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The objective of the study was to examine the apparent influence of soil and near surface rock strata on the severity of damage experienced by surface structures subjected to mine subsidence. The report discusses (1) modes of ground movement that might complicate or be confused with mine subsidence, (2) prevailing concepts concerning mine subsidence in the Northern Appalachian Coalfield and Illinois Basin, (3) apparent influences of soil and near surface rock strata on the character of subsidence-related ground movements, (4) classifications and criteria for characterizing damage due to ground movements, and (5) types of subsidence damage experienced by homes above active mines in western Pennsylvania. Presented is a damage classification system incorporating a severity index for characterizing subsidence damage to homes with basements above active mines. A series of case histories illustrate subsidence damage to structures above active and abandoned mines.

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FOREWORD

This report was prepared by GAI Consultants, Inc., Monroeville, Pennsylvania under USBM Contract number J0295014. The contract was initiated under the Mining Environmental Research Program. It was administered under the technical direction of the Twin Cities Mining Research Center with Richard Thill acting as Technical Project Officer. Ronald Simonich was the contract administrator for the Bureau of mines. This report is a summary of the work recently completed as a part of this contract during the period September 1979 to June 1983. This report was submitted by the authors on June 1983.

The writers gratefully acknowledge the assistance of their consultant, Mr. Kenneth Wardell of Newcastle, Staffordshire, England, who through the course of the study furnished invaluable background and insights on the practice of subsidence engineering in the United Kingdom and Europe. The writers also gratefully acknowledge the assistance of Professor James Geddes of the Department of Civil Engineering, University of Wales Institute of Science and Technology, Cardiff, Wales, who shared his views concerning the soil/structure interaction phenomenon in relation to mine subsidence and graciously arranged meetings with many persons in the United Kingdom--academicians, government personnel, engineering consultants and developers--who have made significant contributions to the practice of subsidence engineering or related fields. These include Dr. Peter Attewell, University of Durham; Mr. Charles Barker, Peterlee Development Corporation, Peterlee; Mr. Kenneth Cole, Ove Arup and Partners, London; Mr. Richard Driscoll, Building Research Station, Garston, Watford; Dr. Ian Farmer, University of Newcastle-upon-Tyne; Dr. David Henkel, Ove Arup and Partners, London; Dr. C. F. P. Jones, West Yorkshire Metropolitan County Council; Mr. Charles Shadbolt, National Coal Board, Nottinghamshire, Mr. Sam Thorburn, Glasgow; Mr. David Turner, Peterlee Development Corporation, Peterlee; Mr. Thomas Walsh, Warrington New Town Development Corporation, Warrington; Professor Barry Whittaker, University of Nottingham; and Mr. Peter Wild, Warrington New Town Development Corporation.

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The writers accept sole responsibility for the analyses, views and conclusions presented in the report.

EXECUTIVE SUMMARY

The objective of the study is to examine the apparent influence of soil and near surface rock strata on the severity of damage experienced by surface structures subjected to mine subsidence, particularly above active mines. The report discusses: 1) modes of ground movement that might complicate or be confused with mine subsidence; 2) prevailing concepts concerning mine subsidence in the Northern Appalachian Coal Field and Illinois Basin; 3) apparent influences of soil and near surface rock strata on the character of subsidence-related ground movements; 4) classifications and criteria for characterizing damage due to ground movements; and 5) types of subsidence damage experienced by homes above active mines in western Pennsylvania.

Two hundred eighty two incidents of subsidence damage to homes above active mines in the western Pennsylvania area have been reviewed through the cooperation of the Pennsylvania Office of Mine Subsidence Regulation along with numerous other subsidence cases above active and abandoned mines identified through other sources. Damage to 134 homes above active mines in western Pennsylvania was tabulated, and the levels of damage of 50 representative cases were evaluated in terms of a newly developed Severity Index. Utilizing this Severity Index as a basis, it was possible to estimate the relative contributions of particular structure, site, geologic and mining factors in determining the severity of damage.

A damage classification system is presented that incorporates the Severity Index for characterizing subsidence damage to homes with basements above active mines. Case histories from Pennsylvania and Illinois are presented that illustrate particular soil and near surface rock effects above both active and abandoned mines.

What is ordinarily termed "normal" or "ordinary" settlement encompasses a wide variety of ground movements that includes consolidation settlement, heave, erosion, piping and many others. Movements can originate from so many individual sources that identification of the responsible source may be difficult. Moreover, more than one type of ground movement may be acting at a given location in response to natural factors or to activities by man. Although each type of ground movement is controlled by certain unique factors, the outward nature of the movements in the absence of a detailed history may be insufficient to identify their source. Virtually all types of ground movement include some component of rigid body translation, rigid body rotation, distortion or volume change. The types of movement most commonly associated with mine subsidence can equally well be associated with at least six other types of ground movement. The identification of subsurface material types can sometimes suggest specific sources of ground movement.

Ordinary ground movements vary widely in magnitude from fractions of an inch to tens of feet and in rate from inches per

year to inches per hour. The severity of damage to surface structures may range from slight to relatively severe for many types of ground movement, although certainly not all could cause total destruction of surface structures. Of the fourteen common types of ground movement, the magnitude of movement can be predicted in only six, and, even then, the prediction is rarely better than ±25 percent.

The art of prediction of ground movements is rather primitive, not simply because of a lack of analytical techniques, but because of deficiencies in our ability to accurately define subsurface soil, rock and groundwater conditions, the distribution, duration and intensity of stress changes, etc. Highly accurate predictions will probably never be possible.

Subsidence above active mines is distinctly different from that over abandoned mines in terms of geometry, time and place of occurrence and, to a certain extent, mechanism, yet the manner in which either type of subsidence occurs is much the same regardless of geographic location -- whether in Appalachia or in Illinois. Geologic differences exist between the Appalachian and Illinois regions, but available subsidence data are not sufficiently refined or extensive to identify reliable relationships between subsidence parameters and basic mine site stratigraphy. The deficiencies in available data make impossible a separate identification of the influence of soil and near surface rock on subsidence parameters.

Very few measurements exist in the United States of ground strain, slope and curvature that are of use in relating mining to severity of damage to structures at ground surface.

A review of case histories of mine subsidence makes clear, however, that soil and near surface rock can influence the character of subsidence through landslides and slippage, block-wise movements of rock strata, reactivation of faults, buckling of strata, loss of soil into open joints, etc. Many of the same phenomena reported in the British and European literature are recognized in the United States. As is typical of a great many forms of ground movement, these near surface phenomena tend to be unpredictable in both magnitude and occurrence. To some degree they are to be expected at every subsidence site.

Even without the anomalous behavior imposed by near surface effects, the soil-structure interaction problem is a difficult one. For this reason, generalized empirical relationships are customarily employed to specify allowable limits of ground movement before the onset of damage to a structure. However, the most common types of damage criteria--those pertaining to self-weight settlements--are generally not applicable to subsidence situations, which commonly involve a significant horizontal component of ground movement.

Establishment of sound, simple, useable criteria encompassing the broad spectrum of damages associated with mine subsidence will require that damage and repair requirements be catalogued at

individual subsidence sites, and that during the subsidence process, building deformation parameters, such as angular distortion, be measured along with ground strain, tilt and radius of curvature.

In the present program, some progress has been made in this direction by reviewing the types of subsidence damage sustained by a series of 134 typical homes in western Pennsylvania located 157 to 650 feet above active mines in which a variety of mining types have been utilized. Damage was evaluated relative to location within the home and to severity. Basements were always found to be damaged, the other home components being damaged in the decreasing order: interior, exterior and roof. Very few data were available to quantify ground movements and structure movements in relation to observed damage. Damage appeared to be due to a combination of hogging, sagging, extension and compression, rather than simply extension and compression. It is clear that U.S. homes, a large proportion of which have basements and include superstructures with only limited mechanical connection to the basement, respond differently to subsidence than do buildings of slab-on-grade construction, which have been the object of study in the United Kingdom.

Inasmuch as (1) basements are virtually always damaged; (2) their repair costs characteristically represent the largest proportion of damage repair costs; and (3) they are rather uniformly constructed from home to home, basement damage has been chosen as the basis for a newly developed subsidence damage classification system applicable to homes in the United States, particularly western Pennsylvania. The Severity Index, a continuous function ranging from 0.0 to 5.0, is a quantitative gage of the severity of damage to a home. The range from 0.0 to 1.0 corresponds to slight damage, where repairs require little more than repointing the basement walls. The range from 1.0 to 2.0 corresponds to moderate damage, where repairs may include some relatively small amount of wall reconstruction. The range from 2.0 to 4.0 corresponds to severe damage, where extensive basement repairs are required, including major wall reconstruction and floor slab replacement. The range 4.0 to 5.0 corresponds to very severe damage where nearly total reconstruction of the basement is required. Superstructure damage does not always conform to basement damage in a predictable manner due to the framing action of the superstructure and the limited mechanical connection between the superstructure and basement. Nonetheless, the more severe cases of superstructure damage are typically associated with relatively severe damage to the basement. In strict terms, the Severity Index is a measure of damage to the basement, but because basement damage dominates the cost of repair, the Severity Index in effect reflects the degree of damage to the entire home. Recognizing that the Severity Index represents a uniform, consistent measure of damage that can be compared from site to site, a predictive model of the Severity Index was developed through a stepwise elimination regression analysis that provided a means for evaluating the relative importance of particular structure, site, geologic and mining factors in influencing the severity of damage to homes. By and large, the

results of the empirical model are as one would expect and are considered reasonably good in light of the limited data available.

Improving case documentation should lead to an even better model for predicting damage severity. Eventual incorporation into the model of ground movement data, including the progression of home damage with development of the subsidence profile, should provide a desired link between damage severity and subsidence that will be of great benefit in predicting damage above active underground coal mines and understanding the processes that control severity. From this knowledge more appropriate building designs might be developed for subsidence-prone areas and more appropriate courses of action might be identified prior to mining to protect existing structures.

Further investigation of the problem of damage prediction should be considered as important, if not more important, than the development of subsidence prediction models. The use in the United States of relationships developed abroad between ground movements and damage severity is to be discouraged.

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1. INTRODUCTION

This report summarizes the work performed for the U.S. Bureau of Mines by GAI Consultants, Inc., under Contract J0295014. The period of the contract was September, 1979, through June, 1983.

Objective of the Study

The purpose of the study is to examine the influence of the soil and near surface rock strata on the severity of damage experienced by surface structures subjected to mine subsidence, with particular reference to the Eastern and Interior Coal Provinces (Figure 1.1).

Background

Damage to man-made structures founded at or below ground surface can result from many causes including natural ground movements, such as those resulting from earthquakes and landslides; man-induced ground movements, such as those resulting from blasting, construction excavation or extraction of minerals from the earth; and alteration of the near surface natural materials, such as might occur under heavy loads from buildings or from changes in chemical, thermal, or hydraulic conditions in the soil and rock comprising the upper layers of the earth's crust. Still other possible causes of building damage not related to ground surface movement include inadequate structural design; alteration of a structure after construction; poor construction (materials and/or workmanship); deterioration due to lack of maintenance; and overloading. It is apparent that assessing the cause of building damage can be a very complex problem, particularly if more than one factor is involved. Fortunately, modern engineering methods and construction technology have reduced the unknowns to a minimum, and, today, a structural collapse, other than a mishap during construction, is a rarity (Johnson, 1965).

Probably the most common cause of structural damage, including both total collapse and repairable distress, is that associated with ground surface movements. Although engineering design and construction technologies can cope with most anticipated loading conditions, ground surface movements, both anticipated and unanticipated, commonly present the greatest uncertainty to structural design. The unforeseen or unanticipated ground movement leads to frustration and anxiety not only in the planning and design stage of new structures, but also in the maintenance and protection of existing structures.

Ground surface subsidence above both active and abandoned coal mines may well be responsible for more serious structural damage than any other form of ground surface movement, excluding earthquakes and landslides. In the 250 years that underground mining of

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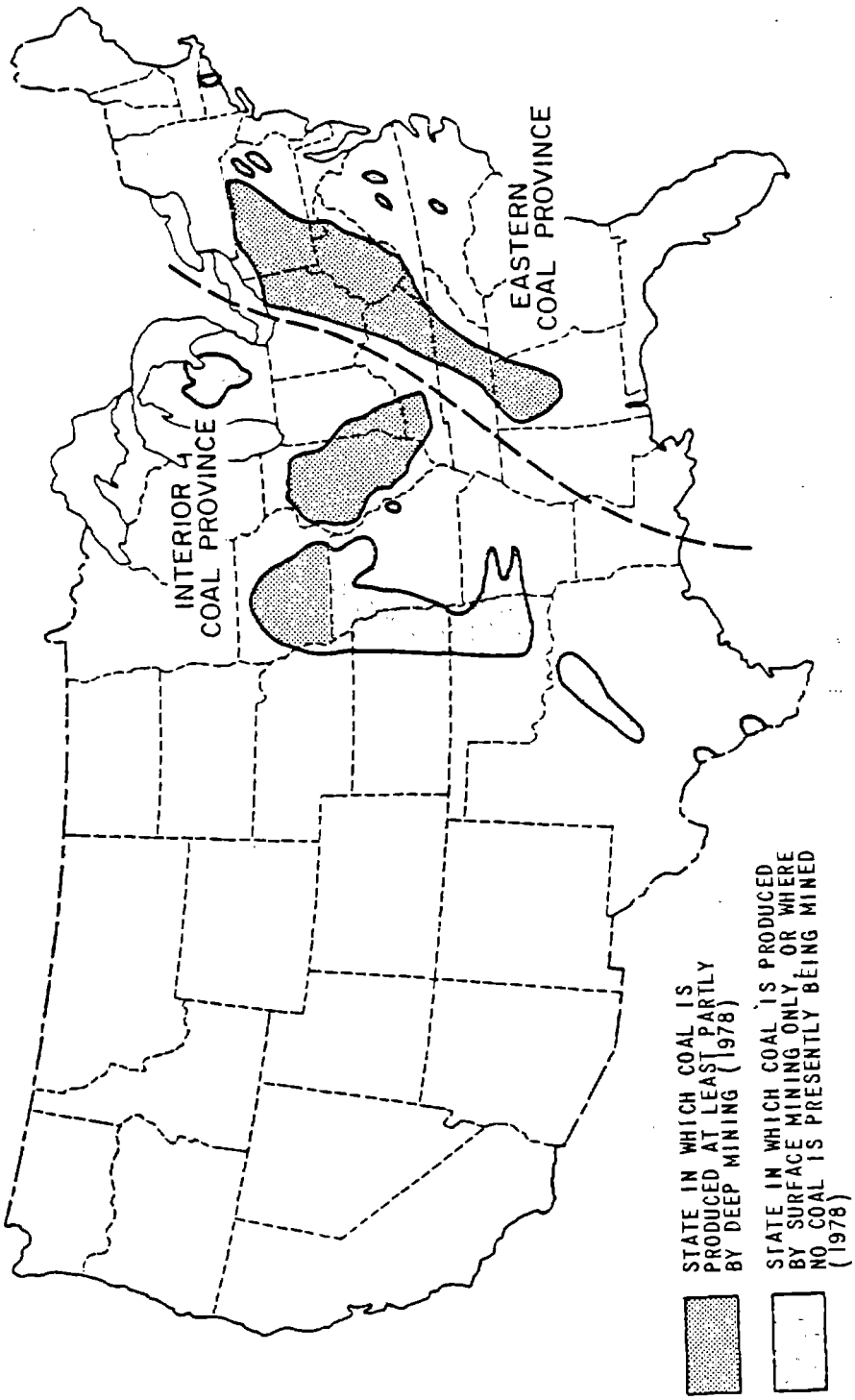


FIGURE 1.1 - EASTERN AND INTERIOR COAL PROVINCES, UNITED STATES

bituminous coal has been practiced in the United States, an estimated 2 million acres of land in 30 states have been affected by subsidence (Johnson and Miller, 1979).

Subsidence of the ground surface has commonly been accompanied by the modification of surface drainage, the modification of the groundwater regime and damage to structures at ground surface. In recent years, more than 30 homes and other structures have been damaged each year by coal mine subsidence in western Pennsylvania--about two-thirds above active mines and the remainder above abandoned mines. Numerous structures have also been reported damaged in Illinois, West Virginia, Iowa and Alabama. Untold numbers of wells have dried up as a result of subsidence-related ground movements, and large expanses of farm land, particularly in Illinois, have required installation of drainage systems to eliminate ponding of water in subsidence depressions.

The high extraction mining methods currently practiced in most states virtually assure significant ground movements during the mining process. While high extraction is consistent with today's energy policy, the federal government has at the same time expressed its commitment to protect the environment. The Federal Surface Mining Control and Reclamation Act of 1977 requires that underground mining be conducted in a manner that will maintain the quality of the environment through measures consistent with known technology to prevent subsidence damage and maintain the value and use of surface lands to an extent that is technologically and economically feasible.

To this end, a great deal of existing information on coal mine subsidence has already been compiled under government sponsorship from field observations and case history studies, and many new data are being collected through current field monitoring studies. However, because of the large number of factors that influence subsidence and the difficulties in quantitatively assessing each factor, attempts at deriving relationships to predict the magnitude of subsidence and deformation of the ground surface have thus far met with only limited success.

Clearly, a major aspect of the problem is to identify the influence of overburden properties on the mechanism of subsidence, since the composition of the rock strata and soil mantle can vary significantly within, and between, coal mining regions. The wealth of soil mechanics and foundation engineering case histories compiled to date show without question that soil types, soil properties, details of soil stratigraphy and groundwater conditions, together with construction details, have an overwhelming influence on the ground surface response in civil engineering projects (Peck, 1969; Attewell and Glossop, 1978). In Stockholm, Sweden, for example, it has been painfully learned that subway tunnels in rock can cause significant building damage when the overburden consists of soft, saturated marine clays. The mechanism responsible for this damage is consolidation of the clay that occurs as a result of water drainage into the tunnels. These experiences suggest that

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the ground surface conditions (soil and near-surface rock layers) have a significant effect on the severity of subsidence-induced damage when material is removed at depth in the earth's crust. In the mining context, the studies of Whittaker and Breeds (1978) in the United Kingdom indicate that ground movements above longwall panels are much more erratic and unpredictable in locations where blocky, jointed Bunter Sandstone lies near the surface than in locations where the more compliant shales or limestones are found.

It is obvious that for practical purposes the study of subsidence must be carried beyond the prediction of ground surface deformations to the more important prediction of the response of structures founded at or below the ground surface. To do so, it becomes necessary to consider the factors that control the transfer of stresses from the ground to the structure, as well as the properties of the structure and the response of the structure to these stresses. The soil-structure interaction problem is among the most complex faced by practicing engineers. That problem associated with subsiding structures, difficult as it is, is complicated by the general inability at the present time to predict ground surface movements in the United States with any degree of reliability. Recent efforts by the National Bureau of Standards to develop guidelines and provisions for the constructon of housing in mine subsidence areas have demonstrated the difficulties in predicting the behavior of the ground surface and of structures located above areas that are already undermined or will be undermined in the future (Yokel, 1977).

Understanding the response of structures to ground movements has the further benefit of permitting mine subsidence damage to structures to be differentiated from damage due to other causes. It is interesting to note that of the 794 claims for subsidence damage to homes above active mines filed in the Commonwealth of Pennsylvania during the period 1966 to mid-1983, somewhat over 50 percent were denied on the basis that damage resulted from a cause other than subsidence (T. Alexander, personal communication).

Formal analyses of structural damage due to subsidence require that the stresses, strains and ground movements associated with subsidence be related to the tolerable or limiting movements of the structure. Significant achievements have been made in establishing damage criteria for various kinds of structures and types of construction (Sowers, 1962; NCB, 1975; Skempton and MacDonald, 1956; Grant, Christian and VanMarcke, 1974; Burland, Broms and DeMello, 1977; Boscardin, 1980).

Because a wide variety of distress and distortion types can occur in a single structure, it is necessary that damage criteria be based on all likely modes of deformation. The most common bases for damage criteria are horizontal strains (extension and compression), vertical differential settlement, total vertical settlement, and angular distortion. Significant data on the subject have been collected by soil mechanics and foundation engineers (Burland, Broms and DeMello, 1977).

Subsidence engineers abroad have also studied building damage extensively. One of the greatest accumulations of knowledge is in the United Kingdom, where measurements and observations of strains and damages have been made for buildings, walls, bridges, highways, and utility lines (NCB, 1975). The knowledge concerning building damage due to coal mine subsidence has been less well documented in the United States.

The emphasis in the present study is on collecting and reviewing subsidence case history data from mining-related sources located primarily in the Eastern and Interior Coal Provinces of the United States and examining the possible effect of ground surface conditions on the response of structures to subsidence through the following avenues:

1. Existing state-of-the-art mine subsidence studies.

2. New mine subsidence case histories.
3. Existing experience and data related to ground surface movements in civil engineering practice.

4. Existing knowledge of building damage due to foundation settlements of structures on soil and rock.

5. Simple model or parametric studies.

Scope of the Study

The project includes the following tasks:

1. Phase I - Background Study

    a. Review of domestic and foreign literature concerned with subsidence and soil/structure interaction and preparation of an annotated bibliography of the principal references (issued under separate cover);

    b. Review of non-mining related ground movements that might be mistaken for mine subsidence;

    c. Evaluation of the quantity and quality of subsidence case documentation in the Eastern and Interior Coal Provinces; and

    d. Recommendation of specific study areas within these provinces to conduct the Phase II Study.

2. Phase II - Data Collection and Analysis

    a. Collection of specific data on documented subsidence cases associated with active and abandoned mines in the Eastern and Interior Coal Provinces, including

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information on geology, hydrology, mining and ground surface conditions;

b. Systematic analysis of these data to identify the effect of soil overburden on subsidence damage; and

c. Formulation of conclusions and recommendations for future research to more clearly identify the effects of near surface materials on the severity of damage to surface structures.

The following pages elaborate on these topics. Section 2 of the report (the next section) reviews the character of non-mining ground movements that may, under certain circumstances, be mistaken for mine subsidence and presents an assessment of how well their magnitudes can be predicted. Section 3 reviews the character of subsidence above active and abandoned coal mines in the Eastern and Interior Coal Provinces in the context of their geologic, topographic and mining settings, while Section 4 discusses observed and suspected influences of soil and near surface rock on ground movements associated with underground mining. Section 5 reviews types of building damage that result from ground movements of various types, as well as existing criteria and damage classifications that relate building damage to distortions associated with differential settlement, ground strains, etc. Section 6 reviews the numbers, types and thoroughness of documentation of subsidence cases above active mines in one specific section of the Eastern Coal Province--western Pennsylvania. It summarizes the types of damage sustained by the structures, presents a tentative damage classification based on the data, and examines the possible influences of factors such as overburden thickness, soil type, and mining type on the severity of damage experienced by the structures. Section 7 presents our conclusions and recommendations for further research. A series of mine subsidence case histories gathered during the study are assembled in Appendix A. Details concerning individual damaged homes, site characteristics, geologic setting and mining types at each of 134 locations of mine subsidence above active mines in western Pennsylvania are presented in Appendices B through D, along with the definitions of variables and functions that have been utilized in regression analyses conducted to identify the factors that govern the severity of damage to homes.

2. GROUND MOVEMENTS OTHER THAN MINE SUBSIDENCE

INTRODUCTION

In 1948, Terzaghi and Peck wrote in their classic Soil Mechanics and Engineering Practice (p. 201):

> "The design of the framework of a building or other structure is, with rare exceptions, based on the assumption that the structure rests on an unyielding base."

Terzaghi and Peck were concerned that, because structures invariably cause compression of the underlying soil, the assumption of an unyielding base is incorrect. Consequently, the structure will be subjected to distortional stresses if the settlement of the soil is not uniform. The term "settlement" is used by Terzaghi and Peck to indicate the sinking of a building due to compression and deformation of the underlying soil.

Although the unyielding base assumption is still probably the most widespread practice for building design, it is becoming increasingly apparent that building designers should consider the possibility that their structures may be subjected not only to the movements caused by soil compression and deformation, but also to those caused by other types of ground movement. The evidence is two-fold:

1. A great many buildings in different parts of the world have been damaged by ground movements quite different from simple soil compression and deformation.

2. A large percentage of buildings to be constructed in the future will be located in areas where it is known that ground surface movements totally unrelated to the presence of any buildings are either taking place or probably will take place in the future.

While analysis of soil settlement due to building and other surface loadings is sometimes sufficient for structural design, designers today should be aware of a much broader spectrum of possible ground movements to which their structures may be subjected. Knowledge of these movements should also be possessed by anyone who is given the task of assessing and evaluating damage to structures.

The nature of ground movements, their possible origins, their analysis and prediction, the evaluation of building damage caused by ground movements, and the relationships between ground movements and building damage are the topics of this section of the report. Emphasis is placed on sources of ground movement other than subsidence due to coal and other solid mineral extraction. That subject is dealt with in Section 3.

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Throughout the following discussion, an attempt is made to point out similarities between ground movements due to mining subsidence and those due to other causes. The underlying thesis is that ground movements cannot be understood, identified or treated within the narrow confines of any one technical discipline.

Classification and Examples of Ground Movement

Any ground movement, regardless of its cause or magnitude, can be reduced to one or more of the four basic modes common to solid mechanics (Timoshenko and Goodier, 1970):

1. Translation--simple uniform rigid body displacement, such as might be associated with sliding on a plane surface.

2. Rotation--angular rigid body displacement, such as might be associated with sliding on a curved surface or tilting.

3. Distortion--change of shape brought about by shearing forces.

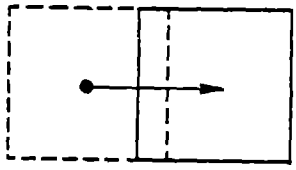
4. Volumetric Change --an increase or decrease in density such as might be associated with shrinking or swelling.

These fundamental modes of movement are illustrated schematically in Figures 2.1-2.4. Simple examples of how each mode of movement can affect a building located at the ground surface are shown together with an illustration of the movement of an element of ground. It is important to recognize that the movements shown in these examples refer to ground movements and not to building or foundation movements, which are described in somewhat greater detail in Section 5 on General Damage Evaluation and Damage Criteria. In Figures 2.1 and 2.2, the movement modes are basically those of a rigid block or mass of soil, whereas those in Figures 2.3 and 2.4 involve a change in volume or shape. As indicated by Figure 2.4, volumetric changes can be accompanied by a distortion of the ground.

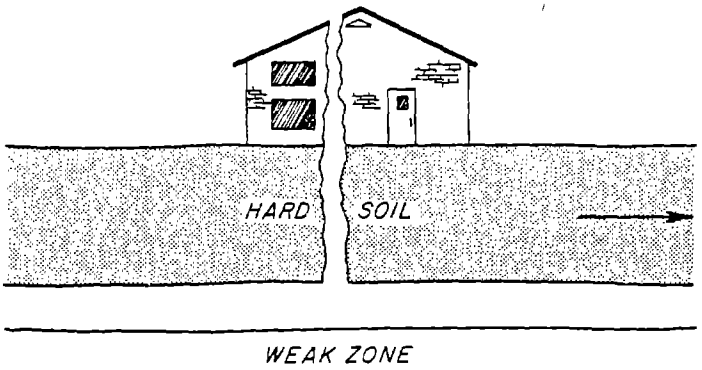
Discussion. Several observations can be drawn from Figures 2.1-2.4:

1. Some ground movements, notably distortional movements including those associated with volumetric changes, may result in a change in profile of the ground surface in which the continuity of the ground surface is preserved. Other movements, notably translational and rotational movements but also certain volumetric and distortional movements may cause discontinuities in the ground surface, such as scarps, tension cracks, etc. The recognition of such discontinuities is important, as it may point to the likely source of ground movement.

A.

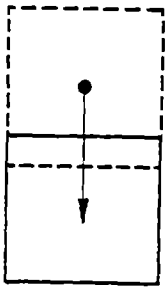


HORIZONTAL DISPLACEMENT

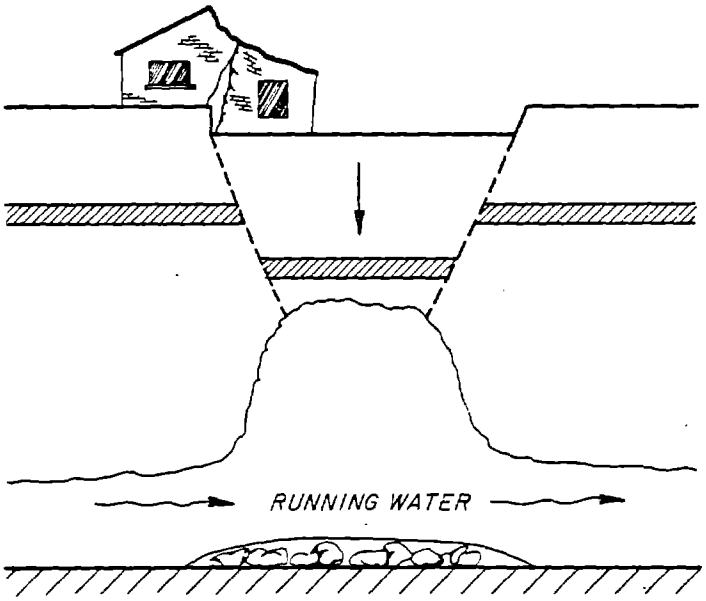


Source of Movement:
Landslide Caused by Earthquake

B.

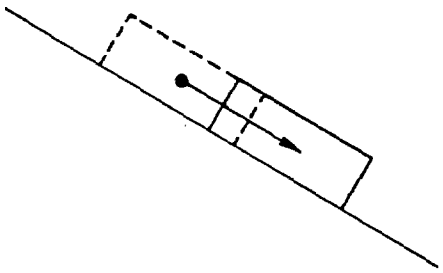


VERTICAL DISPLACEMENT

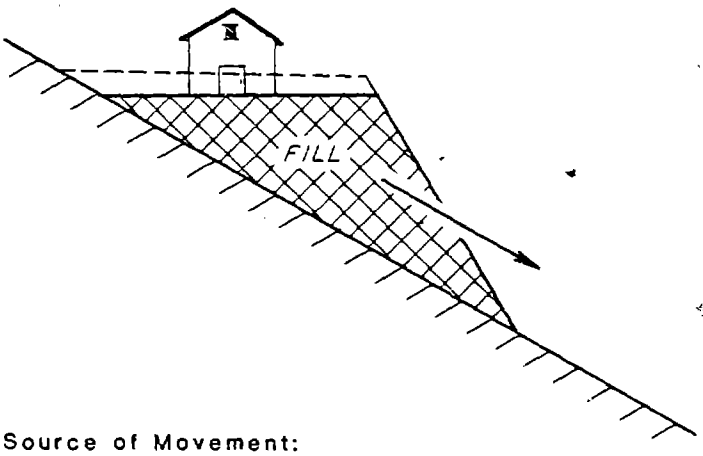


Source of Movement:
Subsurface Erosion and Sinkhole Collapse

C.



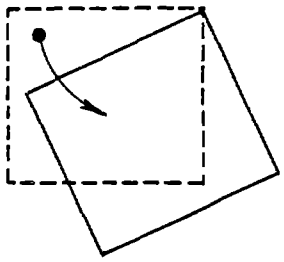
INCLINED DISPLACEMENT



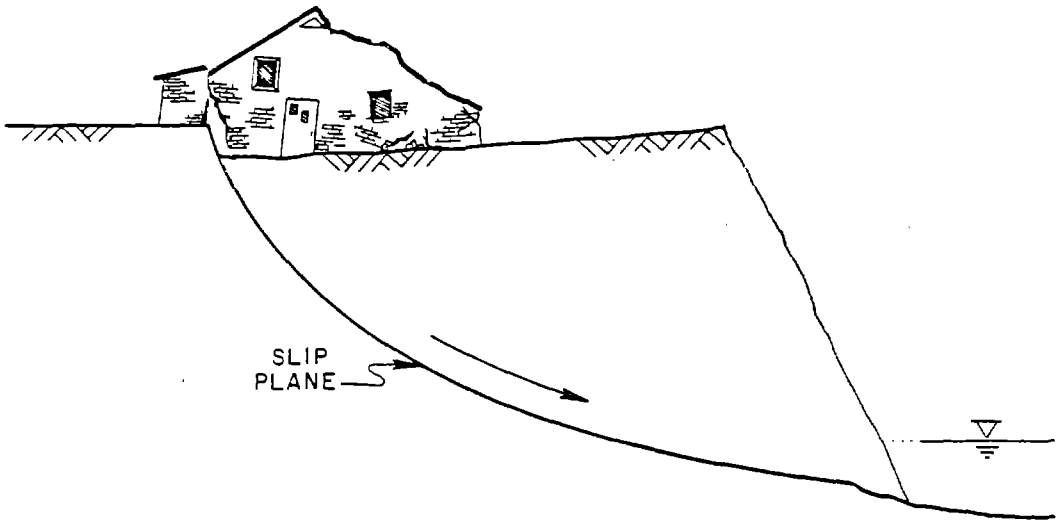
Source of Movement:
Landslide Caused by Fill Placement

FIGURE 2.1 - TRANSLATIONAL GROUND MOVEMENTS

A.

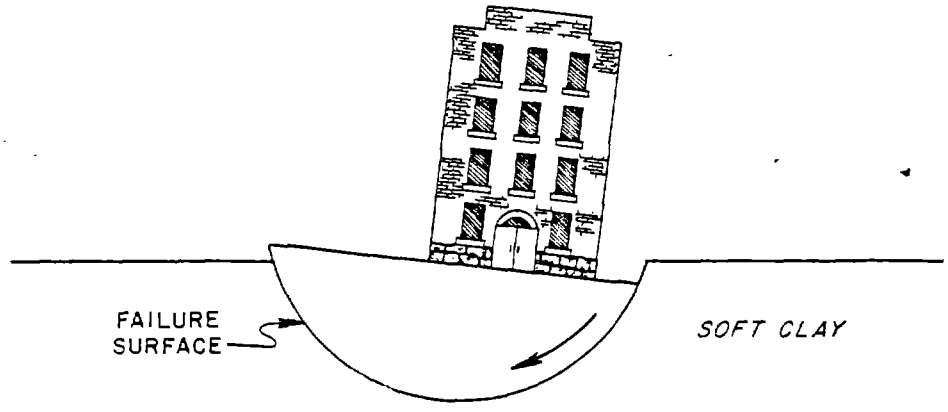


B.



Source of Movement: Landslide

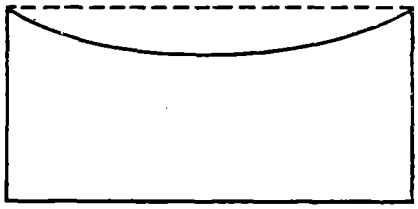
C.



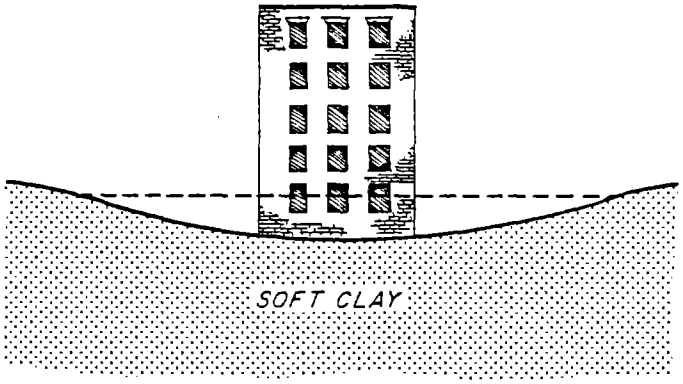
Source of Movement: Bearing Capacity Failure

FIGURE 2.2 - ROTATIONAL GROUND MOVEMENTS

A.



VOLUME CHANGE AND
SHEARING DISTORTION

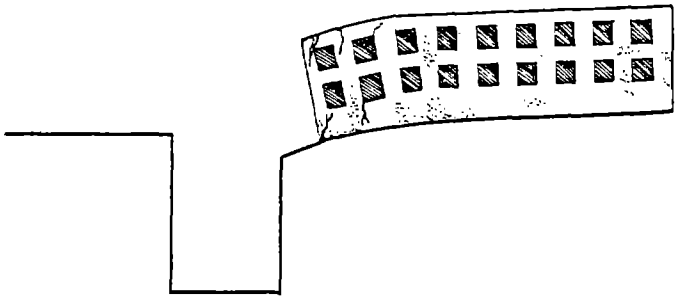


Source of Movement:
Settlement Under Weight of Structure

B.



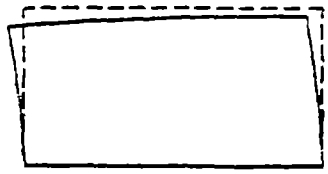
SHEARING DISTORTION WITH
VIRTUALLY NO VOLUME CHANGE



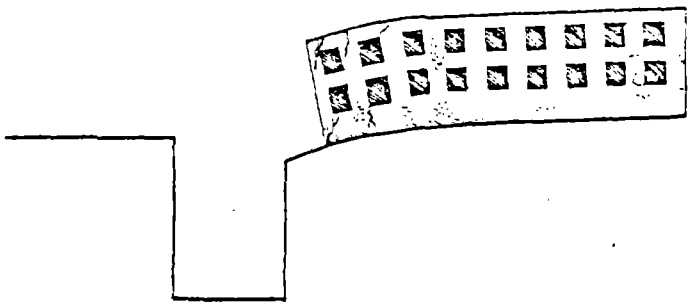
Source of Movement:
Yielding of Soil into an Open Excavation

FIGURE 2.3 - DISTORTIONAL GROUND MOVEMENTS

B.



SHEARING DISTORTION WITH
VIRTUALLY NO VOLUME CHANGE



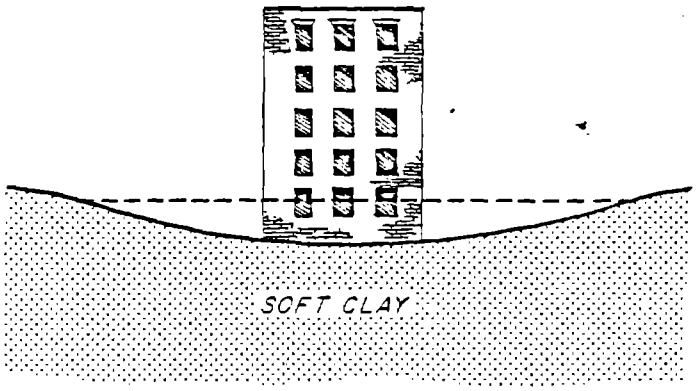
Source of Movement:
Yielding of Soil into an Open Excavation

FIGURE 2.3 - DISTORTIONAL GROUND MOVEMENTS
WITH VIRTUALLY NO VOLUME CHANGE

A.



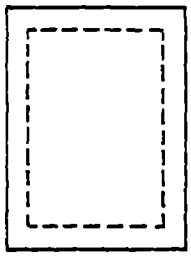
VOLUME CHANGE AND
SHEARING DISTORTION



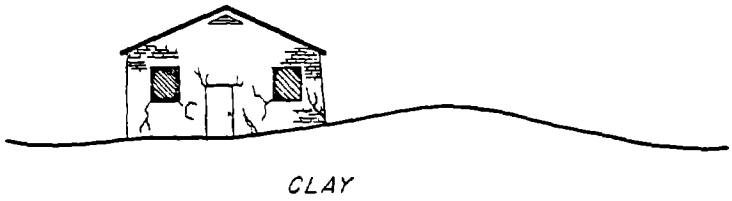
Source of Movement:
Settlement Under Weight of Structure

FIGURE 2.4 - VOLUME CHANGE GROUND MOVEMENTS
ACCOMPANIED BY DISTORTION

B.

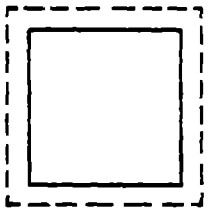


SWELLING

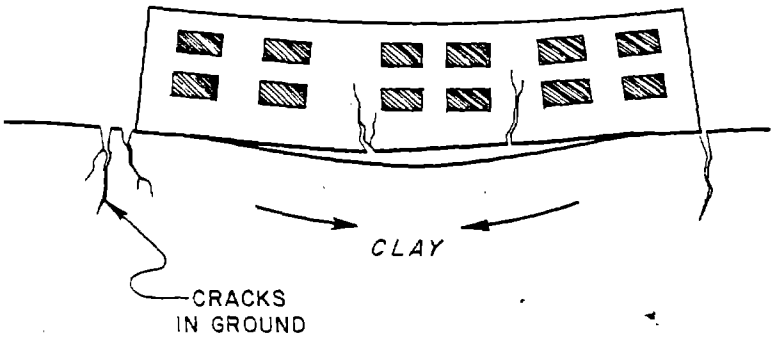


Source of Movement:
Swelling Caused by Wetting of Soil

C.



SHRINKAGE



Source of Movement:
Shrinkage of Clay Under Center of Building

FIGURE 2.4 - VOLUME CHANGE GROUND MOVEMENTS ACCOMPANIED BY DISTORTION

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2. Any one mode of ground movement has the potential to cause building distress.

3. The movements to which a structure resting on moving ground is subjected are not necessarily the same as those of the ground. It is obvious in Figure 2.1B, for example, that distortion of the building results from a vertical translation of the ground surface. In Figure 2.4A, the structure is shown to undergo only a vertical translation while the underlying soil experiences a volume change and distortional movement. Thus the scale on which ground movements take place relative to the scale of buildings founded on the ground surface is a critical factor in determining potential structural damage.

4. As a corollary to item (3), it follows that a structure would not necessarily undergo distress simply because it is resting on a moving soil mass. In Figures 2.1C and 2.4A, for example, uniform movements might cause no damage other than the rupture of utility lines leading into the structures.

5. In some cases, such as Figures 2.1A, 2.1B and 2.2B, the ground movements may not be related to the presence of a structure; that is, the movements might occur even if a structure were not present. In other cases, such as Figures 2.2C and 2.4A, the movements are a direct result of the structure's presence.

Application. The classification of ground movement modes according to the preceding scheme is somewhat arbitrary and simplified. In reality, ground movements are ordinarily a combination of two or more types in which one is more prominent than the others. The purpose in classifying ground movements is:

1. To provide a method for describing ground movement morphologies that is consistent with the underlying physical mechanisms.

2. To provide guidance for identifying sources of ground movements.

3. To indicate the wide variety of associated mechanisms that are driven by ground material properties, ground water conditions, geologic processes, and human influences.

Determination of the mode and origin of ground movement in an actual case may be possible by simple visual inspection. More generally it requires subsurface exploration, ground surface instrument surveys (time-movement history) and possibly in-ground instrumentation to measure soil movements. Sometimes test trenches excavated to shallow depths (under 15 feet) will yield critical

information. Because all of these methods are expensive and require time, it is important to understand the fundamental character of the different ground movements. This is the subject of the following discussion.

SOURCES OF GROUND MOVEMENTS AND PREDICTION OF THEIR MAGNITUDES

General Considerations. In the most basic sense, the source of any ground movement can be traced to a change in the stress field within the ground or to a change in material properties. The response of the ground to either change results in strains, which in turn result in movement.

The prediction or estimation of ground movements is required for two important engineering purposes:

1. As a preliminary aid in the design of buildings that will be subjected to movement, and

2. As an aid in the analysis and treatment of buildings that have been damaged by ground movements.

Any attempt to analyze or predict ground movements requires an understanding of the cause of the change in stress field or material properties. Such an understanding is also necessary to differentiate subsidence from other sources of ground movement.

In the present discussion, potential sources of ground movements in the Eastern and Interior Coal Provinces are reviewed. Mechanisms of movement are discussed together with brief descriptions of their origins. Where available, actual cases of observed movements and damages are cited. Capsule summaries of methods for predicting the magnitude of ground movement are reviewed. All of the movements discussed are permanent. Transient or dynamic ground motions, such as those that occur during earthquakes, are not considered.

The classification of ground movements described in this review is based on the mechanical, physical or chemical processes that cause the movement. The underlying causes are sometimes apparent from the terminology, as in the case of shrinking and swelling soils. In other cases, such as landslides, the underlying causes may lie in a multiplicity of factors. As will become apparent, the state-of-the-art for estimating the magnitude of movement is quite primitive in most cases, a condition which is due not so much to lagging technical developments as it is to an inability to determine quantitatively the factors that are known to govern the magnitude of ground movement. Statistical approaches to the characterization of soil properties and processes are receiving increasing space in the literature (Harr, 1977), but owing to cost of implementation, are only rarely used in practice at this time.

Natural versus Human-Related Ground Movements. Although many of them never affect the activities of humans, natural ground

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movements caused by geologic and other natural processes are probably far more common than those caused by human activities. Geologic and other natural processes are constantly at work, and they result in ground movements that frequently make human-related ground movements appear rather insignificant, both in magnitude and in abundance. Ignorance of these processes has resulted in the imprudent expenditure of millions of dollars in building costs as well as loss of human life.

Because of their potential impact on man-made structures and because ground movements from non-mining causes can sometimes be confused with mining-induced subsidence, a knowledge of these ground movements is important. They are but one of the many so-called "geological hazards," a recently coined term that is applied to earthquakes, floods, avalanches, tsunamis and other processes that disrupt human life (Bolt, 1975). As made clear by Legget (1973), some of these hazards are totally natural in origin; others are almost entirely man-made, assisted by some natural process such as a heavy rainfall. Intermediate are those that are predominantly natural, assisted by some human activity such as excavation or alteration of drainage patterns. This spectrum of causes applies well to ground movements, as will be seen in the following pages.

Leonards (1982) has reviewed a series of case histories where unexpected ground movements have resulted in failures of engineered structures. Allen (1969) has presented a most noteworthy review of different geologic settings of subsidence. Whereas she concentrated on purely natural or geologic processes leading to downward vertical ground movements, the present work treats all forms of ground movement that are likely to occur in the coal mining areas of the central and eastern United States, due to any cause, geologic or human.

LANDSLIDES AND RELATED MASS MOVEMENTS

General Considerations. Ground movements associated with landslides, earth flows, hillside creep and other movements of soil and rock masses are widespread throughout the world. The potential for mass movement cannot be ignored in any area where the land surface has relief, or where excavations are made into the earth.

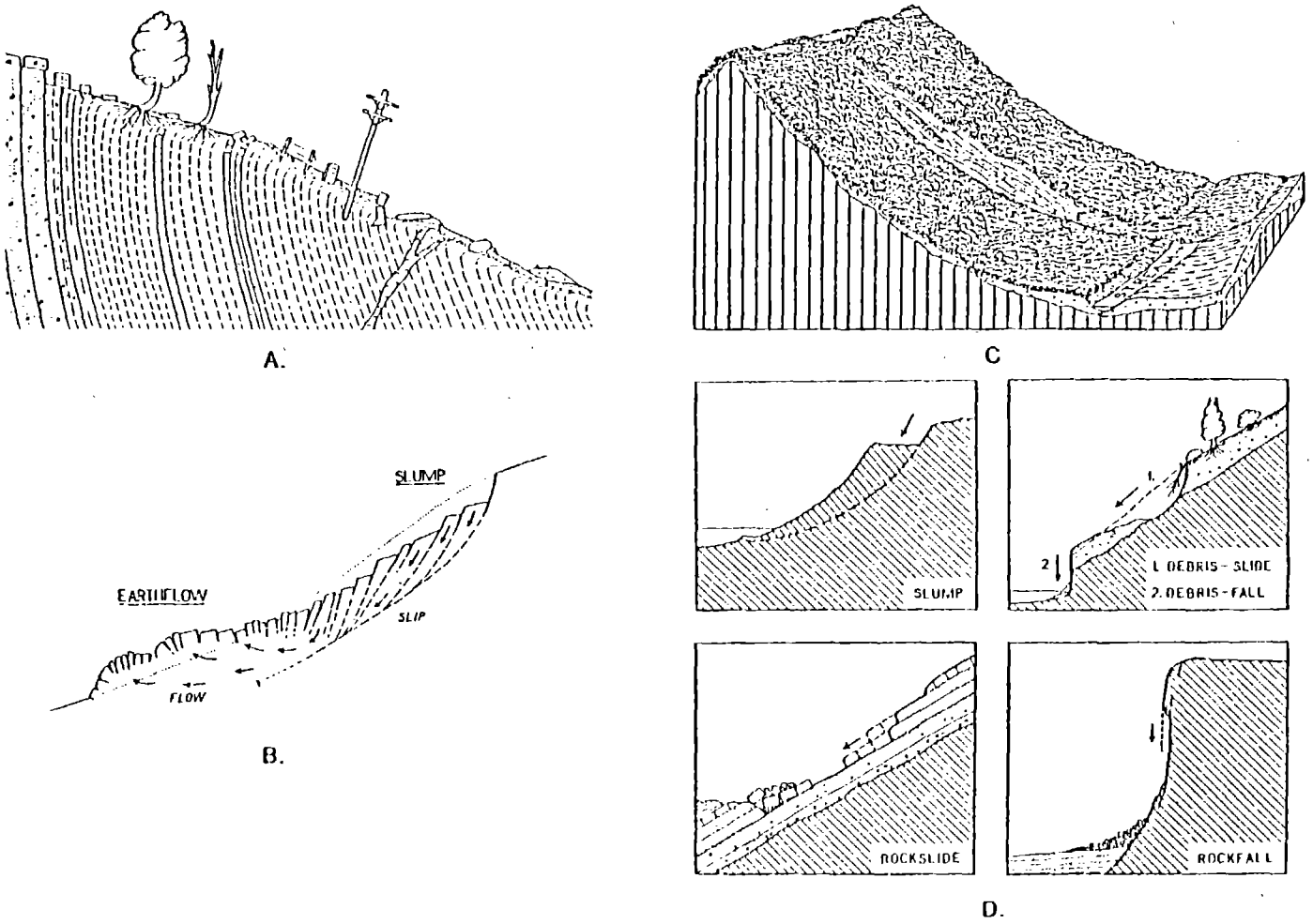
A comprehensive treatment of landslides is beyond the scope of this report. Only a few key considerations are made here. The reader is referred to Landslides and Related Phenomena by C. F. Stewart Sharpe (1938) for a classic treatment of the subject. More recent knowledge and engineering experience is summarized in Landslides - Analysis and Control, edited by Schuster and Krizek (1978).

Classification. Various classifications have been proposed for mass movements (Sharpe, 1938; Varnes, 1978; Skempton and Hutchinson, 1969). Sharpe's classification is reproduced in Table 2.1. Sketches of some of the types of movement are shown in Figure 2.5. More specific variations of mass movements in clay,

Table 2.1

SHARPE'S CLASSIFICATION OF MASS MOVEMENTS

Movement		Earth or Rock Plus Ice	Earth or Rock Dry or with Minor Amounts of Ice or Water	Earth or Rock Plus Water
Kind	Rate			
Slow Flowage	Usually Imperceptible	Rock-Glacier Creep Solifluction	Rock Creep Talus Creep Soil Creep	Solifluction
Rapid Flowage	Slow to Rapid Perceptible Rapid	Debris Avalanche		Earth Flow Mud Flow Semiarid Alpine Volcanic Debris Avalanche
Slip Landslide	Slow to Rapid Perceptible Very Rapid		Slump Debris Slide Debris Fall Rock Slide Rock Fall	



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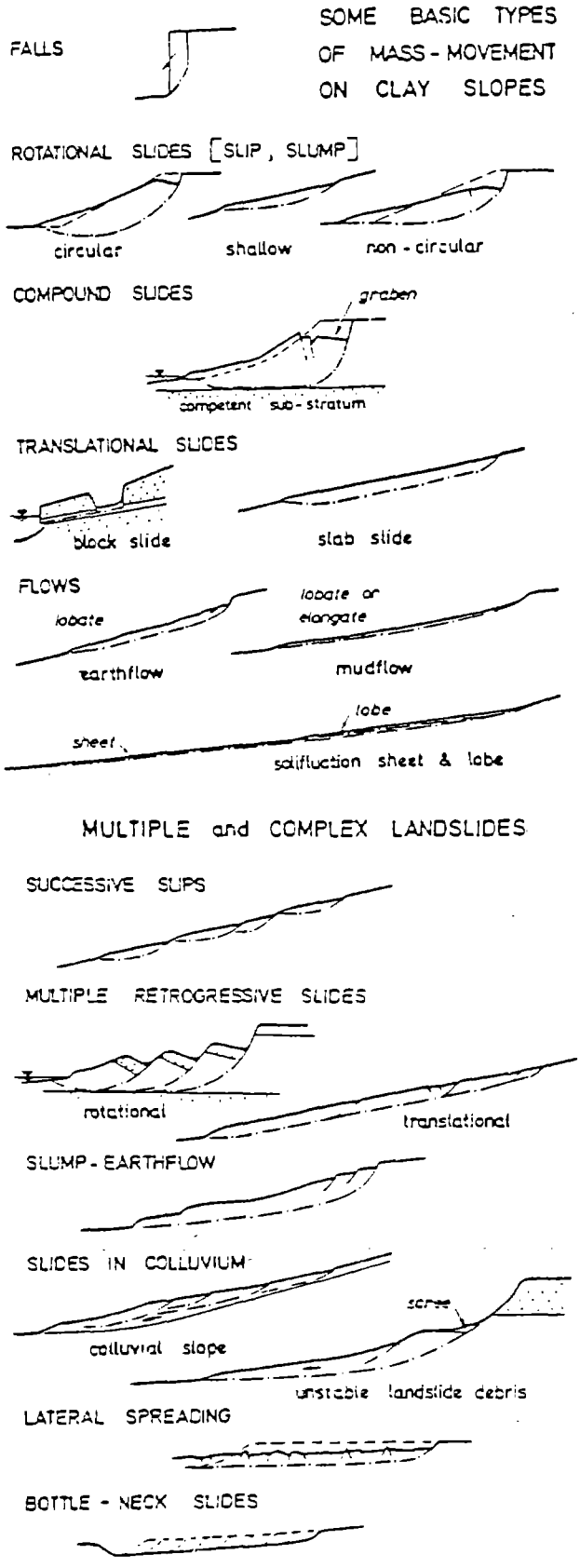
FIGURE 2.5 – ILLUSTRATIONS FROM SHARPE'S MASS MOVEMENT CLASSIFICATION. A., CREEP MOVEMENTS: B., EARTHFLOW AND SLUMP: C., DEBRIS-AVALANCHE: D., LANDSLIDES (after Sharpe, 1938).

according to Skempton and Hutchinson (1969), are shown in Figure 2.6. The purpose of landslide classification is to enable engineers to understand the types of materials and failure mechanisms involved in sliding so that analysis and corrective measures can be carried out (Gray, 1975). Whatever classification system best suits the specific landslide under consideration is the most appropriate one to use. The variety of classifications seems unlimited.

From Figures 2.5 and 2.6 it is apparent that three modes of ground movement--translational, rotational and distortional--are possible in landslides. Vertical movements can be on the order of several feet to tens of feet. Horizontal movements normally fall in the same range, but may be as great as thousands of feet. The nature of ground movements in landslides depends to a large degree on the shape of the failure surface. A wide variety of possible failure surfaces is indicated in Figure 2.6. Wilson (1970) presents numerous cases where movements occurred along horizontal surfaces of sliding. The most prominent and frequently reported damage occurs from the more spectacular forms of mass movements, such as earthflows and slumps.

It is the belief of the authors that soil creep, which is the slow, usually imperceptible downslope movement of superfical soil or rock debris, may be responsible for a good deal of minor structural damage, such as basement wall cracking, that almost never receives the attention of structural, foundation, or mining engineers. Pestrong (1976) reports that such imperceptible movements can only be measured in centimeters per year, and are much more prevalent than is commonly realized. He says that after long periods of time they can cause a great deal of damage to structures with shallow foundations. This condition could be compared with the slow, progressive nature of some observed subsidence-induced building damage. Although creep is generally thought to occur most rapidly in the upper few feet of soil, field measurements indicate that creep movements may occur at depths up to 10 meters (Carson and Kirkby, 1972). One of the most significant features of creep, at least in the Appalachian Plateau section of the Eastern United States, is that it prepares the way for the occurrence of localized and much faster earthflows (Sharpe and Dosch, 1942).

