SESSION 3: MINE SUBSIDENCE AND DAMAGE MITIGATION

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USE OF ANALOG GROUND MEDIUM OF TAYLOR-SCHNEEBELI TYPE FOR MODELLING OF MULTIPARAMETRIC DEFORMATIONS ARISING IN MINING-INFLUENCED AREAS[1]

by
Kazimierz Klosek[2]

Abstract: The results of the author's several years of model research using analog ground medium of Taylor-Schneebeli type to the multiparametric simulations of mining area deformations are presented in the paper. On the basis of the model similarity criteria the complete usefulness was proven of this research approach to the analysis of process of joint action of the mining rock mass and ground structures, underground infrastructures, engineering structures and road surface, also with use of geosynthetics.

Additional Key Words:
- model similarity criteria
- ground analog
- model research
- mining substrate
- kinematics of the disintegrated medium.

Introduction

A number of phenomena happening inside the rock mass and on its surface are impossible to record and to conduct direct surveys of analyse or research on the theoretical grounds. This is due to technical and economical reasons, as well as space-time limitations. Under these circumstances model research, conducted with the use of the equivalent and loose material, is gaining importance. The use of the equivalent materials (sand, gypsum, lime, borax, water mixtures, etc.) requires suitable selection of material constants in order to maintain the similarity laws. As a final result, this research allows to analyse not only the dislocations and deformations, but also forces and stresses occurring in the medium. Although the main domain of its application is ground and exploitation excavation areas [16], it remains beyond the scope of this paper. Models built of media, especially of sand allow conducting research limited to the kinematics of medium reduced to the recording of dislocations and displacements in the medium, caused by initially inflicted limiting displacements. Each of these groups of research has its advantages, but also substantial disadvantages. The paper presents a compromising approach to this problem which connects basic advantages of both main research trends by using the so-called Analog Ground Medium of Taylor-Schneebeli Type [8,15]. The possibility of full optical recording of undisturbed character of deformation of mining area surface and structures on the subsoil is given special consideration.

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THE CRITERIA OF MODELLING SIMILARITY

There are two basic rules of modelling similarity in the model research. The first one is the rule of geometric similarity based on fact that dimensions of modelled space are in proper relation (modelling scale) to the real object. The second is the rule of mechanical similarity, in form of three conditions:

- kinematic model similarity,
- dynamic model similarity,
- rheological (time) model similarity.

The kinematic similarity condition require the relative dislocations of the corresponding points in the model and in nature to be in geometrical interdependence. The comprehensive preliminary research proved [4,6] that corresponding parts of superficial zone of rock mass and of the Taylor-Schneebeli model are the subject to the same linear and formational deformations under identical excitations. For the process of modelling of dislocations of the surface of mining area for dynamic subsiding through, the conformance is shown in Fig.1.

The description of the profile of the subsiding through appearing on the surface as a result of the simulated exploitation show a full correspondence to the Gauss curves (normal distribution curves). This corresponds to the recorded distribution of real effects in the ground [9,10,16]. The other dislocations and deformations, which describe basic deformation parameters of surface, such as depression (W), sloping (T), curvature (K), horizontal dislocation (U) and unitary horizontal deformation (ε) are also fully corresponding. The theoretical shapes of these functions and their interdependence are shown, among others, in [2,3]. The fulfilment of this principal kinematic criterion of similarity guarantees the proper simulation of the behaviour of model surface objects founded on the ground analog T-S (Taylor-Schneebeli) type and related to the planar problems. The condition of dynamical similarity requires the ratio of model and real forces be the same as the ratio of mass and accelerations product, described by the relationship:

$$\frac{F_N}{F_M} = \frac{m_N \cdot a_N}{m_M \cdot a_M} \qquad (1)$$

where: a_N -acceleration of mass m_N

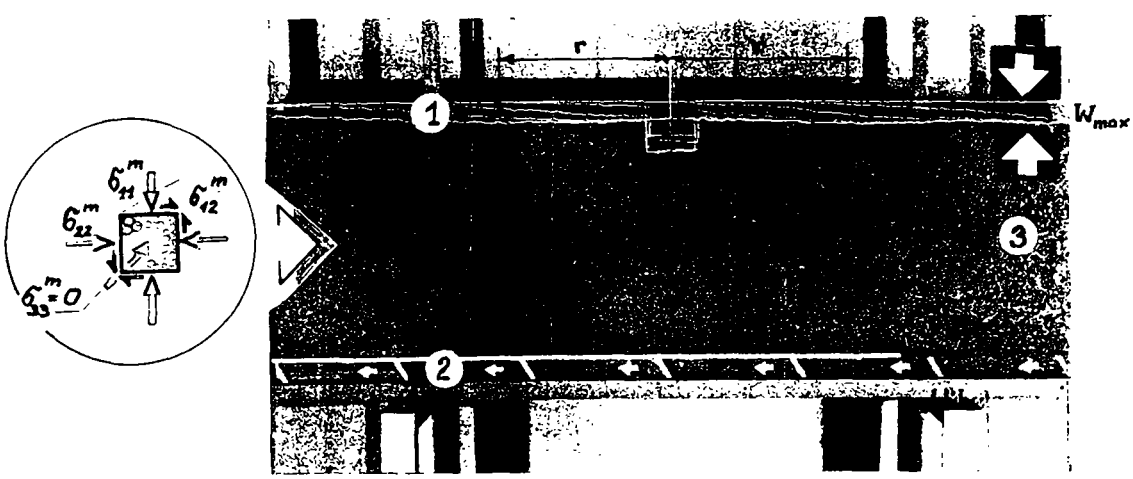


Figure 1. Dynamic subsiding through of a depression
(1) for variable position of exploitation front
(2) in an analog model of rock mass
(3) Taylor-Schneebeli (T-S) type.

under the force F_N in nature.
a_M -acceleration of mass m_M under effect of force F_M in the model.

This condition implies the constant value of the unit mass ratio, the masses being different by the factor b:

$$b = \frac{m_N}{m_M} = const \qquad (1a)$$

In the case of T-S type analog this factor is b=1. The condition of rheological similarity requires that the quotient of deformation velocity in nature and in laboratory was a constant value:

$$\frac{\dot{\varepsilon}_N}{\dot{\varepsilon}_M} = const \qquad (2)$$

Because the tested model was a granular rock medium and its cohesion was zero, this criterion was easily met. The physico-mechanical parameters of the materials used as equivalent should fulfil a number of additional conditions, calculated theoretically basing on the rules of modelling similarities. However, obtaining the wanted values of most of the parameters is very difficult and practically currently not possible. Therefore in practise the rule of functional characteristic is applied. It assumes as a basis of the mapping such mechanical index, which in analysed phenomena plays a vital role. It has been assumed that the principle will concern the preservation of the similarity of the phenomena of ground shearing as a basic strength criterion for this dispersed medium.

THE DESCRIPTION OF THE ANALOG GROUND MEDIUM.

The T-S type analog medium consist of a mixture of cylindrical rollers of two diameters: $\varnothing$4mm and $\varnothing$6mm, L=47mm long and weight by volume of γ=1.4G/cm^3. The side surfaces of the rollers are fine finished, ensuring the full stability of the pile, which then not need head walls. The assumed ground model ensured exactly flat state of strains and deformations. The Schneebeli's experiments [15] have shown that the pile of circular rollers of different diameters fulfil the condition of Coulomb's limiting equilibrium [12,13] . The necessary condition for obtaining the macroscopic and isotropic structure of the medium is the use of a set of rollers of different diameters [1,11]. Deformations of the single-fraction medium have discrete character, caused by structural anisotropy of the homogeneous pile. The anisotropy of the structure leads to the law of anisotropic elasticity of the analog making the research difficult and its results hard to interpret. The example of that is presented in interpret [7]. The conditions of model similarity of the mixed task require the fulfilment of adequate dependencies between strains, weights by volume, strength and geometric parameters of the model and the real object. This can be described as:

$$\alpha_\sigma = \alpha_\gamma \cdot \alpha_L \cong 1$$
$$\alpha_\gamma = \frac{\alpha_\sigma}{\alpha_L} \qquad (3)$$
$$\alpha_\phi \cong 1$$

where: $\alpha_{\sigma,L,\gamma,\varnothing}$:the scale of strains, lengths, weights by volume, internal friction angle. After substitutions it is possible to describe the state of strains in nature on the basis of the ground analog parameters:

$$\frac{\alpha_\sigma}{\alpha_L}\left(\frac{\partial\sigma_{22}}{\partial x_2} + \frac{\partial\sigma_{12}}{\partial x_1}\right)m = 0$$
$$\frac{\alpha_\sigma}{\alpha_L}\left(\frac{\partial\sigma_{11}}{\partial x_1} + \frac{\partial\sigma_{12}}{\partial x_2}\right)m = \alpha_\gamma \cdot \gamma_m \qquad (4)$$
$$\frac{\alpha_\sigma}{\alpha_L^2}\nabla^2(\sigma_{11} + \sigma_{22})m = 0$$
$$(\sigma_{11} - \sigma_{22})m = (\sigma_{11} + \sigma_{22})m \cdot \alpha_\varphi \cdot \sin\varphi_m$$

The parameters with index „m" correspond to the model. The first two equations describe the limiting equilibrium state, the next concern the condition of inseparability and the Coulomb-Mohr plasticity criterion. The homomorphic model of T-S type ground does not meet exactly all of the similarity model conditions. The research results are burdened with certain errors resulting from non-fulfilled criterion of that weight strain scale and geometric similarity of grains criterion. However it does not influence significantly the results, especially of the preliminary, qualitative estimation of the investigated effects of interrelation of structure and mining deformed ground.

USE OF THE ANALOG GROUND FOR THE MODELLING OF THE INFLUENCE OF UNDERGROUND MINING ON SELECTED OBJECTS ON SURFACE

The research carried out by the author using the T-S type analog medium preceded the complementary and verifying experiments realised with the use of sand in a specially constructed box apparatus (Fig. 2, [14]).

This apparatus made possible the physical realisation of the basic index of mining deformation of a terrain. The index were the unitary horizontal deformations of compacting and slackening character ε.

The direct recording of the kinematic effects and verifying the computational models was in this case impossible. Only the T-S analog placed on a specially constructed research stand shown in Fig. 3 makes the recording of these effects fully possible. In the model shown in Fig.1 the displacement of the working face of underground exploitation is simulated by the stages of sliding of a batten in the bottom part of the T-S pile. The contrasting (white) rollers of the T-S pile make possible clear dynamic recording of the freely chosen points of the medium. The research stand creates the possibility of simulation of all the deformation parameters in the near-surface zone as well as in the rock mass.

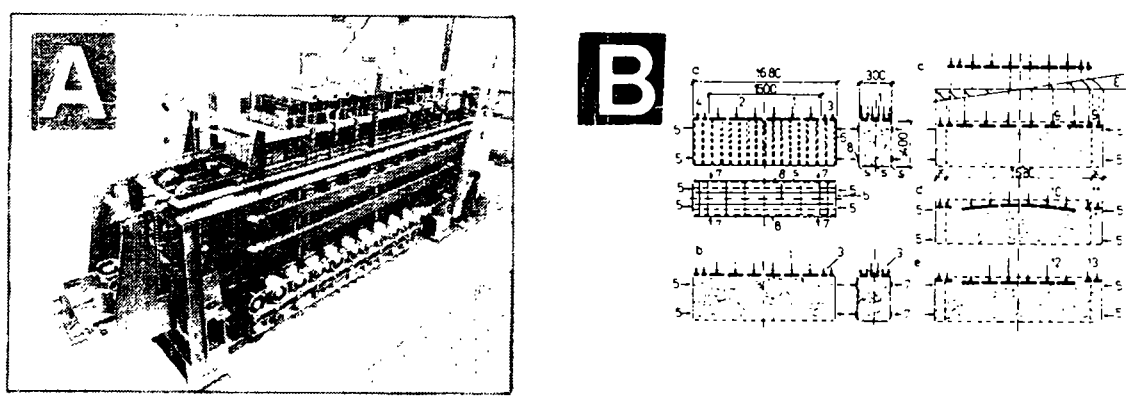


Figure 2. Apparatus for modelling of horizontal mining deformations in loose granular medium
A) General view
B) Principle of apparatus operation taking under consideration an external load.
Denotations: 1-2: vertical load, 3-4: horizontal load, 5-8: head-walls and side-walls, 9: flexible load, 10: stiff load.

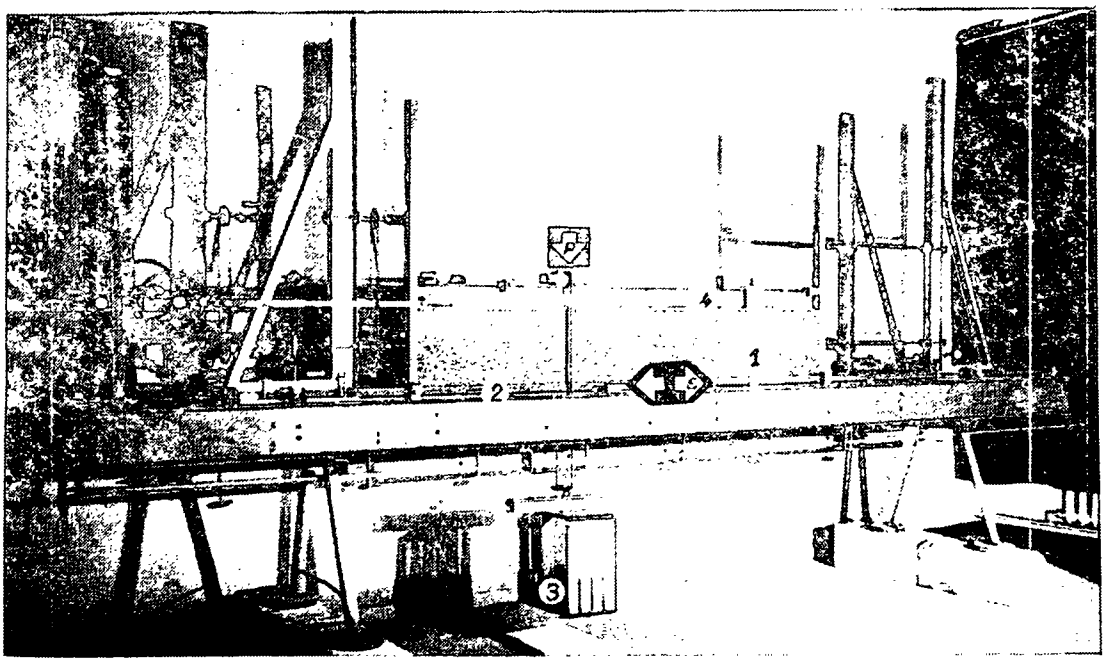


Figure 3. Test rig for modelling of the mining deformations in T-S type analog medium.
Denotations: 1: T-S analog, 2: rubber band, 3: external load, 4: model of the foundation.

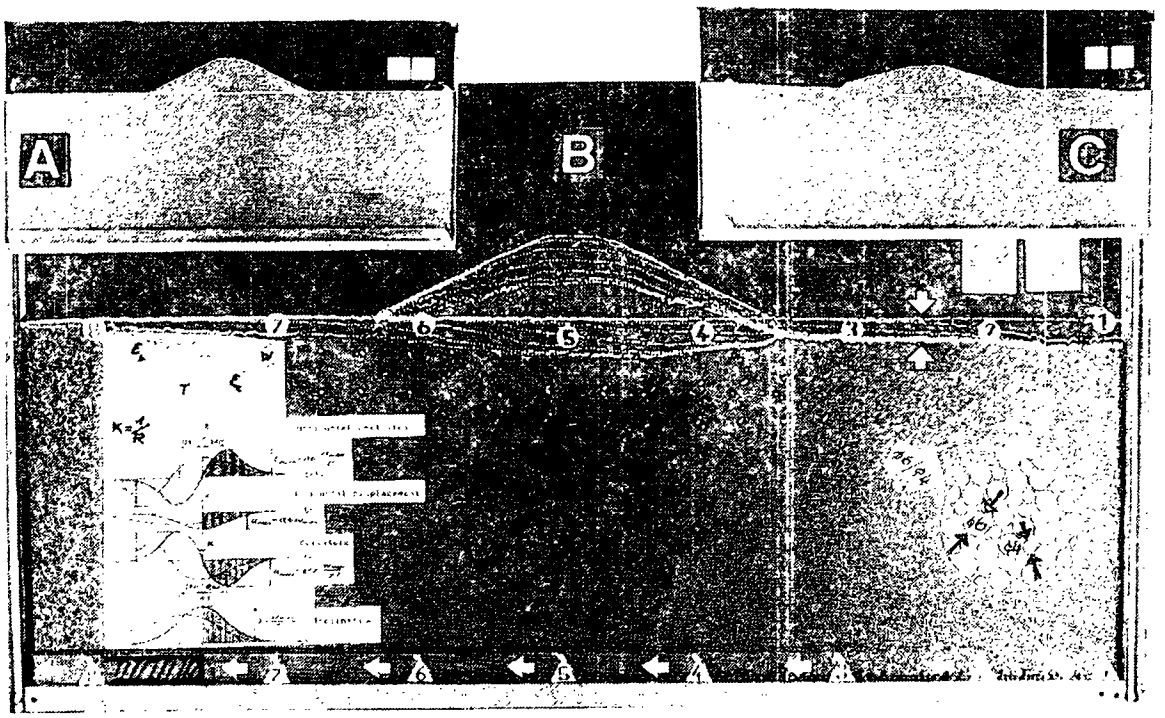


Figure 4. Modelling of the mining deformations within an embankment earth construction
A) Initial state
B) Passing of dynamic subsiding trough under the object, perpendicularly to its longitudinal axis,
C) Final state, after the excitations have ceased.

The research stand shown in Fig.3 makes possible the simulation of the main index of the mining deformation, i.e. the unitary horizontal deformation ε of the compacting and slackening character. The container filled with T-S pile has in its bottom part a rubber batten stiffly attached to the properly bedded side walls. The central drive makes possible exerting a uniform tension on the rubber belt and eventually creates a horizontal simulation of homogeneous deformations ε, typical for a mining area. This apparatus additionally makes possible the simulation of the vertical curvature of the bed K, both concave and convex.

The most important and fully successful tests of application of T-S analog for geotechnic applications in mining-influenced areas include:

- modelling of the deformations arising in mining-influenced areas caused by underground mine exploitation together with the global assessment of the influence of the dynamic subsiding through, on the surface objects such as earth structures (embankments, cross-cuts - Fig. 4), tunnel constructions (Fig. 1), supports, etc.
- estimation of the influence of slackening horizontal deformations of the building foundations in wall footing or grillage systems (Fig. 5a), together with the assessment of the influence subgrade reinforcement on the reduction of additional axial forces in the wall footing - Fig. 5b.
- assessment of the influence of compacting horizontal deformations on the surface retaining constructions - Fig. 6a, b, and the constructions hollowed in the ground - Fig. 6c.
- the determination of the character of interrelation of the road surface and the permanent way with the mining-influenced area; a detailed investigation was carried out to find the sources of additional forces in the rails of the permanent way resulting from thermal effects and braking vehicles - Fig.7. The studies were also aimed at finding the sources of slackening deformations (Fig. 8) of the mining-influenced subgrade and determining the reinforcement of load capacity of track grillage subgrade by geotextile undersleeper containers and horizontal membrane - Fig. 9.

The detailed analysis of the results of this research is impossible to be presented here, because of the space limitations of this paper. More details concerning this subject may be found in [3,4,5,6,11] and other works.

The results of the kinematic research show the essential differentiation of the character of interrelation of structures with the mining-influenced areas opposed to the case of a non-mining area. As a consequence a necessity arises of a separate theoretical approach to this class of complex, interdisciplinary computational problems encompassing the issues of rock-mass mechanics, mining geotechnic and the construction theory. Without applying the described research techniques, it would be extremely difficult, if not impossible, to obtain the described results.

CONCLUSIONS

The application of the Taylor-Schneebeli type analog ground in a model research of mining substrate proved a number of essential advantages in comparison with the traditional models using equivalent materials and natural granular substances. The advantages are:

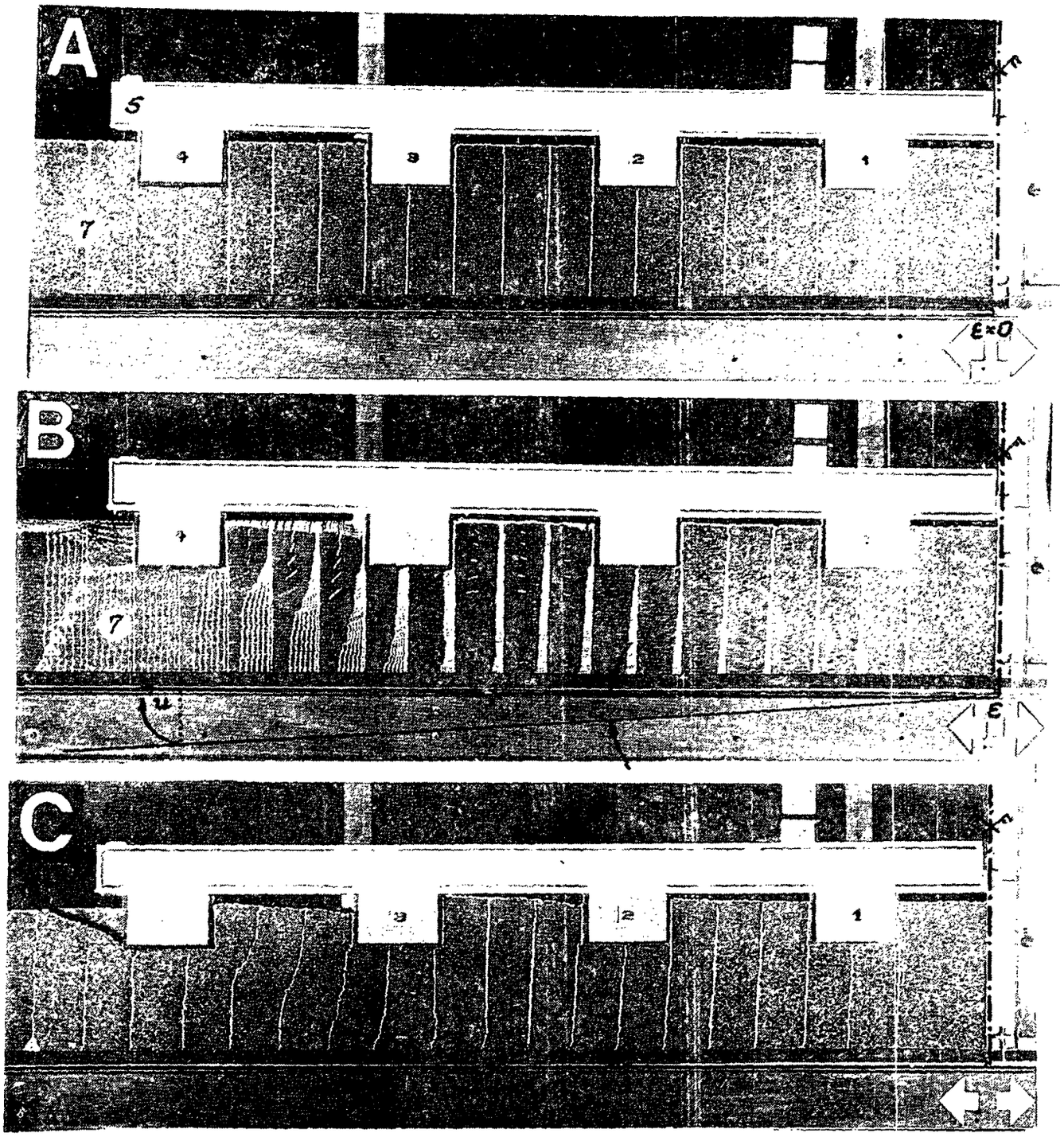


Figure 5. Modelling of the interrelation of the transversal foundation benches under the conditions of horizontal slackening of subgrade without reinforcement
A) Initial state
B) Growth of slackening deformations in mining subgrade
C) Final state, after the excitations have ceased
Denotations: 1-4: transversal benches, 5: longitudinal bench

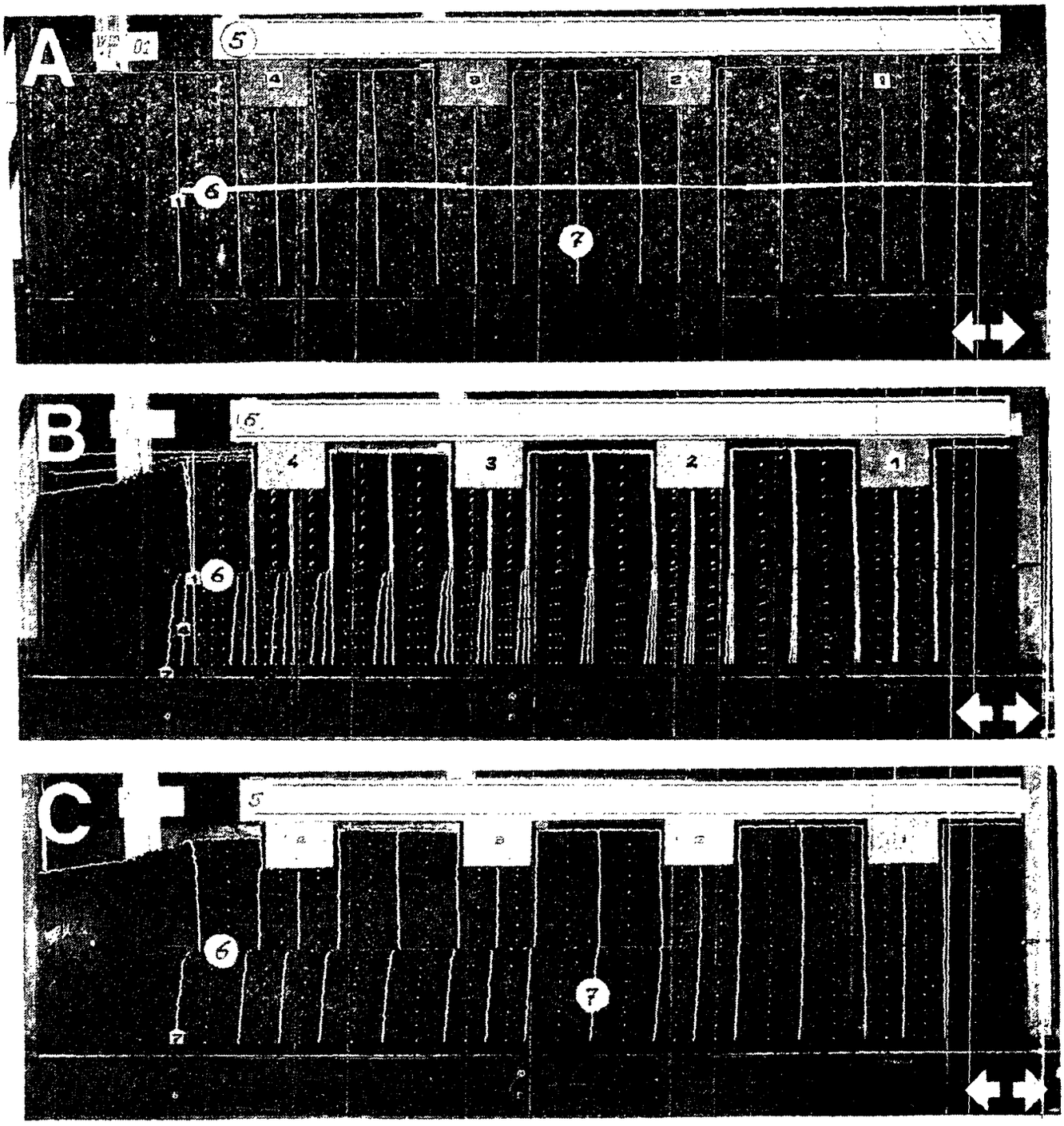
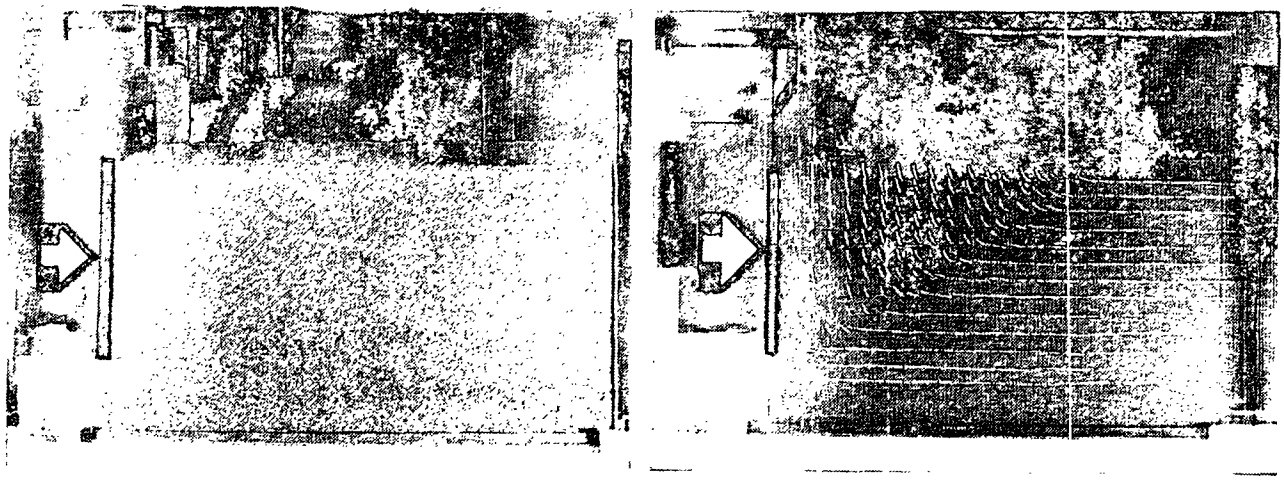


Figure 6. Modelling mining influences on transversal foundation benches under the conditions of slackening deformations of subgrade with geosynthetics strengthening
A) Initial state
B) Growth of slackening deformations in mining subgrade
C) Final state of deformation after the influences ceased
Denotations: 1-4 transversal benches, 5: longitudinal bench, 6: subgrade with geosynthetic strengthening, 7: contrasting elements of T-S pile.



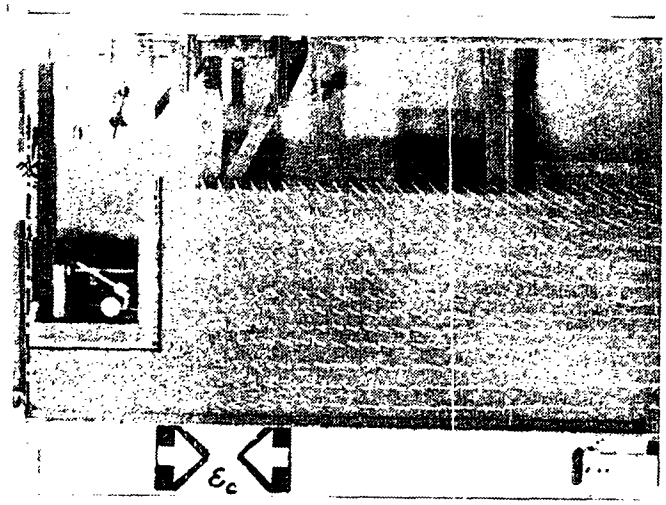


Figure 7. Influence of horizontal thickening deformations on the retaining constructions
A) Passive ground pressure without mining influences (shift of the construction towards the ground-left side, relative shift of the ground in relation to the construction-right side)
B) Pressure of the ground under the conditions of thickening deformations of mining subgrade with kinematic analysis (a-horizontal component, b-vertical component, c-resultant)

- comparatively simple construction of test stands of universal use, allowing the recording of the deformation field and the components of the stress state in complex flat state of deformation, recording being complete and undisturbed by the boundary conditions
- preservation of the main criteria of modelling similarity, creating the basis for the further qualitative and quantitative analyses. The adoption of the obtained results to the engineering practice is possible thanks the values of the scale coefficient α_i being close to those corresponding to the natural conditions.
- relatively short duration of the research and test programme, low cost of labour and apparatus and the consistently good repeatability of the results
- the characteristics of the phenomena of the dislocations and deformations in the rock mass and the sub-surface layer fully correspond to the qualitative representation of the same effects in the experimental research in T-S type analog medium. The latter statement concerns also the process of the interrelation with the surface objects located on the mining-influenced areas, also under different conditions of mining, for instance in case of strip-mining;
- the results of the research on the analog medium have mainly introductory and qualitative character. Therefore, the attempts to transfer directly the qualitative criteria to any real object should be considered with due care and attention.

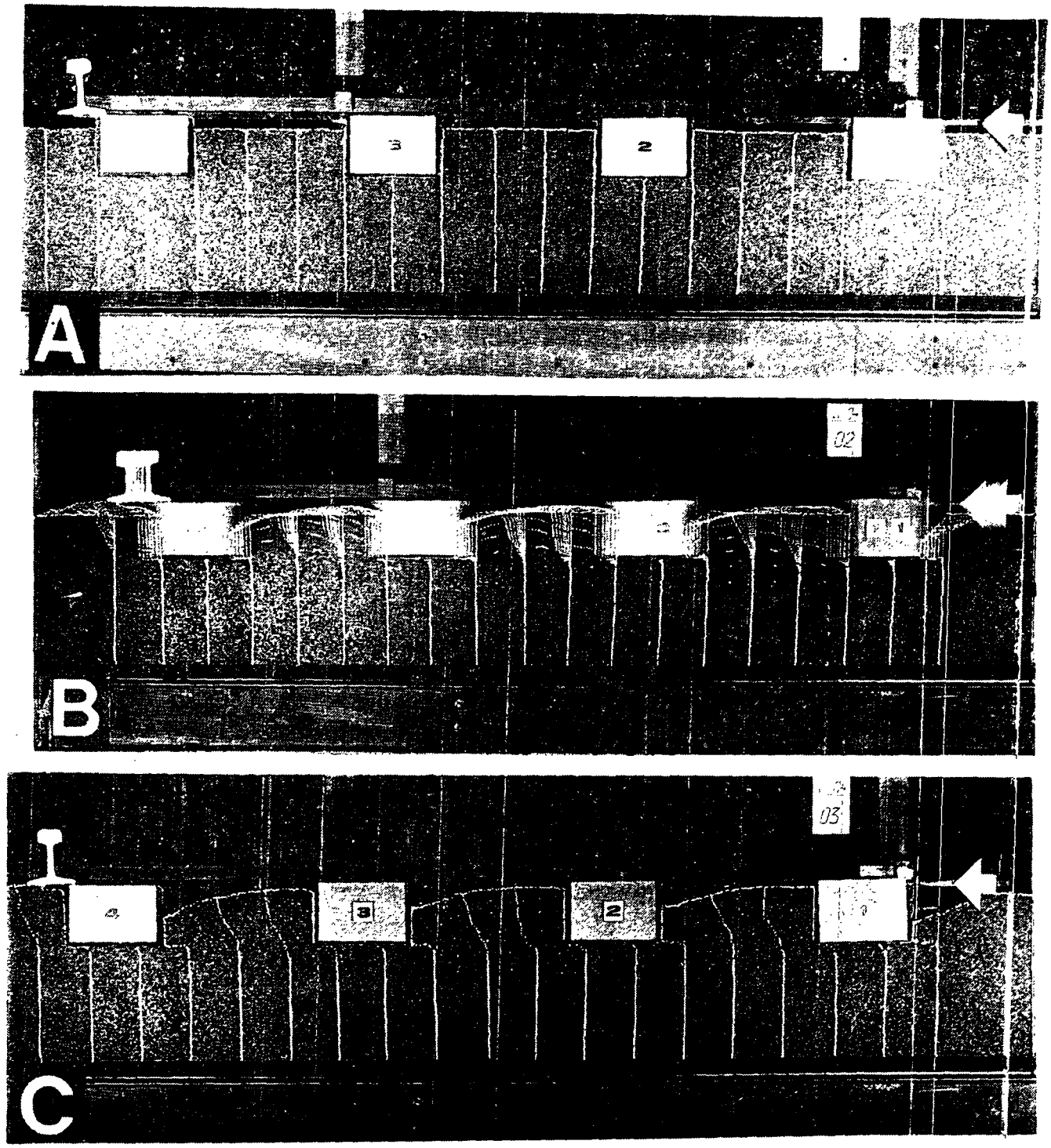


Figure 8. Modelling of kinematic phenomena related to the rise of axial forces in railroad track on the non-mining areas.
A-Initial state
B-Growth of the axial forces in permanent way (temperature, retarding of vehicles etc.)
C-Final displacement state

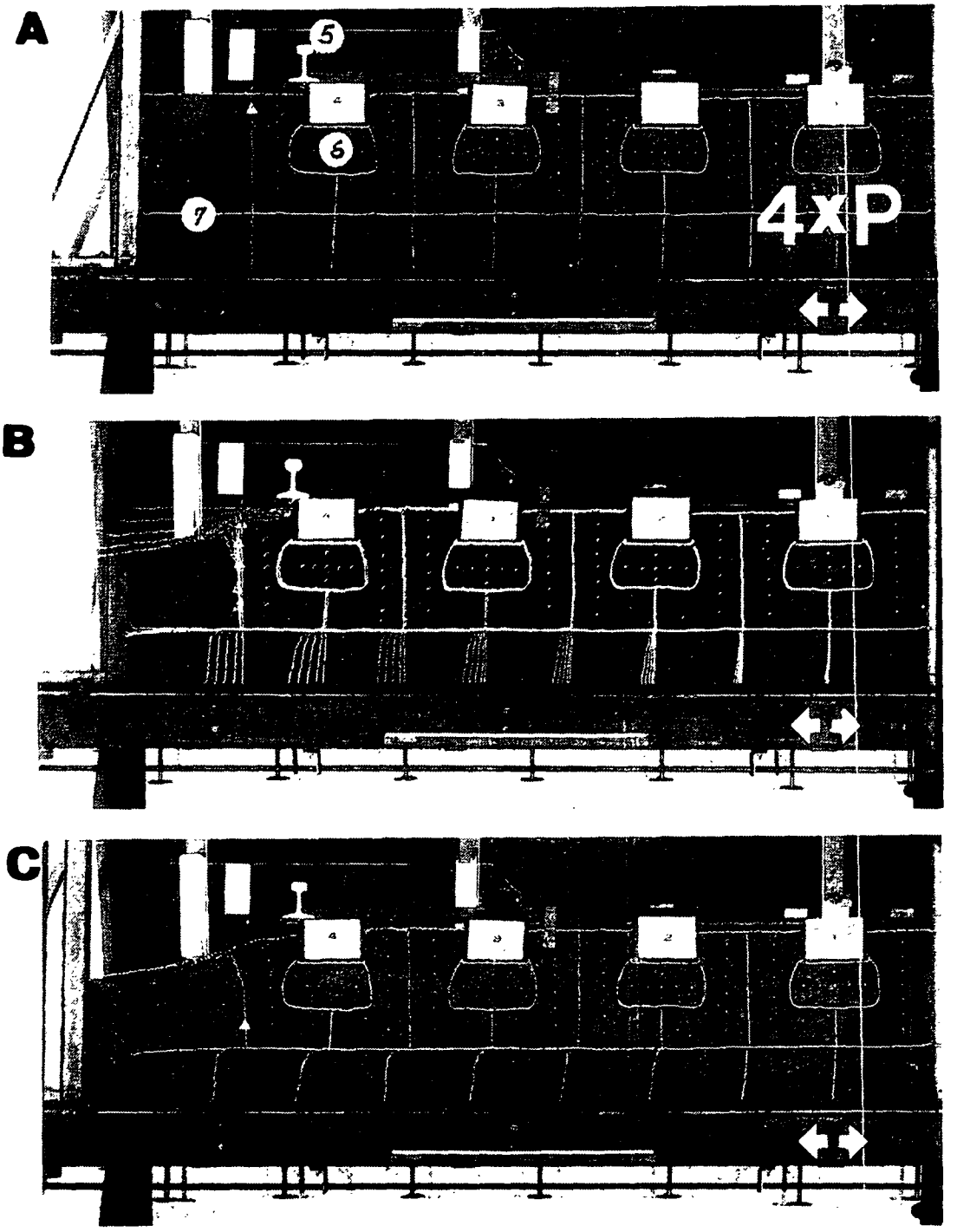


Figure 9. Modelling of the influence of slackening mining deformations on the permanent way strengthened with undersleeper geotextile containers and horizontal membrane.
A) Initial state
B) Growth of slackening deformations in mining subgrade
C) Final deformation state, after the influences ceased
Denotations: 1-4: sleepers, 5: rail, 6: undersleeper geotextile container, 7: horizontal geotextile membrane.

LITERATURE CITED

[1] Drescher A., Josselin de Jong. 1972. Photoelastic verification of a mechanical model for the flow of a granular material. Journal of Mechanics and Physics of Solids, Vol. 20

[2] Klosek K. 1994. Prevention of damage to highways and railroads in mining areas. Int. Land Recl. and Mine Drainage Conf., Pittsburgh, PA, 101-110

[3] Klosek K., 1988. Effect of mining foundation strains on a co-operation of the subgrade with a track structure. Proc. of Techn. Univ. of Silesia, Vol. 66, Gliwice, Poland, p.160.

[4] Klosek K. 1984. Use of analog ground medium of Taylor-Schneebeli type for modelling of the mining exploitation influences on superficial civil engineering structures. Prace Komisji Naukowych Polskiej Akademii Nauk, Vol. 9/1984, Katowice, Poland

[5] Klosek K., Graftiaux H.F., 1982. Quelques problemes concernant la construction d'ouvrages d'art en terrain d'exploitation miniere. Indistrie Minierale-Les Techniques - France, Aout-Sept; p.349-359

[6] Klosek K., 1979, The effects of a mine field subsoil deformations on construction's support. Ph.D. Thesis, Warsaw Univ., Poland

[7] Kratzch H., 1983. Mining subsidence engineering. Springer-Verlag, Berlin-Heidelberg-New York, p.535

[8] Lambe T.W., Witman R.V., 1969. Soil Mechanics. New York-London-Toronto, p.620.

[9] Litwiniszyn J., 1965. Some remarks on the stochastic theory of ground movement. Feldsmechanik u. Ingenieurgeologie. J. Int. Soc. Rock Mech. Vol.3 No 2.

[10] Litwiniszyn J. 1957. Application of the equation of stochastic processes to mechanics of granular bodies. Int. Congr. of Applied Mechanics; Bruxelles.

[11] Litwinowicz L, Klosek K., Kolacz Z., 1978. Application of Schneebeli medium for model research of the influence of mining substrate deformation on the processes of mobilisation of ground pressure. Papers of the 5th National Geotechnics Conference, Katowice, Poland

[12] Oda M. 1974. A mechanical and statistical model of granular material. Soil and foundations. Vol. 14 No. 1

[13] Oda M., Konischi J. 1974. Microscopic deformation mechanism of granular material in simple shear. Soil and Foundations. Vol. 14 No. 4

[14] Rosikon A., 1979. Transportation Engineering on mining damages areas. WKiL, Warszawa, p.315

[15] Schneebeli G., 1975. Une analogie mecanique pour l'etude de la stabilite des ouvrages en terrein a deux dimensions. Proc. IV ICOSMEF, Vol. II, London.

RELEVELING AND BEHAVIOR OF STRAP RETROFITTED DAMAGED TEST FOUNDATIONS EXPOSED TO MINE SUBSIDENCE[1]

by Gennaro G. Marino, Ph.D., P.E.[2]

Abstract. Test foundation walls were constructed in an area of planned subsidence. These crawl space-sized block bearing walls were located in the tension zone of a longwall panel. The test walls were 12.2 m (40 ft) long and were vertically loaded on top with soil bins to simulate the house weight. As the longwall proceeded past these test foundations subsidence movements damaged the test structures. Using a steel strap retrofit and applying a cementitious surface coating, the foundations were structurally and aesthetically repaired. The repaired test foundations underwent significant subsequent subsidence as an adjacent longwall was mined beneath. The response of the repaired foundation is summarized. The second part of the paper describes releveling another set of the damaged test foundations after the first subsidence event by using an innovative procedure. First the straps were applied to the block bearing walls and then with wall jacks the top of the walls were successfully made level. This releveling procedure is outlined with the results.

Additional Key Words: mine subsidence, foundation response, foundation leveling, foundation repair.

Introduction

In the first part of this paper, a field test of a newly developed technique for retrofitting residential foundation walls damaged by mine subsidence is described and discussed. There were three main objectives of the field test: 1. to evaluate the constructability of the new repair technique, 2. to investigate the performance of the retrofit under subsidence induced bending and climatic conditions, and 3. to relate the aesthetic and structural performance of the retrofit to factors such as subgrade bearing capacity, soil stiffness, angular distortion, and tilt. All results from all three objectives are discussed herein except for the influence of the subgrade characteristics on foundation behavior. Discussion of this latter topic would make this paper too lengthy.

The research described in the second part of this paper involved the field testing of an advanced tilt correction procedure that was proposed for houses tilted from mine subsidence (Marino, 1987). The objective of the field test was to determine: 1. the feasibility, 2. the constructability, 3. releveling capability, and 4. the level of difficulty of the proposed procedure.

Complete data summaries and analyses of the work reported herein can be found in Marino et al, 1992A; Marino et al, 1992 C; and Marino and Gamble, 1993.

Background and Site Conditions

The test site was located at a rural location just outside of West Frankfort, Illinois. At the test site a series of foundation walls were built by the University of Tennessee (Bennett, et al, 1992). Originally, the test walls were built to evaluate the effects of different foundation characteristics and subgrade materials on the structural response of similar foundations to mine subsidence movements.

The test walls were built with ordinary concrete blocks. These walls were not grouted nor reinforced. Block dimensions were 8" x 8" x 16". The

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walls were four courses high and 12.2 meters (40 ft) long. All of the walls ran side by side. Each wall was paired with another and supported soil-filled wooden bins which were used to simulate the load of a house (see Figure 1).

One pair of test walls was used for the retrofit test. They were designated as walls T5 and T6. Although the walls were of the same construction, different footing designs were used. The eastern-most foundation wall, T5, was founded on a footing constructed of normal concrete with a 0.004" plastic sheeting around it in an attempt to reduce friction between it and the sub-grade. Foundation wall T6 was on a footing constructed of concrete with steel fibers in it. Figure 1 shows cross-section diagrams of the two footings.

The soil subgrade was a mottled brownish gray and orangish brown silty clay, CL, with trace to little fine sand. Hand penetrometer readings for the soil were consistently above 2.0 kg/cm^2 (4 tsf), so the silty clay is classified as being hard. Moisture content of the material was between 16 and 20 percent. Plastic indices for the clay ranged from about 24 to 36% with the natural moisture content from 5 to 10% greater than the plastic limit.

The walls were erected near two future longwall mine panels. Figure 2 shows the location of the north-south test walls in relation to the relevant longwall panels. The walls were built during the spring of 1990 before any subsidence took place. During mid-May of 1990 the face of the first longwall panel was beneath the test site. Significant damage in the walls was induced by the subsidence movement over that panel. Figure 3 shows the main cracking in both T5 and T6 as of May, 1991. The walls were repaired using the new strap technique between 5/13/91 and 5/17/91. The face of the second panel was adjacent to the test foundations in early to mid-June 1991.

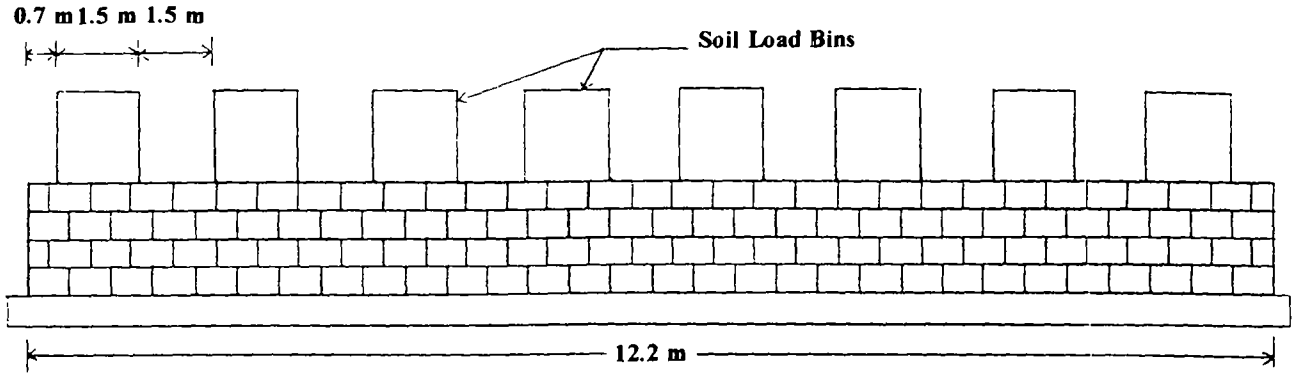
The tilt correction methodology was tested on the middle set of foundation walls (Numbers T3 and T4). These block walls were of the same construction as T5 and T6. The releveling of these walls was performed after the north panel was mined and the foundations were tilted 20-23 cm (8-9 in.) to the south with angular distortions of about 1/200.

