean) 450	2,070	0.22	Solid stowed	
101	W.M.	1	Holditch	128 (Chesterton)			690	1,950	0.35	Caved	
102	E.M.	7	Merry Lees	Newbold Verdon	Yard	Transverse/ Longitundinal	760	426	1.78	Caved	
103	E.M.	6	Gedling	2 (Stoke Bardolph)	High Hazles (A.P.D.s)	Diagonal/ Longitundinal	590	1,337	0.44	Strip packing 30%	
103(a)	E.M.	6	Gedling	2 (Stoke Bardolph)	High Hazles (B2s and B4s)	Diagonal	2,382	1,337	1.78	Strip packing 29%	
104				Trek 81	Maurits	Transverse	1,968	1,406	1.40		Dutch State Mines.
105	E.M.	7	Cadley Hill	Castle Gresley	Main Nether (5s)	Transverse/short longitundinal	390	438	0.89	Solid stowing excellent	See 189.
106	E.M.	7	Cadley Hill	Castle Gresley	Main Nether (7s)	Transverse/short longitundinal	375	353	1.06	Pneumatic stowing excellent	See 189.
107	E.M.	6	Bestwood	Arnold	High Main (30s)	Short Advance longitundinal	1,200	915	1.31	Strip packing good	Two Point observation.
108	E.M.	6	Calverton	Calverton	High Main (46s)	Short longitudinal	1,350	1,242	1.09	Strip packing good	Two Point observation.
109	E.M.	3	Thoresby	Thoresby Park	Top Hard (53s)	Transverse/Advance	951	1,971	0.48	Strip packing 40%	
109(a)	E.M.	3	Thoresby	Perlethorpe Church	Top Hard (53s)	Advance	951	1,971	0.48	Strip packing 40%	
110	E.M.	3	Shirebrook	Hollins Doubling Mill	Clowne	Transverse/Advance	1,870	914	2.05	Strip packing 26%	
111	Yorks.	2	Askern	Instoneville	Flockton (1s)	Transverse/ Longitundinal	690	2,196	0.31	Strip packing	
112	Yorks.	1	Manton	Clumber	Barnsley	Transverse	550	1,973	0.28	Strip packing	
113	Yorks.	1	Treeton	Aston Common	High Hazel	Transverse	1,470	705	2.09	Strip packing	
114	Yorks.	1	Steetley	Welbeck Abbey	Barnsley	Longitundinal	(mean) 1,120	1,513	0.74	Strip packing	4 yd. packs. 20 yd. wastes.
115	Yorks.	1	Steetley	Welbeck (Roomwood)	Barnsley	Transverse	1,120	1,800	0.62	Strip packing	4 yd. packs. 20 yd. wastes.
116	Yorks.	6	Park Mill	Bretton Hall – East Side	Wheatley Lime	Diagonal	1,049	632	1.66		Horizontal movement of pegs measured. Harmonic extraction. Width varies.
117	Yorks.	6	Park Mill	Bretton Hall – West Side	Wheatley Lime	Transverse	556	450	1.24		Horizontal movement of pegs measured.