Mechanisms of Mass Movement. The mechanisms by which mass movements occur is the subject of a classic paper by Karl Terzaghi (1950 a). In that work, causes of landslides are classified as either: (a) external--those which increase the shearing stresses along the potential surface of sliding; or (b) internal--those which lead to a decrease in the shearing resistance along a potential surface of sliding. Common external causes are steepening or heightening a slope by erosion at the toe or by excavation near the toe; deposition or placement of material along the upper edge of the slope; and earthquake shocks. Common internal causes are increase in soil pore water pressure due to a rise in the groundwater level, and the progressive decrease of the cohesion component of shear strength. There are causes, such as rapid drawdown of



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FIGURE 2.6 - MASS-MOVEMENT ON CLAY SLOPES
(after Skempton and Hutchinson, 1969).

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water on a submerged slope and subsurface erosion, which are both external and internal in nature. Some causes, such as excavation of the slope toe and overloading of the top of a slope, are related to human activity. Others, such as buildup of pore water pressure due to heavy rainfall and river erosion of a slope toe, are completely natural. Figures 2.5 and 2.6 show mass movements mostly on steep slopes, but certain types of mass movement are known to occur on slopes so flat that landsliding may not be suspected as a source of ground movement. Gray and Donovan (1971) report evidence of pre-existing failure surfaces (indications of past landslide occurrence) on slopes as flat as 7° to 10° in the Allegheny Plateau region of western Pennsylvania and West Virginia.

Prediction of Landslide Movements

The stability of a hillslope of a given geometry is dependent on the strength of the geologic materials which underlie the slope. Slope failure occurs when the strength of the materials is exceeded by the stresses that act on a potential failure plane. It would seemingly be a straight-forward matter to calculate the stresses on a potential failure plane, measure the strength of the slope materials, and determine the safety factor (SF) by comparing the available strength to the acting stresses:

$$SF = \frac{\text{Available Strength}}{\text{Acting Stresses}}$$

Slope failure should theoretically occur when the safety factor is less than one.

If the safety factor is less than one and if the slope fails, there is no indication from the calculation as to how much movement will take place. If the stresses that caused failure suddenly decrease after failure takes place, movements may cease very quickly. If, on the other hand, the stresses are not relieved, slope movement may continue indefinitely.

The extent to which movement continues after failure depends largely on the stress-strain characteristics of the slope materials (Morgenstern and Sangrey, 1978). Typical stress-strain relationships for materials commonly involved in slope failures are shown in Figure 2.7. If the slope material has the stress-strain characteristics of curve (A), the slope will continue to move after failure as long as the stresses that caused failure remain equal to the ultimate strength. This behavior is typical of loose sand and many clays. If the slope material has the stress-strain characteristics of curve (B), the slope will continue to move after failure as long as the stresses that caused failure equal the residual strength of the material, a value that may be on the order of one-half the ultimate strength. Dense sands and overconsolidated clays exhibit this type of behavior. (Overconsolidated clays are those that exist under ambient effective stresses less than have acted in the past.)

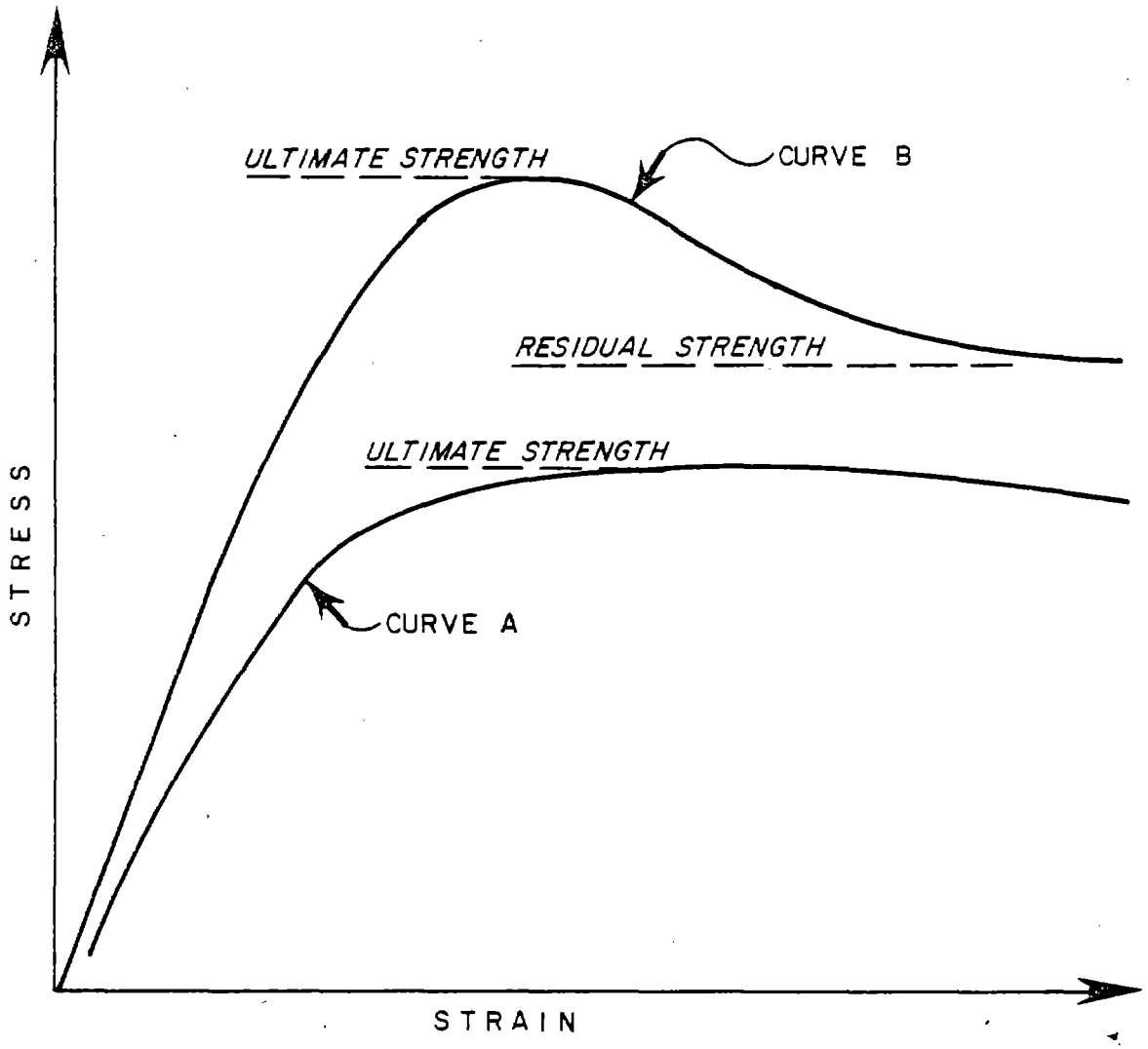


FIGURE 2.7 - STRESS-STRAIN RELATIONSHIPS FOR SOILS. TYPICAL RESULTS FROM CONSOLIDATED UNDRAINED TRIAXIAL TESTS

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Obviously, an attempt to predict ground movements in a landslide would require not only an accurate knowledge of the stress-strain characteristics of the slope forming materials, but also a complete history of stresses (including pore water pressures) acting in the slope before, during and after failure. A great deal of effort has been expended by soil mechanics researchers to develop procedures for estimating the safety factor of slopes, i.e. conditions up to the point of failure. Significant strides have been made in predicting the fluctuation of groundwater levels as a function of precipitation, in this way enhancing the utility of both deterministic and probabilistic methods in evaluating the stability of slopes and developing mitigation measures (Sangrey et al., 1982). Yet, assessing the stability of slopes is commonly fraught with so many uncertainties that it leads experienced professionals to make such comments as:

> "Generally a quantitative slope stability analysis should not be used to speculate on the absolute factor of safety of a present natural slope." (Patton and Hendron, 1974)

After a major mud flow in the Baker River Valley, Washington, completely destroyed a powerhouse, one of the three principal investigators, who had been monitoring the slope's behavior over a one-year period prior to the landslide made the following statement:

> ". . .we have lost much of our confidence in our ability to predict the behavior of a natural hillside or in the results of our remedial measures. On this project, it is evident that nature was able to outwit us, and we fear she can and will do so on similar occasions in the future. This I submit is the present state of the art." (Peck, 1967).

On the basis of these judgments with regard to safety factor analysis, it is no great surprise that experienced professionals hesitate to predict the magnitude of movement in a landslide.

Somewhat more promising is qualitatively assessing the susceptibility of a given slope to failure. Because local geologic conditions such as stratigraphy, soil weathering, rock structure and ground water conditions play such an overwhelming role in slope behavior, landslide conditions in any given geographic area often repeat themselves in specific geologic formations. Sometimes distinct ground surface forms, soil conditions and stratigraphic profiles will give a strong indication of past or probable future behavior of a hillside slope. In western Pennsylvania, for example, slopes in outcropping areas of the Pennsylvanian-age Pittsburgh red beds are notoriously prone to landsliding, both on natural slopes and in construction excavations. The Pierre and Bearpaw shales of the Missouri River Basin hold a similar reputation. Therefore, in attempting to understand, assess, or predict slope behavior in any specific geographic region, one naturally turns to local experience as the best guide.

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Landslide susceptibility on a regional basis is indicated in Figure 2.8. It is interesting to note that some of the areas designated as 'major landslide severity' coincide with areas of major underground coal mining activity, notably the Pittsburgh-Huntington Coal Basin.

Damage. That landsliding and mass-movements are significant agents of building damage in the central and eastern United States is not a matter needing lengthy explanation. Briggs, Pomeroy, and Davies (1975) report that the costs of damages from landsliding in Allegheny County, Pennsylvania (the Greater Pittsburgh area) in the period 1970-1974 have been estimated at nearly two million dollars annually, a figure that does not include many small unreported events. Examples of building damage caused by various forms of mass movement are shown in Figure 2.9. Reported damages, in general, vary from minor cracking to total destruction. Frequently, movements occur rapidly, on the order of feet per day.

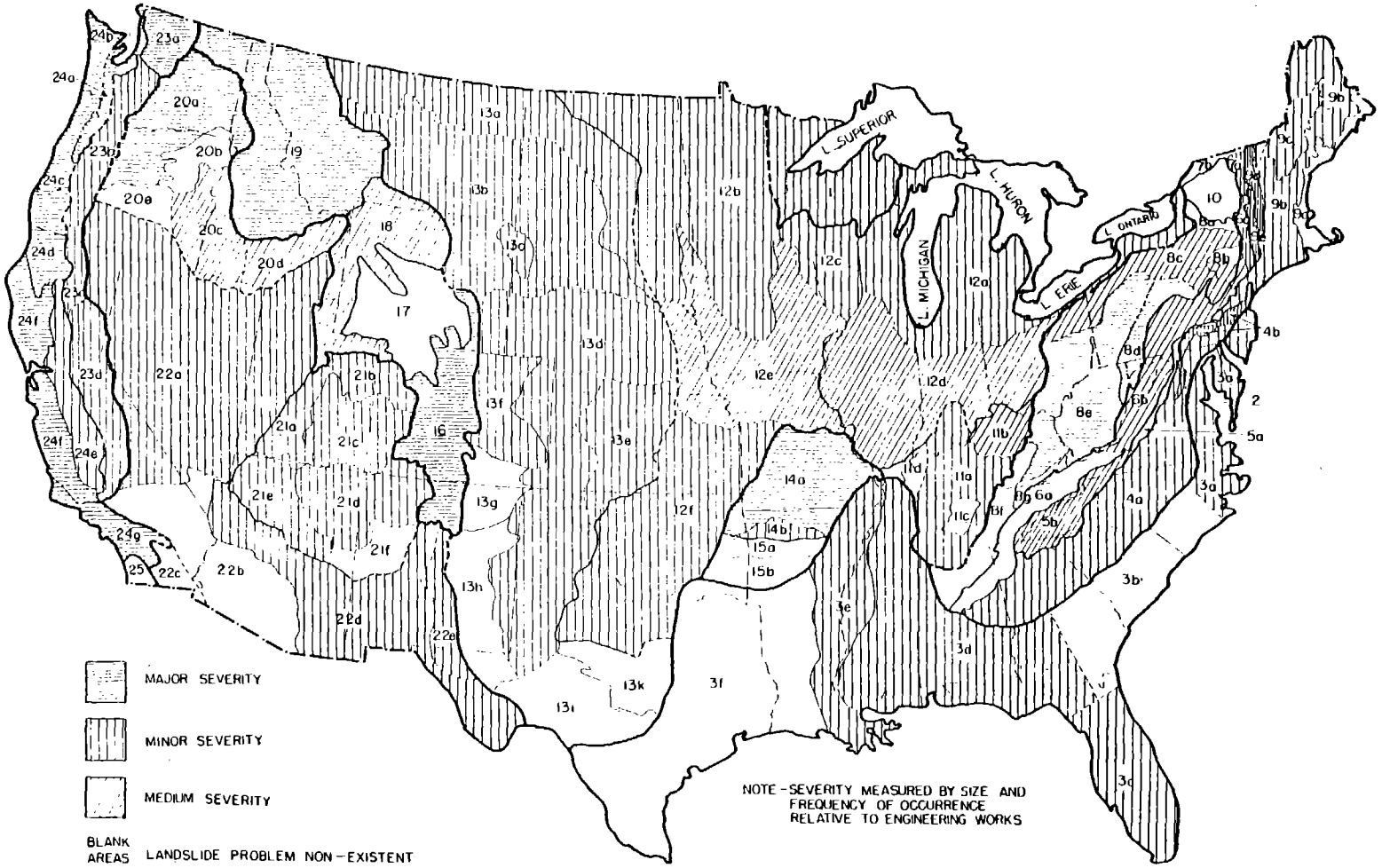
In the Pittsburgh area, backyards are commonly created for small, closely spaced city lots by the informal dumping and leveling of soil and rock fill on unprepared natural hillslopes. Generally, no provision is made for surface or subsurface drainage, nor is the fill keyed into stable hillslope materials to ensure long term stability. The backyard landslides that occur so frequently sometimes bear a vague resemblance to subsidence features above abandoned mines--a point not lost on the calculating homeowner who recognizes he might be able to recover damages to his home through a state operated subsidence insurance program if authorities can be convinced the ground movement is due to previous underground mining, but will recover nothing if the ground movement is simply a landslide. Case History 1, presented in Appendix A, describes one such case.

EROSION

General Considerations. Erosion is perhaps more properly considered a manifestation of ground removal than ground movement, but is discussed here because it can sometimes lead to significant disturbance of the ground surface and building damage.

Ground movements associated with erosion can be translational, rotational, or distortional. Erosion is unlikely ever to occur uniformly beneath a structure, and thus damages most often result from an erratic local loss of support, an insidious condition from a structural viewpoint.

The American Geological Institute (1962) defines erosion as "The group of processes whereby earthy or rock material is loosened or dissolved and removed from any part of the earth's surface. It includes the processes of weathering, solution, corrasion, and transportation. The mechanical wear and transportation are effected by running water, waves, moving ice, or winds, which use rock fragments to pound or grind other rocks to powder or sand."



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FIGURE 2.8 - LANDSLIDE SEVERITY IN THE UNITED STATES. NOTE MAJOR SEVERITY IN REGION UNDERLAIN BY PITTSBURGH COAL (SW PENNSYLVANIA, WEST VIRGINIA, SE OHIO) (after Baker and Chieruzzi, 1959)



A.

FIGURE 2.9 - EXAMPLES OF BUILDING DAMAGE CAUSED BY MASS MOVEMENTS. A. DIAGONAL CRACKS IN BRICKWORK OF HOME SUBJECTED TO SLUMP-EARTHFLOW.



B.

FIGURE 2.9 - B. HEAD OF LATERAL SPREADING MOVEMENT IN QUICK CLAY. BUILDING AT LEFT IS TORN IN HALF, OTHER BUILDINGS MOVED TENS TO HUNDREDS OF METERS WITH MINOR DAMAGE.

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Disregarding the direct-hit effects of such geological agents as floods, tidal waves, avalanches, etc., there are two broad categories of erosion that affect the ground surface: surface erosion and subsurface erosion.

Surface Erosion. Removal of surface soil directly from beneath a building by wind or water is probably an uncommon cause of building damage, but can occur under certain conditions. Johnson and Kavanagh (1968) point out that it is most likely to occur in warmer climates where there is no frost and hence no need to place footings or foundations below a given frost depth (usually 2 to 5 feet). Water originating from downspouts, clogged drains and diverted gutter flow can be responsible for removal of soil from beneath shallow footings, as can wind. Silts, sands and other cohesionless materials are most likely to be affected.

Johnson and Kavanagh (1968) show a case in which wind action was responsible for erosion of 18 to 24 inches of soil from around a footing. The form of damage that might result from such erosion is illustrated in Figure 2.10. Unusually heavy flows of water, such as those from flooding, are capable of removing even greater thicknesses of overburden. Scour of river bed materials from around and beneath a bridge pier is an extreme example.

Subsurface Erosion. Subsurface erosion is a much more hidden, subtle and serious cause of ground movement than surface erosion. It is also much more common. It can occur by several mechanisms: (a) piping; (b) boils and blowouts; and (c) solution. The mechanisms are sometimes interrelated. Any of these mechanisms may lead to ravelling, caving and eventual collapse of the ground surface. The processes can be initiated by human activities or may be natural.

a. Piping. Piping is the movement of individual soil grains by the seepage of water. When unchecked, piping can lead to progressive backward erosion around concentrated leaks or seeps. It can occur by the infiltration of groundwater or surface water into fractured sewers, drains, or culverts, carrying with it fine soil particles and leaving behind void spaces in the soil; or by similar infiltration through shrinkage cracks, joints or other natural open voids in bedrock, or along sand or gravel layers.

   Examples of ground movements resulting from these processes are shown in Figure 2.11.

   According to Sowers (1976), undersized sewers and culverts are particularly dangerous because the water is under pressure during periods of high flow and is forced into the soil through weak connections or breaks. As the flow recedes, water seeps back to the sewer or culvert and carries soil particles with it. Sowers (1974a) describes the collapse of a warehouse wall into a 35-foot wide by 45-foot long hole in the ground that resulted

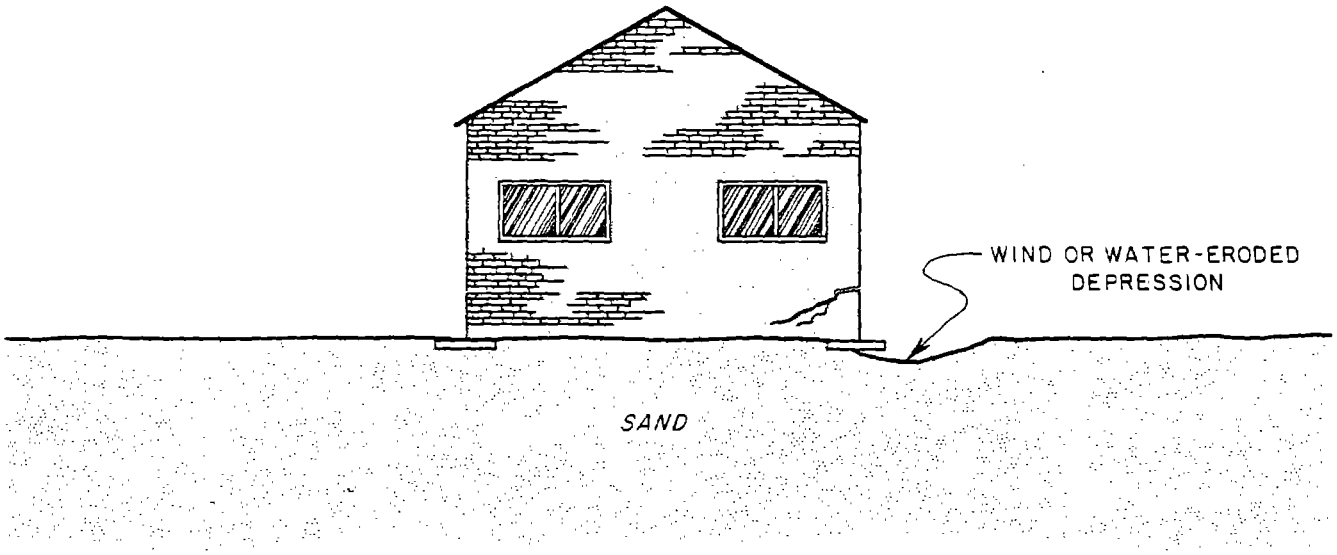
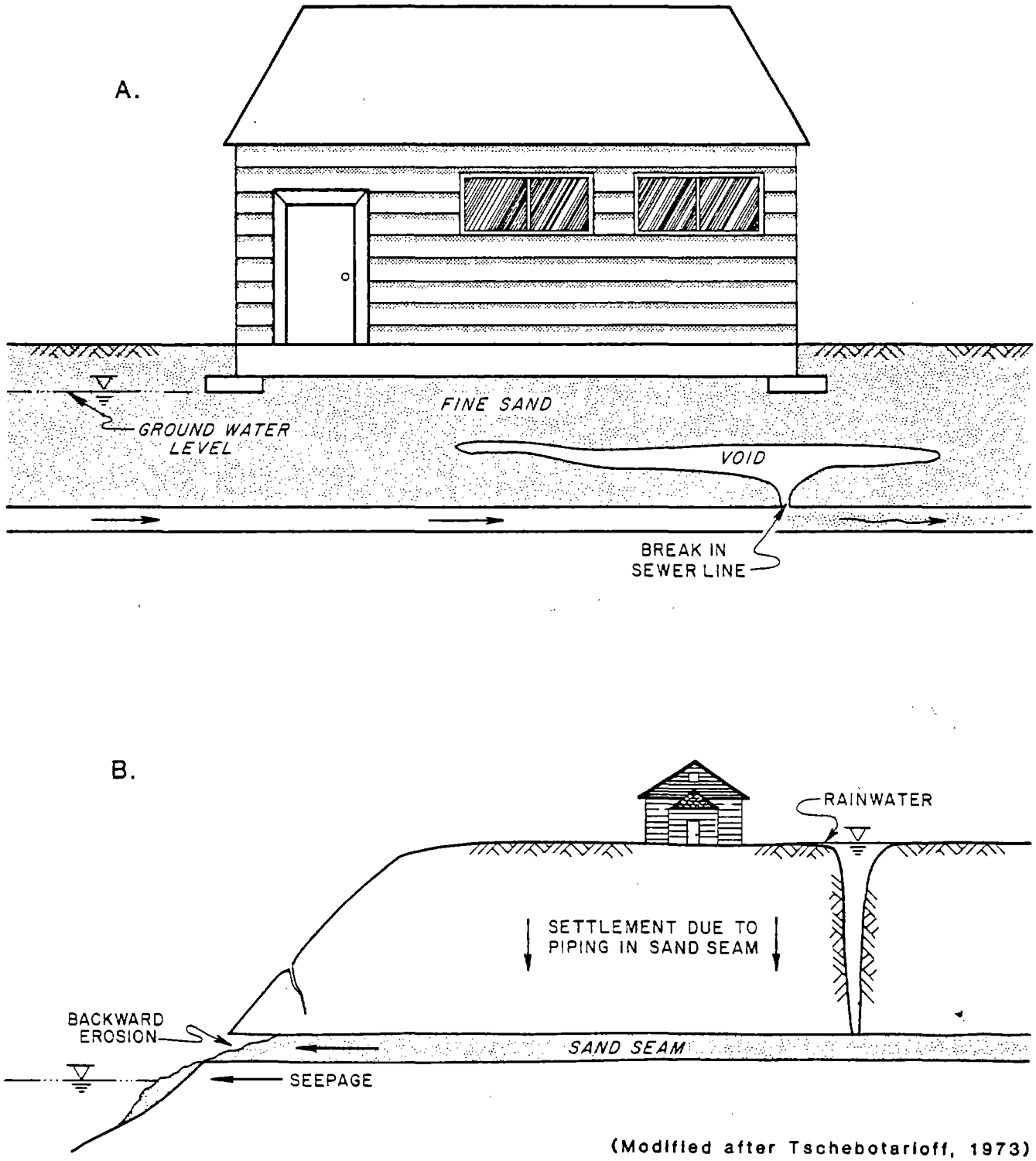


FIGURE 2.10- CRACKING OF MASONRY WALL DUE TO FOUNDATION UNDERMINING BY SURFACE EROSION



(Modified after Tschebotarioff, 1973)

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FIGURE 2.11 - GROUND MOVEMENTS CAUSED BY PIPING. A. VOID CREATED BY PIPING OF SAND INTO BROKEN SEWER: B. SETTLEMENT CAUSED BY PIPING WITHIN NATURAL SAND SEAM

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when an old overloaded storm sewer tunnel carried away a large volume of nearby soil (Figure 2.12). The process apparently initiates by piping, but accelerates by the ravelling of slabs of soil and boulders. Tschebotarioff (1973) discusses two cases of severe building damage that occurred as a result of piping in a natural sand seam (Figure 2.11B). In situations such as this, water could derive from a point source, as shown in the figure, or from an elevated, ground water table.

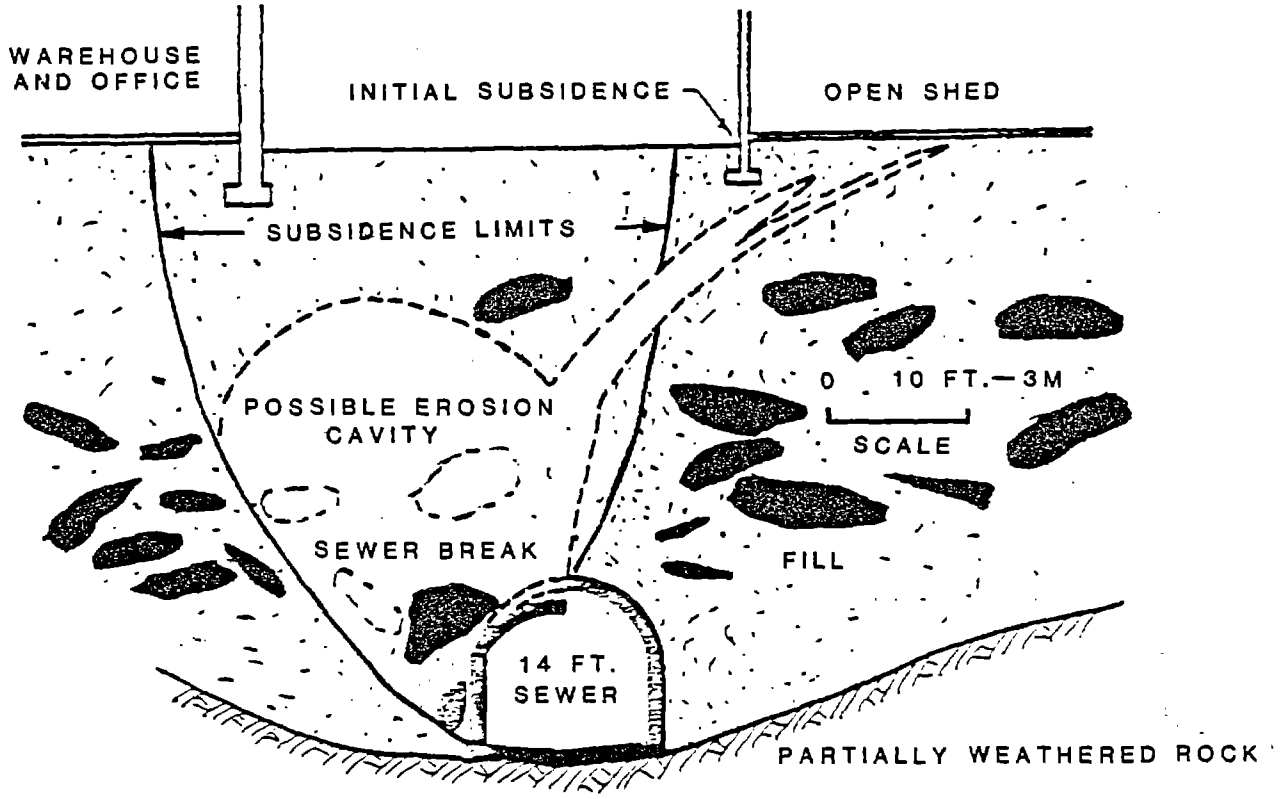
A substantial ground collapse occurred during the construction of a sedimentation basin at a power plant near Harrisburg, Pennsylvania. The basin was within several hundred yards of a river, and apparently large quantities of soil materials had eroded through an undiscovered underground channel of unknown origin before a sinkhole-type opening appeared suddenly in the basin bottom. The occurrence of such phenomena along rivers is not a matter of coincidence. Flood plains typically contain deposits of fine-grained cohesionless soils (silts and sands) that are highly susceptible to piping. Any layer of slightly cohesive soil that overlies the cohesionless materials can temporarily bridge over an underground erosion tunnel. These soil conditions, combined with the usual presence of flowing ground water in a floodplain, may provide the ideal circumstances for subsurface erosion by piping. An outlet for the discharge of flowing water and disposal of transported material is provided by the river.

Feld (1965) illustrates two cases in which uncontrolled water flow into overloaded drain pipes that had burst undermined footings and caused the complete collapse of the wall of a building (Figure 2.13).

A most unusual, and apparently natural, case of ground movement caused by piping occurred in Clarksdale, Mississippi. The top of a subsurface clay stratum located 10 to 22 feet below ground surface contained conspicuous shrinkage cracks that allowed the downward piping of overlying silt and fine sand, eventually leading to collapse of the ground surface (Allen, 1969).

An extreme example of natural subsurface erosion occurred on the east bank of the Mississippi River near Memphis, Tennessee, after a high river stage in 1927. Collapse of the ground surface above an underground sand flow over a period of two months led to 60 feet of surface subsidence in a 700-foot long by 100-foot wide strip of land along the river bank bluff (Terzaghi and Peck, 1967).

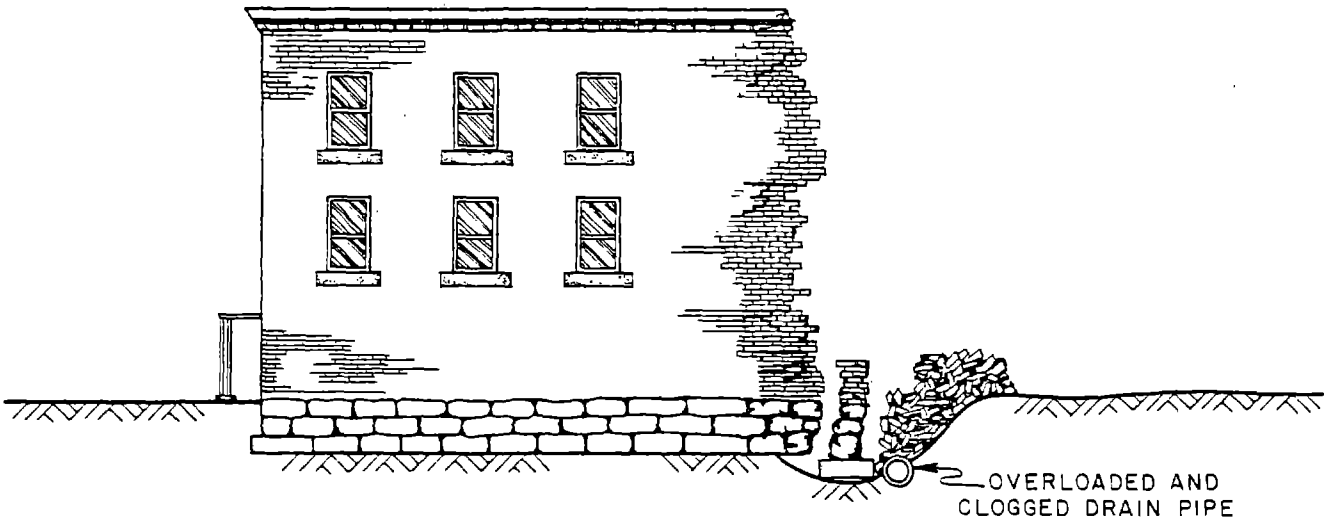
Removal of groundwater in proximity to a construction excavation by deep wells or open sumps is a common source of subsurface erosion. If a filter or screen is not



SUBSIDENCE OVER A 14 FT. TRUNK SEWER UNDER 45 FT. FILL

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FIGURE 2.12 - GROUND SUBSIDENCE AND BUILDING FAILURE PRODUCED BY COLLAPSE OF A CAVITY GENERATED BY A DAMAGED SEWER FLOWING UNDER PRESSURE
(after Sowers, 1974a)



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FIGURE 2.13 - DAMAGE CAUSED BY UNDERMINING OF WALL FOOTING DUE TO CLOGGED DRAIN
(AFTER FELD, 1968)

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provided on the pump, substantial quantities of sand and silt can be pumped with the water. The piping that results can lead to the development of erosion tunnels which may eventually collapse to form sinkholes or subsidence troughs.

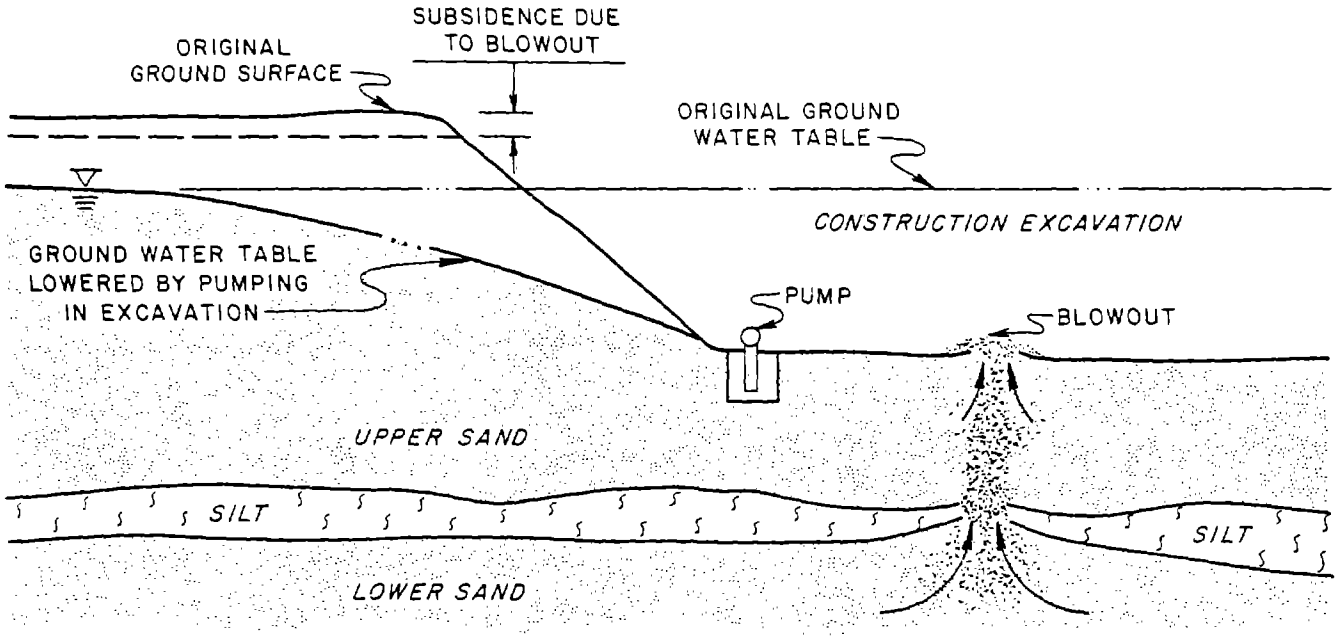
b. Boils and Blowouts. An unusual form of subsurface erosion can occur when the pore water pressure in a deposit of sand or silt exceeds the intergranular pressure between individual soil particles. If the pressure finds a natural "release" by pushing up the ground surface the result is called a sand blow, blowout, or sand boil. Piping usually accompanies this phenomenon, resulting in removal of subsurface soil material and subsequent ground surface subsidence.

Sand boils or blowouts sometimes occur in the floor of an open excavation during the dewatering process. The conditions leading to the formation of a blowout are illustrated in Figure 2.14A. The silt layer shown in the figure represents a relatively impermeable zone beneath the sand deposit being excavated. As the ground water table in the upper sand is lowered by pumping from an open sump in the excavation, the pore water pressure in the lower sand remains unchanged. At some depth of excavation, the hydrostatic pressure in the lower sand would exceed the weight of soil and water between the base of the excavation and top of the lower sand. At this depth, the bottom of the excavation may heave or blowout, spewing sand and water into the bottom of the excavation. If this condition goes unchecked, sufficient soil may be removed from the lower sand layer to cause subsidence of the ground surface surrounding the excavation.

A similar phenomenon can occur in braced excavations where water is allowed to percolate freely into the bottom of the unprotected excavation (Figure 2.14B). Terzaghi and Peck (1967) describe how this condition was responsible for six-inch settlements of buildings adjacent to a New York City subway excavation.

Blowouts can occur well away from any construction activity or pumping. The heaved pavement at the gasoline station shown in Figure 2.15 was caused by the buoyant uplift of a pair of buried storage tanks which had been completely emptied of gasoline prior to the station going out of business. The groundwater table was only a few feet below ground surface at the time of the incident. Correct practice would, of course, have been to fill the storage tanks with water.

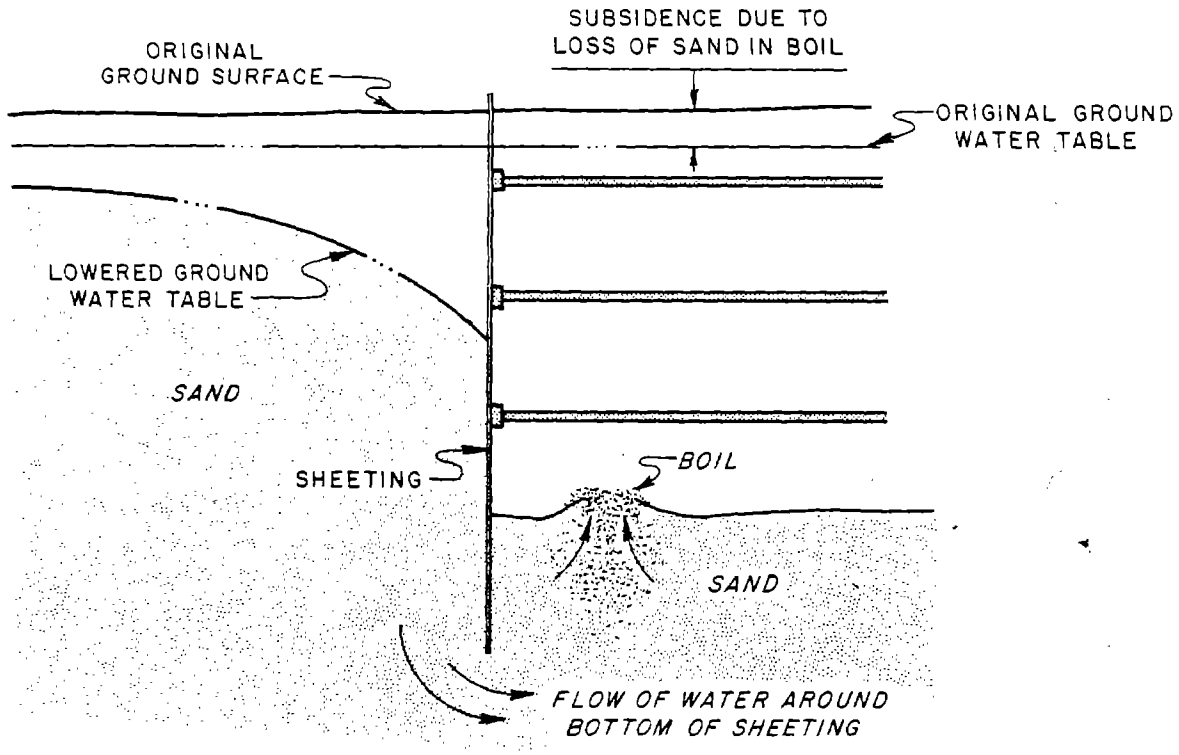
Sand boils can occur under a variety of circumstances. A ruptured water line could produce boils if the soil



(Modified after Terzaghi and Peck, 1967)

A.

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B.

FIGURE 2.14 - EROSION OF SAND IN BOTTOM OF DEWATERED CONSTRUCTION EXCAVATIONS. A. BLOWOUT IN BOTTOM OF UNBRACED EXCAVATION: B. SAND BOIL IN BOTTOM OF SHEETED EXCAVATION.

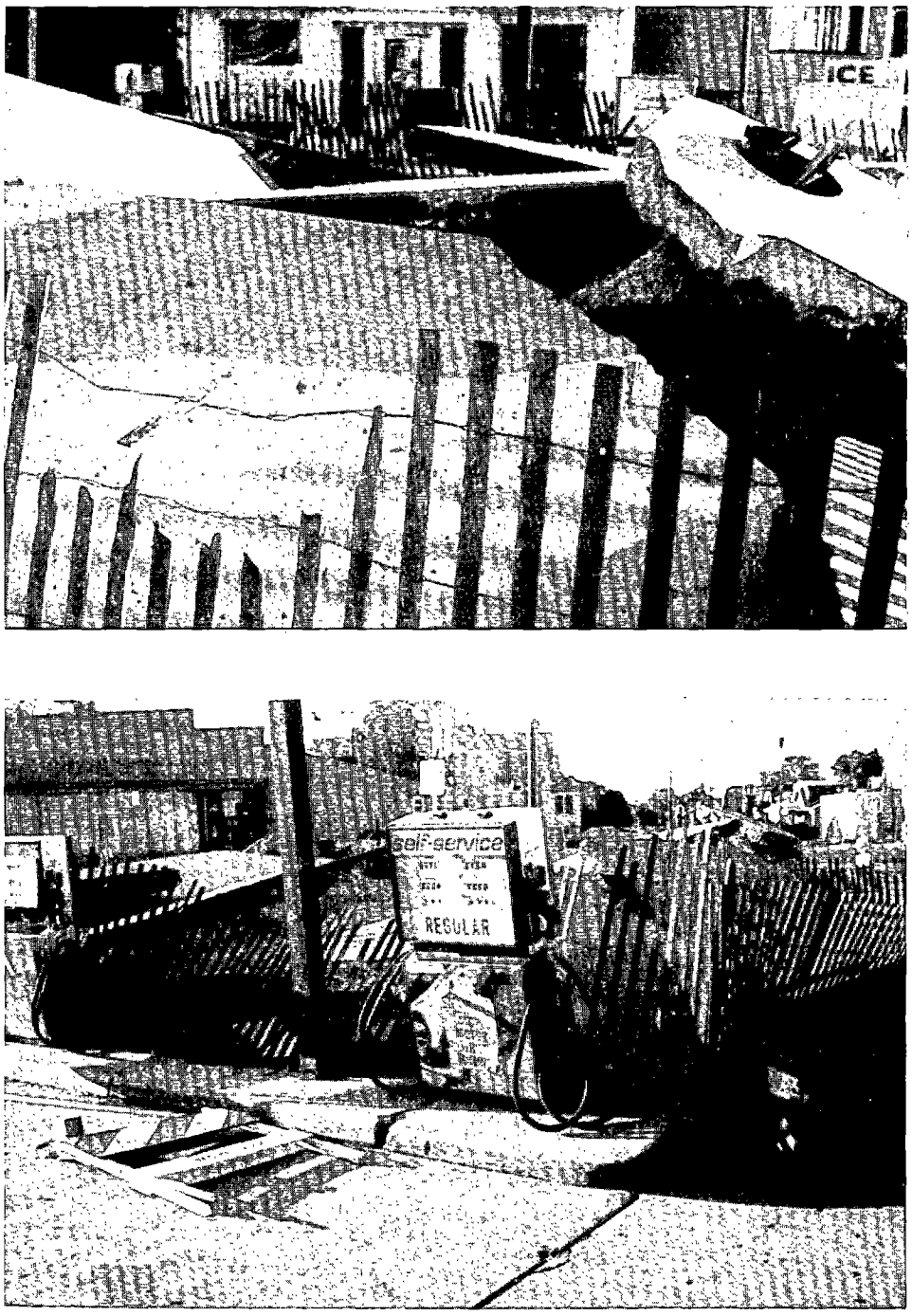


FIGURE 2.15 - BLOWOUT CAUSED BY THE BUOYANT UPLIFT OF BURIED GASOLINE STORAGE TANKS

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overlying the line is cohesionless and the pressure at the leak exceeds the weight of the overburden. Along major rivers that are controlled by levees, such as the Mississippi, boiling may occur on the land side of the levee if the river level rises above the flood plain level and if the flood plain soils consist of cohesionless silts and sands. Such action has resulted in local street subsidence in portions of Wilkes-Barre, Pennsylvania, that lie on very low flood plains adjacent to the Susquehanna River.

Sand boils, or sand blows, sometimes occur naturally during earthquakes. The phenomenon has been described by various witnesses as jets of sand and water spouting four to six feet into the air, leading to the formation of miniature sand volcanoes. Crater diameters have been reported as small as several inches and as large as 20 feet (Housner, 1958). The mechanism is explained by Housner in terms of a dynamic release of overpressure in the vicinity of small regions of higher permeability soils near ground surface. Sand blows are characteristically associated with areas of thick saturated cohesionless soils overlain by more coherent surficial strata (Allen, 1969). In all likelihood they are related to liquefaction, a phenomenon discussed under Bearing Capacity Failures.

Housner (1958) reports that ground surface subsidence in the 1934 Bihar, India, earthquake amounted to 6 to 12 inches in a region where extensive sand blows were observed. It is not known if that subsidence was due to subsurface erosion by sand blows, densification of sand by vibration, tectonic movements in the earth's crust or a combination of all three mechanisms. This lack of distinction is common in most recorded earthquake subsidences.

Fuller (1912) describes the forceful extrusion and ejection of water, sand, mud, and gas from the ground which took place in the New Madrid, Missouri, earthquake of 1817. "Sand blows," "sand scatters" and "sand sloughs" are the terms he uses to describe the surficial deposits resulting from the upward flow and extrusion of materials. He attributes this action to: (a) closing of earth fissures in the alluvium; (b) escape of gases generated by the decay of buried organic matter; and (c) local and temporary development of artesian conditions. Regardless of cause, the extrusion phenomena reflect subsurface erosion, and probably were at least partly responsible for the 10 to 20 feet of subsidence observed over extensive areas of the Mississippi Valley during the New Madrid earthquake.

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c. Solution. The natural process of underground solution of limestone that causes the development of karstic limestone terrains is well known. The progressive collapse of underground channels and upward migration of fractures through the weathered rock and soil mantle may result in the development of sinkholes, or swallow holes, at ground surface. The same processes can take place in gypsum or salt deposits (Emmons, Allison, Stauffer and Thiel; 1960).

Sinkholes may sometimes exceed 100 meters in diameter and 50 meters in depth. The triggering mechanism that leads to their formation can be related to totally natural phenomena, such as heavy rainfall, seasonal fluctuations in the water table, earthquakes, or other changes in the hydrogeologic regime (LaMoreaux and Warren, 1973). Human activities, such as modification of surface drainage, dewatering, breaks in water or sewage pipes, and even overwatering of gardens may provide the impetus for collapse. Benson (1978) states that changes in surface water drainage and groundwater seepage are major factors in triggering the collapse of subsurface voids in limestone. Either a deficiency or an abundance of water can enlarge a cavity and cause the collapse to occur.

Sowers (1976) describes the collapse above underground openings as a three-stage process (Figure 2.16):

1. Collapse of a truncated cone of material into the opening;

2. Sloughing or collapse of the overhanging walls; and

3. Eventual development of an open hole if erosion of the dropout cone occurs by seepage from surface infiltration or subsurface water. Partial collapse leaves a saucer-shaped depression in the ground surface.

The solution openings into which collapse occurs in limestone terrain need not be cavernous. Sowers (1975) explains how collapse can occur by ravelling and erosion of soil into bedrock joints and fissures enlarged by solution (Figure 2.17). An "hourglass" effect can result when solution channels are overlain by dry cohesionless sand. The only requirement for development of surface collapse features is adequate circulation of water through the cavities in the rock to carry the ravelled and collapsed soil away. Allen (1969) points out that transportation of material may occur over a long period of time before a break appears in the ground surface.

Movements are likely to be sudden, catastrophic, and on the order of several feet to tens of feet. Numerous

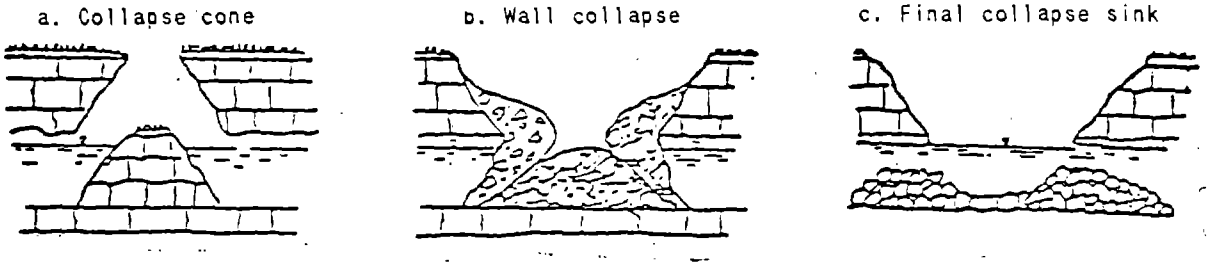


FIGURE 2.16 - PROGRESSIVE DEVELOPMENT OF SINKHOLE COLLAPSE IN LIMESTONE (after Sowers, 1976).

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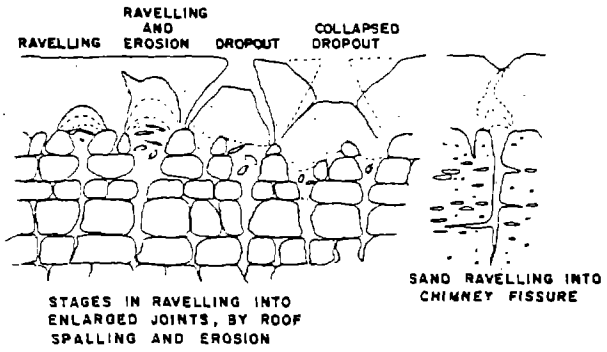


FIGURE 2.17 - MECHANISM OF RAVELLING COLLAPSE IN LIMESTONE SOLUTION FEATURES (after Sowers, 1975).

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writers report sensational cases in which buildings have collapsed into sinkholes and solution cavities (Sowers and Sowers, 1970; Legget, 1973; Feld, 1968). Sowers (1975) reports that ravelling failures through small rock openings of the type shown in Figure 2.17 account for more loss of property and structures than all other failures associated with limestone formations combined. He indicates the actual collapse of underground solution cavities, as shown in Figure 2.16, is not as common as the ravelling-type collapse. Dramatic ravelling type collapses affected the Hershey Valley in Eastern Pennsylvania in the early 1950's. Over one hundred sinkholes, ranging from 1-foot to 20-feet in diameter and in from 2 to 10 feet in depth, opened in the valley when ground water levels dropped 30 to 40 feet due to pumping in a nearby limestone quarry. Foose (1969) describes the sinkholes as follows:

"All of them had very steep walls as though the bottom had suddenly been pulled out and the unsupported earth above had quickly slumped. Essentially that is what happened . . . . Water flowing in the channel of Spring Creek moved downward through whatever openings were available into solution-widened joints and bedding planes of the limestone, carrying along large quantities of silt, sand, and gravel."

The large voids that appeared beneath buildings at the Hershey Chocolate Company caused understandable alarm.

A rather unusual case of ground surface movement caused by the subsurface solution of a salt massif in Rumania is reported by Beles and Stanculescu (1958). A 4-story bathing sanatorium founded on the salt massif settled up to 2.5 feet as a result of the solution of the salt by fresh water in areas where sand and gravel overburden allowed circulation of ground water across the buried surface of the massif. The building suffered cracking damage, but was repaired and returned to use after the solution was stopped by constructing a clay apron over the salt surface. Instances of known subsidence due to gypsum, anhydrite and salt solution in the United States are reviewed by Allen (1969). Of particular note is the account of the sudden overnight disappearance into a sinkhole (presumably a salt solution feature) of a railroad station at Rosel, Kansas. All that remained was a 70-foot deep pond, one acre in extent, surrounded by abrupt walls.

Prediction of Movements due to Erosion

It would be virtually impossible to predict the magnitude of ground movements from any of the erosion processes described. None of them lend themselves to rational analysis or theoretical

treatment. If movement of a structure had been caused by one of the erosional processes, the source of movement may or may not be immediately obvious. Sinkholes and collapse features in limestone terrains usually are conspicuous. Piping, on the other hand, can be subtle, elusive and difficult to detect. It can develop in a matter of days, or may take years. There exists the possibility, too, that subsurface erosion under a given set of conditions will not lead to ground surface movement, provided the near-surface soils or rocks possess adequate strength to bridge over any subsurface voids or cavities that develop.

Actual ground surface movements depend on the following factors: 1) the quantity and velocity of water available for erosion (or solution) and transport of subsurface materials; 2) the erodibility (or solubility) of the materials; and 3) the strength of materials above and around any voids that may form as a result of erosion (or solution). There is no simple theoretical relationship among these factors that determines if subsurface erosion and ground movements will occur, much less the degree of erosion and the magnitude of movement.

In view of the fact that water is the principal agent in subsurface erosion, Benson (1978) lists the following locations as highly susceptible:

1. Areas in and around high rates of ground water withdrawal.

2. Areas where natural surface drainage and ground water patterns have been altered, such as highways, parking lots, airport runways, etc.

3. Areas of leakage from retaining ponds, dams, and other structures.

Although this list strictly pertains to sinkhole formation in karst terrains, it applies equally well to any area where subsurface materials are susceptible to erosion by piping, sand boils, and solution.

Concluding Remarks Concerning Erosion. Ground movements caused by surface and subsurface erosion can occur in a wide range of geologic conditions. The materials susceptible to such action are cohesionless soils and any water-soluble soil or rock, such as limestone. Piping, in particular, has a reputation for occurring in many unlikely areas -- beneath foundations, fills, concrete slabs, and pavements; behind retaining walls; within dams; and in borings and wells. Benson (1978) makes the statement based on years of field experience that "it is reasonable to suggest that all roads and airports have some level of cavity development associated with them." His usage of the term "cavity" includes those caused by various processes described above as well as those caused by subsidence or collapse of abandoned mine shafts, tunnels, and other buried structures.

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FROST ACTION

Frost heave is the uplift of rock fragments or soil caused by the growth of ice (Holmes, 1964). Water, upon freezing, expands about one-eleventh in volume and can exert high pressures that aid in the disintegration of rock and soil material. It is recognized by geologists as a powerful agent of rock weathering and soil formation.

Foundation engineers apply the more general term 'frost action' to a combination of two processes:

1. Freezing of the soil and the formation of ice lenses or layers that cause frost heave; and

2. Thawing of the ice lenses, which provides an excess of free water in the soil and lowers soil strength (Sowers and Sowers, 1970).

Whereas the simple expansion of ice during freezing is important in the natural weathering process of geologic materials, such expansion in soils does not in itself cause significant ground surface movement. Rather, it is the formation of ice lenses caused by the capillary rise of water from the ground water table to an agglomerating ice zone that leads to heaving of the ground surface. Because the formation of ice lenses depends strongly on soil particle size and gradation, both of which can be highly erratic for natural soils, frost heave is usually nonuniform (Terzaghi and Peck, 1967).

The irregularity of frost heaving, together with the conversion of soil to a supersaturated material of mushy consistency upon thawing, create the common problems encountered with highway pavements, concrete slabs, small structures on shallow foundations, culverts, walls and utility lines. Broms and Yao (1964) measured laboratory shear strength losses in excess of 90 percent for specimens of silty clay soil subjected to freezing and thawing.

Building damage can thus result from two causes where frost action takes place:

1. Non-uniform heave, which, like differential settlement, can cause building distortion as shown in Figure 2.18; and

2. Soil bearing failure during and after thaw, which can cause a building to sink into the ground unevenly and distort or cause a wall to topple.