Field Test of Foundation Retrofit Procedure

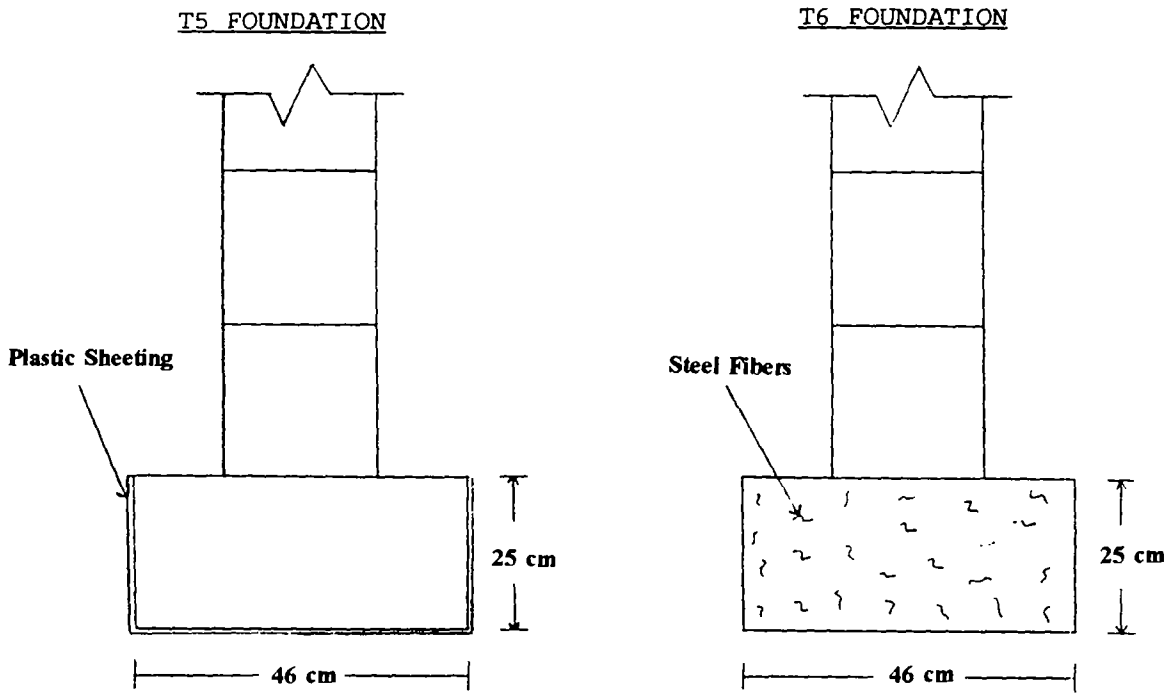
Overview of Wall Retrofit

The strap retrofit system used in the field test is illustrated in Figure 4. For test foundation T5, the strap retrofit was applied to both the east and west sides, and for T6 a strap was applied only to the east side. The total cross-sectional area of the steel was the same, however, for both walls. The purpose of attaching straps to both faces of wall T5, and only one face of wall T6 was to evaluate single strap performance and compare it to the performance of the wall with reinforcing straps on both faces. It was important to evaluate single strap performance, because if the larger single reinforcing straps perform adequately, the necessary reinforcement can be applied entirely to the interior wall faces. This procedure modification would be more cost efficient than applying straps to both sides especially where excavation would be required. Also note as shown on Figure 4, the amount of steel applied to the wall surfaces increased towards the middle of the walls. This increase in steel was calculated based on the increased bending moments estimated from the expected hogging-like subsidence movement resulting from when the southern panel is mined.

There were five main steps taken in retrofitting the damaged test foundations. First, the steel required for the repair was determined. Second, the first stage of repair was done and the walls were cleaned. This included filling open wall cracks in the expected zone of compression with mortar. By doing this large wall deflections due to crack closure were prevented. Shear strength was restored to the walls by grouting reinforcement into the cells of the concrete blocks in areas exhibiting diagonal shear cracks. Cleaning the walls was then completed, thus enabling the epoxy and fiber-cement to adhere to it. Third, the steel reinforcing straps were prepared for attachment. Fourth, the external reinforcing straps were attached to the walls to increase wall bending capacity. Fifth, a surface finish of fiber-cement was applied to the walls to improve the retrofit

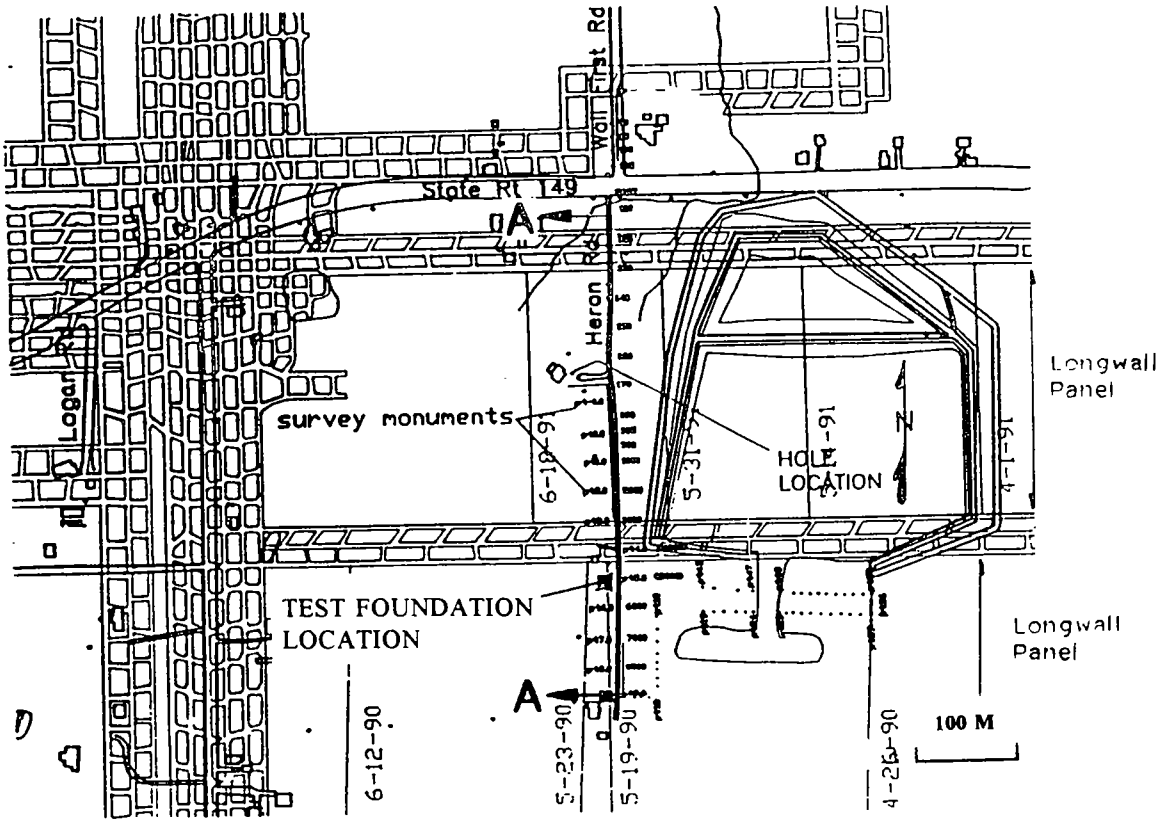


ELEVATION

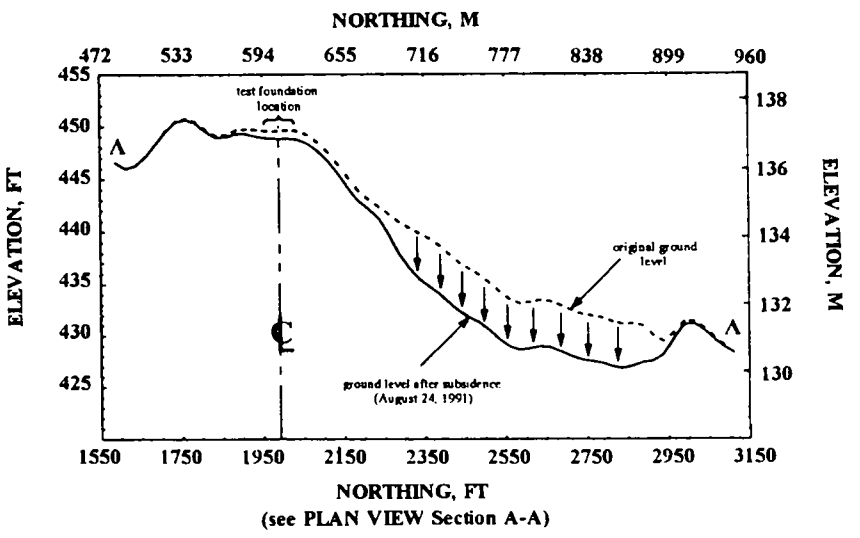


CROSS-SECTION

FIGURE 1 CONFIGURATION OF TEST WALLS



PLAN



PROFILE OF SECTION A–A

FIGURE 2 POSITION OF THE TEST FOUNDATIONS RELATIVE TO MINING ACTIVITY AND SUBSIDENCE PROFILE

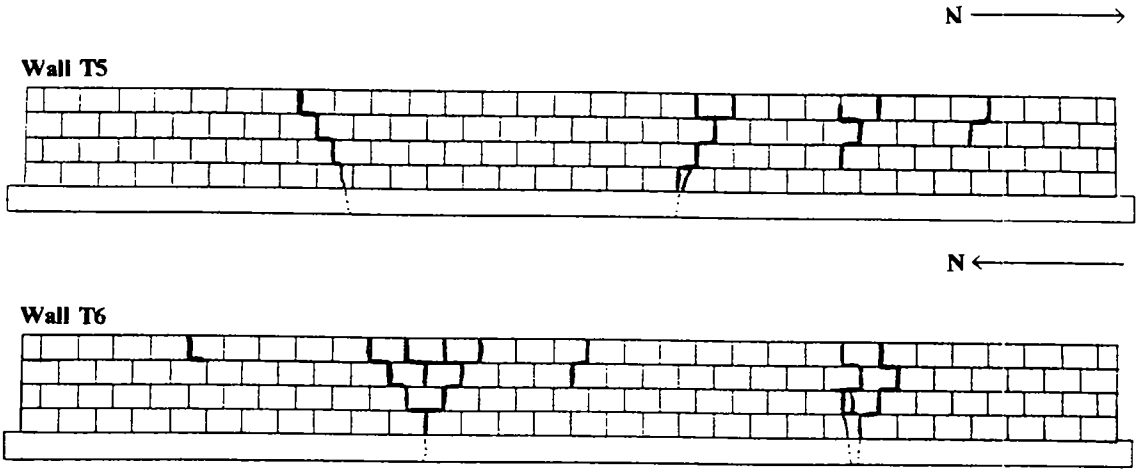


FIGURE 3 MAIN CRACKING IN FOUNDATION T5 AND T6 (MAY, 1991)

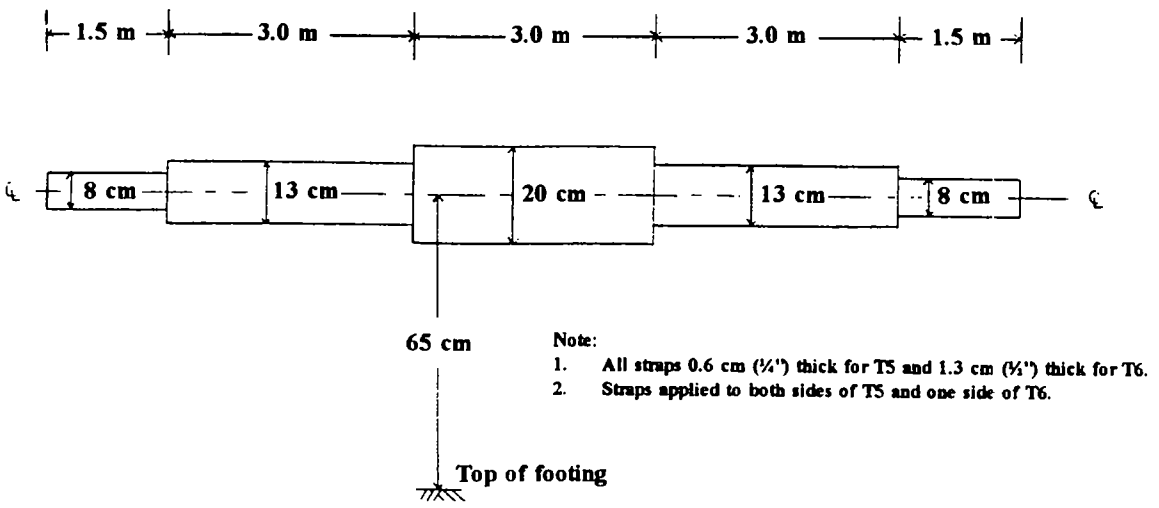


FIGURE 4 STEEL REINFORCING STRAP DIMENSIONS FOR WALLS T5 AND T6

appearance. A detailed description of the application of this strap retrofit process is given in Marino, 1992.

Instrumentation

A set of measurements was obtained periodically for one and one-half years after strap installation. This included: level survey of points on foundation walls, on wall footings, and on soil monuments along the foundations; continuous strain gauge measurements on the foundation walls along the straps and near the base of the retrofitted sides of the foundation walls; and relative vertical and horizontal displacements between the foundation wall and footing.

Consistent measurement procedures were maintained during the monitoring period. Strain measurements along the wall and footing and relative displacements between the same were performed using a Whittemore strain gauge. Details on the instrumentation and measurement specifications are given in Marino et al, 1992A and Marino and Gamble, 1993.

Response of the Retrofitted Foundation

Foundation Settlement

Based on the survey data taken during subsidence, the wall and footing settlement profiles for each retrofitted foundation are similar (even though steel was placed on only one side of T6 compared to both sides of T5). Furthermore, these profiles show the foundation walls and footings undergoing a more definitive hogging curvature with time. To demonstrate these points, foundation settlement measurements for T5 relative to the positions on 14 May 1991 are shown in Figures 5 and 6. Note only selective profiles are shown for clarity.

Using the ground and foundation profiles, the tilt and average angular distortion were calculated for three dates since the repairs were made. These calculated angular tilt and distortion values are compared in Tables 1 and 2 respectively. As can be seen in Table 1, the walls and footings for both foundations T5 and T6 have essentially tilted (or rotated) with the ground, reaching values of 0.48%. On the other hand, the angular distortion experienced by the repaired walls was typically less than that experienced by the footing and about one-half that of the soil at the severest levels (1/1200). Also, it is interesting to note that Wall T6, which has been strap retrofitted only on one side, appears to have bent less than Wall T5 with straps attached to both sides (i.e. 1/3,125 for T6 compared to 1/2,174 for T5).

Foundation Straining

Particle displacement diagrams along the straps and near the bottom of the retrofitted sides of foundation T5 and T6 were prepared and all showed similar trends. As examples of this data the particle displacement diagrams for the east face of foundation T5 at strap level and near the base of the foundation wall are depicted in Figure 7. The particle displacements were determined by adding the displacements measured between each set of Whittemore strain points starting from the north end of the foundation. Therefore, each point plotted, such as on Figure 7, shows the total of all point to point displacements measured south of Whittemore point in question.

As can be seen in Figure 7 little overall change in strain (as determined by the slope of the line) occurred between November 26, 1991 to November 29, 1992. Changes in the average longitudinal strain over this period of time ranged from about -2.0×10^{-5} to 3.0×10^{-5}. Even peak average strains during the entire test period were small and were 2.4×10^{-4} in tension and -4.8×10^{-5} in compression. Even though these strain values may appear low compared to the increase in the angular distortion (or deflection) of the walls (see Table 2), they are actually greater than values that would be predicted by the classical curvature-bending strain relationship (Popov, 1968) which is:

$$c = \epsilon / y$$

where: c = beam curvature
ϵ = longitudinal bending strain
y = vertical distance from neutral axis (assumed at bottom of footing)

The final strains measured in the wall are all tension, which is compatible with the wall being in the

tension zone of the second subsidence event. However, the tensile strains at the bottom of the wall were larger than at the steel straps, 26 in. above the bottom of the wall on all three sides measured. This strain distribution corresponds to a case of sagging curvature in tension, while the measured vertical movements of the walls clearly indicate small hogging curvatures. The reasons for this inconsistency are not clear. A possibility for this inconsistency may be measurement error as the level of straining was low.

No comparison of foundation strain could be made with ground strain based on the soil monuments as these measurements were found to be not accurate.

Overall Performance

The November 1992 readings indicate the amount of ground distortion and tilt experienced by Foundations T5 and T6 since the repair was sufficient to cause damage to conventional residential construction. The most recent ground movements (shown in Tables 1 and 2) have been plotted on Figure 8. In Figure 8 are damage criteria for sag subsidence for houses resting on conventional block crawl-space foundations. This damage relationship assumes that ground distortion is empirically related to ground horizontal strain, i.e., as a house is exposed to some angular distortion level it is also exposed to a certain amount of ground strain. Therefore, ground strain can be considered to be inherently included in the damage plot shown in Figure 8.

As can be seen in Figure 8 a house exposed to the level of ground movement by the test foundations will likely sustain 10 to 40 percent Relative Repair Costs (RRC). RRC is defined as the percentage the necessary repair cost relative to the house replacement cost. This range of RRC values will usually include minor damage to the foundation and superstructure (see Table 3). At the most, possible replacement of accessory foundation element(s) or partial replacement of the brick veneer would be necessary. As noted above, the bending of the walls was about half that of the ground. This level of distortion reduces the estimated damage to Level I (see Figure 8). In other words, no damage to only RRC of 10 percent would be expected. Clearly then, based on this performance, these retrofitted test foundations are performing better than conventional construction since essentially only one minor discontinuous crack has appeared.

This crack is located about 9.40 m (370 in.) from the north end of Wall T6. The width of this crack has increased slightly over the last year of monitoring. This crack was more serious than the shrinkage cracks aesthetically, but is discontinuous and could probably be adequately covered with elastomeric paint if it were a problem. It should be noted, however, that this crack may close if the overall hogging curvatures in the wall become great enough with time as the crack originates at the base of the wall.

Aesthetic performance can be defined as the ability of the fiber-cement (Quikwall) coating to maintain an aesthetic appeal. Deterioration of the aesthetics of the surface cement is considered to occur when the propagation of cracks are visible to the naked eye at a distance of a few feet. The cracks may be load induced or the result of temperature, application, and curing.

Shrinkage cracks form when the fiber-cement coat contracts from drying or lower temperatures. Also, crazing or map-type cracks can occur depending upon the way the cement is applied. This type of cracking was found on all three retrofit faces and has not changed with time. These cracks could only be found through close observation. Therefore, none of the shrinkage cracking that occurred was considered to be an aesthetic failure. Figure 9 contains a photograph of the face of the fiber-cement on November 29, 1992.

Field Test of Tilt Correction Procedure

General Procedure for Releveling

For a tilted and damaged foundation in the tension zone, straps are attached with epoxy to the upper part of the foundation wall (see Figure 10). Before straps are installed, however, any necessary treatment to existing cracks or shear reinforcement to this upper wall section must be completed. Jacks are then installed in windows made in the wall below the

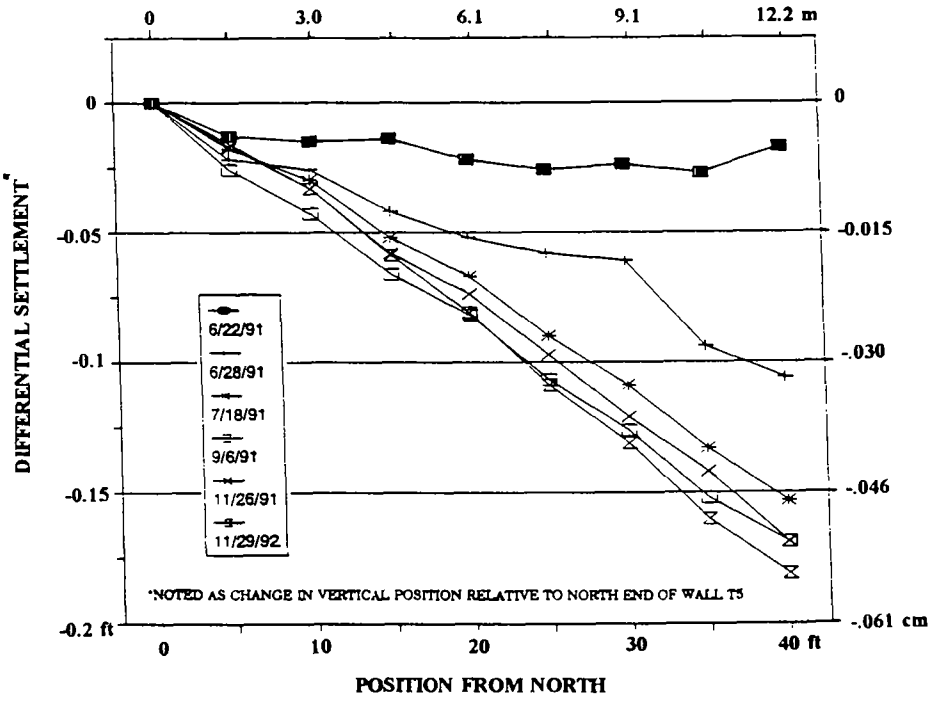


FIGURE 5 DIFFERENTIAL SETTLEMENT OF RETROFITTED WALL OF FOUNDATION T5

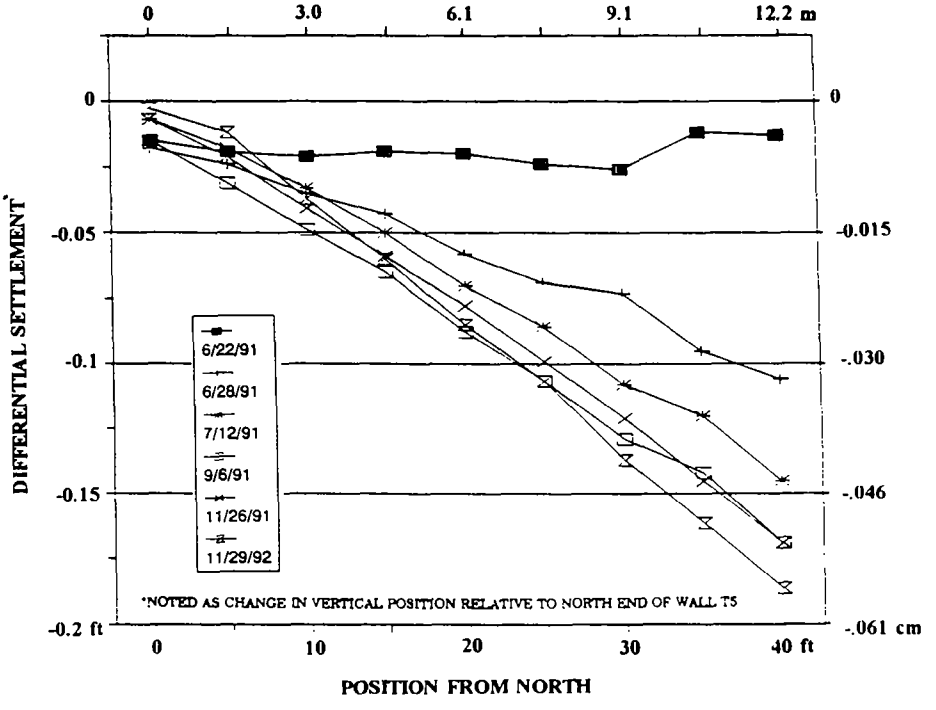
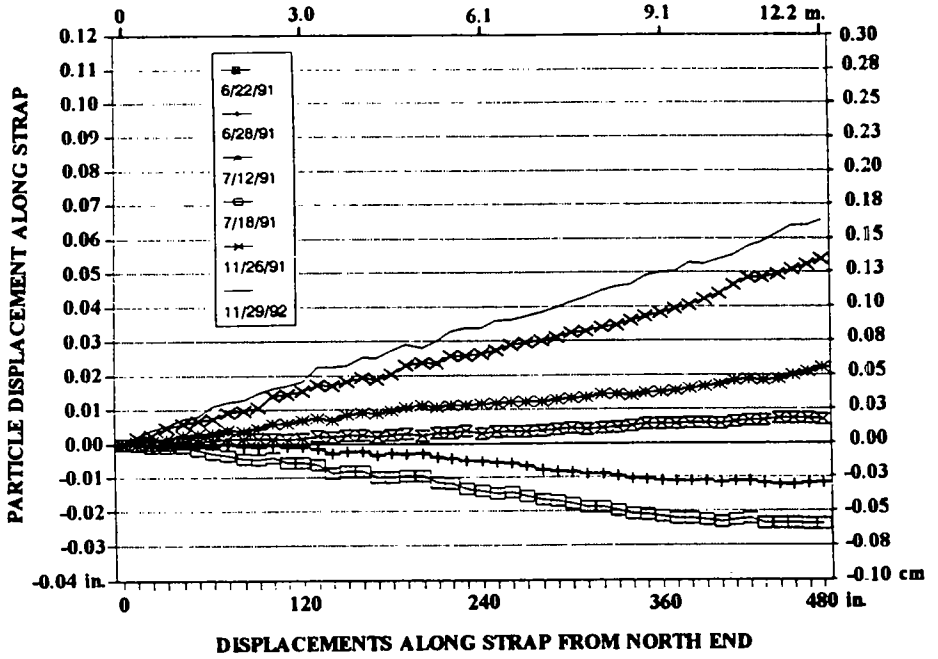


FIGURE 6 DIFFERENTIAL SETTLEMENT OF FOOTING OF FOUNDATION T5



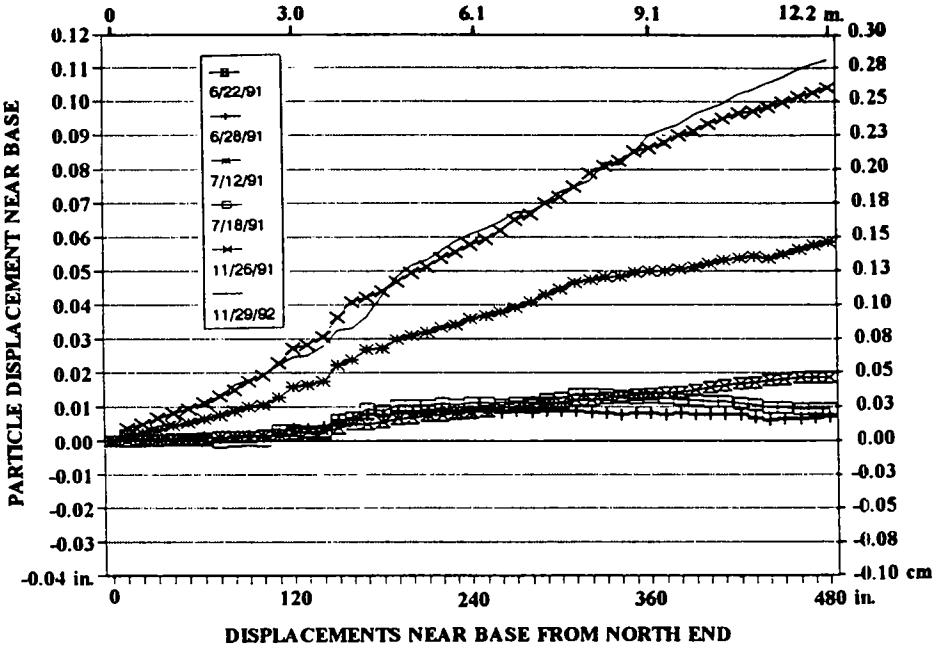


FIGURE 7 RELATIVE PARTICLE DISPLACEMENTS ALONG EAST FACE OF RETROFITTED WALL OF FOUNDATION T5

TABLE 1 TILT COMPARISON*

T5	7-12-91	11-26-91	11-29-92
Retrofitted Wall	0.37%	0.43%	0.45%
Footing	0.35%	0.40%	0.46%
T6			
Retrofitted Wall	0.36%	0.47%	0.48%
Footing	0.36%	0.43%	0.46%
Soil	0.39%	0.44%	0.48%

* Average tilt computed over entire length of profile since 5-30-91

TABLE 2 ANGULAR DISTORTION COMPARISON*

T5	7-12-91	11-26-91	11-29-92
Retrofitted Wall	roughly straight	1/2,500	1/2,174
Footing	straight	1/2,206	1/6,250**
T6			
Retrofitted Wall	straight	1/6,250	1/3,125
Footing	1/1,667	1/2,500**	1/1,500
Soil	1/1,667	1/1,282	1/1,210

* Measurement taken over entire soil monument profile (i.e., middle 30 ft), and includes angular distortion since 5-30-91.

** Possibly low value due to an erratic point.

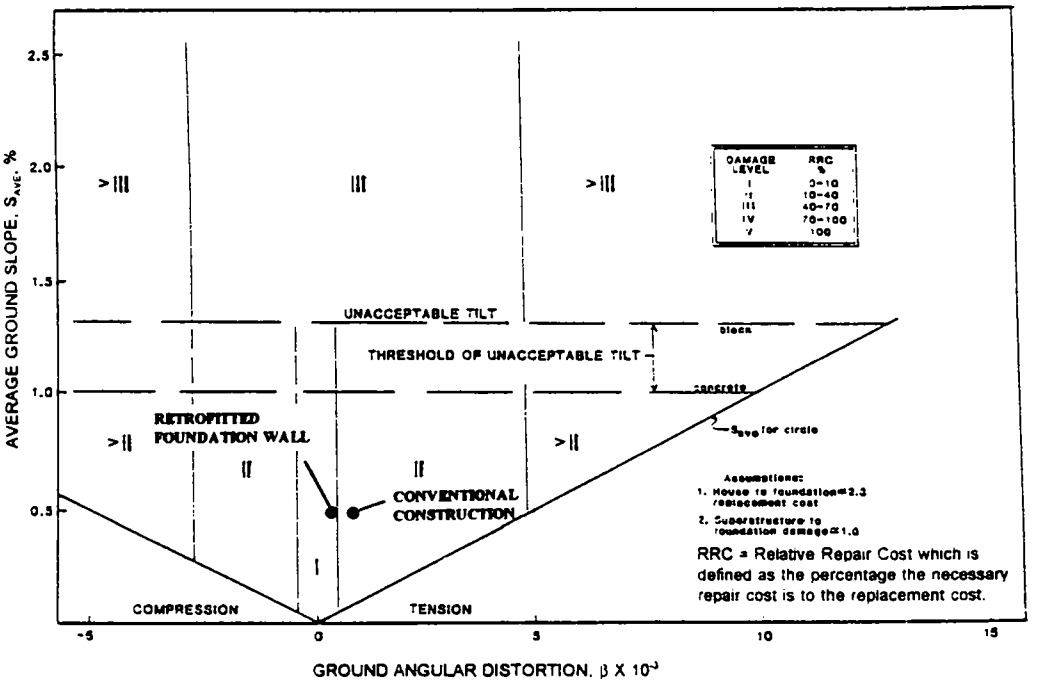


FIGURE 8 DAMAGE LEVELS RELATED TO THE ASSOCIATED ANGULAR DISTORTION AND AVERAGE SLOPE OF THE SAG PROFILES FOR HOUSES ON CRAWL-SPACE FOUNDATION (Marino and Mahar, 1985)

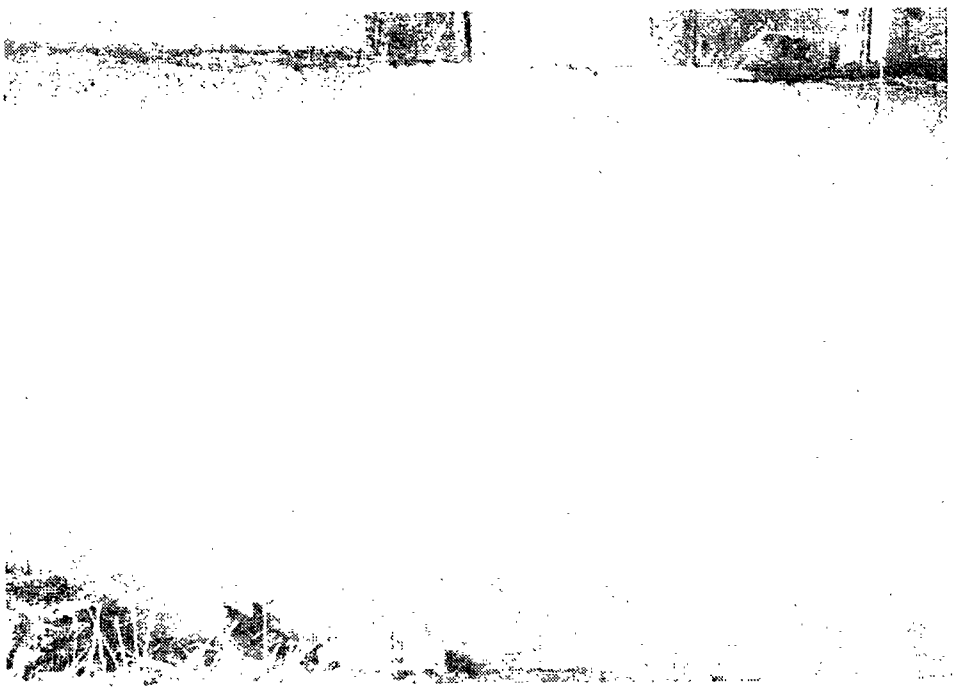


FIGURE 9 PHOTOGRAPH OF FACE OF FIBER-CEMENT COATING ON NOVEMBER 29, 1992

TABLE 3 COMMON SUBSIDENCE REPAIRS FOR HOMES WITH CRAWL SPACES AT DIFFERENT LEVELS OF DAMAGE (Marino and Mahar, 1985)

DAMAGE LEVEL	RRC	COMMON REPAIRS FOR CLASS B HOMES†		
		FOUNDATION	EXTERIOR	INTERIOR
I	≲ 0.1	Patching of minor wall cracks.	Tuck pointing of cracks in brickwork. Possible trimwork of siding/paneling. Repair of molding, and caulking separations. Possible repair of spout-roof separations.	Patching of minor wall cracks and painting of the dry or plaster walls in typically less than half the inside. Racked openings.
II	0.1 - 0.2	Same, possible replacement of accessory foundation element(s) (e.g.; displaced/cracked isolated block crawl-space section, slab foundation, exterior concrete, etc.)	Same.	Same, but with generally larger cracks and possibly paint entire inside. Repair caulk separations around fixtures and openings.
	0.2 - 0.4	Same.	Same, with possibly some replacement of brick.	Same.
III*	0.4 - 0.55	Same, replacement of crawl-space or crawl-space with a small basement section. House jacking necessary. Redecoration of living areas on a slab.	Same, but also partial replacement of siding, replacement of masonry veneer panel (less than one side and one story).	Same.
	0.55 - 0.7	Same.	Same, but replacement of 1 story brick veneer.	Same.
IV*	0.7 - 1.0	Same.	Same, replacement of exterior on wall or roofing. Repair of cracking in chimney-fireplace.	Same.
V*	1.0	REMOVE AND REPLACE HOUSE.		

† A class B home has a masonry crawl-space foundation, with possibly a small basement section. The house is wood-framed with brick, paneled, and sided exterior(s) and can have an attached garage.

The term "same" refers to damage conditions which are present at a lower level and have not been superseded by a worse condition.

* Possible abatement measures necessary.

strapped portion. The jacks are used to lift and level the upper part of the foundation and house. The jacking either occurs directly below a common mortar joint if the foundation is block, or a cut is made parallel and below the strap if the wall is concrete. During jacking a gap in the wall is created and is extensively and continually shimmed to prevent the occurrence of dangerous out-of-plane eccentricities, and to protect against the effects of loss of support due to hydraulic jack leaks.

Once the house and the upper strapped section are level the gap created in the wall can be filled with mortar or concrete and any treatment required for existing cracks can be taken care of. After the foundation walls are completely retrofitted and the house is level, the walls can be covered with fiber-cement for mainly aesthetic purposes. Of course, interior beams and/or walls are also appropriately lifted during the jacking process.

Wall Preparation for Jacking

Structural preparation similar to that conducted on Foundation T5/T6 was performed on T3/T4. (Note here, however, vertical steel straps were used in lieu of grouted-in rebar for shear strength repair. This was the first attempt at using vertical straps on the walls for shear strength repair.) Windows for the jacks were knocked out in the lower course of block at the prescribed places along the walls (see Figure 10). Bottle jacks were then inserted in the windows (see Figure 11).

Because the test foundations to be lifted are essentially two-dimensional structures interior bracing was felt necessary to provide some lateral support. The wood bracing system was designed to allow the jacked part of the foundation to move only in a vertical plane.

Instrumentation

In order to evaluate the amount of lift imposed on the strapped wall sections and soil bins, and to assess the induced strains in the steel straps, survey control points and Whittemore gauge points were established before any jacking was done. The survey control points were set on the top of the wall and were spaced about every 1.5 m (5 ft). Whittemore points were also located on the straps where maximum tensile strains were expected during the jacking process.

Jacking Process and Performance

The jacking process was conducted in three stages: 1. uniformly lifting walls 2.5 cm (1 in.); 2. rotating walls to level positions; and 3. adjusting jacks to straighten the top of the walls (see Figures 11 to 13). The main purpose of Stage 1 was to provide sufficient separation as to allow the north ends of the walls to rotate as the lower south ends were jacked upwards.

It was found during the Stage 2 of the jacking, the test foundation could be lifted 2.3 cm (0.9 in.) without causing excessive strain in the steel (a maximum strain of 0.00022 was measured at 2.3 cm jack displacement). Therefore, 2.3 cm was used as the maximum lift criterion. During Stages 2 and 3 the lifting was monitored mostly by measuring induced separation of the mortar joints with only occasional level surveys. Also when jacking, the surveying effort was lessened because negligible foundation settlement seemed to occur during jacking. Once Stage 2 was thought complete a survey was taken. Jacks were adjusted accordingly to bring the foundation walls level end-to-end. In order to bring the walls level they had to be lifted as much as 20-23 cm (8-9 in.) at the south end.

Although the walls were level end-to-end it was discovered that the middle of the walls were about 3.8 cm (1.5 in.) higher. This was determined by stretching strings from one end to the other at the tops of the walls. The tops of the walls were then straightened out by adjusting the jacks until both strings were coincident with the tops of the walls (see Figure 13). With the walls now straight the jacking was complete. The induced mortar joint separations were supported with additional shims and the jacks were removed.

After all the jacking was complete and the walls were straight and level, the most stressed point measured in the steel (based on Whittemore gauge measurements) reached only about 20 percent of its yield strength.

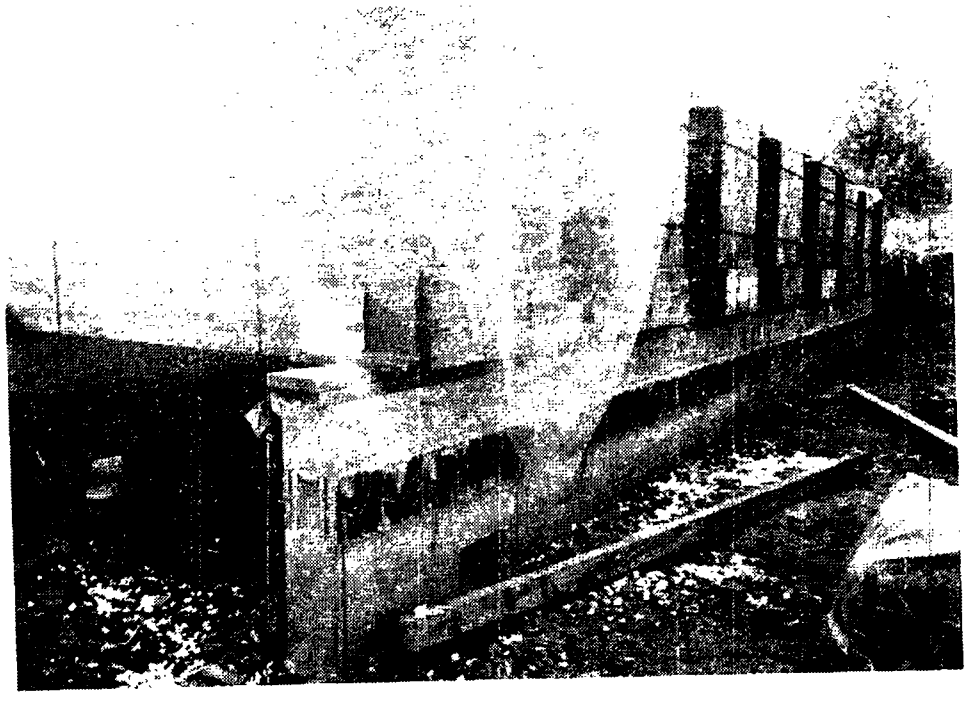


FIGURE 10 ALL JACK WINDOWS KNOCKED OUT

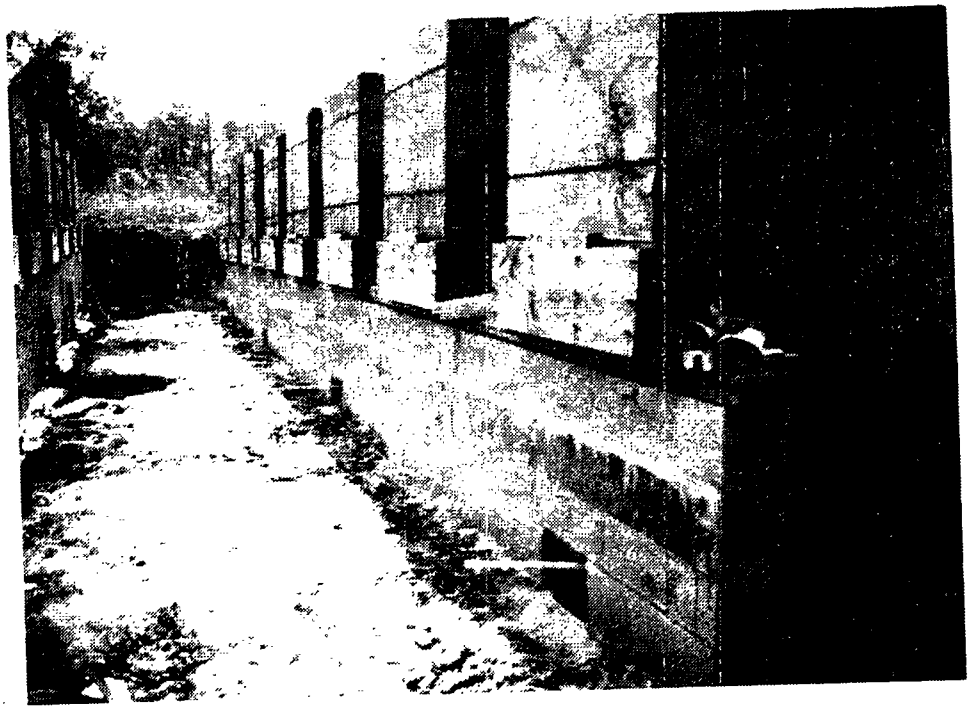


FIGURE 11 JACKS INSERTED IN WINDOWS

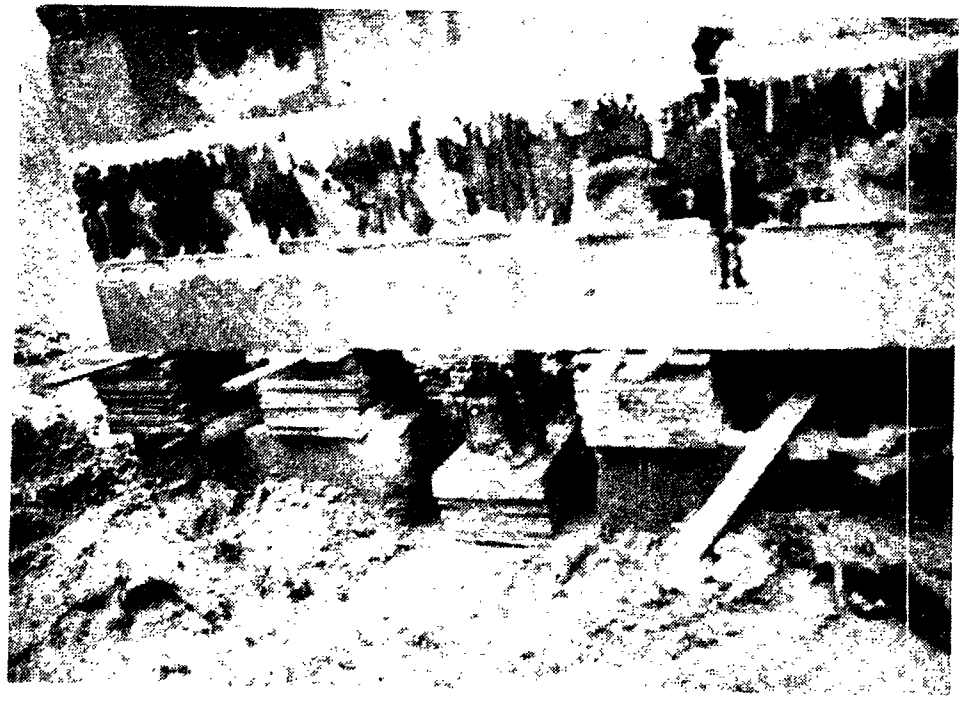


FIGURE 12 JACK ELEVATED IN WINDOW BY THE USE OF PLYWOOD PLATES

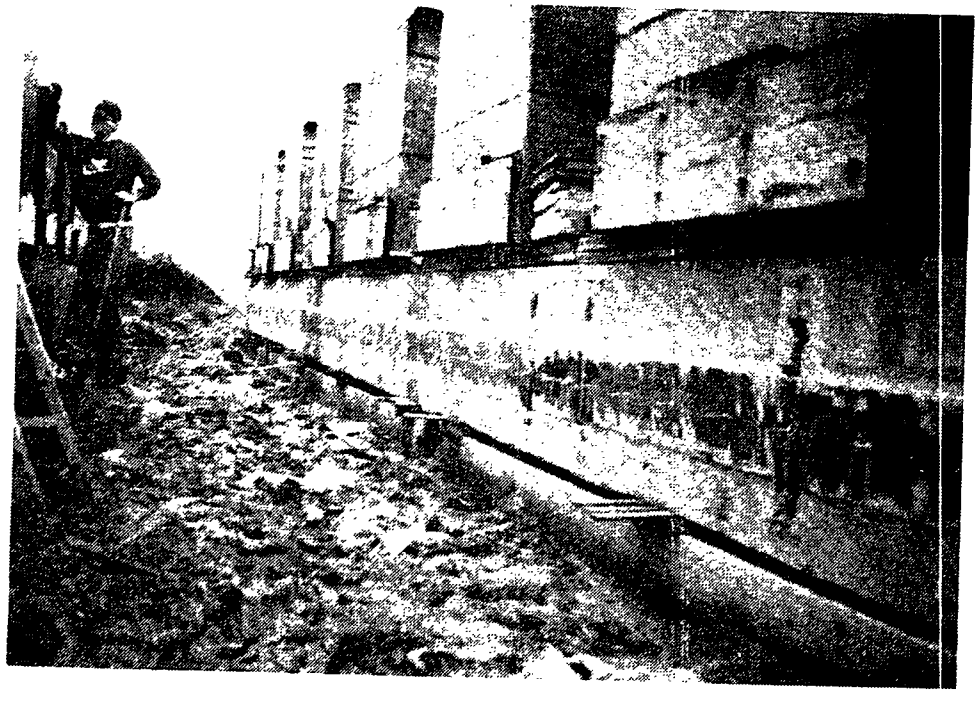


FIGURE 13 FINAL JACKED POSITION OF WALL

Summary and Conclusions

In 1987, new, more cost-effective methods of retrofitting damage foundations and tilt correction to sloped house foundations were proposed. The application of this retrofit technique is described herein and involves a method where steel straps are attached to foundation bearing walls. The advanced tilt correction methodology, also described in the paper, basically consists of: 1. reinforcing the upper section of the foundation walls using the advanced strap reinforcement technique; 2. inserting jacks in windows cut in the walls immediately below reinforced sections; 3. incrementally jacking level the reinforced wall sections and house; and 4. backfilling the separation created by lifting the reinforced section off the remainder of the foundation with mortar/concrete.

In this paper a field test of the proposed repair technique for retrofitting residential foundation walls damaged by mine subsidence is described and evaluated. The retrofit technique was designed to be cost efficient, simple to build, effective in strengthening foundation walls, and crack resistant. The method used consists of repairing wall cracks, externally reinforcing the walls, and covering the walls with a crack resistant aesthetically pleasing fiber-cement coating. The field test was located at a rural location just outside of West Frankfort, Illinois. Four block high, forty-foot long block walls, resting on strip footings were used in the test. These walls were located over a longwall coal mine.

The retrofit was simple to construct using only current methodology and was cost-effective in resisting the subsequent ground movements.

Over the monitoring period of 1.5 years, the hogging curvature of the ground had increased slightly to an angular distortion of 1/1,210. The angular distortion of the retrofitted foundation walls were about one-half that of the adjacent ground. Only slight changes in longitudinal strain have occurred over the monitoring period. Both the two-strap and one-strap foundation walls, T5 and T6 respectively, performed well. Also, the fiber-cement finishes have retained their aesthetic appeal after one and one-half years of weathering.

This proposed method of tilt correction was successfully and quite easily applied to relevel two field test foundations near West Frankfort, IL. These foundation walls were jacked up differentially about 20-23 cm (8-9 in.) In addition to jacking the walls level, it was also possible to eliminate the hogging in the wall. The wall slopes and deformations were caused by longwall mining activity. After all the jacking was complete and the walls were straight and level, the most stressed point in the steel reached about 20% of its yield strength, thus leaving adequate capacity to resist residual subsidence movements. In fact, it was found that the upper wall could be lifted about an inch at one jack without stressing the steel excessively.

Acknowledgements

By invitation from the U.S. Bureau of Mines in Minneapolis, MN and the cooperation of the University of Tennessee the test foundations at West Frankfort, IL became available for performing repair research. The author would like to especially thank Professor R. Bennett at the University of Tennessee, Knoxville, and Dr. L. Powel at the U. S. Bureau of Mines, Minneapolis, MN for their assistance in performing this work. We also are deeply indebted to Mr. E. Murphy of the Illinois Mine Subsidence Insurance Fund for his continuing support. The research was funded by The Illinois Mine Subsidence Insurance Fund. Also, the author would like to thank the staff of the Civil Engineering Department at the University of Illinois, especially Professor W. L. Gamble, for their contributions to the research projects. The manuscript was typed and some of the graphics were prepared by Mr. M. Reinking of Engineering Consultants.