Case No.	Division	Area	Colliery	Observation Line	Seam	Direction of Line	Panel Width ft.	Mean Depth ft.	Width Depth Ratio	Goaf Treatment	Remarks
118	Yorks.	3	Wath Main	Sandygate	Meltonfield	Diagonal	495	333	1.49	Strip packing 60%	Centre gate.
119	Yorks.	3	Wath Main	Burman Road	Meltonfield	Transverse	495	333	1.49	Strip packing 60%	2nd Line over panel in case 118.
120	Yorks.	3	Wath Main	Dearne Road	Newhill	Transverse/ Longitundinal	480	450	1.07	Strip packing 60%	
121	Yorks.	5	Barnsley Main	Ardsley Village	Kents Thick	Transverse	354	948	0.37	Pneumatic	
122(a)	Yorks.	8	West Riding (Fox Pit)	In fields near Fox Pit	Warren House	Transverse/ Longitundinal	358	204	1.75	Caved	
122(b)	Yorks.	8	West Riding (Fox Pit)	In fields near Fox Pit	Warren House	Transverse	715	204	3.50	Caved	Panel worked alongside the one in case 122(a).
123	Yorks.	8	Prince of Wales	Spittal Hardwick Lane	Castleford 4′ (304s)	Diagonal/ Longitundinal	658	750	0.88		Complicated by adjoining workings.
124	E.M.	3	Clipstone	Clipstone Village	Low Main (34s)	Transverse	660	2,730	0.24	Strip packing 28%	
125	E.M.	4	'A' Winning	St. Werburghs Church (Blackwell)	Low Main	Transverse/ Longitundinal	583	880	0.66	Strip packing	Harmonic extraction of support pillar.
126	W.M.	1	Foxfield	137	Dilhorne	Diagonal	900	460	1.96	Strip 53%	Width varies.
127	W.M.	1	Florence	124	Bowling Alley	Transverse	550	2,450	0.22	Caved	Working alongside old goaf.
128	W.M.	1	Silverdale	134 (Knutton)	Cannel Row	Transverse	550	900	0.61		
129	W.M.	1		136 (Knutton)			585	840	0.70	Strip	Centre gate.
130	E.M.	7	Merry Lees	Newbold Verdon	Lower Main	Transverse/Advance	500	471	1.06	Strip packing	See case 102 for workings in the same area.
131	Yorks.	1	Shireoaks	Highgrounds	Clowne (2s)	Transverse	600	1,114	0.54	Caved	
132	Yorks.	1	Shireoaks	Highgrounds	Clowne (20s)	Transverse/ Longitundinal	920	1,140	0.51	Caved	Panel at right angles to case 131, Coal Pillar left.
133(a)	Yorks.	1	Shireoaks	Shireoaks Common	Clowne (1s)	Transverse	560	1,016	0.55	Caved	
133(b)	Yorks.	1	Shireoaks	Shireoaks Common	Clowne (1As)	Transverse	540	1,016	0.53	Strip packing	Panel worked alongside the one in Case 133(a).
133(c)	Yorks.	1	Shireoaks	Shireoaks Common	Clowne (3s)	Transverse	920	1,016	0.91	Strip packing	Panel worked alongside the one in case 133(a). one in case 133(a). Finishes beneath
135											
134	Yorks.	1	Handsworth	Woodhouse	Flockton	Transverse/ Longitundinal	880	420 (mean)	2.10		
136	W.M.	1	Silverdale	Knutton			585	840	0.70	Strip packing	Centre gate. Old workings 25 yd. to dip side of panel. Max subsidence on rise side of surface position of Apedale fault.
137											
138											
139											
140											
141	W.M.	1	Fenton (7)	Longton Gas Works	Banbury	Transverse	120	2,400	0.05	Strip packing	
142											
143											
144	N. & D.	3	Shotton	Area Cases 1 and 2	Low Main	Transverse/ Longitundinal	(mean) 869	889	0.98	Caved	