The severity of frost action is related to the frost line--the depth below ground surface to which a 32° F temperature extends. Ice lenses form above the frost line when soil and water conditions are favorable. Normally frost action problems are avoided by placing the foundations for a building below the maximum depth of frost penetration or by removing frost-susceptible soils and replacing

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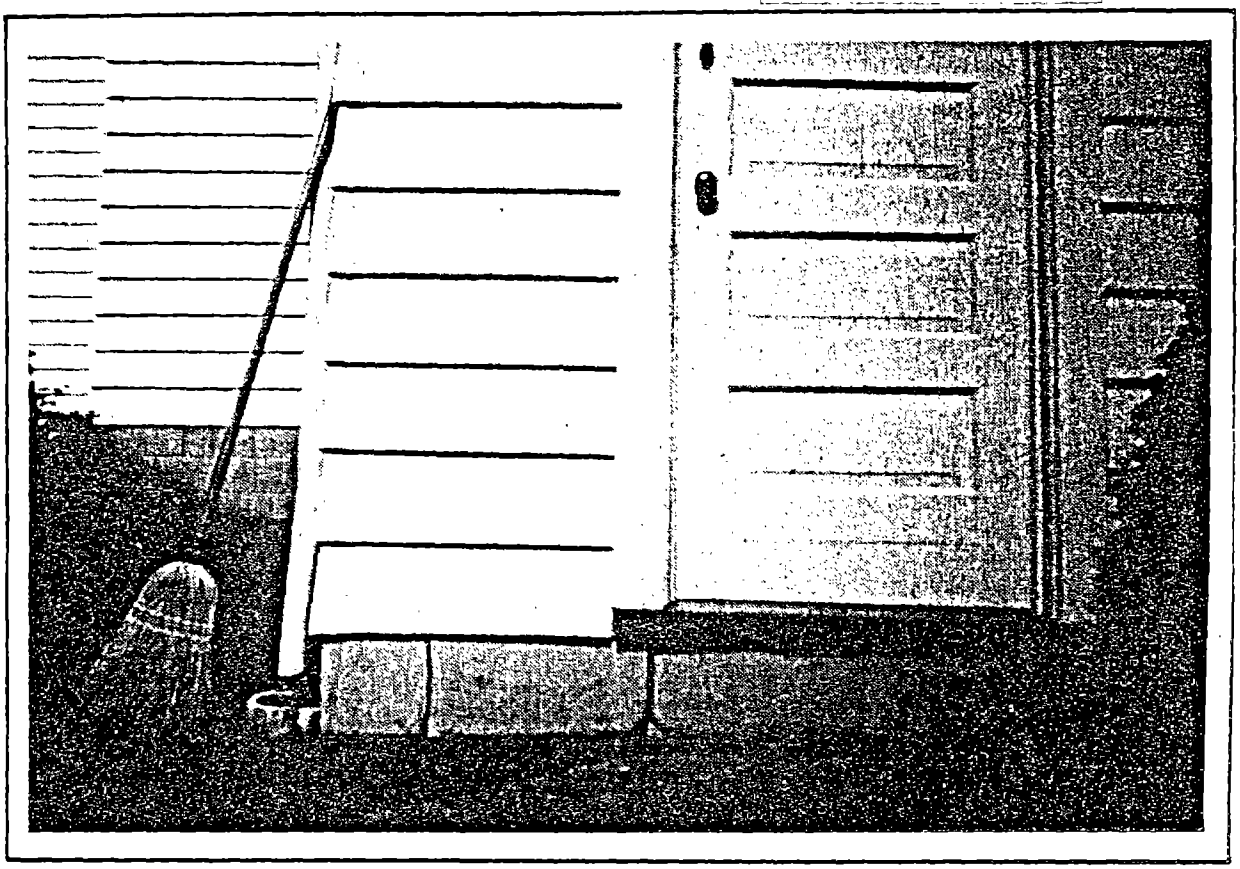


FIGURE 2.18 - BUILDING DAMAGES CAUSED BY FROST HEAVE
A. CRACKED FOUNDATION OF GARAGE

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them with soils that are not susceptible. Frost penetration maps and local building codes provide guidelines for frost penetration depth at specific locations.

Although frost action is a natural phenomenon, human activities can be responsible for its occurrence. Drain pipes and downspouts, for example, are sometimes allowed to discharge into the soil beneath a foundation, providing a source of water that otherwise might not exist. Unusual frost penetration problems have been encountered beneath cold storage areas, ice skating arenas and other refrigerated areas beneath which freezing temperatures may extend far below the normal frost line. Cooling and Ward (1948) report floor heaves up to 8 inches at a cold store in England due to inadequate insulation (Figure 2.19). The floor of a refrigerator storage shed investigated by Tschebotarioff (1973) heaved 18 inches due to the formation of ice lenses to a depth of 23 inches beneath the center of the building. Thorson and Braun (1975) report heaves of seven to 11 inches for three ice arenas that operate year around in the Minneapolis-St. Paul area.

Ground surface frost heaves of 6 to 18 inches in unoccupied and undeveloped ground areas are reported by various sources. Observed upward movements of structures affected by frost heave range from less than one inch to as much as 12 inches (Feld, 1968). Many structural elements reportedly affected by frost action, such as floor slabs and residential dwellings are relatively light in weight. However, even bridge piers and piers supporting building columns have occasionally heaved substantial amounts.

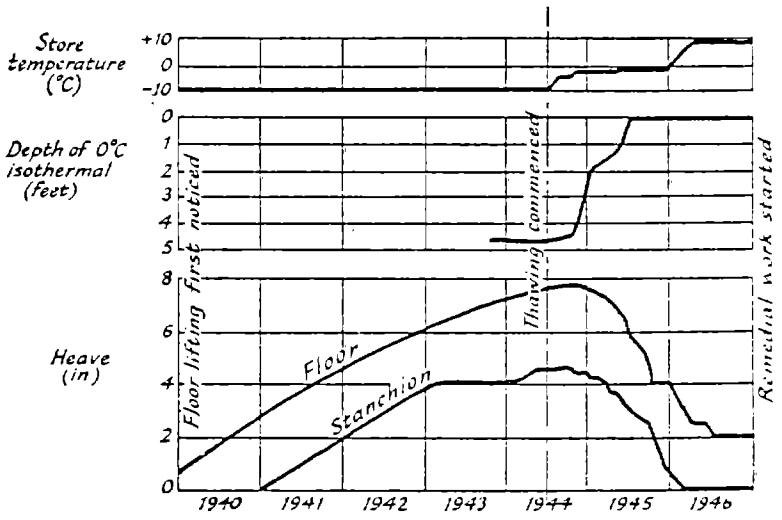
Ground movements caused by either frost heave or thaw could be confused with subsidence and other sources of ground movements, particularly if the frost action is nonuniform beneath a structure, as shown in Figure 2.20. A positive way to differentiate such patterns of ground movement is by reference to a fixed point, structure, or object that has not been affected.

Frost action is fundamentally a vertically directed volume change process, but includes distortion because it occurs in an erratic fashion. Occasionally, structures are seasonally lifted and lowered in a uniform manner and do not sustain damage.

Permafrost or permanently frozen ground, is a different phenomenon from frost action, and is not discussed in this report because it does not occur in the regions of the United States covered in this study. A fairly recent comprehensive review of all aspects of frozen ground, including permafrost, is found in the work of Corte (1969).

Prediction of Movements due to Frost Action

The prediction of ground movement due to frost action is generally not attempted. Where frost action is expected, and there are few locations where it is not, the standard practice is to avoid its effects by placing foundations below the expected depth of frost penetration.



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FIGURE 2.19 - FLOOR HEAVE AT A COLD STORE AS A RESULT OF FROST HEAVE (Cooling and Ward, 1948)

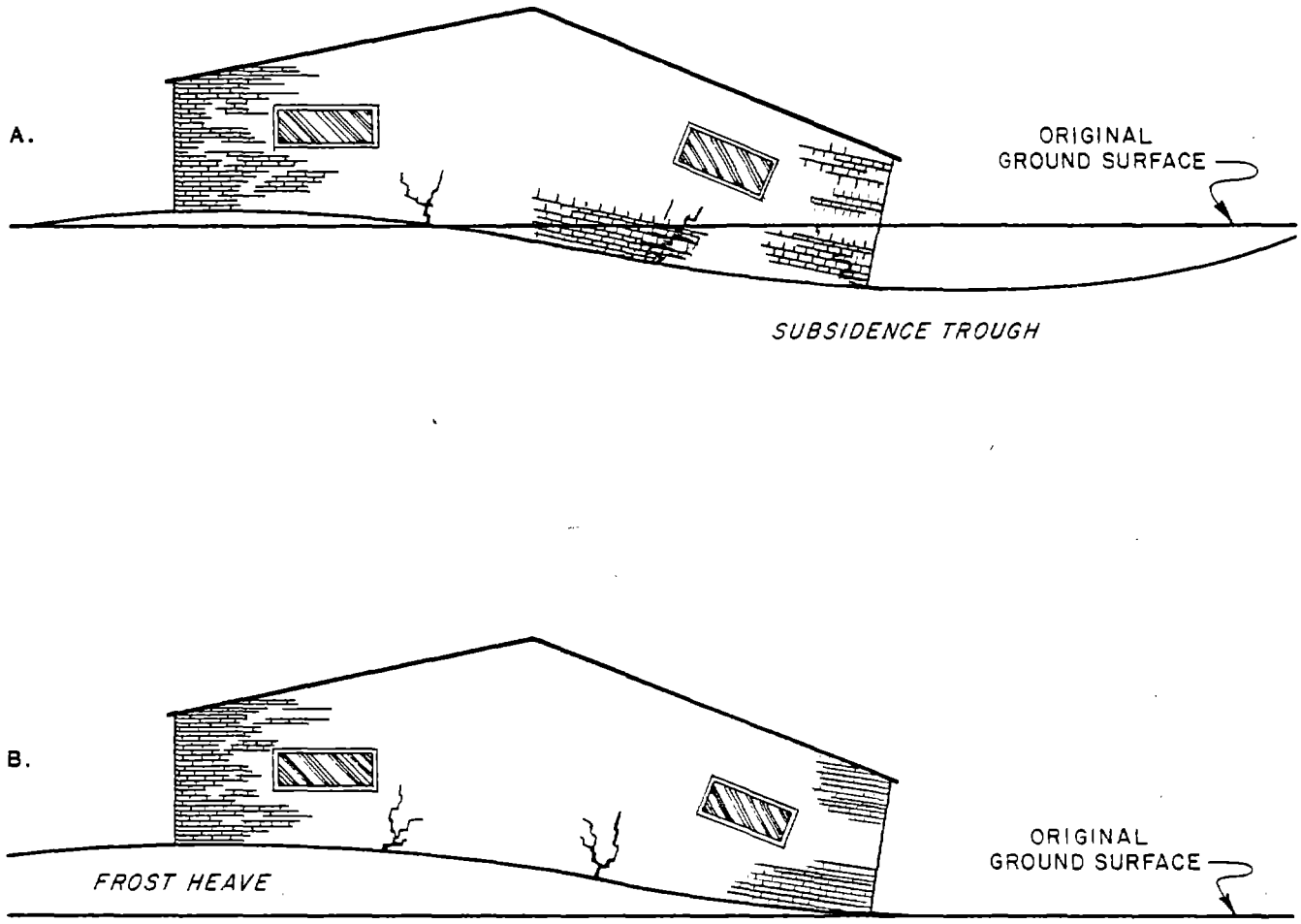


FIGURE 2.20 - IDENTICAL DIFFERENTIAL GROUND MOVEMENTS BENEATH A BUILDING
A. SUBSIDENCE B. FROST HEAVE

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The amount of uplift due to the formation of ice lenses depends on the grain size distribution of the soil, the availability of water, the depth and duration of frost penetration and the variations of these factors beneath the building foundations. Because these factors are not likely to be known and because the mechanics of ice lens formation is not fully understood, the prediction of movements is not within the realm of present day foundation engineering. Terzaghi and Peck (1967) point out that, for an open system in which the soil grain size is optimal (silty soils) and unlimited ground water is available for the growth of ice lenses, the potential thickness of ice lense that can form is theoretically unlimited.

COLLAPSE

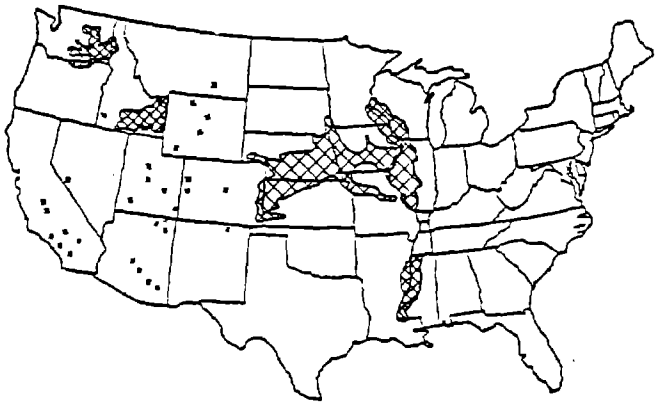
A few special types of natural soil, because of their physical and chemical structure, exhibit the unusual character of collapsing upon wetting. Such soils include loess (wind-deposited silt); clayey loose sands in which the clay serves merely as a binder; loose sands cemented by soluble salts, and certain residual soils, such as those derived from granites under conditions of tropical weathering (Peck, Hanson, and Thornburn, 1974).

The single common characteristic of collapsible soils is the presence of appreciable void space in the soil structure. In the dry state these soils may possess substantial strength. Upon wetting, their strength is drastically reduced, resulting in collapse. The process is sometimes called hydrocompaction.

Plate load tests conducted by the Bureau of Reclamation on loess in Nebraska show that the loess in that area had a bearing capacity in excess of 5 tons/square foot at the natural moisture content, but failed under a load of 0.8 tons/square foot in a wetted condition (Holtz and Gibbs, 1952). The collapsing phenomenon is actually a form of bearing capacity failure, but is treated separately from ordinary bearing capacity failures because it is likely to be manifested under different circumstances.

The water that causes collapse would normally originate from a variety of human activities: irrigation, even on the small scale of lawn watering; concentration of ground surface runoff; discharge from roof drains and downspouts; leaks in drains, sewers, swimming pools and reservoirs; broken water pipes; and discharge from septic fields. Lofgren (1969) reports that many of the loess deposits of the United States do not compact (collapse) when wetted unless a surcharge load has been applied. On the other hand, low-density alluvial deposits of the San Joaquin Valley, California, collapse under their own weight when water is applied.

The distribution of reported cases of collapsible soils in the United States, as determined in a review by Dudley (1970), is shown in Figure 2.21. The behavior of loess as a foundation material is of particular significance because of its widespread occurrence in the Interior Coal Province. Loose sands and silts, which have been



LEGEND

MAJOR LOESS DEPOSITS

x REPORTS OF COLLAPSE IN OTHER TYPE DEPOSITS

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FIGURE 2.21 - DISTRIBUTION OF COLLAPSIBLE SOILS IN THE UNITED STATES (after Dudley,1970)

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cemented by clay or soluble compounds, such as calcium carbonate, gypsum or ferrous iron, are most likely to be encountered in the more arid and semiarid regions.

In the central and western United States collapsible loess soils have been responsible for damage to all types of structures, ranging from houses to irrigation canals and grain elevators. The nature of ground movements associated with wetted collapsed loess depends to a large degree on the extent of wetting. Where large areas are wetted, such as with irrigation, the ground movements are likely to be regional in character, possibly resembling those of regional subsidence. Where wetting occurs in one or two concentrated areas, such as around a broken water pipe, the resulting ground movements are likely to be concentrated around the wetted area in a discontinuous nature, much the same as sinkhole formation above shallow, caved mines.

Private residence damage in the Denver suburbs caused by collapse of wetted loess foundation soils is well documented (Clevenger, 1956). Movements on the order of 5 inches are apparently common in that locale. The reported cases exhibit features similar to a classic, shallow mine subsidence case: broken utility connections, distorted door frames, cracked walls and ceilings, etc. Holtz and Gibbs (1952) report settlements of up to 3 feet in a pumping plant discharge works in Montana due to wetting of loess subgrade soils. Peck and Peck (1948a) report collapse settlements from one to 15 inches for a steam locomotive maintenance building founded on loess. The movements which occurred in an area that previously had supported heavy structures without detrimental settlement, took place very rapidly after the subsoils became saturated with locomotive wash water.

Collapse upon wetting is not limited to natural undisturbed soils. Some poorly placed artificial fills are subject to the phenomenon. Charles, Naismith, and Burford (1977) report ground surface movements as large as 0.5 meter when the backfill in open cast mines at Horsley, England, became saturated by a rising water table. The backfill consisted of loose unsaturated cohesionless sandstone and mudstone material. Where the backfill had been preloaded by an overburden heap, the collapse movements after saturation were significantly reduced, but not eliminated. Terzaghi (1960) and others have postulated that the settlements of saturated rock fills may be caused by softening and crushing of highly stressed contacts between rockfill particles due to weakening of the rock material upon wetting.

In view of the extensive strip mining that occurs in the Eastern and Interior Coal Provinces, it would seem likely that ground movements due to the collapse phenomenon are rather common, particularly in recently stripped areas where laws require backfilling to approximate the original ground contour.

The potential for significant collapse settlements is not limited to cohesionless backfills. Compressions of 10.9 percent

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were measured by Penman and Godwin (1974) upon saturation of one meter diameter laboratory samples of boulder clay taken from a strip mine in Corby, England. Their observations of experimental backfill indicate an increasing settlement rate following a rise in water level within the fill.

A special type of clay soil, referred to as quick clay, is sometimes said to have a collapsible structure. The mechanism of collapse in these soils does not involve a volume change, but rather a disturbance of their highly sensitive structure. Quick clays occur only in a few areas of the world and are not in the Eastern or Interior Coal Provinces.

Prediction of Collapse Movements

Jennings and Knight (1957) propose the use of double oedometer tests to predict movements due to wetting and collapse. One oedometer (consolidation) test is conducted on a soil sample in its natural moisture condition and another on a sample of the same soil in a saturated state. The results from both tests are then adjusted to reflect the amount of consolidation due to the wetting. Jennings and Knight have found this method furnishes results in agreement with observed movements on a structure located above cemented sand in South Africa.

Peck, Hanson and Thornburn (1974) point out several difficulties with this method, and suggest that values of movement so computed are likely to be larger than the actual movements because of the time required for the in-place soils to reach an equilibrium water content.

On-site field load tests conducted in pits that are wetted while the test plate is under load is another more elaborate means of predicting collapse movements. The degree and extent of wetting significantly influence the test results. In order to conduct a meaningful test, knowledge is required of the ultimate percent change and distribution of moisture content, and the rate of change of moisture content.

Available predictive methods should be regarded as semi-rational and of only fair reliability. Dudley (1970) suggests that a correction factor based on local experience be applied to any computed collapse estimates in order to arrive at reasonably realistic values. Such correction factors would typically range from 0.5 to 4.

SHRINKING AND SWELLING

General Considerations. According to one source, shrinking and swelling soils in the United States inflict at least $2.3 billion annually in damages to homes, roads, pipelines, and other structures (Jones and Holtz, 1973). Ground movements caused by such materials are much more common than is appreciated even by many foundation engineers. Jones and Holtz claim that of the 250,000

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new homes built on expansive soils each year, 60 percent will experience minor damage and 10 percent will experience significant damage--some beyond repair. Shrinking and swelling tend to occur in the same types of soils and are most commonly associated with changes in moisture content. Because the two processes occur under different conditions, they are treated separately in the following paragraphs.

Shrinkage. Shrinkage is caused by capillary tensions that develop in the porewater around individual soil particles as the moisture content decreases during drying. The resultant compressive stresses that develop between soil particles may be as high as several thousand pounds per square foot and may cause shrinkage in both the horizontal and vertical directions. Ground movements are likely to include both vertical and lateral components (Tomlinson, 1969). Sowers and Sowers (1970) report that shrinkage cracks one foot wide and as deep as 15 feet have been observed in highly compressible clays. Peck, Hanson, and Thornburn (1974) state that soil shrinkage problems are associated with the presence of soils having moderate to high swelling capacity in more humid regions.

Causes of drying or dessication that can lead to soil shrinkage are several-fold: seasonal moisture changes, especially extended dry spells; protective ground covering, such as buildings, that promote differential drying of the ground surface; removal of soil moisture by tree root systems; and artificial heating of the subsurface, as might occur beneath a boiler, furnace, or kiln.

Cooling and Ward (1948) measured vertical ground surface movements as large as 1.1 inches that were caused by shrinkage of a stiff fissured clay in southeast England due to seasonal changes in soil moisture content (Figure 2.22). Ward (1953) found that cyclic seasonal movements of almost 2.5 inches take place beneath open grassland. The movements decreased with depth, but were measureable even at a depth of four feet. The researchers point out that damages occur either when parts of a structure are founded at different levels, or when changes in soil moisture vary around the building. It appears that movements of bare ground are somewhat less than those that are vegetated.

Mr. Richard Driscoll of the United Kingdom's Building Research Station indicated in personal communication that the 1976 drought in England, the worst in 250 years, resulted in an unprecedented one hundred million pounds ($225 million) in claims to the insurance industry for home damage due to ground movements. The average claim was $10,000. Damage was primarily architectural, taking the form of vertical or stair step cracks in walls, but was enough to make many homes unacceptable for bank financing and therefore unsellable. To combat the problem, many people underpinned their homes with short drilled piers, which are typically 14 inches in diameter and 8 to 10 feet deep. Driscoll stated that differentiating architectural damage due to clay shrinkage from that due to mining is difficult, if not impossible, in many cases.

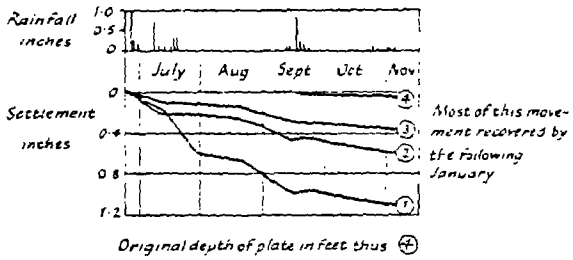


FIGURE 2.22 - SETTLEMENT CAUSED BY SOIL SHRINKAGE DUE TO SEASONAL CHANGES IN SOIL MOISTURE CONTENT (after Cooling and Ward, 1948)

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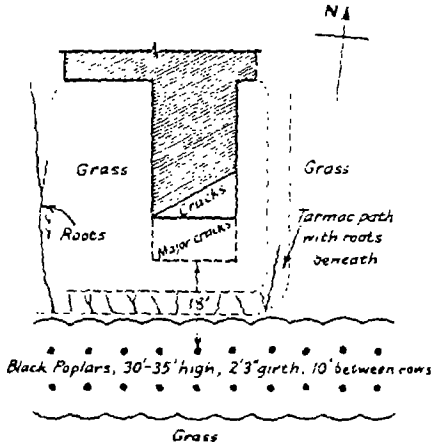


FIGURE 2.23 - SETTLEMENT DAMAGE TO BUILDING DUE TO SOI SHRINKAGE CAUSED BY TREE ROOTS
(after Ward, 1947)

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Hammer and Thompson (1966) report extensive damage to two-story brick structures (differential vertical movements up to four inches and rotations of basement walls) that were attributed to the presence of large elm and poplar trees growing within 15 feet of the buildings. The ground subsidence patterns in the vicinity of the trees and damaged buildings might be difficult to distinguish from those caused by minor subsidence on the basis of appearance alone. Usually the damage caused by moisture removal through tree roots occurs in the dry periods of late summer and fall when transpiration is greatest. Extended dry spells of two to three years tend to worsen the problem and cause damage to buildings that experienced no distress in the preceding several decades. The effect can be felt to the maximum depth of tree root penetration. For the cases in Michigan reported by Hammer and Thompson, tree roots responsible for damage were found at depths of 10 feet.

Ward (1942) reports damage to a building near London bounded on one side by poplar trees (Figure 2.23). Cooling and Ward (1948) report four-inch differential movements where the responsible trees were 80 feet from the damaged building. Legget (1973) cites cases where tree roots penetrate to depths of 30 meters below ground level and up to 30 meters horizontal distance from the tree itself. Some trees use more water than others and consequently present greater hazards. According to Legget, the worst offenders are poplar, alder, aspen, willow and elm, in that order. Professor R. Yarbrough of Southern Illinois University reports in personal communication that the central United States, water-loving floodplain trees, such as pin oak, silver maple and cottonwood, can also contribute to the root penetration problem. Building damage caused by tree roots can be avoided by simply prohibiting trees within a specified distance from the foundation.

Cooling and Ward (1948) report significant vertical shrinkage movements beneath a group of three boilers. The maximum settlement of the 40-foot square foundation slab was six inches at the center and three inches at the edges, causing tilting of the boilers and leakage of the pipes. The moisture content of the clay subsoils, measured after movement had occurred, indicated drying to a depth of five to six feet below the foundations. Two and one-half months after the boilers had been shut down, the temperature at a depth of 15 feet was still 90°F, compared to a norm of about 50°F. In semi-arid regions, such as western Texas, dessication of clays occurs to depths of 20 feet. The shrinkage cracks may later fill with water causing swelling.

Swelling. Swelling that accompanies an increase in soil moisture content is apparently more complex than shrinkage. It may be due to several mechanisms: the elastic rebound of soil grains; the attraction of clay minerals for water; the electrical repulsion of clay particles from one another and their adsorbed cations; and the expansion of air trapped in soil voids (Sowers and Sowers, 1970). Other mechanisms related to osmotic phenomena have been suggested (Gromko, 1974).

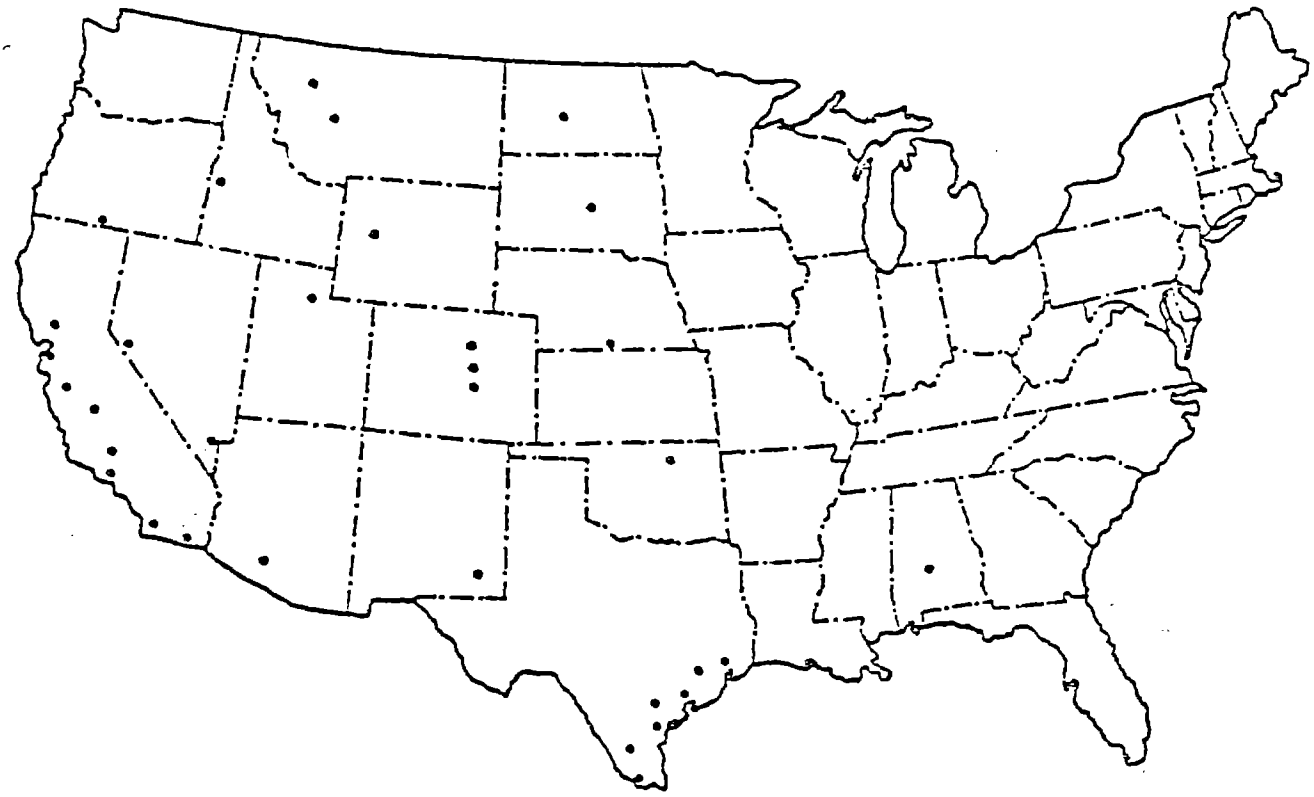
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If soils having a high swelling potential are unrestrained, they may undergo significant volume change--possibly as great as 50 percent--when given free access to water. If such a soil is confined and swelling restrained, the stress required to prevent swelling is called the swelling pressure. Swelling pressures occasionally exceed 20 tons/square foot. Because most building foundation loads are only moderate, uplift from swelling soils is often possible.

Not all soils exhibit swelling potential. The swelling potential of a soil is roughly indicated by its Plasticity Index, which is measure of the range in moisture content over which a soil behaves plastically, or deforms without cracking. If the plasticity index is less than 15, the swelling potential is low. A plasticity index equal to or greater than 35 normally indicates a very high swelling potential. Other more complex criteria for estimating swelling potential have been suggested by Gromko (1974). Murphy (1975) claims that almost any clay, regardless of origin, is capable of undergoing significant volume changes, particularly those with high colloidal contents and high percentages of montmorillonite.

Engineered structures resting on swelling soils will be subject to swelling only if the soil moisture content at the time of construction is less than the equilibrium moisture content that will be reached some time after construction is completed. The availability of moisture during the life of a structure, as well as the present load and the maximum previous load to which a soil has been subjected influence the degree of swelling that will actually take place.

Swelling soils are common in the United States, particularly in a belt extending from Texas northward through Oklahoma and into the Upper Missouri River Valley. According to Jones and Holtz (1973), expansive or swelling soils are found in every state and cover one-fourth of the United States. An indication of the extent of reported structural damage caused by soil heave in the United States as of 1974 is shown in Figure 2.24. No concentrations of reported damages occur in the Eastern and Interior Coal Provinces. Murphy (1975) notes that "the most distinctive characteristic of swelling soils is their close association with climatic extremity." In view of this observation, it might be expected that unusual changes in climate could precipitate swelling problems in areas that previously had not experienced such difficulties. Sowers and Kennedy (1967) report several cases of damage due to swelling in the Southeastern Coastal Plain area of the United States (Georgia, Florida, South Carolina, and Mississippi). Swelling occurred where foundations of structures were constructed either during an extended drought or after soils had been exposed during a hot summer dry period. Subsequent wetting during rainy seasons caused swelling. In one case, upward ground movements were at least four inches.



• REPORTS OF STRUCTURAL DAMAGE DUE TO HEAVE

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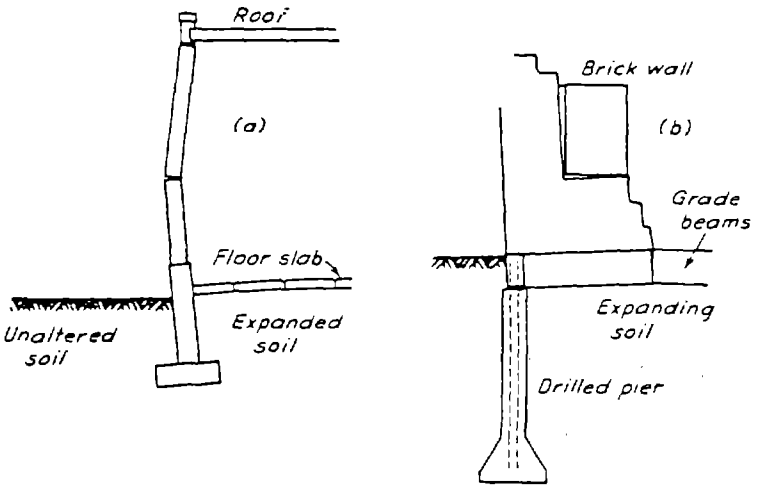
FIGURE 2.24 - REPORTED STRUCTURAL DAMAGE CAUSED BY HEAVE OF SWELLING SOILS (after Murphy, 1975: Gromko, 1974)

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Typical examples of building damage caused by swelling soils are shown in Figure 2.25. Heave normally occurs in dry climates when soil moisture from the ground outside the building migrates to the interior where the ground is shielded from evaporation. Terzaghi and Peck (1967) state that heave movements related to this process may become greater than one foot under unfavorable climatic and soil conditions.

Most reported damage due to swelling ground movements concerns residential buildings where foundation loads are relatively low, although heavier structures are occasionally affected. Cheney and Burford (1974) report vertical heave on the order of 20 mm (0.7 inch) for a three-story office building in the United Kingdom founded on swelling clay. The swell was related to an increase of moisture content in the clay that occurred after the building had been constructed on a site previously covered by trees. Over a long period of time the trees had apparently caused drying and dessication of the soils. Removal of the trees allowed soil moisture to increase to a higher level, a process which was accompanied by swelling. A common recommendation in the United States is to allow cleared land to stand one year before building so that soil moisture can equalize. The sources of water available to swelling soils are as numerous as those discussed for collapsing soils. Broken water pipes, surface runoff concentration from roofs and paved areas, and leaking sewers are the most common sources (Godfrey, 1978).

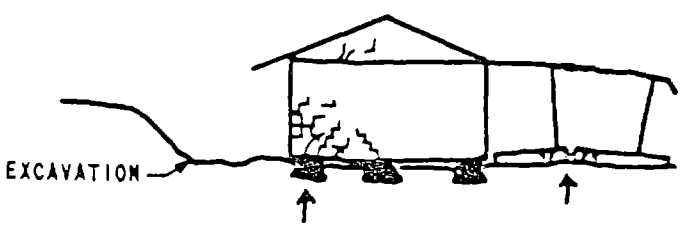
Ground movements caused by swelling materials are not limited to natural soils, nor are volume changes always related to changes in moisture content. Certain rocks and compacted fills may exhibit volume change characteristics. Gardner and Salver (1977) describe two foundation swelling problems encountered in southwestern Pennsylvania. The first problem occurs in shales that contain the mineral pyrite. Oxidation of pyrite may occur after shale is exposed to air and water in a foundation excavation and results in the formation of new minerals of greater volume than the original pyrite. Expansion or swelling may occur very slowly and non-uniformly and continue long after a structure is completed (Dougherty and Barsotti, 1972). The second foundation problem is associated with certain steel mill slags, which are by-products of the iron and steel-making processes. These materials are frequently used as substitutes for sand and gravel beneath foundations and behind retaining walls. Some, such as blast furnace slag, are inert. Others, such as open-hearth, electric steel furnace and Basin Oxygen Furnace (BOF) slag, may contain free calcium and magnesium oxides that combine readily with other substances and expand (Crawford and Burn, 1969). Examples of damages caused by heave of expansive shales and steel mill slag are shown in Figure 2.26. Feld (1968) describes a case in Cleveland in which a building and its floor slab were founded directly on a pyritic shale. Heaving of the structure had been going on since 1920, raising the floor slab one foot and the small footings four inches.



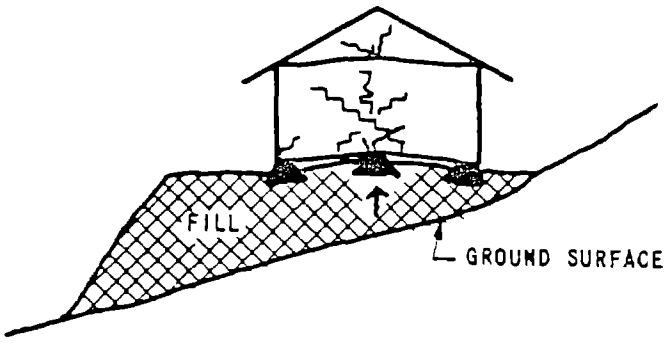
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FIGURE 2.25 - TYPICAL BUILDING DAMAGE CAUSED BY SWELLING SOIL. A., FOOTING SUPPORTED STRUCTURE WITH SLAB-ON-GRADE (after Parcher and Means, 1968). B., PIER SUPPORTED STRUCTURE WITH GRADE BEAM IN CONTACT WITH SWELLING SOIL (after Peck, Hanson, and Thornburn, 1974)

EXPANSIVE SHALE

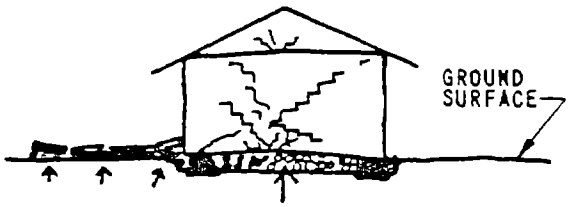


FOUNDATIONS AND SLABS ON RECENTLY EXPOSED EXPANSIVE SHALE

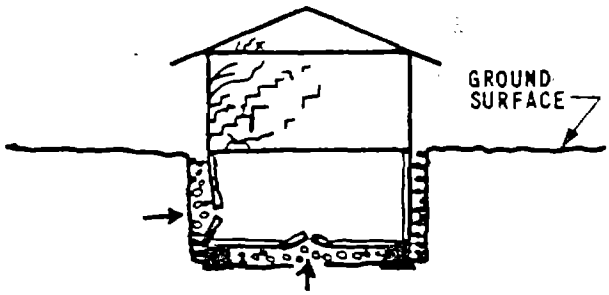


FOUNDATIONS AND SLABS PLACED ON FILL CONTAINING LARGE AMOUNTS OF EXPANSIVE SHALE

EXPANSIVE SLAG



FOUNDATIONS AND SLABS PLACED ON EXPANSIVE SLAG



WALLS BACKFILLED WITH EXPANSIVE SLAG

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FIGURE 2.26 - DAMAGE CAUSED BY HEAVE OF EXPANSIVE SHALES AND SLAG (after Gardner and Salver, 1977).

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Free gypsum and anhydrite are two other minerals that may occur in shale and, like pyrite, cause swelling and expansion due to chemical changes. Both shrinking and swelling are fundamentally volume change processes. Both processes may lead to uneven lateral and vertical ground movements that may resemble movement from other sources, such as frost heave, consolidation settlement, or subsidence.

Prediction of Movements due to Shrinking and Swelling

There are currently no reliable analytical or empirical methods to predict shrinkage movements. The difficulties lie in: 1) identifying the stress-time-moisture relationships for the in-place soils; and 2) conducting a laboratory test that reproduces field conditions. Sowers and Sowers (1970) suggest that shrinkage can be estimated by drying a soil sample and computing the relationship between saturated water content and soil volume. The maximum possible shrinkage of a soil mass is equivalent to the change in volume that occurs as the water content is reduced from the maximum field water content to the shrinkage limit. The shrinkage limit is the water content at which drying ceases to cause a decrease in volume and beyond which the soil is no longer saturated. Estimates of shrinkage ground movements are crude at best.

Oedometer tests are used by some engineers to predict the swelling potential of soil (Murphy, 1975, Peck, Hanson and Thornburn, 1974; Gromko, 1974), but are subject to interpretation problems similar to those for collapsing soils.

Other methods of predicting swell are based on Atterberg limit and clay content determinations (Gromko, 1974 and Murphy, 1975). They have apparently enjoyed some success in the geographic areas where developed. Sowers and Kennedy (1967) propose a "water plasticity ratio" for estimating swelling pressures and percent expansion of clays in the Southeastern Coastal Plain area.

Fuller (1979) claims that the Plasticity Index has not proven reliable in predicting potential volume changes of soils.

The major difficulty in predicting ground movements associated with shrinking and swelling soils lies in defining the actual moisture changes that occur with time in the soil beneath a foundation. Generally foundation engineers must be satisfied with recognizing potential shrinking or swelling soils at a specific site so that safeguards can be taken.

FLUID EXTRACTION

General Considerations. The discussion of erosion indicated how the flow of water can carry away soil materials and cause dramatic ground surface movements. The present section discusses ground surface movements caused by the removal of fluids alone. The term 'fluid' refers both to liquids and gases.

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The mechanism by which fluid extraction causes ground surface movement is quite simple in concept. It is well described by Scott (1978):

> "In a geological layer having some pore space saturated with fluid, the total overburden stress is partitioned between the solid structure and the fluid. When through some process of pumping or lateral drainage, the pressure of the fluid is reduced, its portion of the total, unchanged overburden pressure must be taken up by the solid portion of the formation. The consequence of this is deformation in the solid structure as it adjusts to the new stress conditions. In particular, a volume decrease is induced."

This is a classic application of Terzaghi's concept of effective stress, the effective stress being the difference between total overburden stress and the pore fluid pressure. The consequences of the volume decrease associated with the increased effective stress are described by Scott:

> "Depending on the extent of fluid pressure reduction and the geometry and properties of the region in which pressure is reduced, very complex effects can be developed in the surrounding material. These can include distortions of wells or well casing and subsidence and lateral movements of overlying ground. If the fluid-bearing formation is sufficiently compressible and near the ground surface, very large surface effects can be observed."

This describes the fundamental process that accompanies all the following activities:

1. Pumping of ground water from wells for water supply purposes;

2. Pumping of ground water from wells or sumps for construction dewatering purposes:

3. Construction of tunnels and open excavations through water-bearing soil or rock;

4. Withdrawal of oil, gas, steam and brines from the ground.

The common feature of all of these activities is that they reduce pressures in the pore fluid contained in the soil or rock. In any of these cases it is possible that the removal of fluid would also be accompanied by subsurface erosion, the consequences of which have already been discussed.

Scott (1978) claims that fluid extraction is responsible for the largest and most spectacular instances of ground surface subsidence caused by human activity. Abundant examples of ground surface subsidence attributable to the activities listed in the previous paragraphs can be found in the geologic and civil engineering literature. They are of particular concern for three reasons:

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1. Subsidence patterns associated with fluid extraction sometimes resemble those associated with solid mineral extraction, and might be confused with them;

2. Subsidence due to fluid extraction is fairly common. It can occur over a wide range of geologic conditions, and may occur in the same regions as mining subsidence;

3. Mining itself could cause drainage of overburden soil and rock materials and lead to significant ground surface movements, quite apart from those due to collapse or yield of overburden into the mine openings.

Case Records. No attempt will be made here to cite all records of ground movement due to fluid extraction. It is interesting to note, however, that the first descriptions of subsidence due to fluid extraction were published in 1925 for the Goose Creek Oil field in Texas (Poland and Davis, 1969). The first reported observations of subsidence due to groundwater withdrawal--in the Santa Clara Valley in California--were published in 1933. Most of the more serious problems caused by fluid extraction have occurred since World War II, and it is anticipated that the frequency of incidence will continue to increase, most notably in connection with the development of ground water resources.

Some of the reported cases of subsidence caused by pumping water and oil/gas for resource development are summarized in Table 2.2. Several points should be made in regard to these and other reported cases of subsidence caused by fluid extraction for resource development:

1. Fluid extraction for resource development tends to cause subsidence over large areas, measured in square miles to tens of square miles. The regional nature of the problem in the Houston area is illustrated in Figure 2.27.

2. Settlements are often large, measured in feet to tens of feet.

3. In some instances, different fluids have been extracted from more than one horizon, a factor which complicates the interpretation of subsidence. This is the case in the Houston area.

4. The seat of subsidence--the geologic interval in which consolidation occurs--is not always clearly established. The great depths of fluid extraction and the existence of more than one compressible stratum frequently preclude pin-pointing the location where subsidence is emanating.

5. Fluid extraction does not always cause recognizable subsidence. Pumping from a confined aquifer in Memphis, Tennessee, since 1886 had, as of 1969, caused only negligible subsidence (Poland and Davis, 1969).

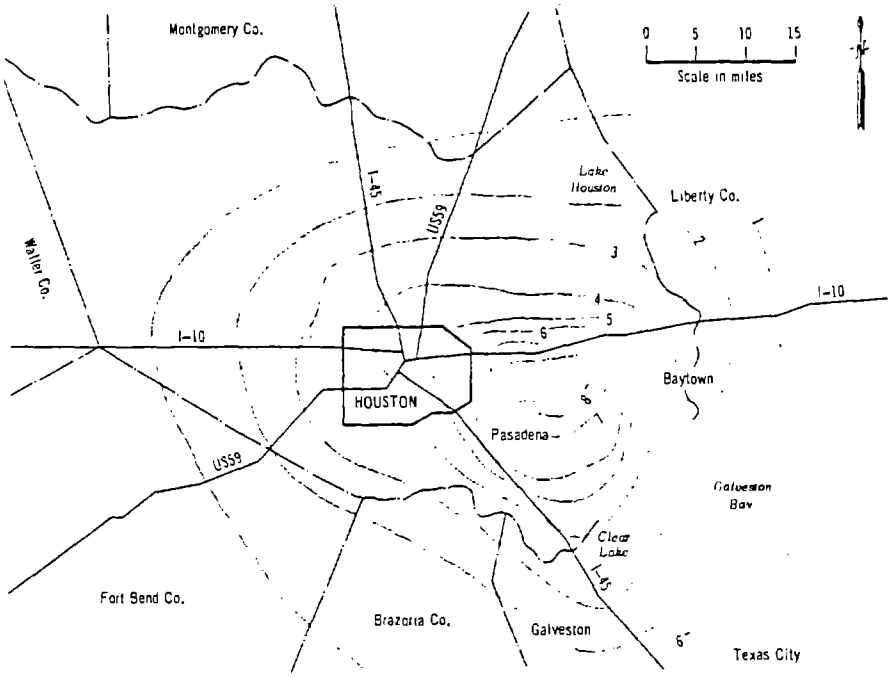
Table 2.2

SOME OBSERVED GROUND MOVEMENTS (SUBSIDENCE) CAUSED BY FLUID EXTRACTION FOR RESOURCE DEVELOPMENT

Location	Fluid Extracted	Area Affected	Maximum Subsidence	Depth of Extraction	Materials in which Consolidation Occurs	Comments
Goose Creek, Texas[1]	Oil/Gas, Water/Sand	1.4 mi x 2.5 mi	3 ft +	700-4500 ft	Unconsol. Sand and Clay	Normal Faulting-Displacement 3 to 16 in.
Wilmington, California[1]	Oil/Gas, Water	≅ 25 sq mi	28 ft	2500-6000 ft	Siltstones, Shales, or Sandstones (?)	Tectonic movement also possible.
Lake Maracaibo, Venezuela[1]	Oil	?	11 ft	?	Unconsol. Sands, Clays, and Silts	
Niigata, Japan[2]	Gas/Water	200 sq km	2.5 m	800-1000 m	Unconsol. Sands, Sandstone, Conglomerate Clay	
Po Delta, Italy[1]	Gas/Water	20 km x 40 km	J 6 ft	?	Unconsol. Deltaic Sand and Clay	
Santa Clara Valley, California[1,3]	Water	250 sq mi	13 ft	200-1000 ft	Unconsol. Alluvial and Bay Deposits: Clay and Silt	
Mexico City Mexico[1,4]	Water	?	9 m	50-500 m	Clay	Most of subsidence due to consol. of top 100 m of soil.
Houston-Galveston Texas[1,5,6]	Water/ Oil/Gas	7200 sq mi	9 ft	Shallow to 2500 m	Clay	240 km faults activated by fluid extraction.
Savannah, Georgia[1,7]	Water	330 km^2	200 mm	50-150 m	Clay, Marl, Silt, or Limestone (?)	
Denver, Colorado[1]	Water	?	1.25 ft	?	?	

Note: Superscripts refer to source of data:

[1] Poland and Davis (1969)
[2] Aoki (1976)
[3] Poland (1976)
[4] Vega (1976)
[5] Kreitler (1976)
[6] Spencer (1977)
[7] Davis and Counts (1976)



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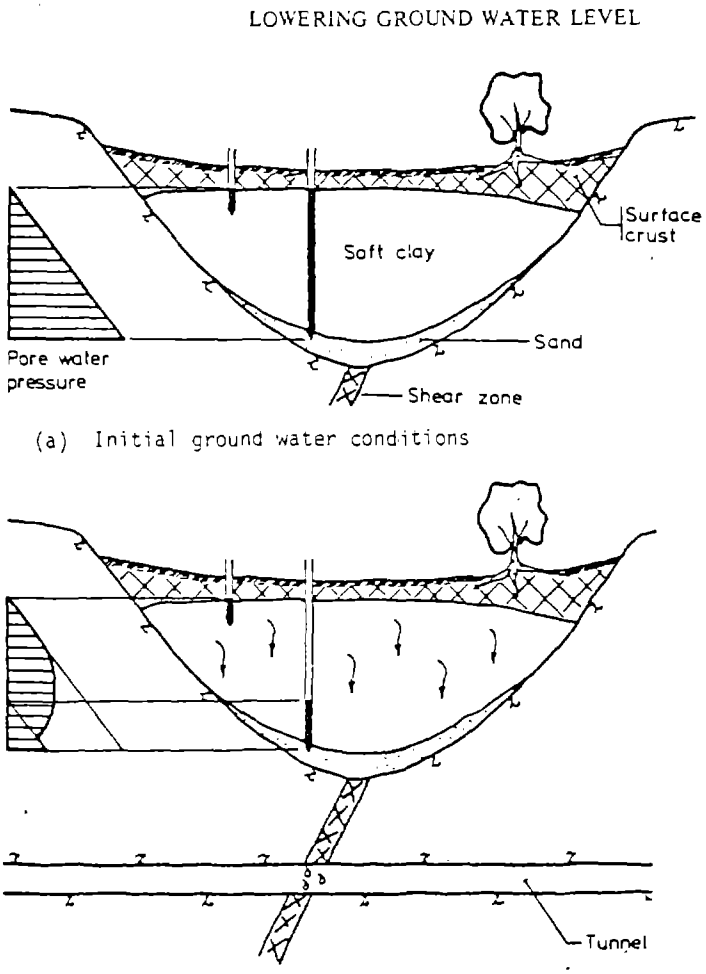
FIGURE 2.27 - SUBSIDENCE CONTOURS (IN FEET) FOR HOUSTON, TEXAS AREA. SUBSIDENCE IS CAUSED BY EXTRACTION OF BOTH WATER AND HYDROCARBONS (after Spencer, 1977).

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Although the subsidence caused by fluid extraction has created significant engineering problems for cities, such as flooding, reports of structural building damage are not common. Many buildings--in fact whole cities--have been significantly lowered without causing widespread structural damage. The most serious threats to building damage are the sharp differential movements associated with local faulting, such as occur in the Houston-Galveston area. Also of concern are the horizontal strains that develop in subsidence depressions. In Long Beach, California, compressive strains over the subsiding Wilmington oil field have caused buckling of railroad tracks and pipelines and structural damage to bridges (Poland and Davis, 1969). Legget (1962) reports that a 6,000-foot horizontal base line was shortened by five feet.

Local ground movements caused by pumping from one or two wells or by lowering the ground water level in connection with construction operations are more likely to inflict building damage than is the regional subsidence caused by fluid extraction. The former are more likely to cause distortional building movements. Feld (1968) reports numerous incidents of building damage caused by local ground water lowering. Several interior columns in a 12-story concrete building in Brooklyn, New York, had to be underpinned after they settled excessively when ground water was pumped for industrial use. No figures for amount of ground surface movement are given. Long term pumping for the dewatering of a subaqueous tunnel in New York caused the overlying streets to settle nine inches and affected structures facing the street. D'Appolonia (1971) reports that dewatering of sand and gravel layers by well points in open excavations on the Massachusetts Institute of Technology Campus has caused minor settlement (0.1 foot) and cracking of many buildings. Settlement has also been reported in Stockholm, Sweden, and Oslo, Norway, where construction of rock tunnels beneath deposits of soft clay caused lowering of piezometric levels and subsequent consolidation of the clays (Broms, 1978; Karlsrud and Sander, 1978; Morfeldt, 1970). A simplified sketch of the problem is shown in Figure 2.28. Leakage of relatively small quantities of water may cause substantial lowering of the piezometric levels and subsequent consolidation of overlying soft clay. The ground movements do not involve yield of soil into the tunnel, but merely drainage through joints and faults. In Oslo, Norway, where clay-filled depressions in the bedrock are normally separated from the bedrock surface by a thin sandy seam, ground surface movements up to 35 cm (13.8 inches) have occurred at distances of 200 to 400 meters from the tunnels (Karlsrud and Sanders, 1978).

Fluid Extraction Ground Movements. Fluid extraction and consolidation produce distortional ground movements, somewhat similar to those associated with subsidence above undermined areas. Fluid extraction over a broad area tends to produce smooth, gradual distortions, whereas mining tends to cause local, sharper, more pronounced distortions. Horizontal ground strains are apparently important in both cases. In Oslo, for instance, the worst building damages caused by the consolidation of clay-filled depressions



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FIGURE 2.28 - CONSOLIDATION OF CLAY-FILLED DEPRESSIONS IN STOCKHOLM, SWEDEN. CAUSE IS DRAINAGE OF GROUND WATER THROUGH SHEAR ZONE INTO ROCK TUNNEL. A. INITIAL GROUND WATER CONDITIONS B. LOWERED GROUND WATER (after Broms, 1978).

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above rock tunnels occurred in buildings situated along the edges of the depressions. The situation is likely attributable to the high tensile ground strains near the edges.

Prediction of Ground Movements due to Fluid Extraction

Analysis and prediction of ground movements due to fluid extraction would, at first consideration, appear rather simple since the two major factors that influence subsidence are the reduction of the piezometric head in the pumped strata and the compressibility of the strata. Simple one-dimensional consolidation theory may be adequate for analysis, although more complex theories involving two- and three-dimensional consolidation are available. Saxena (1978) reviews these and other subsidence prediction methods that have been used in geothermal, ground water and oil-gas extraction analysis. Some of the more sophisticated methods have been verified for specific locations.

Application of all consolidation and subsidence theories is generally limited by a lack of detailed knowledge of the soil compressibility and the location and degree of continuity of drainage layers. In fact, the seat of consolidation is often not known with certainty. Because of the high costs of evaluating these unknowns, the feasibility of making accurate predictions decreases as the size of the area and the complexity of the subsurface increases.

Christian and Hirschfeld (1974) point out that difficulties in obtaining undisturbed samples of soil or rock, unknown subsurface details and the interpretive problems associated with measuring fluid pressures in wells all serve as a strong warning that predictions of fluid extraction subsidence should be viewed with skepticism. Eight of the estimates for subsidence at Long Beach, where subsidence was caused by pumping in the Wilmington oil fields, were too low. After 22 years the subsidence had reached 28 feet and showed no signs of slowing. It was stopped by injecting water into the field.

The prediction problem is further exemplified by the Norwegian experience with ground surface movements caused by leakage of water into rock tunnels beneath clay-filled bedrock depressions. Karlsrud and Sander (1978) report that fairly reliable estimates of both the magnitude and rate of surface movement can be made using conventional consolidation theory provided the reduction in pore pressure at the bedrock level due to leakage into the tunnel is known. The problem lies in predicting leakage and pore pressure changes that might take place. This problem is one that appears in all fluid extraction problems.

In areas where subsidence has been monitored by field measurements, existing data have been used to forecast future subsidence. This procedure has met with limited success in the Sacramento and San Joaquin Valleys of California where subsidence is caused by ground water withdrawal (Prokopovich, 1978). The method suffers

from numerous complexities, including the questionable stability of benchmarks, the effect of tectonic activity, the variability of ground water withdrawal (both past and future), and the assumptions necessary to extrapolate past subsidence to the future.

Concluding Remarks Concerning Fluid Extraction. The potential for ground movement due to fluid extraction is not limited to any specific geographic area. It is likely to be encountered wherever the subsurface contains compressible soils and fluid extraction or drainage takes place. Although soft clays and loose sands are the most susceptible, stiff soils can also consolidate significantly if the reduction in pore water pressure is sufficiently large. The process normally occurs as a result of a human alteration of the ground water regime. However, any natural process that lowers the ground water level could activate the same mechanisms of movement as fluid extraction.

DENSIFICATION BY VIBRATION

Vibrations from sources such as moving equipment, unbalanced operating machinery, reciprocating engines, pile driving or blasting operations, highway or railroad traffic, or natural phenomena, such as earthquakes, can cause some soils to densify. If the degree of densification is significant and if the soil deposit has appreciable thickness, measurable vertical ground surface movements can result. The important factors in the process are: the frequency and duration of vibrations imposed on the soil; the static load on the soil, and the soil properties at the onset of vibration (Barkan, 1962).

The soils most susceptible to densification are cohesionless such as (sands, silts and gravels), whose structure allows individual soil grains or particles to rearrange into a more compact configuration than exists in the original state. Clays and cohesive soils are not as significantly affected by vibrations because the cohesive bonds between clay particles interfere with the slippage and rearrangement of soil particles. Field and laboratory experiments, as well as records of building damage caused by vibrations, have shown that vibration frequencies in the range of 500 to 2500 impulses per minute cause the most serious densification. The frequency of the unbalanced forces in many types of machinery, such as steam turbines, diesel power units, and air or gas compressors lies within this range (Terzaghi and Peck, 1967).