References

Bennett, R. M., Drumm, E. C., and Johnson, D. C., 1992, Behavior of Linear Foundations Subjected to Longwall Subsidence, Proceedings of the Third Workshop on Surface Subsidence Due to Underground Mining, Morgantown, WV, pp 121-8.

Marino, G. G., and Mahar, J. W., 1985, House Damage Criteria for Sag-Subsidence over Illinois Room and Pillar Mines, AIME Trans., 6 pp.

Marino, G. G., 1987, Residential Handbook of Abatement and Reconstruction, Provisional Copy, Published by the Illinois Mine Subsidence Insurance Fund, Chicago, IL.

Marino, G. G., 1992, Reinforcing Block Foundation Walls, Masonry Construction Magazine, pp 185-7.

Marino, G. G., Hardesty, B. M., and Gamble, W. L., 1992A, The Response to Longwall Subsidence of Cracked Block Test Wall Foundations Retrofitted with Steel Straps, West Frankfort IL, Report to Illinois Mine Subsidence Insurance Fund.

Marino, G. G., Bennett, W. R., and Gamble, W. L., 1992B, General Specifications: Retrofitting Cracked Concrete Block Foundation Walls with Horizontal Steel Straps, Published by Illinois Mine Subsidence Insurance Fund, Chicago, IL.

Marino, G. G., Bennett, W. R., and Gamble, W. L., 1992C, Experimentation of Advanced Tilt Correction Methodology at West Frankfort IL, Report to the Illinois Mine Subsidence Insurance Fund.

Marino, G. G., and Gamble, W. L., 1993, A One Year Update of the Response of the Test Foundations to Longwall Subsidence, West Frankfort, IL, Report to the Illinois Mine Subsidence Insurance Fund.

Popov, E. D., 1968, Introduction to Mechanics of Solids, Prentice-Hall, Inc. Englewood Cliffs, NJ, 571 pp.

CONSIDERATION OF SUBSIDENCE IN LAND USE PLANNING POLICIES AND PROCEDURES IN ENGLAND[1]

David Brook[2]

Abstract. A widespread potential for subsidence requires consideration within a planning system which aims to control development in the public interest. A research strategy involving review at the national level of causes of subsidence followed by specific studies to develop techniques for use by planners provides the basis for planning guidance on the development of unstable land. This is now being taken forward with specific respect to subsidence to enable proper consideration in development plans and in considering applications for planning permission.

Introduction

England has a long history of mining dating from prehistoric times, although the principal period of activity was from the late 18th to the middle of the 20th century. Natural underground cavities due to dissolution of soluble rocks (chalk, limestone, gypsum, halite) are also widely distributed. In consequence there is a widespread potential for subsidence which may pose a threat to public safety and to property and land use.

When events occur, sub-editors have a field day with headlines such as:-

- "Crater fright for family" (Hitchen & Stewart 1995)
- "Falling apart at the coal seams" (Prior 1989)
- "Mine of information needed as homes face collapse" (Porter 1992)

Such events often represent a historical legacy of the time when little was known about the potential for subsidence and there was little, if any, control over the location of development. There are, however, a number of examples where recent development has been affected by subsidence. If the developer and those controlling development had taken account of the potential for subsidence most of these events could have been avoided. Whilst recognising that the responsibility for safe development and secure occupancy of a site rests with the developer and the landowner, we need to look at existing controls over development, how they have been exercised in practice and the improvements that can be made.

Controls on Development

The main controls over development and land use in England are through the operation of the planning system and the building regulations.

The Planning System

The legislation is now contained in the Town and Country Planning Act 1990, as amended by subsequent acts. The fundamental requirement is that development may not be undertaken without planning permission.

Development is defined in the 1990

[1] Paper presented at the 1996 National Meeting of the American Society for Surface Mining and Reclamation, Knoxville, Tennessee, May 18-23, 1996.

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Act as :- "The carrying out of building, engineering, mining or other operations in, on, over or under land, or the making of any material change in the use of any buidlings or other land." However, certain works and uses do not constitute development (eg the use of land for the purpose of agriculture or forestry) and the Town and Country Planning General Permitted Development Order 1995 gives general permission in advance for certain defined classes of development, mainly of a minor character (eg small extensions and alterations to dwelling houses).

Subject to these exceptions, specific planning permission is required for development upon application to the local planning authority (lpa). The Secretary of State for the Environment (SoS) has call-in powers to require in exceptional circumstances that applications be referred to the SoS for decision and aggrieved applicants may appeal to the SoS against the lpa's decision, ie against refusal of permission, against conditions imposed on the grant of permission or against failure to determine an application. There is no provision for appeal by other parties, though decisions may be challenged in the courts on legal grounds.

The lpa's decision on an application must "have regard to the provisions of the development plan, so far as material to the application, and to any other material consideration." Other material considerations are not defined and they can cover a wide field. They must, however, be genuine planning considerations, ie they must relate directly to the physical development and use of land. Permission may be granted subject to such conditions as the lpa or the SoS think fit, provided that they are necessary, relevant to planning, relevant to the development to be permitted, enforceable, precise and reasonable in all other respects. There is a presumption in favour of allowing applications for development, having regard to all material considerations, unless the development would cause demonstrable harm to interests of acknowledged importance. A developer is not required to prove the case for the development proposed and, if the lpa consider it necessary to refuse permission or to impose conditions, the onus is on them to give reasons for permission being refused or conditions imposed.

Development plans, as prescribed by the Acts, provide the framework for the control of development. They are intended to provide a firm basis for rational and consistent decisions on planning applications and a means of coordinating the needs of development and the interests of conservation. Where the development plan is material to a development proposal, the Planning and Compensation Act 1991 requires an application or appeal to be determined in accordance with the plan unless material considerations indicate otherwise. The system is thus essentially plan-led.

Structure Plans and Part I of Unitary Development Plans (UDPs) provide the broad strategic framework based on a written statement which may be accompanied by diagrams. Local Plans and Part II of UDPs are more specific, develop the strategic policies in the local context and contain land-use proposals which are map-based. Development plans are mandatory and are required to be consistent with national planning policies, with any regional planning guidance issued by the SoS and with each other.

<u>The Building Regulations</u>

The Building Regulations 1991, made under the Building Act 1984, provide an additional means of controlling built development. They are concerned primarily with the safe construction of a building and the provision of services to and from it so as to secure "the health, safety, welfare and convenience

of persons in and about the building." The regulations apply to building work in general, control of services and fittings and material change of use. However, there are exemptions such as greenhouses and agricultural buildings, temporary buildings, some small detached buildings and extensions.

The building regulations authority has a duty to see that building work complies with the regulations. If the work fails to comply, the developer may be required to alter or remove it. However, the submission of detailed plans for checking of compliance with the approved document is not compulsory and works may commence two days after the submission of a building notice.

Relevance of planning and building regulations to subsidence

Subsidence is not specifically mentioned in the Town and Country Planning Acts. It is clear, however, that planning matters are related to the physical development and use of land. This relates to the content of development plans, to decisions in individual cases and to the imposition of planning conditions.

The preparation of development plans requires lpas to undertake a survey of their area which must include "a review of the principal physical characteristics of the area" and the plan must contain statements on "measures for the improvement of the physical environment." In principle, there is as much justification for including policies on subsidence as a material planning consideration as there is to include other commonly accepted planning constraints such as Sites of Special Scientific Interest, Ancient Monuments, areas of high landscape value or high quality agricultural land.

Similarly, the physical characteristics of a site are relevant to the development and use of that site and can be considered as material considerations when deciding a planning application. The relevance of subsidence to planning decisions was recognised by the Ministry of Town & Country Planning (1949) when arrangements were announced for lpas to obtain the advice of the mineral valuers of the Valuation Office of the Board of Inland Revenue where any question of subsidence is involved.

In 1961, the Minister of Housing and Local Government gave his opinion that "decisions should not be taken to permit surface development without giving due weight to what is known or can be conjectured about the stability of the site" (Ministry of Housing & Local Government 1961). Whilst this advice was directed specifically to lpas in coal mining areas, it is clearly a general principle which is more widely applicable to other mining and other forms of land instability. There had also been a long-standing procedure for consultation with the nationalised coal industry. With privatisation of the coal industry, the Town and Country Planning (General Development Procedure) Order 1995 requires a lpa to consult the Coal Authority before granting planning permission for "development which involves the provision of a building or pipe-line in an area of coal working notified by the Coal Authority to the local planning authority."

Despite these provisions, it was clear by 1990 that the potential for subsidence was often not considered by lpas when deciding planning applications. Many planners regarded subsidence as a matter for the developer alone, or for the building regulations. At that time, the building regulations covered specific causes of ground movement such as swelling, shrinkage or freezing but there was dispute as to whether they covered subsidence due to mining, except in so far as the subsidence was caused by the weight of the building itself. Concern had also been expressed about the liabilities of lpas granting planning

permission for developments which were subsequently affected by subsidence.

In consequence, the Department of the Environment (1990) issued Planning Policy Guidance Note (PPG) 14 - *Development on unstable land* to clarify the position with respect to all forms of ground instability. This explained the responsibilities of the various parties to development and stressed the lpa duty to take all material considerations into account when reaching decisions on development proposals. The stability of the ground, in so far as it affects land use, is a material consideration and should be taken into account when deciding a planning application. Ways in which instability might be treated in development plans and in considering applications for planning permission were outlined and the role of expert advice was emphasised.

PPG 14 explained that it is the function of the planning system to determine, taking account of all material considerations, of which instability is only one, whether a proposed development should proceed. Having made that decision, for certain types of development, it is the function of the building regulations to determine whether the detailed design of buildings and their foundations will allow the buildings to be constructed and used safely.

Subsequently, the Building Regulations 1991 drew attention to potential problems caused by land instability in requiring that "The building shall be constructed so that ground movement caused by (a) swelling, shrinkage or freezing of the subsoil; or (b) landslip or subsidence, in so far as the risk can be reasonably foreseen, will not impair the stability of any part of the building."

Minerals Planning Guidance Note (MPG) 12 (Department of the Environment 1994) has taken forward the general principles in PPG 14 to advise lpas, landowners and developers of the specific consideration which needs to be given to the problem of disused mine openings and to demonstrate the need to compile information on mined ground in readily accessible, well organised data systems. Particular attention was given to the need for and operation of planning control over the treatment of mine openings in different circumstances (eg on closure of a mine, for public safety reasons, to safeguard conservation interests and to facilitate development) and to the consideration which needs to be given to mine entries and mine openings in determining planning applications and in development plans.

Department of the Environment Research on Subsidence

The planning guidance that has been issued has been based on a solid foundation of research on land stability issues which proceeded in line with a well defined strategy. This involved, firstly, reviews on a national scale of individual causes of land instability to determine what work had been done and what was needed. The second element of the strategy was to conduct specific area and topic-studies to develop techniques for the investigation and management of particular problems which would be more generally applicable.

The National Reviews of Land Instability

The general objectives of all the reviews were to examine the geographical and geological distribution of problems, causes and mechanisms, effects on the land surface and on development, methods of investigation, risk assessment and remedial measures, the legislative and administrative provisions for dealing with problems and priority topics for further research.

Review of Mining Instability in Great Britain. Carried out by Arup Geotechnics (1992), this involved the compilation

and interpretation of data from readily available publications and documentary records. For the purposes of the review, minerals were classified into five types :- metalliferous (non-ferrous) ores;rocks (eg sandstone, limestone,chalk); coal and associated minerals (eg fireclay, Coal Measures ironstone); iron ore outside coalfields; and evaporites (eg rock salt, gypsum).

Mining has been found to have taken place throughout England and Wales, although in some counties it is of only minor extent, and in all Scottish Regions except Western Isles (figure 1). Extensive coalfield mining covers more than 20 per cent of the areas of 16 counties in England and Wales and 3 Scottish Regions. Mining methods include both partial extraction (primitive shafts and headings such as bell pits or chalk wells, pillar and stall mines and brine pumping and solution mining) and total extraction methods (pillar and stall, stoping, longwall extraction and sub-level or block caving). The principal types of mining instability which may affect the ground surface are:-

- collapse of mine entries (shafts and adits);
- crown holes (localised collapses into mine voids); and
- general subsidence (also called areal subsidence, a wide saucer-shaped depression of the land surface without the massive discontinuities associated with crown holes).

The results of the review are presented in 3 volumes comprising:-

- Ten regional reports describe the extent of mining for the 5 mineral types on a county and district basis and contain advice to planners to be used in conjunction with PPG 14. Comprehensive schedules of mining areas indexed by reference codes, mine names and National Grid references provide a link to the associated !:250,000 scale maps showing areas where mining is known or suspected to have taken place and, therefore, should be regarded as a planning consideration.
- Five technical reports consider the effects of mines, investigation methods, preventive and remedial measures, monitoring methods and procedures for locating disused mine entries. Guidance is given on the procedures that should be adopted, their fitness for purpose, procurement and the priorities for further research as a basis for better technical guidance.
- Eleven case study reports, which provided the basis for the technical reports illustrate the variety of subsidence problems experienced due to mining instability. Each outlines the characteristics of mining and associated subsidence events in a specific area and summarises the site investigations and preventive and remedial measures which were used to deal with the problems.
- A separately published summary report and an executive summary provide a brief outline of the findings of all elements of the work undertaken for this review together with two summary maps, for northern and southern Britain respectively, at 1:625,000 scale of areas of mining consideration.

<u>Review of Natural Underground Cavities in Great Britain.</u> This review was carried out by Applied Geology Ltd (1993). Natural cavities were classified into 4 categories based on the mode of formation :- by dissolution (limestone, chalk, gypsum, salt and calcareous sandstone); by cambering (of competent cap-rock overlying less competent mud-rocks associated with past glacial and periglacial episodes); by marine erosion (sea caves in coastlines composed of competent rocks); and by other processes (eg soil piping, scour hollows resulting from periglacial processes and erosion of natural discontinuities).

Natural cavities were found to be widespread but not uniformly distributed throughout Great Britain (figure 2); 20,000 occurrences are recorded in the project database. The most significant concentrations are dissolution cavities in the Chalk of south east England and

East Anglia and the Carboniferous Limestone of the Yorkshire Dales, Derbyshire, north and south Wales and Avon and Somerset, with lesser concentrations due to cambering in the Permian, Jurassic and Cretaceous sequences of North Yorkshire, the East Midlands, Gloucestershire and Dorset. The cavities may be filled with air, water or unconsolidated deposits. Most instability is related to the downward movement of cover deposits rather than the cavity host rock. It may be sudden where air-filled voids are destabilised or occur more slowly with consolidation of loose infill deposits. The most common subsidence trigger involves water, often related to human activities.

The results of this review are presented in a similar format to those of the mining review with:-

- 10 regional reports (with accompanying 1:250,000 scale maps and a computerised database) describing the types and spatial distribution of natural cavities related to administrative areas and assessing them as constraints and amenities. Particular attention is given to regional experience of cavities affecting construction and engineering works, mining and tunnelling, mineral extraction, waste disposal, water resource development and public open space. Brief case histories are included within the regional reports.
- 3 technical reports discussing the nature and occurrence of natural cavities and their influence on planning and development, the range of site investigation techniques which may be appropriate for cavity detection and the ground treatment and structural design techniques available to overcome potential problems.
- A summary report with accompanying 1:625,000 scale maps of northern and southern Britain.

Techniques for Use by Planners.

Four examples are given below of the specific area- and topic-studies which have been carried out to assist planners in giving consideration to the problems of subsidence.

Development Advice Maps for Mining Subsidence. This project aimed to develop a method for preparing development advice maps (DAMs) giving advice to planners on abandoned shallow mining in the South Wales coalfield. A pilot study in the Ebbw Vale area (Ove Arup & Partners 1985) was followed by a further study in the Borough of Islwyn (Scott & Statham 1995) to confirm the methodology and to devise procedures enabling the maps to be used in the planning process.

Both studies were supplemented by a detailed review of subsidence incidents in the South Wales coalfield (Statham & Treharne 1991). This showed that 75 per cent of the 400 subsidence events recorded in about 35 years were collapses into workings at outcrop or mine entrances. The remainder were crownholes with an upper limit of migration of 8 to 12 times void height. Almost 70 per cent occurred in open land and less than 20 per cent caused damage to buildings or structures; only one example of injury to people was traced. Overall the risk of subsidence in the South Wales coalfield is very low, though it is higher at mine entrances and near to seam outcrops.

Preparation of the DAMs involved a detailed desk study from geological, mining, historical and other archival records to establish workable seams and their seam reputation for subsidence. Working maps showing modified geology, mining records, rock cover to seams and drift cover were then used to prepare maps showing zones where:-

- abandoned shallow mining is known or anticipated;
- abandoned shallow mining is possible;
- abandoned shallow mining is unlikely; and
- positions of mine entrances.

Draft "Planning guidelines" and "Guidance notes for potential applicants" were incorporated in the planning process by Islwyn Borough Council and the operation of this system was monitored for a year to assess the accuracy and worth of the DAMs and their usefulness in the planning process. It was concluded that the maps are a valuable tool, that their development should be extended to other areas of the South Wales coalfield and that the techniques could be applied to other coalfield areas in the United Kingdom.

Subsidence in Norwich. Following a number of fairly spectacular incidents in the late 1980s, a thorough survey of the causes of subsidence in Norwich was undertaken (Howard Humphreys & Partners Ltd 1993). This involved an examination of the records of subsidence and their causes, determining the extent and finding ways of managing the subsidence problem.

The study showed that not all of Norwich suffers from subsidence problems. Where they have occurred they have often been of a relatively minor nature, though there have been a number of severe subsidence events. The majority of subsidences were found, somewhat surprisingly, not to be associated with underground chalk mines, though the most severe ones were. A high proportion of the subsidences are associated with damaged or leaking water pipes and many of the subsidence problems have been exacerbated by poor construction and maintenance of property. The study concluded that the risk was relatively low, despite the many factors which contribute to the subsidence problem (chalk mines, cellars, undercrofts and wells, natural underground cavities in the chalk, collapse of sewers and leakage from services and the presence and nature of backfills and infills).

Most of the subsidences can be dealt with by simple preventive or remedial works provided appropriate investigations are made by developers and good practice is observed in construction and alteration of buildings. Dealing with subsidence is an integral part of Norwich City Council's local plan and account is taken of potential instability when considering development proposals and planning applications.

The recommendations for similar studies elsewhere emphasise the importance of a comprehensive data collection exercise, the importance of and requirements for underground inspection of old mines and the value of using a computerised database and geographic information system.

Assessment of Subsidence Hazard due to Gypsum Dissolution. The review of natural underground cavities had identified as a priority for further work the gypsum-bearing strata on the eastern side of the Pennines. In furtherance of this a study was commissioned to assess the working of the planning system in reducing hazards due to subsidence related to gypsum dissolution and to prepare a draft framework of advice suitable for use by planners, developers, land and property owners, insurers and others.

The study was focused on the Ripon area of North Yorkshire, where such subsidence was well known through the work of the British Geological Survey (Cooper 1995), but also covered the wider extent of the outcrop of gypsum-bearing strata. The geological, geomorphological and hydrogeological controls on gypsum dissolution and associated subsidence were examined and a conceptual model of gypsum dissolution in the Ripon area was established as a basis for assessment of the subsidence hazard. Recommendations were made on the engineering and planning responses to gypsum-related subsidence (Thompson & others 1995). The wider area study enabled confirmation of the conceptual model and conclusions to be drawn on the wider applicability of the framework of

advice for planners and others.

Treatment of Disused Mineshafts. Because of concern over the incidence of subsidence damage associated with disused mine opening, the Local Authority Associations and the Royal Institution of Chartered Surveyors made separate representations to the DOE about the need for advice. A Working Group was established and a program of research was undertaken on the treatment of disused mine openings and on the information base on mined ground (Freeman Fox Ltd 1988a,b).

Existing practice and methods of treatment were reviewed, classified according to their function and evaluated in respect of the most significant factor, depth to bedrock, and cost. Based on the concept of treatment objectives as a prime starting point for determining treatment policy, procedure, practice and selection of treatment method, a draft framework of advice was presented to indicate the most appropriate and cost-effective measures covering the majority of situations.

Similarly, the need for information on mined ground was examined along with information sources and types. Mining activity on which information is required was classified into shafts, roadways and workings and storage and retrieval options, including current practice in those authorities who already had information systems, were examined and recommendations made.

These two research programs provided the basis for MPG 12.

The Way Forward.

The need for consideration of potential subsidence in land-use planning has long been recognised but there have been some deficiencies in practice. A number of initiatives have been taken in recent years founded on a solid basis of research into the types of problems, their distribution and significance to planning and development and the appropriate planning responses.

Advice which makes it clear to lpas that subsidence and other forms of instability are material planning considerations which should be considered when preparing development plans and determining applications for planning permission has been issued in PPG 14. This advice has been augmented in respect of the treatment of disused mine openings and the availability of information on mined ground in MPG 12. It may yet be too early to see the direct benefits of this advice but a number of other initiatives are being taken forward.

Recent and current research is providing the basis for the planning guidance to be taken further in respect of subsidence from mining, natural underground cavities and other causes of subsidence and settlement. The aim is to reach a wider audience than the technical expert so that awareness is raised amongst land use planners, developers and financial institutions, enabling proper consideration to be given to the issues that arise. The DOE has started to prepare guidance which will advise planners and others on how they can identify and assess the potential for subsidence in order to take it into account in their everyday planning decisions. It will also advise on the information requirements for sound planning and as a basis for investment by developers. This will be based on the research described above.

Proper consideration of subsidence in the planning process will assist in reducing the impact of subsidence on the economy and on people. Whilst the risk has, in most cases, been found to be generally low to very low, when subsidence occurs the results can be spectacular and costly. They cannot be ignored by decision-makers. There will undoubtedly be some impacts due to the historical legacy of development in areas which are

prone to subsidence but it should be possible to avoid these impacts in most new development and redevelopment. It is also recognised that most forms of subsidence can be mitigated by appropriate engineering but that this is generally more cost-effective when undertaken in advance of development rather than as a result of incidents occurring during or after the development has taken place.

It is important to emphasise that the development of policies on subsidence and planning is not aimed at preventing development, though in some cases that may indeed be the appropriate response. Rather it is to ensure that development is suitable, that any risk is mitigated by appropriate engineering and that both natural and human-induced physical constraints on the land are considered at all stages of planning. Proper consideration of the potential for subsidence will thus contrbute to the broad objectives of the planning system to achieve economy and efficiency in the use of land and the protection of the environment.

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References.

Applied Geology Ltd, 1993. Review of instability due to natural underground cavities in Great Britain. Vol. 1, Regional Reports with 1:250,000 scale maps; Vol. 2, Technical Reports; Summary Report with 1:625,000 scale maps. Royal Leamington Spa, Rust Environmental.

Arup Geotechnics, 1992. Review of mining instability in Great Britain. Vol. 1, Regional Reports with 1:250,000 scale maps; Vol. 2, Technical Reports; Vol. 3, Case Study Reports; Executive Summary; Summary Report with 1:625,000 scale maps. Newcastle-upon-Tyne, Arup Geotechnics.

Cooper, A., 1995. Subsidence hazards caused by the dissolution of Permian gypsum in England: Geology, investigation and remediation. p. 457-466. In Maund, J., Penn, S. and M. Culshaw (Editors) Geohazards and engineering geology: International hazard assessment, mitigation and remediation. Preprints of papers for the 31st Ann. conf. Geol. Soc. Engineering Group, Coventry, U.K., 10-14 September, 1995.

Department of the Environment, 1990. Development on unstable land. Planning Policy Guidance Note 14. London, H.M.S.O., 25p.

Department of the Environment, 1994. Treatment of disused mine openings and availability of information on mined ground. Minerals Planning Guidance Note 12. London, H.M.S.O., 31p.

Freeman Fox Ltd, 1988a. Treatment of disused mine openings. London, H.M.S.O., 182p.

Freeman Fox Ltd, 1988b. Methods of compilation, storage and retrieval of data on disused mine openings and workings. London, H.M.S.O., 132p.

Hitchen, A. and F. Stewart, 1995. Crater fright for family. Ripon Gazette, 17 February, 1995.

Howard Humphreys & Partners Ltd, 1993. Subsidence in Norwich. London, H.M.S.O., 112p.

Ministry of Housing & Local Government, 1961. Town & Country Planning Acts, 1947-59: Surface development in coal mining areas. Ministry of Housing & Local Government Circular No. 44/61. London, H.M.S.O., 3p.

Ministry of Town & Country Planning, 1949. Consultation on proposals for development by local authorities and statutory undertakers. Ministry of Town & Country Planning Circular No. 65. London, H.M.S.O., 2p.

Ove Arup & Partners, 1985. South Wales mining subsidence desk study. Report No. 85/1100 to Department of the Environment and Welsh Office. Open-file report.

Porter, R., 1992. Mine of information needed as homes face collapse. Sunday Telegraph. 28 June, 1992.

Prior, G., 1989. Falling apart at the coal seams. Construction News. 20 April, 1989.

Scott, M.J. and I. Statham, 1995. Development advice maps - mining subsidence. p. 482-490. In Maund, J., Penn, S. and M. Culshaw (Editors) Geohazards and engineering geology: International hazard assessment, mitigation and remediation. Preprints of papers for the 31st Ann. conf. Geol. Soc. Engineering Group, Coventry, U.K., 10-14 September, 1995.

Statham, I. and G. Treharne, 1991. Subsidence due to abandoned mining in the South Wales coalfield, U.K.: causes, mechanisms and environmental risk assessment. p. 143-152. In Johnson, A.I. (Editor) Land subsidence. Proc. 4th Int Sympos. on Land Subsidence, Houston, TX, 12-17 May, 1991. Int Assoc Hydrological Sciences Publication No. 200, Wallingford, IAHS Press.

Thompson, A., Hine, P., Peach, D., Frost, L. and D. Brook, 1995. Subsidence hazard assessment as a basis for planning guidance in Ripon. p. 491-505. In Maund, J., Penn, S. and M. Culshaw (Editors) Geohazards and engineering geology: International hazard assessment, mitigation and remediation. Preprints of papers for the 31st Ann. conf. Geol. Soc. Engineering Group, Coventry, U.K., 10-14 September, 1995.

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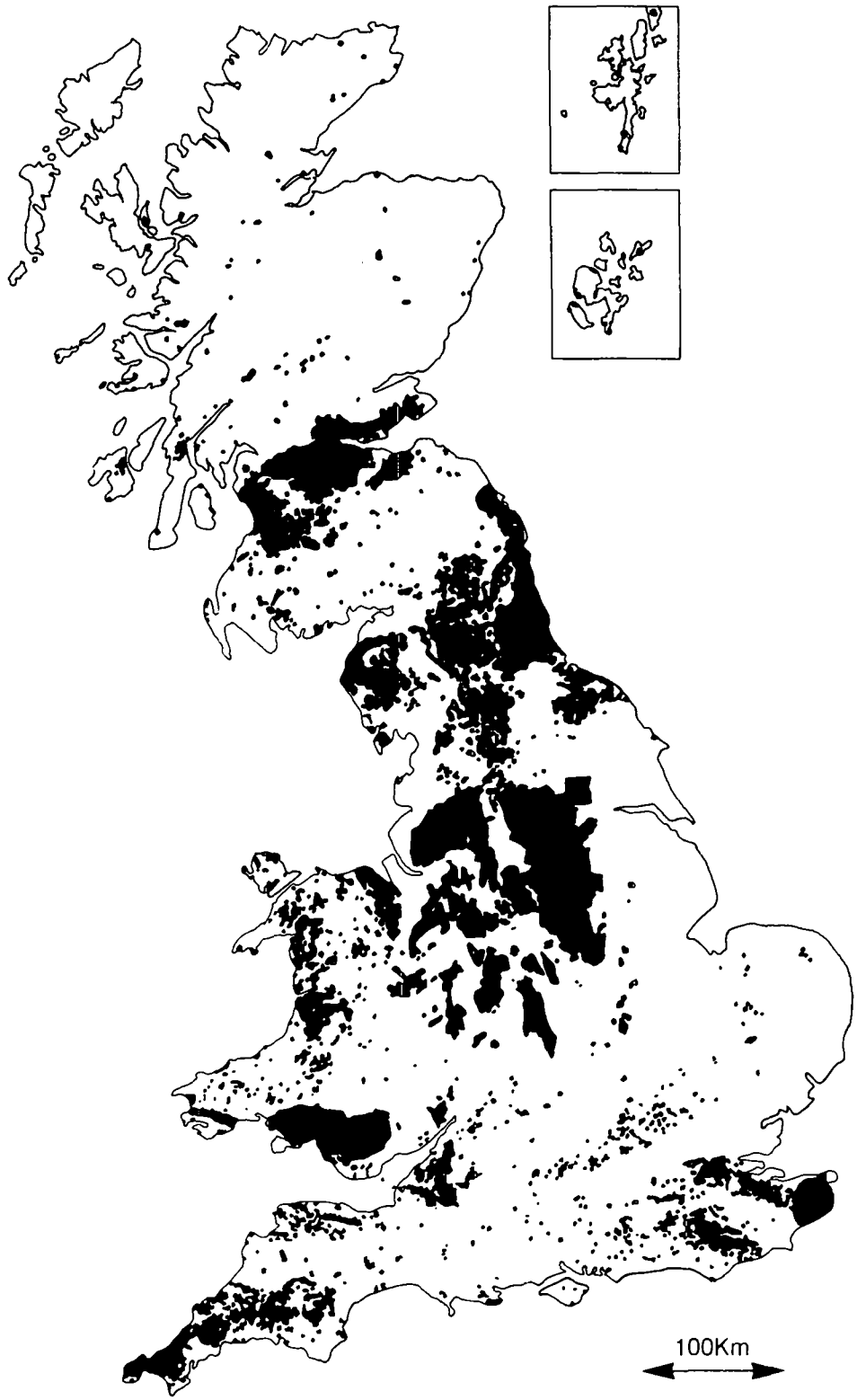

Figure 1. Mining areas in Great Britain (after Arup Geotechnics, 1992)

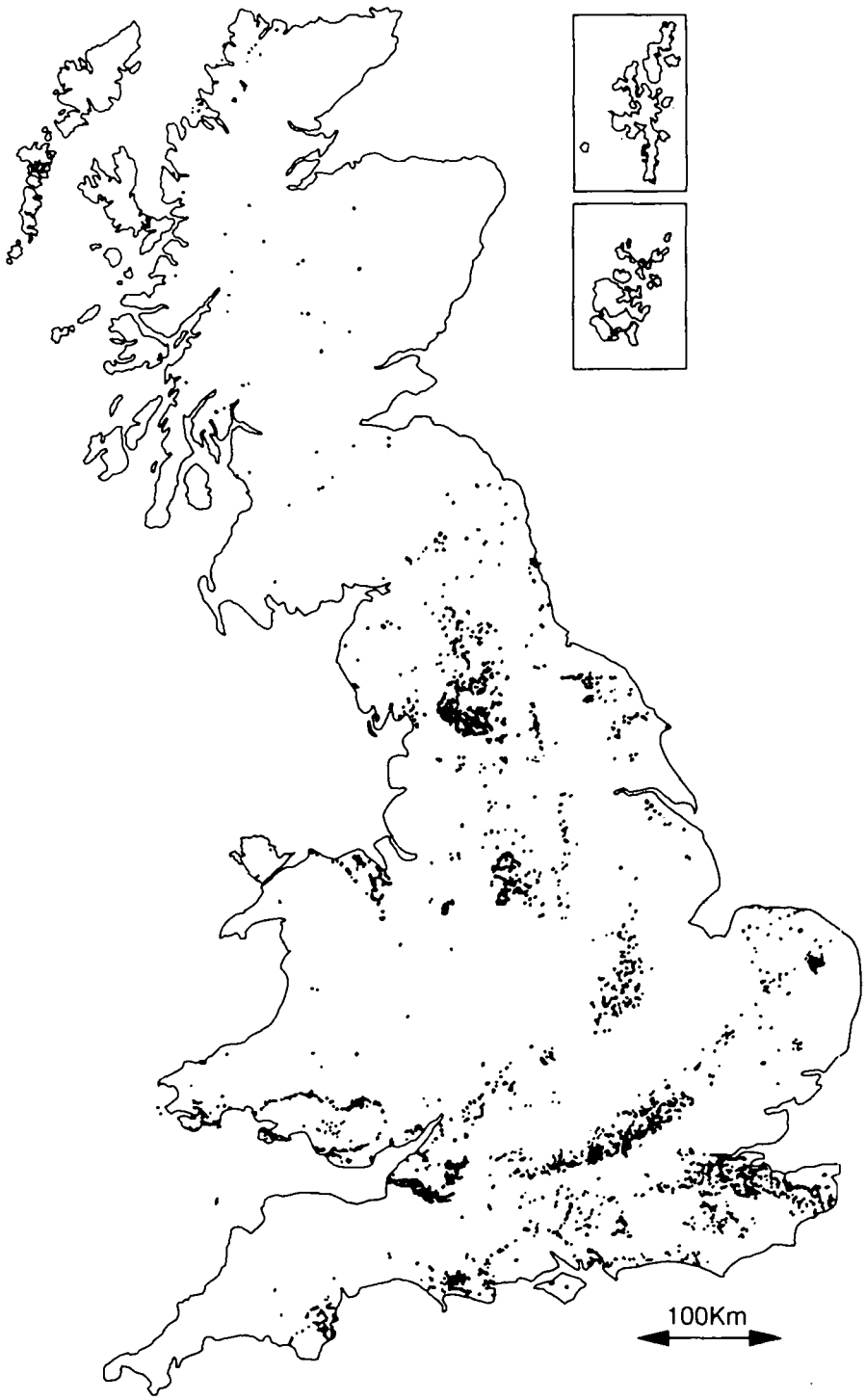

Figure 2. Distribution of natural underground cavities in Great Britain (after Applied Geology Ltd, 1993)

REPAIR OF THE CZAR SHAFT SINKHOLE

by

J. David Deatherage, Jack Starkey, Nils Gustavson

Abstract. After a portion of the Queen Mine parking lot collapsed into a sinkhole, an engineering study was conducted to confirm that the cause of the damage was settlement into the Czar Shaft in Bisbee, Arizona. Historical records were researched to describe and locate the original shaft, and a seismic refraction survey was performed to define the foundation conditions within and around the shaft. Recommended mitigation measures were developed to repair the parking lot. Mitigation included design and construction of a flexible geosynthetic cap and telltale settlement monitoring system.

Additional Key Words: geotextile, settlement, subsidence.

Introduction

The City of Bisbee operates the underground Queen Mine as a tourist attraction. Tourists park in a lot 18 meters by 46 meters (60 feet by 150 feet) in dimension set into the northeast side of a limestone hill overlooking Bisbee. In March of 1995 the center of the parking lot collapsed under the weight of a tour bus. After safely removing the tour bus, a sinkhole roughly one meter deep (three feet) and 4.5 meters (fifteen feet) in diameter was found under the parking lot, as presented in Figure 1. Water was flooded into the sinkhole at a rate of 20,000 to 40,000 liters (five to ten thousand gallons) per day for a one week period, in an attempt to consolidate the underlying fill. Subsequent excavation in search of lateral settlement limits widened the sinkhole to a diameter of 12 meters (40 feet). Settlements within the sinkhole of 25 to 33 centimeters (10 to 13 inches) per day were observed during the flooding. The settlement stopped when the flooding was discontinued. The sinkhole was temporarily backfilled with 32 truck loads of soil and rock. The City of Bisbee contracted with DEI Professional Services (DEI) to confirm the cause of the sinkhole, and to develop a mitigation design to bridge and cap the failure.

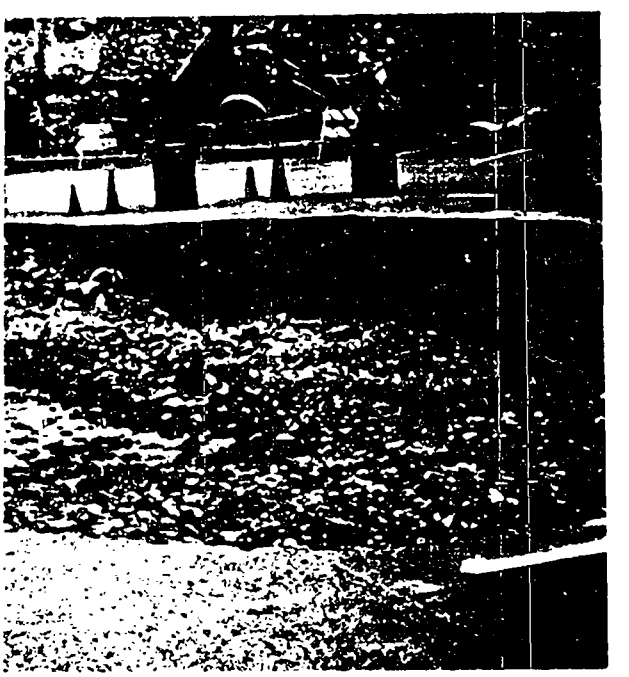

Figure 1: Queen Mine Parking Lot Sinkhole

(1) Presented paper at the Annual Conference of the American Society for Surface Mining and Reclamation, Knoxville, TN, May 19-25, 1996.

(2) J. David Deatherage, P.E., is the senior geotechnical engineer and Jack Starkey, C.D.T., is the senior civil designer with DEI Professional Services in Phoenix AZ, 85254. Nils Gustavson is Public Works Director for the City of Bisbee, AZ, 85603.

Investigation

DEI reviewed available literature relating to the site and conducted a limited geotechnical field investigation as part of the study. The field work included seismic profiling of the sinkhole, and additional sinkhole water flooding and settlement measurement.

Literature Review

Research of historical information relating to the site was conducted from literature and publications in the Bisbee Mining and Historical Museum. The Czar Shaft was sunk by the Copper Queen Company in 1885 to a depth of 134 meters (440 feet). The Shaft provided access to a carbonate copper ore deposit, (Graeme, 1981). The carbonate ore is found within early Cretaceous limestone rock included in the Bisbee Group. (Nations, 1983). The proximity of copper-rich igneous rock and limestone rock produced a classic zone of enrichment. Metallic solutions percolated into seams and fractures in the porous limestone, depositing metal-rich minerals, (Francaviglia, 1982).

Of particular interest in the Francaviglia publication were early 1900 period photos of the Czar Shaft site that show considerable construction related filling of the site. The filling was necessary to produce a flat bench on the side of the limestone hill for the mining buildings relating to the Copper Queen Mine and the Czar Shaft operations.

The Czar Shaft operated for sixty years until closure in 1944. The conditions within the shaft were reportedly very wet and cold. The shaft was capped with concrete in 1961 to stop trespassing. Shaft timbers reportedly decomposed from the lack of air, and in 1973 the head-frame collapsed. The shaft was then closed by backfilling, (Graeme, 1981).

Field Investigation

The parking lot is situated on the northeast side of a limestone hill. The Shaft is approximately 45 meters (150 feet) to the left of the Queen Mine Tour buildings, as shown in Figure 2. The sinkhole area in the parking lot is in the center of Figure 2, and barrels ring the temporarily backfilled area. Considerable filling is evident in the areas around the parking lot.

A limited geotechnical field investigation was conducted to differentiate the loose Shaft fill from the surrounding fill and native rock materials. DEI assisted AGRA Earth and Environmental in performing a refraction seismic survey of the sinkhole area. A 12-channel Geometrics signal enhancement seismograph was used for velocity data recording. Maximum depths of 4.5 to 6 meters (15 to 20 feet) were investigated using ten foot wide intervals for the geophones. Two seismic lines were run through the sinkhole, and two seismic lines were run on the inside and outside portions of the parking lot. Loose fill inside the sinkhole exhibited compression wave velocities of 225 meters per second (mps). Fill on the sides of the sinkhole exhibited significantly higher velocities below a depth of three meters, with a range of 430 to 800 mps. The line closest to the limestone hill detected the limestone at a three meter depth, with a velocity of 2,100 mps.

Figure 2: Temporarily Backfilled Sinkhole

The shallow seismic investigation confirmed the horizontal limits of the sinkhole to the immediate area of the surface expression of the settlement. The investigation also revealed that the pad for the original mining operations and parking lot was composed of a mixture of loose and medium dense fill overlying a dipping contact with the original limestone hill.

Additional water flooding was conducted in an attempt to initiate settlements within the fill over the Shaft. From May 15 through May 29 (15 days) from 27,000 to 40,000 liters (7,000 to 10,000 gallons) per day of water was run into the depression. Four measurement points were monitored within the depression, and at the end of the flooding no significant settlement was observed in any of the

measurement points. Approximately 15 centimeters (six inches) of total settlement was observed in the fill on the northeast side of the sinkhole when the material was first wetted. No additional settlement was observed after repeated flooding.

Results and Conclusions

The investigation confirmed that the Czar Shaft was originally located under the parking lot sinkhole. The historic review revealed past foundation distress problems related to settlement movement within the Shaft fill materials. These movements include the collapse of the original headframe over the Czar Shaft. Soils surrounding the Czar Shaft fill are not sufficiently dense to support concentrated loading from a rigid structural cap without possible significant settlement and movement.

The settlement into the Czar Shaft is similar to problems experienced with limestone sinkholes in karst areas of the country. Construction techniques employed to stabilize these features include the placement of thick reinforced concrete slabs, injection of cement grout under high pressure, and filling of voids with solids and concrete. Complicating geotechnical conditions at the Czar Shaft site include significant thickness of non-engineered fill in the upper portion of the shaft area.

DEI concluded that a structurally reinforced cap bearing on the adjacent fill was not a positive or economical alternative to repair the sinkhole. A mitigation measure was selected to bridge over minor future settlements beneath the parking lot. The mitigation allows for monitoring and future additional corrective measures for more significant settlements, should they occur.

A flexible geosynthetic cap was designed to distribute the parking lot loading over the densified existing and new Czar Shaft fill. The design was based on engineering judgement that considered the removal of as much loose overburden as site access and utilities would allow, and replacement of the loose overburden with geotextile reinforced fill.

The planned construction consisted of excavation of the sinkhole area to a depth of 4.5 meters (15 feet). The excavation had a flat bottom ten feet wide, and was nine meters (thirty feet) in length. Side slopes were designed as one to one (horizontal to vertical), with a rectangular top of excavation 12 meters (40 feet) wide and 18 meters (60 feet) long. Fill was specified as imported or excavated material consisting of clean sand, gravel and cobbles. The material was well blended with no isolated nests of gravels and cobbles, no particles larger than 30 centimeters (12 inches), and less than 8 percent passing the No. 200 sieve. The fill soils were free of organic matter, construction debris, and other deleterious materials. Due to the coarse nature of the fill, the compaction was specified as 85 to 95 percent of the maximum relative density. The compaction was evaluated visually by the engineer's representative without testing.

High strength woven geotextile fabric was detailed in an alternating pattern between subsequent fill layers. A wide roll width of 5.5 meters (18 feet) was specified to reduce overlap connections between sheets.

The geotextile was specified as a woven polypropylene fabric with the following minimum average roll values:

Property	Method	Value
Grab Tensile	ASTM D-4632	500 lbs.
Elongation	ASTM D-4632	20%
Wide Width	ASTM D-4595	4800 lbs. / ft.
Elongation	ASTM D-4595	18%
Mullen Burst	ASTM D-3786	1350 psi
Puncture	ASTM D-3786	140 lbs.
Trap. Tear	ASTM D-4533	250 lbs.
Roll Width	-	18 ft.
Roll Length	-	150 ft.

The geotextile was selected based on its high strength, flexibility, and resistance to biological and chemical degradation. Availability and shipping delivery also were important to the timing of the construction. For estimating purposes the material cost of the geotextile was approximately $2.00 per square meter.

Details of the design showing the placement of the geotextile sheets into each of the five layers filling the excavation are presented in the following Figures 3 through 7.

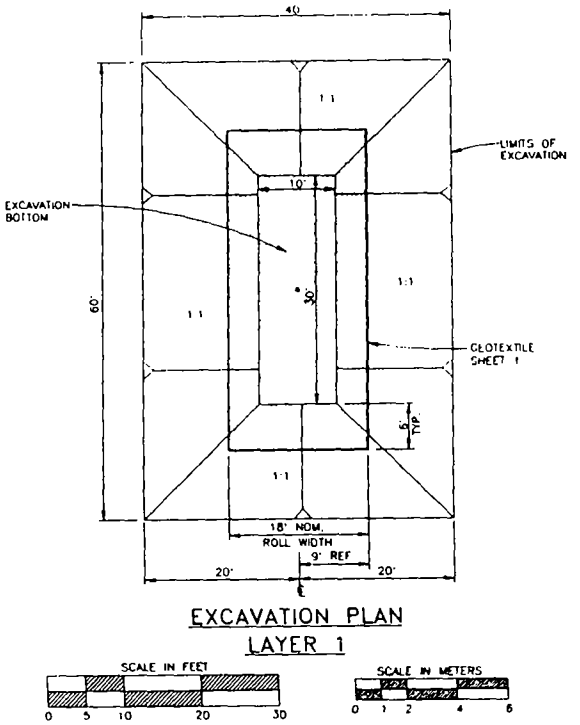

Figure 3: Plan View of Layer 1

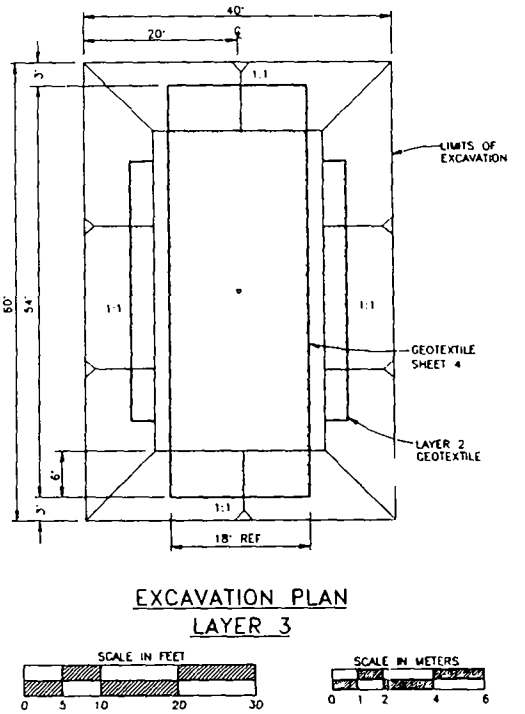

Figure 5: Plan View of Layer 3

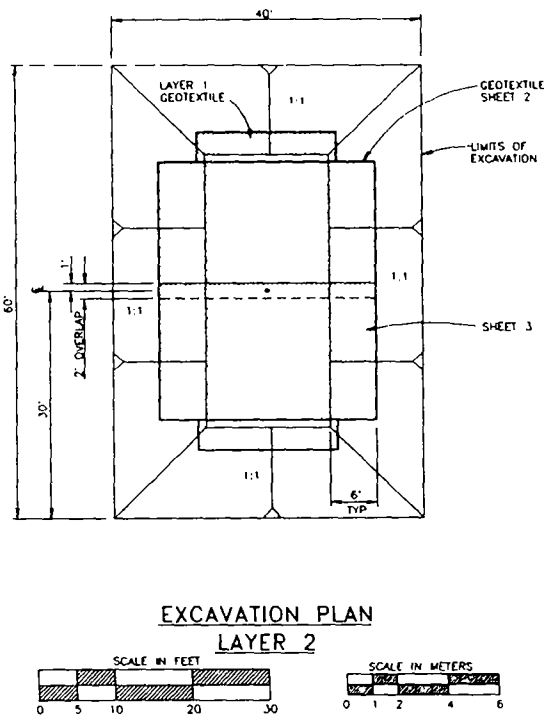

Figure 4: Plan View of Layer 2

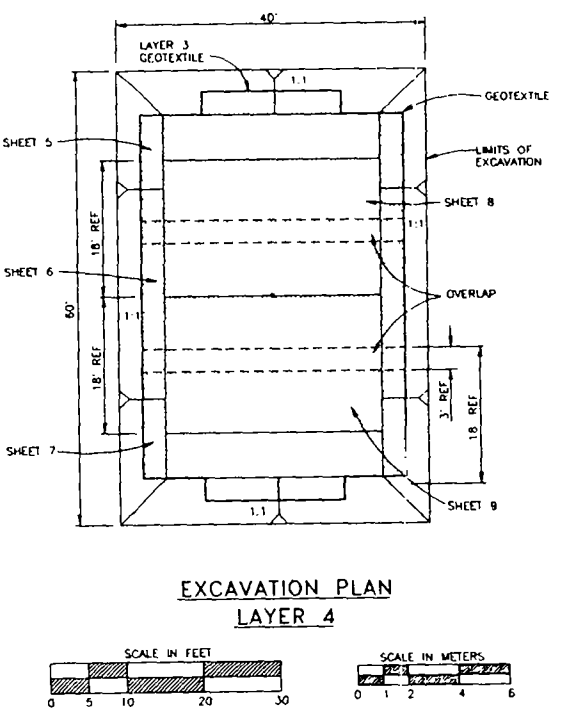

Figure 6: Plan View of Layer 4

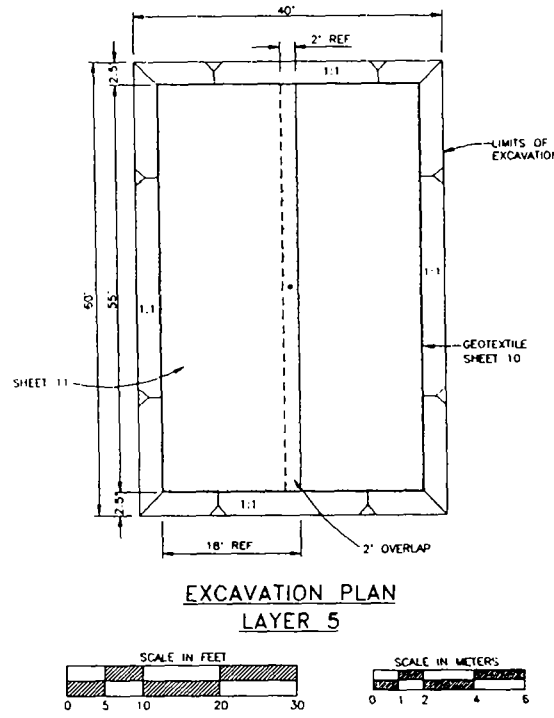

Figure 7: Plan View of Layer 5

Geotextile sheet arrangement on the side slopes of the excavation was carefully controlled to develop the maximum frictional resistance in the outer portions of each geotextile sheet. Care was taken not to overlap geotextile from one layer on top of another layer. This mitigated the significant loss in frictional resistance when one side of the geotextile is bedded against another sheet of geotextile. Frictional resistance of the geotextile sheet is similar to the backfill soil frictional strength if the geotextile is bedded on both sides by the soil. The full tensile capacity of the geotextile can be mobilized after a few feet of burial in the backfill soil. Figures 8 and 9 present sections through both the long and short axis of the excavation and geosynthetic cap system.