Case No.	Division	Area	Colliery	Observation Line	Seam	Direction of Line	Panel Width ft.	Mean Depth ft.	Width Depth Ratio	Goaf Treatment	Remarks
145(a)	N. & D.	3	Shotton	Area Cases 3 and 4	Five Quarter	Transverse/ Longitudinal	883	951	0.93	Strip packing	
145(b)	N. & D.	3	Shotton	Area Case 5	Five Quarter	Diagonal	883	951	0.93	Strip packing	2nd line over panel in Case 145(a).
146	N. & D.	3	Horden	Area Cases 6 and 7	High Main	Transverse/ Longitudinal	627	902	0.70	Pneumatic	
147	N. & D.	3	Horden	Area Case 8	High Main	Transverse	604	922	0.65	Poor pneumatic (35%)	Panel worked alongside the one in Case 146.
148	N. & D.	1	Hylton	Hylton Site Nos. 119–102	Maudlin	Transverse	689	1,263	0.55	Caved	See Cases 96 and 97.
149	N. & D.	1	Washington Glebe	North Biddick Hall Site	Brass Thill	Transverse	679	625	1.09	Caved	
150											
151											
152(a)	E.M.	6	Babbington	Rushcliffe Road, Hucknall	Deep Soft (2s)	Short longitudinal	984	1,601	0.61	Caved	"Two point" observation.
152(b)	E.M.	6	Babbington	Annie Holgate School, Hucknall	Deep Soft (2s)	Diagonal	984	1,647	0.60	Caved	Line outside panel.
153	E.M.	6	Wollaton	No. 3 Wollaton Park	Tupton (45s)	Transverse/ Longitudinal	1,148	869	1.32	Strip packing followed by caving	Longitudinal line over ribside.
154	E.M.	5	Mansfield	Nottingham Road	High Hazles (4s)	Transverse/ Longitudinal	696	553	1.26	Strip packing 30%	Data incomplete due to premature stopping of face.
155	N.W.	E. Lancs.	Agecroft	Prestwick District	Worsley	Transverse/diagonal	710	1.020	0.70	Strip packing	
156	E.M.	6	Bestwood	No. 1 Sherwood Lodge	High Main (5s and 7s)	Transverse/short longitudinal	1,518 (2 panels)	932	1.63	Strip packing	Harmonic Extraction – Line over common cutting side. (See Cases 59 and 60.)
157	E.M.	6	Bestwood	No. 2 Sherwood Lodge	High Main (5s and 7s)	Transverse/ Longitudinal	1,518 (2 panels)	957	1.59	Strip packing	Second line over common cutting side in Case 156. Width varies.
158	Yorks.	1	Dinnington	Todwick Grange Line	Barnsley	Transverse/ Longitudinal	600 (appox.)	1,300	0.46		
159	Yorks.	2	Brodsworth	Woodlands Village	Dunsil (123s)	Transverse/short longitudinal	700	1,920	0.36	Caving	
160	Yorks.	2	Brodsworth	Green Lane, Scawthorpe	Thorncliffe (314s)	Transverse/short longitudinal	750	2,589	0.29	Caved	
161(a)	Yorks.	2	Hatfield	Fish Lake, Right Barrier Bank	High Hazel (56s)	Transverse	420	2,010	0.21	Strip packing 3 ydP 8 ydW	
161(b)	Yorks.	2	Hatfield	Fish Lake, Left Barrier Bank	High Hazel (56s)	Transverse	420	2,010	0.21	Strip 3 yd Packs 8 yd wastes	Second line over panel in Case 161(a).
161(c)	Yorks.	2	Hatfield	Fish Lake, Left Barrier Bank	High Hazel (57s)	Longitudinal	420	2,010	0.21	Strip 3 yd Packs 8 yd wastes	Worked from stop position of panel 58s in Case 161(b).
162	Yorks.	5	Skiers Spring	"Doric Lodge"	Swallow Wood (228s and 227s)	Diagonal	735	198	3.71	Strip 41%	
163	Yorks.	5	Dodworth	Gawber	Haighmoor	Transverse/ Longitudinal	570	324	1.76	Strip 34%	
164(a)	Yorks.	5	Smithy Wood	Wentworth	Lidgett (76s)	Transverse	675	330	2.05	Strip 26%	
164(b)	Yorks.	5	Smithy Wood	Wentworth	Lidgett (76s)	Transverse	675	330	2.05	Strip 26%	2nd line over panel in Case 164(a).
165	Yorks.	5	Dodworth	Rowland Road	Haighmoor	Transverse/ Longitudinal	600	417	1.44	Pneumatic	

Bibliography

This list is by no means complete, but contains the literature most relevant to ground movement studies.