Soil densification due to vibration is a volume change-type movement. If the soils are uniform in both thickness and physical properties, then ground movements will be uniform and only minimal damage may result to structures founded at ground surface. More commonly, granular soils occur in deposits of nonuniform character. Densification may, therefore, lead to differential movements and a distortion of the ground surface profile.

Because of the common occurrence of cohesionless soils and the wide variety of vibration sources, particularly those brought about

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by mankind, ground movements caused by vibrations are not uncommon. Building damage is frequently involved. Terzaghi and Peck (1967) describe several case histories in which settlements of 6 to 14 inches were experienced as a result of soil densification by pile driving, turbogenerators and train traffic. Severe damages occurred where the movements were unequal. Feld (1968) describes a case in which a 400-foot frontage of buildings in New York City had to be demolished because of damage resulting from an adjacent pile driving operation. The street pavement sank up to 16 inches.

Fine sands below the water table are among those soils most affected by vibrations. In a case described by Terzaghi and Peck (1967), pile driving caused the ground surface within the pile clusters to settle as much as three feet. The soil porosity was reduced from 44 to 38 percent. The effects of densification by vibration from pile driving may extend as far as 50 feet from the source of vibration. The effects are most pronounced for groups of piles for which basin-like subsidence depressions have been observed (Swiger, 1948).

Earthquakes have caused some spectacular ground surface subsidences due to densification of cohesionless soils. Seed (1975) describes cases in Alaska in which layers of alluvial soils decreased in thickness by 2.5 to 4 feet during the Good Friday earthquake of 1964. Similar ground movements have been reported for earthquakes in Chile and Japan. At least part of the 10 to 20 feet of ground surface subsidence that took place in the Mississippi Valley in the 1811 New Madrid earthquake was probably due to densification of the alluvial soils that underlie that area. Movements due to densification are to be distinguished from those due to soil liquefaction and earth crust adjustments, which are discussed later in this section.

In most cases the ground surface movements resulting from densification in earthquakes occur in saturated cohesionless soils that lie below the water table. The densification results from the dissipation of excess hydrostatic pressures induced in the soil by the earthquake ground motions. Seed and Silver (1972) have shown that significant densification can also occur in dry or moist cohesionless soils as a result of compaction induced by earthquake ground motions. They cite cases where significant densification of dry sand occurred during earthquakes: 4 to 6 inches of settlement in a 40-foot thick sand fill in the 1971 San Fernando earthquake; and severe damage to major structures due to compaction of loose sand pockets in the 1963 Skopje, Yugoslavia, earthquake.

Prediction of Movement due to Densification by Vibration

Terzaghi and Peck (1967) state that "Any structure founded on cohesionless soil is likely to settle excessively if the soil is subject to vibrations from such sources as moving machinery, traffic, pile driving, blasting or earthquakes." Although it should be easy for a foundation engineer to recognize conditions under which unusually large soil densification could occur, it is quite another

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matter to predict the amount of densification that might take place for any given condition. No standard procedures are available.

For even the simplest case of perfectly uniform soils, the problem of forecasting or analyzing movements is made difficult by two factors: 1) the inability to obtain undisturbed soil samples for laboratory testing; and 2) the uncertainty concerning the intensity and duration of vibration to which different portions of a soil deposit beneath a foundation will be subjected. In-place field testing is sometimes meaningful, but might itself induce the movement that is to be avoided.

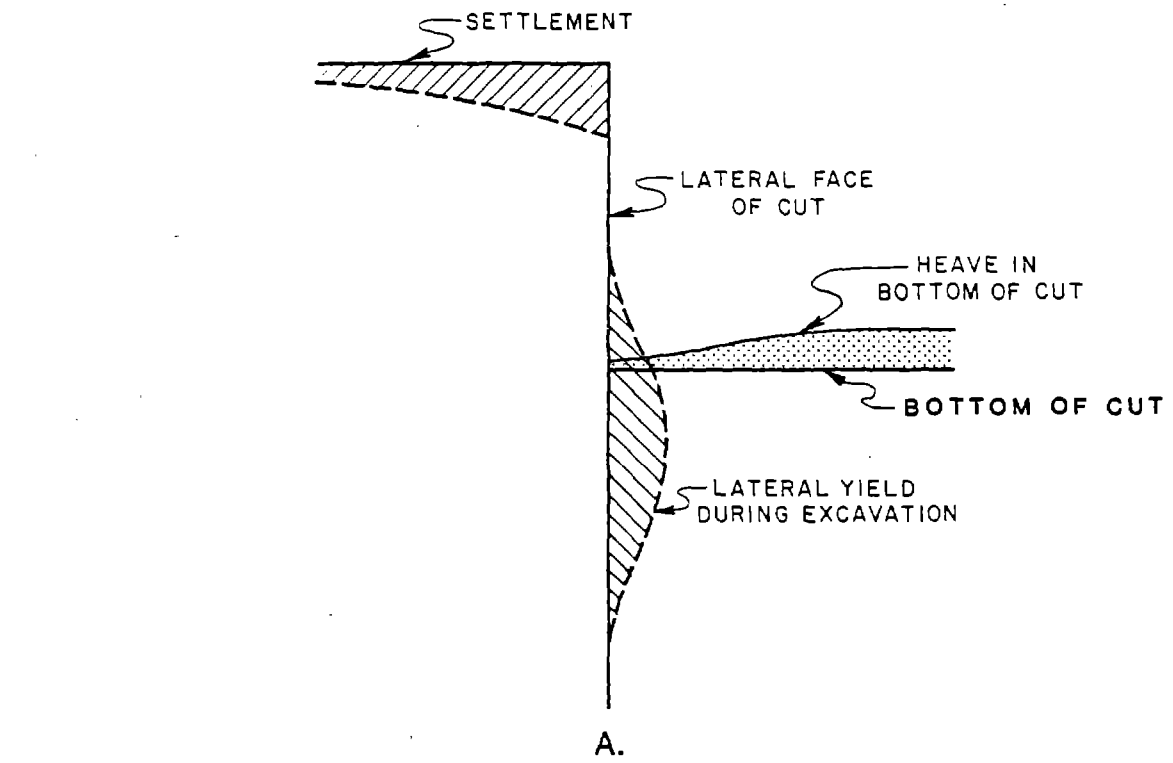
YIELD OF SOIL INTO CONSTRUCTION EXCAVATIONS

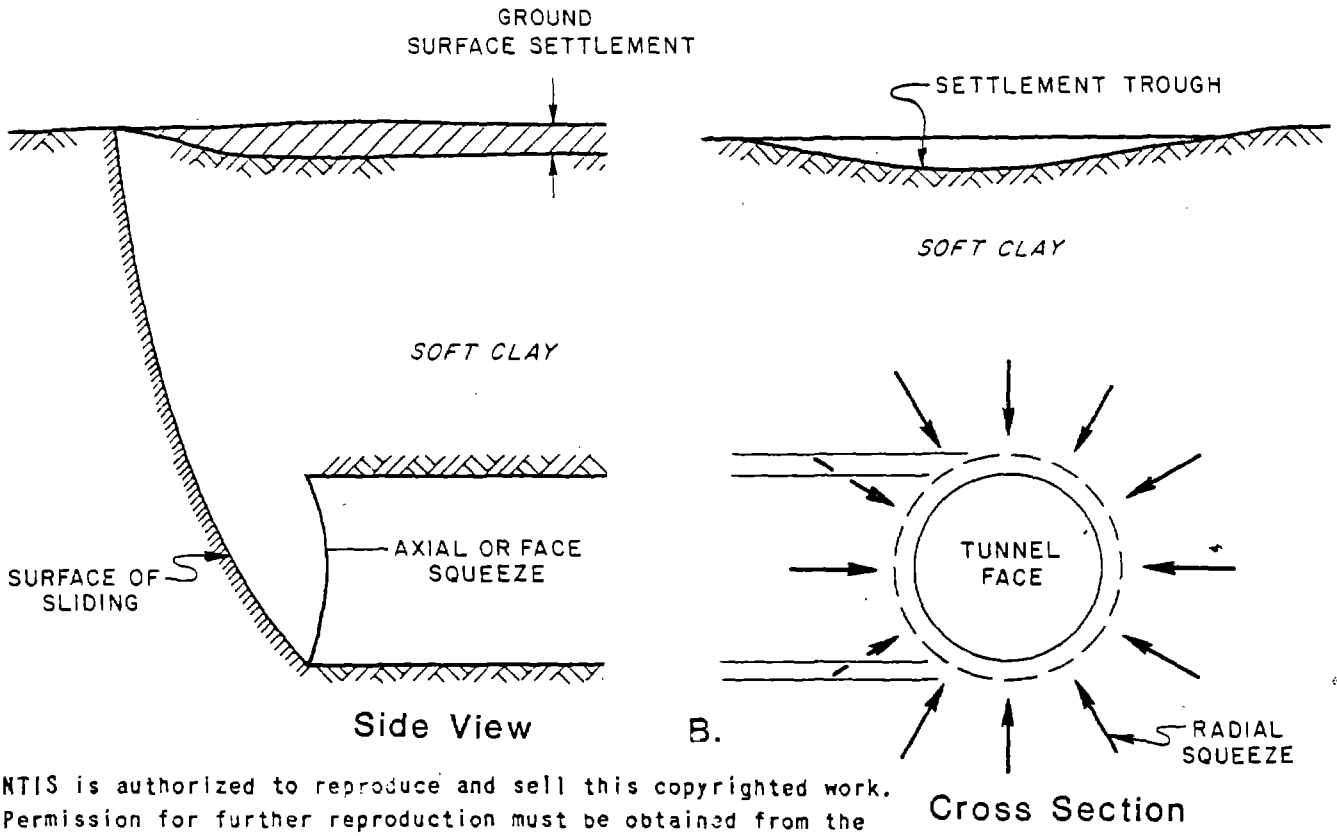
General Considerations. In a preceding discussion, dewatering of construction excavations was described as a possible cause of ground surface subsidence. Significant ground surface movements adjacent to excavations also can result from yielding or flow of soil, as shown schematically in Figure 2.29.

The ground surface movements are caused by the removal of soil material from the excavation and may be influenced by the presence of buildings located adjacent to the cut or above the tunnel. Movements generally occur rapidly during construction and in some cases correspond to a state of plastic failure in the soil mass (Terzaghi, 1943; Flaate, 1966). Additional time-dependent movements, quite independent of the initial movements that occur during construction, may take place in cuts and tunnels as a result of drainage and consolidation if the soils are saturated and soft (Peck, 1969). In some soils, consolidation may represent a significant part of the total settlements (Flaate, 1966). Long term movements above tunnels may also occur as a result of recompression of high-density overconsolidated soils loosened during construction (Farmer, 1977).

Ground movements associated with open cuts and tunnels are of special interest to soils engineers because of the potential damage to buildings located in the immediate vicinity of the construction activity. Such excavations are very common in cities and heavily populated areas. Open cuts may be made for foundations, utility lines, highways, subways and retaining structures. They may be backfilled soon after construction, or they may remain open permanently. Shallow, near-surface tunnels may be used in heavily populated areas for a wide variety of purposes: highway, railroad, subway, utility lines, water conveyance, and storage, among others.

Significant movements may occur adjacent to open excavations and tunnels even when they are promptly provided with structural support during construction. Movements have been measured in soil below the level of excavation in open cuts (Flaate, 1966) and ahead of the excavation faces in soft-ground tunnels (Terzaghi, 1943). These movements are virtually inevitable.





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FIGURE 2.29 - GROUND MOVEMENTS DUE TO YIELD OF SOIL INTO CONSTRUCTION EXCAVATIONS. A. DEEP CUT IN SOFT CLAY (modified from Terzaghi and Peck, 1967): B. NEAR-SURFACE TUNNEL IN SOFT CLAY (modified from Terzaghi, 1950b.).

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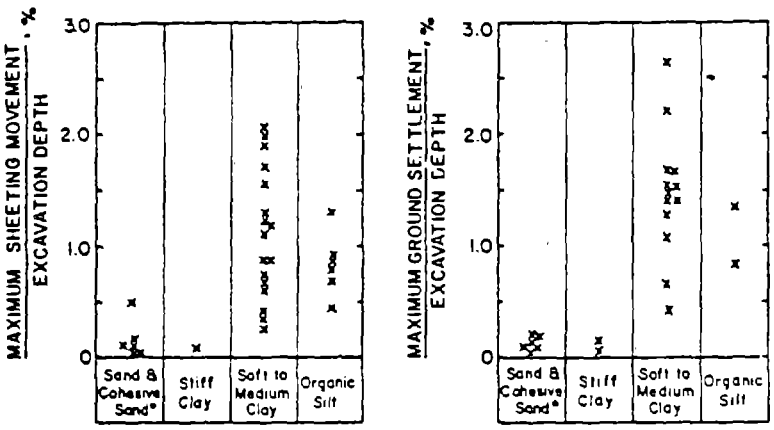
The magnitude of ground surface movements caused by any excavation depends on the following factors: the arrangement of the soil layers; soil properties; dimensions and depth of the excavation or tunnel; the sequence of excavation and support; and construction workmanship (Peck, 1969; Schmidt, 1969; Attewell, 1977a). Quite a bit of detailed field work has been done in measuring ground movements associated with open cuts and near surface tunnels in various kinds of soil and rock. Data available prior to 1969 have been analyzed and summarized by Peck (1969). D'Appolonia (1971) has updated Peck's work for open cuts, and Attewell (1977a) has added considerably to Peck's 1969 state-of-the-art paper on soft ground tunneling by drawing from recent British experience. Ground surface movement data for open cuts and tunnels are presented separately in the following paragraphs.

Open Cuts. Measured ground movements associated with open cuts supported by soldier piles and lagging or steel sheet piling have been collected by Peck (1969) and D'Appolonia (1971) and are shown in Figure 2.30. For all soil types, observed vertical ground surface movements as well as lateral movements of the excavation bracing are less than three percent of the excavation depth, and generally less than two percent. Vertical and lateral ground movements for a 40-foot deep cut in soft clay might be on the order of 10 inches (two percent of excavation depth), whereas for a similar cut in sand or stiff clay the movements may be only one inch (0.2 percent).

The distribution of vertical ground movements adjacent to open cuts in various clay soils is shown in Figure 2.31. The large movements in Zone III correspond to cases where the factor N_b (ratio of overburden pressure to undrained shear strength of soil below base of cut) equalled or exceeded a critical value, N_{cb}, which ranges from 6 to 7.5 for most cuts. This Zone III condition is associated with heave or bearing capacity failure of the bottom of the excavation, and obviously leads to larger ground surface movements.

Soft Ground (Soil) Tunnels. Considerable effort has been expended by researchers to identify the shape and volume of the ground surface settlement trough above soft ground tunnels (Schmidt, 1969; Peck, 1969; Attewell, 1977a). For medium to stiff clays, the trough has been shown to approximate a normal probability curve. Other soils exhibit a more triangular-shaped trough. Attewell (1977a) has presented available case history data in a plot of maximum surface settlement versus volume of settlement trough (Figure 2.32). He suggests that the volume of the settlement trough for tunnels in both cohesive and noncohesive soils is related to the maximum surface settlement, δ_{vsmax}, by the relationship:

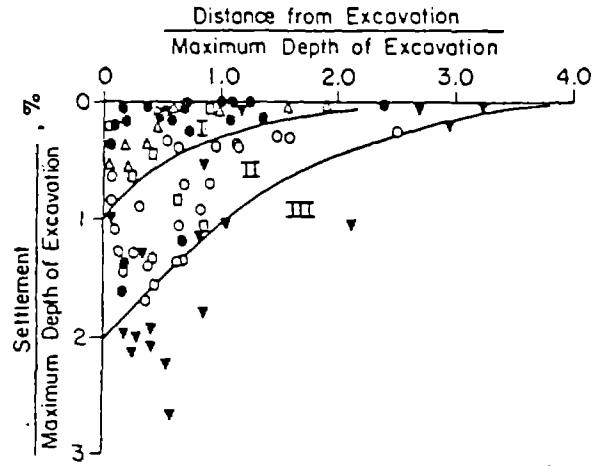
$$V_s = 8.2\ \delta_{vsmax}^{0.84}$$



*Does not include cases where large movements have been caused by subsurface erosion and piping.

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FIGURE 2.30 - MAXIMUM GROUND MOVEMENTS ADJACENT TO OPEN CUTS IN SOIL. EXCAVATIONS EMPLOYING SOLDIER PILES AND LAGGING OR STEEL SHEETING (after D'Appolonia, 1971).



Soft to Medium Clay	Depths of Excavation, ft.
• Chicago, Illinois	30 - 63
o Oslo, Norway, Excluding Vaterland 1,2,3	20 - 38
▼ Oslo, Norway, Vaterland 1,2,3	32 - 35
▵ Stiff Clay and Cohesive Sand	34 - 74
▫ Cohesionless Sand	19 - 47

Zone I
Sand and Soft to Hard Clay
Average Workmanship

Zone II
a) Very Soft to Soft Clay
   1) Limited depth of clay below bottom of excavation
   2) Significant depth of clay below bottom of excavation but $N_b < N_{cb}$

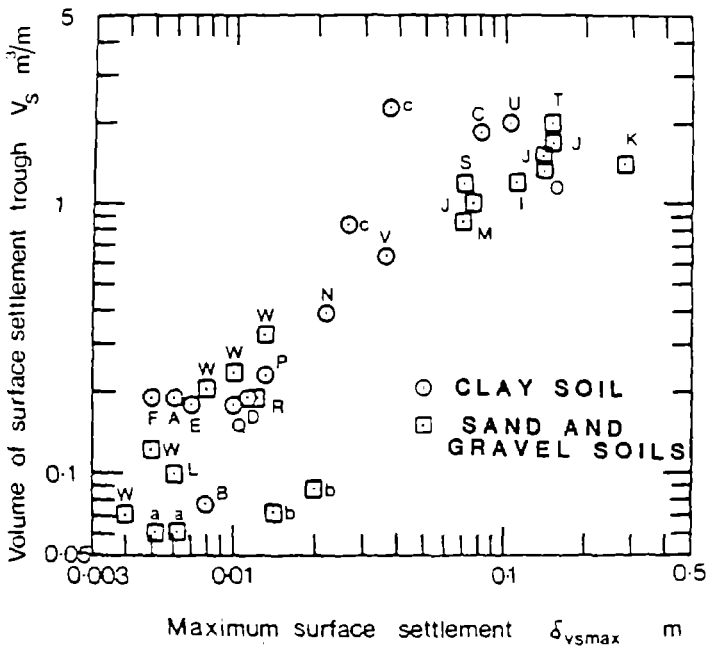
b) Settlements affected by construction difficulties

Zone III
Very Soft to Soft Clay to a significant depth below bottom of excavation and with $N_b \leq N_{cb}$

Note:
All data shown are for excavations using standard soldier piles or sheet piles braced with cross-bracing or tiebacks.

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FIGURE 2.31 - DISTRIBUTION OF VERTICAL GROUND MOVEMENTS ADJACENT TO OPEN CUTS IN CLAY (after Peck, 1969)



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FIGURE 2.32 - SETTLEMENTS ABOVE SOFT-GROUND TUNNELS. (after Attewell, 1977a).

It is obvious from Figure 2.32 that:

1. The magnitude of ground surface settlement is not related strictly to soil type; and,

2. Surface settlement may reach a foot, but is often less.

A rather unusual source of ground surface movement not related to construction but of a somewhat similar effect to tunneling is described by Deere (1961). The extraction of sulphur by the Frasch process of melting and pumping from the cap rock of a salt dome in the Gulf Coast region of Texas produced a classic dish-shaped depression over a ground surface area of approximately 3,000 feet by 5,000 feet. Maximum settlements up to 4.9 feet and horizontal movements up to 1.3 feet were observed over a 31-month period of extraction from depths of 1300 to 1600 feet. The strata overlying the cap rock consist of slightly cemented to uncemented sands, gravels, clays and clay shales. The mechanism of subsidence is not described, but it seems to have involved yield of the overburden into the cavity. Measurements of ground movements suggest displacement along a normal fault on one side of the subsidence depression. There is no discussion of building damage that might have been caused by the movements.

Types of Ground Movement Associated with Open Cuts and Soft Ground Tunnels. As a general rule, ground surface movements adjacent to open cuts and soft ground tunnels in dense noncohesive or stiff ground deformations tend to be localized around an open excavation or above a tunnel, and are commonly distortional. They tend to occur within a lateral distance from the side of an open excavation of one to three times the excavation depth and within one to eight tunnel diameters from the centerline of a tunnel. Hogging is a prominent form of distortion.

Building damages caused by ground movements adjacent to open cuts and above tunnels are probably more common than one would be led to believe from the literature. It is probably easier to find accounts of total failure than to find case histories where ground movements were measured and minor damages were recorded. Notable documented accounts of damage around cuts and tunnels for which ground or building failure did not occur are given by Attewell (1977b) and Cording (1978). The lack of well-documented records of damage due to these causes should not be taken as an indicator of either the infrequency or the low potential for such damage.

Prediction of Movement due to Yield of Soil into Construction Excavations

Because of the overwhelming influence of construction methods (excavation technique, type of support) and quality of workmanship, ground movements associated with surface and near surface excavations cannot be predicted by calculation (Terzaghi and Peck, 1967). In fact, the overall design of open cuts and tunnels suffers from a

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great many shortcomings which, according to Peck (1969), have their origin in the engineers' commonly held notion that design and construction are two separate endeavors. Because of this situation and the fact that communication and understanding between designers and constructors are all too often poor to non-existent, it is not surprising that failures still occur in open cuts and tunnels. (ENR, 1973; ENR, 1974; Sowers and Sowers, 1967; Feld, 1968).

When excavation failures do occur, the source of movements is normally apparent because of the catastrophic nature of the failure. For ordinary construction carried out in such a way that failure is avoided, the only available guide for estimating ground movements is reliable, well-documented case records. Guidelines developed from such information are summarized below.

Open Cuts. The work of Peck (1969) and D'Appolonia (1971) in summarizing movements surrounding open cuts allow the following generalizations to be made:

1. For cuts in loose sands and gravels the maximum vertical ground settlement adjacent to the cut may be on the order of 0.5 percent of the cut depth. For cuts in dense sand, the settlements are likely to be inconsequential. This is for cases in which the soil is above water table or the ground water has been lowered by pumping and brought under control. If the ground water has not been brought under control, large, erratic and damaging settlements may occur as a result of piping or erosion.

2. For granular soils that contain a small amount of cohesion, limited field data suggest movements in accordance with item 1 above. Even a limited amount of cohesion greatly reduces the potential for erosion of cohesionless sands and gravels.

3. For cuts in soft and medium clay, the maximum inward movements of the sheeting and maximum surface settlements are commonly on the order of 1.0 to 2.0 percent of the excavation depth.

4. Open excavations in stiff clay normally produce maximum movements that are only a small fraction of one percent of the excavation depth.

5. Limited data for cuts in compressible silt indicate that ground movements, both lateral and vertical, are of the same order of magnitude as those for cuts in soft clays (1.0 to 2.0 percent of excavation depth).

These movements include the effects of consolidation as well as flow or yield of soil.

Soft Ground Tunnels. The best available guidelines for predicting ground surface movements above soft ground (soil) tunnels are those

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suggested by Attewell (1977a). He used case history data to relate the maximum ground surface settlement to the volume of the settlement trough and suggests the following guidelines for estimating the volume of the settlement trough per unit length of tunnel Vs:

1. For tunnels in dense non-cohesive soils (sand and gravel), Vs is commonly one to two percent of the tunnel face area, provided good ground control measures are employed in driving the tunnel. In difficult ground conditions, for which ground loss at and behind the face is excessive, Vs may approach five percent of the tunnel face area.

2. In loose sand, Vs is so highly dependent on the quality of workmanship in the tunnel driving operation that it is difficult to generalize. Attewell (1977a) suggests that Vs will usually not exceed five percent of the tunnel face area. If total face failure occurs, not an uncommon incident in loose sands, the ground surface movement may equal or exceed that associated with sinkhole development.

3. For tunnels in clay (cohesive soil), the settlement trough volume depends on the depth of the tunnel relative to the strength of the soil. This ratio is expressed as the Simple Overload Factor, OFS:

$$OFS = \frac{T_z - T_i}{C_u}$$

where

T_z = overburden pressure

T_i = supporting internal pressure within tunnel, such as compressed air pressure

C_u = undrained shear strength of soil

Attewell suggests that the surface settlement volume Vs is about 1 percent of the excavated tunnel volume when the OFS is less than 4. If the OFS lies between 4 and 6, Vs typically lies between 1 and 5 percent of the tunnel volume. If the OFS exceeds 6, failure by flow of clay into the tunnel is likely to occur and ground surface movements will often be unpredictably large, as in the case of landslides.

As additional data from field measurements become available, the guidelines and empirical relationships presented above will no doubt be refined and improved. However, the strong dependency of ground surface movements on construction methods will doubtlessly remain, and it seems unlikely that the engineer will ever completely escape educated-guess technology.

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MOVEMENTS ASSOCIATED WITH DRIVEN PILES

A comprehensive review of this subject is given by D'Appolonia (1971). Three primary types of ground movements may result from pile driving:

1. Transient motions or vibrations.

2. Densification of cohesionless soils; and,

3. Lateral and heave movements in soft and medium clays.

Driving of displacement piles into clay causes the soil around the pile tip to displace laterally and upward, resulting in ground surface heave. During this process, excess pore water pressures are built up and the soil is greatly disturbed, resulting in an increase in its compressibility. As the pore pressures dissipate, the ground surface settles, usually more than the amount of the heave because of the increased soil compressibility due to disturbance during driving.

The effects of the soil behavior discussed above may also result in significant lateral movements of nearby open excavations, retaining walls and slopes within distances of the piles equal to the thickness of the clay penetrated by the piles. D'Appolonia (1971) summarizes numerous case histories of movements due to pile driving. Most measured heave lies in the range of 0.3 to 2.0 inches; net settlements following heave have been as high as four inches. Lateral movements up to two feet were observed in one instance where piles were driven adjacent to an anchored bulkhead on a riverfront in Chicago.

Building damages are known to occur from the effects described above. In one city in Norway, the use of pile foundations was discontinued because of the settlements resulting from driving. Pile driving is frequently carried out in connection with open excavations, and it may sometimes be impossible to distinguish ground movements caused by these two sources.

BEARING CAPACITY FAILURES

General Considerations. If the weight transferred to the ground by buildings, fills or other artifical loadings exceeds the strength of the subsoil, a bearing capacity failure ensues. As long as the imposed loading does not exceed the ultimate soil strength, the movements are usually finite, sometimes tolerable, and may even be predictable. After the ultimate strength is exceeded, movements are usually so excessive and the damage so great that there is little concern with the actual magnitude of the movement.

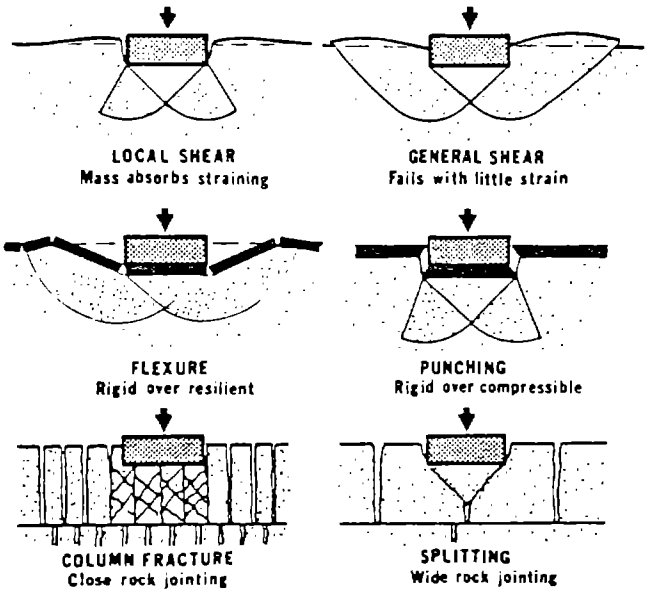
Bearing capacity failures in soils can take several forms, depending on subsurface conditions and foundation type. In some cases the evidence of bearing capacity failure is so unique that it leaves no doubt as to the cause of the ground movement. In other cases it is clear only that excessive settlement has taken place.

Sowers (1974b) suggests the six mechanisms of bearing capacity failure for soil and rock shown in Figure 2.33. All are characterized by the downward movement of a wedge of ground immediately beneath the foundation that results in displacements of the surrounding soil and permits continued penetration of the foundation and wedge. The mechanisms apply to deep foundations (end-bearing piles or drilled piers) as well as to shallow foundations. The more common forms of bearing capacity failures are discussed in the following paragraphs. Although bearing capacity failures are uncommon and usually result from gross ignorance or negligence on the part of the foundation engineer or contractor, there do exist in the literature case history records of some notable failures. All of the cases discussed below concern shallow foundations.

General Shear Failure. Sometimes referred to as a general bearing capacity failure, this is the most characteristic mechanism and can occur for a wide variety of soil and rock types. If the foundation loading is perfectly symmetrical and if the ground is uniform in the horizontal direction, failure occurs along two well defined symmetrical surfaces of rupture that propagate to the ground surface and cause surface heave adjacent to the foundation. A feature characteristic of general bearing capacity failures is that ground movements continue until either the soil strength is changed, such as by densification, or the soil loading is changed due to excessive tilt, increase in the bearing area or some other factor.

If the foundation loads are slightly eccentric, which is likely to be the case as failure progresses, the general shear failure results in tilting to one side. According to Tomlinson (1969), the inevitable variations in soil shear strength and compressibility beneath foundations nearly always cause greater yielding on one side of the foundation, forcing the center of gravity of loads to shift, leading to an asymmetrical increase in pressure and further tilting.

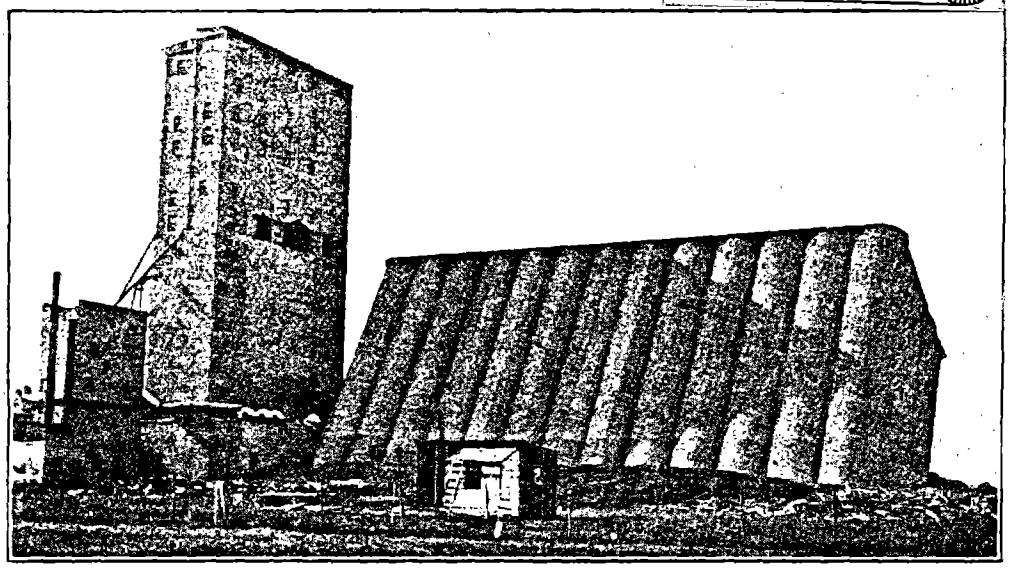
One of the most classic bearing capacity failures on the North American continent involved a grain elevator at Transcona, in the province of Manitoba, Canada. The million-bushel grain elevator was constructed in 1913 on a reinforced concrete raft foundation measuring 77 feet by 195 feet in plan embedded at a depth of 12 feet below the ground surface. It is underlain by 30 to 50 feet of clay. Within an hour of the first filling of the elevator, the structure settled uniformly about one foot and within 24 hours tilted about 27 degrees from the vertical, as shown in Figure 2.34. Soil investigations made after the failure indicated that the bearing capacity beneath the elevator was about 2.6 tons per square foot, the same order of magnitude as the average bearing pressure beneath the structure when it failed (Peck and Bryant, 1953). Nordlund and Deere (1970) describe the collapse of another grain elevator at Fargo, North Dakota, located in a geological setting similar to the Transcona elevator. Strangely enough, load-settlement data were obtained for the Fargo elevator as it was being filled, but were not plotted until after the elevator had failed. Had the data been plotted during filling, they would have clearly



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FIGURE 2.33 - BEARING CAPACITY FAILURE MECHANISMS
(after Sowers, 1974b).

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A.



B.

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FIGURE 2.34 - BEARING CAPACITY FAILURE OF TRANSCONA GRAIN ELEVATOR. A. OVERALL VIEW: B. CLOSE UP SHOWING HEAVED GROUND
(Courtesy of Canadian Pacific Railway)

indicated the trend shown in Figure 2.35 and given clear warning of the impending failure. The last set of settlement readings, taken four days prior to failure, showed settlements ranging from 9.5 to slightly over 12 inches, approximately double the settlement observed only one week earlier. Displacement of the elevator foundation after failure was on the order of ten vertical feet. The soil to the side of the elevator heaved about six feet.

Tschebotarioff (1973) illustrates the collapse of heavy reinforced concrete cement silos that were founded on varved clay, the same type of deposit on which the Fargo and Transcona elevators were founded. The failure mechanism, shown schematically in Figure 2.36, is essentially the same as for the Fargo and Transcona grain elevators and is characteristic of plastic cohesive soils.

General bearing capacity failures, involving curved shear surfaces similar to those of the previous examples, can also occur for deep foundations, such as piers and pile groups, which are not founded in firm substrata. Usually the settlement of such foundations becomes excessive before a complete failure of the type shown in Figure 2.36 occurs. Bearing capacity failures of pile foundations are virtually absent among case histories in the literature (McClelland, 1972). Whether this is due to the rare occurrence of failures or to the misinterpretation of observed settlements is not known.

Although angular displacements associated with general shear failures may be quite large, building damage does not necessarily follow. The bearing capacity failure at Fargo completely destroyed that structure, but the Transcona elevator remained intact and was successfully righted, underpinned and returned to use (Prentis and White, 1950). Large movements therefore do not necessarily portend structural damage, provided distortions of the structure do not occur.

Punching Shear Failure. This type of bearing capacity failure occurs when an overloaded foundation sinks into the ground with only slight disturbance to the surrounding ground. Soil failure occurs by vertical shear around the foundation perimeter with no well-defined failure plane (Vesic, 1975). The punching shear failure may appear simply as excessive settlement. This mechanism of failure is characteristic of foundations on stiff or dense strata underlain by soft or loose soils (Figure 2.33) and of foundations on a variety of highly porous soils that are capable of substantial compression (Figure 2.37).

A punching-type failure may occur suddenly, yielding a load-settlement relationship like that in Figure 2.35, or it may occur gradually yielding a load-settlement curve as shown in Figure 2.38. The former is typical of cemented loose sands, certain relatively wet loess deposits and soft clay, whereas the latter is typical of uncemented loose sands, loess deposits of low natural water content, and weak clays that undergo large strains before failure.

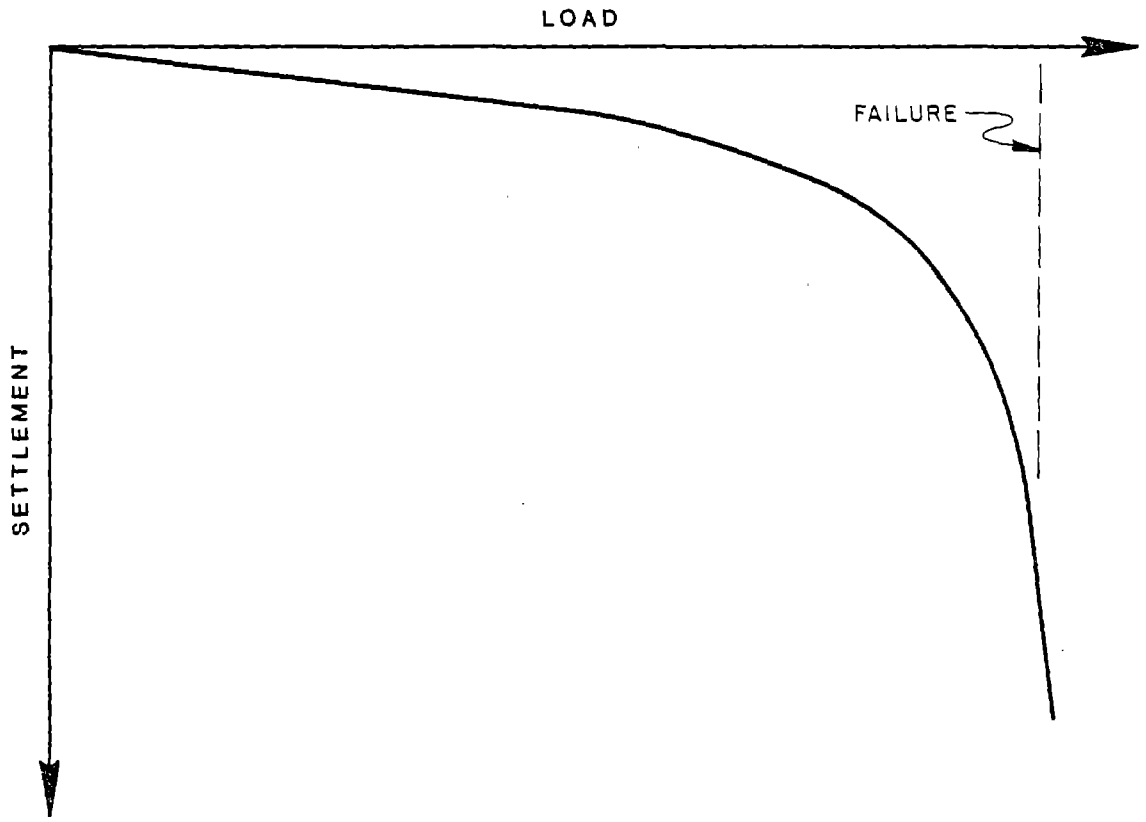
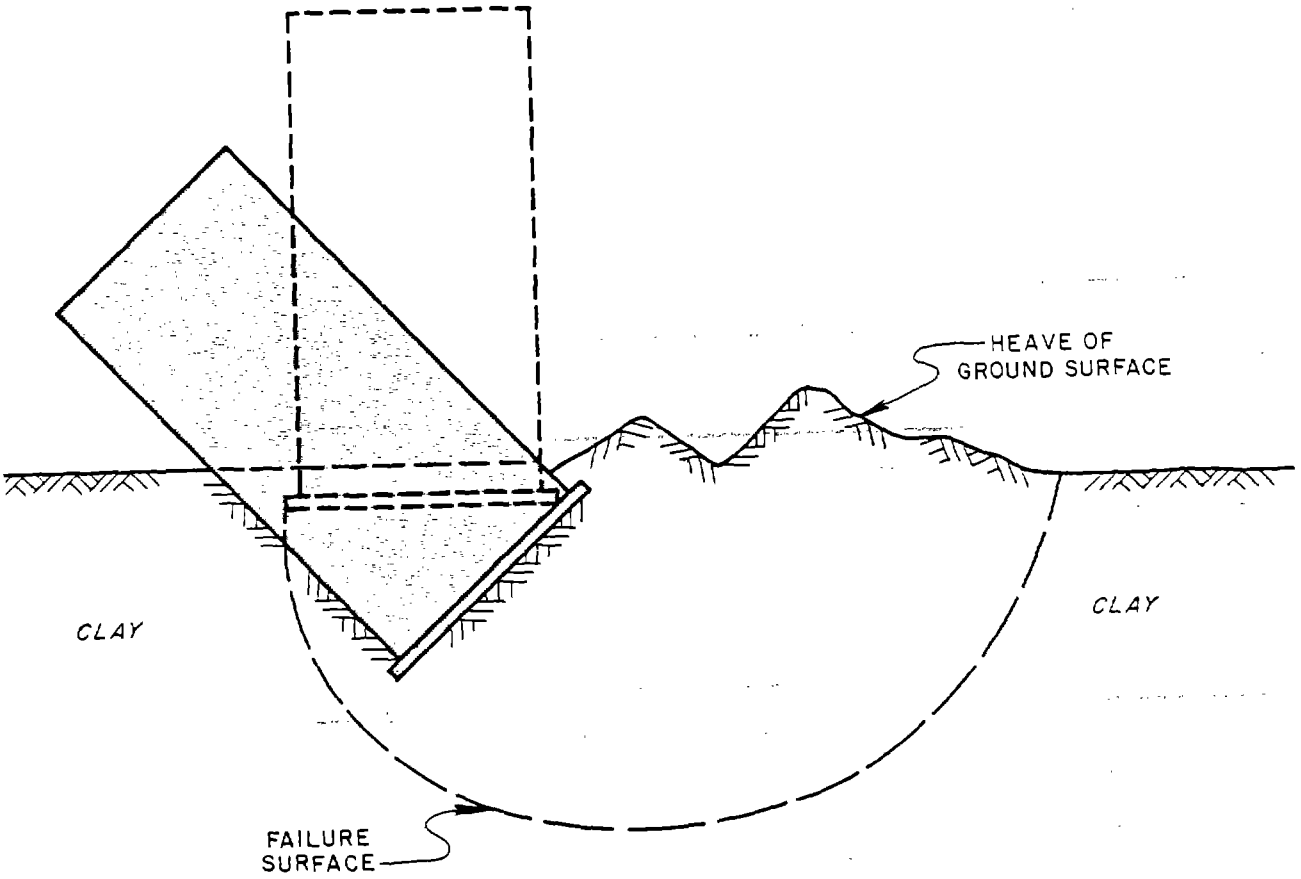


FIGURE 2.35- LOAD-SETTLEMENT CURVE FOR BEARING CAPACITY FAILURE IN SOFT CLAY



GURE 2.36 - BEARING CAPACITY FAILURE OF GRAIN ELEVATORS AND SILOS FOUNDED ON SOFT CLAYS AND SILTS

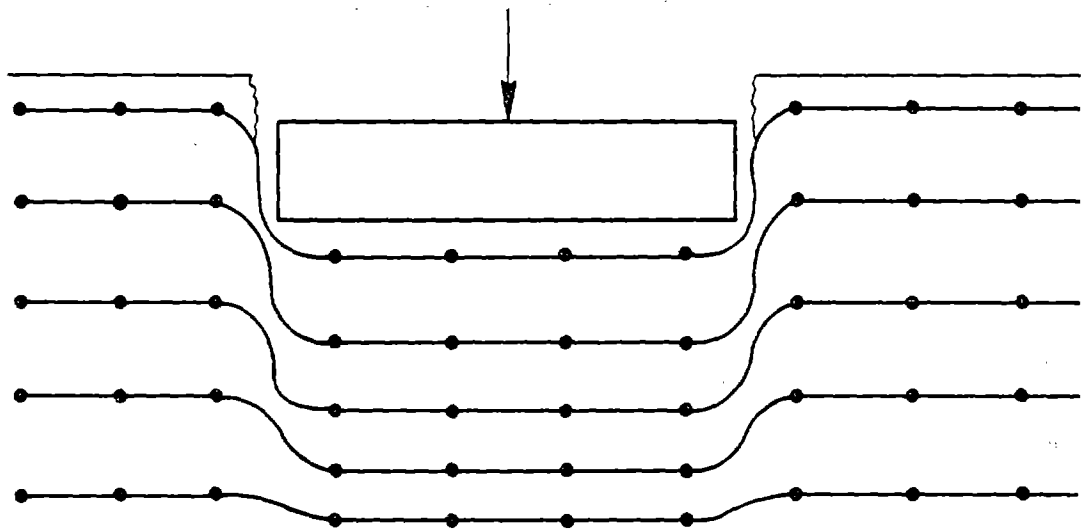


FIGURE 2.37- PUNCHING-TYPE BEARING CAPACITY FAILURE OF FOOTING IN LOESS OR LOOSE SAND

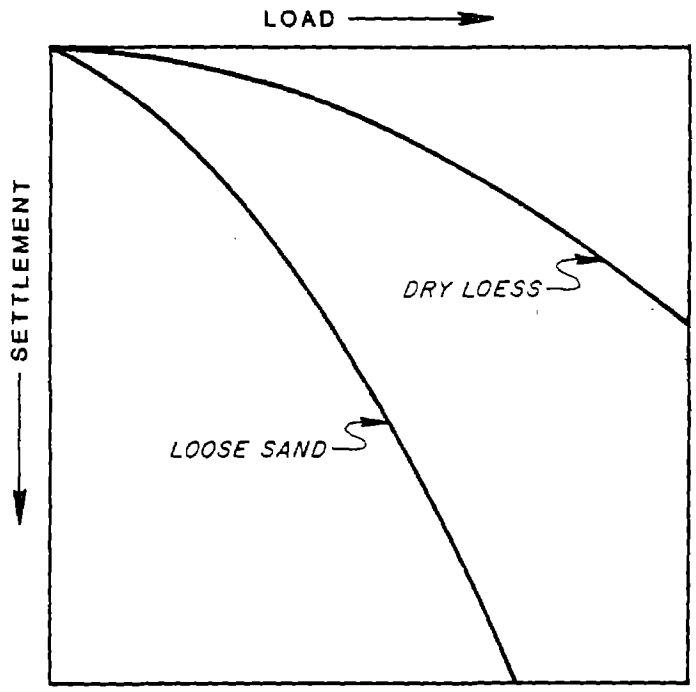


FIGURE 2.38 - LOAD-SETTLEMENT CURVES FOR LOOSE SAND AND DRY LOESS

Peck and Ireland (1957) discuss a punching-type bearing capacity failure of a grain elevator in Kansas that was founded on loess. Maximum settlement exceeded 2.5 feet, and the differential settlement was about one foot, resulting in substantial tilting of the structure. The failure discussed by Peck and Ireland was not due to wetting and collapse, although that type of failure is well known in loess.

A special case of punching failure is that which occurs in sanitary land fills where a thin crust of soil covers the more spongy and resilient sanitary fill (Sowers, 1968). The mechanism is illustrated in Figure 2.33. A similar failure could occur in a natural soil deposit or on a poorly compacted man-made fill.

An unusual case of a punching type bearing capacity failure may have been partly responsible for the 19 feet of settlement of a railroad girder bridge, reported by Tschebotarioff (1973). The bridge piers were underlain by layers of mud, clay and fine sand with many hollow animal shells. Tschebotarioff speculates that crushing of the shells may account for much of the settlement.

Punching-type bearing capacity failures normally result in vertical translational ground movements in uniform soils. Many loessal soils in the central United States experience this type of failure and settlements up to one foot have been tolerated for heavily loaded structures such as grain elevators, provided eccentric loading can be avoided (Peck, Hanson and Thornburn, 1974). If the soils are nonuniform in horizontal extent, or if loading is eccentric, punching-type failures will likely lead to rotational as well as translational movements, causing tilt of the structure. Even these movements have been tolerated in some instances where building damage was not severe.

Local Shear Failure. Local shear failure is a transitional mode of failure that lies between general shear failure and punching shear failure. The mass accommodates part of the displacement by compression; well defined shear surfaces below the foundation terminate somewhere in the mass (Vesic, 1975). Visible ground bulging may occur to the sides of the foundation, but slip surfaces do not appear at the ground surface until vertical footing displacement on the order of one-half the width or diameter of the footing has occurred. Load-settlement curves for local shear failure resemble those in Figure 2.38. Because of the large settlements, catastrophic tilting or collapse of a foundation is unlikely to occur.

Because of the difficulty in recognizing this type of failure, some reported cases of large settlements may in reality have been local bearing capacity failures. Bolt (1975) suggests that the initial tilting of the famous Leaning Tower of Pisa, which occurred after only three galleries had been constructed, is a result of a partial bearing capacity failure in the early stages of construction. Because the Tower never toppled, and because it is underlain by a two-meter thick layer of compressible clay, a local bearing capacity failure would seem the most probable mechanism. The

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100-year delay in construction is believed to have allowed the compressible clay strata beneath the tower to increase in strength by consolidation, thereby permitting construction to be completed. The centuries of consolidation and long term creep settlements have long since masked the original movements.

Ground movements caused by local shear failure are a combination of translational or rotational and distortional movements.

Liquefaction. In a preceding section it was explained how shocks and vibrations can densify a sand deposit. If a saturated sand possessing a ratio of void volume to mineral volume in excess of a critical value is subjected to shock or vibration and drainage cannot occur (if, for example, the sand deposit is located beneath the groundwater table), the presence of water in the soil voids may temporarily prevent densification of the sand. When this occurs, the pressure in the water-filled voids (porewater pressure) increases to the point where the stress between soil particles (intergranular stress) is carried entirely by the porewater. Because water cannot sustain shear stresses, the sand deposit loses its strength and behaves like a viscous fluid (Seed and Lee, 1966). This phenomenon is called liquefaction.

Liquefaction can be responsible for several important ground movements. Flow slides occur in certain loose, saturated sand deposits when they are disturbed by a shock from an earthquake or construction activity, or by a rapid rise in the groundwater level. Flow slides are known to occur on natural sand slopes as flat as 15 degrees (Terzaghi and Peck, 1967). Many landslides that occur during earthquakes are a result of liquefaction of entire soil deposits or of sand seams and layers (Seed, 1968). Lateral movements associated with flow slides vary from several feet to hundreds of feet (Youd, 1973).

Of special interest is the reduced bearing capacity that occurs when a sand deposit becomes liquefied or "quick". Buildings resting on sand that becomes liquefied may literally sink into the ground, as happened in the Niigata, Japan, earthquake in 1964 (Seed, 1975). Building settlements sometimes exceeded three feet, and one apartment building rotated through a vertical angle of 80 degrees (Figure 2.39). Hundreds of buildings were damaged, including port facilities and bridges that experienced substantial lateral ground movements. Several buried structures, such as sewage treatment tanks, floated to the ground surface.

The mechanism of failure by sinking apparently is related to:

1. The upward flow of sand and water through springs and boils that erupt at the ground surface, resulting in a loss of ground from beneath structures, and

2. Displacement of the liquefied sand by the sinking structure.



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FIGURE 2.39 - TILTING DUE TO LIQUEFACTION BEARING CAPACITY FAILURE, 1964 NIIGATA EARTHQUAKE (after Seed, 1975).

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The severity of building damage in the Niigata earthquake was correlated with the relative density of the underlying sand deposits--loose sand yielding the worst damage for shallow foundations. Because the occurrence of liquefaction depends on the development of a critical hydraulic gradient, the location of the groundwater table, the permeability of the sand, and the boundary drainage conditions are also important factors.

Blasting and shock waves generated by some construction activities can cause the liquefaction of very loose sands and cohesionless silts. The nature of ground deformations and the duration of the induced vibrations are important variables in determining liquefaction in any situation where ground motion is the initiating factor.

Failure of Brittle Materials. Brittle materials such as rock and hard soil may fail by general shear or punching shear if they are intact. Typically, such materials contain joints, open seams, and other discontinuities, so that failure occurs by one of the mechanisms shown in Figure 2.33 or by shear displacement along discontinuities. Movements prior to failure are likely to be small. Consequently, there is little forewarning. Bearing capacity failures in brittle materials are not common.

Bearing Capacity Failures Caused by Ground Surface Loadings Other Than Buildings. The majority of bearing capacity failure cases discussed in the preceding paragraphs resulted from overloading the subsurface by the weight of a building. Far more common, according to Ireland (1969), are bearing capacity failures beneath artifical fills or "made" ground (Figure 2.40). This general shear-type failure resembles those described earlier for grain elevators--particularly, the rotational movement along a circular failure plane and the heave of the original ground surface. Haupt and Olson (1972) describe such a failure in a soft varved silt during construction of a 37-foot high highway embankment fill in Vermont. The failure encompassed 300 feet of embankment and penetrated 50 to 60 feet into natural soil, displacing approximately 150,000 cubic yards of material. The embankment dropped 10 to 12 feet in elevation while material in a counterberm at the toe of the embankment moved laterally 25 to 30 feet. Failure evidently took place rapidly, the rate of movement exceeding one foot per minute. Settlement of the embankment prior to failure was on the order of 2 to 2.5 feet. Bjerrum (1972) reviews similar failures for embankments and test fills at 12 other locations throughout the world where the seat of failure was soft ground beneath the fill. In some cases cracking of the fill and settlements greater than two feet preceded failure, while in others no cracking was noticed and observed movements were very small (less than one inch), indicating a brittle behavior.

The same failure mechanism shown schematically in Figure 2.40 has occurred beneath earth fill dams. Petersen, Iverson, and Rivard (1957) describe several dam failures in Canada in which

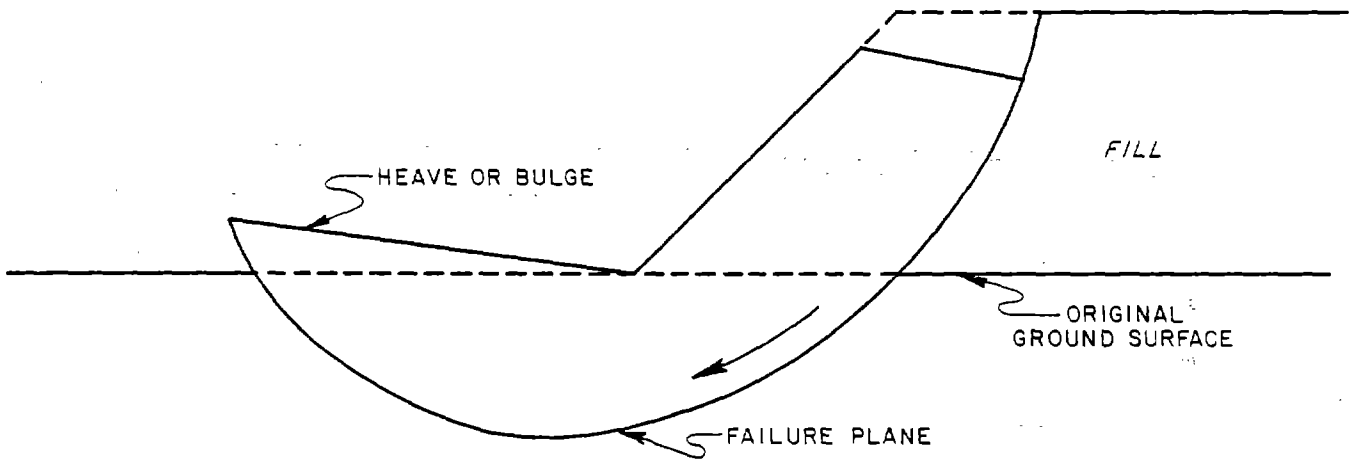


FIGURE 2.40 - BEARING CAPACITY FAILURE BENEATH AN ARTIFICIAL FILL

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shear failure occurred through saturated highly plastic clay foundations. Visible surface cracks up to 1 foot wide and 22 feet deep preceded failure movements of 3 to 6 feet (horizontal and vertical).

Peck, Ireland and Teng (1948) made a study of railroad retaining wall failures in the United States and Canada and found that the most common cause of trouble is the overloading of clay foundations by the weight of the retaining wall backfill. Although poor subsurface conditions may be recognized in subsurface exploration and acknowledged in design by placing the retaining structure on pile foundations, there is an apparent tendency to disregard the load that the backfill is likely to place on the subsoil. If the fill height is such that the imposed load exceeds the bearing capacity of the subsoil, shear failure will occur. Many times the resulting movements involve the retaining wall and its foundations. Typically the retaining wall tilts toward the backfill and sinks into the ground, movements ranging from several inches to more than ten feet.

Huntington (1957) describes several modes of bearing capacity failure for retaining wall structures that are not provided with adequate foundations. Ireland (1969) points out that the frequency of bearing capacity failures beneath bridge abutment fills is probably a result of engineer's attention being diverted to the details of structural design of the abutment. This unfortunate circumstance could occur in building design also, where placement of fill around or under a building could add enough load to the subsurface to cause failure. The subtleties of site grading, building design, and landscape architecture could combine to make the recognition of such a failure difficult. It is to be expected that ground movements due to bearing capacity failures of fill could be confused with mine subsidence movements, particularly in local or punching shear failures where movements may not be catastrophic.