A telltale system was incorporated into the design to allow for periodic monitoring of the settlement of the Czar Shaft fill under the geosynthetic reinforced cap. A steel plate was specified to be placed in the center of the excavation, beneath the first layer of geotextile. A 15.25 centimeter (six inch) diameter Schedule 40 PVC pipe extends from the bottom of the excavation to the surface. The pipe provides an access port for the 2.0 centimeter (0.75 inch) diameter telltale pipe that extends from the bottom steel plate to the surface.

Construction

Construction started on August 11, 1995 with material removed from the planned excavation by a Cat 235L Trackhoe. All excavation and compaction operations were continuously monitored by a field engineer representative of DEI. Workers and equipment were not allowed into the bottom of the excavation because of the unknown Shaft fill conditions below the bottom of the excavation.

Excavated material was picked up with a Kawasaki 85Z2 loader and placed into a stockpile. DEI originally specified compaction with a vibrating plate compactor or compaction wheel attached to the end of the trackhoe bucket. The contractor was unable to provide the proper equipment, and elected to use the trackhoe as a battering ram to compact the soil with multiple impacts of the bucket. The base and side slopes of the excavation were compacted with repeated blows of the trackhoe bucket. A fire hose was used to supply water to the fill prior and during compaction.

The settlement plate and pipe were installed through the first layer of geotextile. Fill was then placed and compacted in two lifts totaling 90 centimeters (36 inch) thick over the geotextile. A minimum overlap of adjacent geotextile sheets was maintained at 0.6 meter (two feet). The geotextile was held in place during backfilling with steel spike nails driven into the walls of the excavation. The first 90 centimeter (36 inch) layer of fill was complete by the end of the second day of construction. By the end of the third day the third layer of geotextile was in place and covered with a lift of fill.

The settlement monitoring pipe was extended upward through each layer of the filling operations. On the fourth day the fill was placed over the fourth layer of geotextile, and a self propelled tamping foot compactor (Ingersoll Rand SD40D) was used to compact the fill layers. By the fifth day of construction the final geotextile layer was installed and the fill over the geotextile was compacted. Figures 10 and 11 present details of the settlement plate and telltale.

The contractor returned to the site several weeks after finishing the filling of the excavation to pave the filled portion of the parking lot, and to finish the surface completion of the telltale monitor. DEI

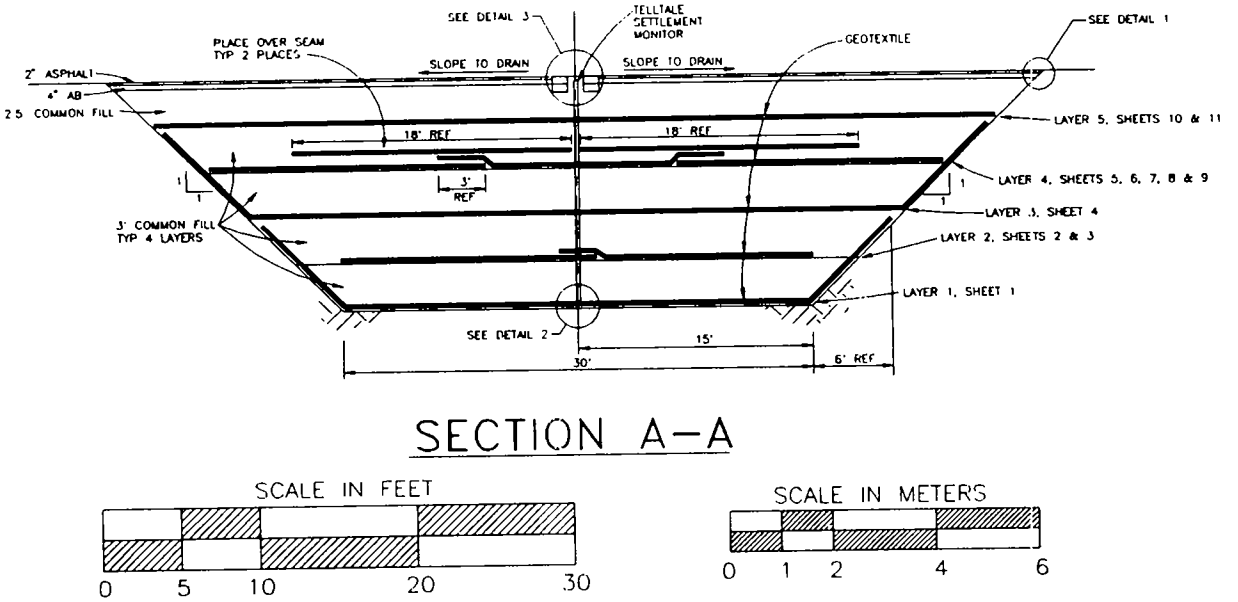

Figure 8: Section through the long axis of the cap.

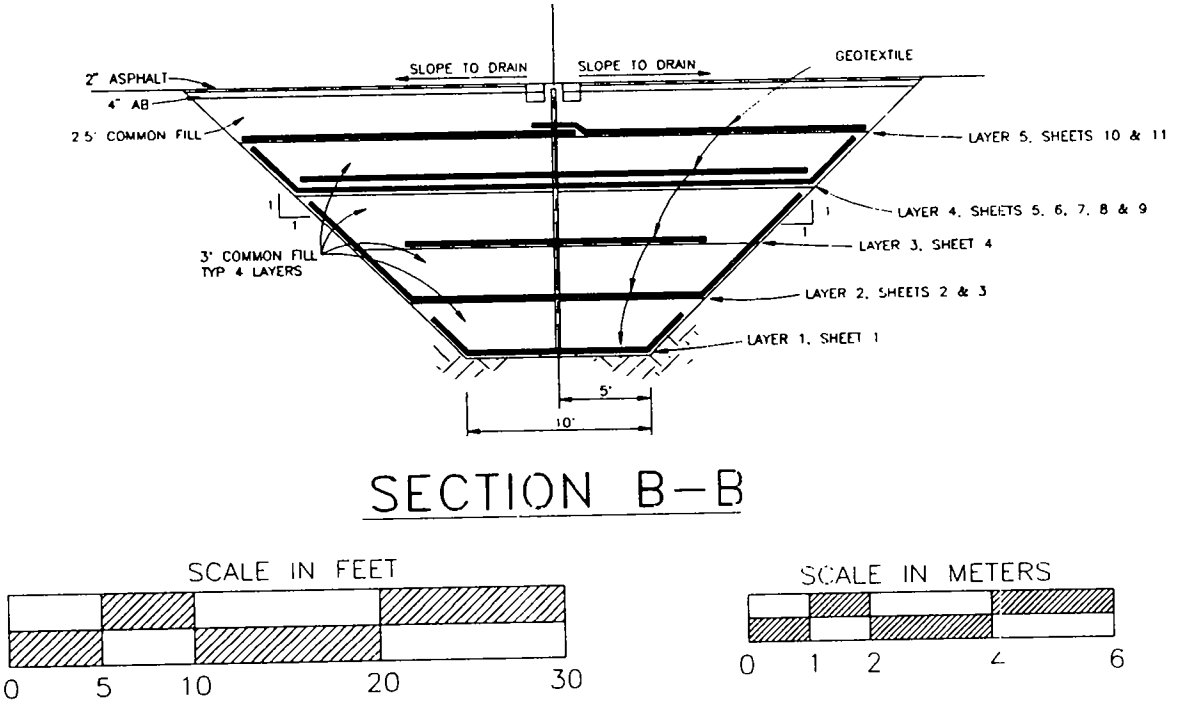

Figure 9: Section through the short axis of the cap.

returned to the site on November 6, 1995 to observe the completed construction. Figures 12 through 17 present photographs of the construction.

Telltale Settlement Monitoring

The telltale settlement monitor will be periodically inspected by representatives of the City of Bisbee. DEI suggested a monitoring frequency of weekly for the first month, then monthly for one year, then quarterly or twice yearly after the first year. Through the end of March 1996 there had been no measurable settlement beneath the geosynthetic cap. The intent of the geosynthetic cap and telltale settlement monitoring is to provide an engineering system to safely bridge minor settlements in the underlying shaft. The system will also provide a means to monitor and mitigate more significant settlements that may occur in the future. Monitoring will detect significant settlement and allow possible mitigation before the progression of damaging and potentially unsafe settlements. Mitigation will consist of injection of concrete grout through the monitoring pipe to fill any voids forming under the cap. If necessary this process can be repeated to maintain the integrity of the cap.

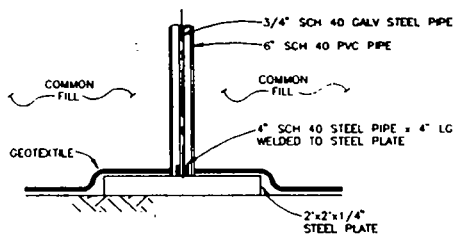

Figure 10: Detail of Settlement Plate

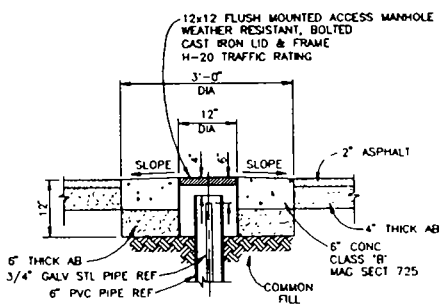

Figure 11: Detail of Settlement Telltale

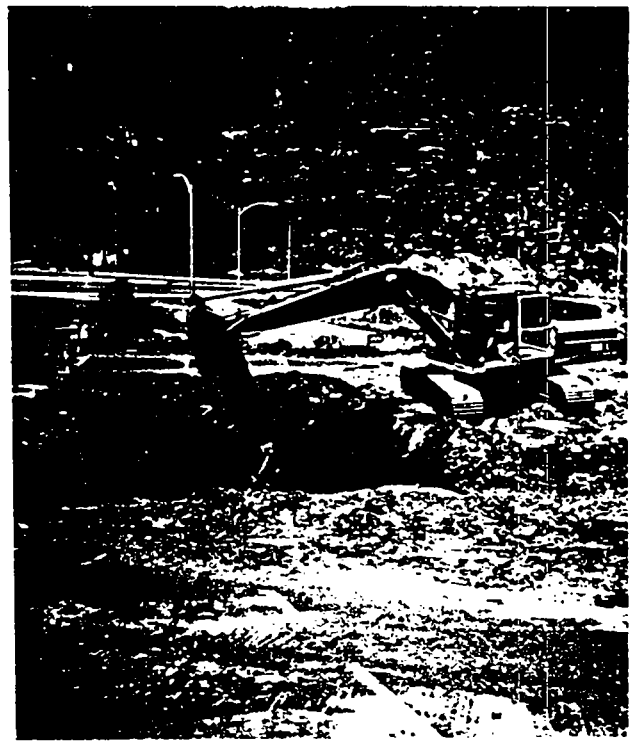

Figure 12: Excavation of Sinkhole

Figure 13: Compaction of Fill Over Layer 1

Figure 14: Placement of Layer 2

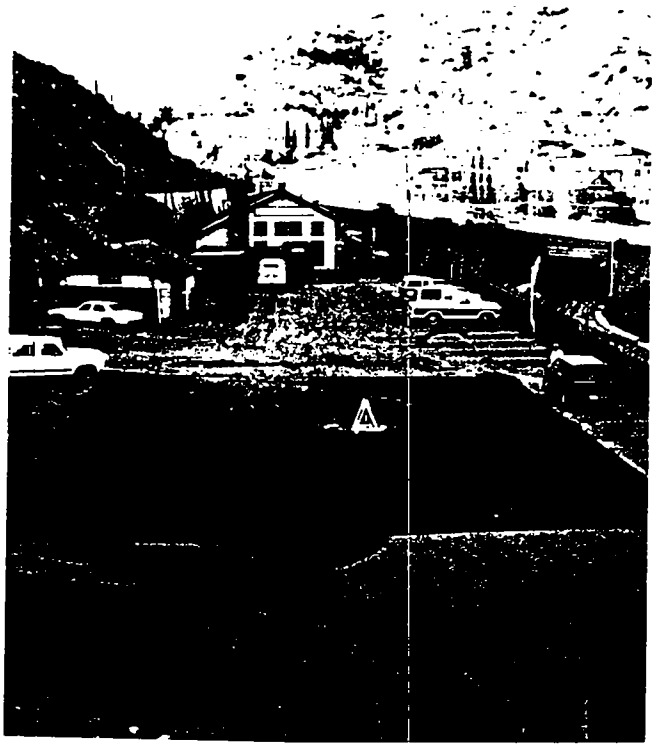

Figure 16: Completed Repaved Parking Lot

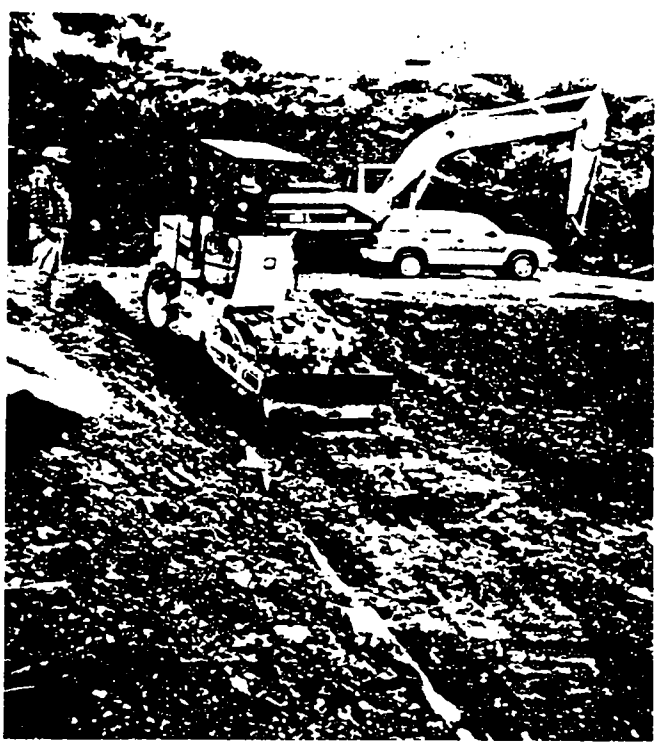

Figure 15: Compaction of Fill Over Geotextile

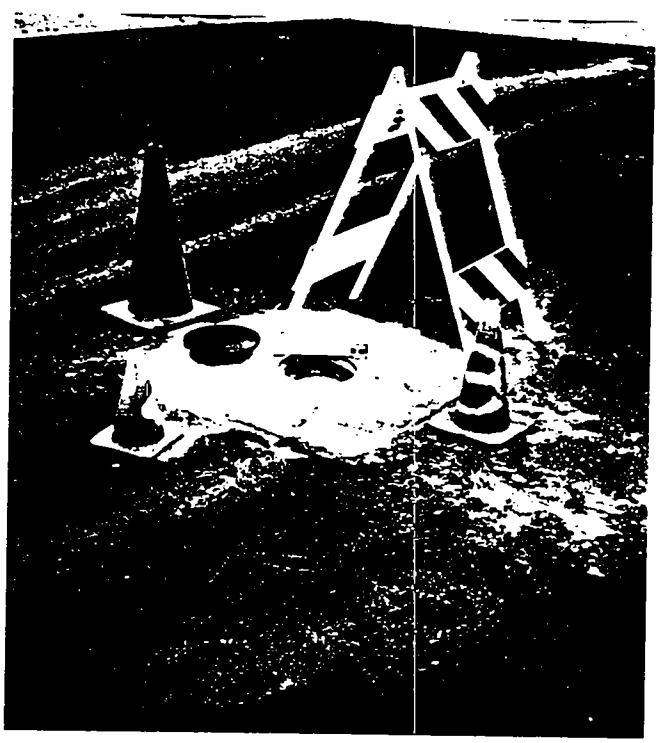

Figure 17: Settlement Monitoring Pipe

References

Francaviglia, Richard, Autumn, 1982, "Copper Mining and Landscape Evolution- A Century of Change in Arizona's Warren Mining District," Journal of Arizona History, Vol. 23, No. 3.

Graeme, Richard, Sept.-Oct., 1981, "Famous Mineral Localities: Bisbee, Arizona," The Mineralogical Record, Vol. 12.

Nations, Dale, Stump, Edmond, 1983, "Geology of Arizona," Kendall/Hunt Publishing Company, Dubuque, Iowa.

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MONITORING OF A HOUSE DURING MINING SUBSIDENCE

G. Lin[1], E.C. Drumm[2], R.M. Bennett[2], L. Powell[3]

Abstract: The structural response and subsequent damage to a residential house were monitored during an active longwall mining operation. The monitoring program consisted of survey monuments on the ground surface around the house and at various locations on the structure. Structural tilt, soil strain and soil pressures were also recorded. The structure was subjected to a cyclic loading consisting of initial tension, then compression, followed by a second tension in the ground surface. The unreinforced foundation experienced extensive cracking and was left with permanent displacements. Although the structure experienced significant shear and tilt during the subsidence, it was left with only minimal damage. The observations suggest that structures in subsidence-prone areas must be able to withstand significant angular distortions, especially during the initial tensile phase of the ground deformation. Current construction practices can be modified to induce composite action between the foundation and superstructure and thus improve the resistance to subsidence-induced ground movements.

Additional Key Words: subsidence, structural damage, foundation damage

Introduction

With the growth in demand for coal, and the resulting increase in coal production, underground mining will be performed under existing structures. In addition, new structures will be built over areas that were mined previously. Mining induced ground subsidence is estimated to result in damage to residential structures costing between 25 and 35 million dollars each year. This damage is estimated to exceed $1 billion over the final quarter of this century (Gray 1988).

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New construction procedures and techniques are needed to reduce structural damage due to mining induced ground movements. However, the structural response to ground movements is very complex due to interaction effects between the ground, foundation and superstructure. It is essential to understand these interaction mechanisms for the development of new damage mitigation procedures.

Although the subsidence above abandoned room-and-pillar mines is very unpredictable, the ground subsidence induced by a longwall mining is generally predictable both in terms of magnitude and timing. As such, longwall mining subsidence provides a unique opportunity to investigate structural response to subsidence.

A one-story ranch-type house in southern Illinois was monitored during longwall mining subsidence. The monitoring program consisted of survey monuments on the ground surface around the house and at various locations on the structure. Structural tilt, soil strain and soil pressures were also recorded. This paper presents some results from the monitoring program and discusses the interaction process between the ground, foundation and

superstructure. Some recommendations are provided in terms of inexpensive modifications to current construction procedures to minimize structural damage during mining induced ground subsidence.

Site and Structure Background Information

Structure

The house was a one-story wood framed ranch-type house constructed over a crawl space. The house has a concrete slab porch at the front and an annex on the back. The main structure has a plan dimension of about 17 m long and 8 m wide. The floor plan and elevation are shown in Figures 1 and 2, respectively. The walls were composed of fiber board and wooden siding over wooden studs. The interior walls were covered with gypsum wallboard. The house appeared to be in relatively good condition prior to the mining subsidence.

The main structure was supported on continuous unreinforced concrete strip footings about 0.6 m wide as shown in Figure 3. Although the footings along the four sides were connected at the corners, they were not at the same elevation. The footings at the south and west sides were poured approximately 20 cm higher than the footings at the north and east sides. A wooden beam, consisting of three nominal 50 by 200 mm boards, ran longitudinally down the center of the house. The beam was supported on six evenly spaced isolated concrete footings of approximately 0.6 m square. Wood joists spanned between the top of the foundation and the main beam. The house was anchored to the foundation with 9 mm L-shaped anchor bolts at 2 m on center. The sill was a 50 mm by 150 mm board with 18 mm holes for the anchor bolts. There was evidence of some rot in the sill.

Site Conditions and Mining Operations

A borehole was drilled to investigate the subsurface materials above the mine seam. It was found that the overburden consisted primarily of shale and siltstone, with some interbedded sandstone and three thin coal seams (Mehnert et al. 1992). The surface soils were loess deposits consisting primarily of low plasticity silts and clay. The soil layer was approximately 7 m thick with the soil having a plasticity index of 10, angle of internal friction

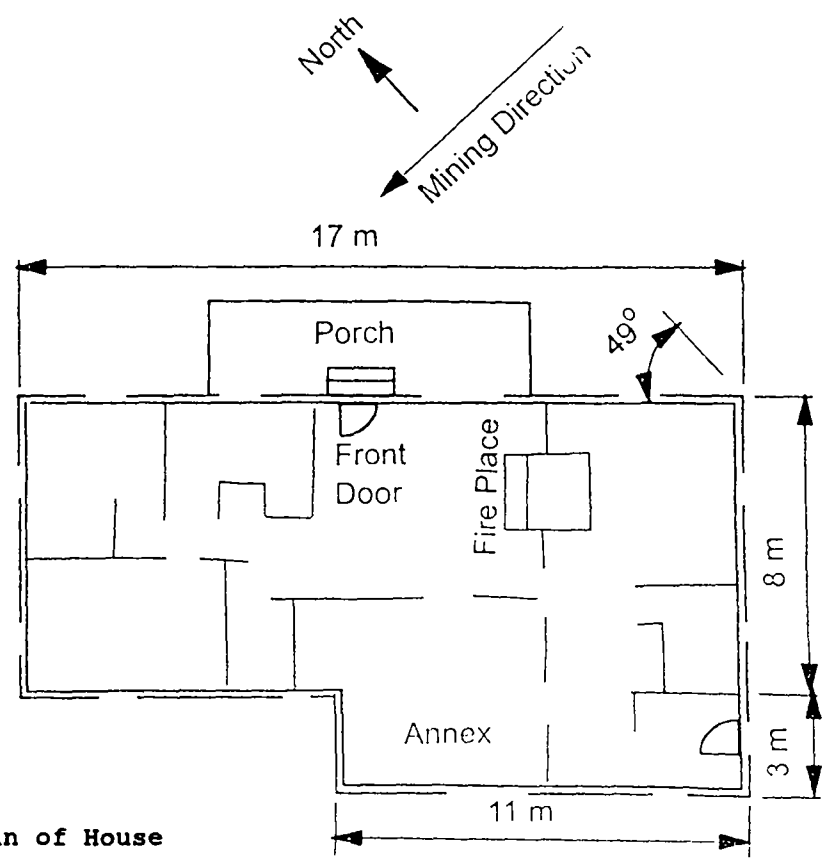
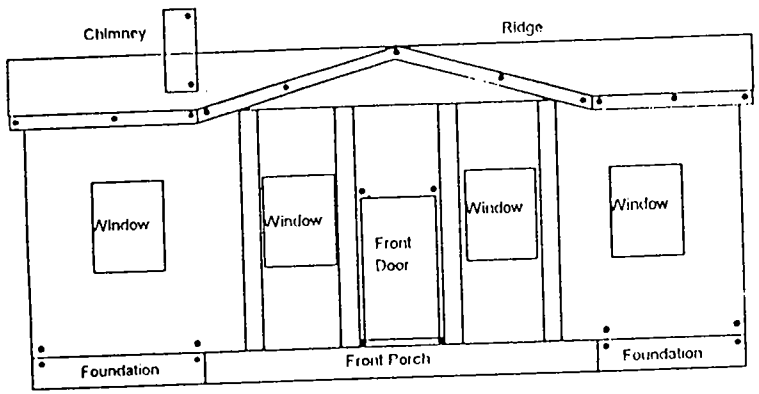
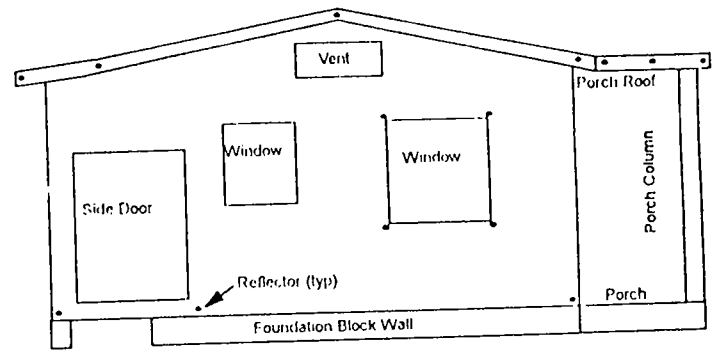


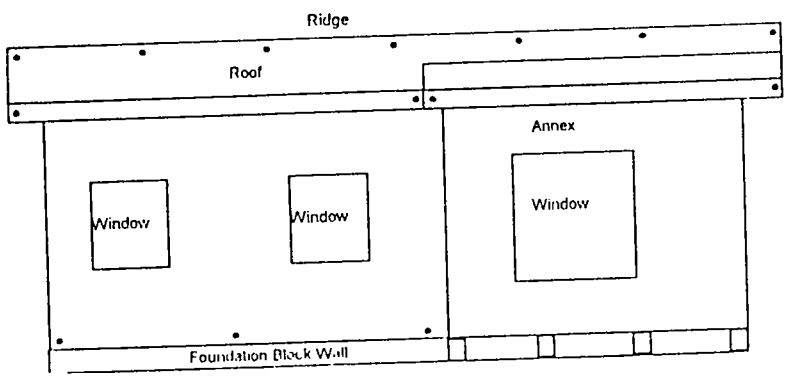
Figure 1 Floor Plan of House



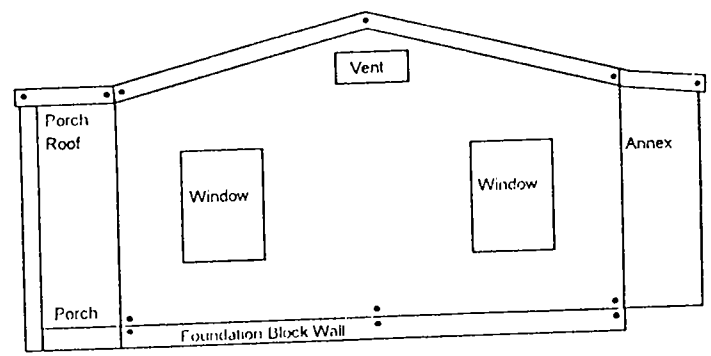
North Elevation



East Elevation



South Elevation



West Elevation

Figure 2 Elevation Views of House

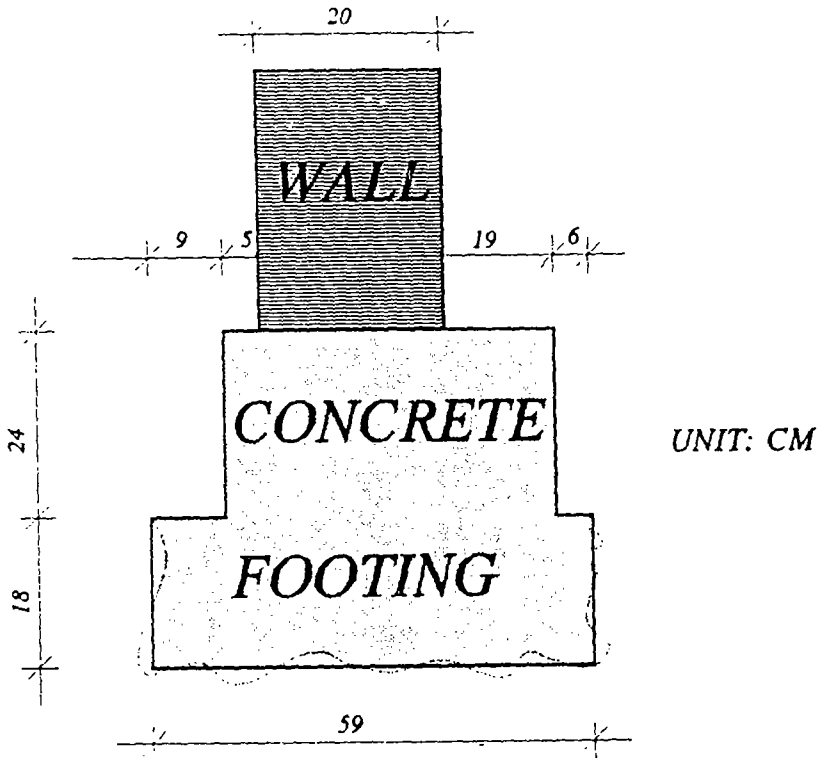


Figure 3 Typical Section through the Foundation

of 35 degrees, a cohesion value of 4.1 kPa and an elastic modulus of 44 MPa (Lawrence, 1992).

The longwall coal mine panel was 280 m wide, with the house being near the center of the panel. The house was oriented at 40.8 degrees with respect to the mining direction (east-west). Therefore, one diagonal of the house was roughly aligned with the mining direction. The panel was supercritical, meaning the subsidence transverse to the mining direction near the house was relatively uniform. The depth of mining was 160 m, with the coal seam being 2.3 m thick. Mining speed averaged 10.7 m/day and mining progressed from east to west. The mining resulted in a moving ground subsidence wave along the surface approximately 100 m wide perpendicular to the centerline of the mining. It was anticipated that the subsidence effects perpendicular to the mining direction would be minimal.

Measured Ground and Structure Movements

Ground Movements

Survey monuments along the access road to the house were monitored during the subsidence event. Figure 4 shows the subsidence profile along the road over a horizontal distance of about 700 m. The relative location of the house is also shown. Figure 4 indicates that a maximum subsidence of about 1.4 meters was measured in the location of the house.

Forty survey monuments were installed on the ground surface around the house. Vertical ground movements were measured with an optical level, and horizontal ground movements measured with an electronic total station survey instrument. The measured vertical ground movements (subsidence) adjacent to the

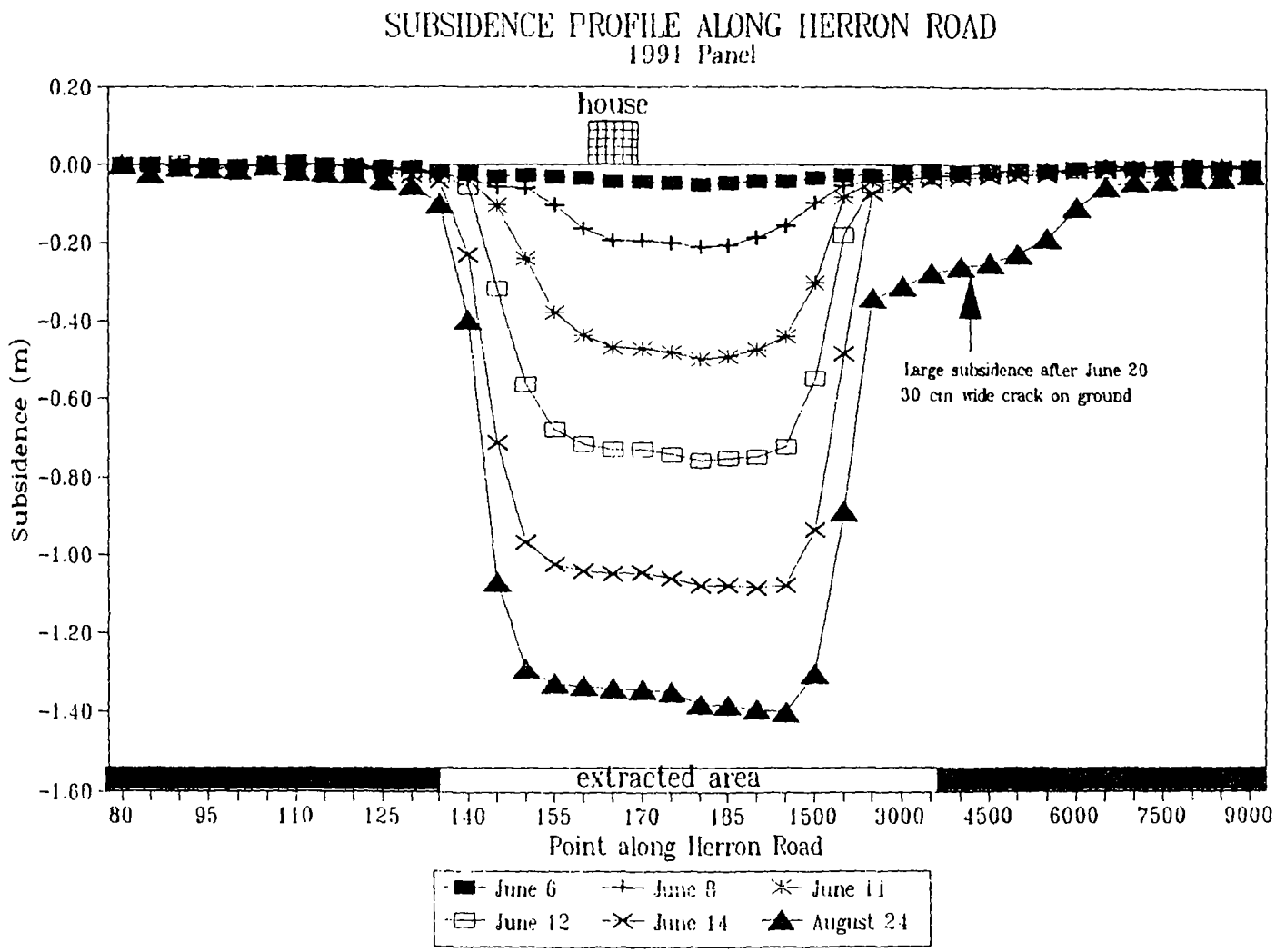


Figure 4 Subsidence Profile Perpendicular to Mining Direction along a 700 m Survey Line

house are presented in Figure 5. The horizontal ground movements adjacent to the house are shown in Figure 6.

Figure 5 depicts ground subsidence along the mining direction and ground deformation at different phases of the subsidence event. The large span in the east half of the curves corresponds to the rigid front porch. The curves indicate that subsidence started at the east end and moved from east to west. The subsidence profiles on June 15 and 16 indicate that the ground subsided considerably more at the east end than at west end, and suggest the east half of the ground was in tension. On June 17, the tension wave on the ground surface reached the western end of the structure, and the eastern half experienced near constant slope. The entire ground around the house was in compression as the subsidence profiles were slightly concave-upward. The ground returned to almost level by June 24, as the mining face moved well beyond the house. The maximum subsidence around the house was approximately 1.2 m, while the the maximum subsidence in the mining panel was approximately 1.4 m. As anticipated, the subsidence around house was caused primarily by the dynamic subsidence moving in the east- west direction; the subsidence perpendicular to the mining direction was negligible.

The horizontal ground movements in Figure 6 were magnified twenty times for clarity. The ground first moved eastward, then moved westward back to the initial position, with the movements in the north-south direction being negligible. Between June 8 and June 16, the ground at the east end moved the maximum amount (0.164 m) while the ground at the west end only moved about one-third of the maximum (0.046 m). This differential movement perpendicular to the mining direction was negligible.

The horizontal ground movements in Figure 6 were magnified twenty times for clarity. The ground first moved eastward, then moved westward back to the initial position, with the movements in the north-south direction being negligible. Between June 8 and June 16, the ground at the east end moved the maximum amount (0.164 m) while the ground at the west end only moved about one-third of the maximum (0.046 m). This differential movement of 0.118 m subjected the house to tension. On June 17, the east end remained almost in the same position while the west end moved to almost the maximum amount (0.139 mm). On June 18 and 19, the east half of the ground started to move back westward while the west part continued to move slightly eastward, subjecting the house to compression.

House Movements

The horizontal and vertical movements of the house were measured with a total station survey instrument by monitoring the position of seventy reflector monuments mounted on the structure (Lin 1993). In addition, a series of structural tilt measurements were obtained with tiltplates, and soil strain and lateral soil pressures were measured. The locations of the tilt plates, soil strain gages, and soil pressure gages are shown on a plan view of the house in Figure 7.

In general, the house experienced the same movement trends as the ground, being subjected to tension (convex-upward bending), compression (concave-upward bending), and returning to a final state with a small amount of residual tension. However, the magnitudes of deformation were different. The magnitude of the vertical movements of the house were very close to the ground subsidence at the adjacent location, but the horizontal movements of the house were typically smaller than the adjacent ground surface. This difference is a result of the interaction between ground, foundation and superstructure.

Due to the orientation of the house relative to the direction of mining, one diagonal of the house was roughly in the mining direction. The subsidence wave traveled from the northeast corner to the southwest corner. The average horizontal strain of the ground, foundation, wall and roof were calculated along the diagonal to demonstrate the transmission of horizontal strain from the ground to the structure. The results in Figure 8 show discrete tension and compression phases. The maximum tensile strain and compression strain are tabulated in Table 1. Table 1 indicates that the maximum tensile strain of the ground was about 1.6 times the maximum compressive strain of the ground. More tensile strain was

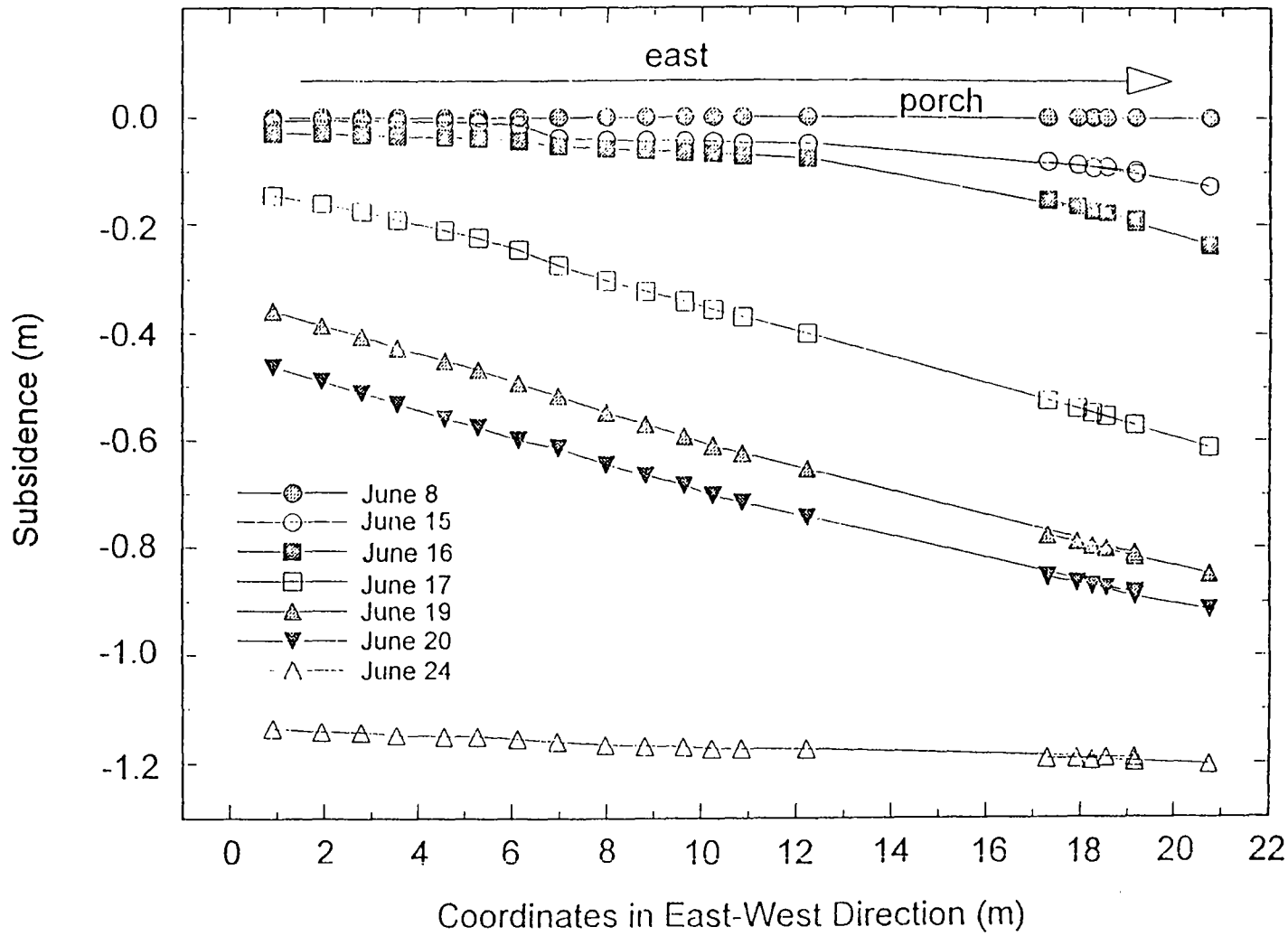


Figure 5 Subsidence of the Ground near the House

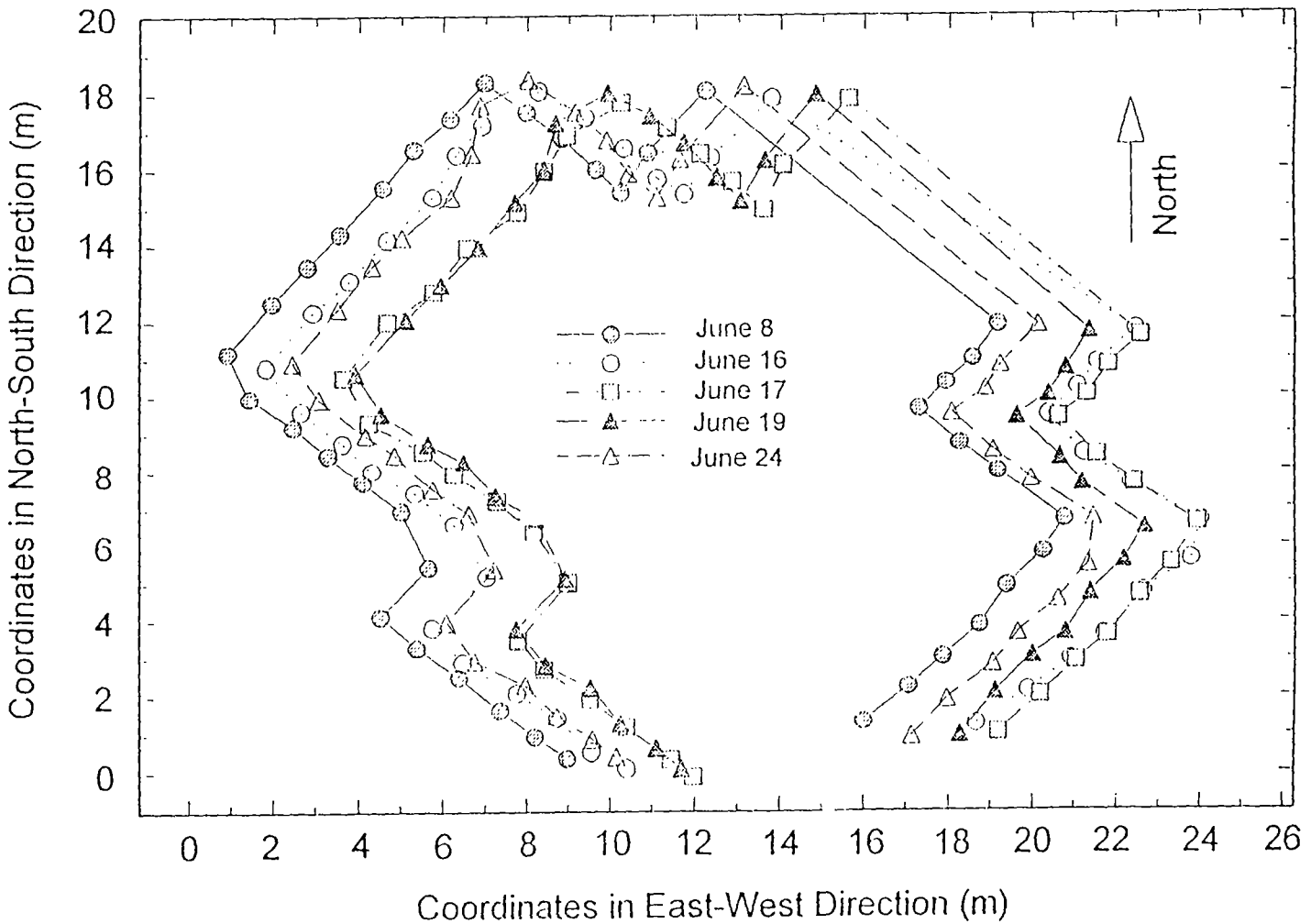


Figure 6 Horizontal movement of the Ground around the House

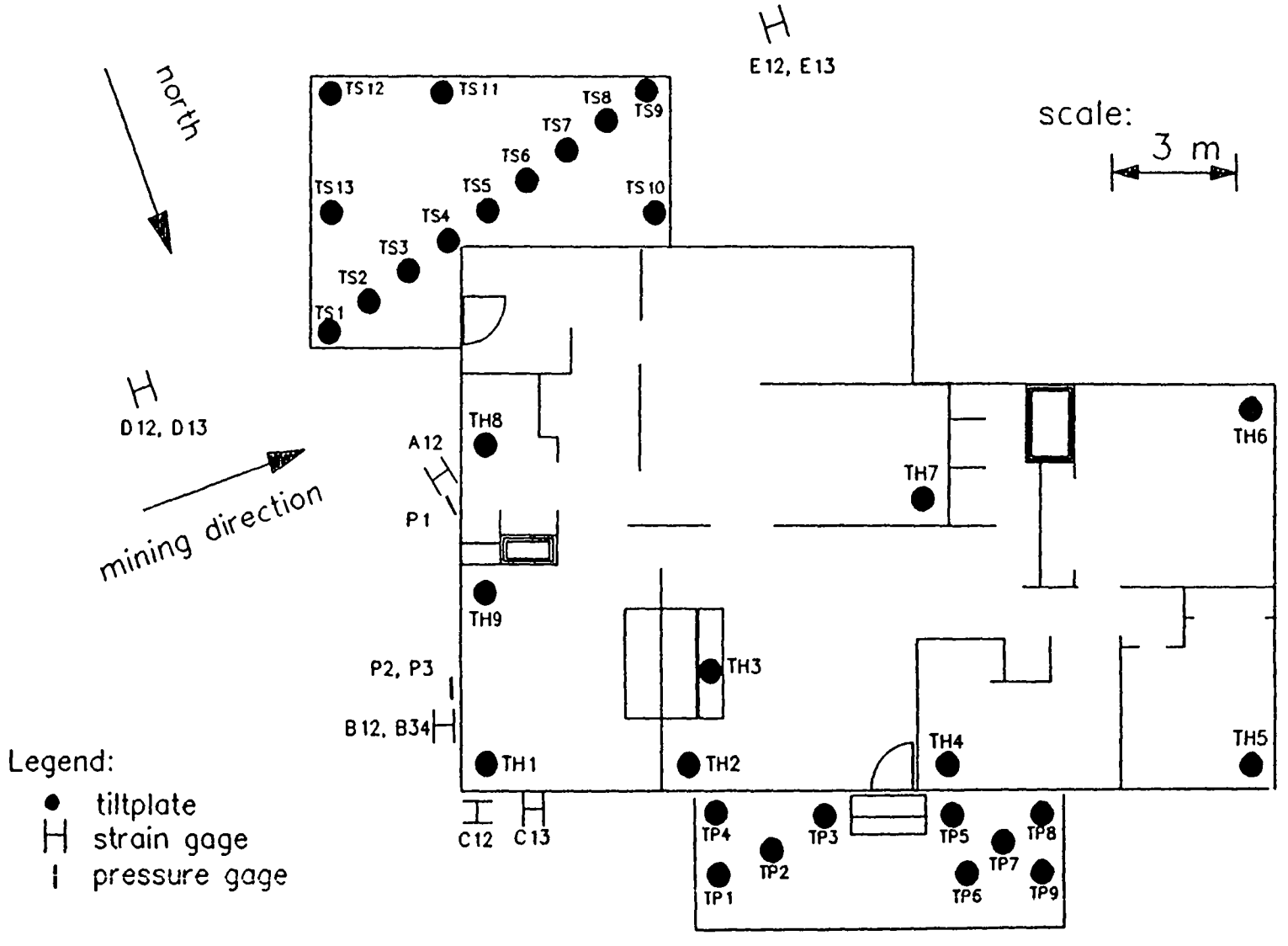


Figure 7 Instrument Locations on the House

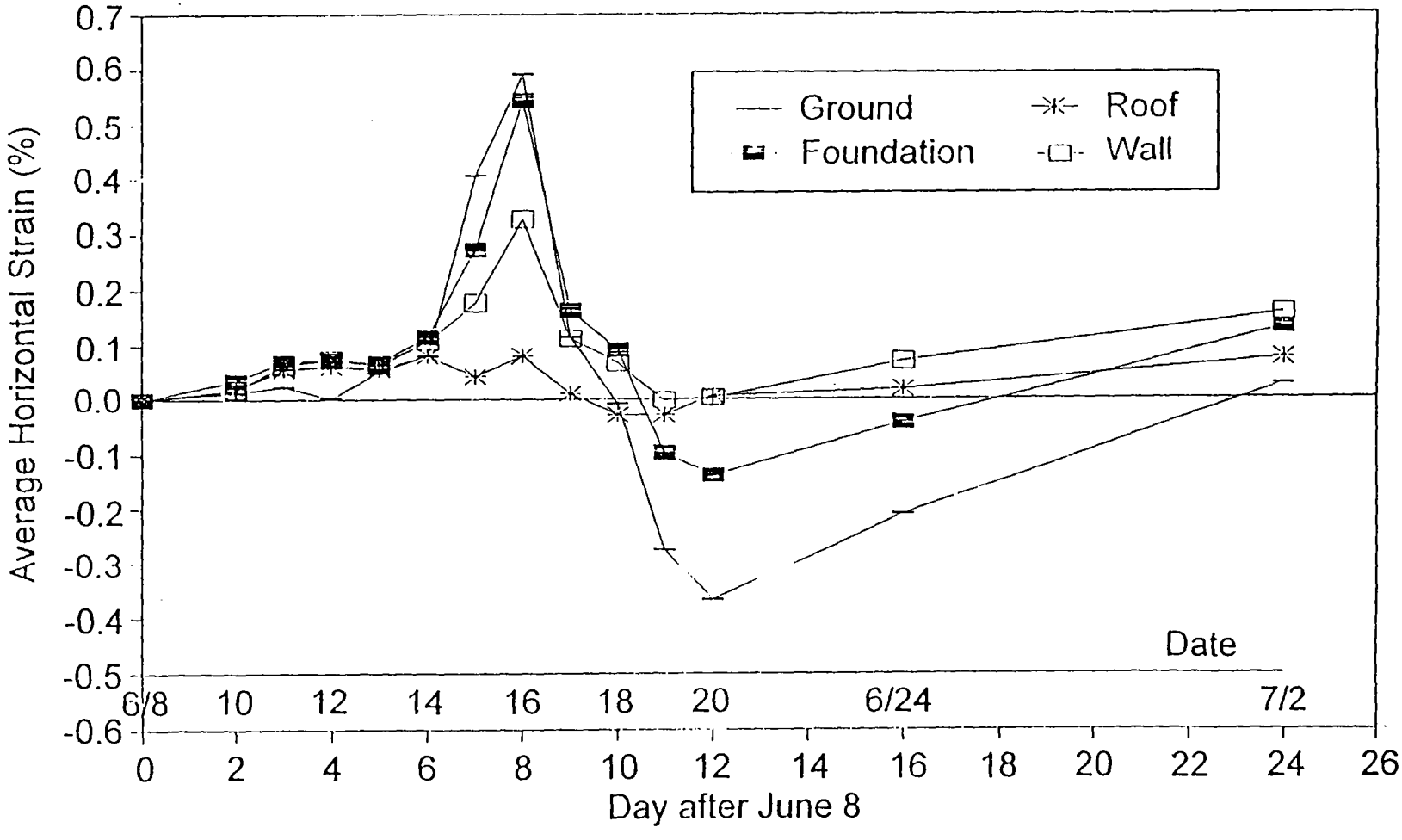


Figure 8 Average Horizontal Strain of the Ground, Foundation, Wall and Roof

transmitted to the superstructure components than compressive strain. The maximum tensile strain of the foundation was very close to the tensile strain of the ground, while the maximum compressive strain of the foundation was only about one-third of the maximum compressive strain of the ground. Both tensile and compressive strains decreased substantially from the foundation to the roof. Both the wall and the roof sustained virtually no compressive strain along the mining direction, which suggests there were relative movements between the foundation and the superstructure.