General

1. AYNSLEY, W. J. and HEWITT, G., "Subsidence observations over shallow workings, including pneumatic stowing and rapidly advancing faces". *Min. Engr.* Vol. 120. No. 7, April 1961, pp. 552–569.
2. BEEVERS, C. and WARDELL, K., "Recent research in mining subsidence". *Trans. Instn. Min. Engrs.*, Vol. 114 (1954/5), p. 223.
3. CORDEN, C. A. H. and KING, H. J., "A field study of the development of surface subsidence". *Int. Rock Mech. Min. Sci.*, Vol. 2, Pergamon Press, March 1965.
4. GROND, G. J. A., "Ground movements due to mining". *Colliery Engng.*, Vol. 34 (1957), pp. 157, 197.
5. HALL, M. and ORCHARD, R. J., "Subsidence profile characteristics". *Chart. Surv.*, Vol. 95, No. 8, Feb. 1963, pp. 422–428.
6. KAPP, W. A. and WILLIAMS, R. C., "Extraction of coal in the Sydney Basin from beneath large bodies of water". 1972 Conference of the Australasian Inst. of Mining & Metallurgy.

*7. KING, H. J. and WHETTON, J. T., "Mechanics of mining subsidence". *Colliery Engng.*, Vol. 35 (1957), pp. 247, 285.

*8. KNOTHE, S., "Observations of surface movements and their theoretical interpretation". *Colliery Engng.*, Vol. 36 (1959), p. 24.

9. LEE, A. J., "The effect of faulting on mining subsidence". *The Mining Engineer,* Aug. 1966.
10. MARR, J. E., "A new approach to the estimation of mining subsidence". *Trans. Inst. Min. Engrs.*, Vol. 118, Pt. II, Aug. 1959, p. 692.
11. MARR, J. E., "The effects on surface property by a modified mining method". *Chart. Surv.*, Vol. 97, Jan. 1965.
12. N.C.B. Production Department 1952. Information Bulletin 52/78. "Investigation of mining subsidence phenomena."
13. N.C.B. Production Department 1961, Info. Bulletin 61/231. "Partial extraction as a means of reducing subsidence damage."
14. OGDEN, H. and ORCHARD, R. J., "Ground movements in North Staffordshire", *Trans. Inst. Min. Engrs.*, Jan. 1960, Vol. 119, Part 4, p. 259.

*Denotes papers discussed at the European Congress on Ground Movement held at Leeds in June 1957. They were included in the Proceedings of the Congress.

15. ORCHARD, R. J., "Recent developments in predicting the amplitude of mining subsidence". *Jl. R. Instn. Chart. Surv.*, Vol. 33 (1954), p. 864.

*16. ORCHARD, R. J., "Prediction of the magnitude of surface movements". *Colliery Engng.*, Vol. 34 (1957), p. 455.

17. ORCHARD, R. J., "Partial extraction and subsidence". *Min. Engr.*, No. 43, April 1964.
18. ORCHARD, R. J., "Surface subsidence resulting from alternative treatments of colliery goaf". *Colliery Engng.*, Oct. 1964.
19. ORCHARD, R. J. and ALLEN, W. S., "Ground curvature due to coal mining". *Chart. Surv.*, May 1965.
20. ORCHARD, R. J. and ALLEN, W. S., "Longwall partial extraction systems". *Min. Engr.*, June 1970, pp. 523–535.
21. ORCHARD, R. J., "The control of ground movements in undersea working". *The Mining Engineer,* Vol. 128, Feb. 1969, p. 259.

27. ORCHARD, R. J., "Some aspects of subsidence in the U.K." Australasian Institute of Mining and Metallurgy Symposium, Feb. 1973.

23. PRIEST, A. V. and ORCHARD, R. J., "Recent subsidence research in the Nottinghamshire and Derbyshire coalfield". *Trans. Instn. Min. Engrs.*, Vol. 117 (1957/8), p. 499.
24. ROM, H., "A limit angle system", *Mitteilungen aus dem Markscheiderwesen* 71, 4 pp. 197–199.
25. SPENCER, L. H., "Subsidence research carried out in the Bestwood area of Nottinghamshire'. *Min. Engr.*, Vol. 120, No. 3, Dec. 1960, pp. 201–210.
26. WARDELL, K., "Some observations on the relationship between time and mining subsidence". *Trans. Inst. Min. Engrs.*, Vol. 113 (1953), p. 471 (Discussion p. 799).

*27. WARDELL, K., "The minimisation of surface damage". *Colliery Engng.*, Vol. 34 (1957), p. 361.