Prediction of Bearing Capacity Movements

The prediction of the ultimate bearing capacity of foundations on soil materials has advanced to the point that relatively little research effort is currently being expended in this area. The historical development of bearing capacity analysis for shallow foundations and a summary of available equations for ultimate bearing capacity, based on the general shear type of failure, is given in an excellent state-of-the-art report by Moorhouse (1972). The bearing capacity of deep foundations such as piles and piers is not as well understood as for shallow foundations, but reasonably reliable methods for estimating bearing capacity are available (McClelland, 1972; Meyerhoff, 1977). The biggest problem confronting the practitioner in predicting the ultimate bearing capacity of foundations is in the determination of the appropriate strength parameters (Burland, Broms, and de Mello, 1977). For loess and other soils that fail by mechanisms other than general shear, the ultimate bearing capacity is frequently determined from in situ plate load tests.

Whereas the foregoing permit prediction of the susceptibility or likelihood of failure, there is no technique that can be used to predict the amount of ground movement that will take place after a bearing capacity failure has occurred. Thus, ground movements due to bearing capacity failure cannot readily be distinguished from those of other sources on the basis of the magnitude of observed movement. The most reliable means of making the distinction is an examination of the load-settlement history of the structure under consideration. Unfortunately, this information is rarely available.

SETTLEMENT UNDER SURFACE LOADS

General Considerations. "Settlement" refers to ground movements that are caused by compression of the underlying materials by the weight of surface loads.

Surface loads include, among others: buildings; man-made fills, such as highway embankments, earth and rockfill dams and building site fills; resource and refuse storage piles; and reservoirs, such as sedimentation ponds. Loads may be permanent or temporary. Dynamic, impact and so-called live loads from snow, ice, wind and moving vehicles can also cause settlement.

Any form of surface loading, no matter how limited in extent or magnitude, will cause some surface settlement. An exception is where a thickness of soil or rock equivalent to the imposed weight of a building is removed prior to the construction of the building. In this case, there is no net increase in loading of the subsurface, and settlement is limited to recompression of the soil.

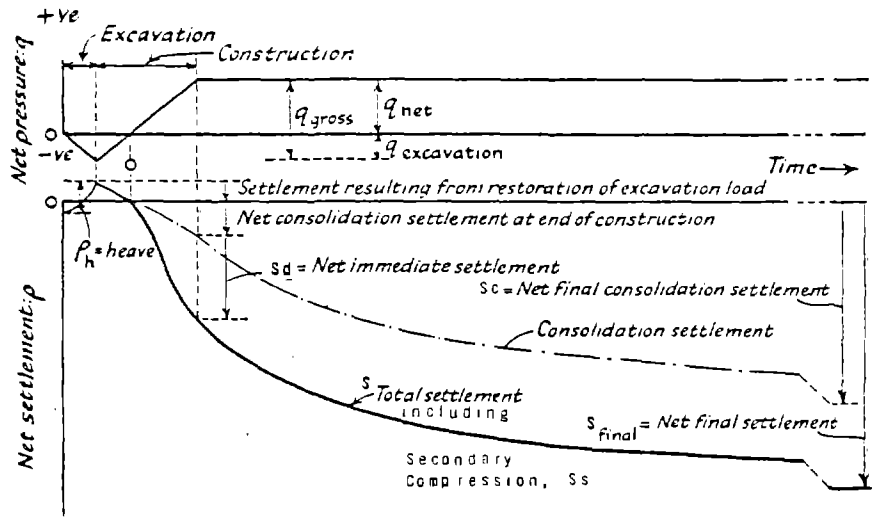
All geologic materials undergo some settlement when subjected to surface loading. Rock is no exception. The settlement may not be appreciable under ordinary circumstances, but is real nonetheless.

As a result of these circumstances, the subject of settlement due to surface loads has received a great deal of attention from civil engineers, and a great deal of theoretical knowledge and practical experience on this subject has accumulated over the past 40 years. The term "normal ordinary settlement" is frequently used in reference to settlements caused by surface loads.

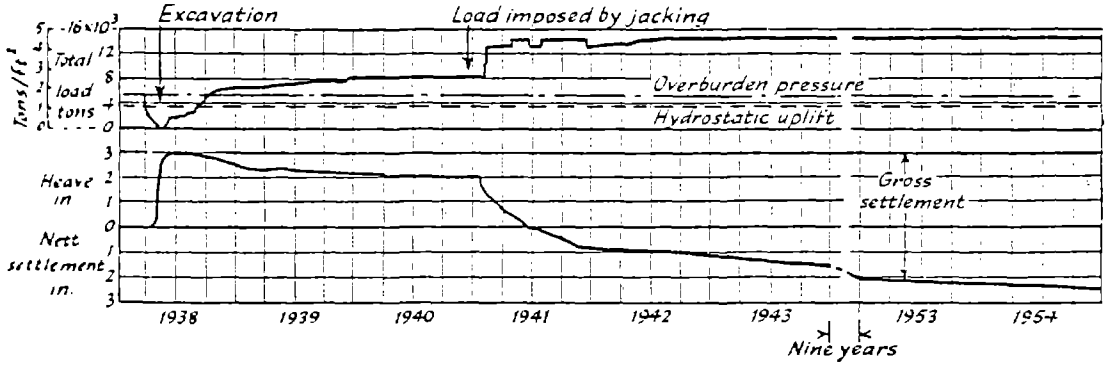
Types of Settlement. Generally, the settlement of a point on a statically-loaded ground surface area is considered to be the sum of three components:

$$S = S_d + S_c + S_s$$

in which S is the total settlement (Figure 2.41), S_d is the immediate or distortion settlement, S_c is the consolidation or hydrodynamic settlement, and S_s is the secondary compression, secondary settlement, or creep settlement (Perloff, 1975). The use of this equation implies that the loads are less than those required to produce failure. Immediate or distortion settlement



A.



B.

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FIGURE 2.41 - SETTLEMENT

A. DEFINITIONS (modified from Skempton, Peck and McDonald)

B. TYPICAL CURVE, SETTLEMENT OF PIER NO. 4, WATERLOO BRIDGE, LONDON (after Cooling and Gibson, 1955)

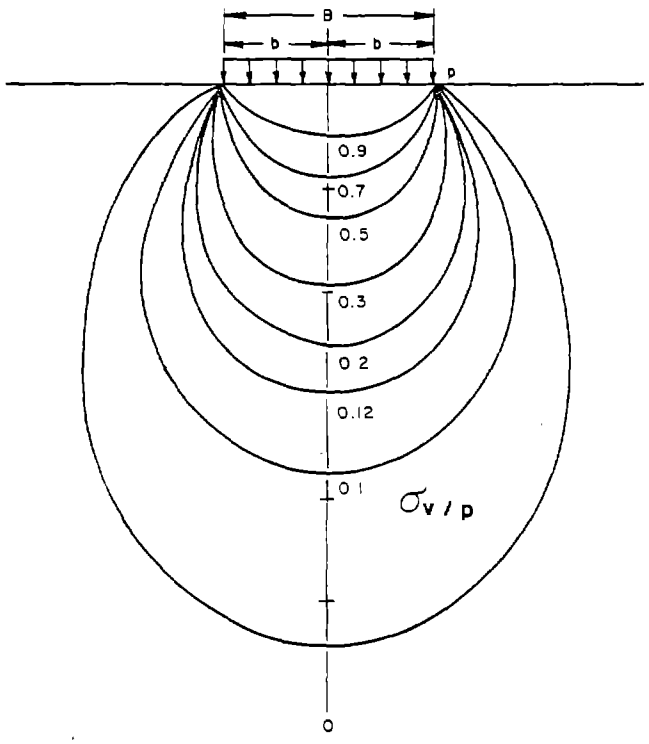
occurs concomitantly with load application and is analogous to the initial deformation that occurs when a weight is hung on a simply-supported beam. Consolidation settlement occurs as a result of the expulsion of porewater as the soil skeleton adjusts to the newly imposed state of stress. There is no analogy between consolidation and the steel beam model. Secondary compression occurs after the excess porewater pressure generated by the surface loading has completely dissipated and is controlled largely by the rate at which the skeleton yields and compresses. This is comparable to time-dependent creep of a steel beam under a suspended weight.

The time-rate analysis of settlement is common to all soil and rock although in specific cases one or more of the settlement components may be either negligible or non-existent. For instance, in clean sand or gravel, the flow of water from the pores is generally rapid and settlement is practically instantaneous. Secondary compression is normally negligible, and total settlement occurs immediately. In most rocks loaded within the normal range of building practice, pore pressure effects are minimal and the settlement is immediate. If the rock is weak and highly fractured or if the imposed loads are high, consolidation and creep settlement could contribute to total settlement. Soft clays typically exhibit all three components of settlement. The consolidation component is usually the most significant.

Types of Ground Movement. Because a load imposed at ground surface is distributed laterally with depth (Figure 2.42), ground surface settlements partially derive from the compression of material that lies outside the vertically projected area of the surface load. A loaded surface area of diameter B will cause a settlement depression at the ground surface with a diameter greater than B, as shown in Figure 2.43. The shape of the settlement depression depends on the shape of the loaded area (square, round, semi-infinite, etc.); the distribution of the load (uniform, triangular, irregular, etc.); and the stress-deformation characteristics of the subsurface materials. The overall shape of the ground settlement profile represents a distortion of the original ground surface and in some cases resembles subsidence patterns above undermined areas. Even if a structure located on the ground surface settles uniformly, as would be the case for a uniformly loaded rigid foundation resting on a uniform soil, the soil beneath the foundation is distorted because of shear stresses induced by the loading.

Case Histories of Settlement. There are undoubtedly more accounts of settlements due to surface loading than any other type of ground movement. A collection of all the available literature would constitute a substantial library. A few of the more significant sources of case histories and techniques for analysis are listed below.

- Proceedings of the International Conferences on Soil Mechanics and Foundation Engineering. Nine conferences, 1936-1977.



σ_v IS VERTICAL STRESS IN THE GROUND

ρ IS APPLIED SURFACE LOAD

BULBS ARE CONTOURS OF EQUAL VERTICAL PRESSURE

FIGURE 2.42 - "PRESSURE BULBS" BENEATH A UNIFORMLY LOADED CIRCULAR AREA.

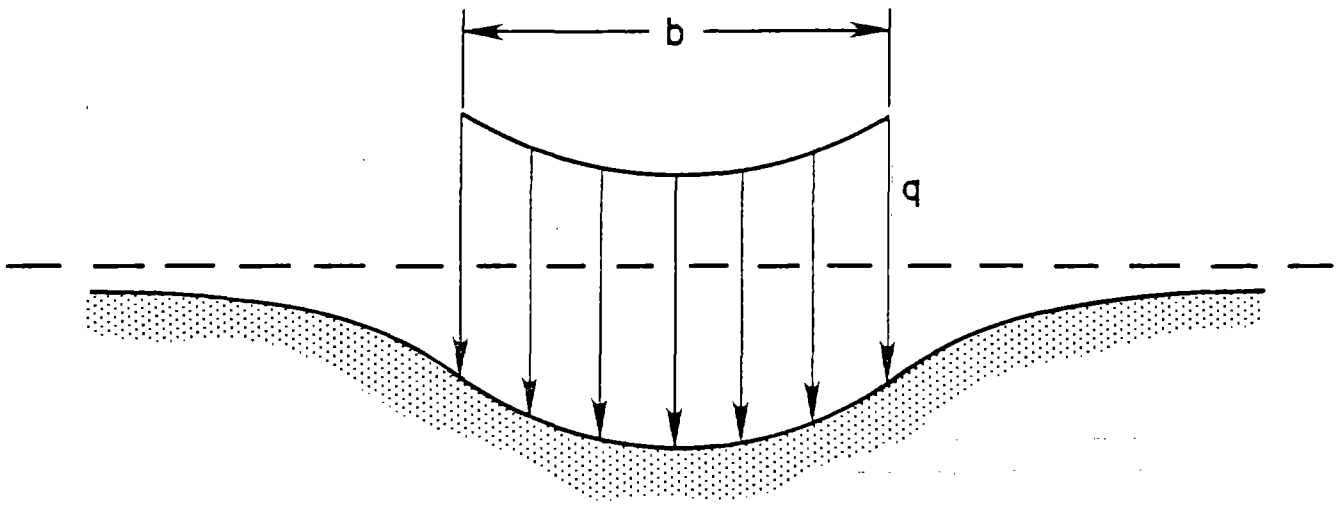


FIGURE 2.43 - SETTLEMENT DEPRESSION CAUSED BY UNIFORM SURFACE LOADING, SUCH AS OCCURS BENEATH OIL TANKS AND OTHER FLEXIBLE STRUCTURES THAT SUPPORT UNIFORM LOADS.

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- Proceedings of the American Society of Civil Engineers, Journal of the Soil Mechanics and Foundation Engineering Division, through December 1973; since January 1974, Journal of the Geotechnical Engineering Division. Of particular interest is the September 1964 Journal which contains papers from the conference "The Design of Foundations for Control of Settlements."

- Proceedings of the Specialty Conference on Performance of Earth and Earth-Supported Structures, American Society of Civil Engineers, three volumes, 1972.

- Settlement of Structures, Conference organized by the British Geotechnical Society, 1974, proceedings published by John Wiley and Sons, Inc., 1975.

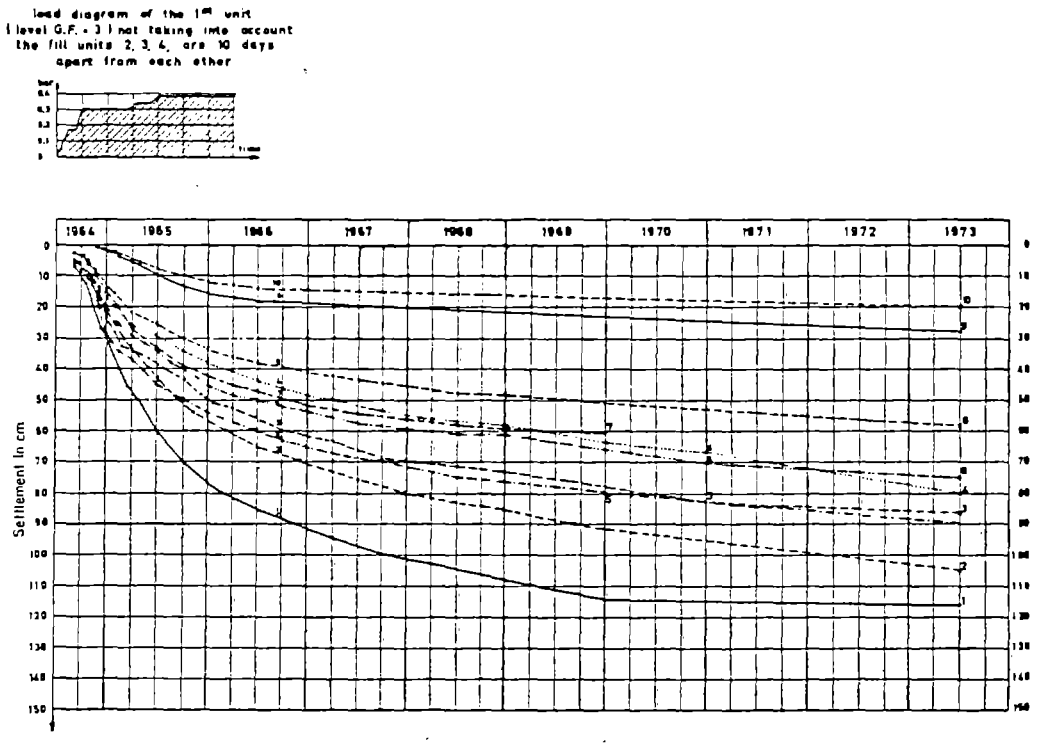
These sources serve as a starting point for a study of the subject. There are hundreds of others.

A few cases of settlement will be described in the following pages to illustrate the diversity of recorded experiences with settlements due to surface loads.

Cases of Large Settlements of Buildings. In the First Conference on Large Ground Movements and Structures, held at the University of Wales Institute of Science and Technology, Geddes (1977a) defined large ground movements as those in excess of 50 mm (about 2 inches). If this criterion is adopted, a cursory review of the literature will lead to the conclusion that countless cases exist where large ground movements resulted from settlement under surface loads. It is a fair guess that at least one-third of the reported case histories of settlement under surface load are in excess of 50 mm.

Among the more spectacular reported settlements due to surface loads are those described by Sanglerat, Girousee, and Gielly (1974) for three-story residential buildings at Nantua, France. These buildings are founded on cellular raft foundations underlain by soft organic clays of considerable and variable thickness. The total settlements observed over a nine-year period range from 200 millimeters to 1160 millimeters (0.65 feet to 3.8 feet), an astonishing amount considering the relatively light structures. The time-settlement charts are shown in Figure 2.44. Most of the settlement is probably consolidation. Because the buildings and utility lines were provided with movable connections, no damage occurred despite differential settlements of 6 to 16 centimeters (2.4 to 6.3 inches).

A general review of the literature reveals that large settlements of structures founded on clay soils are rather common. Simons (1974) summarizes 15 cases of buildings founded on clay in which the actual or extrapolated settlements after 20 years exceed 200 mm (about 8 inches). Nine exhibited settlements in excess of 400 mm. Apparently none of the structures were rendered unserviceable.



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FIGURE 2.44 - TIME-SETTLEMENT CURVES FOR THREE-STORY RESIDENTIAL BUILDINGS AT NANTUA, FRANCE. CURVES ARE TYPICAL FOR CONSOLIDATION SETTLEMENT OF CLAY (after Sanglerat, Girousse, and Gielly, 1974).

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Sometimes the large settlements of buildings founded on soft clay terminate after a period of time, as shown in Figure 2.45. Such behavior suggests consolidation settlement is predominant. In other cases, the settlement log-time curves display non-terminating settlement, indicating significant secondary compression. This is shown in Figure 2.46 for Norwegian and Swedish clays.

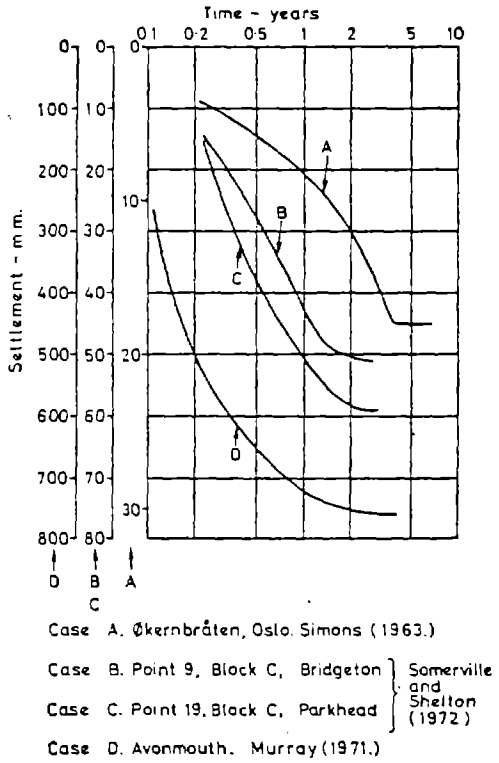
Large settlements of buildings are not limited to those founded on soft clay. Gielly, Lareal and Sanglerat (1974) report settlements of 26 centimeters (about 10 inches) for a 22-story raft-supported building in Cartagena, Columbia, that is underlain by 28 meters of loose to dense, highly variable sandy marine deposits. The settlements at four corners of the building were almost identical at the time the last observations were made, eight months after the end of construction. No damage was reported.

Chellis (1961) summarizes the settlement failure of the Naval Aircraft Building in Philadelphia, which is underlain by 35 to 55 feet of compressible silt. The building was originally founded on clusters of 30-foot long concrete piles whose tips rested in the silt stratum. Three years after construction, the settlements had reached 12 to 24 inches and the floors and walls of the building had cracked. In a prior load test, one foundation pile settled three inches after 45 days under full building load. Settlements six months after construction were already 6 to 16 inches. The time rate of settlement for silt is relatively fast, somewhere between the nearly instantaneous settlement of sand and the slower consolidation of clay.

Settlements on Dense, Stiff, or Hard Subsurface Materials. All but the lightest of buildings, or those provided with compensated or floating foundations, are likely to experience large settlements when founded on soft or loose normally consolidated or lightly over-consolidated soil deposits. The foundation engineer is, therefore, inclined to seek out more firm subsurface materials on which to place building foundations. The following case histories show the range of experiences with settlements on dense, stiff, and hard subsurface materials.

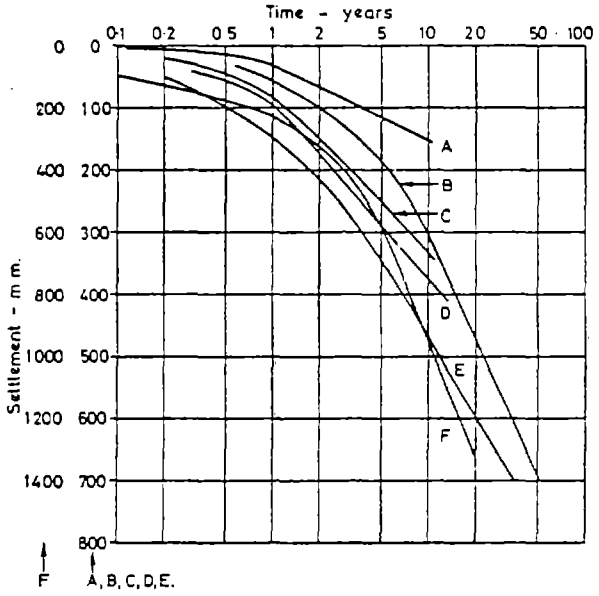
The settlement behavior of 340 column footings for steel mill facilities in Indiana demonstrates the behavior of medium dense sand under moderate loads. The footings were designed for an average bearing pressure of 3.0 tons per square foot, and carried dead loads from the structure as well as live loads of snow, wind and an overhead crane. Only eight percent of the measured settlements were greater than 0.5 inch. The maximum settlement was 0.75 inch. Most differential settlements were less than one quarter the total settlement. About one-half the measured settlement took place during mill construction (D'Appolonia, D'Appolonia, and Brissette, 1968).

Dense sand and gravel are among the best bearing materials for heavily loaded foundations, but will still experience some settlement. Breth and Chambosse (1974a) report the settlement behavior



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FIGURE 2.45 - SETTLEMENT-LOG TIME CURVES INDICATING TERMINATING SETTLEMENT (after Simons, 1974).



Case A. Scheitliesgate 1, Drammen, Bjerrum (1967)

Case B. Jernbanetollsted, Oslo, Anderson and Frimann Clausen (1974)

Case C. Konnerudgate 16, Drammen, Bjerrum (1967)

Case D. Skabo office block. Oslo, Simons (1957)

Case E. Turnhallen, Drammen, Simons (1957)

Case F. Test fill. Vasby, Sweden. Chang, Broms and Peck (1973)

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FIGURE 2.46 - SETTLEMENT-LOG TIME CURVES INDICATING NON - TERMINATING SETTLEMENT (after Simons, 1974).

of a nuclear reactor in Germany founded on dense to very dense sands and gravels in the Rhine River Valley. The foundation is a 60-meter diameter, 3-meter thick plate with an average bearing pressure of 4.25 kg/cm^2 (4.35 tsf). Near the end of construction, the maximum observed settlement at the center of the reactor was about 55 mm (about 2.2 inches). The plate foundation deflected into a dish-shaped pattern with a center depression about 10 mm below the edge.

Stiff clays are usually overconsolidated and behave differently than normally consolidated clays, which have never experienced a stress greater than exists at the time of loading.

Heavily overconsolidated clays usually can support much greater loads than normally consolidated clays without the problems of large settlement and potential bearing capacity failure. A great many buildings have been built on overconsolidated clays in England and elsewhere. Butler (1974) summarizes 29 case histories of settlements of structures founded on the London, Woolwich, and Reading and Gault Clays, which are overconsolidated by virtue of removal of the overlying sediments by erosion. Shallow foundations averaging 200 kN/m^2 (2.12 tons per square foot) experience settlements in the range of 50 to 100 mm (2 to 4 inches). Typically these settlements are reached two to three years after construction. Settlements by the end of construction are generally 40 to 60 percent of the maximum observed settlements.

Settlements of Atypical Subsurface Materials. Conventionally, foundation engineers categorize settlement experience by material: sand, soft clay, stiff clay, and rock. There are a number of subsurface materials, natural and artificial, that do not fit these general categories and whose compressibilities lead to significant settlements from surface loading. Settlements of structures founded on three such materials are considered below: peat, strip-mine backfill and sanitary landfills.

Peat is an accumulation of partially decomposed and disintegrated plant material that has been fossilized under conditions of incomplete aeration and high water content. It may occur as a surface deposit in swamps and bogs, or as an interbed within deposits of silt and clay. Normally peat is avoided as a foundation material or is removed by excavation and replaced with compacted backfill. When peat occurs as an interbedded deposit, these measures may not be feasible. In some cases, its presence has not been known prior to construction.

Bell (1978) discusses a settlement case from the United Kingdom in which a house founded on peat settled about one meter below street level. Tomlinson (1969) cites another case in the UK in which a 5-foot thick layer of peat within a soil deposit compressed 10 inches when subjected to the weight of: 1) a 12-foot thick layer of hydraulically placed sandfill; and 2) a brick store imposing a surface loading of 0.5 ton per square foot. The degree of damage is not reported in either reference.

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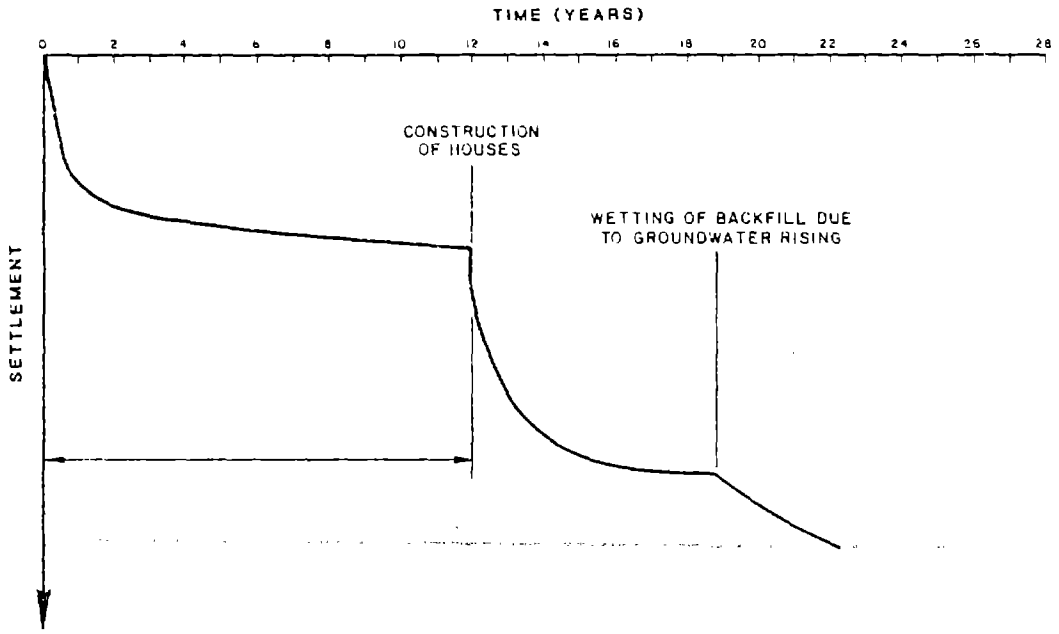
Compression of a 2.4-meter thick stratum of peat is believed responsible for the serious damage that necessitated foundation underpinning of the Winchester Cathedral in England (Legget, 1973). Settlements in parts of the cathedral reached 76 mm (3 inches). The structure was built in the period 1079-1202 and was founded on a raft of beech logs and short oak piles, 1.5 to 1.8 m in length.

The settlement of peat is a complex process that involves an initial period of rapid consolidation that is associated with the dissipation of excess pore pressures, followed by significant secondary compression, which, in fibrous peats, may be due to the extremely slow drainage of water from micro-pores. Some researchers even postulate tertiary and quarternary phases of compression. To complicate the matter, serious settlements can result from movements not related to volume change (consolidation), such as creep and lateral spreading.

Remarkable settlements have been recorded for roadway embankments constructed over peat. Weber (1969) reports settlements of 3 to 18 feet for highway embankment fills placed across peat deposits in San Francisco Bay. The peat thickness was between 12 and 28 feet and the embankment fill thickness was between 1.5 and 11 feet. Primary consolidation, referred to by Weber as loading settlement, was usually complete within a few weeks after the fills were placed. Long-term settlements on the order of several feet followed the loading settlement and were still taking place after 15 years, presumably by secondary, tertiary, or quarternary creep mechanisms.

A most informative and pertinent case of settlement has been reported by Penman and Godwin (1974) for experimental houses constructed on loose opencast mining backfill at Corby, England. The material on which the houses are founded consists of about 18 meters of boulder clay and oolitic rock dumped in piles by a dragline and leveled with a bulldozer. The two-story houses were of brick construction founded on either strip or raft foundations. Approximately five years after house construction, the settlements had leveled off to values ranging from less than 10 mm (1/2 inch) to about 180 mm (7 inches). The variations reflect erratic backfill conditions. Settlement rates began to increase once again with the rise in water level in the fill apparently due to soil structure collapse (Figure 2.47). Damage to the houses included diagonal stepped cracks in brickwork and 2 mm wide cracks. The maximum differential settlement was about five inches.

The most significant settlements lasted about five years and are thought to be due to near surface compaction of the boulder clay under the weight of the houses. Because the ground was not saturated during that time, the settlement is probably due to simple compression of the fill material and gradual softening of near surface boulder clay as it becomes wetted from surface water infiltration. The settlement rate increase at the 19-year mark is attributed to a reduction in strength of the rockfill upon saturation and subsequent crushing at the points of contact of larger



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FIGURE 2.47 - PROBABLE SETTLEMENT HISTORY OF OPENCAST BACKFILL AT CORBY, ENGLAND
(Adapted from Penman and Godwin, 1974)

rock pieces. Penman and Godwin conducted large-scale laboratory oedometer tests on the boulder clay and oolitic rock backfill materials and found that compression upon saturation amounted to 10.9 percent for the boulder clay and 3.7 percent for the oolitic rock. Their short-term (three-day) laboratory creep measurements of the rockfill prior to saturation suggested that the measured creep rate can account for only a small part of the observed settlements during the nine years between the time the houses were built and the time the water level started to rise in the fill. It is apparent that several settlement mechanisms are responsible for the total settlements measured at Corby: compression and creep of the fill under its own weight; compression and creep of the fill under the weight of the houses; and compression or collapse of the internal structure of the fill due to wetting.

Sanitary landfills consist of garbage, paper, fibrous wastes, metal cans, glass, bedsprings, old tires, large organic material (stumps, trees), junk auto bodies and refrigerators, demolition wastes (brick, earth, stone, etc.), ash and other waste material (Sowers, 1968). The thought of eventual construction atop sanitary landfills was almost certainly of no concern when the first fills or dumps were started centuries ago. As suitable building sites become more expensive and less available, landfills will no doubt come into greater utilization for that purpose.

Sanitary landfills are often highly erratic in both composition and engineering properties. Old "dumps" were simply open masses of waste. Today, dumps are commonly controlled insofar as wastes are separated according to size or composition and spread, compacted and covered daily with thin layers of soil. The settlement of landfills occurs by a number of different mechanisms (Sowers, 1973):

1. Mechanical compression of loose, resilient materials under the weight of: the fill itself, including its cover; added leveling fill; and structures placed on the fill.

2. Ravelling or movement of fine fill particles into the open voids between larger particles or pieces of junk and rubbish.

3. Physico-chemical changes such as corrosion, oxidation, and combustion.

4. Biochemical decay and fermentation.

5. Interaction of the above mechanisms, such as organic acids produced from decay causing corrosion, or volume changes associated with compression triggering ravelling.

The rates of settlement depend on the composition of the fill and a host of environmental factors, including temperature and ground water conditions.

Sowers (1968) reports settlements of 4 to 10 inches for buildings founded on sanitary landfills. Typically, most settlement occurs as an initial compression during construction and the year following. Occasionally significant settlements occur after this period.

Tan (1977) describes the settlement of three single- and double-story office buildings in Singapore built on shallow raft foundations over four meters of uncompacted rubbish. The rubbish was underlain by soft clays and peaty clays that were generally poor for any foundations other than piles. Settlements after 40 months amounted to 2.5 to 17 centimeters (1 to 6.6 inches), but because of their uniformity, had caused only minor structural damage. Settlements of two of the buildings had slowed, but not leveled off, after 40 months.

Sometimes the occupants or owners of buildings on fill are not aware that their buildings rest on "made ground". A careful study of original ground surface contours and subsurface exploration may be required to determine the existence of the landfill after it has been built upon, planted, paved or otherwise masked by human activities.

Several types of fill are shown in Figure 2.48. Valley and lowland fills are commonly created for industrial or residential development where suitable natural land is unavailable. They may occupy several hundred acres or several square miles. Lowland fills are common along the Atlantic Coast where wetlands are being reclaimed for urban development (Rutledge, 1970). Embankment fills are used in highway and railroad construction, earth and rockfill dams, canals, etc.

Settlements of fills may derive from two sources:

1. Compression of loose or soft natural materials beneath the original ground surface.

2. Compression of the fill material itself.

The weight of the fill itself or the weight of structures placed on the fill can both cause settlements. Figure 2.49 shows settlement cracks that have formed in the rear wall of a restaurant located 10 feet from the crest of a 20 foot high compacted earth and rock fill constructed in a lowland adjacent to a highway.

Fill settlements are significant in the present study for several reasons:

1. Fill weights can be considerable and can conceivably initiate subsidence above undermined areas. If the fill itself is settling, differentiation of fill settlement from subsidence is difficult.

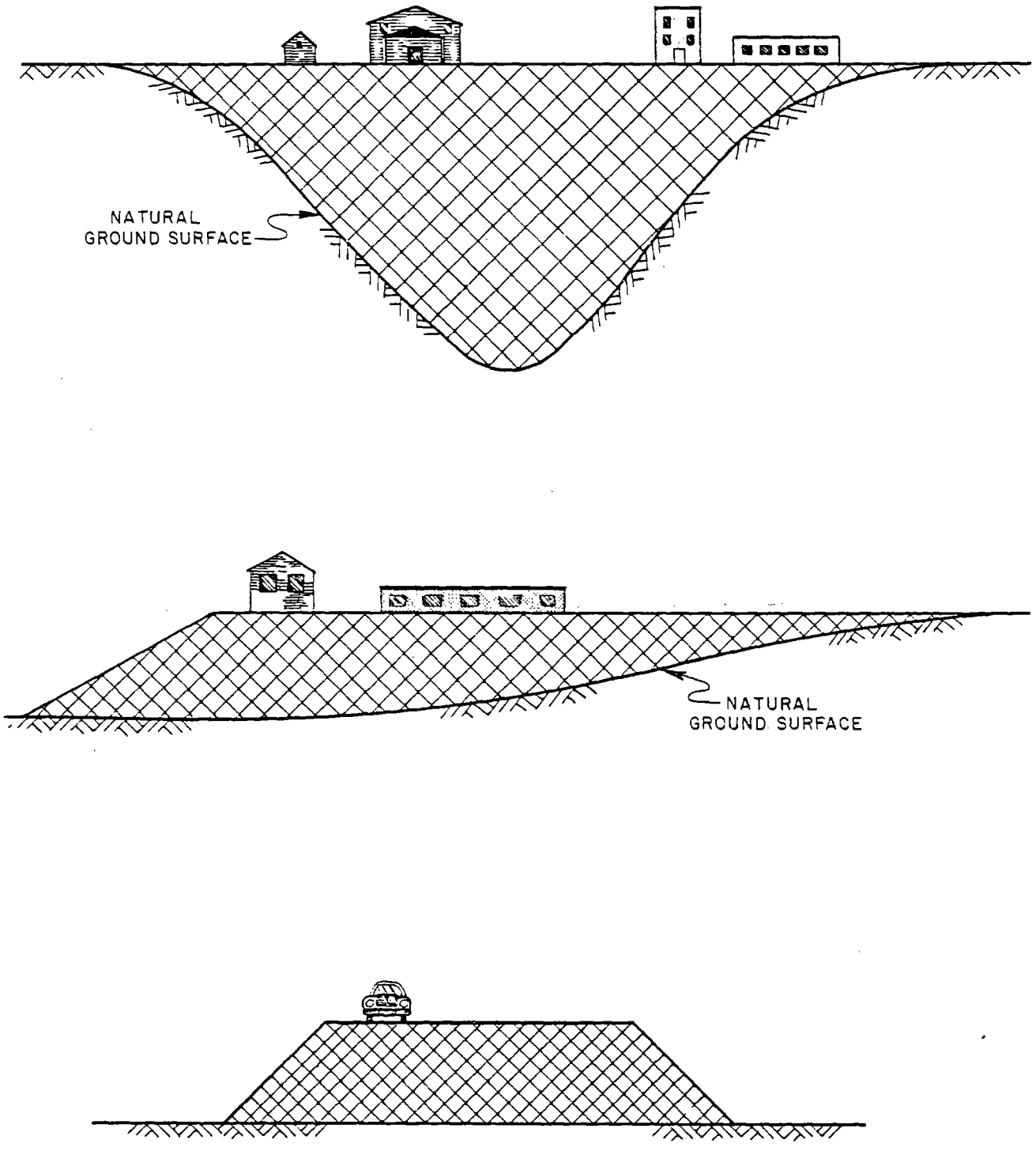


FIGURE 2.48 – TYPES OF ARTIFICIAL FILLS: A. VALLEY FILL: B. LOWLAND FILL: C. EMBANKMENT FILL

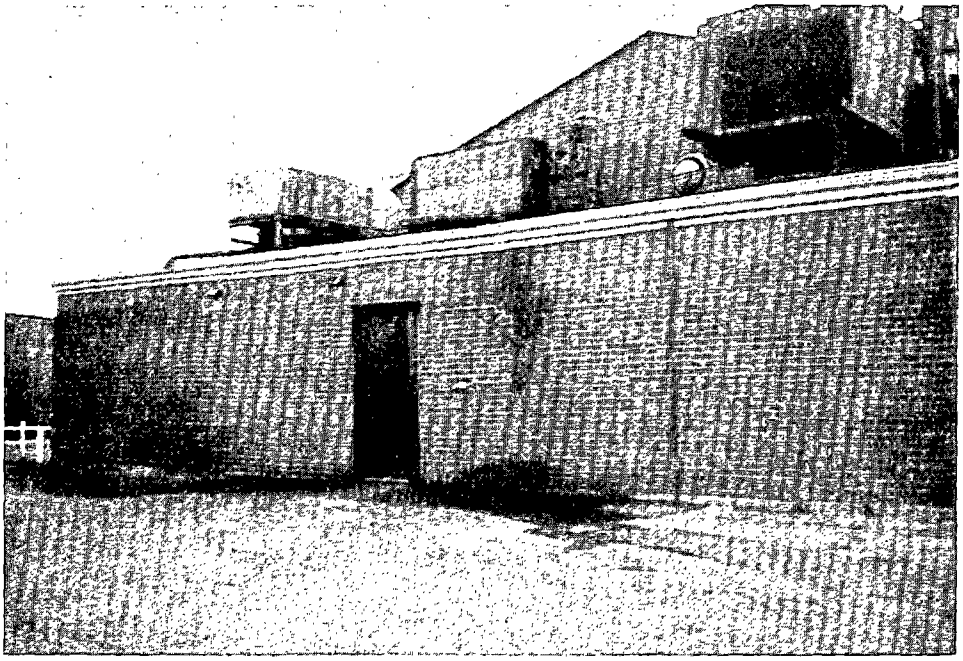


FIGURE 2.49 - SETTLEMENT CRACKS IN THE REAR WALL OF A RESTAURANT SITUATED ON FILL

2. Patterns of fill settlement can resemble mining subsidence--broadly distributed or regional-type patterns for extensive fills and dish-shaped patterns for smaller fills.

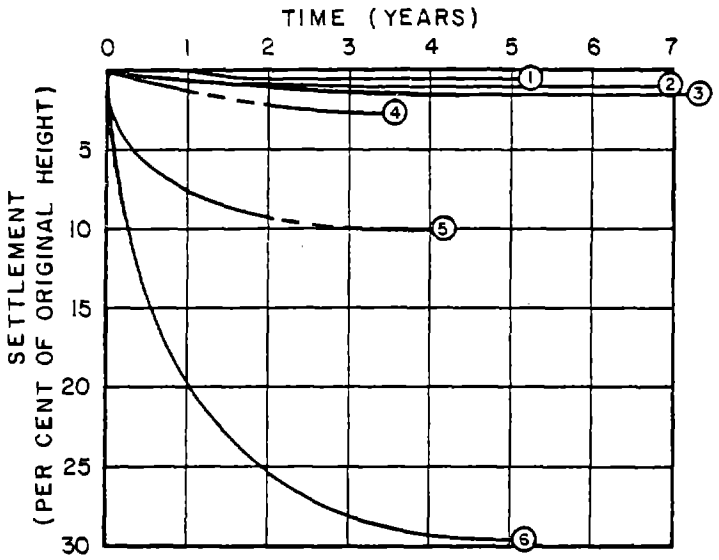
3. Artificial fills are used extensively in the Eastern Coal Province for land development. There is no question that a great many artificial fills lie above undermined areas, notably in the Pittsburgh-Huntington Coal Basin.

Although many fills today are well engineered, their construction is another matter. Poor construction practice that may lead to large post-construction settlement includes placement of improper materials (organic or waste soils and construction debris); placement of material too wet or too dry to achieve suitable compaction; and inadequate compaction, among others. Examples are given in the following paragraphs.

Fills may consist of anything. Engineered fills--those specifically designed and constructed for use as building sites--normally consist of compacted soil or rock. Settlement of various types of fill are shown in Figure 2.50. These curves show that the total settlement due to self weight is dependent on the fill material type, the degree of compaction and the total fill thickness. Other factors such as moisture content of the fill material can also influence settlement. The curves suggest that most fill settlement occurs within the first year of placement and is relatively small after two years. This is not always true, particularly in non-engineered fills.

As a general rule, settlements of well-compacted rock, gravel, sand, shale, and clay fills do not exceed 0.5 percent of their height (Bell, 1978). For a 10-foot height of fill, this amounts to about 0.5 inch. An uncompacted clay fill of similar height might settle 12 inches, according to Curve 5 in Figure 2.50. If the same fill should contain pockets of refuse, which is not uncommon in hastily constructed fills where no engineering supervision is provided, settlements could reach 30 percent, or 3 feet. Thus, the potential for differential settlement across a clay fill can be significant and is highly dependent on the variation of construction methods and fill materials.

Thousands of fills are well constructed and never experience damaging settlements. Probably the highest quality engineered fills are earthfill dams. Post-construction crest settlements of United States Bureau of Reclamation (USBR) dams average less than 0.2 percent of their height in the first three years after completion and less than 0.4 percent in periods up to 14 years (Sherard, 1963). The fill of a 100-foot high earthfill dam would therefore be expected to settle little more than 0.4 foot. The fine silt core of the 50-foot high Tustervatn Dam in Norway, which was constructed by the so-called Swedish wet compaction method, had settled two percent (one foot) in the first five years after construction. During construction (fill placement), earth dams con-



DESCRIPTION OF CURVES

NO.	STATE OF COMPACTION	MATERIAL
1	WELL-COMPACTED	WELL-GRADED SOIL
2	MEDIUM	ROCK FILL
3	LIGHTLY-COMPACTED	CLAY AND CHALK
4	UNCOMPACTED	SAND
5	UNCOMPACTED	CLAY
6	WELL-COMPACTED	MIXED REFUSE

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FIGURE 2.50 - OBSERVED SETTLEMENTS OF FILLS
(after Meyerhoff, 1951).

structed according to USBR practices are likely to compress between one and four percent. Well-compacted and wetted rockfill dams have experienced post-construction settlements similar to those of earthfill dams (0.1 to 0.4 percent of fill height). Post-construction crest settlements of dumped rockfill embankments may exceed one percent of the embankment height and may continue for 10 years or longer (Sowers, Williams, and Wallace, 1965).

A dam embankment is a special class of fill that would normally be constructed with much greater care than the average valley or lowland fill. The settlements described in the preceding paragraph should be regarded as the minimum or lower limit settlement for other well constructed fills that are underlain by relatively incompressible soils or rock.

At the other extreme are fills where either highly compressible fill or subsurface materials have caused large ground surface settlements. Taylor and Buchignani (1972) describe the settlement behavior of a debris dike fill placed over San Francisco Bay Mud. The fill consisted of demolition and packing materials, metal, waste paper, scrap lumber, tree trimmings, household discards, rock, brick, glass, broken pavement, and concrete. Soil was mixed with the debris so that at least 25 to 40 percent of the compacted fill consisted of inert material. The fill was mixed, placed, and compacted by bulldozers and rested on about 90 feet of soft, highly compressible bay mud. It is difficult to imagine a poorer combination of fill materials and subsurface conditions. An instrumented test section revealed settlements of two to four feet under a surcharge loading of about five feet of soil. A tension crack on the soil surcharge and heave of the bay bottom in proximity to the fill slope toe indicated that failure was impending. Even where the debris dike was not load tested, vertical settlements due to the weight of the fill exceeded four feet. These large movements are most likely a combination of compression and consolidation of the fill material, consolidation of the bay mud, and secondary compression of the bay mud through creep and viscous flow. This is a case where secondary compression will almost certainly contribute significantly to the ultimate total settlement. Numerous structures have been built on filled ground over the bay mud. The apparent absence of serious structural distress, as of 1972, would seem to indicate that structures can be designed to adjust to the creep movements.

Charles, Naismith, and Burford (1977) present settlement data for surface (opencast) mining sites at Horsley, England, that were backfilled with sandstone and mudstone fragments by end tipping and casting by dragline. The average silt content of the spoil was 10 percent. At one point where the fill depth was 46.5 meters, ground surface settlements over a 20-month period amounted to 37 mm (about 1.5 inches), or about 0.08 percent. Twenty months after the water table rose into the bottom eight meters of the fill, the ground surface settlement had increased to 70 mm, still only 0.15 percent. The maximum settlements measured in the area after 37 months were on the order of 0.5 to 500 mm, or about one percent of the fill

depth. These data are somewhat misleading in that settlement observations for the youngest fills were not initiated until three years after the fill had been placed. Kilkenney (referenced in Bell, 1978) reports that the settlements of opencast backfill at another site in England were 50 percent complete after two years. Charles, Naismith and Burford may not have measured the maximum settlements at Horsley.

Sowers (1968) reports that, according to the U. S. Department of Health, Education, and Welfare, sanitary fills of mixed composition typically settle from 10 to 30 percent of their original height under their own weight. Most settlement takes place in the first year or two, but settlement continues indefinitely at a much decreased rate.

A special case of fill settlement--one that has more geologic than engineering significance--is the natural consolidation ('compaction') of deltaic and alluvial sediments. Subsidence of the Mississippi River Delta is probably the best known example. Up to 20 feet of vertical movement has been recognized in natural levees, and one town in Louisiana abandoned in 1888 subsided as much as four feet over a period of 50 years (Allen, 1969).

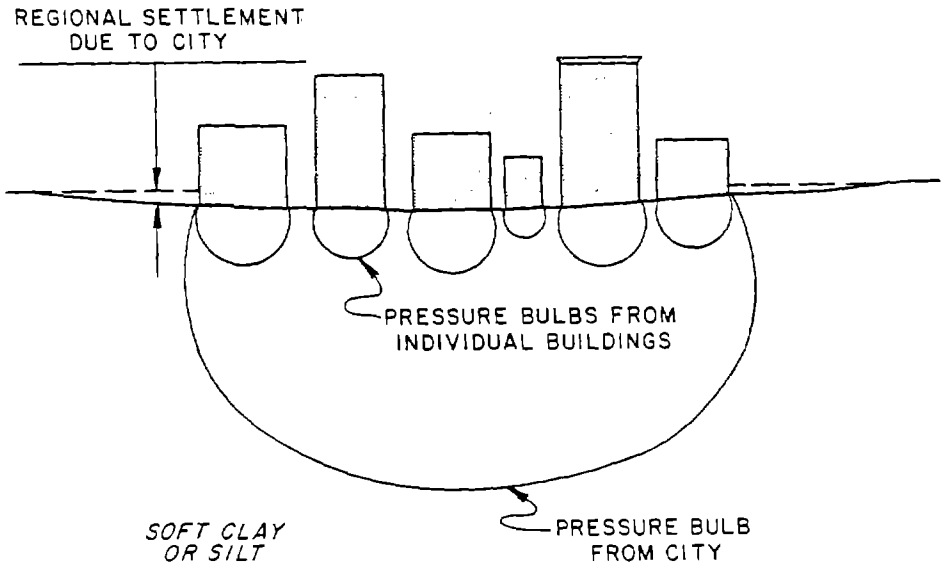
Perhaps the most common fill settlement that affects ordinary structures is that of backfills around retaining walls, foundation walls and foundations. Good practice dictates controlled compaction of backfill, but it is common practice to simply dump material without compaction. The settlements that result may lead to broken utility lines, and cracked pavements and sidewalks.

Settlements Due to Inconspicuous Loading Conditions. There exist a number of surface loading conditions that might not immediately be associated with ground surface settlements. Three such conditions are reviewed.

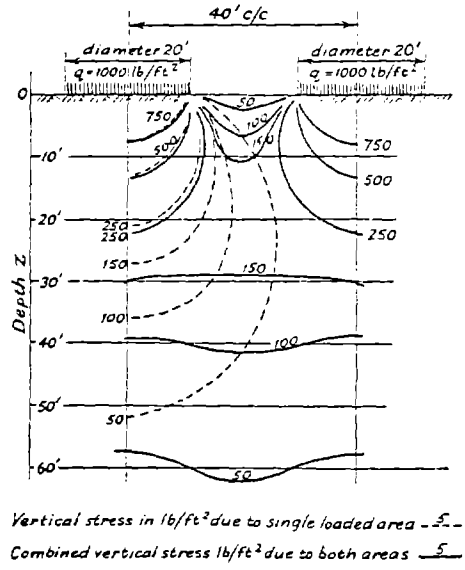
According to Terzaghi (1938), if an entire city or part of it is situated above a bed of silt or clay, a progressive general irregular subsidence of the loaded area should be expected. Terzaghi explained:

> "In Cambridge, Massachusetts, the entire area on both sides of Massachusetts Avenue, settled from 1869 to 1909 (a period of 40 years) through distances ranging between 0 and 2 feet." "A similar general subsidence was found to have occurred in San Francisco, California, in the district adjoining Market Street. From 1923 to 1928, the greatest subsidence was more than 0.2 feet."

This type of settlement is illustrated schematically in Figure 2.51. Pressure bulbs beneath individual buildings result in settlements that are characteristic of the loads transmitted to the subsurface by each building. Collectively, a group of buildings creates a pressure bulb that extends to a much greater depth than any individual pressure bulbs. It would be difficult to attribute



A.



B.

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FIGURE 2.51 – SETTLEMENT DUE TO COMBINED LOADS: a. REGIONAL SETTLEMENT OF A CITY LOCATED ON SOFT SOIL, b. EFFECT ON STRESS DISTRIBUTION OF COMBING TWO LOADED AREAS (after Mitchell, 1946).

such subsidence to surface loading alone if significant quantities of ground water were pumped from beneath the area.

Some evidence is available that suggests reservoirs behind high dams can also cause compression of the underlying rocks, a phenomenon that would superimpose settlements on those due to isostatic adjustments in the earth's crust. Tizdel (1970) reports that filling of the 100-meter high Bratsk Dam in Russia resulted in a sharp 70 mm (2.75 inch) increment in the settlement of the dam foundation as well as the appearance of a settlement cone about 2 to 3 km in diameter and joints opening in the foundation of the dam. Two kilometers from the dam, settlements of about 0.1 inch were observed after filling the reservoir. At a distance 350 meters downstream of the dam, the observed settlement was 9.5 mm (0.37 inch). The dam and reservoir are underlain by a thin layer of diabase, which is believed to act as a natural impervious apron over a thick, more compressible and permeable sandstone. Piezometers placed in the sandstone layer indicated a minor (10 meter) change in water level upon reservoir filling, giving credence to the hypothesis that filling caused an immediate or distortional settlement of the sandstone without any sustained increase of pore pressure.

Vertical settlements beneath and around material storage piles are normally of no concern, although lateral ground movements may be significant. Peck and Raamot (1964) describe lateral movements at 12 iron ore storage yards in the Great Lakes region and on the Atlantic seaboard. They point out that iron ore storage yards constitute some of the largest, most heavily loaded soil in the world. The concern for lateral movements arises out of the necessity to keep ore unloaders, ore handling facilities, retaining walls, and water front walls intact, reasonably straight, level and operable. It has been found that the plastic squeezing of clays beneath and around such yards can produce lateral movements as great as five feet and vertical settlements up to three feet. Some outright failures have occurred, resembling the bearing capacity failures described for fills in the previous section. Iron ore storage piles typically are 40 to 50 feet high and impose a load of 3 to 4 tons per square foot on the ground surface. Analyses of ground movements that take into account lateral flow must be utilized if the subsurface materials consist of clays.

Settlements of Unusual Structures. A number of structures, because of their shape, design or function, exhibit unusual settlement behavior. Two deserve special mention--steel storage tanks and chimneys.

Steel storage tanks are frequently placed directly on a prepared earth surface, eliminating the need for a concrete pad or raft foundation. Because of their inherent flexibility, the tanks transmit nearly uniform pressures to the subsurface and for this reason cannot "bridge" over soft or loose spots in the subsurface as can a rigid footing. Tanks therefore require special attention with regard to differential settlement to avoid weld cracks and buckling of the shells. Despite these potential failure modes,

tanks are frequently placed over relatively soft soils that are initially too weak to support them without large settlements that would be unacceptable for most other structures (Greenwood, 1974).

Davisson and Salley (1972) discuss the settlement of four large grain storage tanks in Kansas City, Missouri, founded on low to medium dense alluvial sand. The tanks were founded on a two-foot thick macadam base and had 3/16-inch thick steel floor plates. One of the tanks, 125 feet in diameter with 84-foot high walls, experienced the settlement pattern shown in Figure 2.52. The average net bearing pressure under the tank due to a full load of soybeans was 2.28 tsf. Despite the fairly large settlements that occurred on the first filling, the tank experienced no structural distress.

Sullivan and Nowicki (1974) report maximum edge settlements up to 46 cm (18 inches) and a center settlement of 83 cm (33 inches) for a 41-meter diameter steel oil tank founded on soft clay and sand in Colombia, South America. No damages were reported.

Green and Hight (1974) report the failure of two oil storage tanks caused by excess differential settlements believed to be 3 percent of the tank diameters.

Tall narrow structures such as chimneys and monuments are unusual in that they are sensitive to tilt and, therefore, to differential settlement. A 150-foot high brick smokestack in Connecticut had to be underpinned shortly after construction was completed due to compression of a peat layer under one side of the stack (Prentis and White, 1950). The 14-foot diameter stack, constructed on a spread footing, tipped 35 inches to one side and was moving at the rate of one inch per day.