Since the longitudinal direction of the house was not aligned with the mining direction, the house was distorted during the passing of the subsidence wave. This distortion consisted of both in-plane and out-of-plane deformation of the structural members. The distortion of the house was measured by the angle formed by the north and west foundation walls as listed in Table 2. The wall angle was observed to increase and then decrease, and be left with an overall decrease in wall angle after subsidence.

Structural Tilt

Tiltplates were installed at numerous locations around the floor ofthe house. The tiltplates (SINCO 1985) permit the measurement of structural tilt in two orthogonal directions. Figure 9 illustrates the East-West tilt measured at two locations (tiltplates TH1 and TH6) over a period of 40 days. It is noted that the maximum tilt occurred about June 18 at the eastern most plate, TH1, while the maximum tilt occurred about 2 days later on the western tiltplate, TH6. There was very little tilt measured at either location in the North-South direction perpendicular to mining.

The gradient of the tilt with respect to horizontal distance is the curvature, which is useful for the comparison of structural damage. A linear regression of the tiltplate data was performed to determine the curvature along the diagonal of the structure. Figure 10 shows the curvature of the house floor over time. During the tension phase (about June 17) a convex-upward or negative curvature of 0.0003 m^{-1} was recorded. On June 21, a concave-upward or positive curvature of 0.00015 m^{-1} was recorded. A second negative curvature of about the same magnitude as the first was recorded on July 2.

Table 1 Measured Ground and Structural Strain

	Maximum Tensile Strain	Maximum Compressive Strain
Ground	0.59%	0.37%
Foundation	0.54%	0.14%
Wall	0.31%	<0.1%
Roof	<0.1%	<0.1%

Table 2. Measured Distortion of the House

Distortion of House as Measured by Angle of the north and west foundation wall					
Date	June 8	June 16	June 17	June 18	June 20
Angle (°)	90.01	90.22	90.02	89.93	89.28

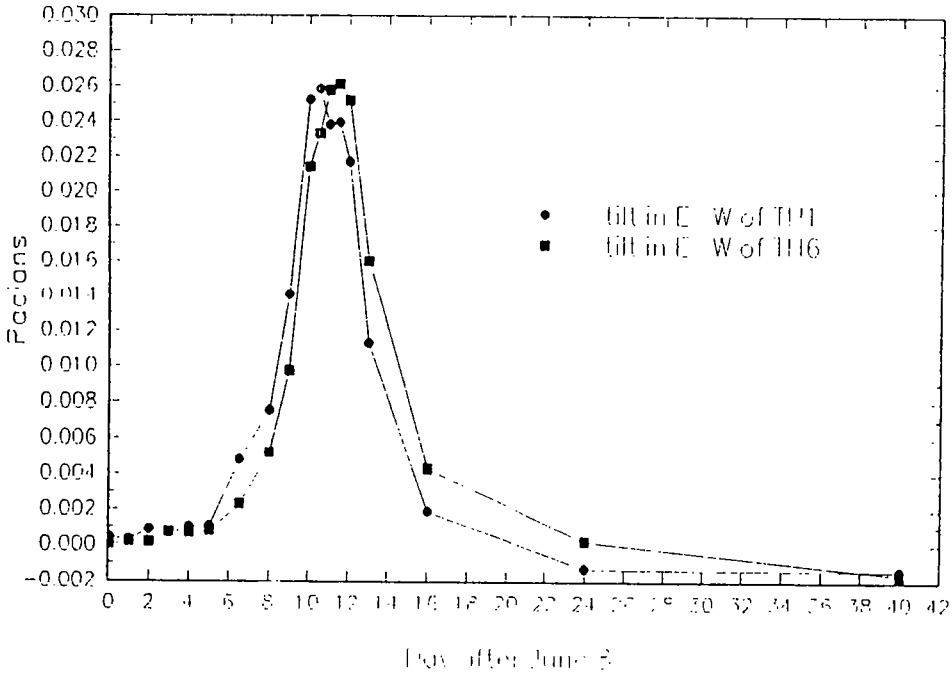


Figure 9 Measured Tilt of the East and West Corners of the House

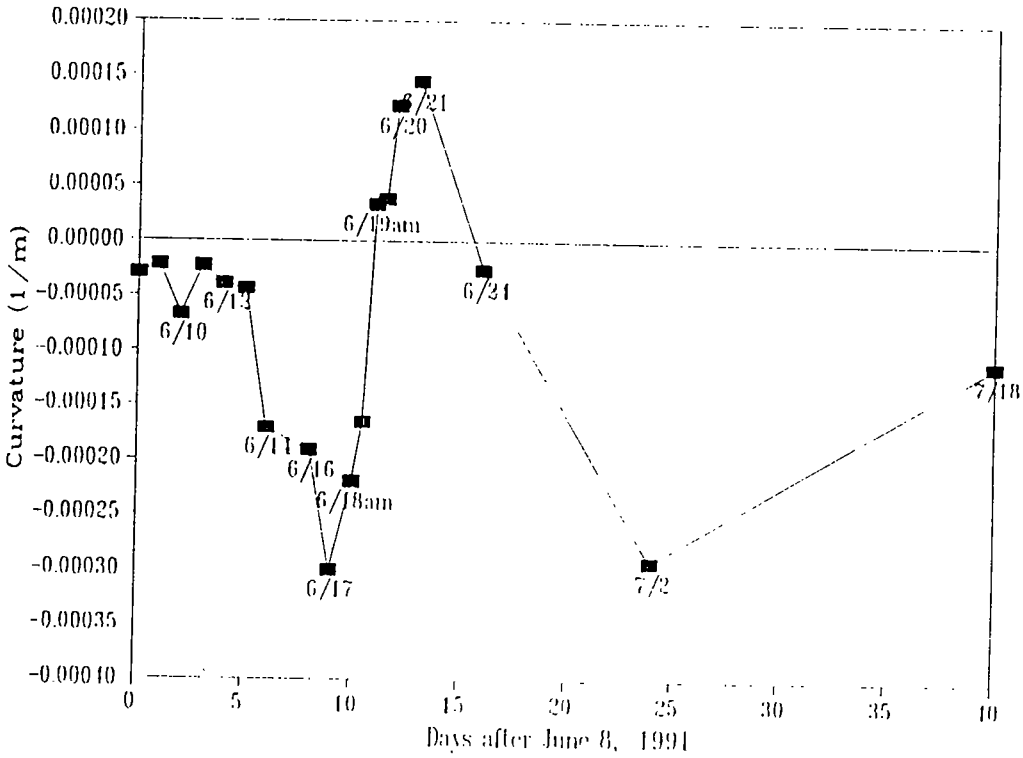


Figure 10 Calculated Curvatures along the House Diagonal

Soil Strain

Bison soil strain gages (Selig 1975; Bison Instruments 1989) were installed at several locations to record the soil strain. The soil strain in the horizontal direction is shown in Figure 11 for four locations. Gages A12, located next to the structure, and gage D12 located away from the structure, were oriented in the direction of mining. Gages B34 and C12 were located adjacent to the structure but oriented relative to the structure as shown in Figure 8. Gage A12 was installed at a depth of 0.6 m, while B34, C12, and D12 were installed at a depth of 0.4 m. The overall soil strain response is similar to the average soil strain over the length of the structure shown in Figure 7, with clear periods of tension (positive strain), followed by compression (negative strain). However, the strain magnitudes are different, and the soil strain gages located adjacent to the foundation were left with residual compressive strain after the subsidence wave passed. The residual compressive strain may reflect the plastic deformation that takes place near the footing during the compression phase. This plastic deformation of the soil is thought to cause the residual tension in the footing after the subsidence is complete. The gage length of the soil strain gage measurements was typically about 100 mm, which is much smaller than the gage length of nearly 20 m used to calculate the soil strain in Figure 7. Therefore, differences in the measured strains are not unexpected.

Soil Pressure

Soil pressure gages were installed to record the lateral earth pressures during the subsidence event. Lateral earth pressures can cause significant damage, especially to structures with basement walls. The soil pressure gages were located between the foundation wall and the surrounding soil. As shown in Figure 8, the pressure gages were oriented with the Bison soil strain gages. The recorded change in soil pressure during the subsidence event is shown in Figure 12. Although the magnitude of the compressive pressure differs from gage to gage, the trend of the pressure variation is similar. A slight negative change or reduction in lateral soil pressure was recorded early in the subsidence event, as the tension wave approached the gage (structure). About June 18, a compressive peak in pressure was recorded corresponding the arrival of the compression wave. This peak was followed by a slight tension period before the pressure returned to nearly that which existed prior to subsidence. The magnitude of the peak compressive stress recorded in the three gages ranged from about 10 kPa to nearly 25 kPa. This is likely a result of differences in the manner in which the gages were installed or seated. In all cases however, the measured stresses were well below the theoretical passive earth pressure which is the maximum pressure that the soil could resist assuming that the wall did not fail. For example, the 25 kPa pressure recorded at gage P2 which was located at a depth of 0.55 m, can be compared to a Rankine passive earth pressure of 35 kPa. This suggests that even though the 25 kPa soil pressure is significant, it is well below the theoretical upper limit.

Since soil pressure gage P1 and soil strain gage A12 were installed at nearly the same location, the results can be combined to produce a lateral stress-strain response curve for the soil as shown in Figure 13. In this curve, the initial tensile strain and pressure are shown as positive, with the subsequent compression loading shown as negative. These instruments recorded much larger strain and stress during the compression phase than during the tension phase.

Structural Damage

As a result of the ground movements and deformation, the house suffered two types of structural damage: cracking of the concrete footings and block walls, and distortion of the superstructure. The development of cracks in the block wall was recorded during the subsidence. The cracks on the concrete footings were mapped after the subsidence event. The superstructure damage is quantitatively described in terms of strain, tilt and angular distortion.

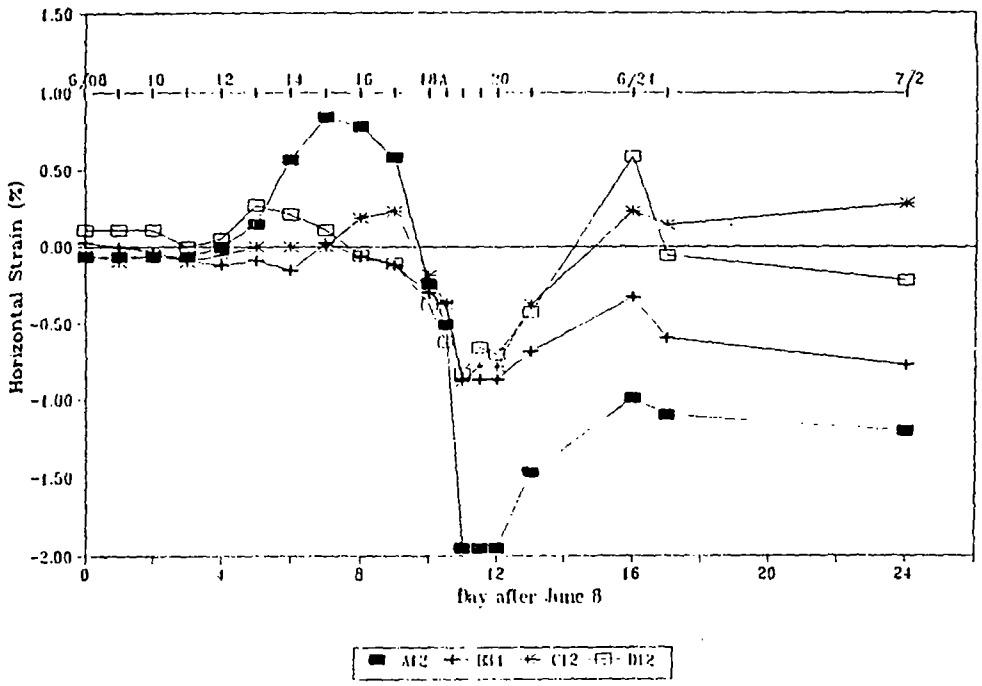


Figure 11 Measured Soil Strain over Time

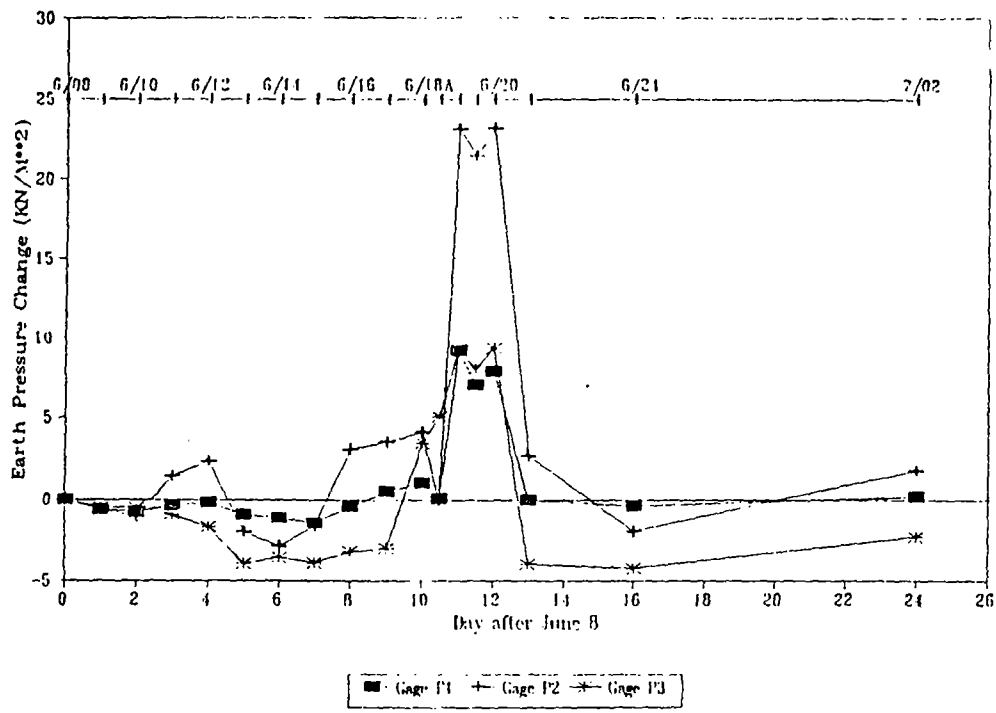


Figure 12 Change in Lateral Soil Pressure over Time

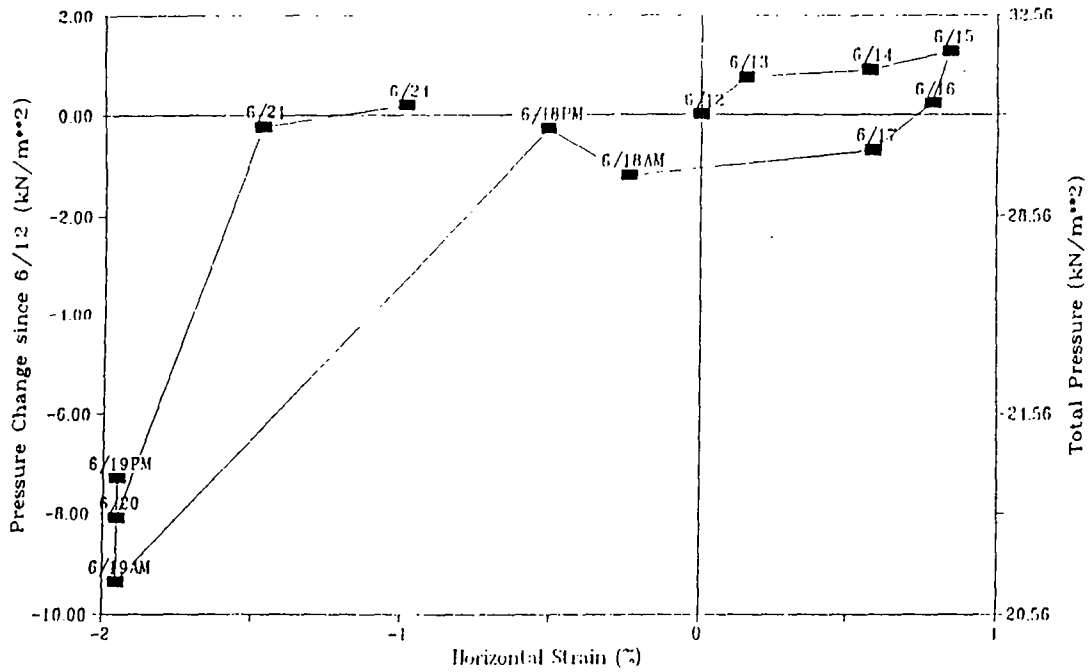


Figure 13 Soil Stress-Strain response during Subsidence

Figures 14 and 15 show the location and residual crack width in the concrete footings and block walls. The residual crack width was considerably smaller than the maximum crack openings developed during the tension phase of the subsidence event, and ranged from 3 mm to 10 mm. The cracks typically extended up through the masonry wall, generally in a stair step fashion and slightly increasing in width. No cracks were observed in the east side footing, but a large portion of the footing was covered by a utility box. Cracks were first found on the east side of the masonry wall on June 15, and by late afternoon six cracks were found in the masonry wall.

The development of the crack on the east section of the front wall, Figure 14, can be used to illustrate the effects of subsidence on the house. This crack started on June 15 and by the morning of June 16, the blocks on the sides of the crack moved apart about 25 mm both in the direction of the wall plane and perpendicular to the wall plane. On the morning of June 17, the cracks closed to about 10 mm in both the in-plane and out-of-plane directions. The crack decreased to about 2 mm in both directions on June 18. Subsequently, the crack reopened in the wall plane and had a residual width of about 10 mm. This process of tension, compression and a second tension has been described previously (Geddes 1977) and was also observed in a nearby series of test foundations (Lin et al. 1994). Due to the high compressive stiffness of the structure and yielding of the soil, the full compressive deformations from the soil
are not induced into the structure. As the ground returned to a state of zero tension, it induced a second tension in the structure.

Additional damage to the masonry wall was observed where the center beam rested on the wall. During the compressive phase of the subsidence, the beam pushed two blocks out about 10 mm on both the east and west sides, causing some local crushing of the block. This is indicative of the relative movements between the foundation and superstructure.

Cracks were also observed on gypsum wallboard inside the house. These cracks typically developed at an about 45° angle from the corner of openings such as windows and doors. These cracks were

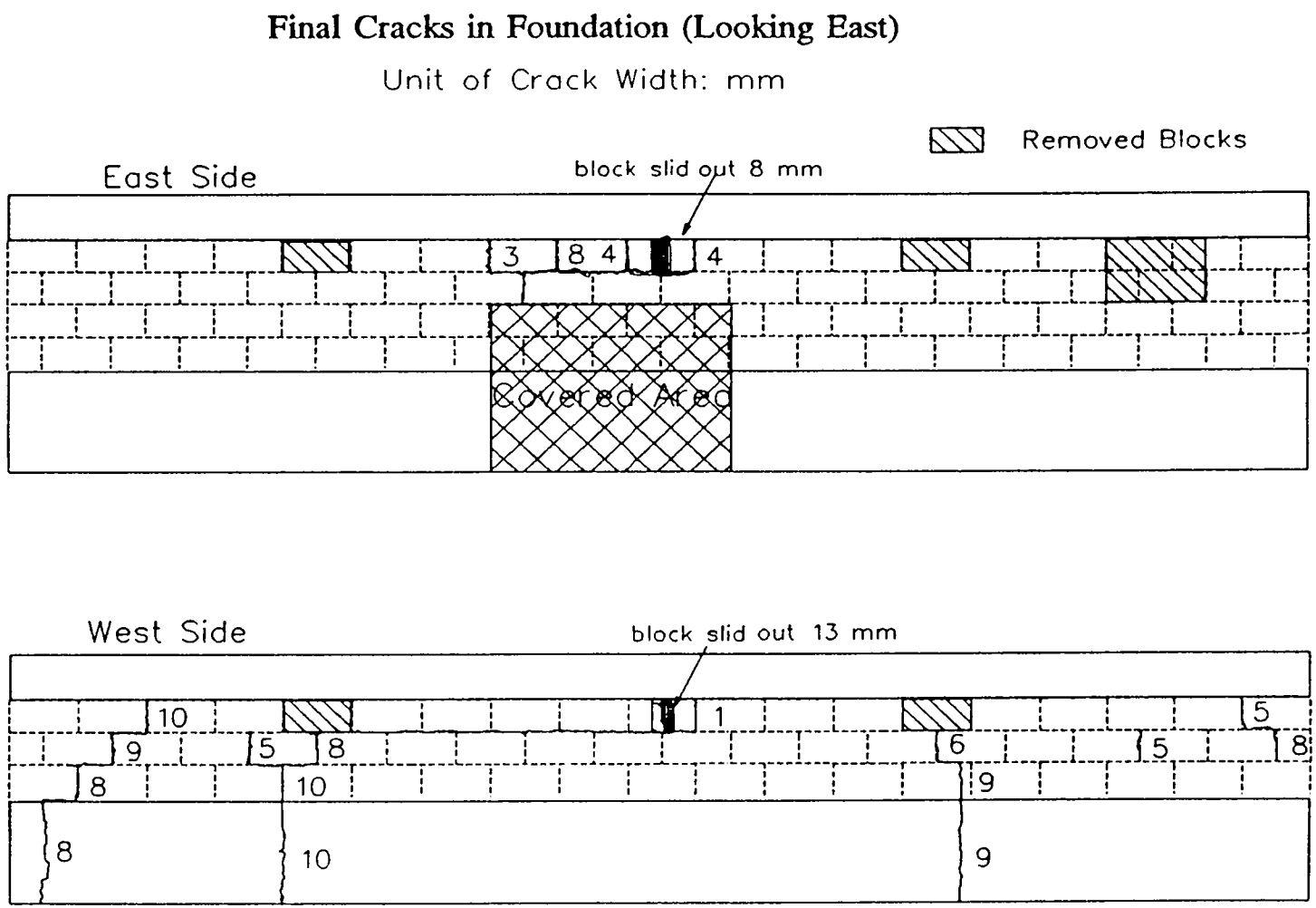
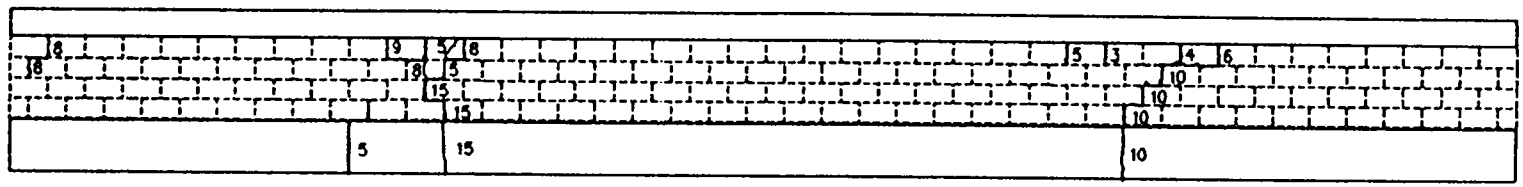


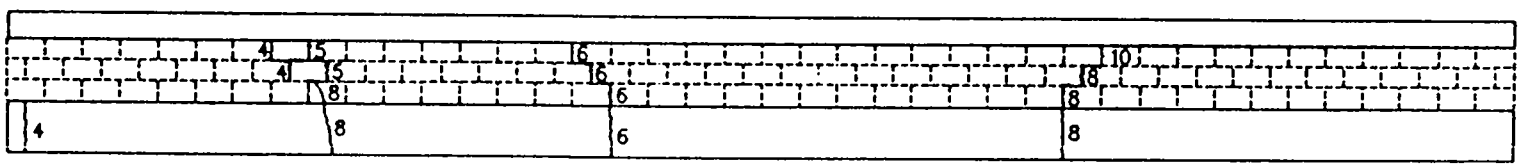
Figure 14 Location and Widths of Final Cracks (Front and Rear of House)

Final Cracks in Foundation (Looking North)

Unit of Crack Width: mm



Front Side



Rear Side

Figure 15 Location and Widths of Final Cracks (East and West Sides of House)

typically very small in width and extended less than 300 mm.

It appears that the superstructure damage was primarily due to differential vertical movements, and secondarily due to horizontal movements. The structural deformation and damage were quantitatively described in terms of angular distortion. Angular distortion is the change of slope between two points divided by the horizontal distance between the two points. Figure 16 shows the angular distortion of the ground, foundation wall, and house wall immediately above the foundation wall. The angular distortion decreases moving from the ground up to the foundation, and continues decreasing with elevation into the structure. The change in angular distortion is indicative of interaction between ground, foundation and the superstructure.

The calculated angular distortions are relatively consistent with the limits of angular distortion suggested by others corresponding to certain levels of structural damage. Bjerrum (1963) suggested an angular distortion of 1/500 as a safe limit for no cracking. Cracking was observed in the house foundation when this limit was exceeded. Marino (1985) suggested that a house with a crawl space that had an angular distortion exceeding 1/208 would require foundation repair. Once the house foundation cracked, it essentially followed the ground movements, resulting in high angular distortion of much greater than 1/208 and complete failure. Wahls (1994) indicates that cracking for plywood or fiberboard on wood frames occurs at an angular distortion between 1/60 and 1/170. Angular distortions greater than 1/170 were observed during the investigation, but only minimal cracking was noticed. However, serviceability problems such as inoperable doors were experienced.

Conclusions and Recommendations

The foundation of the monitored house cracked in numerous places during the mining subsidence. Therefore, extensive repair and replacement work would be required before the house could be returned to normal use. Despite the extensive damage to the foundation, the superstructure of the house suffered minimal damage. The superstructure damage was primarily serviceability issues such as sticking doors and

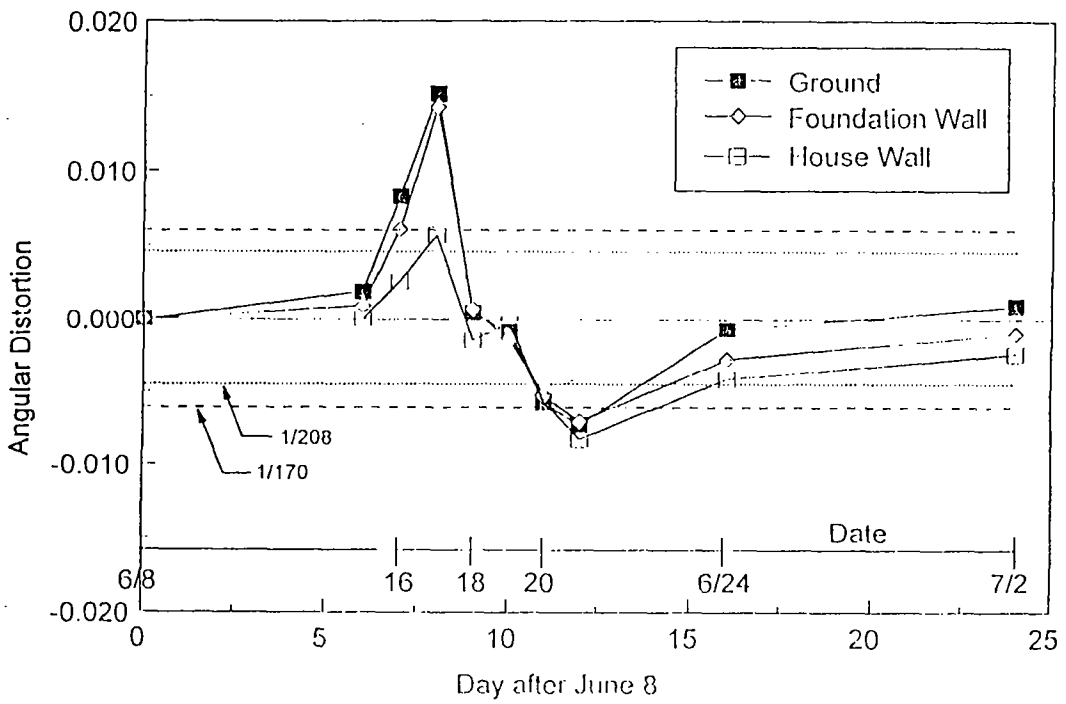


Figure 16 Angular Distortion of Ground, Foundation Wall, and House Wall

aesthetic problems such as cracking of the interior gypsum-board wall. The wall deformations were mainly in-plane shear and out-of-plane movements resulting from twisting of the house.

The house approximately followed the nearby ground subsidence in the vertical direction. Measurements of strain over the length of the house suggest that the foundation experienced tensile strains close to those in the ground during tensile period, resulting in cracking of the foundation and wall. However, during the compression period, the foundation cracks closed and the house frame distorted, resulting in foundation strain that was only about one-third of the strain in the ground. The foundations and walls experienced both in-plane and out-of-plane deformation but the walls appeared more vulnerable to the out-of-plane deformation. The deformation in the foundation was transferred to the superstructure primarily through the mechanism of shearing, and its magnitude decreased from foundation to roof.

The minimal damage to the house superstructure can be attributed to the lack of any significant connection between the superstructure and the foundation. Although this lack of connectivity between the structure and foundation tended to reduce the damage, this cannot be recommended as an damage mitigation technique. Good construction practice dictates anchoring the structure to the foundation to resist lateral loads due to wind and earthquakes, even if the anticipated lateral loads are small.

The purpose of a foundation is to transfer structural load to the ground and to minimize settlements. In an area subject to mining-induced settlements, there are several other considerations during foundation design, most at very little additional construction cost. Damage mitigation schemes for mining subsidence or other similar types of ground movements should be directed towards strengthening the foundation and improving the connection between the foundation and the superstructure. Reinforcing steel and post-tensioning were proven to be effective in minimizing foundation cracking or controlling crack widths in a series of test foundations constructed over a similar mining panel (Lin et al. 1995). By reinforcing and tieing the basement/crawl space walls and possibly the superstructure walls to the foundation so that they act compositely, a stiff and strong beam is formed to resist the ground movements. Additional research is being conducted to further develop this concept.

Mine subsidence results in a dynamic subsidence wave that moves along the ground surface. This wave produces three phases deformation for a structure on the surface: a) convex-upward bending or tension phase, b) concave-upward bending or compression phase, and finally c) a second tension or return to near horizontal configuration. In general, the ground returns to a near zero state of stress in the third phase, but the structure may be left with some residual tension.

When analyzing structures for subsidence deformation, it is important to keep in mind that due to soil-structure interaction effects, the ground deformations in the vicinity of the structure are smaller than those away from the structure. Therefore, a structure need not resist the full magnitude of ground deformation measured during subsidence event. The degree of soil-structure interaction depends upon the relative stiffness of the soil and foundation. The vertical deformations are believed to cause the structural damage, with horizontal deformations serving to open up any cracks that form due the vertical deformation.

Acknowledgments

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References

Bison Instruments (1989). Instruction Manual for Soil Strain Gages, Minneapolis, Minnesota.

Bjerrum, L. (1963). "Allowable Settlement of Structures." *Proceedings* European Conference on Soil Mechanics and Foundation Engineering, Weisbaden, 3, 135-137.

Geddes, J.D., (1977). "The Effect of Horizontal Ground Movements on Structures." *Proceedings*, Conference on Large Ground Movements and Structures, Halstead Press, New York, N.Y., 623-646.

Gray, R.E. (1988). "Coal Mine Subsidence and Structures" *Mine Induced Subsidence and Structures: Effects on Engineering Structures*, Geotechnical Special Publication No. 19, ASCE, New York, N.Y. 69-86.

Lawrence, T.S. (1992). "Soil Testing and Numerical Modeling of a Subsidence Event." Thesis submitted in partial fulfillment of the requirements of the degree of Master of Science, The University of Tennessee, Knoxville, TN.

Lin, G. (1993). "Structural Damage Mechanisms Associated with Differential Ground Movements." Dissertation submitted in partial fulfillment of the requirements of the degree of Doctor of Philosophy, The University of Tennessee, Knoxville, TN.

Lin, G., Drumm, E.C., Bennett, R.M., Powell, L., and Triplett, T. (1994). "Soil-Structure Interaction During Mining Induced Ground Movements." *Vertical and Horizontal Deformations of Foundations and Embankments*, ASCE Geotechnical Special Publication No. 40, 786-797.

Lin, G., Bennett, R.M., Drumm, E.C., and Triplett, T.L. (1995) "Response of Residential Test Foundations to Large Ground Movements,"ASCE *Journal of Performance of Constructed Facilities*, Vol 9, No. 4, pp 319-329.

Marino, G.G. (1985). "Subsidence Damaged Homes Over Room and Pillar Mines in Illinois." Dissertation submitted in partial fulfillment of the requirements of the degree of Doctor of Philosophy, The University of Illinois, Urbana-Champaign, IL.

Mehnert, B. B., Van Roosendaal, D.J. and Kawamura, N. (1992) "Overburden Characterization for Heron Road Subsidence Site." Final Report to U.S. Bureau of Mines, P0213293, Illinois State Geological Survey, Champaign, Ill.

Selig, E.T. (1975) "Soil Strain Measurement using Inductance Coil Method" *Performance Monitoring for Geotechnical Construction*, ASTM STP 584, ASTM, Philadelphia, Pennsylvania, 141-158.

SINCO (1985) "Geotechnical, Geophysical, Groundwater and Structural Instrumentation" Slope Indicator Company, Seattle, Washington.

Wahls, H.E. (1994). "Tolerable Deformations." *Vertical and Horizontal Deformations of Foundations and Embankments*, ASCE Geotechnical Special Publication 40, 1611-1628.

USING GIS AND NUMERICAL MODELING TO ASSESS SUBSIDENCE OVER ABANDONED MINES

by

Kevin M. O'Connor, John A. Siekmeier, and Jan Stache

Abstract. The U. S. Bureau of Mines (USBM) has been developing techniques to assess surface subsidence over abandoned coal mines. One technique has involved estimation of stress acting on every pillar of an abandoned mine then comparing it to pillar strength and floor bearing capacity. This required computations for several thousand pillars for one mine. Mine maps are digitized and saved as a computer drawing file. Then the tributary area loading each pillar was determined graphically and outlined. Geographic Information System (GIS) software was used to compute the ratio of pillar area to tributary area for each pillar and then divide the average overburden stress by this ratio to compute an estimated pillar stress. Numerical modeling was then used to analyze a two-dimensional cross section of the overburden and mine, and provide an independent estimate of stresses. Based on published data for floor bearing capacity and pillar load capacity, GIS was used to perform a mine-wide classification of pillars according to stress level. An example of this analysis and classification is presented in this paper for an abandoned coal mine in the Illinois Basin. The mine had been operated in the Herrin No. 6 Seam at a depth of 60 m with an average overburden stress of 1.4 MPa. It was found that pillars with estimated stresses greater than 5 MPa correlated with historical subsidence events. Due to the greater detail considered in this approach, it provides a fundamental basis for the assessment of subsidence risk since it incorporates the geometry of mine pillars and entries as well as the ultimate strength of the pillars and floor.

Additional Key Words: pillars, stress, animation, CAD

Introduction

Coal mining was active in Collinsville, Illinois from 1870 to 1964, and the area is underlain by a network of mine openings. Support for the overlying rock is provided by stable pillars and blocks of coal as well as those that may be failing or possibly punching into the underlying claystone. As a result of failures within the abandoned mines, there have been localized mine subsidence occurrences throughout the city. Movement of the overlying rock, and ultimately the surface, has subjected structures, streets, and utilities to strains and stresses that have caused damage. Occurrences of subsidence and damage have been regarded as random events. Current research indicates that this may not be the case and rational assessment of subsidence potential may be possible.

Subsidence characterization over abandoned room-and-pillar mines remains relatively undeveloped due to a lack of understanding about time-dependent rock mass behavior at mine level and within the overburden. The effectiveness of early warning technology inherently requires that the timing and extent of a subsidence event can be anticipated. In the case of important structures located over abandoned mines, it is not adequate to say that subsidence may occur. A means must be provided to indicate if strata movements are occurring (and the rate at which they are occurring)

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[1]Paper presented at the 13th Annual National Meeting of the American Society for Surface Mining and Reclamation, Knoxville, Tennessee, May 18-23,1996.

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beneath these structures so that appropriate measures can be taken to mitigate damage. One component of research being conducted by the USBM has involved monitoring of subsurface movements which are precursors of surface subsidence. This paper concentrates on another component involving the use of historical subsidence data and mine maps to identify areas over abandoned mines with a high likelihood of subsidence occurrence.

The USBM in cooperation with the Illinois Mine Subsidence Insurance Fund (IMSIF), Illinois Abandoned Mined Land Reclamation Council (AMLRC), and Collinsville Community Unit School District No. 10 (CUSD #10), has implemented these techniques to allow proactive land use planning over abandoned mines. This joint project was initiated because of subsidence damage to one school, the Dorris Elementary School (figure 1 and figure 2). It will be replaced by a new building and two sites are being considered. One site is an athletic field adjacent to the existing school. The alternative site is an athletic field adjacent to the Lincoln Elementary School (figure 2). Both sites are undermined so it is a situation in which the school district must assess the relative risk of future subsidence.

The USBM conducted a program of site characterization involving core drilling, downhole camera inspection of mine conditions, monitoring of subsurface movements, and data analysis. This paper focuses on efforts to estimate pillar stresses and correlate occurrences of historical subsidence with pillars that are highly stressed. This effort was undertaken using two independent approaches. The first approach involved GIS analysis of the geometry of pillars and entries to estimate stresses assuming tributary loading on pillars (Whittaker and Reddish, 1989). The second approach involved numerical modeling of a two-dimensional cross section of the abandoned mine and overburden to estimate the distribution of stress.

Geometric and Subsidence Data Collection

The project sites are located in Collinsville (figures 1 and 2) which has been extensively undermined. Historically, the old high school and Dorris Elementary School have been damaged by subsidence. There has been no evidence of subsidence at the Lincoln Elementary School.

Available data which were collected for this projected included: 1) U.S. Geological Survey (USGS) 7.5-minute Collinsville quadrangle map, 2) mine maps, 3) subsidence survey data, 4) drilling logs, 5) structural damage reports, and 6) Madison County plat map and orthophoto overlays. Maps were assembled and digitized for the following mines (ISGS, 1991):
Bullock Mine, operated 1899 - 1946;
Heinz Bluff Mine, operated 1883 - 1907;
Huntley Bluff Mine, operated 1886 - 1887;
Lumaghi Mine No. 1, operated 1883 - 1903;
Donk Bros. Mine No. 1, operated 1900 -1922.

Surface subsidence has been monitored by the IMSIF and the AMLRC (Gibson and Schottel, 1990) in response to occurrences of subsidence damage. Collectively, ninety-eight benchmarks in the area of Dorris School and the old high school have been monitored at various times during the period from February 1984 to August 1994. This monitoring has been performed in response to claims of subsidence damage so it was started after "events" had affected the ground surface. Furthermore, data were collected as a series of nets with local reference systems that were not tied to a common origin.

All surface feature and mine maps were digitized and assembled as a drawing file using computer-aided drafting (CAD) software. This allowed all spatial data to be assembled, viewed, and plotted with a common origin, reference system, and graphical scale. With the survey monuments now tied to a common reference, it was possible to assign global coordinates to each point. A drawing of the Dorris Elementary School with surveyed benchmark locations was obtained from the AMLRC. Drawings and survey elevation measurements of monitored benchmarks and structures in the surrounding area were obtained from the IMSIF. The mine maps and drawings were scanned using DeskScan II, then digitized using AutoCAD and assigned to different drawing layers. The historical survey data from both AMLRC and IMSIF were entered into a spreadsheet.

The digitized base map was created using section corners and road intersections as control points for positional accuracy of the mine and subdivision drawing layers. This made it possible to register the map with USGS 7.5 minute Collinsville quadrangle digital line graph (DLG) features and the Universal Transverse Mercator (UTM) projection coordinate system. Road alignments, property boundaries, and the addition of cultural features (baseball field, power poles, fences) for the two school sites were entered from overlay maps obtained

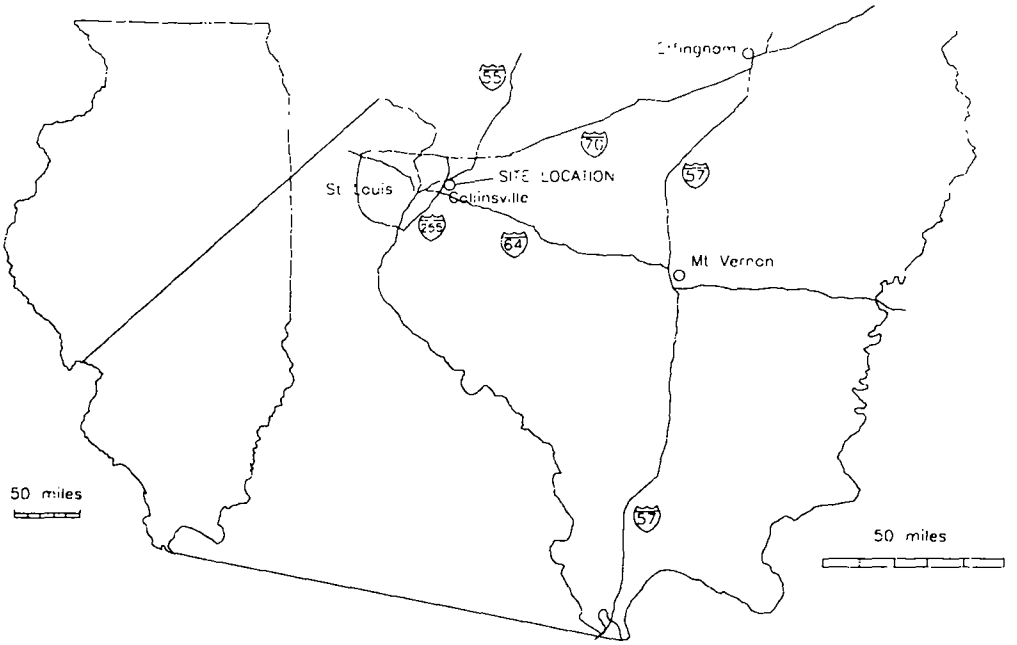

Figure 1.-Site location in Illinois.

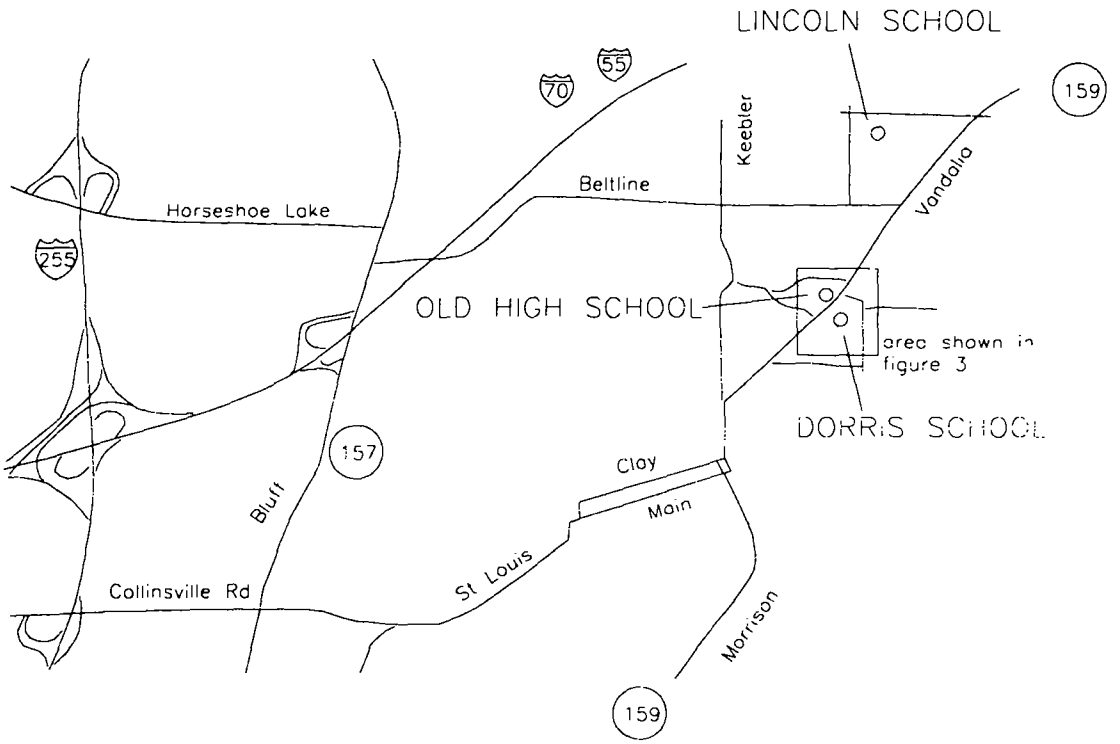

Figure 2.-Site location in Collinsville, Illinois.

from the Madison County Maps and Plats Department. These maps display property boundaries as an overlay with aerial photography at a scale of 1:1200 (1 in.= 100 ft).

Map scale errors may have been introduced or compounded as a result of scanning or digitizing copies of the maps used for data entry. USBM researchers recognize that accuracy of the base maps is an approximation and therefore use the maps to assess general tendency correlations between structural damage and ground surface curvatures (Triplett et al, 1992), and as a tool for display of spatial analyses. However, the elevations of subsidence monitoring points used to build the research database were measured using modern survey techniques with a precision of ±1.5 mm (±0.005 ft).

Initial survey data were used to establish baseline elevations and subsequent surveys were used to calculate vertical displacement. Subsidence contours for each date were generated using a statistical technique known as kriging available within SURFER for Windows. The subsidence contour lines were then imported into AutoCAD and contours for each date were placed on separate layers. Figure 3 displays the last date of survey measurements available overlaid on the mine map. The first contour represents 0.05 m (0.16 ft) of subsidence and the contour interval is 0.025 m (0.08 ft).

Analysis of Subsidence Occurrences

By selectively activating different drawing layers, it was possible to generate a series of slides showing subsidence contours for each date that measurements were made overlaid with the mine geometry. The slides were compiled as an animation to enhance the changes in subsidence contours as a function of time. The animations allowed an assessment of subsidence activity and its relation to underlying mine geometry. This was critical since mine level changes are the cause of subsidence and time is a major factor when considering subsidence over abandoned mines. With this visual aid it was possible to qualitatively assess: 1) the rate of surface subsidence, 2) the rate of lateral progression, and 3) the correlation with mine geometry.