28. WARDELL, K. and WEBSTER, N. E., "Surface observations and strata movement underground". *Colliery Engng.*, Vol. 34 (1957), p. 329.
29. WARDELL, K. and EYNON, P., "Structural concept of strata control and mine design". *The Mining Engineer,* Aug. 1968.
30. WHETTON, J. T. and KING, H. J., "The time factor in mining subsidence". International Symposium on Mining Research, Feb. 22-25, 1961, Rolla, Missouri, Vol. 1, Ppr. 31, pp. 1-29.
31. WHETTON, J. T., "A general survey of the ground movement problem". *Colliery Engng.*, Vol. 34 (1957), p. 153.
32. ORCHARD, R. J. and ALLEN W. G. "Time Dependence in Mining Subsidence". Minerals and the Environment, pp. 643-659, *Inst. Min. Met.*, March 1975.

Legal
101. BUSH, T. W., "Statutory requirements in new coal mining areas". *Trans. Instn. Min. Engrs.,* Vol. 114 (1954/5), p. 752.
102. ENEVER, F. A., "Law of support in relation to minerals". Solicitors' Law Stationery Soc., Lond., 1947.
103. GLOVER, C. M. H. and WEBSTER, N. E., "The Law relating to damage by mining subsidence and its effect on mining practice". *Trans. Instn. Min. Engrs.,* Pt. 2, Vol. 118 (1958), p. 75.
104. LANE, W. T. and ROBERTS, J. H., "The principles of subsidence and the law of support in relation to colliery undertakings". Knopf Ltd., Lond., 1929.
105. LISSANT, P. E., "Law of support". River Boards' Ass. Year Book 1956, p. 42.
106. THE COAL MINING (Subsidence) ACT, 1950. H.M.S.O., Lond., 1950.
107. THE COAL MINING (Subsidence) ACT, 1957. H.M.S.O., Lond., 1957.
108. THE COAL MINING (Subsidence) (Notice of Uninhabitability) REGULATIONS, 1957. No. 1404.
109. THE COAL MINING (Subsidence) (Damage Notice) REGULATIONS, 1957. No. 1405.
110. THE COAL MINING (Subsidence) (Notice of Works) REGULATIONS, 1957. No. 1406.
111. THE COAL MINING (Subsidence) (Further Damage) REGULATIONS, 1957. No. 1407.
112. THE COAL MINING (Subsidence) (Assessment of Disablement) REGULATIONS, 1957. No. 2199.
113. THE COAL MINING (Subsidence) (Valuation) REGULATIONS, 1958. No. 1307.
114. THE COAL MINING (Subsidence) (Land Drainage) REGULATIONS, 1958. H.M.S.O., Lond., 1958.

Structures (Including Bridges)
201. DAVIES, J. D., "Circular tanks on ground subject to mining subsidence". *Civil Eng. and Pub. Works Rev.,* Vol. 55, No. 648, July 1960, pp. 918–920.
202. GIBSON, D. E. E., "Buildings without foundations". *J.R.Inst. Brit. Archit.,* Vol. 65 (1957), p. 47.
203. GOODYEAR, H. K., "Construction of an open-hearth melting shop on dangerous ground – Westfalenhuett, Dortmund". *Instn. Civ. Engrs.,* Culman Fellowship Report (unpublished), 1957.
204. LACEY, W. D. and SWAIN, H. T., "Design for mining subsidence". *Archit.* (*Build.*) J., Vol. 126 (1957), p. 557.
205. LACEY, W. D. and SWAIN, H. T., "The development of the Notts. system of construction". *Archit.* (*Build.*) J., Vol. 126 (1957), p. 631.
206. MINISTRY OF WORKS, "Mining subsidence. Effects on small houses". *Nat. Build. Studies Spec. Rep.,* No. 12, H.M.S.O., Lond. 1951.
207. MORSE, C. F. R., "Some mining problems". (M6 Motorway.) *The Chartered Surveyor,* Nov. 1967, p. 236.
208. PRYKE, J. F. S., "Eliminating effects of subsidence". *Colliery Engng.,* Vol. 31 (1954), p. 501.
209. SKEMPTON, A. W. and MACDONALD, D. H., "Allowable settlements of buildings". *Proc. Instn. Civ. Engrs.,* Pt. III, Vol. 5, p. 727 (Dec. 1956).
210. WASILKOWSKI, F., "Ruchy Budowli na terenach gorniczych". *Inzyn i Budownictwo,* Vol. 12 (1956), p. 135. (Movements of structures in mining areas.)
211. WITT, H. P., "Huettenweksbauten in Bergsenkungsgebeit". *Bauingenieur,* Vol. 30 (1955), p. 431. (Construction of a foundry in a mining subsidence area.)