Complications in Settlement Behavior. Numerous factors may complicate the settlement behavior of a structure, among them:

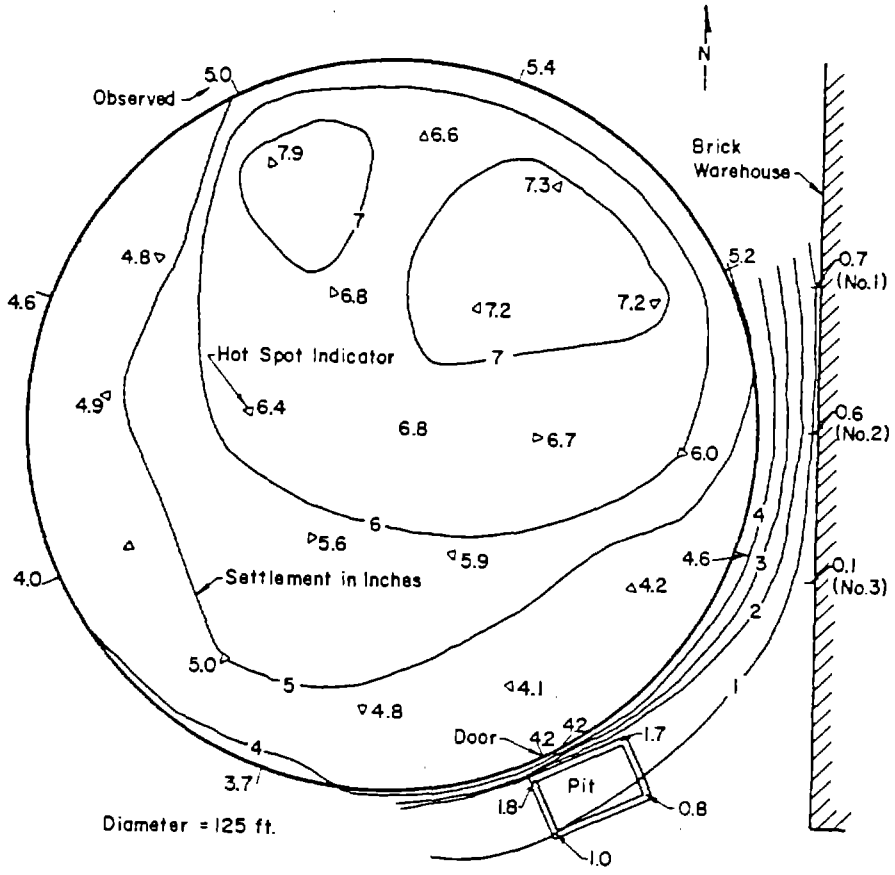
1. Multiple mechanisms of settlement, such as consolidation of a soft clay deposit and compression of a sand layer.

2. Multiple sources of ground movement due to the same surface loading, such as a partial bearing capacity failure and consolidation settlement of a building founded on soft clay.

3. Multiple sources of ground movement, due to different causes, such as settlement due to surface loading and subsidence due to underdrainage or withdrawal of fluids.

4. Movements due to the settlement of an adjacent structure.

Case histories are cited which demonstrate each of these complicating factors.



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FIGURE 2.52 - SETTLEMENT CONTOURS FOR GIVEN STORAGE TANKS FOUNDED ON SAND (after Davisson and Salley, 1972).

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1. Peck and Peck (1948b) analyzed the settlement of a 35-foot diameter, 60-foot high water tank founded on loose sand and soft clay at Salt Lake City, Utah. The clay contained many interbedded layers of sand. During the process of filling, the tank settled an average of about 0.4 foot. Two days later, the settlement had increased to 0.49 foot and after a week to 0.58 foot. After five years, the total settlement was 1.1 feet and increasing at a reduced rate. The observed settlement history clearly indicates that several mechanisms of settlement were involved. The rapid initial settlement of 0.2 foot was undoubtedly due to compaction of the loose sand that immediately underlies the tank. Because of the presence of many interbedded sand layers in the clay, a time period as short as two days or as long as three months would have been sufficient for primary consolidation of the clay. Yet, after one year the tank was still settling, indicating the likelihood of a secondary compression component. Because of uncertainties in the exact network of drainage paths in the sand layers which determine the time rate of consolidation, it was not possible to divide the time settlement record into distinct parts.

2. The famous Leaning Tower of Pisa in Italy, built in the period 1173-1350, is believed to be an example of multiple sources of ground movement due to the same surface loading. The tower is founded on shallow footings at a depth of only two meters below the ground surface, and is underlain by compressible clay strata. The original tilt of the Tower may have been the result of a partial bearing capacity failure during the early stages of construction. While the tilt has occupied the attention of tourists over the centuries, few are aware that the Tower has also undergone settlements of 6 and 12 feet (Bolt, 1975; McKinley, 1964). Differential movement across the 60-foot diameter of the foundation, probably due to both the partial bearing capacity failure and to subsequent differential settlement, is approximately four feet, a condition which gives rise to the infamous lean of 14 feet (Tower height is 180 feet). Despite these dramatic movements, the Tower has apparently not suffered severe structural damage. The settlements undoubtedly include distortion, consolidation, and secondary compression. The latter is still occurring, probably as a consequence of viscous flow or creep of the underlying soil material at a relatively constant volume (Bolt, 1975).

3. Multiple sources of ground movement give rise to complicated ground movements because two or more entirely different movements are superimposed on each other, and may be inseparable. Buildings founded on the soft, highly compressible volcanic clay subsoils of Mexico City are an example of this condition. The 165 feet of clay

that underlie the city are interbedded with layers of sand from which water has been pumped for over a century (Poland and Davis, 1969). Regional land subsidence resulting from consolidation of the clay under the reduced artesian head ranged from 16 to 23 feet over the period 1898 to 1956. Local settlements first occurred under the weight of ancient Aztec structures, then more recently from Spanish buildings. Today large structures founded near the surface undergo consolidation settlements due to their own weight as well as to consolidation from ground water withdrawal. The Palace of Fine Arts, completed in 1934, has settled ten feet and in 1962, was still settling at the rate of 1.5 inches per year relative to the outside (Legget, 1962). Although efforts are being made to recharge the ground water system by injecting water into the subsurface, the settlement of the structure is complicated by the ongoing secondary compression of the clays under the influence of the previous ground water withdrawal and the present surface loading.

4. Existing buildings may be affected, even damaged, if they lie within the settlement depression of new buildings. Particularly susceptible to such influences are tall structures that experience tilting, as shown in Figure 2.53. Movements due to construction excavation for the foundations of the new building may worsen the problem.

The zone of influence for the condition shown in Figure 2.53 depends upon the subsurface soil profile as well as the dimensions of the loaded surface area. Terzaghi and Peck (1967) cite a case from New York in which a 20-story building was constructed on a lot between two existing 7-story buildings supported by spread footings on a deposit of fine sand. The new building settled uniformly 1.8 inches and remained intact. The neighboring buildings were damaged by shear cracks and by distortion of door and window frames. As might be gathered from the case histories described in this section of the report, settlements due to surface loading are quite frequently the result of more than one factor.

Prediction of Settlement under Surface Loads

General Considerations. The settlement of the ground surface under the action of surface loads is dependent upon two factors:

1. The magnitude and distribution of loads transferred to the ground by the structure; and

2. The stress-deformation characteristics of the subsurface materials.

In comparison to the difficulties described for predicting other ground movements, settlement prediction under surface loads is somewhat advanced, although not without its limitations.

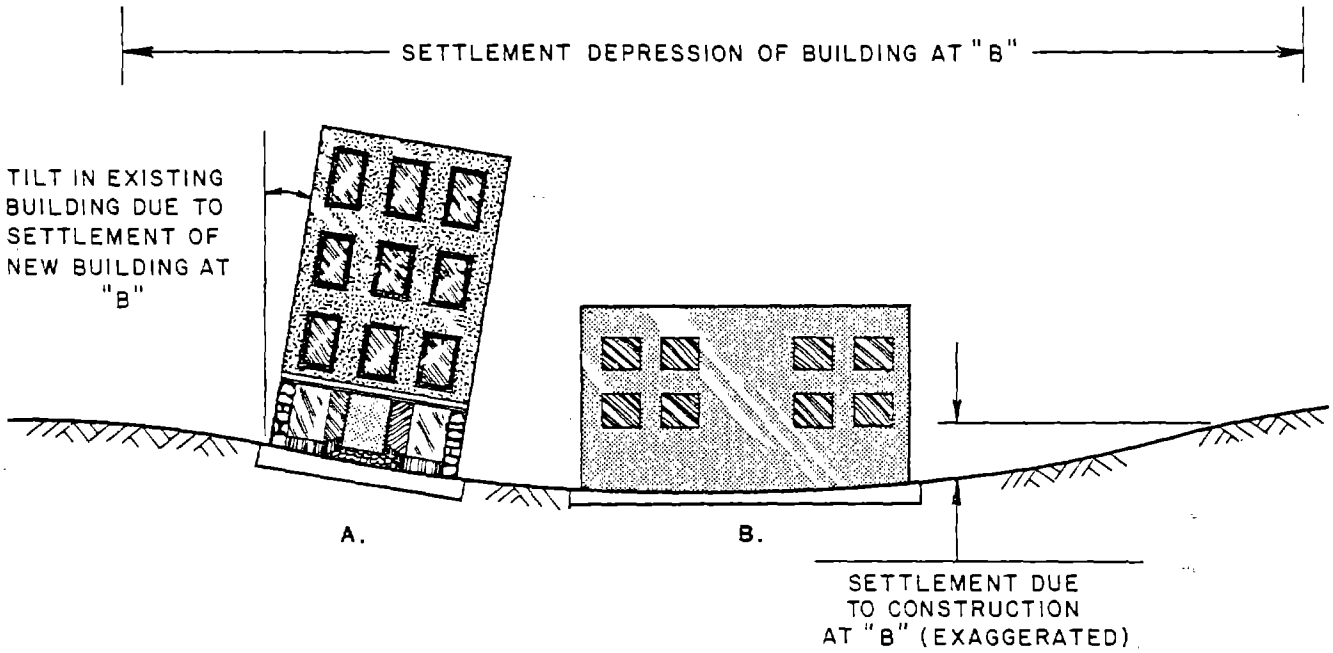


FIGURE 2.53 - INFLUENCE OF SETTLING STRUCTURE ON EXISTING STRUCTURE

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Sources of Difficulties. The principal difficulties in predicting settlements are as follows:

1. The magnitudes of the loads imposed by the structure may not be accurately known. Loads used by the structural engineer to design a building are rarely the same as the real loads, which may be difficult to estimate and may partly be transitory.

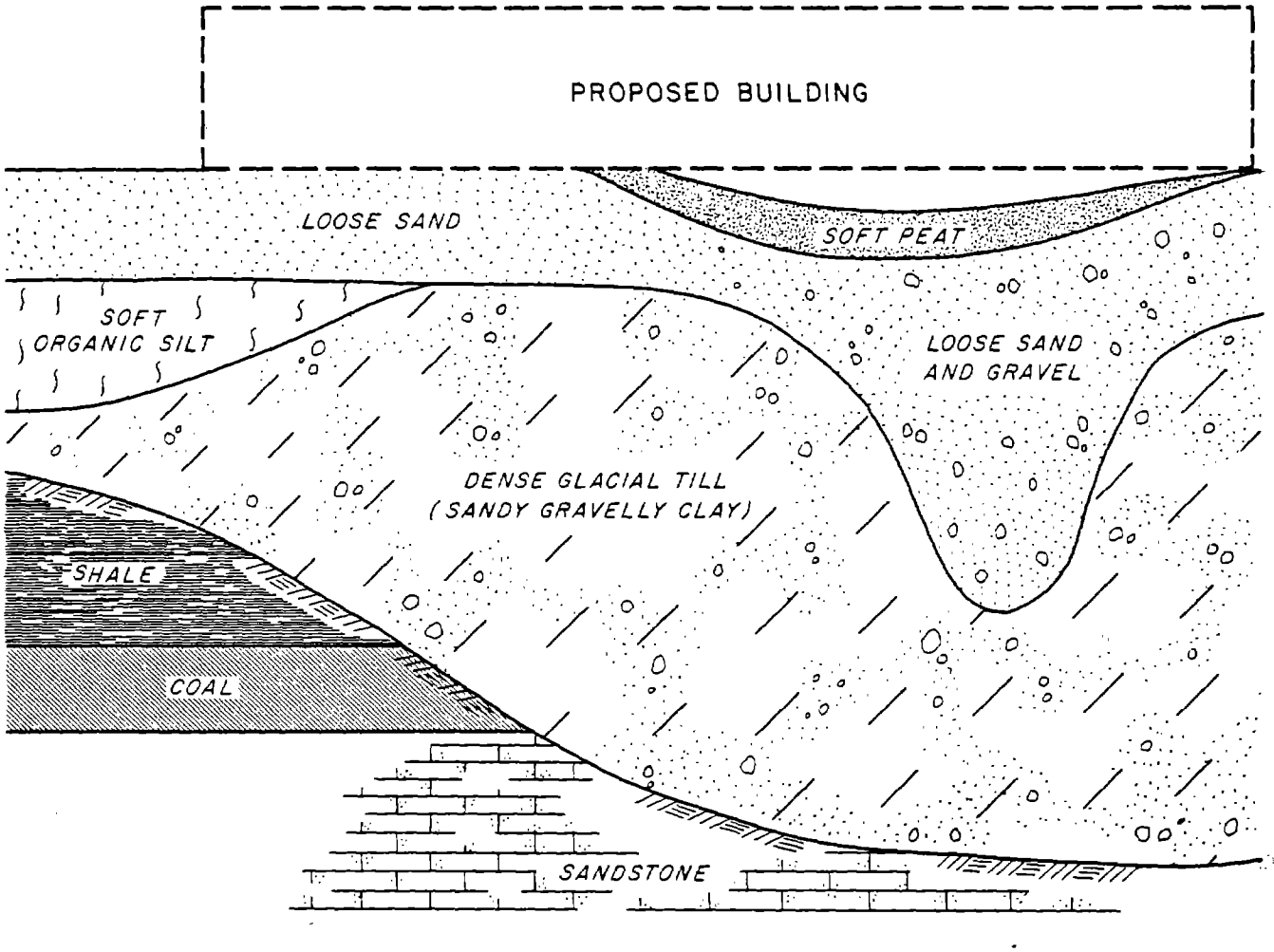
2. The manner in which loads on a structure are transferred to the ground surface is usually complex and depends on the stiffness of the structure, its foundation, and the underlying ground. For an elementary structure, such as a flexible steel oil tank, the transfer of load to the ground is relatively simple. By comparison, the load transferred to the ground beneath a semi-rigid concrete mat subjected to a series of concentrated, line and distributed loads is very complex. The average bearing pressure, obtained by dividing the sum of all structural loads by the foundation area, generally is only a rough representation of the actual pressures transferred to the ground.

3. The distribution of imposed stresses within the ground is difficult to evaluate with any exactitude. A few subsurface materials behave as ideal homogeneous, isotropic substances, in which case simple theories of mechanics may be applied to calculate stress distributions. More often, subsurface materials are somewhat heterogeneous and anisotropic. Sophisticated stress distribution theories are available, but are cumbersome and are rarely employed.

4. The subsurface profile beneath a structure may be difficult to characterize with regard to the distribution, orientation and extent of soils, boulders, rock, groundwater and artificial objects. The complexity of this problem is illustrated in Figure 2.54, a subsurface cross-section in a glaciated area. Variations in composition can also exist in the third dimension.

5. If the subsurface profile is complex, determination of the stress-deformation characteristics of each component in the profile can be expensive.

6. There are no universally acceptable methods to measure stress-deformation characteristics of subsurface materials. Although the in situ behavior of some soils can be characterised reasonably well by laboratory tests on small samples taken from boreholes, sample disturbance generally influences test results to some degree and reaches unacceptably high levels for cohesionless materials like sand. The stress-deformation characteristics



Note erratic variations across area of proposed building: soft silt under one end, soft peat, loose sand and gravel under the other end, dense glacial till under the center. Prediction of settlement of the building would be difficult.

FIGURE 2.54 - SUBSURFACE CROSS-SECTION IN GLACIATED AREA

of materials like rock may be governed by geologic features such as joints and faults for which neither laboratory nor in situ testing may be feasible.

These six problems are normally handled as follows:

1. The magnitude and distribution of structural loads are estimated where they are not known.

2. Load transfer from the foundation to the ground surface is assumed to occur uniformly or according to some other simple distribution.

3. The distribution of stress in the ground is assumed to occur in accordance with simple elastic theory.

4. The subsurface profile is interpreted from the results of available borings, test pits, soundings and other subsurface exploration techniques.

5. The interpreted subsurface profile is partitioned into zones of material whose stress-deformation characteristics are considered to be approximately uniform.

6. Stress-deformation characteristics for each zone are approximated from what is hoped to be a reasonable number of laboratory or in situ tests.

The process of predicting settlements clearly involves many estimations, assumptions, simplifications, interpretations, and judgments. According to Peck (1965), the greatest errors in settlement analysis occur as a consequence of misjudgment of loading or of the subsurface profile, and not of inadequacies in theory. For practical purposes, then, emphasis ought to be placed on items (1) and (4). Peck explains,

> "By and large throughout the world, soil conditions are erratic rather than homogeneous. Yet the implications of the heterogeneity of soil deposits are still not properly appreciated. Few soil deposits are uniform enough to warrant an elaborate investigation of their properties."

Beyond the engineering stage, the reliable prediction of settlements may be marred by construction defects such as poor construction practices and poor construction materials. Good plans, specifications and construction inspection are as much a part of reliable settlement prediction as are the factors discussed in the preceding paragraphs.

Analytical Methods. Soils are typically divided into two broad categories: clay and sand, or similarly, cohesive and cohesionless. Different methods of analysis are used for each. Peat, silt and fill do not consistently fit into either of the categories, so that methods are generally tailored to the particular soil type, as

they are for rock. Conventional methods to analyze settlements in clay, sand and rock are discussed separately in the following paragraphs.

Clay. The total settlement experienced by structures on clay is the sum of three components: the immediate (distortion) settlement, (2) the consolidation settlement and (3) the secondary (creep) settlement. Although the individual components are computed separately, all three generally occur simultaneously. Immediate settlements are normally calculated by applying stress-deformation relationships from the theory of elasticity. Excellent reviews of these methods are contained in articles by Simons (1974) and Butler (1974). The prime concern in computing immediate settlement is the method of evaluating the undrained modulus of the clay. No universally satisfactory method exists and experience indicates that the calculated immediate settlement will vary by a factor of 5 to 10 depending on the method used to determine the modulus. One interesting finding based on field observations is that the ratio of immediate settlement to total settlement ranges from 0.077 to 0.212 for normally consolidated clays and from 0.315 to 0.735 for over-consolidated clays. (Simons, reported in Bell, 1978).

Consolidation settlement was first recognized by Terzaghi in the 1920's. His theory is still widely used today and is described in most textbooks on soil mechanics. Modifications to the original theory have been made by numerous researchers to account for sample disturbance and for pore pressure variations different from those assumed by Terzaghi. These modifications, together with interpretations and applications of consolidation concepts are discussed by Simons (1974), Butler (1974) and Poulos (1975). The original Terzaghi theory has given fairly reliable results in actual practice when the subsurface conditions approximate those around which the theory was developed. The modification proposed by Skempton and Bjerrum (Butler, 1974) has given the method even broader application.

Burland, Broms and deMello (1977) point out that for soils that are approximately "elastic" in their response to vertical loads, probably including a wide range of overconsolidated clays, the classical Terzaghi one-dimensional consolidation analysis can be used to calculate the combined immediate plus consolidation settlement. For soft "yielding" soils (normally consolidated clays), the Terzaghi method provides only the consolidation component of total settlement to which must be added the immediate component.

Generally, the estimate of consolidation settlement is accompanied by an estimate of the time-rate of settlement. The settlement rate can be highly variable, depending on the permeability of the soil and the presence of sand or gravel layers in the subsurface profile that hasten drainage of soil pore water pressures built up under the surface loading. The accuracy of time-rate predictions is strongly dependent on the veracity of the subsurface

profile. Simons (1974) points out that the observed rates of settlement of structures are almost always more rapid than the rate calculated from one-dimensional theory based on laboratory oedometer tests on small samples. Large scale laboratory tests, in situ permeability tests, and two- and three-dimensional consolidation theories are suggested for accurate predictions of settlement rates.

Stress path testing is a concept introduced by Lambe (1967) for predicting settlements from laboratory triaxial tests. Lambe proposes that a series of laboratory test specimens be subjected to stress changes equal to those the soil will experience under actual field loading. This method is particularly suitable for cases where the areal extent of the applied load at ground surface is small compared to the thickness of the consolidating soil layer, making the applied load non-uniform with depth, contrary to the requirements of Terzaghi's one-dimensional theory. The technique requires rather exacting triaxial testing procedures as well as knowledge about the initial in situ stress conditions that exist prior to surface loading. The prediction of settlements of structures by the stress path method has proven reliable in many cases. Simons (1974) reports results for heavily over-consolidated clay.

An important factor that strongly influences consolidation settlement predictions regardless of method is the time-sequence of loading. Loads are almost never applied instantaneously. They change as construction proceeds, as well as throughout the life of a structure. Consolidation settlements depend on the past loading history, and it is impossible to predict reliably the consolidation settlement if this sequence is not known.

No reliable, universally applicable methods have been developed for the prediction of secondary settlement. Experimental, empirical and theoretical approaches have been employed to characterize the phenomenon, but success has been limited. Bjerrum (referenced in Simons, 1974) suggests that structures subjected to large variations in live load, such as silos, storage tanks and high structures under wind action, may experience larger secondary settlements than structures subjected to static loading. Secondary settlement is generally ignored in analyses, and local experience is the best guide in evaluating its magnitude.

Sand. Because of the difficulties inherent in obtaining undisturbed samples of sand, methods for predicting the settlement of structures on sand are based on field tests. An excellent review of this subject is given by Sutherland (1974). Concise summaries are given by Jorden (1977) and Arnold (1980). Until recently, field plate load tests were regarded as the most reliable method for estimating settlements in sand. However, the extrapolation of results from a small-scale test to a full-scale prototype footing is not always reliable.

Two forms of field penetration tests are used for cohesionless soils: the standard penetration test (most common in the United

States) and the Dutch Cone test. The former is a dynamic penetration method; the latter is a static penetration method. Both rely on empirical correlations derived from the penetration resistance of the instruments driven. Terzaghi and Peck (1948) proposed empirical relationships between the standard penetration resistance and the allowable soil pressures for footings of various sizes whose settlements would equal one inch. This correlation was established from load tests observations. Since 1948, a number of modifications have been made to reflect the actual behavior of foundations and to correct for various factors such as overburden pressure, the position of the ground water table, and footing depth and width. The original "black box" approach has been preserved--the method does not attempt to calculate settlement from an assumed stress distribution and stress-deformation parameters. The Dutch Cone method involves correlations between the cone resistance and various measures of sand compressibility. The compressibility is then used in various stress-strain calculations to arrive at settlement.

Most predictive methods regard settlement as an instantaneous process. Schmertmann (1970) has introduced a creep factor. Time effects may derive from cyclic loading and resultant densification of the underlying sand.

Burland, Broms, and deMello (1977) attempt to circumvent the complications of penetration test result interpretation by developing envelope curves for plots of footing breadth versus the ratio of settlement to average bearing pressure. The curves, based on measured foundation settlements and load tests, are suggested for medium and dense sands. Insufficient data exist to establish a curve for loose sand. The method is similar to that of Terzaghi and Peck. No distinction is made for the position of the ground water table beneath the foundation, a factor which is regarded by some authorities as important in determining settlement behavior (Peck, Hanson and Thornburn, 1974). Burland, Broms and deMello (1977) suggest that the method of Schultze and Sherif (1973), based on the standard penetration test, offers one of the more thorough approaches to estimating settlements on sand. Sutherland (1974) suggests the method proposed by Meyerhof (1965) generally provides a satisfactory estimate of maximum probable settlement.

A most informative overview for methods of settlement prediction on sand comes from a study by Simons, Rodrigues and Hornsby (1974), who compared observed settlements of six different structures with settlements predicted by eight modifications of methods based on the Standard Penetration test and the Dutch Cone test. The ratio of predicted to observed settlements ranges from 0.1 to 4.8. Some methods consistently underestimated settlements; some consistently overestimated settlements; most underestimated in some cases and overestimated in others. Petley and Bell (1978) summarize very appropriately:

> "In view of the wide scatter in results obtained from the various methods it is important that undue reliability is not

placed on any one method. It would appear to be wise to calculate settlements based on as many different methods as possible, in order to assess the possible range in uncertainty."

Rock. Settlements of structures founded on rock are usually predicted with the aid of the theory of elasticity and either large scale in situ load tests or empirical correlations between rock compressibility and some relatively simple index of rock quality. The approach is basically the same as that used to calculate the immediate settlement of foundations on soil. By introducing into the analysis values of compressibility that reflect the presence in the rock mold of joints, faults, seams and other geologic discontinuities, settlement can be estimated, at least roughly.

Over the past 20 years, rock engineers have been searching for inexpensive ways to empirically relate rock type and brokeness to rock mass compressibility. Rock mass compressibility is sometimes an order of magnitude less than that of the intact rock and is closely related to the orientation and spacing of the discontinuities, their openness, and the character of any infilling. If the joints are tight or are not open more than a fraction of an inch, the compressibility is reflected by the Rock Quality Designation, as defined by Deere (1966). Hobbs (1974) has extended Deere's work to British rocks. Bieniawski (1978) provides an alternative Geomechanics Classification Rating for relating rock mass characteristics to the compressibility of the mass.

Priest and Hudson (1981) discuss statistical methods for characterizing the distribution of joint spacings and RQD in one-, two- and three-dimensions using data from core borings and rock exposures.

Kulhawy (1978) presents a rational geomechanical model for estimating the settlement of isotropic, transversely isotropic and orthogonally jointed rock masses using rock mass compressibility values based upon the characteristic discontinuity spacing (or, alternatively, the RQD) and the corresponding ratio of the modulus of the intact rock material to the normal stiffness of the discontinuities. Kulhaway's approach seems promising, but at the present time there are few documented cases to attest to its reliability.

In closing the discussion of predicting settlements under surface loads, the remarks of Karl Terzaghi delivered at the First International Conference on Soil Mechanics and Foundation Engineering in 1936 are as true today as they were then:

"Whoever expects from soil mechanics a set of simple, hard-and-fast rules for settlement computations will be deeply disappointed. He might as well expect a simple rule for constructing a geological profile from a single test-boring record. The nature of the problem strictly precludes the possibility of establishing such rules" (Terzaghi, 1936).

Given this perplexing situation, it is not too surprising that experience and "engineering judgment" (the latter frequently amounting to an outright guess) are widely resorted to. Completing Terzaghi's message:

> "If a supervising or construction engineer wants to enjoy the benefits of recent developments in this field, he should first of all study the rules for securing reliable settlement records and then start to observe the buildings of his district. After he has done this for a certain period he will discover for himself the value of the information which he can obtain from soil mechanics." (Terzaghi, 1936).

TECTONIC MOVEMENTS OF THE EARTH'S CRUST

General Considerations. All sources of ground movement discussed to this point occur at or near the surface of the earth. With the exception of fluid extraction, which might take place at great depths in the case of oil and gas, the movements take place as a result of stress or strength changes in the near-surface earth materials, even though the cause of the movement may not be surficial. Landslides, for instance, may originate from disturbances at the ground surface, such as construction excavations, or may be triggered by deep-seated earthquakes. In either case they are surficial phenomena.

Some ground movements, unlike those described previously, are merely surface expressions of deep-seated movements of the earth's crust. They are permanent displacements of crustal material that result in changes to the structure of the crust and are sometimes referred to as tectonic movements. Beloussov (1962) discusses three basic types of tectonic movements:

1. Oscillatory movements, manifested as upwarping and downwarping of the earth's crust;

2. Folding movements manifested as crumpling of the strata of the crust into folds; and

3. Rupturing movements, manifested as fracturing of the crust.

For sake of clarity, scientists generally regard the earth's "crust" as that portion of the earth which lies above a seismic discontinuity known as the Mohorovicic' discontinuity. This feature has been found by geophysical seismic methods to lie at a depth ranging from 20 to 70 km beneath the continents and 10 to 12 km below sea level beneath the oceans.

Tectonic movements are discussed in this report for the sake of completeness and because some have been observed and recorded within the coal mining areas. An awareness of tectonic movements would seem useful in considering the possibility that mining subsidence might be influenced by certain forms of tectonic movements, particularly earthquakes.

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Oscillatory Movements. The most common oscillatory movements are those related to isostatic adjustments of the earth's crust which are occuring as a result of the retreat of glaciers 8,000 to 11,000 years ago. Isostasy refers to the state of mass compensation that maintains equal hydrostatic forces in the earth (Andrews, 1974). In the Airy model, isostasy is considered to be a theoretical balance of all large portions of the earth's crust that occurs as a result of slow subcrustal flow of mantle material in response to changes in crustal load at the earth's surface (Verhoogen, 1970). The retreat of continental glaciers from North America and Northern Europe is firmly related to significant crustal rebound in these areas. In Scandinavia, present rates of rebound range from zero to ten millimeters per year (Donner, 1974). In the United States contemporary crustal deformations, calculated from tide-gage records, range from zero at the southern limit (Lake Erie) to three millimeters per year in Northern Michigan (Flint, 1957).

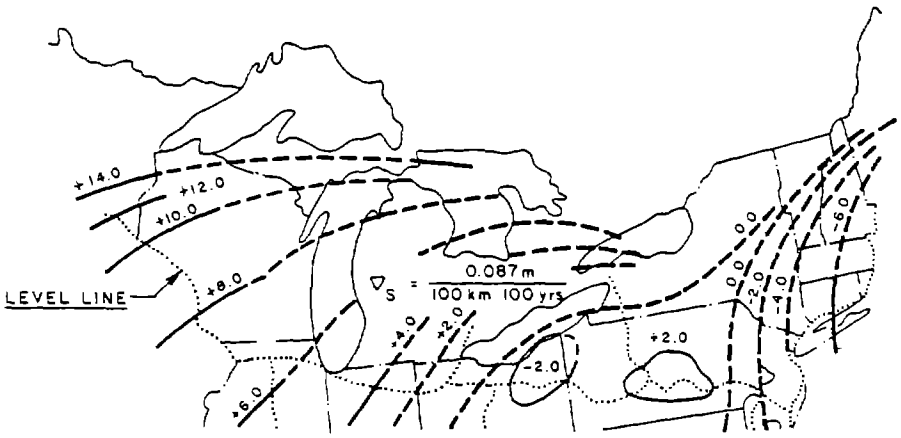
Beloussov (1962) gives data for the Atlantic Coast of North America that indicate the coastline is subsiding on the order of 0.17 to 0.22 inch per year between Boston and Charleston. No explanation is given for this apparent movement. If the coastline submergence is actually due to crustal subsidence, then portions of land between the east coast and the Great Lakes should be experiencing a slow warping. Hands (1976) cites data from geodetic releveling surveys conducted in 1955 to demonstrate that crustal warping has indeed occurred in this area in the period 1929-1955, and is on the order of 0.2 inch per mile per 100 years across western Pennsylvania (Figure 2.55).

Small crustal movements, of the type described in previous paragraphs, have been studied extensively in the Soviet Union because of their possible effects on large hydroelectric development in that country. In the Eastern and Interior Coal Provinces, isostatic crustal movements apparently have not yet influenced human activities. In the Lake Michigan basin crustal movements have been related to coastal submergence and severe lake shore erosion (Hands, 1976).

It is worth noting that isostatic adjustments of the earth's crust have been observed as a result of human activities. Scott (1978) regards the regional settlement associated with the creation of Lake Mead as an isostatic adjustment. The settlement in the center of the reservoir area behind Boulder Dam amounted to seven inches (Sherard, 1963).

The implications of crustal warping and isostatic adjustments as possible mechanisms for inducing or influencing subsidence above mined areas has, to the best of the writers' knowledge, not been considered.

Folding Movements. Folds are undulations in rock strata that may vary in width from a few inches to many miles (Billings, 1954). Although they are most commonly associated with mountains, folded rocks underlie many portions of the world that today have little or



Contours indicate movement in mm/yr
over the period 1929 - 1955.
Plus is uplift, minus is subsidence.

SCALE
50 0 100 200
(Miles)
50 0 100 200 300
(Kilometers)

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FIGURE 2.55 - CRUSTAL MOVEMENTS BASED ON GEODETIC RELEVELING
(Data after Meade, referenced in Hands, 1977)

no topographic relief. Present day folding movements that are reflected on the ground surface are similar to oscillatory movements in their rate and general nature (Beloussov, 1962).

Rupturing Movements. Actual failure or rupture of the earth's crust is associated with the geologic processes that have been responsible for the formation of the earth's crust. Although these processes have produced dramatic deformations of the crust in the geologic past, relatively few of them are currently producing ground movements of consequence to the engineer.

The horizontal movements of the earth's outer shell that have been used to explain the plate theory of tectonic development lie in the range of one to 10 centimeters per year (Verhoogen, 1970). Vertical movements resulting from horizontal movements are not yet thoroughly understood by geologists (Hobbs, Means, and Williams, 1976). Scott (1978) reasons that the increasing separation of the American and African continents by two centimeters per year must be accompanied by vertical deformations. The impact of such surmised movements on subsidence mechanics seems minimal.

Earthquakes represent the principal source of rupturing movements significant to human activities. Permanent ground movements associated with earthquakes include uplift, subsidence and faulting. Although faulting is usually the only observable rupture movement in an earthquake, subsidence and uplift have, in at least one instance, been related to crustal and subcrustal rupture.

In the Good Friday earthquake that shook Alaska in 1964, deformation of the earth's crust was more extensive than any yet studied in a single earthquake. At least four different sources of ground movements were caused by the earthquake:

1. Subsidence, uplift and faulting due to tectonic movements;

2. Subsidence due to densification of soils;

3. Subsidence due to extrusion of materials from the earth; and

4. Landslides, mainly translational and rotational movements, due to liquefaction of sand seams and layers.

Vertical displacements included uplifts as great as 11 meters and settlements as great as two meters (Bolt, 1975). These movements occurred over a 65,000 square mile coastal area and have been attributed to low thrusting southward of Alaska over a fracture lying along a submarine trench. Two surface faults from the earthquake were observed. Vertical displacements of six meters were measured on the longer of the two. A particularly relevant characteristic of ground movements caused by earthquakes can be gained from observations of subsidence at several points in the deeply-filled alluvial valleys of the Kenai peninsula. The total ground

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surface subsidence at a well casing near Homer, Alaska, was observed to be 4.5 feet (Figure 2.56). The casing had been installed into firm rock at a depth of 468 feet before the earthquake, and projected one foot above the ground surface. After the earthquake the casing projected 3.5 feet above the ground surface, indicating a decrease in thickness of the soil layer by 2.5 feet. These observations suggest that tectonic movements caused the rock surface to be lowered by two feet, which together with the 2.5 feet of movement due to soil compaction, resulted in a total ground surface movement of 4.5 feet (Grantz, 1964). In another area, four feet of tectonically-induced rock settlement and four feet of soil densification led to a total surface settlement of 8 feet. Two obvious conclusions can be drawn from these observations:

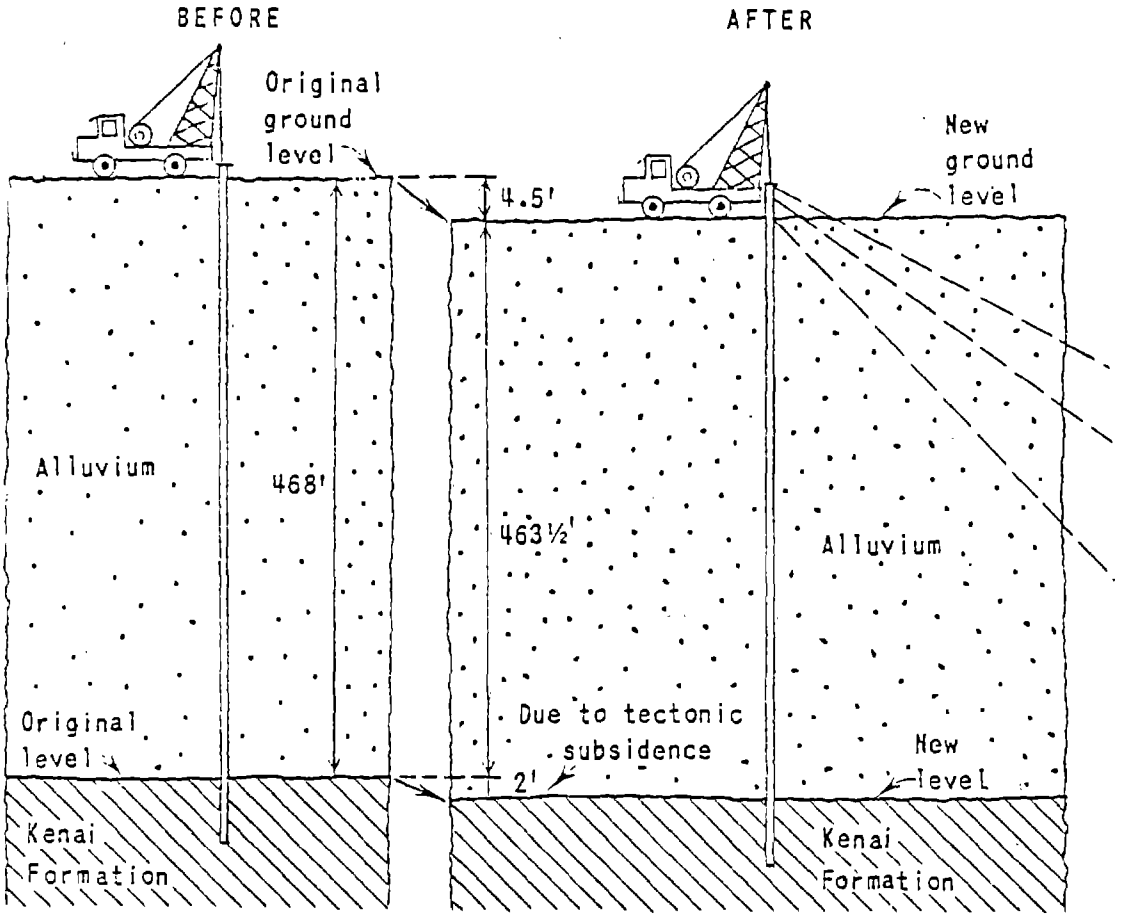
1. Multiple modes of ground surface movements can arise from one cause; and

2. Overburden soil characteristics influence the ground surface manifestation of deep-seated subsidence.

The writers feel that this observation is relevant to the question of the influence of overburden characteristics on ground movements caused by mining subsidence.

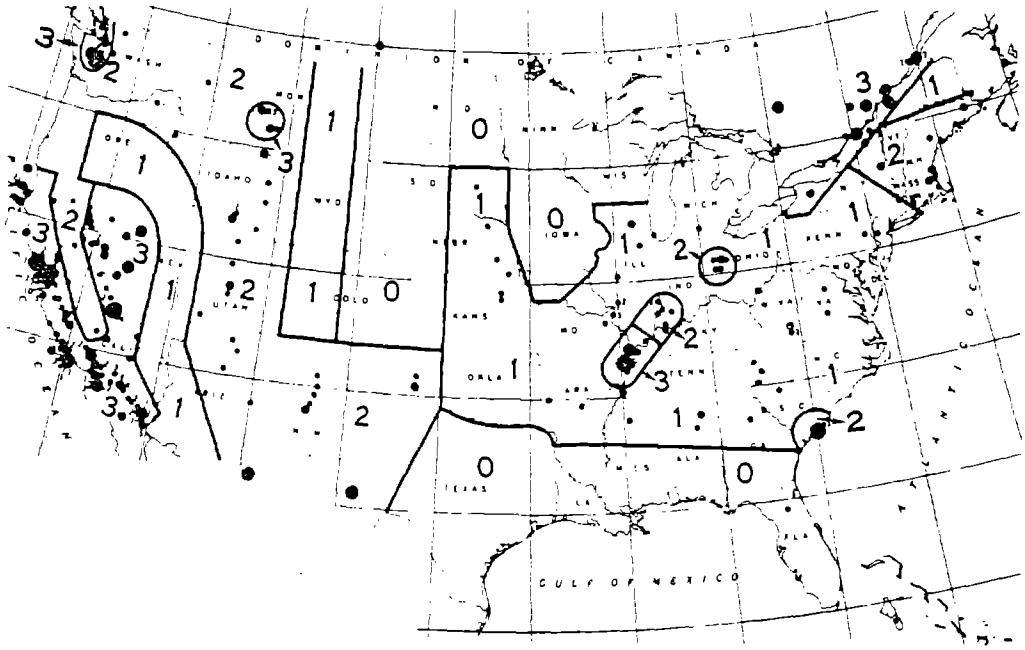
The potential significance of earthquake-induced ground movements is indicated by the seismicity map for the Eastern United States shown in Figure 2.57. Noteworthy are the concentrations of earthquake events in the Southern Illinois-Indiana area and in west-central Ohio. According to Bolt and others (Bolt, 1975), active fault zones in the central United States are not well delineated. They claim there is no definite evidence of surficial fault breakage for any central or eastern United States earthquake. However, this does not preclude the existence of subsurface faults nor does it preclude the possibility of surface subsidence or uplift associated with subsurface rupture movements.

The most significant earthquake is the New Madrid earthquake that occurred in the southern Illinois-southeastern Missouri area of the central Mississippi Valley in 1811. Although accounts of this event are primarily verbal, it is well-established that significant permanent ground movements took place. Fuller (1912) describes uplifts and doming, depressed "sunk lands", and areas of marked fissuring over a 100-mile long stretch of flat river bottom land. The uplifts and doming appear somewhat conjectural, but ground surface subsidence on the order of 10 to 20 feet is apparently confirmed from several accounts and field studies. Fuller, in 1912, attributed the "sunk lands" to "a warping and lateral movement incident to a subsidence in the floor of the basin," and to extrusion of quick sand along creek banks. Subsidence of the basin floor might have been associated with movements of tectonic origin. In view of the extensive complex deposits of unconsolidated loose alluvial soils in the valley of the lower Mississippi River (Kolb and Shockley, 1959), densification due to shock from the earthquake could also have been a significant source of subsidence.



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FIGURE 2.56 - LAND SUBSIDENCE ON THE HOMER SPIT, KENAI PENINSULA, ALASKA. ALLUVIUM COMPACTED 2.0 FEET DURING EARTHQUAKE OF 1964 (after Grantz, 1964).



NOTE CONCENTRATIONS OF EVF EVENTS IN SOUTHERN ILLINOIS-INDIANA AREA AND WEST CENTRAL OHIO. NUMBERS 0 TO 3 INDICATE SEISMIC RISK ZONES (from Bolt,et,al. 1975).

FIGURE 2.57 - EPICENTERS OF SOME MAJOR U.S. EARTHQUAKES COMPILED FROM U.S. COAST AND GEODETIC SURVEY DATA.

The mechanics of earthquakes is not well-understood. According to Okamoto (1973), two principal explanations have been offered:

1. Sudden changes in the earth's crust due either to forceful entrance of magma into the semi-hardened crust or to abrupt variations in the magma; and

2. Sudden releases of strain energy stored in the crust on account of orogenic (mountain forming) forces produced by convection within the mantle.

Variations of these basic explanations have been advanced and apparently have merit in certain geologic environments.

The filling of large reservoirs behind dams is known to cause significant earthquake activity (Housner, 1969). Since the filling of Lake Mead in 1935, over 10,000 earthquakes have occurred in the area, about 1,000 of which have been felt by local residents (Bolt, 1975). There was no record of earthquake activity in the area prior to that time. Since the Lake Mead experience, seismic events have been recorded at 15 dam sites around the world. Coupled with the isostatic sinking that may accompany a filled reservoir, the seismic activity could be responsible for an unusual combination of ground movements of different types. At Lake Mead, for instance, it has been suggested that observed downward movements have been caused by renewed movements along a pre-existing fault, which, in turn, may have caused the earthquakes.

Types of Tectonic Ground Movement and Damage. Most tectonic movements, such as regional subsidence, would appear translational, particularly when ground surface rupture does not occur. When ground surface rupture does occur, the overall movements may still be of a translational nature, but with local distortional movements in proximity to the fault scarp.

Building damage in the case of regional subsidence or uplift due to tectonic movements could be insignificant or nonexistent if large areas translate uniformly. Major damage or total distruction is likely to result if buildings are located on surface faults that undergo either vertical or horizontal displacements. In the case of earthquakes, however, most damage usually occurs from: 1) transient motions; 2) landslides; or 3) soil liquefaction, all of which may be induced by ground shaking and not from permanent ground displacements directly related to tectonic movements.

Prediction of Ground Movement due to Tectonic Processes

The analysis or prediction of tectonic movements in the earth's crust is in a far more primitive stage than the engineering analysis of other sources of ground movement described in this review. In fact, not even the mechanisms of tectonic movements are fully understood. Predictions of the magnitude of tectonic movements are not possible at this time. Much effort has been expanded

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in the prediction of earthquakes. Yet, scientists are still far from predicting the occurrence of these events much less the magnitudes of movement to be expected.

MISCELLANEOUS SOURCES OF GROUND MOVEMENT

Three rather unusual sources of ground movement that deserve consideration involve the chemical change of subsurface materials either by physical or biological processes.

Subsurface Combustion. At least three situations exist where subsurface combustion can cause ground surface movements. The first was mentioned in the previous section in connection with sanitary landfills, where settlement caused by oxidation may occur simultaneously with other movements. The second is the subsurface combustion of coal waste dumps. Such dumps may consist of coal mine waste (sometimes referred to as gob) or stoker ash (incompletely burned coal and ash materials) from coal-burning power plants. In 1968 the U.S. Bureau of Mines identified 292 smoldering or burning piles located in the 26 coal-producing states (Magnuson and Baker, 1974). Such fires occasionally affect buildings or structural performance. In 1974, a major fire was detected beneath the coal storage yard and coal crusher plant of a power plant in northeastern Ohio. Concern about the fire was centered primarily around the integrity of buried transmission buss ducts that traversed the burning area. It was feared that the development of any large cavities in the burning fill would remove structural support and damage the concrete encased ducts. During an investigation and subsequent fire abatement program, voids up to three feet in size were detected at a depth of 10 feet in the burning fill. The ground surface had subsided more than one foot, probably by collapse not unlike that above areas undermined by subsurface erosion. Ground movements were translational and distortional.

A third case of ground movement related to underground combustion has been predicted as a result of underground coal gasification (Shoemaker, 1978). The observed compression, tension and shear failure of coal and overburden materials in a thermo-mechanical model suggests that subsidence may reach 52 percent of the seam thickness for a 50-foot extraction width. Ground surface movements would probably resemble those due to conventional mining.

Chemical Attack. Lukas and Gnaedinger (1972) report cases from the midwestern United States in which chemical spillages of acid and caustic materials leaked into the ground and reacted with the natural soil deposits The resulting weight loss in the soil led to a volume reduction and loosening of the soil deposit and subsequent settlement of soils beneath the foundations of existing buildings. Observed ground movements at different locations varied from 1.5 inches to 2.5 feet. At one site, a 6-foot void was found beneath a floor slab. Differential foundation movements of 3.5 inches caused building damage at one site.

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At each of the sites, ground movements due to conventional sources, such as compressibility of soils under building loads, subsurface erosion and densification by vibration were investigated and ruled out. Laboratory tests on soil samples showed substantial weight losses (up to 25 percent) when non-contaminated soils from each site were soaked in the chemicals that had leaked into the ground (hydrochloric and acetic acids and an aqueous solution of sodium hydroxide).

Ground movements caused by chemical attack could occur in a wide variety of circumstances. The possibilities for chemical spill (type, quantity) are limitless and the experiences of Lukas and Gnaedinger (1972) suggest that even soils containing silica particles are susceptible to attack by some caustic chemicals. The pattern and magnitude of movements are highly erratic and are dependent on the spill pattern. Ground movements are similar to those caused by subsurface erosion or combustion. This type of ground movement is not limited to soils. Carbonate rocks would be highly susceptible to hydrochloric acid spills.

The recognition of chemical attack as a cause of ground movement could be difficult for several reasons:

1. It is an uncommon cause of ground movement, yet one that has widespread potential;

2. The limits of the affected area might be uncertain, if, for example, the contamination is the result of a leak in an underground pipeline conveying a chemical at an industrial site. Ground movement may or may not be confined to the area directly above the contaminated area; the affected soils may not be recognized without excavation;

3. The chemical process could be very slow, and when movement is finally detected, evidence of the source of trouble (spill, etc.) could be lacking.

Biochemical Oxidation. Drainage of arable organic soils in the United States, England, Russia and the Netherlands is known to cause substantial ground surface subsidence. Reported subsidence movements are summarized in Table 2.3. Although the authors are not aware of structural damage caused by these movements, the magnitudes and rates are such that they could be significant for structures founded at ground surface.

Stephens and Speir (1969) list the following as known causes of subsidence in drained organic soils: 1) shrinkage due to dessication; 2) consolidation by loss of the buoyant force of groundwater and loading; 3) compaction tillage; 4) wind erosion; 5) burning; and 6) biochemical oxidation. Of these six mechanisms, the first five have been discussed as general sources of ground movement. According to Stephens and Stewart (1976), the initial subsidence in drained organic soils, most of which are peat, is mainly due to densification of the soils by drying, consolidation

Table 2.3

SUBSIDENCE OF ORGANIC SOILS

Country	Surface Subsidence	
	Rate	Amount (Interval)
USA		
Everglades	1.3-7.7 cm/yr	1.8-2.4m (1913-1968)
Minnesota		15-34 cm
Michigan		9.2 cm (5 yrs)
California	7.6 cm/yr	
England		
Holme Fen		3.25m (1848-1932)
USSR		
Minsk bog		1.0 m (1914-1961)
Netherlands		
Polder Mastenbrock		0.1-0.65m (50 yrs)

Data sources: Stephens and Speir (1969)
Stephens and Stewart (1976)

and compaction, and does not result in loss of soil mass. Such densification is largely a non-recurring phenomenon. Wind erosion and burning may occur periodically and can result in significant soil loss in certain bog deposits. Oxidation, on the other hand, is a long-term process that continues as long as temperature, pH, and aeration are conducive to biochemical action. Oxidation is believed to be responsible for 65 percent of the recorded subsidence in the Florida Everglades, 20 percent in the Netherlands polder Mastenbrock and 13 percent at the Minsk Experimental Bog in Russia. It is a volume change process.

The significance of oxidation subsidence to the behavior of buildings founded on peat has apparently not been studied by engineers, probably because there has been little intentional construction on peat deposits. One locale, however, where extensive marshland has been recovered for development, is the New Orleans-Jefferson Parish area of Louisiana. Drainage in that area was initiated in the mid-1920's, and by 1978, the total subsidence at Kenner, Louisiana, where the peat thickness is eight feet, was about 80 inches and continuing at a rate of about one inch per year. Most subsidence has been attributed to lowering of the water table and subsequent consolidation of the peat, but the upper one to two feet have been affected by shrinkage. Traughber, Snowden, and Simmons (1978) feel that dessication is the predominant factor in causing shrinkage of the peat in that area. The significance of oxidation as a contributing factor to long-term subsidence apparently has not been investigated.

Prediction of Miscellaneous Sources of Ground Movement

The magnitude, distribution and time rate of ground movements caused by subsurface combustion or chemical attack do not generally lend themselves to prediction. Neither source is amenable to rational analysis by any theory, and the number of documented case histories is small. It may be possible to estimate the susceptibility of a site to particular sources of movement.

Stephens and Stewart (1976) present an empirical technique for predicting the subsidence due to organic soil decomposition in Sourthern Florida. They feel the method is sufficiently accurate to justify field estimates of biochemical subsidence for engineering predictions at least within the geographic area for which the method was developed.

SUMMARY AND CONCLUSIONS

Summary. With the exception of a few natural sources of ground movement, such as subsidence due to volcanic activity, not likely to be encountered within the coal mining areas of the central and eastern United States, virtually all significant geologic processes of ground movement known to the writers have been described in the previous pages. A few processes, such as tree-root heave, meteorite impact, and animal burrowing have been intentionally omitted because of their limited interest. Several human activities which

can lead to permanent disruption of the ground surface profile, such as nuclear cratering, have been ignored. On the other hand, one process that does not classify as a ground movement, erosion, has been included because of its demonstrated significance as an agent of ground movements.

The amount of attention given to each source of ground movement in the previous pages should not necessarily be taken as an indication of its relative importance. Settlement under surface loads has been covered more thoroughly than the other sources simply because there is more documentation of this phenomenon than others and because of its almost universal occurrence at building sites. However, in a specific geographic and geologic setting, settlement under surface loads may be a far less likely source of ground movement than, say, landslides, which have been discussed in relatively few pages.

A summary of some important characteristics of ground movements is presented in Table 2.4. The relative predictability of the magnitude of ground movements from the sources discussed is summarized in Table 2.5. For the sake of comparison coal mining subsidence is also included.

The classification of each source of ground movement according to type of movement, most susceptible material, order of magnitude of movement, etc., is intended to reflect the typical or common situation, and does not take into account exceptions and extremes. For instance ground movements due to swelling soils may occasionally exceed 12 inches, but movements in the range of 1 to 12 inches are far more common. Similarly, dense and hard soils are not normally associated with cases of bearing capacity failure, but the presence of these soils does not necessarily preclude such movements.

Conclusions. The following conclusions can be reached regarding the various sources of ground movement that have been discussed:

1. Ground movement of virtually any source (Table 2.4) includes some component of rigid body motion (translation, rotation), distortion and volume change. The most common modes of ground movement appear to be vertical translational, distortional and volume change. The least common modes are horizontal translational and rotational.

2. The modes of movement most commonly associated with subsidence due to coal mining (vertical and horizontal translation and distortion) can also be associated with ground movements of at last six different sources.

3. In conjunction with items (2) and (3), it can be said that positive identification of the mode of ground movement could be helpful in narrowing down the identification of the source, particularly if the movement is horizontal translational or rotational. The major

Table 2.4

SUMMARY OF GROUND MOVEMENT CHARACTERISTICS

Source of Ground Movement	Principal Modes of Ground Movement IΔ	Subsurface Materials Normally Most Affected	Subsurface Materials Normally Not Affected	Movement Possible Source II	Movement Range Magnitude III	Movement Range Rate IV	Severity of Damage to Surface Structures V
Landslide and Related Mass Movements	T_{VH}, R, D, V	All Materials	-	N, H	I-TF	VS-VF	SL-S
Erosion	T_V^*, D, V	Cohesionless Soils, Soluble Rocks	Cohesive Soils; Insoluble Rocks	N, H	I-F	F-VF	SL-S
Frost Action	D, V,	Silt, Jointed & Layered Rocks	Well-Drained & Cohesive Soils, Intact Rocks	N, H	I	S-F	SL-M
Collapsible Soils	D, V	Loose, Cemented Cohesionless soils	Cohesive Soils, Most Rocks	N, H	I-F	F-VF	SL-S
Shrinking and Swelling Soils	D, V	Clay, Some Shales & Artificial Materials	Cohesionless Soils, Most Rocks	N, H	I	S	SL-M
Fluid Extraction	D, V	Loose & Soft Soils, Weak Porous, Jointed Rock	Dense & Hard Soils, Intact Rock	H	I-TF	VS-S	SL-M
Densification by Vibration	D, V	Cohesionless Soils Rocks	Cohesive Soils, Most	N, H	I-F	F-VF	SL-S
Yield of Soils Into Construction Excavation	D	Loose & Soft Soils	Dense & Hard Soils, Most Rocks	H	I	S-VF	SL-S
Bearing Capacity Failures	T_V, R, D	Loose & Soft Soils Weak Rocks	Dense & Hard Soils Most Rocks	H	I-F	F-VF	SL-S
Settlement Under Surface Loads	D, V	All Materials	-	H	I-TF	VS-VF	SL-S
Tectonic Movements of the Earth's Crust	T_{VH}, R, D, V	All Materials	-	N, H	I-TF	VS-VF	SL-S
Miscellaneous	T_V^*, D, V	Coal Refuse	All Other Material	H	I-F	S-VF	SL-S
Chemical Attack	D, V	All Materials	-			VS-VF	SL-M
Biological Oxidation	D, V	Peat, Organic Soils	All Others	N, H	I-F	VS	SL
Coal Mining Subsidence	T_V^*, D, V	All Materials	-	H	I-F	S-VF	SL-S

Note: Δ Key: I. T = Translational (V = Vertical; H = Horizontal), R = Rotational, D = Distortional, V = Volume Change
II. N = Natural, H = Human
III. I = Inches (<12"), F = Feet (1'-10'), TF = Tens of feet (>10')
IV. VS = Very Slow (in/year), S = Slow (in/month), F = Fast (in/week), VF = Very Fast (in/hour)
V. SL = Slight (cracking), M = Moderate (extensive repairs), S = Severe (instability, collapse)
*Assuming collapse of ground surface

Table 2.5

RELATIVE PREDICTABILITY OF
MAGNITUDE OF GROUND MOVEMENT

HIGHEST LEVEL PREDICTION USUALLY POSSIBLE
AT A PARTICULAR SITE

Source of Ground Movement	Qualitative	Quantitative	
	Determine Susceptibility to Movement from a Particular Source	Determine Likelihood of Occurrence of Movement from a Particular Source	Estimate Magnitude of Movement from a Particular Source
Landslide		X	
Erosion	X		
Frost Action	X		
Collapsible Soils			C-F
Shrinking and Swelling Soils			C-F
Fluid Extraction		X	
Densification by Vibration		X	
Yield of Soil into Construction Excavations			C
Bearing Capacity Failures		X	
Settlement Under Surface Loads			F-A
Tectonic Movements of The Earth's Crust	X		
Miscellaneous			
Subsurface Combustion	X		
Chemical Attack	X		
Biological Oxidation	X		(C)
Mining Subsidence		X+	C-F*

KEY:
- C = crude (±50 to 75%)
- F = Fair (±25 to 50%)
- A = Accurate (±10 to 25%)
- \+ Abandoned Mine Sites
- * Active Mine Sites
- () Prediction sometimes possible

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problem in utilizing the type of ground movement as an aid in identification of source of ground movement is that the mode of movement is not always readily apparent. Subsurface exploration, ground surveys, instrumentation and testing may be required. Also, it is important to recognize that due to the complexities of soil-structure interaction, the movement a building undergoes is not necessarily the same as that of the ground.