Based on the computed subsidence contours, it was hypothesized that the extent of pillars associated with subsidence for each date must be within the limits of the contours. This hypothesis is based on experience (Hunt, 1980) and the conventionally accepted geometrical relationship between subsurface openings and surface subsidence (Whittaker and Reddish, 1989; Kratzch, 1983). The expansion of surface subsidence bowls can be attributed to increasing mine level span widths. The plot in figure 3 illustrates how the bowl width is controlled by the position of pillars at mine level. However, the bowl depth (i.e., maximum subsidence) is controlled by several factors including the magnitude of convergence at mine level and the overburden stratigraphy.

Estimation of Pillar Stresses

In figure 3, it can be seen that the length and width of pillars are on the order of 20 m (64 ft) by 6 m (18 ft) beneath the Dorris School (center bowl). By contrast, they are 17 m (57 ft) by 13 m (43 ft) beneath the old high school (northernmost bowl). Also note the entry widths are different so the pillar stresses must be different. Assume that each pillar supports a column of overburden extending from the mine to the surface and that the column boundaries are the midpoint of entries between adjacent pillars (table 1) (Whittaker and Reddish, 1989). The weight of this column can be computed by assuming that the overburden has a unit weight of 20.3 KN/m^3 (144 lbs/ft^3).

It is estimated that the pillars beneath Dorris were subjected to a stress of 5.8 MPa (840 psi) compared with 2.4 MPa (345 psi) beneath the high school. The capacity of pillars to support this stress is not only controlled by the pillar strength (e.g., 10 MPa) but also floor bearing capacity (e.g., 5 MPa) (Pula et al, 1990). In the case of pillar crushing, convergence could occur rapidly at mine level with rapid subsidence at the surface. In the case of pillars punching into the floor, convergence may be long term. In summary, the difference in maximum subsidence for the three bowls in figure 3 can be attributed to different stress conditions, support capacities, and failure mechanisms.

GIS Analysis of Pillar Stresses

Since the hypothesis of tributary area loading on pillars is based almost exclusively on geometry, this type of analysis lends itself to the use of GIS software to estimate the distribution of pillar stress on a mine-wide scale. The AutoCAD drawing

Figure 3.-Estimated subsidence contours with hypothetical associated pillars shaded, minimum contour is 0.05 m, contour interval is 0.025 m; *see figure 4 for location.*

Table 1.-Estimation of Pillar Stresses

	Old High School	Dorris School
Pillar Dimensions		
length.....	17 m	20 m
width.....	13 m	6 m
area.....	221 m^2	120 m^2
Overburden Supported		
length.....	22 m	26 m
width.....	20 m	22 m
area.....	440 m^2	572 m^2
volume.....	26,400 m^3	34,320 m^3
weight.....	537 MN	698 MN
Pillar stress	2.4 MPa	5.8 MPa

Table 2.-Material Properties

	Density kg/m3	Bulk Modulus MPa	Shear Modulus MPa	Constitutive Law	Friction Angle Degrees	Cohesive Strength MPa	Tensile Strength MPa
Soil	2090	330	200	Mohr-Coulomb	30	0.03	0
Rock Mass	2410	630 to 7370	380 to 4420	Mohr-Coulomb	31	2.6	1.0

Table 3.- Horizontal Joint Properties

Normal Stiffness MPa/m	Shear Stiffness MPa/m	Slip Criterion	Friction Angle Degrees	Cohesive Strength MPa
30000	1500	Coulomb	15	0

was converted to GIS coverages using ArcCAD. ArcCAD is a bridge between CAD and GIS integrating the editing and graphical tools from AutoCAD with the topological and analytical tools of a GIS program. A user can toggle between AutoCAD and ArcCAD user interfaces, as well as being able to call upon third-party extensions available to AutoCAD. ArcCAD allows simple conversion of the graphical "entities" of the AutoCAD drawings to GIS "features" without requiring additional data input. ArcCAD is a vector-based GIS that enables the coordinates of the CAD graphical representation to be related to real-world coordinates and map projections. A GIS allows integration of data and maps of disparate scales and sources into one set of geographically referenced data files and establishes topology (relationships) between the geographically referenced features.

The tributary area polygon surrounding each pillar was defined by the centerline of the entries surrounding each pillar. This tributary area network was created using AutoCAD then converted to a GIS POLYGON coverage. Since POLYGON coverages automatically calculate area measurements and are accessible in the database, a ratio between the tributary area and pillar area was calculated by merging the two POLYGON coverages. The tributary area/pillar area ratio was multiplied by an estimated overburden unit weight (20.3 KN/m^3 = 144 lbs/ft^3) to compute pillar stress. Computations were required for several hundred to several thousand pillars for each of the four mines shown in figure 4 which encompassed the entire study area. GIS was then used to classify the mine pillars into four levels of stress:

Class	Stress Level
1	< 3 MPa
2	3 - 4.99 MPa
3	5 - 5.99 MPa
4	>6 MPa

Figure 5 displays pillars below Dorris School. The class 3 stress level is consistent with statistical values for bearing capacity of weak floor strata in the Illinois Basin (Pula et al, 1990).

Numerical Modeling of Pillar Stresses

Numerical modeling was done to provide an independent assessment of the GIS analysis. The methodology was patterned after guidelines outlined by Starfield and Cundall (1988) who encouraged researchers to remember that the focus of numerical analysis should be on gaining an understanding of the mechanisms that characterize the system and not on obtaining unique numeric values for specific parameters. Hart and Cundall (1992) go on to explain that this is accomplished by using the computer model as a laboratory to perform experiments on the system. This can result in an improved understanding of mechanisms, knowledge of parameter dependence, and a means by which to check theories or hypotheses. Ultimately, the new knowledge may lead to new theories or simple conceptual models that can be used in design (Hart and Cundall, 1992). In the present study, the numerical analysis increased understanding of stress levels and the distribution of stresses acting on pillars in the abandoned mines.

The Universal Distinct Element Code (UDEC) (Itasca, 1992) was used to perform desk top experiments and identify parameters that significantly influence model behavior. The computer code allows internal deformation of discrete blocks and also allows large displacements and separations along discontinuities. UDEC utilizes the distinct-element method which is a particular type of numerical model that explicitly incorporates contacts between deformable blocks and explicit time-stepping solution of the equations of motion (Cundall and Strack, 1979). Output in the form of displacements, velocities, and stress distributions can be compiled into movies that make it possible to visualize the influence of various parameters on rock mass response.

Stratigraphy

The distinct element model is composed of deformable blocks separated by discontinuities. Since it is hypothesized that overburden response is affected by the location and character of discontinuities within the rock mass, Bureau researchers used the modified Rock Mass Rating (RMR) system (Bieniawski, 1989) to systematically define the location of horizontal discontinuities in the model. A commercially-available spreadsheet program was used to calculate a RMR for each lithologic bed based on drill core logs and engineering property tests. The RMR was then used to calculate a deformation modulus and bending stiffness for each lithologic bed (figure 6). Large contrasts in the bending stiffness between adjacent lithologic beds have been shown to correlate with measured horizontal shear displacements (Siekmeier et al, 1992). Thus the locations of these large contrasts in bending stiffness were used to define

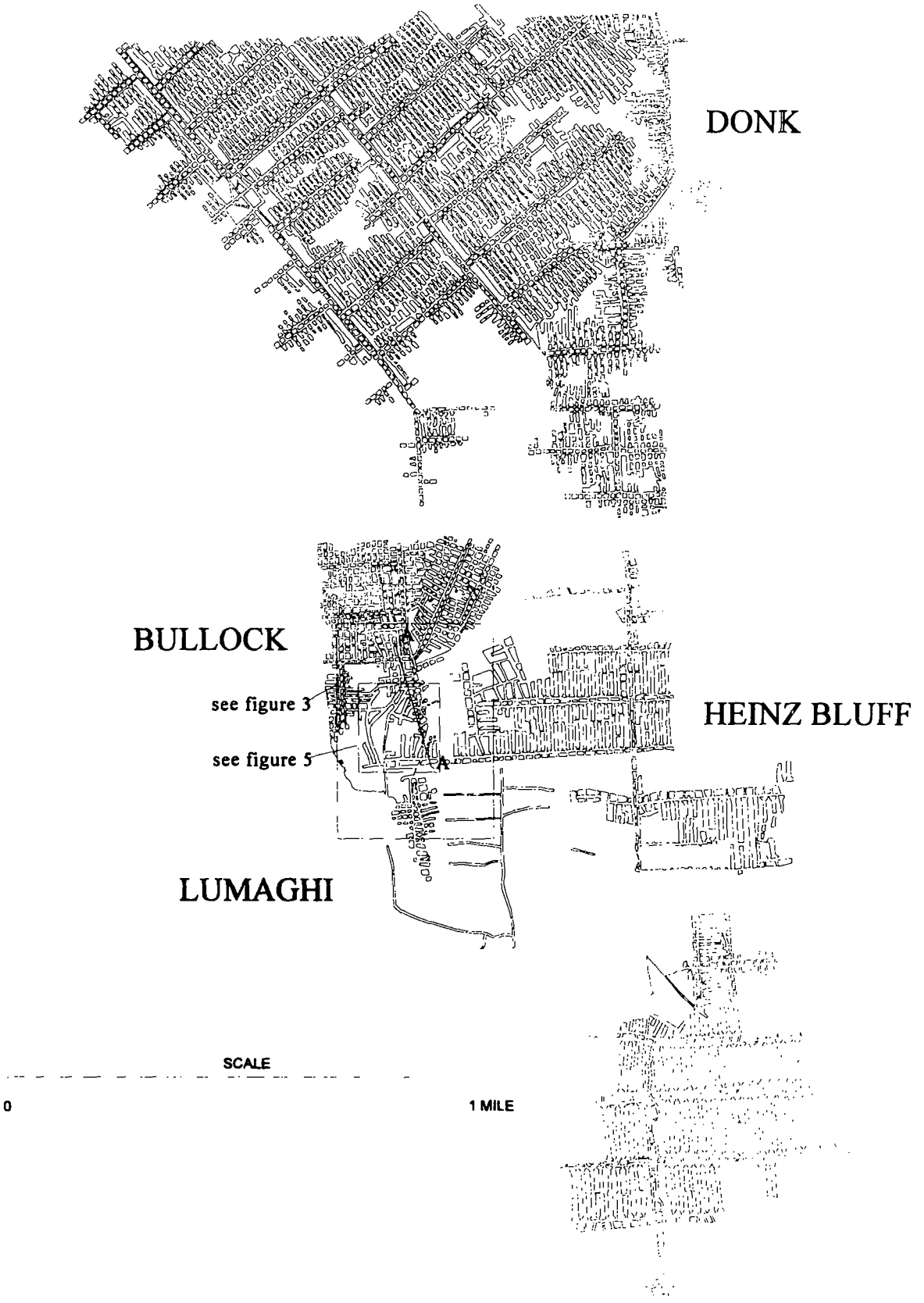

Figure 4.-Display of the four mines in the research area.

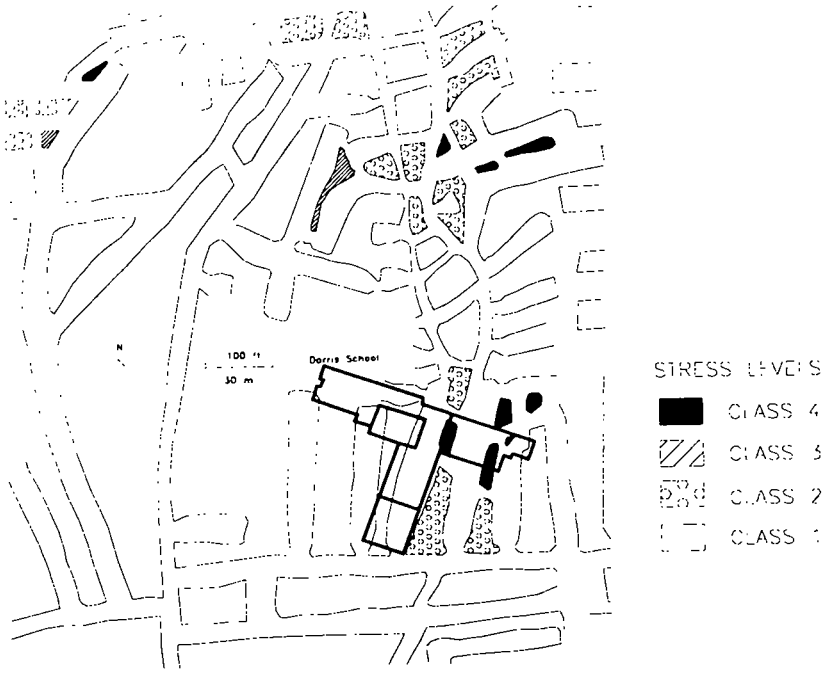

Figure 5.-Dorris Elementary School with pillars from Bullock Mine shaded according to stress level, class 4 is the highest stress; see figure 4 for location.

horizontal discontinuities in the distinct element model (horizontal lines on the right side of figure 6) .

The mines are approximately 60 m (200 ft) below the surface, overlaid by 30 to 34 m (100 to 110 ft) of glacial material and 27 to 30 m (90 to 100 ft) of Pennsylvanian age rock The glacial material consists of 9 to 12 m (30 to 40 ft) of loess overlying silty clay till and sandy stream deposits. The topmost rock stratum is a claystone that has altered to a silty clay of high plasticity. This altered zone is approximately 8 m (25 ft) thick and washed out easily during drilling. The significant features are the two stiffer limestone strata of the mine roof. The upper one is about 1.2 m (4 ft) thick, and the lower one is 6 to 8 m (20 to 25 ft) thick. This lower unit forms the immediate mine roof.

Based on observations made with a downhole camera as well as water level measurements, the mines are flooded and the water is pressurized. An attempt was made to obtain a sample of the mine floor underclay and assess its thickness. The stratum is a highly plastic gray clay at least 1.2 m (4 ft) thick. This thickness was estimated by allowing the drill string to penetrate under its own weight.

Boundary Conditions and Material Properties Used in Numerical Model

A two-dimensional, plane-strain, distinct element model was used to simulate the overburden and abandoned mine along line AA shown in figure 3. The horizontally bedded rock mass and overlying glacial materials were modeled as eleven deformable blocks stacked as a layer cake (figure 6). The blocks are composed of finite-difference triangles and represent layers defined by horizontal discontinuities between strata with significant changes in stiffness.

The deformable block properties are listed in Table 2 and properties of the horizontal discontinuities are listed in Table 3. These values were considered to be reasonable approximations. In order to simulate tributary loading over entries adjacent to the pillars but outside the cross sectional plane, overburden density was doubled. Once again, the objective was to determine if the pillar stresses estimated using the assumption of tributary area loading was reasonable so that any correlation with historical surface subsidence could be considered valid.

Results of Computer Simulations

To simulate abandoned coal mines, blocks representing entries were deleted and gravity loading was applied for 15000 calculation cycles. Support provided by water pressure was not considered and the resulting distribution of vertical stress at two depths is shown in figure 7. The lower line graph is the computed vertical stress acting on the pillars and the upper line graph is the computed vertical stress within the rock mass above the mine roof. Note that the larger stresses occur over pillars adjacent to wider mine openings which is consistent with the tributary area assumption. The maximum value of stress computed for the mine pillars (the lower line graph) is 8.0 MPa which is consistent with maximum stresses on class 4 pillars in figure 5. These results lend validity to the assumption of tributary loading on pillars.

Comparison Between Pillar Stress Conditions and Subsidence Occurrence

While numerical modeling provided an independent check of the tributary loading assumption made in the GIS analysis, the objective was to correlate pillar stress conditions with occurrences of subsidence. Compare the Class 2, Class 3, and Class 4 pillars in figure 5 with the shaded pillars in the center of figure 3. There is good correlation between the subsidence bowl which impacted Dorris Elementary School and the level of pillar stress. This would appear to imply that the GIS analysis assuming tributary loading can provide a basis for assessing the likelihood of subsidence occurrence.

This hypothesis does not appear to be valid for pillars in the top of figure 5. Although there are pillars rated at stress levels of Class 2 and greater, subsidence has not been measured in this area. This does not invalidate the hypothesis but it does demonstrate difficulties with proving that the hypothesis is valid. It is likely that subsidence will occur over highly stressed pillars, but pillar stress analysis cannot predict when subsidence will occur. Furthermore, subsidence may have occurred but it was not measured since there are no structures in this area which could have suffered damage.

Another compounding factor is the influence of mine flooding. Based on downhole camera inspection of mine conditions and water level

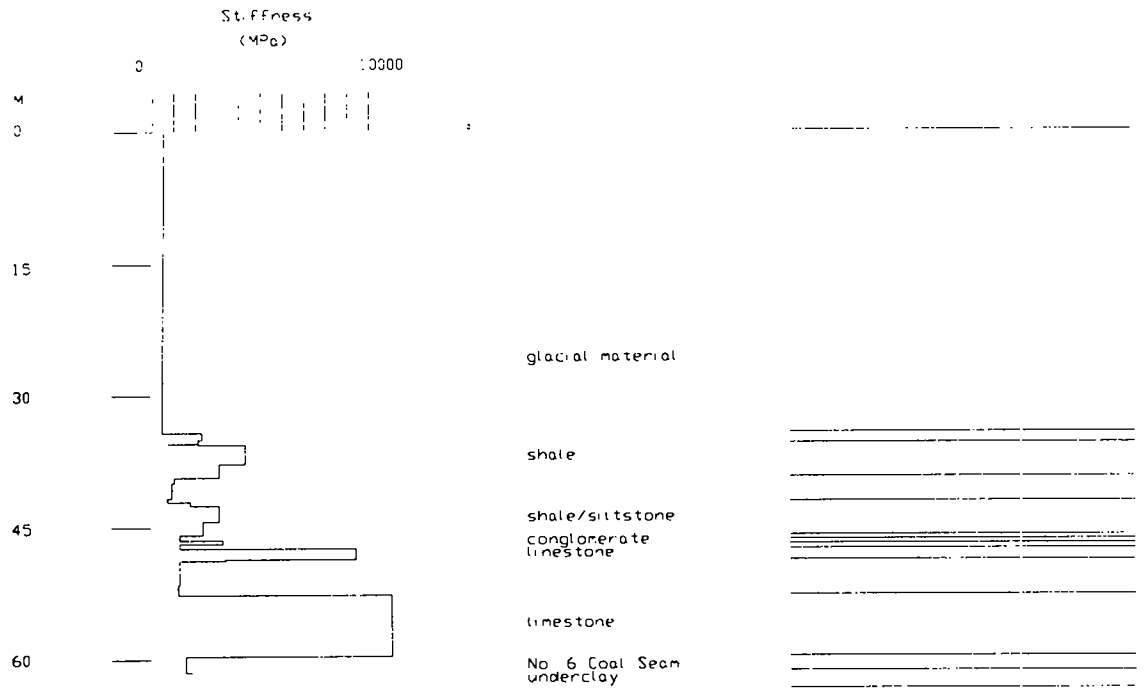

Figure 6.-Stiffness histogram and geologic model used for numerical simulation.

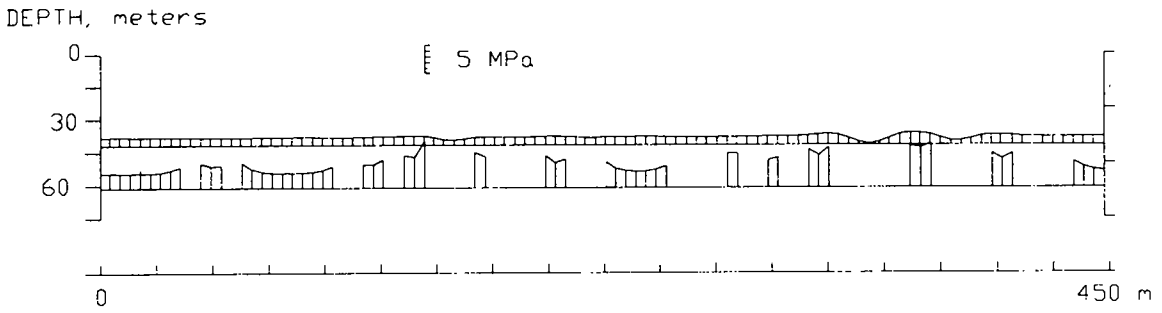

Figure 7.-Distribution of computed vertical stress acting on pillars and along top of main roof stratum; cross section is along line AA in figure 3.

measurements, it is known that the mine is flooded and the water is pressurized. Support for the mine roof is provided simultaneously by pillars and water pressure. Consequently, the actual pillar stress is less than that computed by tributary area loading. If the water pressure dissipates then pillar stresses will increase and the likelihood of subsidence occurrence will increase (USEPA, 1981).

Summary

This paper presents a study which was conducted to assess the correlation between pillar stress and subsidence occurrence over abandoned coal mines. Historical data were assembled and subsidence contours were generated for each date that measurements were made. These contours were overlaid on a CAD base map which included the digitized mine pillars and entries. The CAD layers were selectively activated and a time lapse animation was created to observe the areal progression of subsidence occurrences. Assuming tributary loading on pillars, stresses were computed for several thousand pillars using GIS software. An independent check of the tributary loading assumption was made by numerical modeling of the overburden and abandoned mine. Pillars were then classified by stress level and this classification map was overlaid with the subsidence contours.

Pillar stresses estimated using the hypothesis of tributary loading are consistent with those estimated using the numerical model. Subsidence occurrences are not random events and have developed in a manner which is physically consistent with the layout and stress level of mine pillars. The combination of CAD techniques and GIS analysis offers a valuable tool for subsidence risk assessment.

Literature Cited

Bieniawski, Z. T. Engineering Rock Mass Classification. John Wiley and Sons, New York, NY, 1989, 251p.

Cundall, P. A. and O. D. I. Strack. A Discrete Element Numerical Model for Granular Assemblies. Geotechnique 29, 1979, pp. 47-65.

Gibson, R. D., and B. C. Schottel. A Case History Illustrating the Application of Computerized Modeling of Coal Mine Subsidence Profiles and the Development of a Settlement Prediction Technique. Proceedings, Third Conference on Ground Control Problems in the Illinois Coal Basin, Mt. Vernon, IL, August, 1990, pp. 369-381.

Hart, R. D. and P. A. Cundall. Microcomputer Program for Explicit Numerical Analysis in Geotechnical Engineering. Presented at the International Seminar on Numerical Methods in Geomechanics, Moscow, March 1992.

Hunt, S. R. Surface Subsidence Due to Coal Mining in Illinois. Ph.D. dissertation, University of Illinois at Urbana-Champaign, 1980, 129 pp.

Illinois State Geological Survey. Directory of Coal Mines in Illinois, Madison County. Natural Resources Building, Champaign, Illinois, January, 1991.

Itasca Consulting Group (Minneapolis, MN). UDEC Version 1.8. Vol. 1 and 2, June, 1992.

Kratzch, H. Mining Subsidence Engineering. Springer-Verlag, Berlin, 1983, pp. 145-155.

Pula, O., Y. P. Chugh, and W. M. Pytel. Estimation of Weak Floor Strata Properties and Related Safety Factors for Design of Coal Mine Layouts. Proceedings, Mine Subsidence - Prediction and Control, 33rd Annual Meeting, Assoc. of Eng. Geologists, Pittsburgh, PA, October 1-5, 1990, pp. 91-103.

Siekmeier, J. A. , K. M. O'Connor, and L. R. Powell. Rock Mass Classification Applied to Subsidence Over High Extraction Coal Mines. Proceedings, Third Workshop on Surface Subsidence Due to Underground Mining (Morgantown, WV, June 1-4, 1992), West Virginia Univ., 1992, pp. 317-325.

Starfield, A. M. and P. A. Cundall. Towards a Methodology for Rock Mechanics Modeling. Int. J. Rock Mech. Min. Sci. and Geomech. Abstr., 25 (3), 1988, pp. 99-106.

Triplett, T., G. Lin, W. Kane, and R. Bennett. The Effects of Undermining on Various Types of Linear Foundations. Proceedings,

Symposium on Construction over Mined Areas. Pretoria, South Africa. May, 1992. pp. 99-106.

U. S. Environmental Protection Agency. Rehabilitation of Wastewater Facilities, Streator, Illinois. Final Environmental Impact Statement, EPA-5-IL-LASALLE-STREATOR-WWTP AND CSO-1981, Region V, Chicago, IL, February, 1981, 277p.

Whittaker, B. N., and D. J. Reddish. Subsidence Occurrence, Prediction and Control. Elsevier Science Publishing Company, Inc., New York, 1989, pp. 173-189.

MINE DESIGN: LONG TERM EFFECTS OF HIGH EXTRACTION MINING[1]

by

Paul W. Jeran[2]

Abstract. A consideration when designing a high extraction coal mine is the effects that mining will have on the ground above the mine. This becomes particularly important when the surface has been improved or is inhabited. Surface owners are concerned about; when the effects will begin?, how large will they be?, and how long they will last?. Each of these should be addressed by the designer.

For more than a decade, the U.S. Bureau of Mines (USBM) has been monitoring subsidence at various sites. Based upon the data gathered, some inferences may be made regarding the above stated questions. Essentially surface movement begins with undermining. The magnitude of the movements are proportional to the thickness extracted and the width of the mined area, and inversely proportional to the depth of the mine below surface. The duration of the subsidence process in the northern Appalachian Basin is approximately one year.

The USBM has developed a computer model which predicts the final subsidence profile across a longwall panel in the northern Appalachian Coal Basin. USBM studies on the dynamic development of subsidence have shown that the magnitude of the deformations developed during the subsidence process never exceed those exhibited in the final subsidence profile. Use of the model will provide engineers with a starting point in the design process.

INTRODUCTION

The mining of a large rectangular block of coal by the longwall method results in the development of a trough-shaped depression of the surface above the extracted area. The process of subsidence is dynamic as surface movement begins with undermining and continues until some maximum displacement has occurred: the magnitude of which is controlled primarily by the extracted thickness, the width of the panel, and the overburden thickness and geology. At the end of the subsidence process, equilibrium is achieved and the resulting surface deformations become static. Any subsequent movement of the surface does not result from subsidence but from the altering of the conditions affecting the rock strata disturbed by subsidence (i.e., injection or removal of fluids, erection of structures).

During any discussion of subsidence with longwall mine operators and particularly surface property owners, several questions always arise: How much subsidence will occur?; What will be affected?; When will subsidence begin?; and How long will the process of subsidence last? The first two questions can be estimated using the geometry of mining as input into the U.S. Bureau of Mines' (USBM) subsidence

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prediction model for the northern Appalachian Coal Field (Jeran 1986). The latter two are the subject of this paper.

Subsidence monitoring is typically carried out in two directions: along the centerline of the panel to obtain data which will show the maximum amount of subsidence and the duration of the dynamic phase of movement; and, across the panel (perpendicular to the centerline) to evaluate: the lateral extent of surface deformations, the final cross-section of the developed subsidence trough, and the distribution of final deformations. This is the static result of the subsidence process and any further movement must be attributed to some other cause. Subsidence prediction is typically limited to the final shape of the subsidence trough. Practically all predictive models address this aspect of mining-induced ground movement.

Study of subsidence data gathered along the centerlines of a number of longwall panels in the northern Appalachian Coal Basin, by Adamek and Jeran (1992), has shown that the subsidence process starts with the undermining of a surface point and is 90 percent completed by the time the longwall face has been advanced the thickness of the overburden beyond that point. In this study, it was determined that: the speed with which the face is advanced has no effect on the magnitude of surface deformations; the final movement is usually achieved with mining of the adjacent panel; and, the magnitude of surface deformations that occur during dynamic subsidence are always less than the static case (Adamek 1992)

Examination of manmade features on the surface has shown that they are affected by the forces applied to them throughout the subsidence process. Therefore, the final static shape of the subsidence trough cannot be used to fully explain their degradation or how they came to their final condition. It has also been observed that the degree of degradation is dependent upon location within the subsidence trough (Walker 1990).

Field Studies

As has been noted above, the description of dynamic subsidence has been limited to movement along the centerline. However, does this behavior occur equally across the developing subsidence trough? In an effort to obtain some insight to this question, long term subsidence data were needed. At one site, in northern West Virginia, the USBM monitored a series of longwall panels, in the Pittsburgh Coalbed, remote from previous high extraction mining. Monitoring was conducted for over two years during which time four adjacent longwall panels were mined. Surface monuments were installed over the first three panels (figure 1) and monitoring was conducted over the portions of the array that were actively moving due to undermining. Periodic measurements were also made of the previously undermined portions of the array to determine final movement. Above the first two panels there was no further vertical movement detected after the subsequent adjacent panel was mined (Jeran 1988).

To simplify the analysis, four monuments across the first panel, as shown in figure 1, were selected: (A) above the centerline, (B) 30.5 m (100 ft) from the centerline, (C) 30.5 m (100 ft) inside the rib abutting the barrier pillar, and (D) above the rib abutting the barrier pillar. Figure 2 shows a plot of the subsidence of these four monuments verses time. Also included are the face positions showing the progress of mining for the studied panels. The last measurement was made after mining of the fourth panel was completed. Each face position line is marked to indicate the location of the

face when the four monuments were either undermined or when the longwall face passed by them. From this, we can see that each monument experienced the majority of its movement with the mining of Panel 1 and had completed movement with the mining of Panel 2. Figure 3 shows the first 60 days of the study. From this, it can be observed that the monuments were undermined on the 25th day and some minor movement had been detected as much as 10 days earlier. Furthermore, most movement had occurred within 30 days after undermining.

Since each monument subsided a different amount, the percentage of final movement for each measurement was computed to provide some basis of comparison. Please note that negative values of time and face position, where they occur on graphs, indicate data acquired prior to undermining of the monuments. When the percentages of final subsidence are plotted against face position (figure 4) it can be seen that the three interior monuments (A, B, and C) subsided more than 85 percent of the final movement by the time the face was 245.4 m (800 ft) past the monument line. Movement of the monument over the rib (D) was only 70 percent complete at this time. Plotting the percentage of final subsidence against time (figure 5) shows that all of the monuments behaved similarly for the first 10 days after undermining. After that period, the two central monuments (A and B) behaved similarly and the monument 30.5 m (100 ft) inside the rib (C) lagged slightly behind. The rib monument (D) behaved differently.

From the above, it can be concluded that subsidence of the monuments within the panel limits is governed by face position and time. While subsidence of the monument over the rib, after its initial movement, is more governed by time.

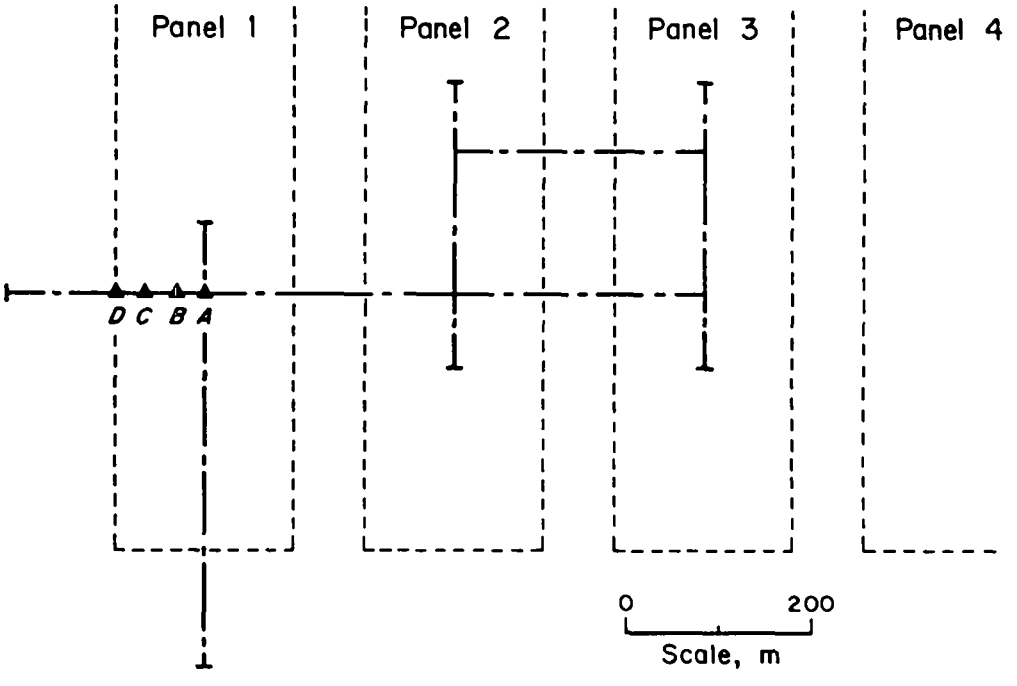

Figure 1. Monitoring Array

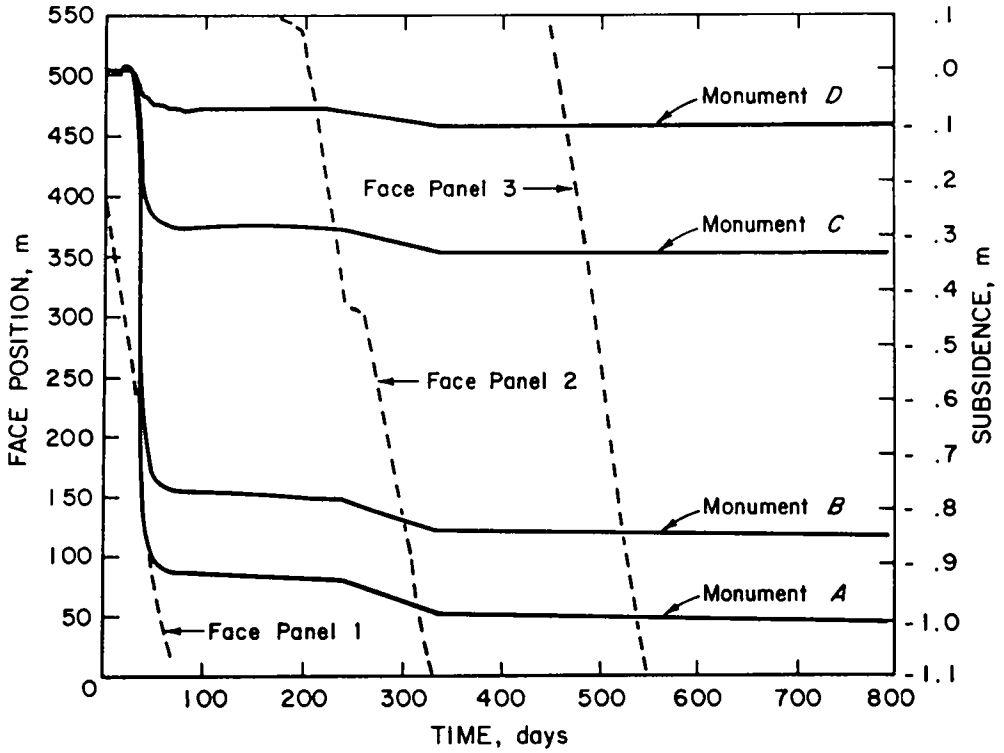

Figure 2. Subsidence during the study versus time

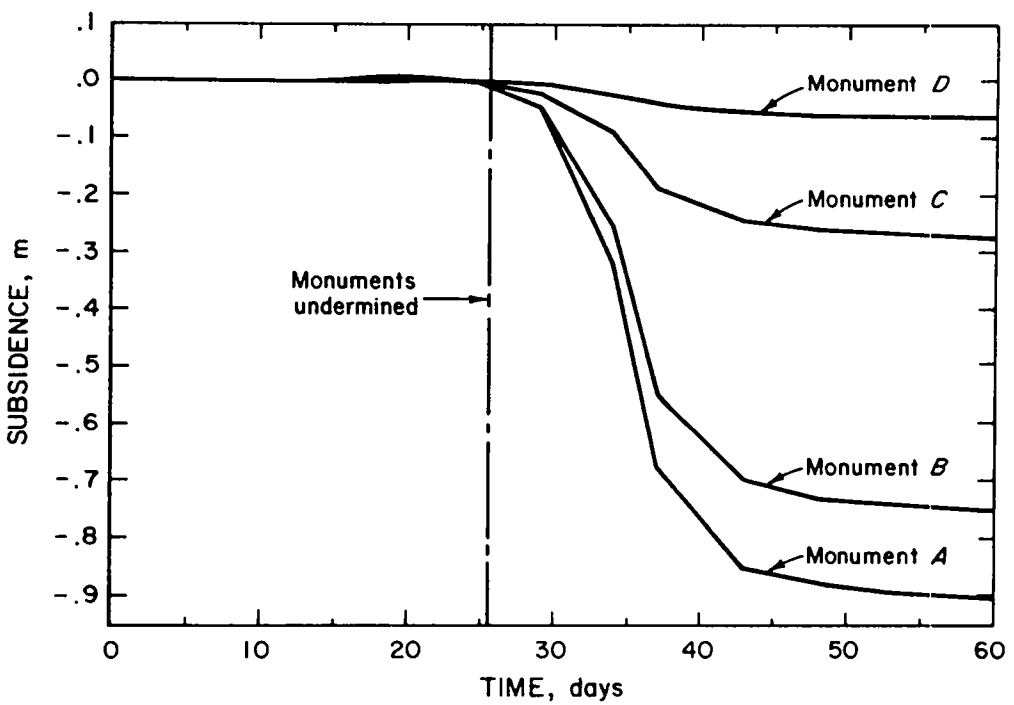

Figure 3. Subsidence during first 60 days

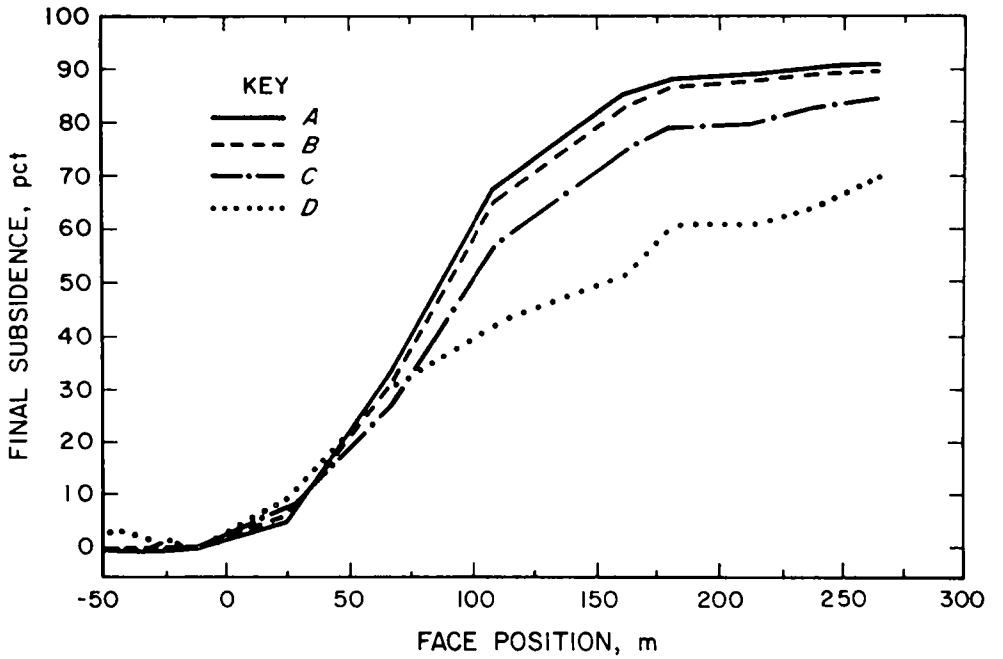

Figure 4. Percent final subsidence versus face position

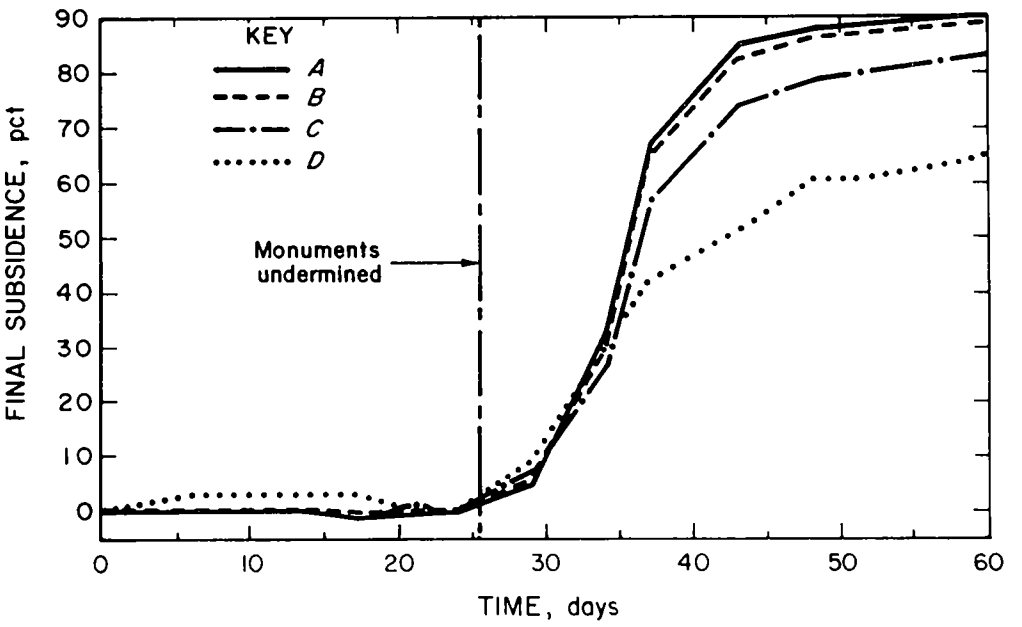

Figure 5. Percent final subsidence versus time

Since these conclusions only represent information from one site, other sites, with shorter duration of study, were investigated to substantiate our observations. Mine sites were selected, one in each of three different coal beds: Pittsburgh, Lower Kittanning, and Freeport. Figure 6 shows the locations of these sites.

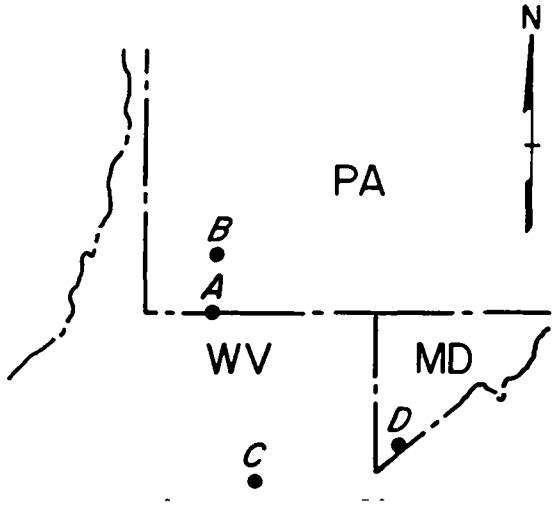

Figure 6. Location of Mines

Mine A is the original site. The Pittsburgh Coalbed site was chosen to determine if the observations were limited to operations in this coalbed (the original site was in the Pittsburgh Coalbed). Overburdens, extracted thicknesses, rates of face advance and geometries varied among the sites. Figure 7 is a plot of face advance for each of the sites. The average rates of mining ranged from 3.1 m/day (10.0 ft/day)at Mine B to 11.8 m/day (38.8 ft/day) at Mine D. Table 1 contains data describing overburden thickness, panel width, average rates of face advance, and coalbed for the sites.

In all cases, the monuments chosen were from the profile line extending from the centerline outward away from subsequently mined panels. Since the extracted thicknesses were different and resulted in differing magnitudes of subsidence, the percentage of the final movement of each monument was calculated and used in the comparisons. It should be noted that the process of subsidence at all sites was completed within one year of undermining.

Data from the centerline monuments at each of the 4 sites plotted against time (figure 8) show that subsidence at mines C and D generally agreed with that at Mine A, while Mine B was much slower. Mine B had the slowest face advance and greatest overburden thickness. Considering that dynamic subsidence is proportional to face advance and inversely proportional to overburden thickness (Adamek 1992) this could be the expected result. When the percentage of final subsidence was plotted against face position, (figure 9) Mines A and B (Pittsburgh Coalbed) are in agreement with Mines C and D differing. It should be noted however, that Mines C and D had lesser overburden thicknesses. When plotted against face position in terms of overburden thickness (figure 10) all sites are in closer agreement. Note, in figure 10, for the centerline monuments, subsidence at all sites was about 90 percent complete when the face had advanced the thickness of the overburden beyond the monument location. These observations agree with the findings of the earlier mentioned study by Adamek and Jeran (1992).