Roads
301. HAKELBERG, F., "Flexible bituminous bases for area of mining subsidence". *Strassen-Asphalt und Tiefbau-Technik,* Vol .9 (1956), p. 657 (in German see Road Abstract No. 247, Mar. 1957).
302. McCALLUM, T., "The maintenance of roads in a mining area"., *J. Instn. Highway Engrs.,* Vol. 2 (Jan. 1952), p. 6.

Utility Services
401. BRITISH STANDARDS INSTITUTION, "B.S.2760: Pitch-impregnated fibre drain and sewer pipes." B.S.I., 1956.
402. CLARKE, N. W. B., "The causes and prevention of fractures in salt glazed ware or other ceramic pipelines". The public Works and Municipal Services Congress 1958.
403. DUMAS, F., "L'amenagement du canal de Lens consecutif aux affraisements miniers". *Travaux,* Vol. 34 (1950), pp. 657, 712, 743. (Works on the Lens Canal as the result of ground movements due to mining.)
404. FARRAN, C. E., "The effect of mining subsidence on land drainage". *J. Instn. Wat. Engrs.,* Wat. & Wat. Engng., Vol. 58 (1954), p. 197.
405. GARNER, J. H., "Report on the effect of mining (coal) subsidence on sewers and sewage disposal works in the West Riding of Yorkshire". The West Riding of Yorkshire Rivers Board, Dec. 1945.
406. LACKINGTON, D. W. and ROBINSON, B., "Articulated service reservoirs in mining subsidence areas". Institution of Water Engineers Journal, Vol. 27 (1973), No. 4, p. 197.
407. McCALLUM, T., "Mineral subsidence and local authority services". *Proc. Instn. Munic. Engrs.,* Vol. 70 (1943/4), p. 411.

408. **MINISTRY OF WORKS**, "Simplified tables of external loads on buried pipelines". *Nat. Build. Studies Spec. Rep.*, No. 32, H.M.S.O., Lond., 1962.
409. **ORCHARD, R. J.**, "The effect of mining subsidence upon public health engineering works". *J. Instn. Publ. Health Engrs.*, Vol. 56 (1957), p. 188.
410. **ORCHARD, R. J.**, "Vitrified clay pipes in areas of mining subsidence". Clay Pipe Development Association 1972.
411. **SERPELL, C. A.**, "The laying of a steel pipeline". *J. Instn. Wat. Engrs.*, Vol. 3 (1949), p. 17.
412. "The Redhill service reservoir No. 2 of the Corporation of Nottingham Water Department." *Wat. & Wat. Engng.*, Vol. 58 (1954), p. 197.
413. "Trackside foundations in subsidence areas." Construction of concrete rafts to carry steel structures for overhead electrification. *Railway Gazette*, April 3, 1959, pp. 390–391.

Strata Control

501. **DOWDELL, R. S.**, "Contribution to discussion on K. Wardell and P. Eynon's Paper: "Structural concept of strata control and mine design". *The Mining Engineer*, Aug. 1968, pp. 649–651.
502. **WILSON, A. H.**, "An hypothesis concerning pillar stability". *The Mining Engineer*, June 1972.
503. N.C.B. Production Department 1972. "Design of mine layouts, with reference to Geological and Geometric Factors." Working Party Report.

Geology

601. **CLARKE, A. M.**, "Some structural, hydrological and safety aspects of recent developments in South-East Durham". *The Mining Engineer*, Dec. 1962.