4. Identification of subsurface material type could point to possible sources of movement. The value of this information is enhanced if the mode of ground movement is also known. For instance a volume change movement in clay soil is often associated with a shrinking or swelling phenomenon. But all soil and rock types can be affected by a wide variety of movements. Probably the most useful application of soil/rock material type in identifying source of ground movement is the possibility of elimination, at least preliminarily, of a number of ground movement sources that are <u>not</u> likely to be related to the identified soil or rock type. For instance, if ground movements have occurred in a stiff clay deposit, the following could normally be ruled out: frost action, collapse, densification by vibration, subsurface combustion and biological oxidation. If this elimination process is utilized, it is important to be certain that the identified material types are not underlain by soil deposits that may be susceptible to other types of ground movement. The other cautions relate to the behavior of materials such as coal refuse. It is important to recognize that soil and rock fills may behave either as cohesive or cohesionless materials depending on their composition.

5. Eight sources of ground movements can derive solely from natural causes; that is, they can occur without any provocation from human activity. These same eight, however, may also result from human activity. Seven sources of ground movements including mining subsidence, can occur only as a consequence of human activity. The presence of buildings or other structures at ground surface may initiate or influence ground movements in some cases, while in others their presence may have absolutely no influence on the movements.

6. Ground movements of several sources may be in action in a specific location, some being related to human activities or to the presence of human objects and others being related to natural causes (e.g., mine subsidence and settlement under surface loads).

7. Ground movements of several sources may be caused by a single human activity. For example, a construction excavation may lead to landslides, erosion, consolidation

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by drainage (fluid extraction), yield of soil into excavation, and bearing capacity failure. Underground mining may promote erosion, fluid extraction, densification by vibration, and landsliding, in addition to subsidence.

8. The magnitudes of movements of all sources can vary over an extremely wide range. Many may be at least as significant as those due to mining subsidence. Extremely large movements (greater than ten feet) can normally be related to one of the following four: landslides, tectonic movements (faulting), fluid extraction, and bearing capacity failure.

9. Movement rates for all sources taken collectively range from inches per year to inches per hour. Ten sources may lead to sudden movements. The rate of movement refers to actual ground movement and not to the process that causes movement. Erosion or chemical attack, for example, could be progressing for years prior to actual ground movement, even though the movements themselves may be quite sudden.

10. The severity of known damage may range from slight to severe for ground movement from any source. Characteristically, the damage tends to be less severe for such phenomena as frost action, biological oxidation, fluid extraction and settlement under surface loads than for bearing capacity failures, landslides, yield into construction excavations and coal mining subsidence.

11. In conjunction with the items above, it is apparent that "normal (ordinary) settlement" may result from a variety of sources. In fact, movements can originate from so many sources that identification of the responsible one may be quite difficult. Although ground movement of each source is associated with certain features unique to its development (soil or rock properties, influence of human activities and objects, presence or movement of ground water, specific natural geologic processes), the surficial character of the movements, in the absence of any detailed subsurface information or historical record of development, might lead to some ambiguity in identifying their source.

12. The state of relative predictability of ground movements is rather primitive (Table 2.5).

    a. In only six of the sources that have been discussed is it possible to predict magnitude of movement. In five of these six, the prediction cannot consistently be made to an accuracy greater than $\pm$50 percent.

    b. The likelihood of occurrence of movement (as indicated by the factor of safety or safety margin) can be reliably predicted for only five sources of

movement while four (at least in certain scenarios) rest in the unfortunate state which allows only recognition of the susceptibility to movement. Unfortunately, two of these four, landslides and erosion, are important and relatively common.

13. The accuracy of ground movement predictions is limited for all types of ground movement. Generally, settlements under surface loads and settlements associated with shrinking, swelling and collapsible soils can be predicted somewhat better than others.

14. Movements take place by different mechanisms, and it is not to be expected that any one analytical technique would fit all mechanisms. Predictive methods must therefore be adapted for each. However, the primitive state-of-the-art in the prediction of ground movements cannot simply be attributed to a lack of analytical techniques. Barriers lie in several areas. In the case of ground movement from natural causes the principal barriers lie in the inability to predict such factors as: the magnitude, intensity, and fluctuations of surface and ground water distribution and flow; and the occurrence, duration, and intensity of seismic and tectonic events. The intricacies of ground water, in particular, probably has a greater influence on all sources of ground movement, both naturally- and humanly-activated, than any other single factor.

    The principal barriers to better prediction in the cases of ground movement due to human causes lie in these areas: the inability to predict the distribution, duration and intensity of stress changes in the ground caused by various activities that lead to ground movement; the inability to accurately define the details of subsurface conditions that are likely to be affected by human activities; and the inability to secure undisturbed samples for testing.

15. Save for those few exceptional cases in which all of the factors mentioned above are known, highly accurate analytical predictions of ground movements will probably never be possible. The greatest hope seems to lie in the development of empirical guidelines based on field observations, but even this approach seems unlikely to produce predictive techniques for processes such as erosion, chemical attack or tectonic movements. The prospects appear dim of ever distinguishing, by calculation alone, the amounts of ground movement due to individual sources where multiple modes of ground movement have acted. The only exception may be where one of two sources of movement is known to be settlement under surface loads.

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3. MINING AND SUBSIDENCE IN THE NORTHERN APPALACHIAN COAL FIELD AND THE ILLINOIS BASIN

Introduction

Coal bearing strata of the Northern Appalachian Coal Field and Illinois Basin are of comparable age and have been mined in much the same way. As might be expected, subsidence features in the two regions are quite similar, although some distinctions exist owing to differences in geologic setting, mining and site characteristics.

Parallel data that would permit a close comparison of subsidence features between the two regions are only partially available. In recent years, the U.S. Bureau of Mines, the Department of Energy, the National Laboratories and state agencies have been collecting and synthesizing subsidence data above active and abandoned mines by way of contract and in-house studies. Progress has not yet reached the stage where a standard reference comparable to the National Coal Board Handbook can be developed. Enough studies have been conducted in the Northern Appalachian Coal Field and the Illinois Basin to achieve a fair indication of subsidence parameters, such as angle of draw and amount of subsidence, and a limited indication of ground strain, slope and curvature, which are important to damage prediction.

In the following pages, the qualitative and quantitative aspects of subsidence in the two regions are reviewed in light of their respective geologic settings and mining practices.

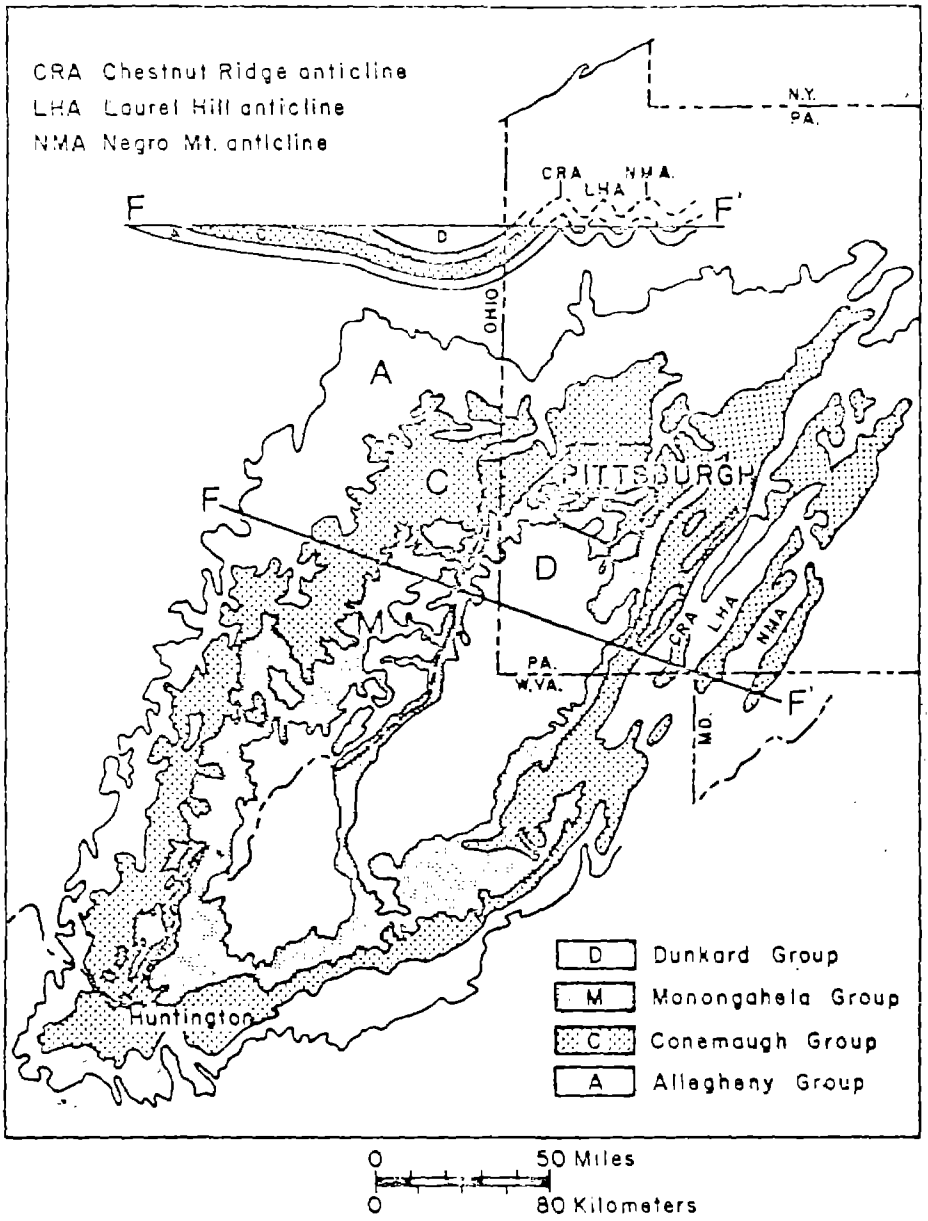
Geologic Setting

1. Northern Appalachian Coal Field

The Northern Appalachian Coal Field encompasses the Pittsburgh-Huntington Basin, which occupies portions of western Pennsylvania, West Virginia, Ohio and Maryland (Figure 3.1). This area is of much interest to the present study owing to the coincidence of underground mining with centers of population, such as the Greater Pittsburgh area. The coal field lies within the Appalachian Plateau, which extends from New York to Alabama in a band 50 to 200 miles wide and includes numerous coal producing areas. The following thumbnail characterization focuses primarily on the Pittsburgh-Huntington Basin.

Surface Topography--hilly to mountainous, with local relief of 500 or more feet near rivers and major tributaries. Hillslopes are commonly inclined at 5 to 20 degrees and locally may steepen to 40 or more degrees.

Surface Deposits--characteristically residual, colluvial and alluvial soils; glacial soils are absent. Residual soils, usually found on uplands, are formed in place by weathering and grade into the underlying parent sedimentary rock.



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FIGURE 3.1 – GEOLOGIC MAP OF THE PITTSBURGH–HUNTINGTON BASIN. (From Wagner et.al. to Pennsylvania, 1970)

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Colluvial soils develop along the lower sections of upland slopes from soil and rock material that has moved downhill by gravity and flowing water. Alluvial soils are deposited by streams and are found in the major stream valleys. Occasionally, alluvial soils occupy ancient, abandoned stream channels on the uplands.

Residual soils are commonly one to five feet thick and consist of 40 or more percent silt, the balance being sand, clay and rock fragments. Colluvial soils tend to have the same composition as the residual soils and parent rock from which they are derived and usually contain fragments of sedimentary rock. Colluvial soils are generally 5 to 15 feet thick but may reach thicknesses of 100 feet. Alluvial soils vary in character from gravels to clays as a consequence of local variations in stream velocity and sediment load at the time of deposition. Terrace deposits may reach thicknesses of several tens of feet. Figure 3.2 shows the soil deposits typical of a slope in this region.

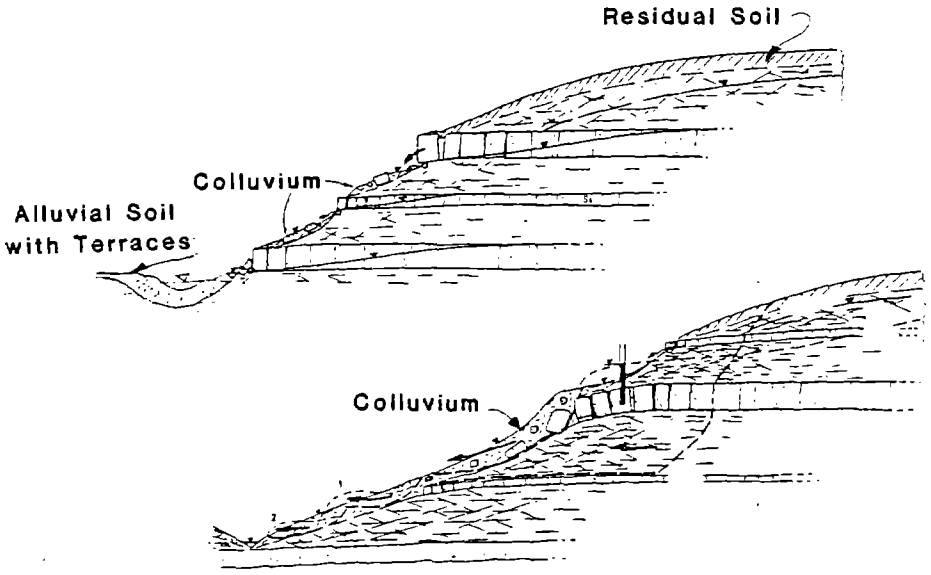
Bedrock Topography--generally conforms to surface topography.

Bedrock Lithology--Pennsylvanian and Permian-age sedimentary cyclothemic strata. Shales and claystone may represent 40 to 70 percent of the rock units, limestone 10 to 30 percent, coal 1 to 2 percent, and siltstone and sandstone 20 to 40 percent. Due to the nearness of the Appalachian depositional area to the parent landmass, these repetitive, rhythmic sequences of rock strata do not exhibit the great lateral persistence or regularity of the strata farther west in the Illinois Basin. Both marine and continental depositional environments are represented (Figure 3.3).

Structural Geology--minimal geologic structure. Rock strata dip gently over much of the region and are commonly described as 'horizontal'. Folds are open and ill-defined, with sinuous and discontinuous structural axes. Fold structures become better defined and of greater amplitude in the easterly portion of the basin (Figure 3.1). Occasional faults with limited vertical displacements are present. Major faulting is not common.

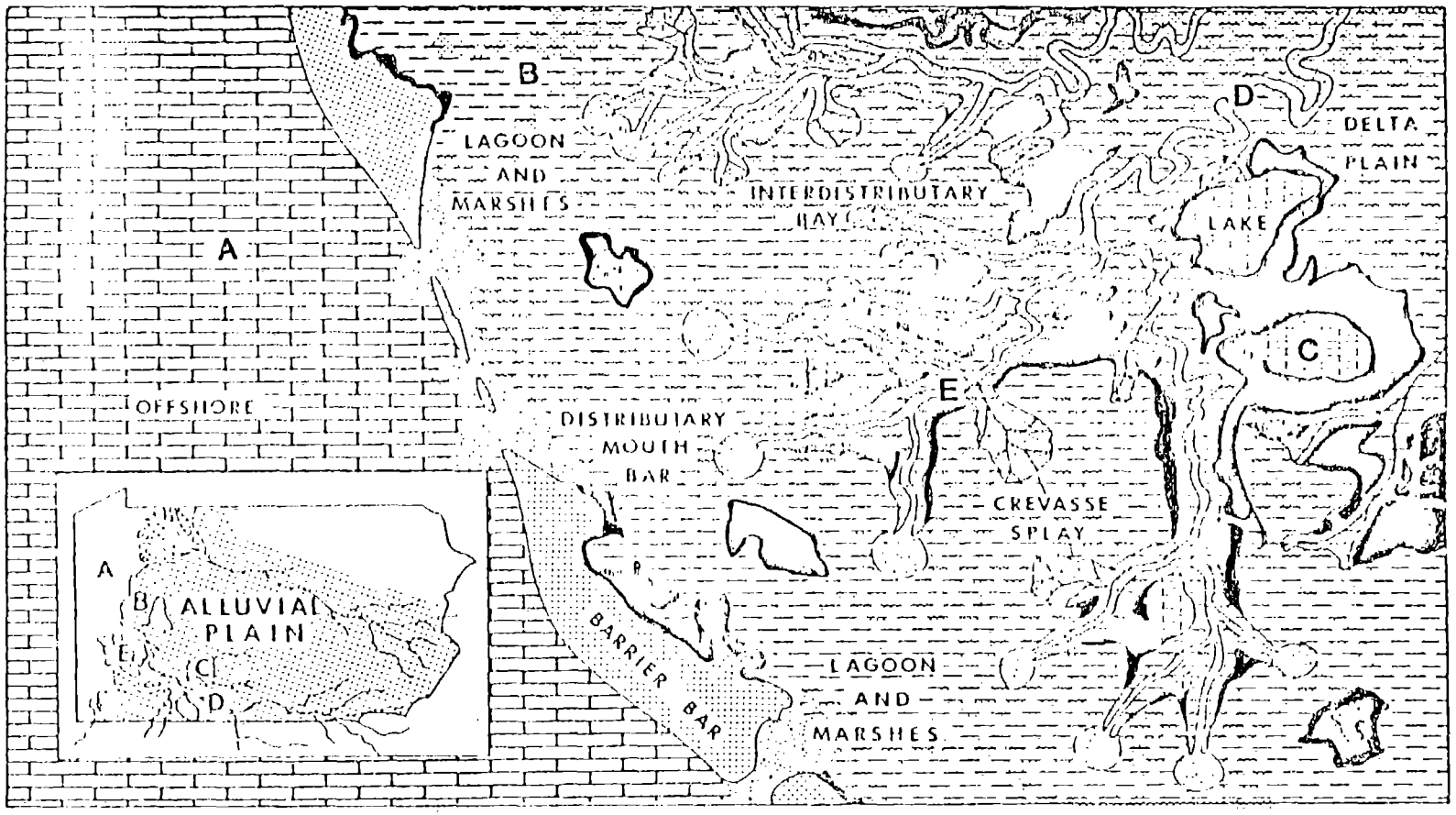
The ratio of horizontal to vertical in place stress in the region may be on the order of 1 to 4 (Haimson and Fairhurst, 1969).

Coal--more than 100 coalbeds occur in the "Coal Measures" of the Appalachian Plateau, 30 or more of them being sufficiently thick to be mined commercially. Five are prominent in the Northern Appalachian Coal Field. They are very uniform and continuous over tens or hundreds of square miles and lie within the Monongahela and Allegheny Groups.



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FIGURE 3.2 – SOIL DEPOSITS TYPICAL OF A SLOPE.
(Modified from Deere and Patton, 1971).



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FIGURE 3.3 – SCHEMATIC DIAGRAM OF THE DEPOSITIONAL ENVIRONMENTS FOR THE ROCKS IN WESTERN PENNSYLVANIA (Adapted from Pa G.S., 1970)

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The most prominent seam in the region is the Pittsburgh Coal, which extends over a 6000-square mile area in Pennsylvania, West Virginia, Ohio and Maryland. The Pittsburgh Coal is now nearly worked-out beneath the Pittsburgh area (Allegheny County), but still contributes more than 80 percent of the coal produced by underground mining in southwestern Pennsylvania. Deep mining in the Pittsburgh Area is conducted primarily in the Upper Freeport Coal (known locally as the Thick Freeport or Double Freeport) which is found in an economically mineable thickness under the northeast portion of Allegheny County (Figure 3.4). The Lower Kittanning Coal is prominent in northern West Virginia as well as in Cambria and Clearfield Counties, Pennsylvania. The Bakerstown, Clarion, Redstone, Sewickley and Waynesburg Coals have also been mined in the region at various times, but to a more limited degree. Figure 3.5 lists the principal coals mined in the Northern Appalachian Coal Field and their stratigraphic positions.

2. Illinois Basin

O'Rourke and Turner (1979) and Hunt (1980) report the following general geologic setting for the Illinois Basin:

Surface Topography--flat-lying to rolling.

Surface Deposits--mostly Quaternary-age glacial or glacially-derived sediments, ranging from 10 feet to more than 200 feet in thickness, and include till, loess and recent alluvial sediments. Till represents unstratified, unsorted pebbly silty clay, interspersed with layers and lenses of sand, gravel, silt and clay. Loess represents windblown silt and ranges in thickness from 2 feet to approximately 25 feet, except for a portion of southwestern Illinois where its thickness may reach 100 feet (Killey, 1975).

Bedrock Topography--gently rolling, except near buried bedrock valleys where relief is greater. The soil may be thicker than the rock strata above the coalbed in certain bedrock valleys.

Bedrock Lithology--shale-and claystone-dominated Pennsylvanian - age, flat-lying, cyclothemic strata, as thick as 800 feet, containing occasional limestones, siltstones, and sandstones. Shales and claystones may constitute 65 to 70 percent of the sequence; limestones 5 to 10 percent, coal 1 to 2 percent, and siltstones and sandstones 20 to 30 percent. Uninterrupted sequences of shale exceed thicknesses of 100 feet, but thicknesses of 20 to 40 feet are more common. Underclays are commonly 2 to 5 feet thick. All rock units except the sandstones exhibit great lateral persistence. Sandstones frequently occur as channel deposits.

Structural Geology--gently dipping rock strata encompassing several broad fold belts, none with more than 200 feet of

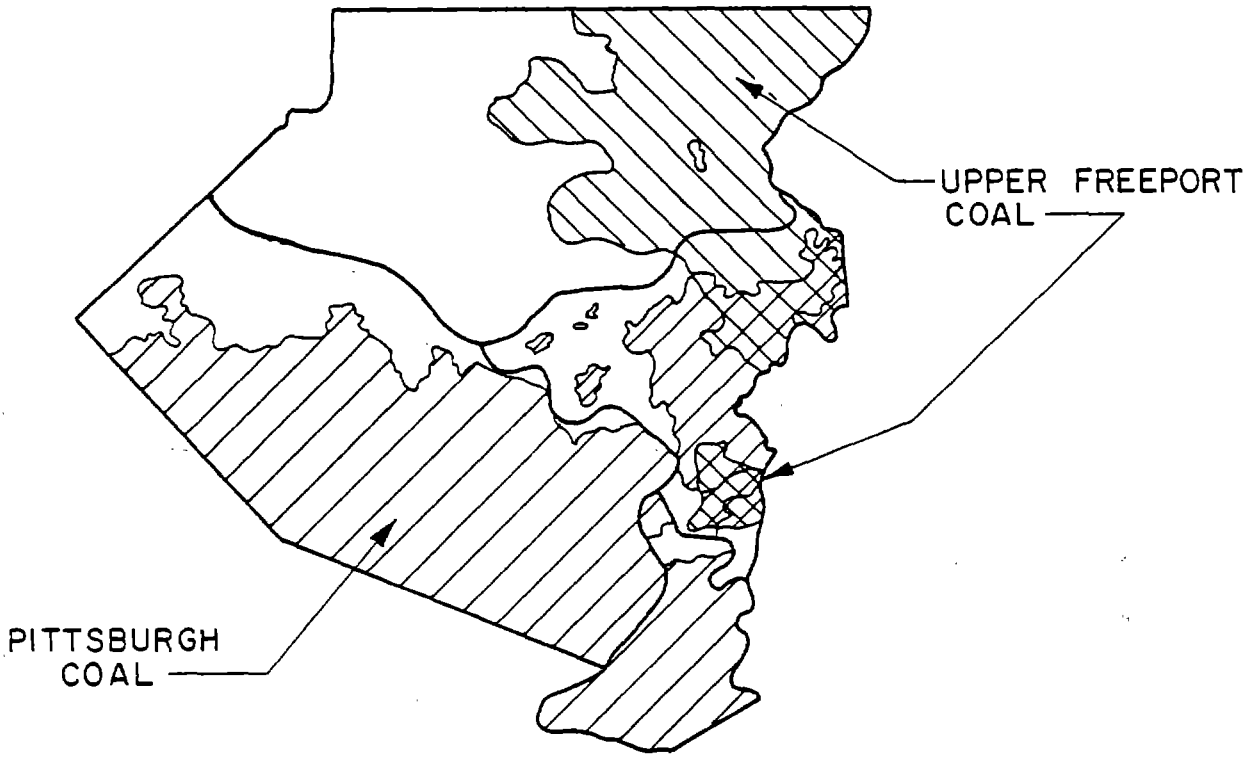
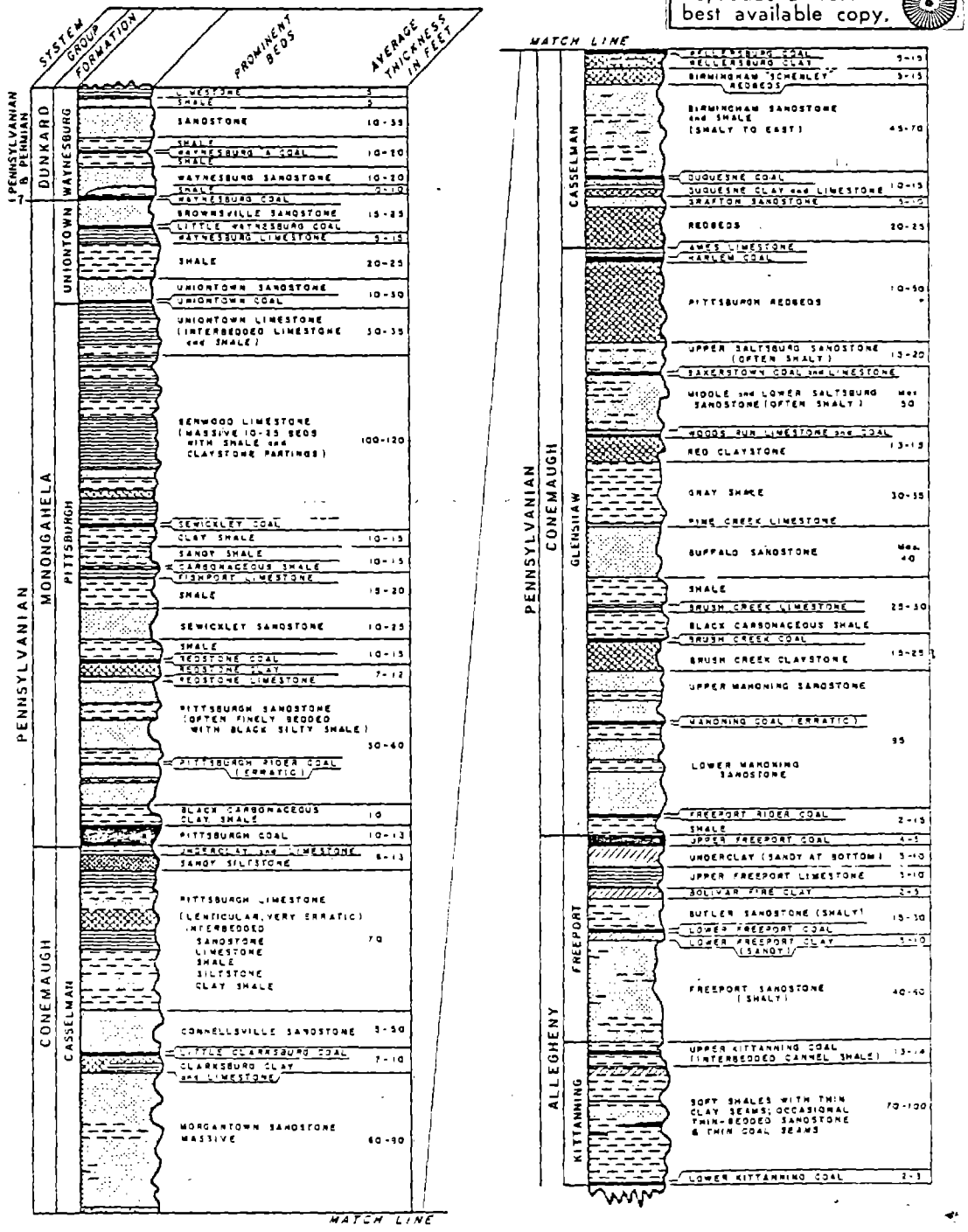


FIGURE 3.4 – AREAS OF ALLEGHENY COUNTY, PENNSYLVANIA, UNDERLAIN BY COAL MINES.

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FIGURE 3.5 – GENERALIZED STRATIGRAPHIC COLUMN FOR NORTHERN APPALACHIAN REGION.
(After Ackenheil and Associates).

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relief. Faulting is prominent in the southeastern part of the basin; displacements are generally less than 1000 feet. The ratio of horizontal to vertical in place stress may be on the order of 1.5 (Ganow, 1975) to perhaps 3 (Haimson and Fairhurst 1969).

Coal--generally flat-lying, 6 to 9 feet in thickness, very uniform and continuous. The principal economic seams are the Herrin (No. 6), the Harrisburg or Springfield (No. 5) and the Colchester (No. 2).

O'Rourke and Turner (1979) point out the striking geologic similarities between the Illinois Basin and the Midlands and Yorkshire Coalfields of the United Kingdom. The similarities include lithology, paleoecology, tectonic activity, erosional history of overlying rock strata and glaciation.

Underground Mining

1. Northern Appalachian Coal Field

Nearly all of the coals have been mined by room and pillar methods. Longwall mining is becoming more common, but still represents only about 5 percent of the underground operations in the region.

The presence of shale in the overburden has promoted the development of mining methods that permit full extraction of coal (85 to 100 percent recovery) by capitalizing on the ability of the strata to fracture in a controlled manner and thereby relieve the load from coal still to be mined. Notable exceptions occur in the northern Panhandle of West Virginia, the Ohio counties immediately to the west and parts of extreme western Pennsylvania, where poorly breaking limestones pose roof control problems that prevent the full extraction of coal in the Pittsburgh seam.

The ready access to the coal offered by outcrops along the major rivers in the northern part of the coalfield led to the early development of the Northern Appalachian Coal Field as a center of mining and industry. The first reported mining dates to the mid-1700's when British troops stationed near what is now central Pittsburgh mined coal from the hillside along the Monongahela River.

During the 18th and 19th centuries, mines were small hand-operations under shallow cover. Coal was cheap and the spacing, size and regularity of pillars were somewhat arbitrary. Coal pillars were left in place as a matter of convenience and safety to the miners. Increased production by the mid- to late-19th century brought mechanization and ventilation requirements to mines that necessitated a systematic arrangement of pillars, but still resulted in considerable

coal being left underground. Mining often extended to where the overburden was only 25 feet thick.

At some time in the 1880's and 1890's, extraction was increased by widening rooms to 20 to 24 feet, leaving pillars 10 to 18 feet wide, thus increasing coal recovery by 10 to 20 percent. These dimensions were characteristic until the advent of the block system of mining several decades later.

In the latter part of the 19th century, total extraction mining was initiated to achieve greater production of the coal that was becoming increasingly valued for its coking properties by the steel industry and was first implemented in partial extraction mines of the day. The distinction from partial extraction mines was that the long narrow pillars left between rooms during first mining were now extracted in a second stage of mining. Subsidence of the ground surface in a properly executed operation took place contemporaneously with pillar extraction.

Extraction of the pillars was accomplished by cutting successive pockets into the lateral faces of the pillars, or alternatively, by cutting successive slabs off the end of the pillar facing the mined-out area. With either method, the pillars were extracted along a common front, so that when the temporary props set to support the roof were removed, the roof, which cantilevered out over the newly mined area, would fracture along the break line induced by the last line of pillars and collapse to the floor.

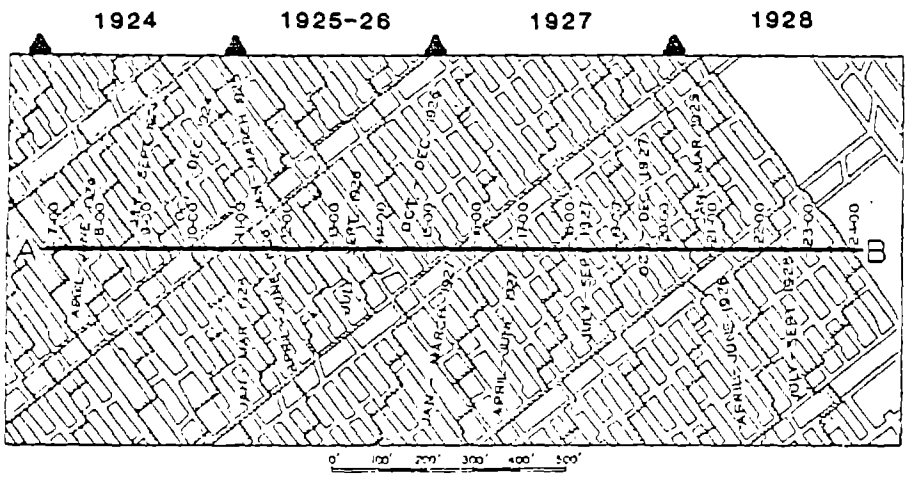
In certain early methods, extraction was coordinated so that two break lines each 150 to 200 feet long formed a chevron that moved from the far end of the panel to the near end.

By about 1930, many mines had accepted long break lines 1,000 to 1,500 feet long as preferable to short ones of a few hundred feet, since the former provided superior roof control as well as concentration of haulage, trackwork, ventilation, and supervision. This fostered lower production costs and compensated for the advantages of quick production and minimum development offered by the other methods that had been developed originally for shallow cover. The pillar retreat line shown in Figure 3.6 for a mine in the Thick Freeport Coal in northeast Allegheny County, Pennsylvania, is over 1000 feet long at the panel midsection and has been likened to a longwall retreating operation by Rice (1930).

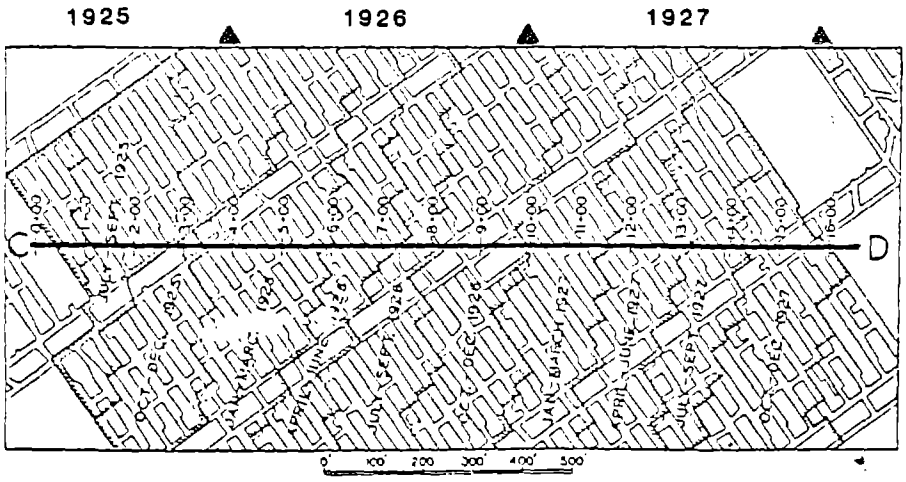
In an extensive survey, conducted nearly 50 years ago, of mining practices current in the Pittsburgh Coal Region at that time, five principal types of total extraction room and pillar mining were recognized, with an equal number of subtypes. The classification was based upon the configuration of rooms and entries, the order of mining the property and the technique

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PILLAR
RETREAT YEAR:



No. 1 South rib section.



No. 2 South rib section.

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FIGURE 3.6 – PILLAR RETREAT MINE PLAN LAYOUT, SPRINGDALE (Allegheny County), PENNSYLVANIA. (Rayburn, 1930)

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employed to extract the pillars. The choice of process depended on production and development requirements, the thickness of the overburden, and the preferences of the operator. The basic characteristics of these methods have been reviewed by GAI (1977). Although differences existed between mining operations due to size, property line constraints, and other factors, most mines were developed in much the same manner.

For a long while, wide rooms and narrow pillars, such as shown in Figure 3.6, were employed in total extraction mines because it was believed that more lump coal could be produced by room mining than by extracting the pillars left between rooms. However, by the 1920's, block systems of mining came into favor, wherein square or rectangular pillars 60 to 100 feet on a side were separated by narrow rooms and entries, reducing roof deterioration, roof falls and support problems during pillar extraction (Paul and Plein, 1935). In the period 1948 to 1952, most remaining mines of the old pattern were converted to the block system as continuous mining machines were introduced on a large scale. Subsequently, as break lines parallel to the pillar faces replaced angled break lines, the transition to the relatively efficient pillar extraction methods of today was essentially complete. Current methods for conducting room and pillar retreat mining operations are reviewed in detail by Kauffman et.al. (1981).

In all of the cases above, the chain pillars and room stumps that protect the entries are removed wherever possible after panel extraction has been completed. This process is sometimes not accomplished until decades after first mining by small operators who can undertake the work economically.

Coal pillars are often left in place in locations where it is necessary to protect structures at ground surface. Although some mine operators had been providing surface support since at least 1900, it was not until April 1966 that mine operators in Pennsylvania became obligated by law to take such action. The Pennsylvania Bituminous Mine Subsidence and Land Conservation Act of 1966 requires that the operator provide, at his own cost, permanent pillar support for public buildings, churches and cemeteries, as well as for homes built prior to 1966. It is the mine operator's further obligation to offer the owners of homes built after 1966 the opportunity to purchase pillar support at reasonable cost if they so choose.

Formulas developed by the Pennsylvania Office of Mine Subsidence Regulation for prescribing permanent pillar support specify that: (1) fifty percent of the coal be left in place, uniformly distributed beneath the protected structures; and (2) the pillar support area be delineated by a 15 degree limit angle that extends to mine level from the outer edge of a 15 foot wide 'safety zone' drawn around the protected structure

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(Figure 3.7). Where the ground slope exceeds 5 percent, the width of the safety zone on the down slope side is increased by an amount equal to one percent of the overburden thickness for each percent of ground slope. No mining is conducted beneath structures where the mine depth is less than 100 feet. More than twenty-thousand buildings in western Pennsylvania have been protected in this fashion. Special provisions are made for dams and other exceptional structures. Provisions for permanent surface support do not exist in other states within the Northern Appalachian Coal Field.

About 30 mines in Western Pennsylvania and 30 in West Virginia employ the retreating longwall system alone or in combination with pillar retreat mining. The greatest concentration of longwalls is in Cambria County, Pennsylvania, where the Lower and Upper Kittanning Coal Seams are mined. Although the longwall method is being used with increasing frequency, it still yields only about five percent of the coal produced annually in the region.

The key features of a typical large underground coal mine in the Northern Appalachian Coal Field that incorporated both longwall and room-and-pillar sections is shown in Figure 3.8.

2. Illinois Basin

Mining practices and subsidence in Illinois have recently been reviewed by Hunt (1980), the Illinois State Geological Survey (1980), and Hunt, Bauer and DuMontelle (1981).

In Illinois, 72 counties have been a source for the underground extraction of minerals, and in 34 counties, at least one percent of the land surface is undermined. Most mining has been for coal, although clay, lead, zinc, fluorite, limestone and tripoli have been mined to some extent. Areas encompassing Pennsylvanian system coal bearing strata are shown in Figure 3.9.

Underground coal mining has been practiced in Illinois for well over 100 years, and the evolution of mining practice parallels that of coal fields in the eastern United States. Early mining was by room and pillar methods that were characterized by irregular development, poorly defined panels and differing sizes of barrier pillars between panels and mines. As time went on, mines became more uniform in layout in order to increase productivity and to improve ventilation and safety standards. Typical mining types are enumerated below. Characteristic dimensions and percent extraction are shown on the accompanying figures.

1. Old Systems

   a. Basic room-and-pillar system--common at the turn of the Twentieth Century (Figure 3.10A).

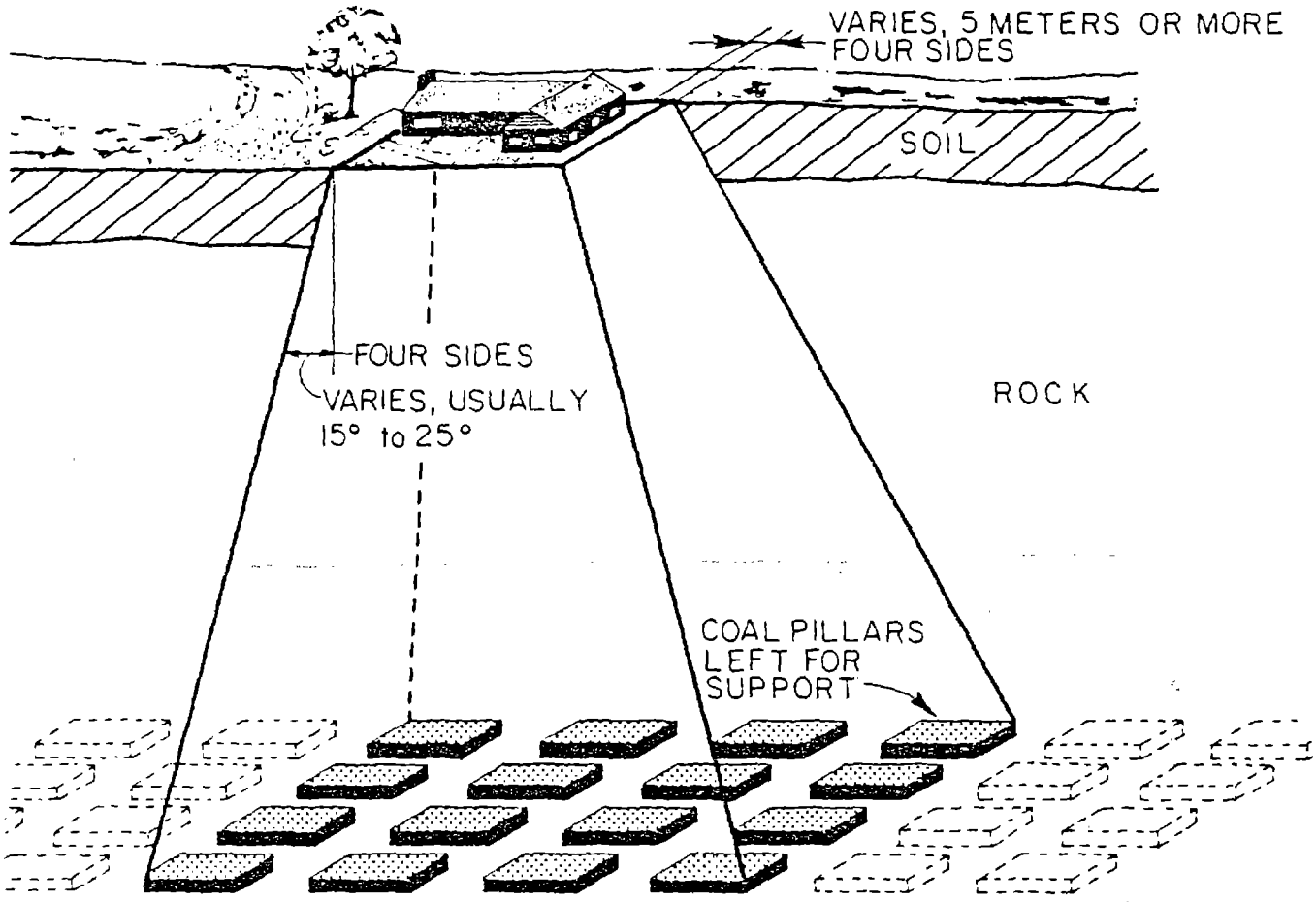
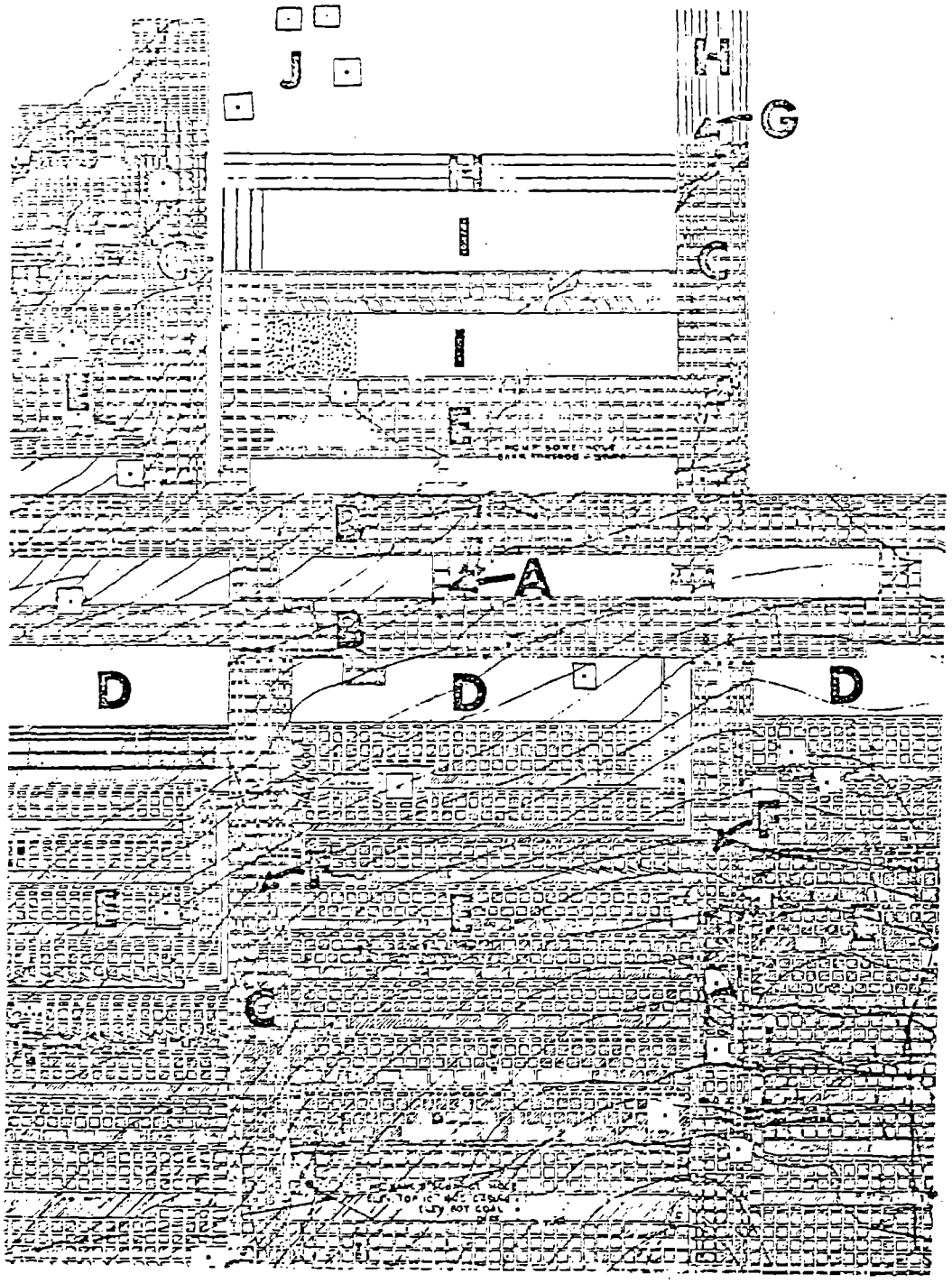


FIGURE 3.7 - PERMANENT PILLAR SUPPORT

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FIGURE 3.8 – MAP OF PART OF LARGE UNDERGROUND MINE (Collins, 1980).

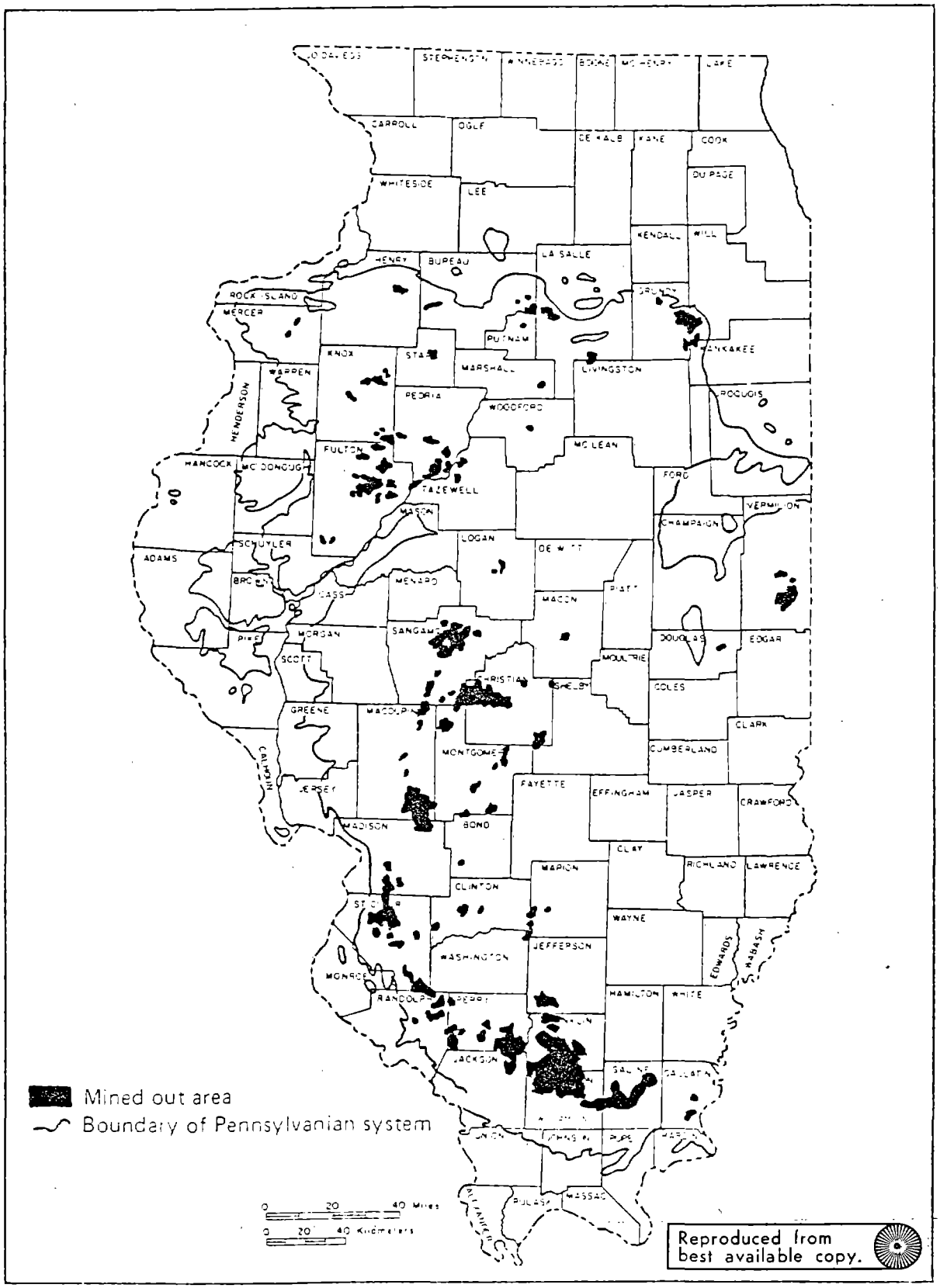
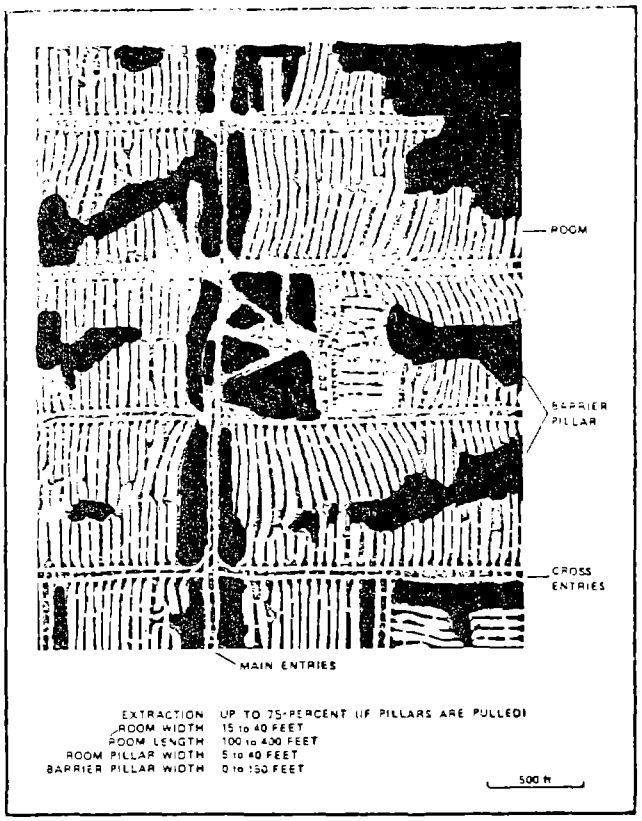
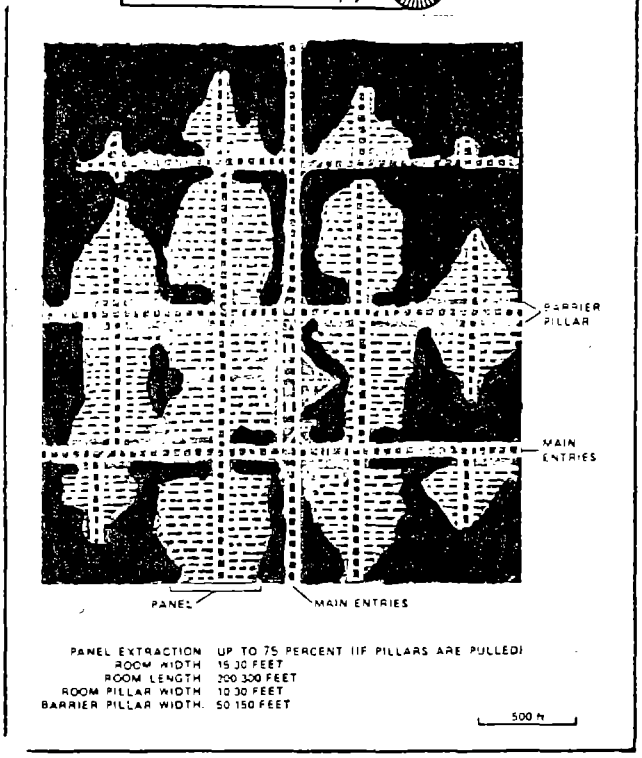


FIGURE 3.9 – AREAS OF ILLINOIS MINED FOR COAL
(DuMontelle et al., 1980)

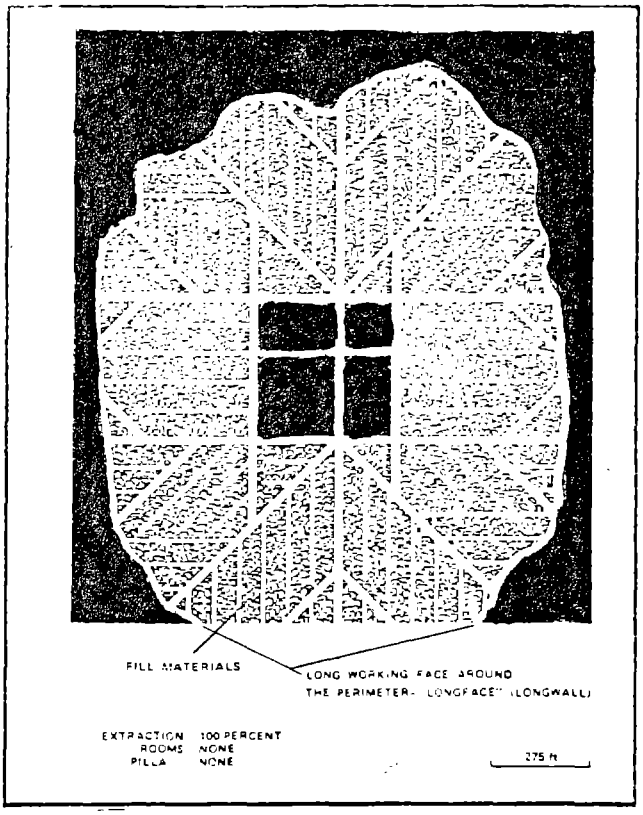
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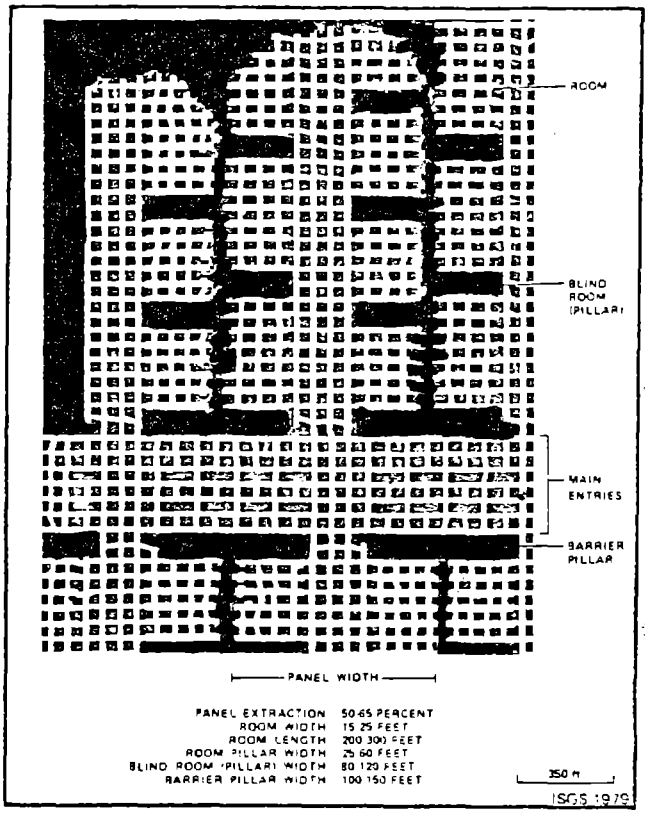
A. EARLY METHOD (BASIC). IRREGULAR LAYOUT WAS TYPICAL OF SOME EARLY MINES (AFTER ANDROS, 1914b).