The same plots were made for the monuments 30.5 m (100 ft) from the centerline for the 4 sites under study (figures 11-13). Again, as for the centerline points, the plot versus face position in terms of overburden thickness shows the closest agreement and again, over 90% of the final subsidence was completed when the face

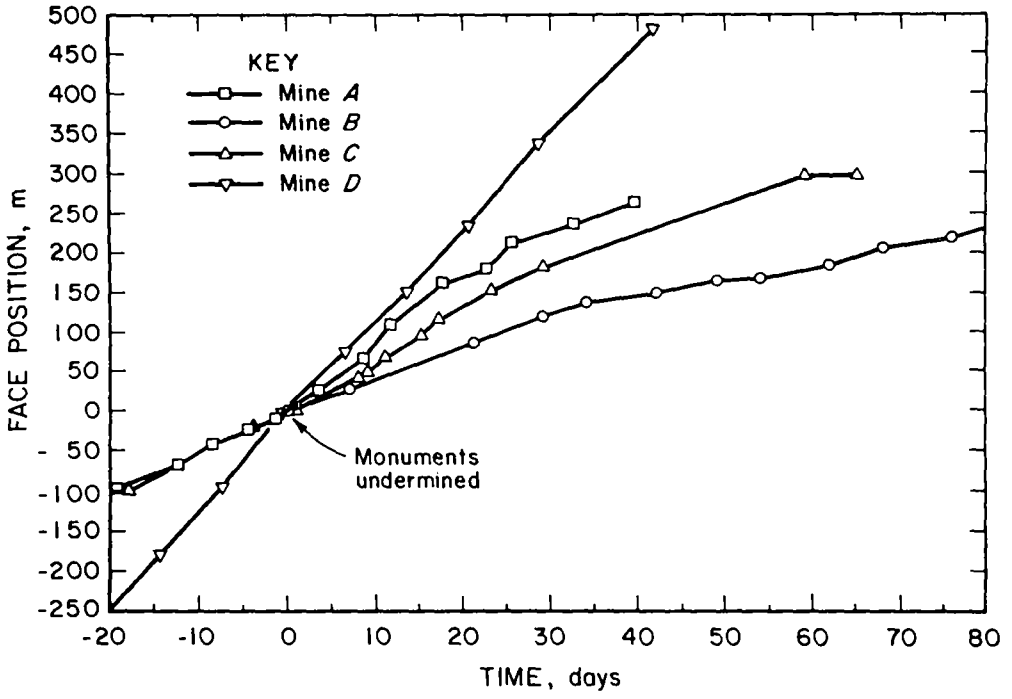

Figure 7. Face advance for each site

Table 1. Study Sites Information

Mine	Panel width, m(ft)	Overburden, m (ft)	Average face advance m/day (ft/day)	Coalbed
A	190.5 (625)	210.3-213.4 (690-700)	6.55 (21.5)	Pittsburgh
B	182.9 (600)	277.4-298.7 (910-980)	3.05 (10.0)	Pittsburgh
C	289.6 (950)	118.9-126.5 (390-415)	4.94 (16.2)	Kittanning
D	182.9 (600)	143.3-155.4 (470-510)	11.83 (38.8)	Freeport

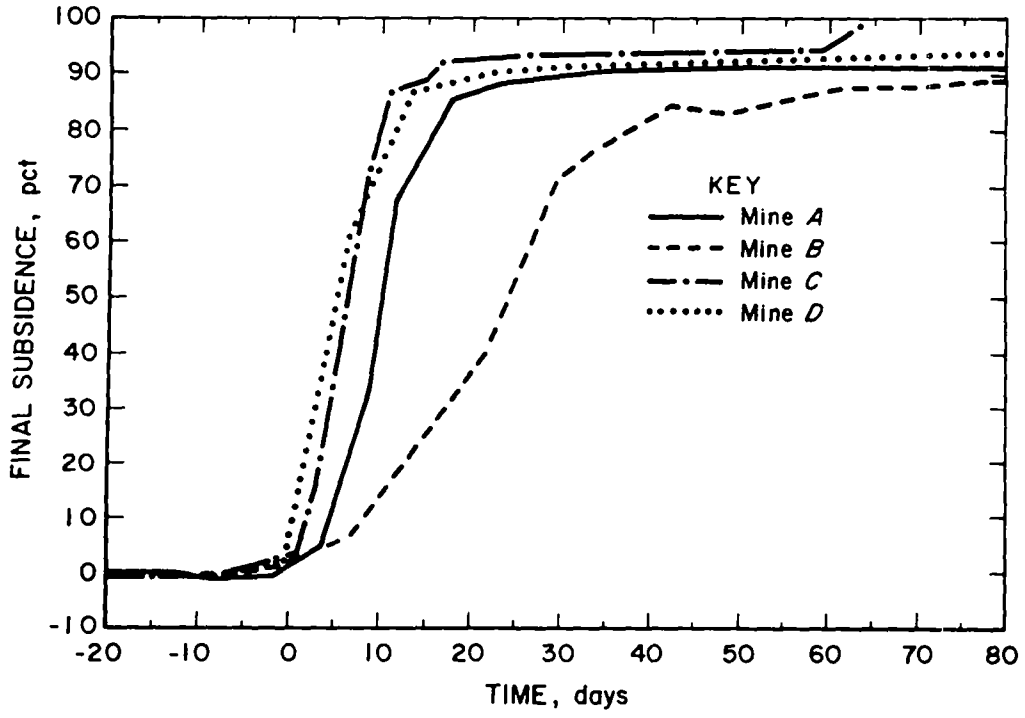

Figure 8. Centerline poinst versus time

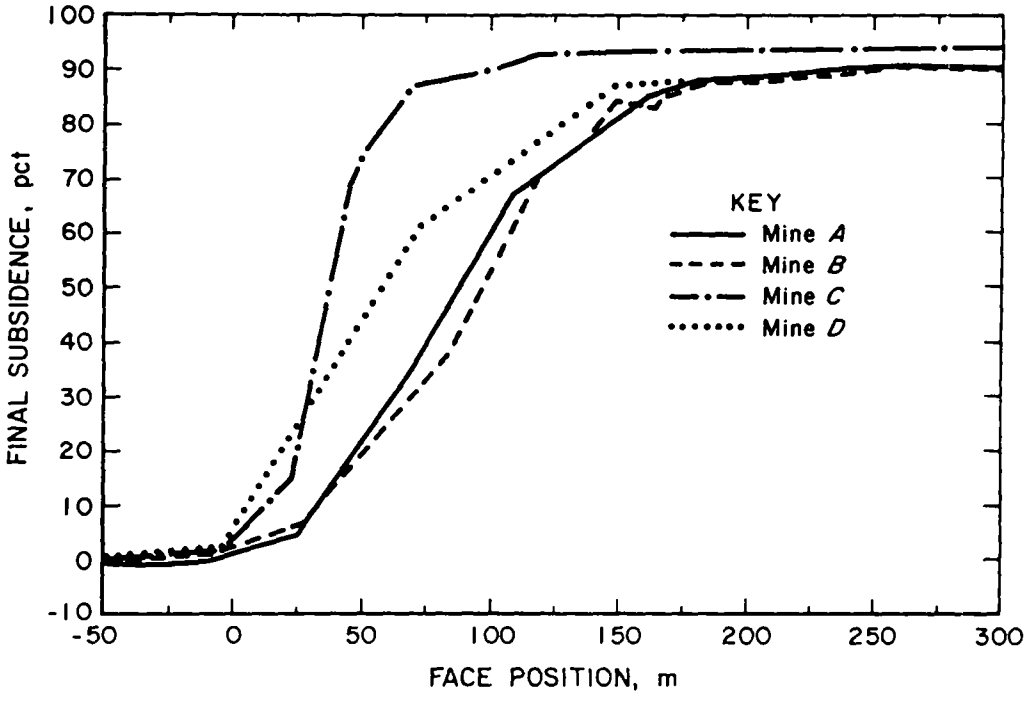

Figure 9. Centerline points versus face position

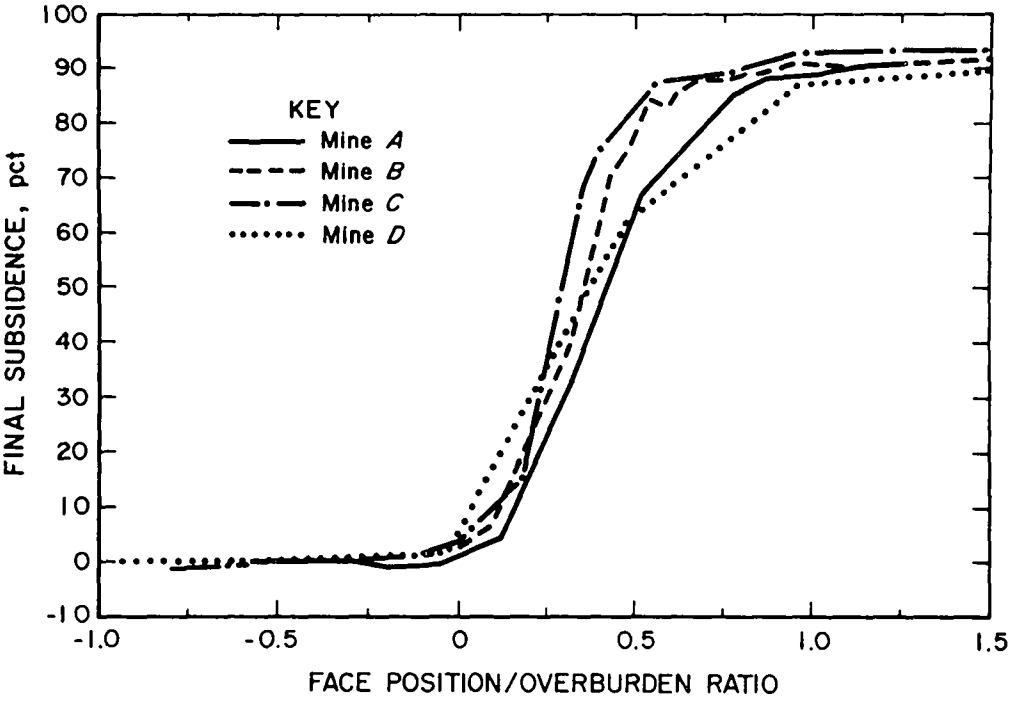

Figure 10. Centerline points versus face position/overburden

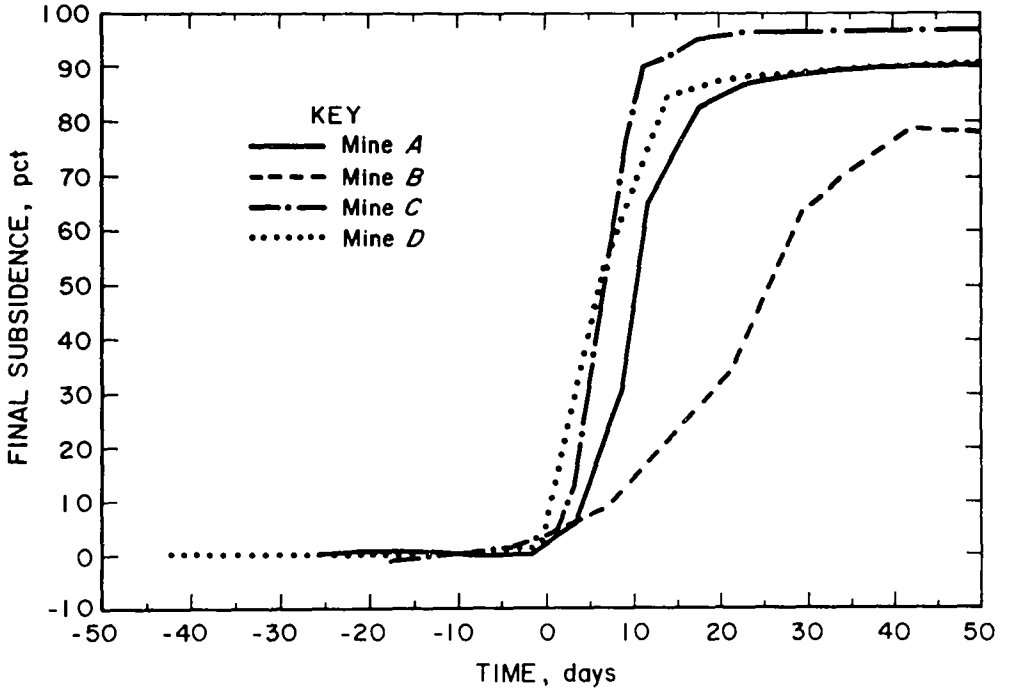

Figure 11. Centerline +100 points versus time

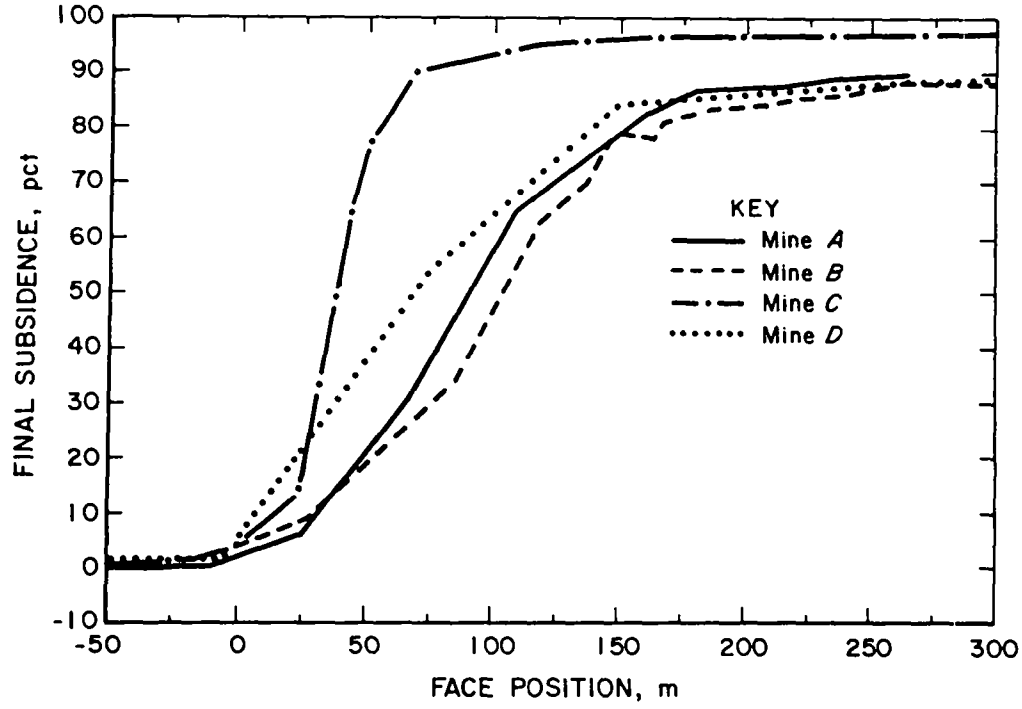

Figure 12. Centerline +100 points versus face position

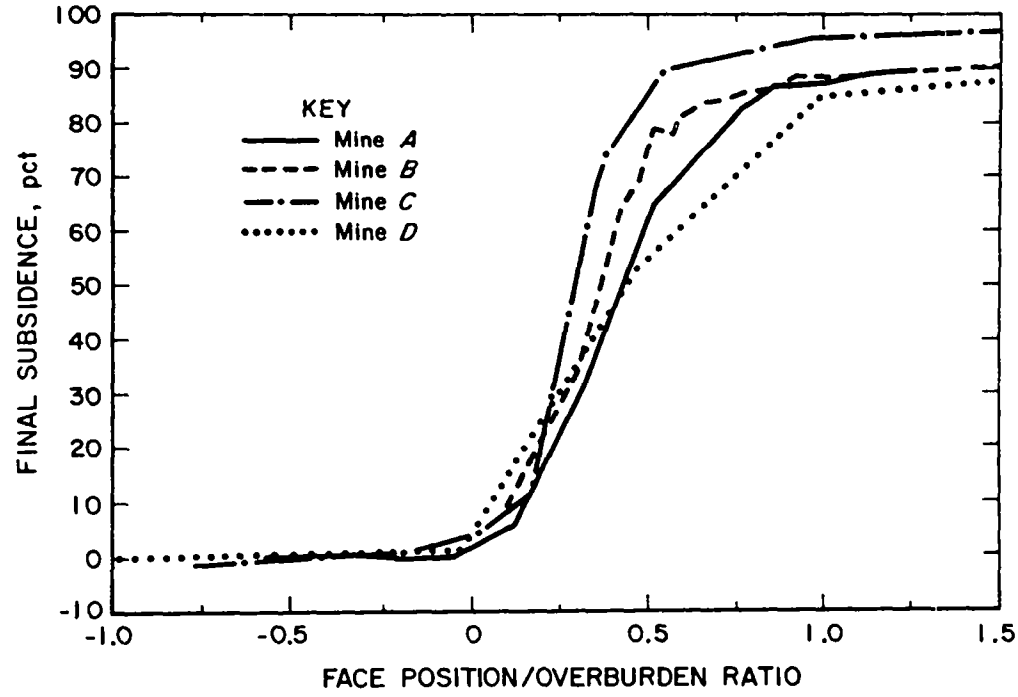

Figure 13. Centerline +100 points versus face position/overburden

had advanced the thickness of the overburden past each point.

Plots for the monuments 30.5 m (100 ft) inside the rib versus time and face position (figures 14 and 15) are similar to those for the more centrally located monuments. However, the plot the percentage of final subsidence versus face position/ overburden ratio (figure 16) shows that this location at Mine D lagged behind the same location at the other 3 mines indicating retardation of the subsidence process. The overburden at this site contains significantly more and thicker sandstone units. These stiffer members bridge further over the gob than the less resistant strata at the other sites. Additional time is needed for the stiffer units to bend into the gob area.

The similar plots for the rib monuments (figures 17-19) show a different characteristic. Plots of the percentage of final subsidence versus face position (figure 18) and face position in terms of overburden thickness (figure 19) do not show the same trends as observed for the other points. The plot of percentage of final subsidence versus time (figure 17), however, shows that within a month of undermining these monuments had completed from 40 to 70 percent of their final subsidence. This indicates that the process of subsidence over the rib is very much site dependent and time appears to play a very significant role.

Deformations

Underlying all discussions of subsidence resulting from longwall mining is the concern for damage. From the above discussion it can be seen that except for the area above the rib most of the surface movement is accomplished by the time that the longwall face has mined past the thickness of the local overburden. It is therefore logical to assume that most of the deformations of the surface are also completed by this time.

The study of dynamic subsidence showed that the magnitude of dynamic deformations (inclination, curvature, and horizontal strain) are always less than the static values (Adamek 1992). Therefore, if the difference between the values of final static deformations and those developed at the time the longwall face has mined one overburden thickness past the profile are small, then the additional deformations from a dynamic point of view should also be small.

The distribution of static inclinations were calculated for each of the profiles used in this study at the point where the longwall face had mined 1 overburden thickness past and again when subsidence had been completed. These are shown in figures 20 - 23. They show that static inclinations increased by less than 3 mm/m. Therefore, dynamic inclinations should be smaller than these values. From these we must conclude that, for these sites, the surface deformations occurred primarily during the mining of the longwall face one overburden thickness past the profile. Additional deformations were small. Since there are no criteria established in this country correlating magnitude of deformation with structural damage it is impossible to state that no additional damages would result from the additional deformation. If criteria developed in Europe (Adamek 1982) are any indication, then these additional deformations should not significantly contribute to surface damages.

Summary

Subsidence data from four sites in the northern Appalachian Coal Basin were studied. These data show that for locations in the central portion of the

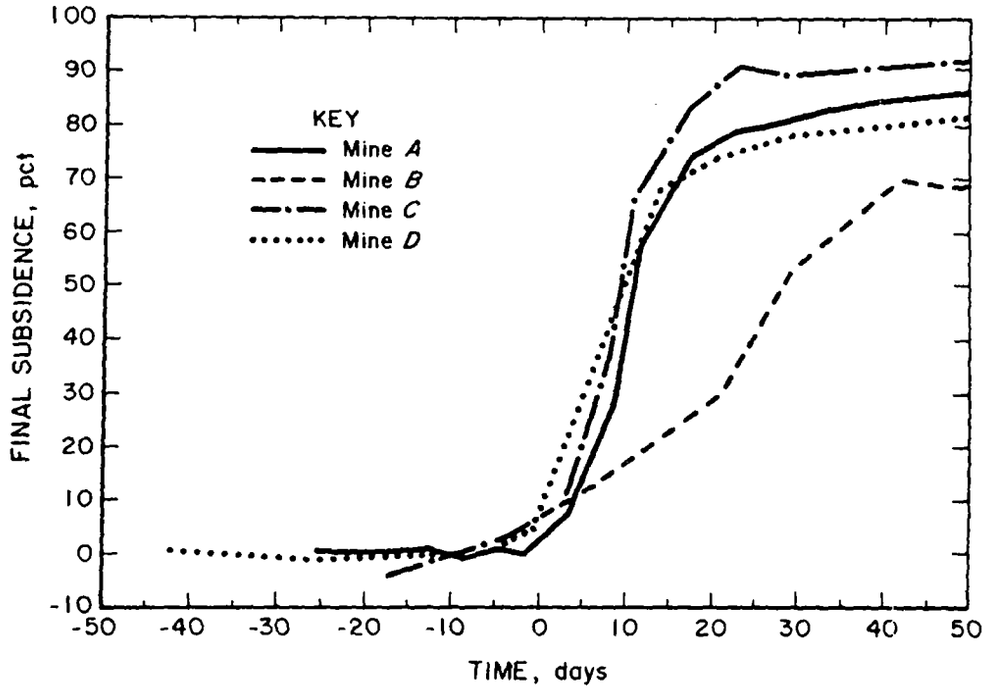

Figure 14. Rib -100 points versus time

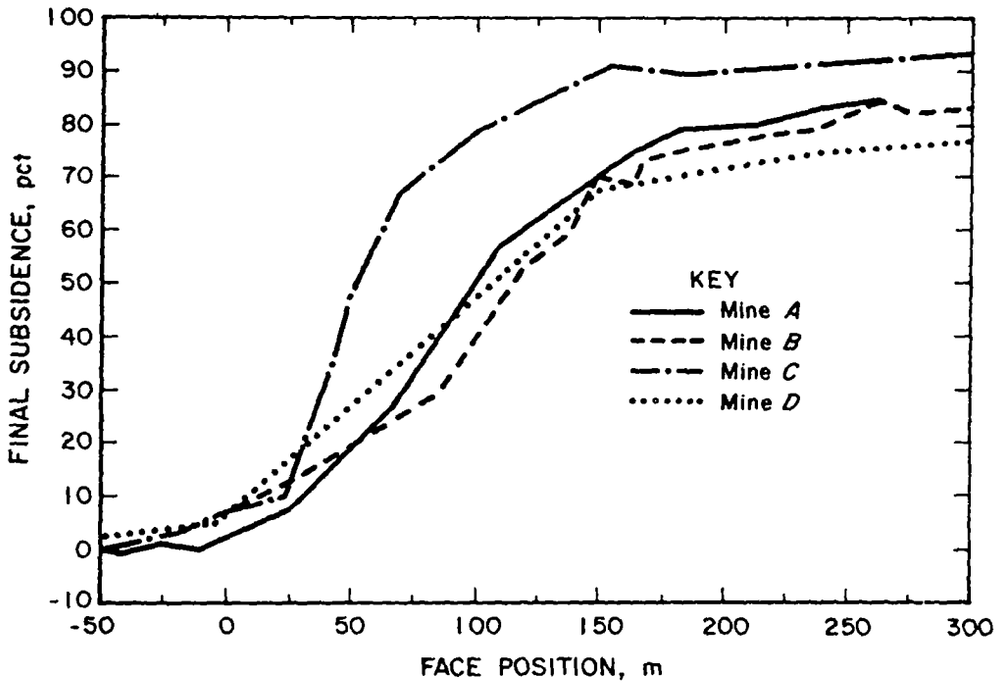

Figure 15. Rib -100 points versus face position

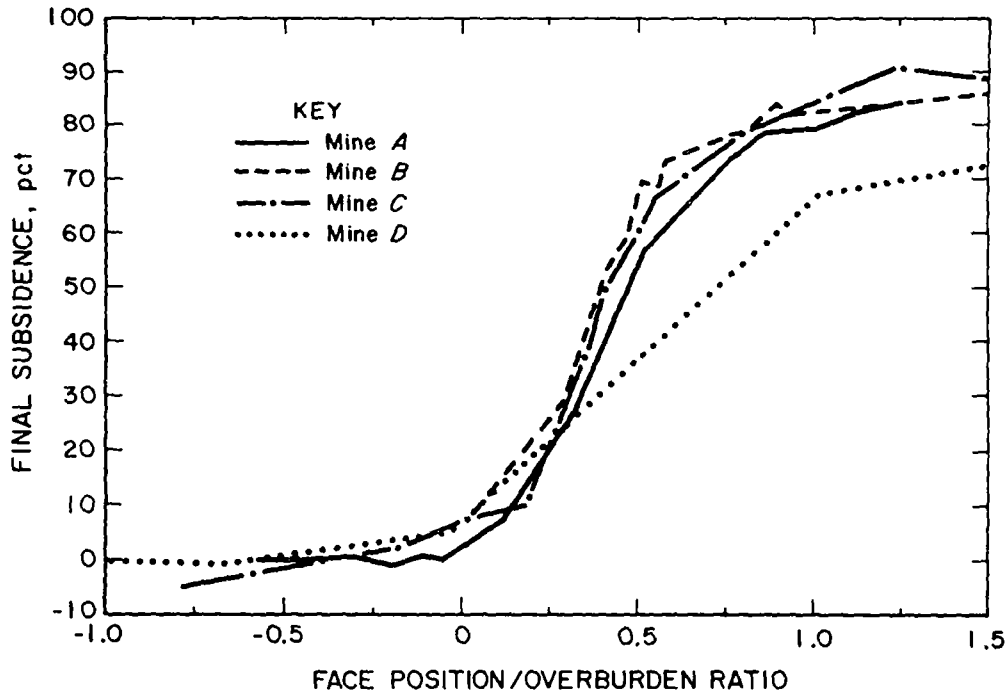

Figure 16. Rib -100 points versus face position/overburden

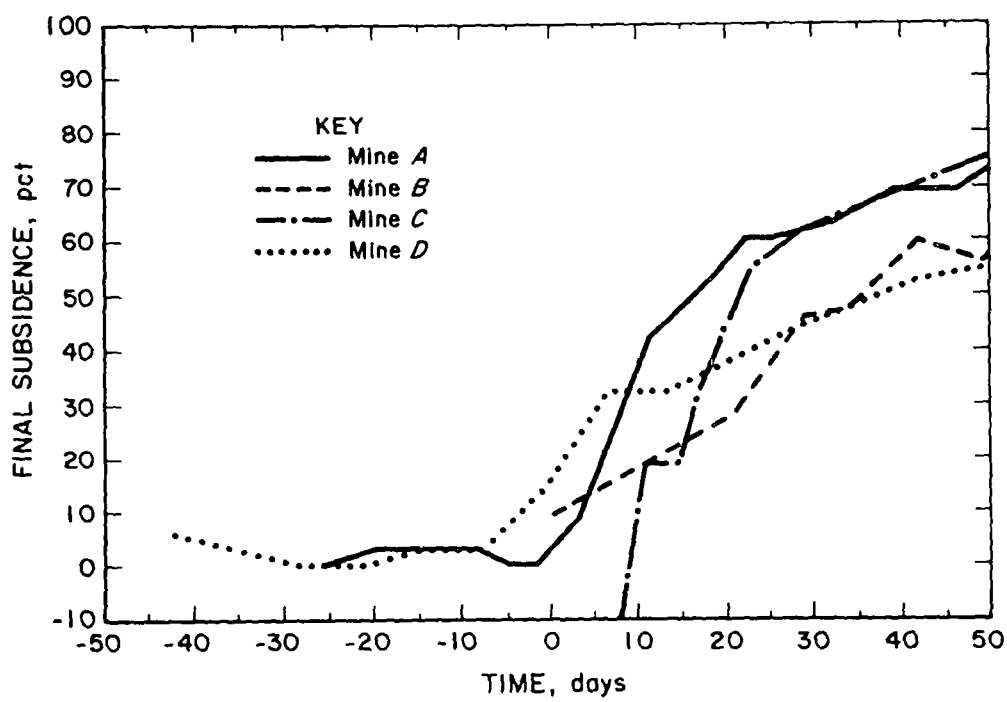

Figure 17. Rib points versus time

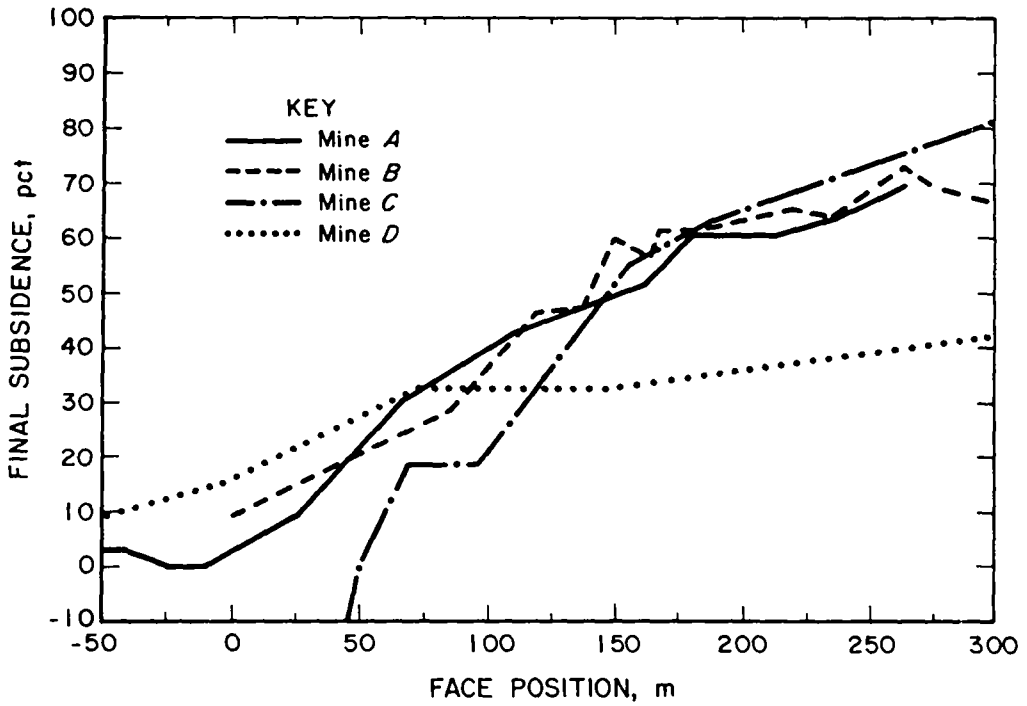

Figure 18. Rib Points versus face position

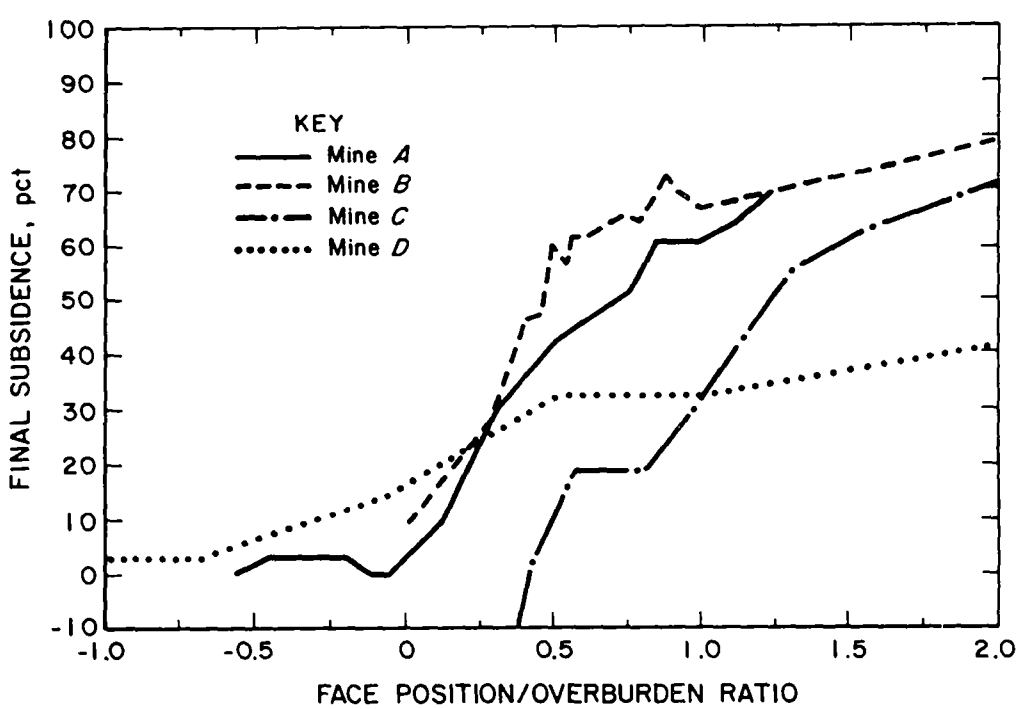

Figure 19. Rib Points versus face position/overburden

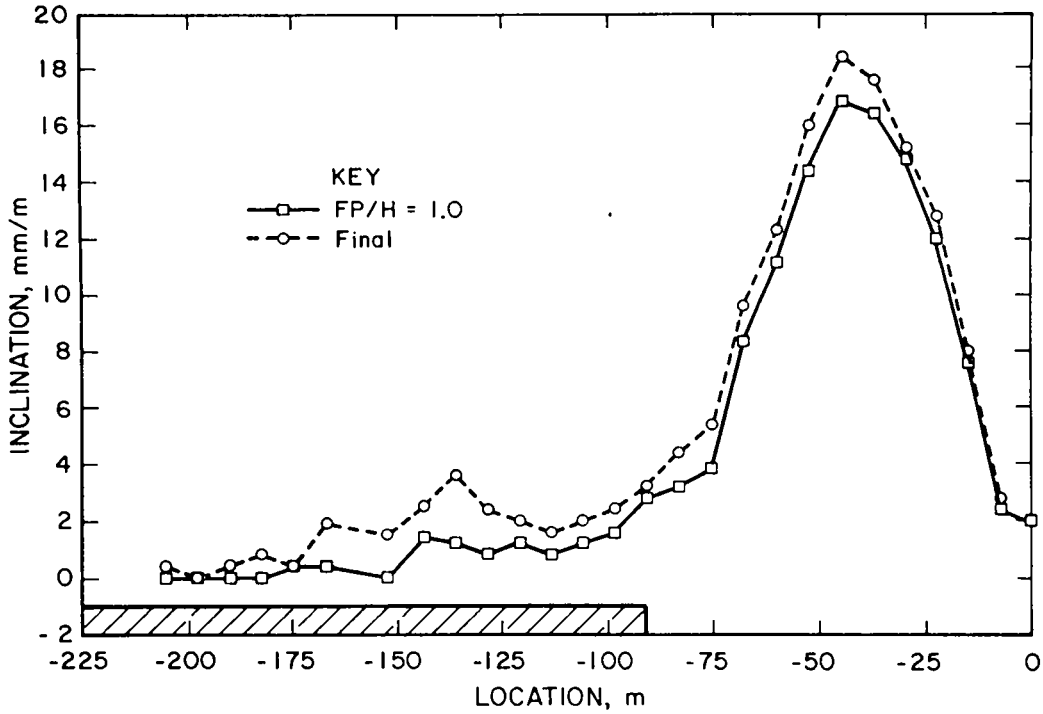

Figure 20. Inclination difference at Mine A

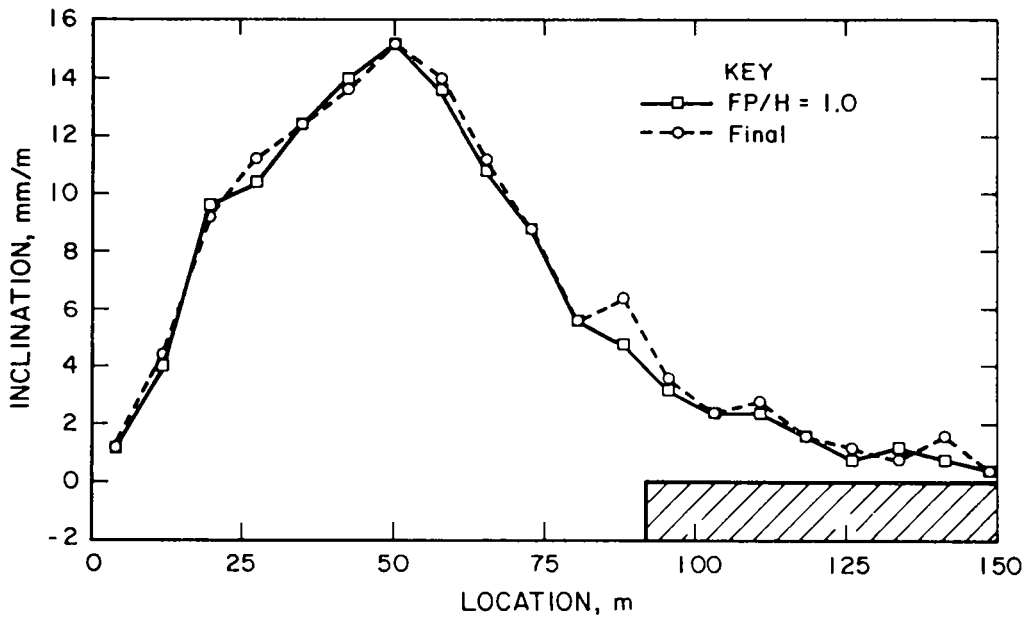

Figure 21. Inclination difference at Mine B

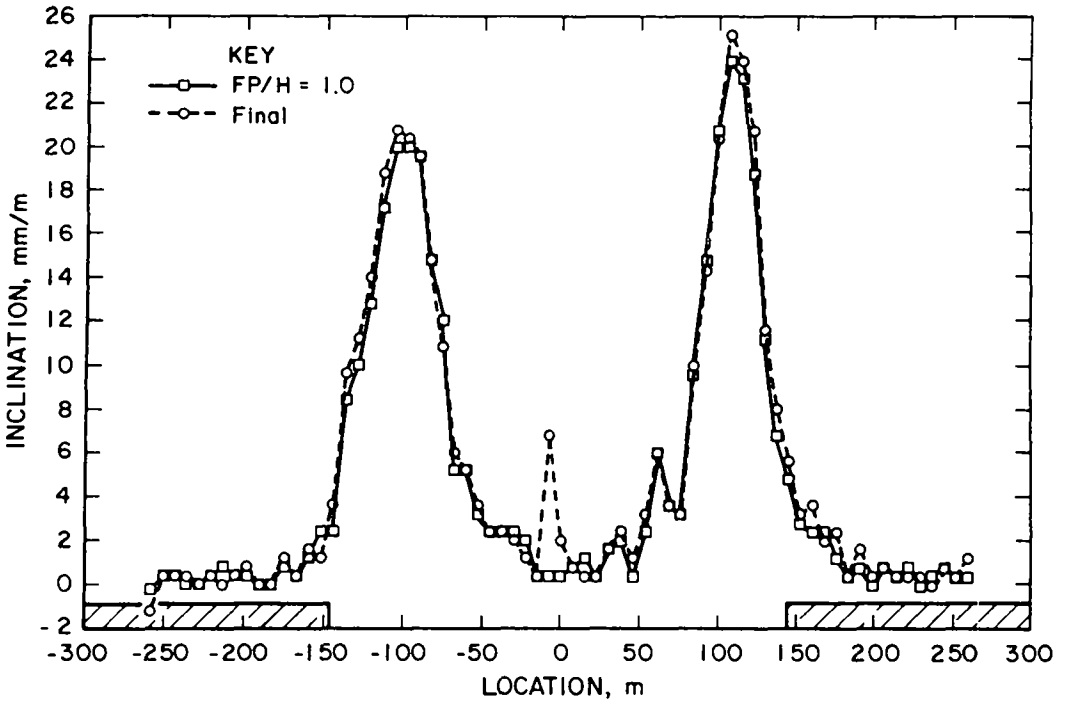

Figure 22. Inclination difference at Mine C

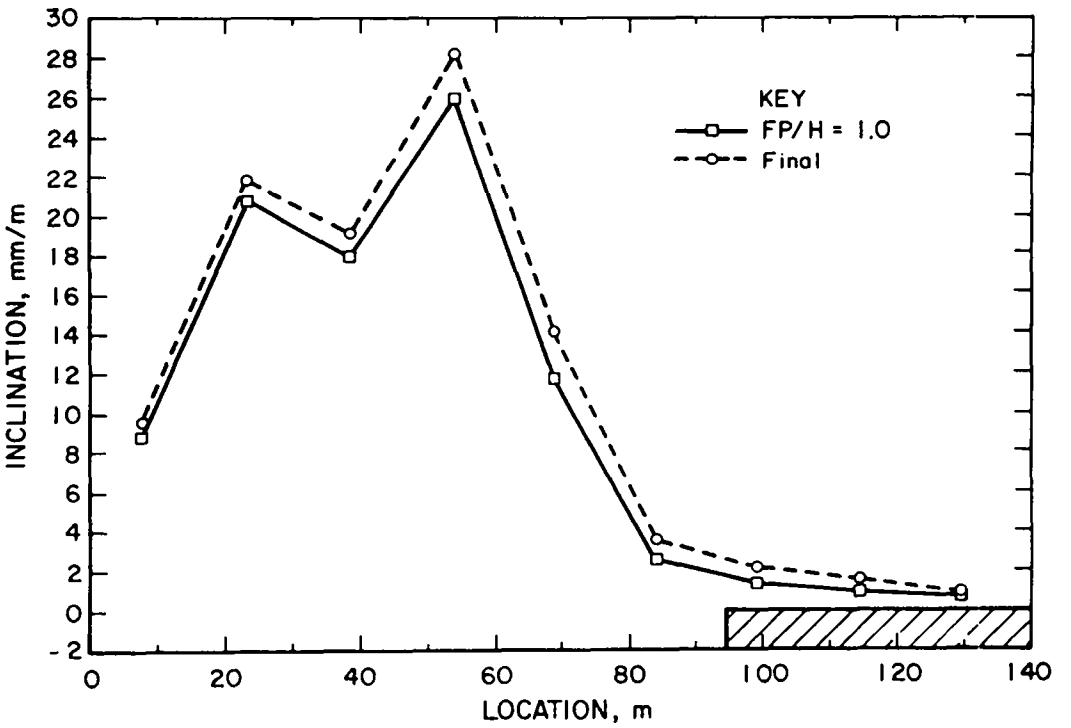

Figure 23. Inclination difference at Mine D

developing subsidence trough (within 30.5 m (100 ft) of the centerline) subsidence develops relative to face position and overburden thickness and is about 90 percent complete when the face has advanced the thickness of the overburden beyond the point in question. Above the rib of the panel the process of subsidence is very much site dependent and time appears to play a significant role, in that, when the face has advanced the thickness of the overburden past a monument only 40 to 70 percent of the movement has taken place. The remainder of the surface movement, irrespective of location above the longwall panel, takes up to a year to complete.

The monuments 30.5 m (100 ft) inside the rib exhibited movements that were characteristically between that of the centerlines and rib areas. With slow to moderate face advance (3.1 to 6.6 m/d), the subsidence process is similar to that over the central portion of the panel with over 80 percent of the final movement completed when the face has advanced the thickness of the overburden past a monument. However, with fast face advance (11.8 m/d) there appears to be some retardation or lag in the subsidence process.

At the sites studied, the preponderance of the surface deformations had occurred by the time that the longwall face had mined one overburden thickness past each profile. The additional deformations were small. All of these factors should be taken into account when designing a high extraction mine to evaluate the potential for damage resulting from mining-induced subsidence.

References

A Subsidence Prediction Model for Longwall Mine Design. P. W. Jeran, V. Adamek and M. A. Trevits, Proceedings of the Longwall USA Conference, Pittsburgh, PA, June 1986, pp. 101-112.

Static and Dynamic Subsidence Prediction in the Northern Appalachian Coal Region Based on the Use of a Variable Subsidence Coefficient. V. Adamek, P. W. Jeran, and M. A. Trevits. Procs. 3rd Workshop on Subsidence Due to Underground Mining, West Virginia University, Morgantown, WV, June 1-5, 1992.

Effects of Longwall Mining on Surface Sructures. J. S. Walker and M. A. Trevits. Procs. Mine Subsidence: Prediction and Control, 33rd Annual Meeting of Association of Engineering Geologists, Pittsburgh PA, October 1990, pp.141-143.

Subsidence Due to Undermining of Sloping Terrain: A Case Study. P. W. Jeran and V. Adamek. USBM RI 9205, 1988. 10 p.

Evaluation of Surface Deformation Characteristics Over Longwall Panels in the Northern Appalachian Coalfield. V. Adamek and P. W. Jeran. Proceedings Soc. Min. Eng. AIME 1 st Conf. on State-of-the-Art of Ground Control in Longwall Mining and Mining Subsidence. Honolulu HI, Sept 5-9, 1982, pp.183-198.

ERROR CORRECTION FOR VERTICAL SURVEYS CONDUCTED OVER A SUBSIDING LONGWALL MINING PANEL

by

D. Allen Hughes

Abstract: The difference between a conventional land survey and a survey of subsiding ground is discussed and a correction method was formulated for surveys conducted on subsiding ground. The area over the longwall mining panel subsided detectible amounts during the time required to conduct the survey when subsidence was at its highest rate, which introduces error into the survey. When the ground subsides before the survey is completed, the survey no longer represents the locations of all points at a common point in time, which is a basic assumption of conventional land surveying. Conventional methods of correction average movement of subsiding points and apply those amounts of movement to points which were unaffected by subsidence, a different correction method was needed. A correction method was used which uses multiple surveys to calculate rates of subsidence for each point in the survey. Subsidence rates were used to estimate the location of each point at a common time. Results are presented using the correction for subsiding ground and using no correction. Different results of the same surveys are shown in terms of elevations and curvatures. The significance of the different types of corrections is discussed and the compounding of error is demonstrated when calculating curvatures.

Additional Key Words: Longwall Mining, Error Correction, Subside.

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[3]Publication in this proceedings does not prevent authors from publishing their manuscripts, whole or in part, in other publication outlets."

Introduction

Mine subsidence is the product of a chain of events that begins several hundred feet below the surface of the ground and propagates upward. The end result is a subsidence trough. Strains caused by subsidence in the area of the trough is the direct cause of structural damage. An accurate, time efficient and cost-effective method of measuring the behavior of the trough is critical. Perhaps the most reliable and accepted method of monitoring mine subsidence is by the use of conventional surveying techniques. However, some modifications to the surveying procedure must be made when monitoring mine subsidence.

Survey Setup

Multiple surveys were conducted over a longwall mining panel near West Frankfort in southern Illinois. Monitoring by surveying was started before the area began to subside, conducted at daily intervals during the peak subsidence and continued until subsidence had ceased. The area subsided between the months of May and August of 1990.

Soils of southern Illinois are typically highly expansive. To accurately measure subsidence, it was necessary to avoid measuring movements caused by other sources, such as freezing and thawing of the soil, and shrinking and swelling of the soil caused by changes in the moisture content. These types of movement occur most often near the surface.

Each survey monument had to be installed so that it would remain as stable as possible. A steel bar was isolated from the soil by surrounding it with sand. The sand allowed some movement near the surface while the bar remained in the ground below the freeze line (Figure 1).

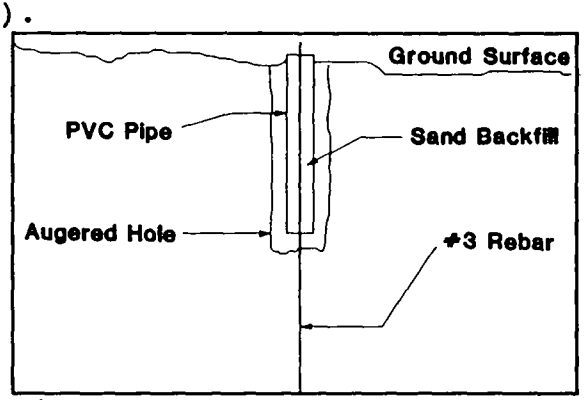

Figure 1. Survey Monument Construction.

The survey monuments were constructed of 1.5 m (5 ft), #3 steel reinforcing bar, 0.075 m (3 in.) O.D. PVC pipe, and sand. A 0.15 m (6 in) diameter hole was augered into the ground to a depth of about 0.5 m (1.5 ft). The reinforcing bar was then driven into the bottom of the hole until about 0.05 m (2 in) of the bar protruded above the surface of the ground. The PVC pipe was then placed in the hole around the bar and back-filled with tamped sand. The hole around the pipe was filled with tamped soil. The PVC pipe was then capped to isolate the sand from moisture. Lines of survey monuments were set up crossing and along the centerline of the subsidence trough. Shorter lines of monuments were also installed adjacent to test footings constructed on the center-line and on the edge of the subsidence trough.

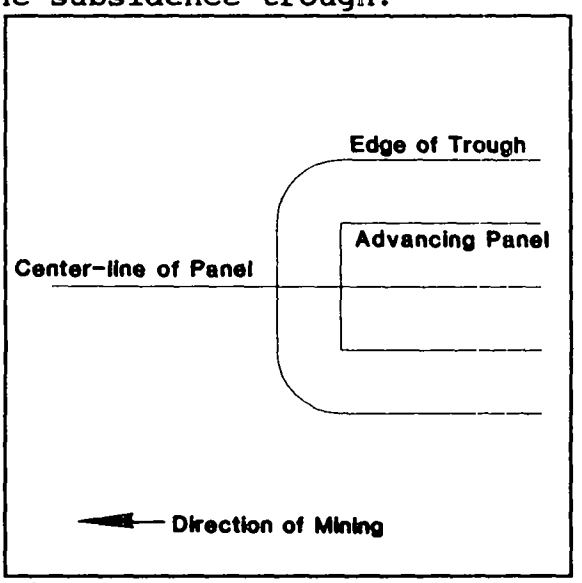

Figure 2. Layout of Survey Monuments.

To demonstrate the correction, a survey of fifteen monuments, numbered 1 through 15, located along the centerline of a subsiding longwall mining panel will be used (Figure 2). Level surveys were conducted on

surveys used to demonstrate the correction method were conducted when the rate of subsidence was at its highest. Two surveys were conducted on the day which the maximum amount of subsidence occurred.

Vertical Survey

To obtain accurate measurements, a Pentax level with a micrometer accurate to 0.0001 m (0.0003 ft) was used with a Nedo temperature invariant leveling rod. The elevation of a point is determined by sighting the leveling rod through the optical level. The reading on the leveling rod indicates the elevation difference between the bottom of the leveling rod and the sights on the level. This is known because the optical level sights the leveling rod on the horizontal plane. Each time the level is set up, the operator back sights on a point of known elevation. Since the difference in elevation of the point of known elevation and the level can be determined, the elevation of any new measurement can be determined by finding the elevation difference between the level and the new point.

The initiation of subsidence was determined to be on May 10, 1990 and the end of subsidence was determined to be on August 8, 1990. The magnitude of subsidence was determined by subtracting the elevation of a given point before subsidence from the elevation of the same point after subsidence. The maximum amount of subsidence observed along the centerline was 1.425 m (4.674 ft). Over fifty percent of the total subsidence occurred in 5 days, when the subsidence rate was also at its greatest (Bennet et al).

Conventional Error Correction

Conventionally, when a survey is conducted, a loop is closed and a closure error is calculated. Closure error for a vertical survey is determined by measuring a series of elevations consecutively to create a loop. At some point, a point in the survey is used to calculate the elevation of a point which has already been surveyed. If the second measurement differs from the first, this is considered to be the amount of error in the survey loop. In a conventional survey, the total amount of error is then distributed evenly among all the points in the loop. For example, if a loop of ten survey points is determined to have a total error of 1 cm then each elevation in the loop would be adjusted by one tenth of a centimeter.

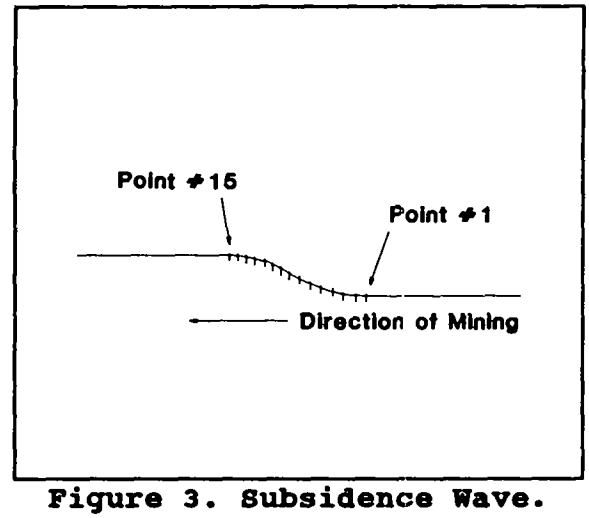

Figure 3. Subsidence Wave.

Typical Subsidence Surveys

One of the basic assumptions of conventional surveying is that all distances

remain constant and any discrepancy is due to error or lack of precision. This is not always the case when surveying mine subsidence. Most published results of subsided mine surveys were begun at day 0 and continued for months or even years after the surface was undermined or collapsed. These surveys were conducted at monthly or weekly intervals. Since most surveys of subsided areas are started after subsidence has ceased, it is correct to use conventional error correction techniques. Surveys of subsided areas commonly apply no correction at all to the survey (Peng, 1992). In this project surveys were conducted either once or twice per day during the time of the maximum rate of subsidence. Subsidence occurred in measurable amounts during the two hours required to conduct the survey. Since the ground subsided before the loop could be closed, adjusting the distances would create more error than would be corrected.

It is possible to correct for subsidence that occurs during the survey, by obtaining rates of subsidence and simulating an instantaneous survey. A correction method was tested over the center line of an advancing longwall mining trough. Subsidence induced by longwall mining is the most convenient type of subsidence to monitor, because it is known when subsidence will begin in the area above the mine.

At its maximum rate, the ground surface over a longwall mining panel can subside at about 1 cm (2.5 in) per hour (Table 1). The rate of subsidence depends on the position of a given point on the subsidence wave (Figure 3).

The problem with calculating closure error of a subsiding surface is that the survey cannot be completed before the ground moves. Closure error could be calculated on a subsiding surface only if an instantaneous survey is conducted. Since this is impractical, if not impossible with current technology, the only logical alternative is to simulate an instantaneous survey.

If a line of points is surveyed, it takes some finite period of time to conduct the survey. That finite period of time is the difference between the initial time , when the first measurement is recorded, and the final time when the final measurement is taken.

In order to know precisely what the profile shape is, one would need to know what the profile is at an instant in time. As the survey is conducted, the survey monuments subside from the initial instantaneous profile to the final instantaneous profile. The problem is that the survey spanned a finite time period and does not represent either of the true subsidence profiles.

An instantaneous profile can be simulated by calculating a rate of subsidence for every point on the survey. After the subsidence rates are calculated, all points can be projected forward or backward in time (Table 1).

Table 1. Comparison of Actual and Corrected Profiles

Point	5-23-90 AM Elev. Drop (m)	AM Time of Measure	5/23/90 PM Elev. Drop (m)	PM Time of Measure	AM/PM Delta Elev. (m)	Subsidence Rate (m/hr)
1	0.737	8:00	0.820	4:00	0.083	0.01038
2	0.667	8:04	0.756	4:04	0.089	0.01113
3	0.598	8:08	0.687	4:08	0.089	0.01113
4	0.571	8:12	0.659	4:12	0.088	0.01110
5	0.541	8:16	0.630	4:16	0.089	0.01113
6	0.515	8:20	0.601	4:20	0.086	0.01075
7	0.487	8:24	0.572	4:24	0.085	0.01063
8	0.458	8:28	0.540	4:28	0.082	0.01025
9	0.435	8:32	0.516	4:32	0.081	0.01013
10	0.409	8:36	0.487	4:36	0.078	0.00975
11	0.383	8:40	0.458	4:40	0.075	0.00938
12	0.361	8:44	0.434	4:44	0.073	0.00913
13	0.334	8:48	0.404	4:48	0.070	0.00875
14	0.278	8:52	0.340	4:52	0.062	0.00775
15	0.221	8:56	0.273	4:56	0.052	0.00650

One foreseeable source of error would be a change in subsidence rate during the time taken to conduct the survey. For this reason loops should be arranged so that completion can be achieved within as short a time span as possible.

To calculate subsidence rates in the field, the elevation of each point must be measured twice. Two surveys should be conducted on each loop with the time of each measurement also recorded. The measurement of the first point is taken at the initial time, t_1, the second at t_2, the third at t_3, etc. With two different elevations for each point at two different times, the data can be used to calculate the subsidence rate for each point. Figure 4 plots projected data points shown with the two sets of actual data used to calculate the subsidence rates. The surveys were conducted approximately eight hours apart. An eight-hour interval was used because this was the smallest interval available. Ideally one would want two surveys conducted about an hour apart to insure that the subsidence rates were relatively the same.

Figure 5 compares the curvature of the actual profiles of points 1 to 15 with the corrected profiles of the same points.

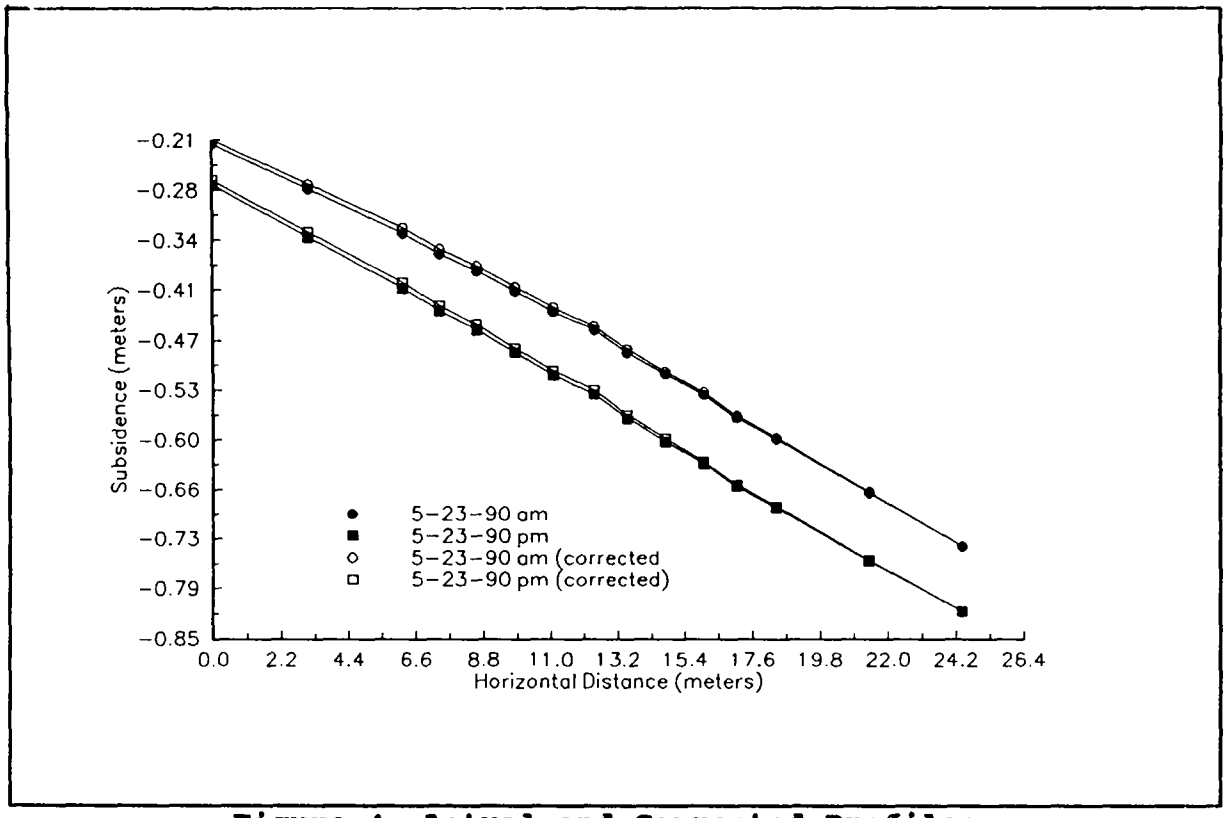

Figure 4. Actual and Corrected Profiles

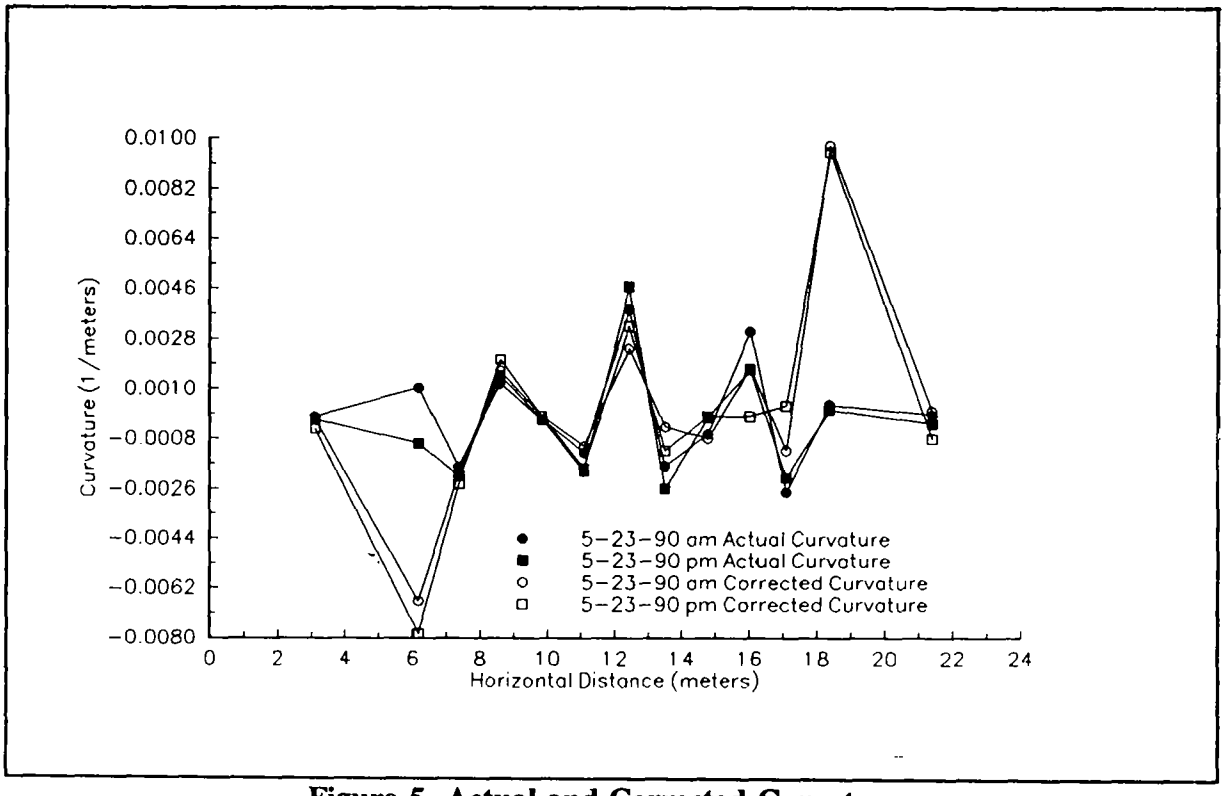

Figure 5. Actual and Corrected Curvatures

Table 2. Comparison of Actual and Corrected Curvatures

Point	AM/PM Correction	AM Corrected Elevation (m)	PM Corrected Elevation (m)	AM Curve 1/(m)	PM Curve 1/(m)
1	0.0000	0.7370	0.8200		
2	0.0008	0.6662	0.7552	-0.0002	-0.0008
3	0.0015	0.5965	0.6855	-0.0007	-0.0095
4	0.0022	0.5688	0.6568	-0.00012	0.0004
5	0.0030	0.5380	0.6270	0.00017	0.0000
6	0.0036	0.5114	0.5974	-0.0008	0.0000
7	0.0042	0.4828	0.5678	-0.0004	-0.0012
8	0.0048	0.4532	0.5352	0.0024	0.0033
9	0.0055	0.4295	0.5105	-0.0011	-0.0019
10	0.0058	0.4032	0.4812	0.0000	0.0000
11	0.0062	0.3768	0.4518	0.0016	0.0020
12	0.0067	0.3543	0.4273	-0.0020	-0.0024
13	0.0070	0.3270	0.3970	-0.0067	-0.0079
14	0.0067	0.2713	0.3333	-0.0001	-0.0005
15	0.0061	0.2149	0.2669		

The differences in curvature are substantial. Since the correction adjusts each point according to the time that elapses after the first measurement is taken, one end of the survey line is adjusted more than the other (Figure 4). The first point required no adjustment. This is the most significant aspect of this correction method. If the correction only shifted the whole line some uniform distance as done with closure correction, then it would be trivial. Since the correction is nonuniform, it deserves more attention.

Conclusions

It appears that the calculation of closure error is of little significance compared to the correction from projecting the points to time t_1. A plot of the points after calculation of closure error is indistinguishable from that of the projected points. Since the rate of subsidence can vary significantly over short horizontal distances, i.e. distances of 5 m (16 ft) or less, the survey should be conducted as quickly as possible, regardless of the correction method used.

In most cases, the most precise methods and equipment for surveying are used to monitor mine subsidence. Greater than normal care is taken to conduct the surveys required to monitor mine subsidence. A monitoring method which produces more accurate results would be beneficial. Since the profile which most nearly represents an instantaneous survey of the subsiding area is desired, this correction method should de used for surveys which have detectable amounts of movement taking place during the survey.

This method would be useful for measurement of any subsiding area or structure which subsides at a rate which would introduce measurable amounts of subsidence before the survey could be conducted. When the survey data is intended to be used for calculation of curvatures or other properties which involve the multiplication of error, the correction has the greatest significance.

Literature Cited

Bennet, R.M. and Drumm, E.C. 1992. Subsidence Effects on Structures. p 5-6. Institute for Geotechnology Report #92-01.

Peng, S.S. 1992. Surface Subsidence Engineering. Braun-Brunfield, Inc. Ann Arbor, MI. 69.

A HISTORY OF METAL SHAFT/PORTAL CLOSURES IN UTAH[1]

by

Louis A. Amodt[2]

Abstract. Over the last twelve years the Utah Abandoned Mine Reclamation Program has utilized metal shaft and portal closures when physical constraints required an alternative method of closure to backfill or masonry seals. Eight types of metal closures have been installed with varying degrees of success. Relative advantages/disadvantages of these closures are discussed. The eight closure types are 1) A-frame "bird cages", 2) large diameter cyclone fence-type grid, 3) bar grates, 4) rebar grates, 5) cable nets, 6) steel doors, 7) angle iron bat grates, and 8) "jail bar" steel bat grates. The primary application of the fabricated metal closures has been in non-coal mines. Only the angle iron bat grate has been installed in abandoned coal mines to date. Bird cages placed in avalanche zones failed and the shafts were later backfilled. Large cyclone fence-type grids were placed in the Wasatch and Tintic mountains in 1985. Rebar grates have been used in the Wasatch and Tintic mountains as an alternative to backfill. Bar grating was used in the Tintic mountains when the aesthetics of an historical headframe dictated a less visible closure. Cable nets have been installed in Canyonlands National Park utilizing a design pioneered in Death Valley National Monument. Steel doors have been utilized where the landowner requests access or a need to access the adit exists. Two types of bat grates have been utilized in both coal and non-coal mines where sensitive or endangered bat species have been identified. The bird cage design has not been as effective as the others. Indications are the rebar grates, angle iron bar grates, and steel doors excel in giving long term protection to the public, with the second generation bat grate giving the maximum protection to the public.

Additional Key Words: Shaft and Adit Reclamation, Reclamation Costs, Metal Closures, Bat Protection, Hazard Mitigation

Introduction

During the last twelve years the Utah Abandoned Mine Reclamation Program (AMRP) has been utilizing a variety of metal closures to address the problem of effectively sealing abandoned shafts and adits when standard techniques are unsuitable. Securing dangerous abandoned mines with a less than permanent seal is affected by the location and ease of accessibility to the site. The easier the accessibility, the higher the visitation and possible vandalism. Thus, the closer and more accessible the site, the stronger the design must be to deter vandalism. Sites located on steep slopes at high elevations may experience not only vandalism problems but also snow loads and possible avalanche damage.

Eight designs of fabricated steel closures have been employed by the Utah AMRP with varying degrees of success in both longevity and durability. The eight closure types are 1) A-frame "bird cages", 2) large diameter cyclone fence-type grid, 3) bar grates, 4) rebar grates, 5) cable nets, 6) steel doors, 7) angle iron steel bat grates, and 8) "jail bar" steel bat grates. The primary application of the metal closures has been in non-coal mines. Only the first generation (angle iron) bat grate has been installed in abandoned

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Publication in this proceedings does not prevent author from publishing the manuscript whole or in part, in other publication outlets.

coal mines in Utah to date.

Costs for the fabricated metal closures range from a high of $2,675.37/m² ($247.69/ft²) for a one time use bar grate closure to a low of $72.06/m² ($6.68/ft²) for installation of large rebar grate closures. A cost analysis was performed and shows a comparable cost for the metal closure when compared to the volume of backfill that would be required for filling the shaft or adit. Table 1 summarizes the costs of each closure completed by the Utah AMRP over the last twelve years.

A-Frame "Bird Cage"

The A-frame bird cages were first used in the Wasatch Mountains above Salt Lake City in 1982 and 1983 (Figure 1). The design consists of a 95 mm (3/8-inch) square mesh on 25 cm (10-inch) centers bolted to a 5 cm (2-inch) by 8 cm (3-inch) by 48 mm (3/16-inch) tubing frame on 61 cm (2-foot). The square mesh was secured by u-bolts to the framing. The bird cages were adjacent to and within the Wasatch-Cache National Forest near four ski resorts. Visitor use days within the Wasatch-Cache National Forest were in excess of 2,000,000 user days in the 1990-91 ski season alone. (Fjeldsted and Hachman 1991) The 2,000,000+ figure does not include the hiker traffic during the summer months. Many shafts in the area extend over 300 meters (1,000 feet) in depth and may be used as recharge or ventilation of the aquifer. Backfill material is limited or nonexistent at the sites.

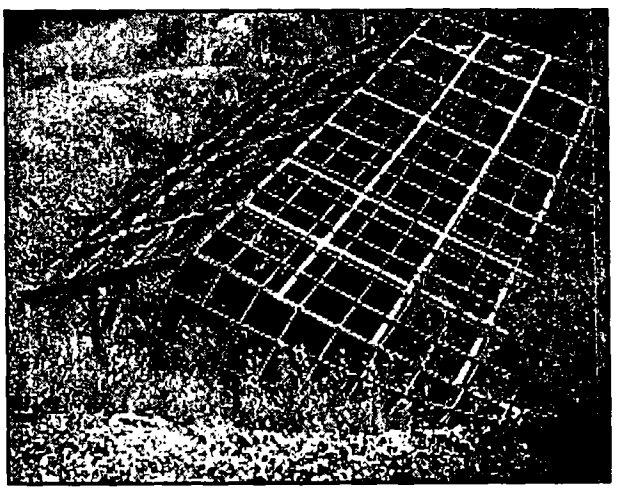

Figure 1. A-Frame "Bird Cage" Grate closure in Alta-Brighton Project, 1984.

The bird cages were designed to protect the public from entering the shafts and performed well to restrict the recreational skier or hiker. The more inquisitive public found that the square mesh could be bent back and the framing used to anchor ropes for descending down the shafts. Sloughing of the shaft collar was a problem with a loss of support for the concrete pillars or epoxy roof bolts near the edge of the collar. The bird cages were transported to the sites by ski, snow cat, and helicopter. Three of the bird cages collapsed due to heavy snow loads from avalanches. The location of the shaft in relation to potential avalanches and competence of the collar material must be evaluated when any type of closure other than backfill is utilized.

The costs for this type of closure range from $1,200 for a 1 m x 1.4 m (3.5' x 4.5') cage to $6,640 for a 13 m x 7 m (42' x 23' m) cage with an average size of 5 m x 4 m (15' x 12') cage at an average cost of $3,512 or $483.76/m² ($46.63/ft²) including the fabrication of the custom fit cages.

To date eight of 15 bird cages are still secure in the Wasatch Range. Five shafts with failed bird cages ranging in depth from 6 meters (20 feet) to 212 meters (400 feet) were later backfilled by hand. One bird cage was repaired by expanding the size of the 95 mm (3/8-inch) mesh by 3 m (10 feet).

Large Cyclone Fencing Grid

The large cyclone fencing grid was designed to withstand the heavy snow loads encountered high in the Wasatch Range, and in response to the failures seen with the bird cages under heavy loading (Figure 2). Based on the known capability of cyclone fencing to distribute the load and flex, a design consisting of 95 mm (3/8") diameter hot dipped galvanized wire twisted into a cyclone fence on 15 cm (6-inch) centers was fabricated. Agutter Engineering of Salt Lake City designed a jig to turn the 95 mm (3/8") diameter bar for the cyclone fencing in 4 m (12') lengths which were threaded together for the required length of "fencing". Where the length exceeded 4 m (12'), the sections

Table 1 - Metal Closure Costs by Site

SITE	SIZE (FEET)	SIZE (METERS)	TYPE	COST
AB-1	3.5' x 4.5'	1.07 m x 1.37 m	Bird Cage	$1,200
AB-2	6' x 8'	1.83 m x 2.44 m	"	$3,575
AB-3	6' x 10'	1.83 m x 3.05 m	"	$2,950
AB-4	6' x 5'	1.83 m x 1.52 m	"	$2,750
AB-5	10' x 10'	3.05 m x 3.05 m	"	$3,000
AB-6	42' x 23'	12.8 m x 7.01 m	"	$6,640
AB-7	17' x 14'	5.18 m x 4.27 m	"	$4,000
AB-13	26' x 8'	7.92 m x 2.44 m	"	$5,575
AB-15	13' x 17'	3.96 m x 5.18 m	"	$3,650
AB-18	15' x 15'	4.57 m x 4.57 m	"	$2,600
AB-22	5' x 8'	1.52 m x 2.44 m	"	$2,450
AB-38	6' X 8'	1.83 m x 2.44 m	"	$3,750
BB-BB	5' x 11'	1.52 m x 3.35 m	Bar Grate[3]	13,623
BB-EH	16' x 32'	4.88 m x 9.75 m	Cyclone Fence[4]	$6,245
BB-GE	24' x 34'	7.32 m x 10.36 m	"	13,007
BB-RV	44' x 36'	13.41 m x 10.97 m	"	13,781
BB-SA	32' x 38'	9.75 m x 11.58 m	"	11,136
A-17	12' x 16'	3.66 m x 4.88 m	"	$4,375
W-2	4' x 8'	6.10 m x 6.10 m	Rebar Grate[5]	$ 384
W-35	4' x 6'	1.22 m x 1.83 m	"	$ 792
W-119	4' x 4'	1.22 m x 1.22 m	"	$ 500
W-312	12' x 15'	3.66 m x 4.57 m	Rebar Grate[6]	$1,485
W-324	18' x 24'	5.49 m x 7.32 m	"	$2,376
W-350	8' x 16'	2.44 m x 4.88 m	"	$3,200
T-3	20' x 20'	6.10 m x 6.10 m	Rebar Grate[7]	$2,300
T-7	36' x 36'	10.97 m x 10.97 m	"	$8,976
T-8	13' x 16'	3.96 m x 4.88 m	"	$1,700
T-13	35' x 40'	10.67 m x 12.19 m	"	$5,400
T-14	15' x 18'	4.57 m x 5.49 m	"	$2,000
T-15	9' x 13'	2.74 m x 3.96 m	"	$1,000
T-16	24' x 24'	7.32 m x 7.32 m	"	$3,400
T-17	34' x 35'	10.36 m x 10.67 m	"	$7,100
T-19	24' x 25'	7.32 m x 7.62 m	"	$3,500
T-21	12' x 14'	3.66 m x 4.27 m	"	$2,100
T-22	20' x 25'	6.10 m x 7.62 m	"	$3,000
T-27	12' x 16'	3.66 m x 4.88 m	"	$1,092
T-28	14' x 14'	4.27 m x 4.27 m	"	$1,100
T-30	18' x 22'	5.49 m x 6.71 m	"	$2,300
T-36	32' x 35'	9.75 m x 10.67 m	"	$6,632
T-38	10' x 10'	3.05 m x 3.05 m	"	$4,000
L-H1b	7' x 6'	2.13 m x 1.83 m	Cable Net	$ 882
L-H3b	11' x 6'	3.35 m x 1.83 m	"	$1,386
L-H4a	19' x 7'	5.79 m x 2.13 m	"	$3,234
L-H4c	10' x 7'	3.05 m x 2.13 m	"	$1,995
L-H4d	7.5' x 7'	2.29 m x 2.13 m	"	$1,113
L-H4e	7.5' x 6.5'	1.98 m x 1.98 m	"	$ 903
L-HO5	10' x 8'	3.05 m x 2.44 m	"	$1,680
L-HO6	7' x 9'	2.13 m x 2.74 m	"	$1,323
L-HO7	9.5' x 7'	2.90 m x 2.13 m	"	$1,407
L-HO8	7' x 8.5'	2.13 m x 2.59 m	"	$1,260
W-67	6' x 6'	1.83 m x 1.83 m	Steel Door[8]	$ 965
W-89	4' x 4'	1.22 m x 1.22 m	"	$ 975
W-103	6' x 6'	1.83 m x 1.83 m	"	$ 975

FN-B1	10' x 6'	2.90 m x 2.44 m	Angle Iron	$4,500
FN-B3	12' x 6'	3.66 m x 1.83 m	Steel Bat	$5,400
FN-B4	10' x 6'	3.05 m x 1.83 m	Grate	$4,500
FN-F1	6' x 6'	1.83 m x 1.83 m	"	$3,000
FN-K1	13' x 8'	3.96 m x 2.44 m	"	$8,150
O-P1	9.5' x 8'	2.90 m x 2.44 m	"	$2,160
O-P2	6' x 9'	1.83 m x 2.74 m	"	$2,835
O-P3	8' x 6'	2.44 m x 1.83 m	"	$2,160
O-P4	9' x 7'	2.74 m x 2.13 m	"	$2,160
O-P5	6' x 8'	1.83 m x 2.44 m	"	$3,040
W-77[9]	5' x 7'	1.52 m x 2.13 m	"	$2,725
W-291	7' x 9'	2.13 m x 2.74 m	"	$4,975
S-3	3' x 2'	0.91 m x 0.61 m	Jail Bar Steel	$1,500
S-4	7' x 6'	2.13 m x 1.83 m	Bat Grate	$5,250
S-38	8' x 6'	2.44 m x 1.83 m	"	$7,280
S-42	3' x 6'	0.91 m x 1.83 m	"	$2,250
S-43	4' x 6'	1.22 m x 1.83 m	"	$3,000
S-46	5' x 3'	1.52 m x 0.91 m	"	$3,640
S-52	5' x 5'	1.52 m x 1.52 m	"	$3,125
S-68	5' x 4'	1.52 m x 1.22 m	"	$3,125
S-73	6' x 5'	1.83 m x 1.52 m	"	$3,750
S-78	8' x 5'	2.44 m x 1.52 m	"	$5,000
S-79	5' x 6'	1.52 m x 1.83 m	"	$4,000
S-80	4' x 6'	1.22 m x 1.83 m	"	$3,120
S-84	4' x 2'	1.22 m x 0.61 m	"	$5,000
S-85	5' x 8'	1.52 m x 2.44 m	"	$5,970
S-105	6' x 6'	1.83 m x 1.83 m	"	$4,500
S-107a	4' x 5'	1.22 m x 1.52 m	"	$2,700
S-107b	7' x 6'	2.13 m x 1.83 m	"	$5,460

[3] Cost includes concrete stabilization of collar area and repair of headframe.
[4] Cost includes fabrication and installation without cost of access improvements or site grading.
[5] Rebar grate consists of #5 rebar on 6" centers, rebar drilled into collar without the placement of a concrete grade beam.
[6] Rebar grate consists of #5 rebar on 6" centers with the placement of a concrete grade beam.
[7] Rebar grate consists of #6 rebar on 8" centers with the placement of a concrete grade beam.
[8] Steel Door closure includes concrete block placed around the steel structure to secure the door.
[9] Angle Iron Steel Bat Grate closure includes mild steel frame allowing bat grate to be opened like a gate.

Figure 2. Cyclone Fence Grid closure in Bullion-Beck Project, 1985.

were bolted together with 12 mm (½") bolts. The design was made to allow the grid to be installed over shafts with uneven collar configurations. However, more grooming of the collar was required than had been anticipated. The first cyclone grid was installed in the Alta area above Salt Lake City in 1985. The grid overlapped the collar 90 cm to 120 cm (3 to 4 feet) with roof bolts installed a minimum of 30 cm (1-foot) to anchor the grid into the bedrock. A 95 mm x 5 mm (2" x 3/8") bar strap was threaded around the perimeter of the grid for attachment of the anchor.

Later in 1985 the same type of grid was used in the Bullion Beck Project with one alteration to the design. A S12 x 22.5 I-beam was installed around the perimeter and

anchored to the dump material utilizing a 61 cm (24") anchor disk as a deadman placed 3 m to 5 m (10' to 15') back from the collar on all four sides of the grid. The anchor disks were secured to the grid with 12 mm (½") galvanized steel cable. The anchor disks were used to allow for possible failure of the collar without compromising the integrity of the closure. Site preparation for the grid required a relatively flat surface with the anchors placed in the dump material surrounding the collar. The perimeter I-beam was covered with dump material to blend the closure with the surrounding terrain. This design allowed for placement around the base of headframes without disturbing the integrity of the structures. None of the grids placed in 1985 have been compromised by collar failure or vandalism. Equipment was utilized to place the grid, I-beam, and excavate for the deadman anchors. Equipment access is required for this type of installation.

The grid size ranged from 4 m x 5 m (12' x 16') in the Alta Project to a maximum size of 14 m x 11 m (44' x 36') at the Bullion Beck Project. The average cost was $148.01/m² ($13.81/ft²) including the fabrication. The cost of site grading and covering of the perimeter of the grate with soil materials is not included in the above costs.

Bar Grate

A bar grate design was developed as a specialized application for the Bullion Beck Project in 1985 when the cyclone fence grid design was determined to be obtrusive to the visual appearance of a historic headframe (Figure 3). The wooden collar lining of the shaft was deteriorating near the main support for a 2-post type headframe. A concrete collar support was installed with bar grating sized to approximate the original dimensions of the collar of the shaft. The final design with the skip guides attached to the wooden beams around the collar gives the appearance of the original structure while securing the collar from safety hazards. The bar grate is constructed of 12 mm (½") diameter bar on 5 cm (2") centers with 25 mm (1") square bar on 61 cm (2') centers supporting the bars.

The cost of the 1.5 m by 3 m

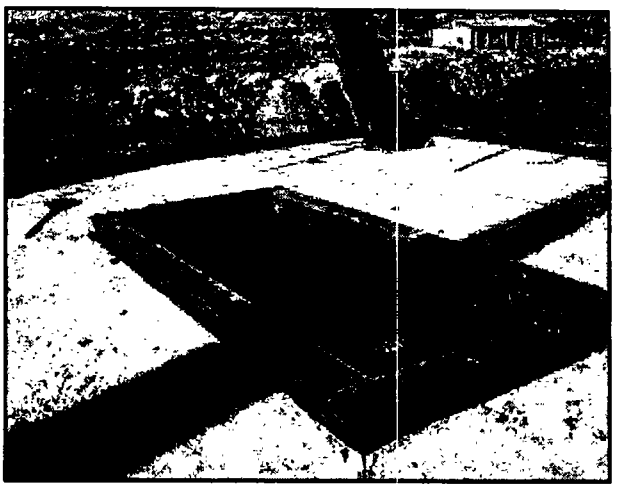

Figure 3. Bar Grate closure in Bullion-Beck Project, 1986.

(5' by 11') grate including concrete work to prepare the site and anchor the grate was $13,623.00 or an average of $2,675.37/m² ($247.69/ft²). This number is high due to the amount of concrete used to stabilize the collar before installation of the grate. The cost of the installation of the bar grating would be substantially lower if a minimum of site preparation were to be required. Equipment access was required for this installation.

Rebar Grate

Rebar grates were utilized in the Wasatch Project in 1990 and 1991 where deep shafts over 300 m (1,000') deep and a lack of available backfill material available prohibit a more permanent closure (Figures 4 and 5). A design which would require a minimum of site preparation and field fabrication was desired. The rebar grate design developed for the Wasatch Project is constructed from 16 mm (5/8") diameter rebar (#5 rebar) welded on 15 cm (6") centers. A 30 cm x 30 cm (1' x 1') concrete grade beam with two additional 16 mm (5/8") diameter rebar (#5 rebar) placed within the concrete beam was placed around the perimeter of the grate. Field fabrication and adjustments could be accomplished more easily by the contractor using rebar construction. The first rebar grate installation consisted of repair to an existing small grate by extending the rebar over the collapsing collar approximately 3 m (10'). Two of the six rebar grates were accessible only by foot and were pinned by drilling

into the bedrock collars, inserting the #5 rebar and welding the rebar at all intersections of rebar. Two rebar grates were assembled in a ski resort parking lot approximately 1.8 km (1/4 mile) below the sites. The welded grates were then transported to the sites by helicopter along with concrete and water for the perimeter grade beam. The rebar grate design used in the Tintic Project, modified to withstand higher vandalism risk, is constructed from 25 mm (1") diameter rebar (#8 rebar) welded on 20 cm (8") centers. A 30 cm x 30 cm (1' x 1') concrete grade beam reinforced by two additional 19 mm (3/4") diameter rebar was placed around the perimeter of the grate.

The grate sizes within the Wasatch Project range from 1.2 m x 1.2 m (4' x 4') to a maximum size of 6 m x 7 m (18' x 24'). The average cost of $273.22/m^2 ($25.42/ft^2) allows for variations in site access ranging from vehicle access in moderate terrain to foot only access in very steep terrain. The larger sites within the Wasatch project had an average cost of $138.88/m^2 ($12.92/ft^2) which reflects a reduced cost for volume pricing for large grates. The cost of minor site grading and revegetation of disturbed areas are not included in the above costs.

The grate sizes in the Tintic Project, utilizing the larger grate design, range from 3 m x 4 m (9' x 13') to a maximum size of 10 m x 12 m (35' x 40'). The average cost of $72.03/m^2 ($6.68/ft^2) reflects costs for sites that are equipment

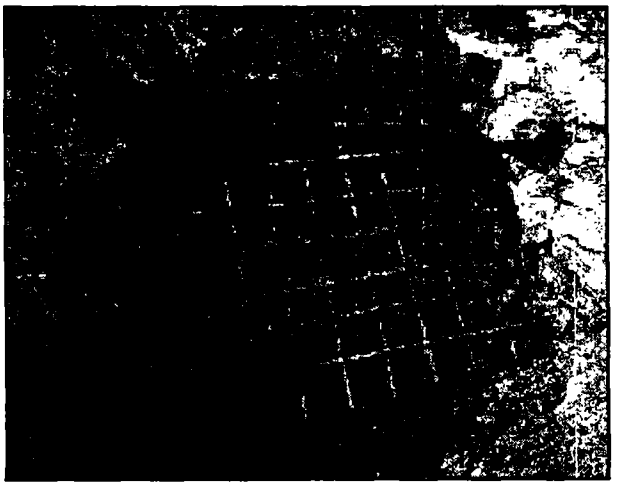

Figure 4. Pinned Rebar Grate closure in Wasatch Project, 1990.

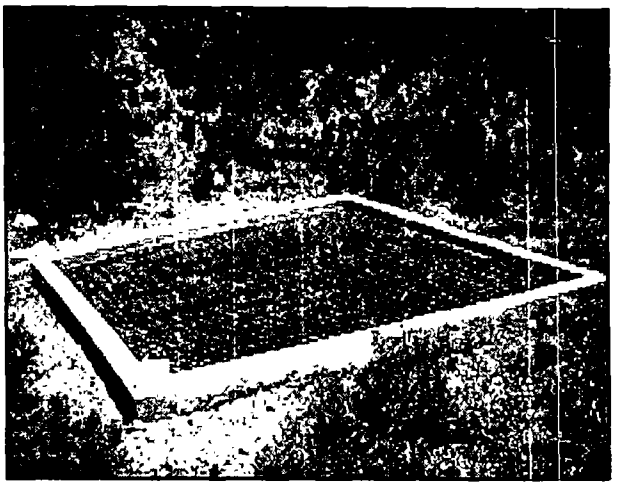

Figure 5. Rebar Grate closure in Tintic Project, 1992.

accessible. One site required transport of all materials by foot giving an average cost of $429.99/m^2 ($40.00/ft^2). The cost of minor site grading and revegetation of disturbed areas is not included in the above costs.

Cable Net

The cable net closure was utilized in the Lathrop Canyon Project in Canyonlands National Park in 1990 where the National Park Service (NPS) requested this type of closure (Figure 6). The cable nets used are fashioned after the design pioneered by the NPS in Death Valley National Monument in California. The design incorporates the use of a single length of preformed 7 x 19 construction (7 strands of 19 wires each) 64 mm (1/4 inch) diameter galvanized aircraft cable to form the grid. A perimeter cable of 79 mm (5/16 inch) of the same type construction is threaded around the grid and through rings secured to the portal wall by 46 cm (18") resin bolts. A lock box was bolted to the rib to allow access into the portals by the NPS. The contractor fabricated the nets at his facility from a jig of his own design. This design allows for small discrepancies in measurements of the portal and allows for irregularities in the ribs to be easily secured. Equipment access is preferred for installation of the nets due to the requirement of drilling the collar for placement of the resin bolts. However, no equipment access was allowed on this project by the NPS resulting in all materials and

equipment transported to the sites by foot.

The net sizes within the Lathrop Project range from 2 m x 2 m (7' x 6') to a maximum size of 6 m x 2 m (19' x 7'). The average cost of $238.88/m² ($22.17²/ft) included transporting all materials to the site by foot. The cost of revegetation of disturbed areas is not included in the above costs.

Steel Door

Steel doors have been used to a minor degree by the Utah AMRP. These doors were installed in the Wasatch Project in the Wasatch Mountains above Salt Lake City in 1990 and 1991 (Figure 7). Steel doors have only been used in non-coal applications. The doors were installed at the request of landowners for access for probable mineral development or access to water supplies. The steel door construction includes the installation of a solid concrete block wall to secure the door to the portal configuration. A 15 cm x 10 cm (6" x 4") angle iron is mounted to the block wall with a steel gate of 38 mm (1-1/2") diameter pipe frame. A 25 mm x 25 mm (1" x 1") sheet of expanded metal was welded at 15 cm (6") intervals over the pipe frame and secured to the angle iron with heavy duty hinges. The purpose of using an expanded metal cover rather than a solid metal closure was to allow for ventilation of the adit. A lock box installed on the gate allows for access to the adit.

Figure 6. Cable Net closure in Laythrop Canyon Project, 1990.

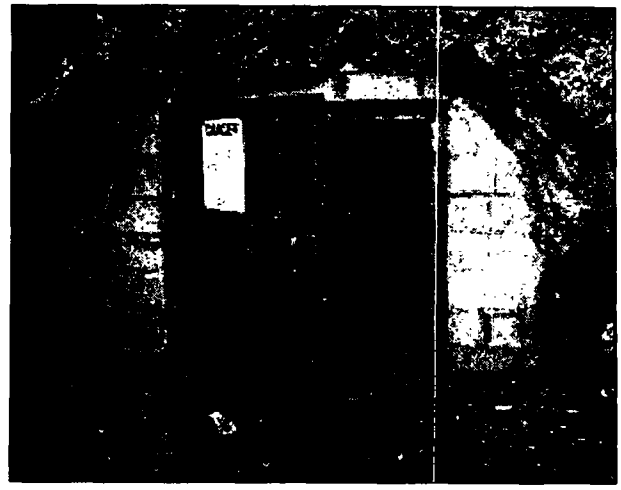

Figure 7. Steel Door closure in Wasatch Project, 1990.

The steel door sizes used in the Wasatch Project range from one small door 1.2 m 1.2 m (4' x 4') installed with foot access over an inclined shaft to larger doors 2 m x 2 m (6' x 6'). The cost of the small door with foot access was $655.07/m² ($60.94/ft²) and is high due to the single site located in steep terrain. The cost of the larger door with equipment access was lower at $289.65/m² ($26.95/ft²) due to vehicle access and a larger size. The cost of revegetation of disturbed areas is not included in the above costs.

Angle Iron Steel Bat Grate

Angle iron steel bat grates were used in the Ferron North Project in 1992, the Oyler Project in 1993, and the Wasatch Project in 1994 (Figure 8). The angle iron steel bat grate design used by the Utah AMRP is a combination of designs pioneered by Roy Powers of Mountain Empire Community College, Virginia, and is presently being used by eastern and western states. Modifications of early designs included hanging the angle iron in front of vertical supports rather than butting the angle iron into the vertical supports. The bat grates are constructed of 12 mm x 12 mm x 79 mm (4" x 4" x 5/16") angle

iron with two 38 mm x 38 mm x 64 mm (1-1/2" x 1-1/2" x 1/4") angle iron stiffeners welded inside of the horizontal bars. The horizontal bars are either welded or bolted with 12 cm (½") carriage bolts to 10 cm x 10 cm x 79 mm (4" x 4" x 5/16") square steel tubing posts. The steel posts are anchored to the roof of the adit by 16 mm (5/8") resin roof bolts placed a minimum of 20 cm (8"). The horizontal bars and base of the posts are anchored to the ribs and floor of the adit by concrete grout. The design of bolting the bars to the post simplifies field installation and allows one or two of the bars to be secured with lock boxes to allow entry into the adit. The 15 cm (6") spacing between bars allows bats to enter and exit freely while restricting public access.

The angle iron steel bat grate sizes in the Ferron North Project range from 2 m x 2 m (6' x 6') to a maximum of 4 m x 3 m (13' x 8'). The cost of $831.60/m² ($77.34/ft²) is high due to helicopter access required to deliver the materials to the sites. The sites in the Oyler Project located within Capitol Reef National Park range in size from 2 m x 2 m (6' x 8') to a maximum of 3 m x 2 m (9.5' x 8') with an average cost of $481.06/m² ($44.71/ft²). The sites in the Wasatch Project range in size from 1.5 m x 2 m (5' x 7') to a maximum of 2 m x 3 m (7' x 9') with an average cost of $844.66/m² ($78.50/ft²). The cost of revegetation of disturbed areas are not included in the above costs.

Figure 8. Angle Iron Steel Bat Grate closure in Wasatch Project, 1994.

During the late summer of 1994, one of the angle iron steel bat grate installed in the Wasatch Project was vandalized by cutting of the horizontal angle iron steel support with a hacksaw. The site located in Big Cottonwood Canyon to the east of Salt Lake City, was repaired by hard-facing all angle points exposed on the outby side of the bat grate. In the summer of 1995, the site was again vandalized by the removal of a portion of the lock-box mechanism on the grate. This vandalism was accomplished by reaching through a 64 mm (¼-inch) space between the lock box and frame with a hacksaw blade. The mild steel used in the angle iron steel bat grate is easily breached by vandals armed with hacksaws and time.

"Jail Bar" Steel Bat Grate

Jail bar steel bat grates were been used in the Summit Project in 1995 (Figure 9). The "jail bar" steel bat grate design used by the Utah AMRP is an attempt to thwart the vandalism of the mild steel angle iron bat grates utilized in previous projects. The bat grates are constructed of 25 mm (1") diameter solid manganese steel bar with two or more 12 mm x 10 cm (½" x 4") manganese steel strap vertical supports. All components are made of 12-14% manganese steel. The vertical supports have 25 mm (1") diameter holes cut on 16 cm (6-1/2") centers with the horizontal bars either electric welded or brazed to the vertical support. The vertical supports are anchored to the roof of the adit by 25 mm (1") diameter 12-14% manganese bars placed a minimum of 20 cm (8") and anchored with resin. The base of the vertical supports are anchored to the floor of the adit by concrete grout. The design and installation of a lock box to the vertical support allows one of the bars to be removed allowing entry into the adit. The manganese steel may be rough cut off-site and cut to fit onsite utilizing a cutting torch. The 14 cm (5-1/2") spacing between bars allows bats to enter and exit freely while restricting public access. Modifications of this design were warranted when a small child (6 years

old) was observed crawling between the lower bars of the closure. The modification consists of reducing the spacing of the bars to 11 cm (4-1/2") on bars located below 1.5 m (60") from the floor of the adit. The spacing of 14 cm (5-1/2") is maintained above the 1.5 m (60") threshold. This allows for one or two spacings of 14 cm (5-1/2") for the average adit. After the 1995 construction season, bats were observed in adits known to have no bat use prior to the installation of the bat grates. The design of the jail bar steel bat grate will be modified to a 10 cm (4") spacing below 1.2 m (48") threshold for all grates after the 1995 season. This change is in response to a change in the national building code standards for constructed gates in public areas.

The jail bar steel bat grate sizes in the Summit Project range from 1 m x 0.6 m (3' x 2') to a maximum of 2.4 m x 1.8 m (8' x 6') with an average cost of $1,582.37/m^2 ($147.06/ft^2). The average cost is increased to $1,884.90/m^2 ($175.18/ft^2) when including a small site (S-84) requiring helicopter access to deliver the materials. The cost of revegetation of disturbed areas is not included in the above costs.

Conclusions

Over the last twelve years the Utah AMRP has used a variety of metal closures with varying degrees of success. The AMRP closure process has been a dynamic evolution as different closure methods are tried and tested.

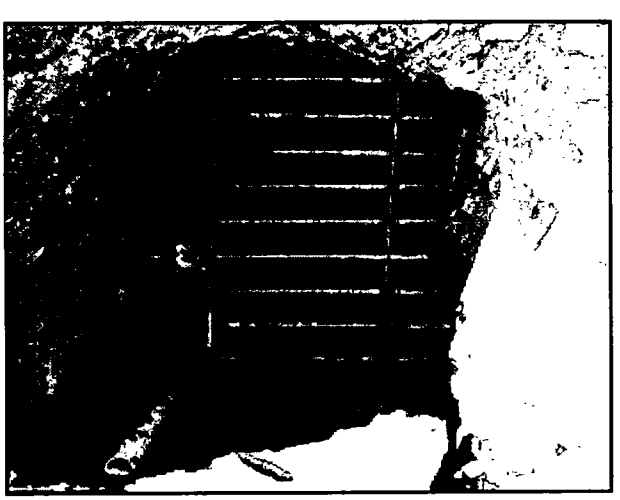

Figure 9. "Jail Bar" Steel Bat Grate closure in Summit Project, 1995.

The A-frame bird cage was the first attempt for the AMRP to find a closure method which would be both cost effective to secure the opening and withstand minor vandalism. With this design, the custom fit, offsite fabrication, difficult installation, and transport to high elevations drove the cost up. The cage is susceptible to failure under snow loads, collar failure, and vandalism.

The large cyclone fencing gives protection for minor collar failure while allowing ventilation, visibility for shaft inspection, and snow loading. The fencing does require moderate site preparation, offsite fabrication and transport to the site. A specialized jig is required to turn the bar stock for fabrication of the fencing design. To date vandalism of the fencing has not been a problem. For large sites with unstable collar configurations the cyclone fencing closure would be the best fit to provide maximum security.

The bar grate provides site ventilation, security, and high loading capacity. Equipment, offsite fabrication, large volume of concrete reinforcing of the collar, and a high amount of site preparation for the single use by the AMRP gave the bar closure an inflated cost for the project. The bar grate provides a secure closure with a high visual compatibility to the historic headframe above the shaft.

The rebar grate gives a secure closure while allowing ventilation, high loading, and visual inspection of the shaft. Minimal site preparation is required allowing the grate to be placed on slopes and uneven collar configurations. The grate is constructed of widely available rebar with onsite fabrication and modifications easily made during assembly without compromising the integrity of the structure. Equipment access to the site reduces the cost significantly due to the volume of rebar and concrete required for the installation. A relatively competent collar configuration is required to anchor the structure while the configuration and flexibility of the grate would allow for minor collar failure. For large, deep shafts with equipment access the grid is a very cost effective way to achieve security. Though configured differently than bat grates, there is anecdotal evidence suggesting bat

utilization of this design in the Tintic Project area in the spring of 1993.

The cable nets offer security while allowing ventilation of adits with a minimum of site preparation. Equipment access to the site significantly reduces costs and the cable net's light weight and capability to be rolled increases the ease of transport. The nets are less durable than the other closures requiring some maintenance to guarantee the security of the closure. The lock is still the weak link in the system with the closure only as secure as the lock. Offsite fabrication, difficult field alterations, and possible vandalism lower the effectiveness of this closure method. The National Park Service applications of this closure technique provide low visibility and minimal impact to the surrounding area. Where the closure can be monitored, this method works well.

Steel doors offer moderate security while allowing controlled access to workings. The steel doors constructed of expanded steel are susceptible to vandalism. The doors must be sized for the opening and then secured to the walls and roof of the mine with block and grout. As with the cable net closure, the lock placed on the door is the weak link in the system. Equipment access to the site significantly reduces the cost of the closure method. In Utah, the doors have only been installed at the request of landowners. This method works well and is cost effective in locations where the doors can be monitored.

The angle iron steel bat grate design offers ventilation of the working along with controlled entry. The bars are fabricated offsite but can be transported in manageable pieces and assembled onsite. Some designs require detailed cutting and welding for installation. This design has minimal effect on the ventilation and a maximum flyway for bats. The grate also gives a secure closure with a higher cost than other methods with equipment access. This method may be acceptable in locations of bat sightings where infants or small children would not be present. In Utah, footprints and other indications of visitation by small children are found in even the most remote locations.

The "jail bar" steel bat grate design also offers ventilation of the working along with controlled entry and vandalism resistance. The bars may be cut offsite with final fit onsite. This allows for the transport of the grate in manageable pieces to be assembled onsite. This design allows for welding by electric or gas with detailed cutting and welding onsite. The grate has a minimal effect on the ventilation and a maximum flyway for bats. The grate also gives a secure closure with a lower cost than angle iron steel bat grates. This method is the preferred closure in locations of bat sightings where infants or small children may be present.

Table 2 summarizes the closures by type, cost, advantages, and disadvantages

Of the eight fabricated metal closures used by the AMRP, the most successful in terms of cost, effectiveness and closure security have been the jail bar steel bat grates.

References

Fjeldsted, B.L. and Hachman, F. C. 1991. Results of the 1990-1991 Utah Skier Survey. Utah Economic and Business Review, Vol. 51, 16 P, August/ September 1991.

Table 2 - Fabricated Metal Closures by Type

TYPE OF CLOSURE	COST $/ft^2$	COST $/m^2$	ADVANTAGES	DISADVANTAGES
A-Frame "Bird Cage"	$46.63	$483.76	Ventilation, possible re-entry	Custom fit, easily vandalized, low snow loads, offsite fabrication, no protection if collar fails
Large Cyclone Fencing	$13.81	$148.01	Ventilation, secures shaft, high loads, bar stock widely available, shaft visible for inspection, collar failure not a problem	Equipment required, offsite fabrication, specialized jig required, site preparation
Rebar Grates	$ 6.68[10] $40.00[11]	$ 72.03[10] $429.99[11]	Ventilation, secures shaft, high loads, minimal site preparation, fabrication onsite, rebar available and inexpensive, field alterations easy	Equipment required, cut with hacksaw
Bar Grates	$247.69	$2,675.37	Ventilation, secures shaft, high loads	Equipment required, offsite fabrication, site preparation
Cable Nets	$22.17	$238.88	Ventilation, secures shaft/adit, minimal site preparation, light weight, transport ease	Custom fit, offsite fabrication, less durable than others, field alterations difficult
Steel Doors	$26.95[10] $60.94[11]	$289.65[10] $655.07[11]	Ventilation, controlled entry, minimal wildlife use	Offsite fabrication, vandalism
Angle Iron Steel Bat Grates	$44.71	$481.06	Ventilation, controlled entry, stock materials, allows bat and other wildlife use, onsite assembly	Offsite fabrication, equipment required, some detailed cutting and welding, cut with hacksaw

"Jail Bar" Steel Bat Grates	$147.06[12] $175.18[13]	$1,582.37[12] $1,884.90[13]	Ventilation, controlled entry, stock materials, allows bat and other wildlife use, onsite assembly	Limited availability of material, gas and/or electric welder required, some detailed cutting and welding

[10] Average site cost for equipment accessible.
[11] Average site cost for single site or foot access to site only.
[12] Average site cost for closure.
[13] Average site cost increased by helicopter access to remote site (S-84).