B. EARLY METHOD (MODIFIED). MORE REGULAR LAYOUT OF ROOMS AND PILLARS; PANELS WERE SEPARATED FROM MAIN ENTRIES (AFTER ANDROS, 1914a).



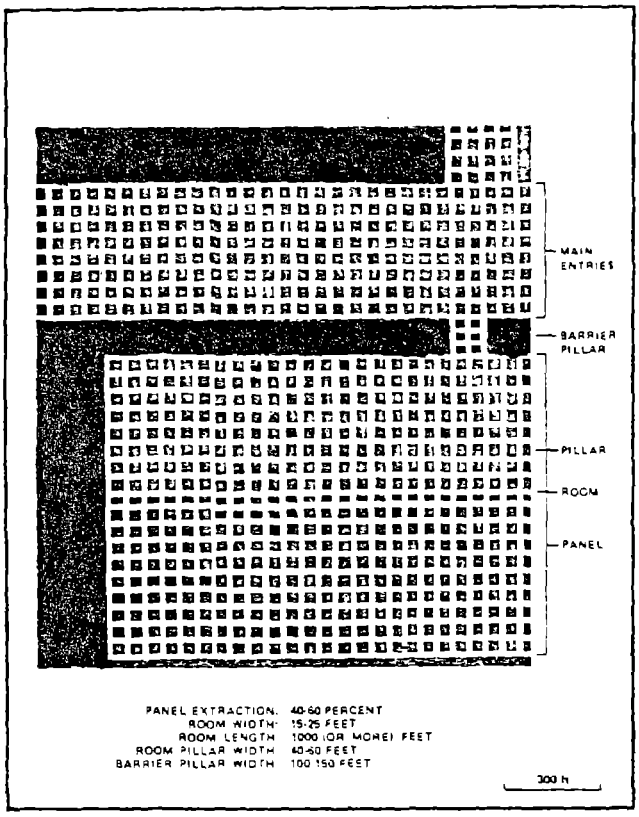
C. EARLY METHOD (LONGFACE).



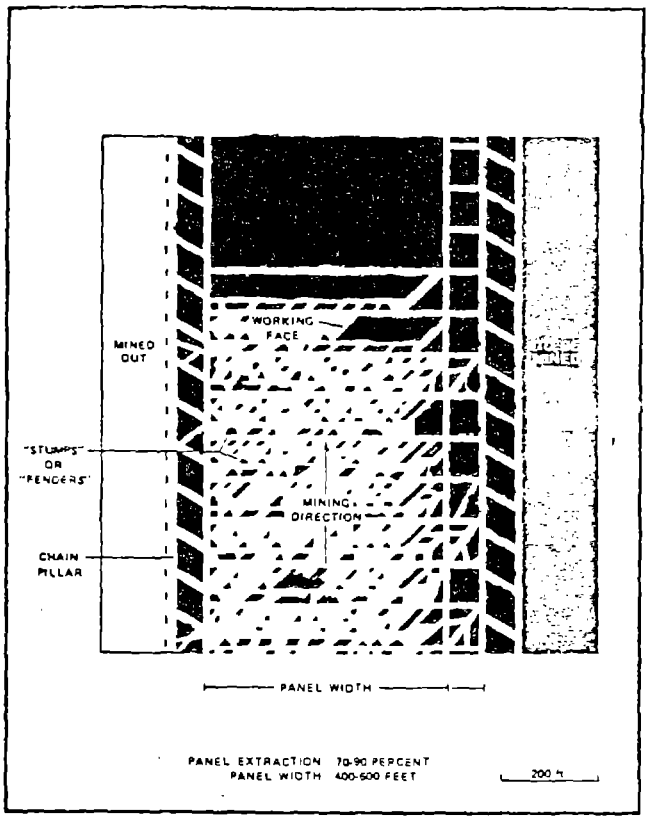
D. MODERN METHOD (BLIND ROOM).

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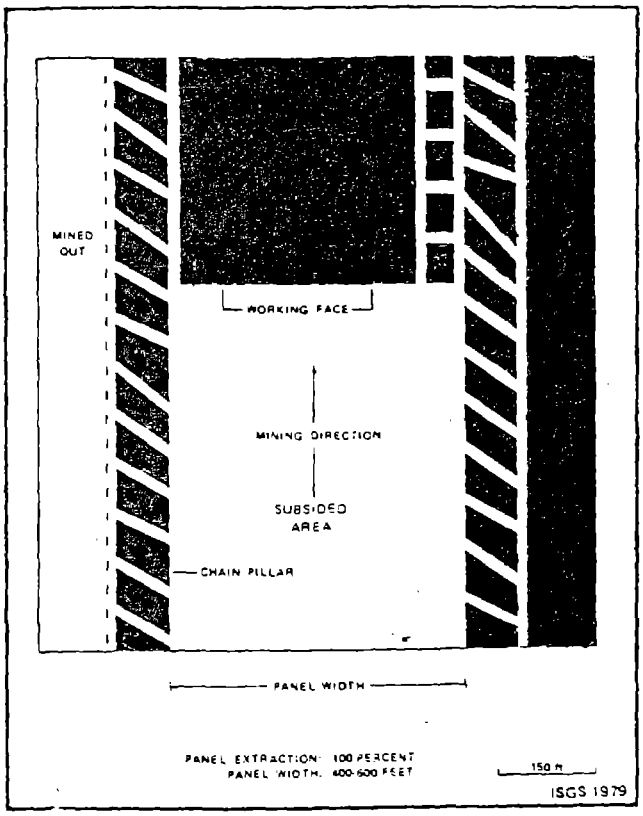
FIGURE 3.10 - MINING TYPES TYPICAL OF ILLINOIS. (AFTER HUNT, 1980)



E. MODERN METHOD (CHECKERBOARD).



F. MODERN METHOD (HIGH EXTRACTION RETREAT).



G. MODERN METHOD (LONGWALL).

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FIGURE 3.10 - MINING TYPES TYPICAL OF ILLINOIS.
(AFTER HUNT, 1980)

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b. Modified room-and-pillar system--a refinement of (a) employing a more uniform layout (Figure 3.10B) and akin to the room-and-pillar systems in use today, although having fewer main entries.

c. Longface system--an early non-mechanical longwall system utilized primarily in northern Illinois (Figure 3.10C). Subsidence was often reduced by stowing or packing waste rock behind the working face.

2. Current Systems

a. Room-and-pillar system--partial extraction (40 to 65 percent recovery) practiced by either of two methods in locations where ground surface movements must be avoided.

1. Blind room--every sixth or seventh room is left unmined for ground control purposes, subdividing the mine panel into smaller areas (Figure 3.10D).

2. Checkerboard--square pillars evenly spaced in both directions (Figure 3.10E).

b. Retreat systems--room-and-pillar method in which pillars are extracted in a systematic manner to achieve a recovery of 70 to 90 percent. The "Old Ben" system, described by Flowers (1957), is commonly employed (Figure 3.10F).

c. Longwall system--complete extraction of coal within the panel by drawing a shearer across the face (Figure 3.10G).

Panel widths of 400 to 600 feet are typical for retreat and longwall systems and 1000 feet for room-and-pillar systems. Both the retreat and the longwall systems result in panel-wide subsidence at ground surface. Coal has been mined to depths of almost 800 feet. There are few areas where more than one seam has been mined.

Subsidence Above Active and Abandoned Mines

1. Types of Subsidence

Subsidence features above mines in the Northern Appalachian Coal Field and Illinois Basin are categorized as either sinkholes or troughs (Figure 3.11).

A sinkhole is a crater-like depression in the ground surface that occurs from collapse of the overburden into a mine opening -- generally a room or an entry in an abandoned

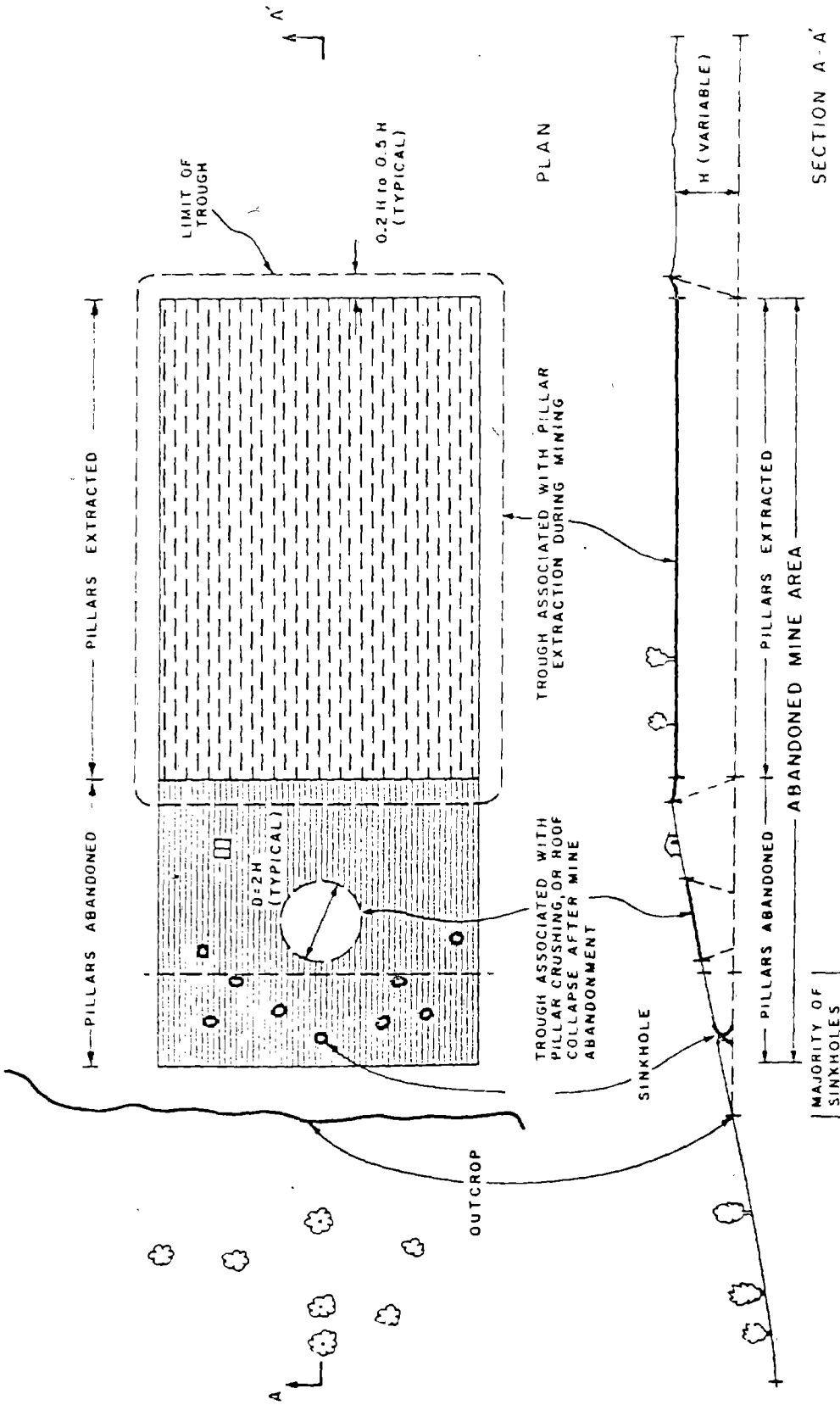


FIGURE 3.11 - TYPICAL MODES OF MINE SUBSIDENCE IN WESTERN PENNSYLVANIA.

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mine operated decades ago beneath shallow cover. Because the surface opening generally bells out with depth, and because erosion may increase the diameter near ground surface, the sinkhole may resemble an hourglass in profile. Sinkholes are sometimes termed 'pits' in Illinois.

A trough is a shallow, often broad, dish-shaped depression that develops when the overburden sags downward into a mine opening in response to coal extraction or as a result of roof collapse, pillar crushing, or mine floor failure. Troughs are sometimes termed 'sags' in Illinois.

Under certain circumstances, ground cracks and slope instability are also manifestations of mine subsidence. The collapse of a concrete or wood cover over an abandoned mine shaft is more properly regarded as a structural failure.

2. Geometry of Subsidence Features

Active Mines: Most troughs above active mines are of regular nearly rectangular geometry, 600 or more feet in width and 1000 or more feet in length (Figure 3.11). They commonly extend a distance 20 to 50 percent of the overburden thickness beyond the boundary of the area where pillars have been extracted, the distance being dictated by the mechanical properties of the overburden, topography, state of stress, etc. The maximum subsidence at the center of a trough is generally 40 to 70 percent of the height of the mine opening--a maximum of 2.5 to 4.2 feet of subsidence for the 6-foot Pittsburgh or Herrin No. 6 coals.

Most, but certainly not all, subsidence features over active mines conform to the description above. Figure 3.12 shows one of the exceptions--a semi-circular trough about 150 feet in diameter that developed in Allegheny County, Pennsylvania, as a result of extracting a chain pillar along a haulageway 300 feet below ground surface. Localized subsidence of this type is not uncommon above older mines now in their final years of operation, where remaining coal is being recovered as local conditions permit.

Abandoned Mines: Subsidence troughs associated with abandoned mines typically resemble those above active mines, but are almost always elliptical in plan, since they generally do not conform to panel boundaries. Also, they usually measure only a few tens of feet to a few hundreds of feet in diameter. Trough diameters above abandoned mines in the Northern Appalachian Coal Field commonly measure 1.5 to 2.5 times the overburden thickness, reflecting the limit to which the overburden can bridge over local crushed pillars or roof failures before sagging into the distressed area (GAI, 1977). Similar trough diameter/mine depth ratios have been found for several subsidence cases reported in Illinois. This is

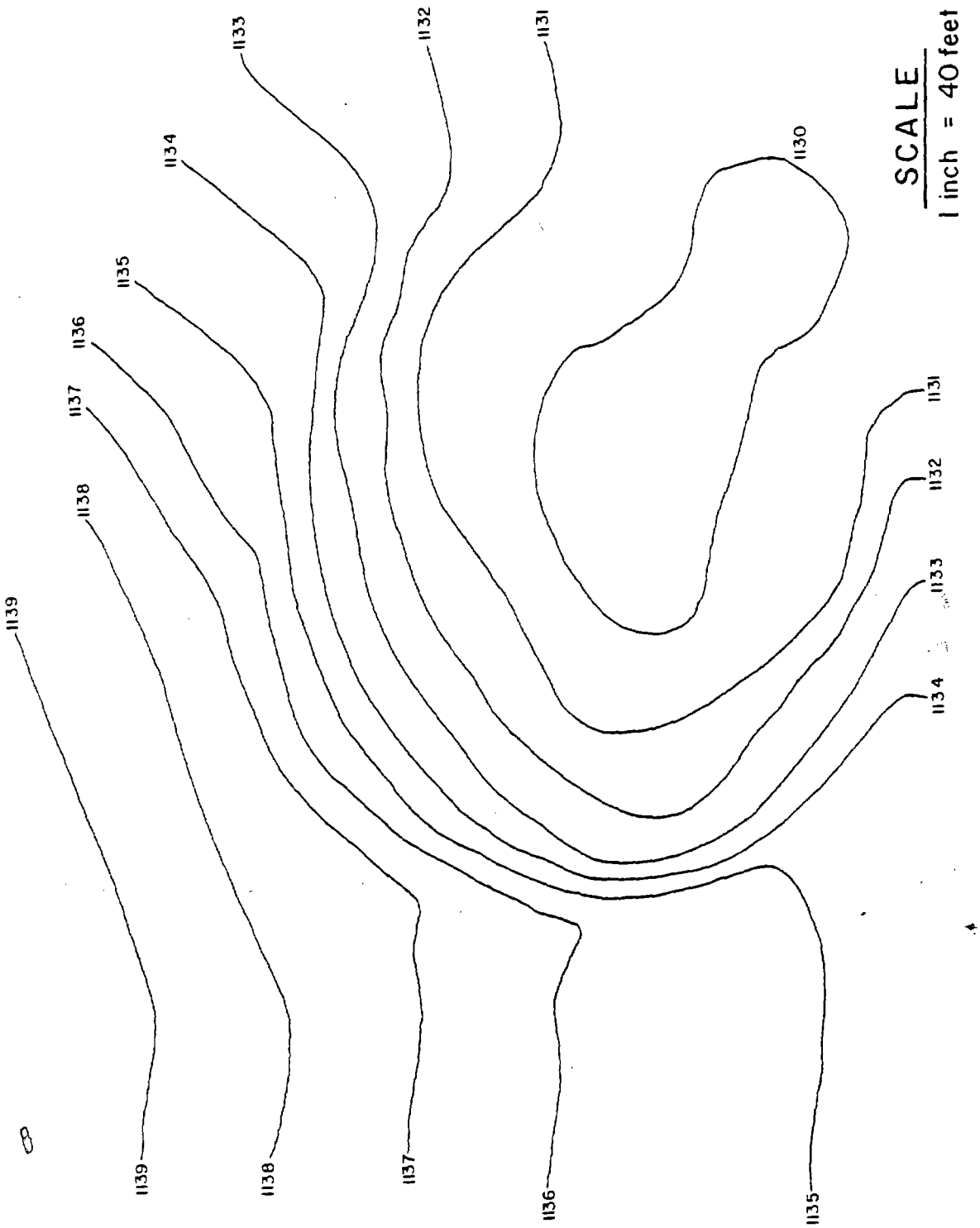


FIGURE 3.12 – SUBSIDENCE TROUGH RESULTING FROM EXTRACTION OF CHAIN PILLAR AT DEPTH OF 300 FEET, ALLEGHENY COUNTY, PENNSYLVANIA

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interesting in that soil often composes 40 to 50 percent of the overburden in Illinois, but only 10 to 20 percent in the Northern Appalachian Coal Field. Trough diameter appears to bear no relationship to overburden thickness when subsidence is caused by punching of pillars into the mine floor (GAI, 1977).

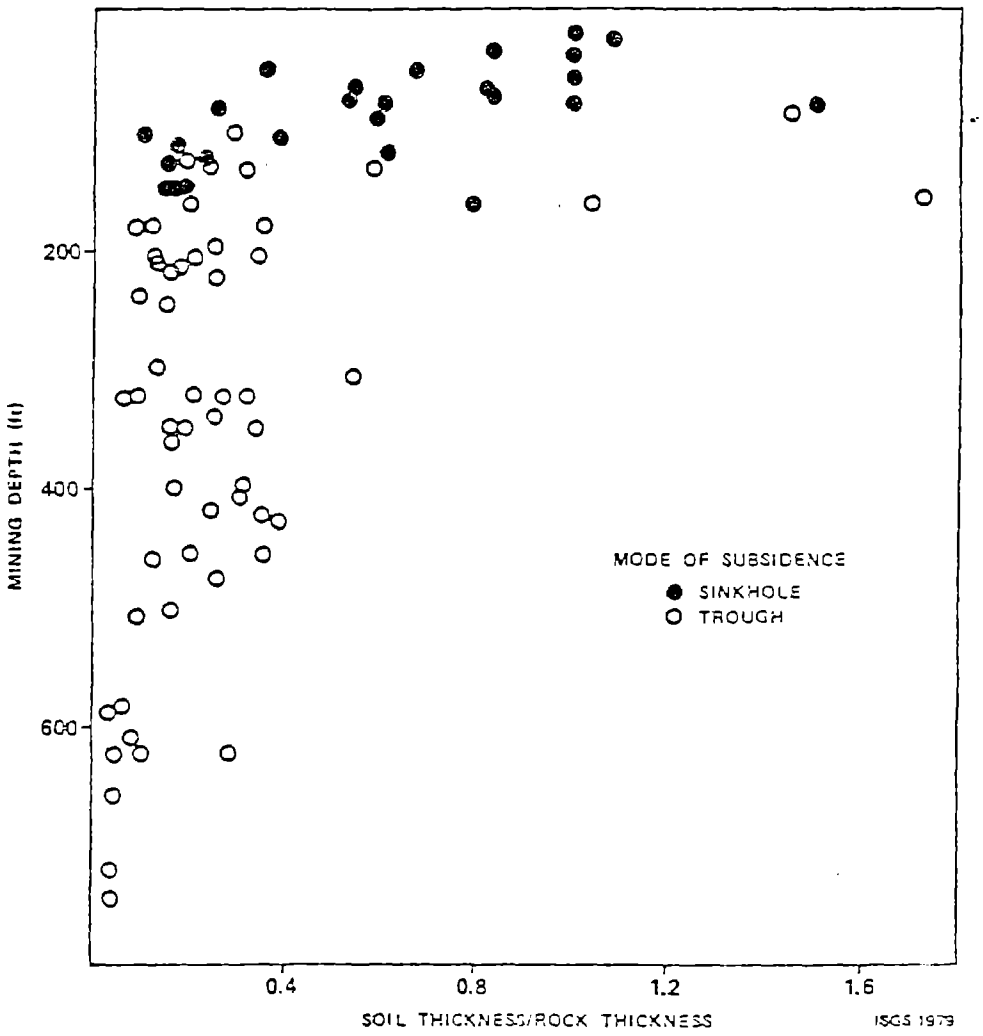
Whereas troughs in the Northern Appalachian Coal Field are generally found where the overburden thickness exceeds 70 feet, sinkholes are generally found at lesser overburden thicknesses (GAI, 1977). Mine sinkholes in the Northern Appalachians are generally less than 15 feet in diameter and 10 feet in depth--a consequence of their collapse origin and the 20-foot width of most abandoned mine openings in the region. Competent strata above the coal are known to limit sinkhole development in some areas, but not enough information is available to determine the effect region-wide. Wildanger et al. (1980) speculate that sinkholes in St. David, Illinois, tend to form where the St. David limestone is thin or absent and the mine roof is occupied by relatively weatherable shale. Hunt (1980) reports that sinkholes in Illinois have been found where the depth of mining is as great as 200 feet. The greater thickness of soil in Illinois appears to permit sinkhole development far higher above mine level than in the eastern states.

The Illinois State Geological Survey (ISGS, 1980) reports that one type of subsidence feature might predominate in certain localities, depending on the comparative thicknesses and properties of the soil and rock overburden. In shallow mines in Fulton County, sinkholes up to 15 feet in diameter are common, whereas in Williamson County above mines of similar depth, troughs up to 80 feet in diameter are common. These phenomena are currently under study by the ISGS. Data to compare these respective areas have not yet been collected and analyzed.

The distribution of reported subsidence features with depth in Illinois is shown in Figure 3.13. Table 3.1 summarizes data from more than one hundred incidents of sinkhole and trough subsidence above abandoned mines in the Pittsburgh Coal region of southwestern Pennsylvania. Similar subsidence features can be found above the other mined-out coals in the Northern Appalachian Coal Field. Local differences in overburden strata and soils may influence dimensions to some extent. Thick terrace soils promote somewhat broader sinkholes and troughs than thin colluvial and residual soils.

3. Time of Occurrence of Subsidence

Active Mines: Eighty-five to 90 percent of the ground movements associated with pillar retreat and longwall mining take place when the coal is extracted. Almost no movement is associated with the advance phase of mining in a pillar

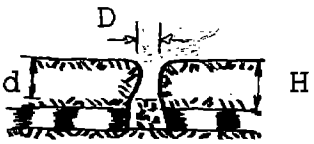
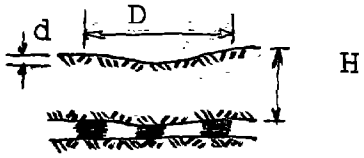


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FIGURE 3.13 - RELATIONSHIP BETWEEN SOIL THICKNESS/ROCK THICKNESS AND MINING DEPTH SHOWING THE MODE OF SUBSIDENCE (Hunt, 1980)

Table 3.1

CHARACTERISTICS OF SUBSIDENCE FEATURES
ASSOCIATED WITH ABANDONED MINES IN THE PITTSBURGH
COAL REGION (Bruhn, Magnuson and Gray, 1980)

		Sinkholes	Troughs
1.	Mode of development	Roof collapse	Roof collapse, Pillar crushing, Pillar punching
2.	Geometry		
	a. Characteristic Profile:		
	b. Diameter, D:		
	Range	1.5 - 45 ft°	30 - 1600 ftΔ
	Mean	9 ft [170]+	300 ft [15]
	80%	≤ 12 ft	≤ 300 ft
	c. Depth, d:		
	Range	1.5 - 45 ft	0.2 - 3 ft
	Mean	14 ft [170]	<3 ft [15]
	80%	≤ 18 ft	≤ 3 ft
3.	Occurrence		
	a. Overburden thickness, H:		
	Range	2.5 -150 ft	35 -450 ft
	Mean	45 ft [212]	120 ft [15]
	80%	≤ 70 ft	≤ 150 ft
	b. Distance from outcrop	75% of occurrences within 500 ftX	No apparent relationship

*Height of typical mine opening is 5.5 to 6 feet.

+Number in brackets denotes size of sample.

°Sinkholes are roughly circular in plan, i.e., $D_{max} = D_{min}$.

ΔNominal trough diameter $D = (D_{max} + D_{min})/2$.
D_{Max}/D_{min}: Range = 1 - 3.2; Mean = 1.45 [15].

Relationship of trough diameter D to overburden thickness H, D/H: Range = 0.4 -4.7; Mean = 2.0 [15]; 80% lie between 0.5 and 2.5; and 50% between 1 and 2.

XHillslope inclinations commonly range between 10 and 20 degrees.

retreat operation. Residual movements above pillar retreat and longwall panels are generally complete within two years after mining.

Abandoned Mines: In western Pennsylvania, subsidence incidents in abandoned mines have been reported less than a decade and more than a century after mining. More than half of all reported subsidence incidents have taken place 50 or more years after mining (GAI, 1977). The time of occurrence of subsidence is undoubtedly governed by the rate of deterioration of the rock strata and coal pillars and is probably hastened by the robbing of pillars by small operators years after initial mining, as well as by modification of the ground surface through excavation or fill placement, by modification of surface drainage, and other factors. It is interesting to note that the frequency of reports of sinkhole development in western Pennsylvania appears to reflect the quantity of precipitation experienced in the preceding three to eight months, a probable consequence of the weatherability and strength losses of rock strata that have been responsible for the numerous landslides in the region. Similarly, Wildanger et al. (1980) report that sinkholes in St. David, Illinois, have tended to appear after rainstorms.

It is interesting to note that recurrent episodes of subsidence have been reported at more than 30 sites in western Pennsylvania in the past twenty years--ground movements sometimes taking place two or three times at intervals of a year or more (GAI, 1977).

Burns (1981) has conducted some preliminary statistical studies concerning the frequency of occurrence of subsidence over abandoned mines in western Pennsylvania and Illinois. Defining the subsidence phenomenon as a Poisson process, he has formulated probability density functions for the time rate of subsidence occurrences in these localities. Unfortunately, the usefulness of such an approach is limited to making region-wide predictions of the probability of occurrence of a certain number of subsidence incidents in a given year. The central issue--prediction of the time at which subsidence will take place at a particular site--remains unanswerable.

4. Locations of Subsidence

Active Mines: Subsidence is assured wherever coal pillars are extracted. Permanent pillar support areas provided locally to protect homes, schools and other buildings have also sometimes subsided, generally as a result of pillars punching into the mine floor or as a result of pillar crushing. In western Pennsylvania the frequency of subsidence of support areas has been low--less than one-half percent of the more than twenty thousand buildings that have been supported by pillars over the past ten years (T. Alexander, Pennsylvania Office of Mine Subsidence Regulation, personal

communication). Some incidents of subsidence in western Pennsylvania have taken place over mined-out areas due to mining of the adjacent panel, but this is not common. Damaged homes are discussed in a subsequent section of report. It is worth noting that documentation concerning the locations of homes damaged by subsidence above active mines is available only for the Commonwealth of Pennsylvania.

Abandoned Mines: Most sites of subsidence above abandoned mines represent locations where rooms and entries in the mine were not completely collapsed upon completion of mining. In western Pennsylvania, sinkholes, which originate from roof collapses, are commonly located in a 500 feet wide band along the outcrop, where the overburden thickness is commonly less than 70 feet (Figure 3.11). In Illinois, proximal distances of sinkholes to the outcrop have not been reported.

Trough subsidence is not limited by overburden thickness and has been reported several hundred feet above abandoned mines in Pennsylvania, West Virginia, and Illinois.

Experience shows that future subsidence above any mined-out area cannot be ruled out unless it can be established that properly designed permanent pillar support has been provided or that the mine openings are completely closed. It is to be noted that surface development, as well as mine dewatering, fires set at outcrop, and other activities by man have sometimes been the immediate cause of subsidence.

Quantitative Aspects of Subsidence Profile Geometry

A subsidence trough is conventially characterized by stationary ground surface profiles in the longitudinal and transverse directions and by a non-stationary ground surface movement profile that accompanies the mine face in its passage from one end of the mine panel to the other (Munson and Eichfeld, 1980). The non-stationary profile is commonly represented as a subsidence development curve, in which downward movement of the ground surface is expressed in terms of advance of the mine face.

1. Subsidence Development

The non-stationary ground surface profile measured at a point near the panel centerline is termed the subsidence development curve and furnishes certain insights concerning the influence of overburden properties on subsidence.

Subsidence development curves for four pillar retreat mines in southwestern Pennsylvania are presented in Figure 3.14. The curves were synthesized from monument survey data available from the Republic, Nemacolin, Montour #10 and Springdale Mines. The Springdale Mine is in the Upper Freeport Coal; the other three are in the Pittsburgh Coal. All are representative of areas where subsidence incidents have been reported.

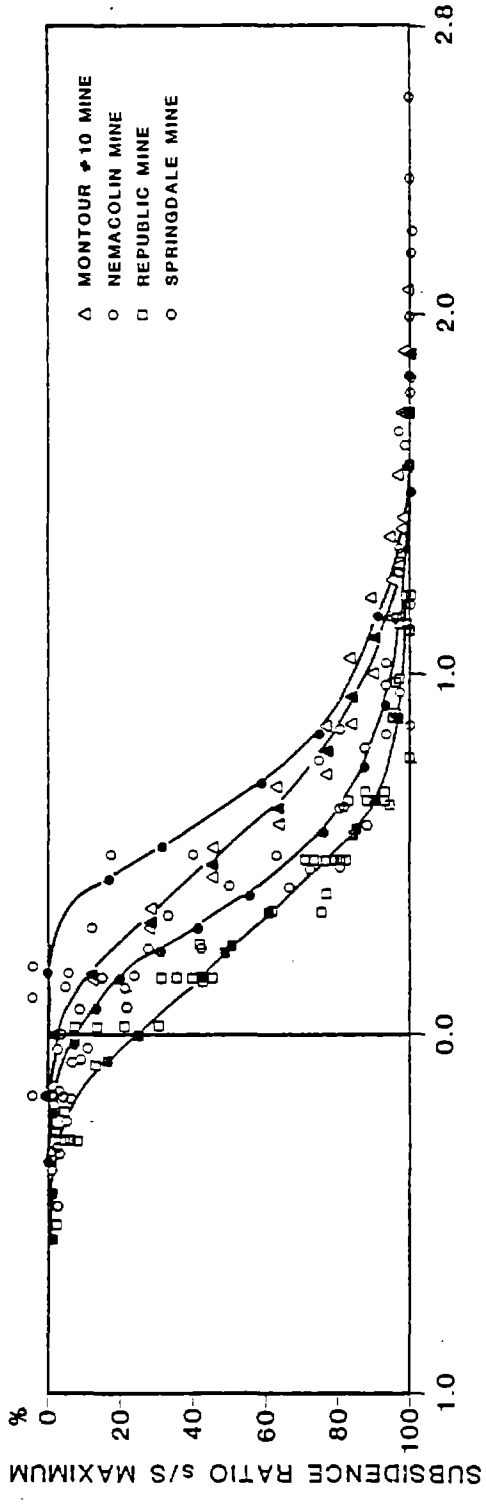


FIGURE 3.14 SUBSIDENCE DEVELOPMENT CURVES OF FOUR PILLAR EXTRACTION MINES IN SOUTHWESTERN PENNSYLVANIA

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Several characteristics emerge from these curves:

a. The initial downward movement of the ground surface is generally first recognized when the mine face has approached to within 0.4 h to 0.5 h of the monitoring point, h being the overburden thickness. Notably, initial movements at the Nemacolin Mine do not begin until the mine face is at or beyond the monitoring point, yielding what is sometimes termed a negative angle of draw. The location of the point of first movement relative to the face position is controlled by the arching action of the overburden strata (Wardell and Eynon, 1968). It may be significant that limestone and sandstone represent 65 percent of the overburden strata at the Nemacolin Mine, but only 35 to 45 percent at the other mines.

b. Subsidence that has taken place by the time the face is directly beneath the point does not exceed 10 percent of the maximum subsidence and may typically be half that amount.

c. Subsidence is commonly 90 percent complete and progressing at a much diminished rate when the face has advanced 0.75 h to 1.0 h beyond the monitoring point.

d. Subsidence is virtually complete when the face has advanced 1.25h to 1.75h beyond the monitoring point.

e. The rate of subsidence of the linear, rapidly descending portion of the curve ranges from about 15 Smax/h to Smax/h, and is generally nearer the latter.

Very similar characteristics have been reported for longwall mines in the Northern Appalachian Coal Field. Cheng and Peng (1981) report that subsidence is generally first noted when the face has approached to within 0.3 h of the monitoring point and does not exceed 8 percent of the maximum when the face is directly beneath the point. Subsidence increases rapidly thereafter and reaches 50 percent when the face has advanced 0.4 h beyond the monitoring point. It exceeds 90 percent of the maximum when the face has advanced 0.94 h, corresponding to a critical area of extraction. Kohli et al (1980), also studying longwall mines in the Northern Appalachian Coal Field, report similar numbers. They recognized first movement of the ground surface when the face approached to within 0.4 to 0.8 h of the monitoring point, 3 percent of the maximum settlement when the face was directly below the monitoring point and 95 percent of the maximum when the face was 1.0 to 1.6 h beyond the point. Residual (time-dependent) subsidence represented 5 to 13 percent of the total.

Figure 3.15 presents a subsidence development curve for the Old Ben No. 24 longwall in Illinois. In general respects, it resembles subsidence development curves for mines in the eastern states, but is much steeper. Subsidence at Old Ben is not measurable until the face is within about 0.2 h of the monitoring point and is well over 90 percent complete when the face is 0.5 h beyond the monitoring point. The steeper profile in Illinois may be due to the relatively few competent beds capable of arching over the mined out area, and possibly, the greater proportion of soil to rock composing the overburden.

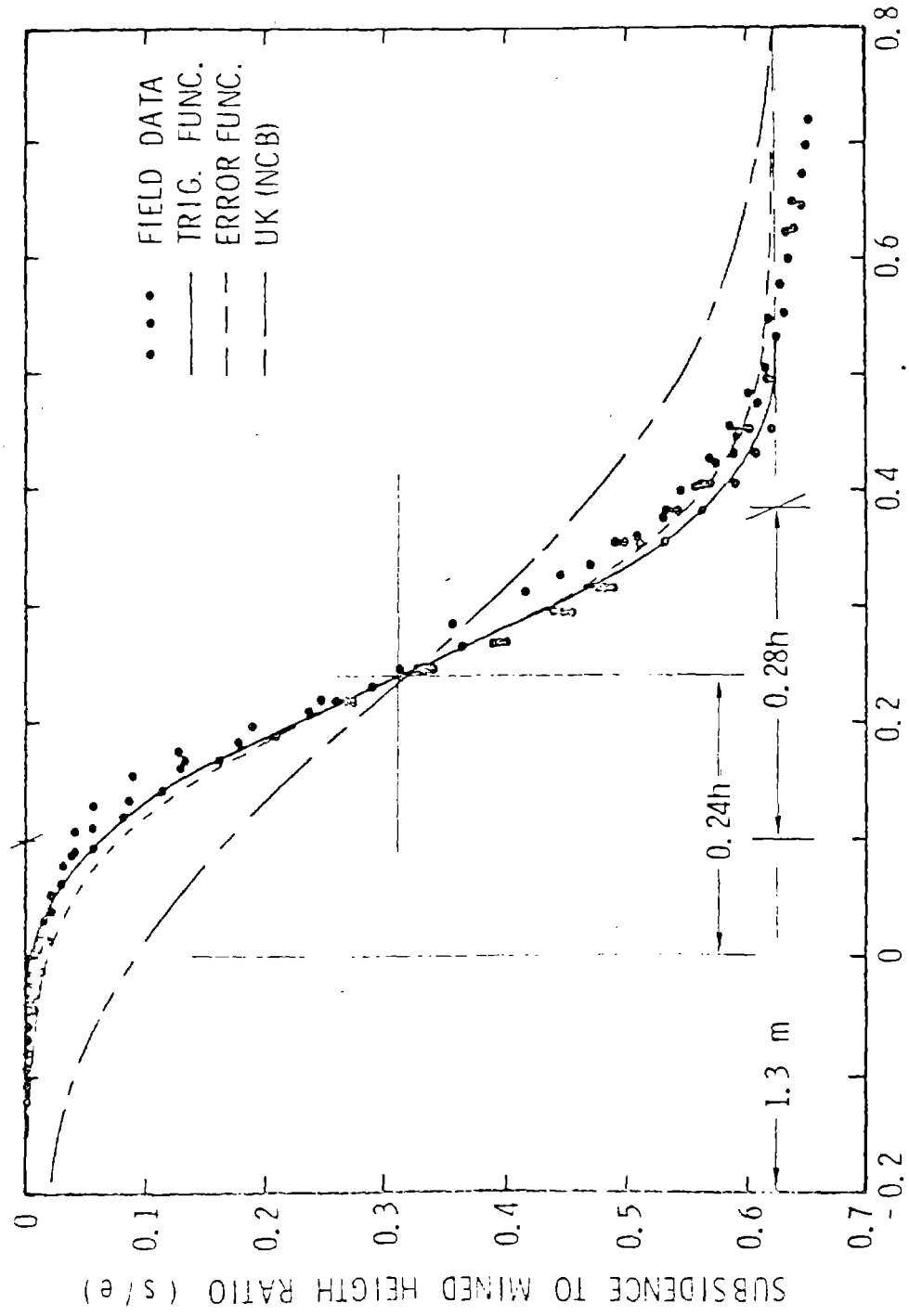
The duration of active surface subsidence in the Northern Appalachian Coal Field has typically ranged from 48 to 149 days, as dictated by the rate of mining. Young and Stoek (1916) concluded that the deeper the seam, the greater the duration of surface movement.

Cheng and Peng (1981) consider subsidence to begin and end when the subsidence rate first exceeds and then drops below 10 mm/month. Within this total period of subsidence T_1, active subsidence is taking place during the period T_2 when the subsidence rate exceeds 50 mm/month. T_1 equals $2.6h/V_{fa}$ and T_2 equals $1.4h/V_{fa}$, where V_{fa} is the rate of face advance. The maximum rate of subsidence V_o is equal to $1.8m\ V_{fa}/h$, where m is the extracted thickness of coal.

Adamek and Jeran (1980) have recognized that longwall subsidence profiles in northern West Virginia and eastern Ohio are sometimes markedly different from one another, even within a single mine. Principal differences between profiles reside in the position of the inflection point, which they define as the point of half maximum subsidence (Figure 3.16). The displacement of the inflection point toward the panel centerline is sometimes attributed to incomplete compaction of the gob close to the panel edges. Adamek and Jeran suggest this phenomenon is controlled largely by the percentage and distribution of resistant beds within the overburden, as shown hypothetically in Figure 3.17. The greater the bridging effect, the closer is the inflection point to the centerline of the panel.

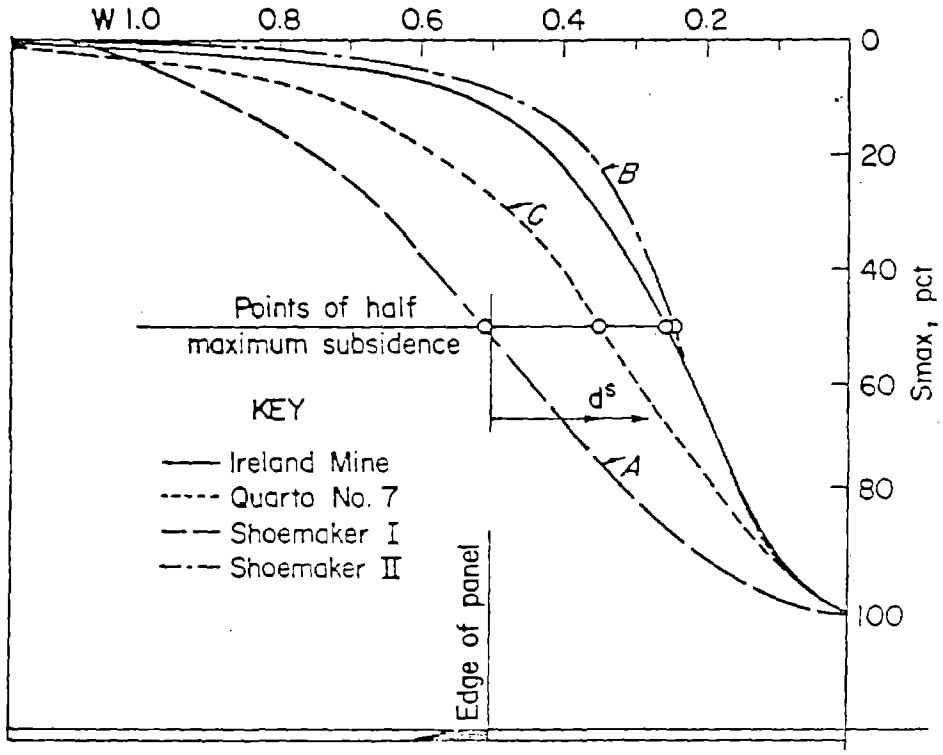
Heaving of the ground surface that accompanies advance of the mine face is another phenomenon that seems to be associated with the bridging action of the rock strata. Upward ground movements have been reported virtually as long as ground surveys have been conducted (e.g. Smith, 1930). Although once ascribed to survey error, they have now been confirmed by numerous careful field measurements.

The magnitude and limits of heaving are not well defined, but fluctuations of at least a few inches have been recognized. Heaving is often attributed to a cantilevering action of joint-bounded blocks of near surface competent strata that lift up and then settle down as the face passes each rock



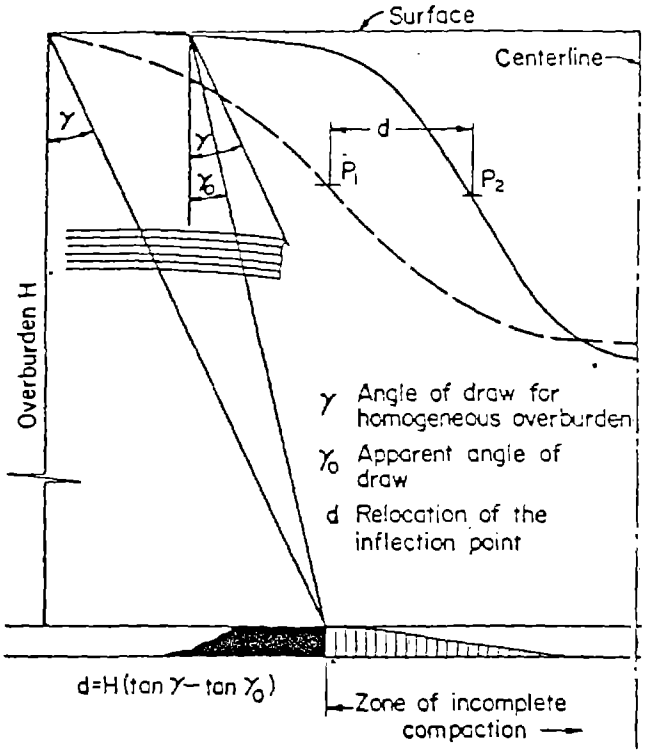
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FIGURE 3.15 – SUBSIDENCE DEVELOPMENT CURVE, OLD BEN NO. 24 MINE, BENTON, ILLINOIS (From Munson and Elchfeld, 1980)



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FIGURE 3.16 - SUBSIDENCE PROFILES OVER FOUR LONGWALLS IN NORTHERN APPALACIAN COAL FIELD (Adamek and Jeran, 1981).



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FIGURE 3.17 - HYPOTHETICAL ILLUSTRATION OF THE BRIDGING EFFECT ON A SUBSIDENCE PROFILE (Adamek and Jeran, 1981)

joint. Heaving obviously introduces an imponderable into the prediction and interpretation of ground movements.

2. Maximum Settlement

Figure 3.18 presents maximum subsidence data that have been reported over longwall and pillar retreat panels in the United States.

As per National Coal Board convention, the maximum settlement has been normalized relative to the thickness of the coal extracted (i.e. height of the mine) yielding a 'subsidence factor'. Data have been compiled by O'Rourke and Turner (1979), Kohli, Peng and Thill (1980, 1981), Karmis et al. (1980) and Abel and Lee (1980).

The subsidence factor becomes greater as the width of the panel increases relative to the mine depth. The limiting subsidence observed for all types of mining in the United States is slightly in excess of 0.7 mine height. Kohli et al. (1980) have found the subsidence factor for a given panel width increases somewhat with depth. The effect is minor and is probably due to the compression of the broken rock under the weight of the overburden. Some variation in measured subsidence is also probably related to local variations in mine height (Munson and Eichfield, 1980).

Additional subsidence sometimes occurs over a mined-out panel due to mining a new panel alongside. Kohli et al. (1980) report a case where settlement along the centerline of a mined-out longwall panel increased 15 percent from this cause. Conversely, the new panel experienced no greater maximum subsidence as a result of its being located adjacent to a mined-out panel. Although not documented, the writers believe the extent of interaction is much less pronounced for mined-out panels that exceed critical dimensions prior to mining alongside. Comparable movements take place above room and pillar mines.

Subsidence in pillar retreat panels tend to be less than in longwall panels of similar dimensions, probably due to the presence of remnant pillar stumps that retain some load carrying capacity upon crushing. Wardell (1958) and Wardell and Eynon (1968) have postulated that the subsidence factor above a room-and-pillar mining operation that results from pillar failure or squeeze is a function of the average vertical stress at the abutments, and the height/width ratio of the pillars. Figure 3.19 presents a logarithmic relationship between these parameters based on data assembled by Abel and Lee (1980) from room-and-pillar mines in Pennsylvania, Illinois, New Mexico and the United Kingdom. The relationship states that:

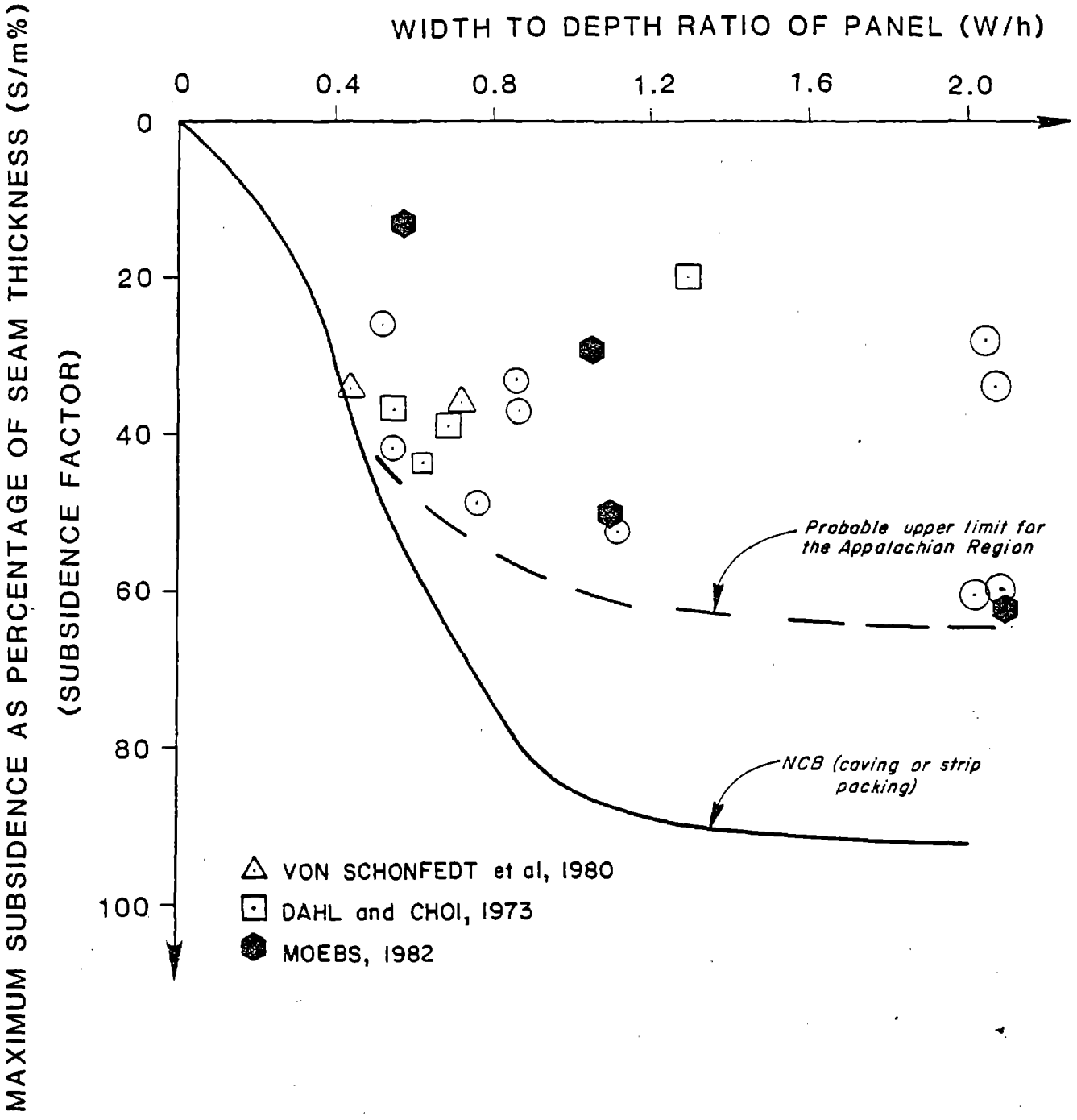
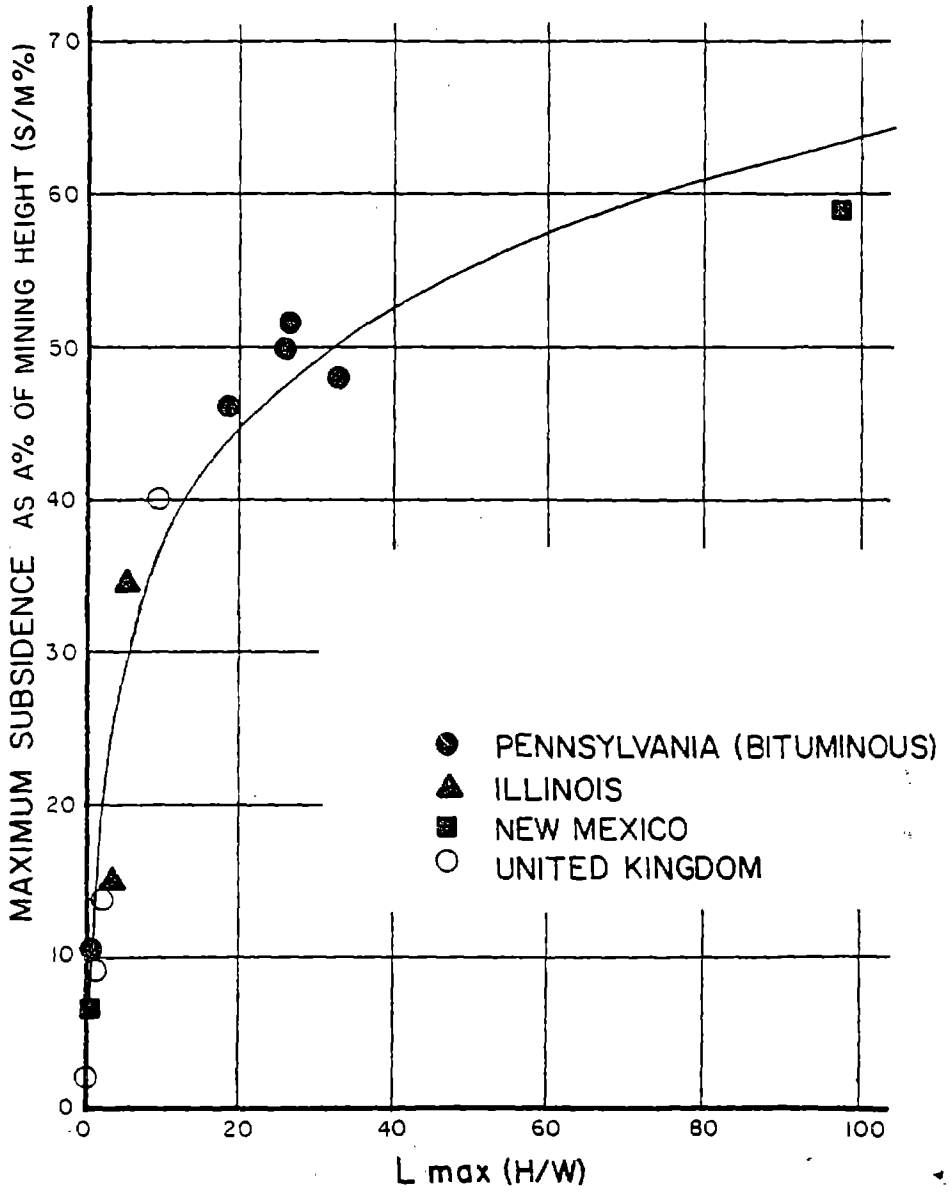


FIGURE 3.18 INFLUENCE OF PANEL WIDTH TO DEPTH RATIO ON SUBSIDENCE FACTOR



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FIGURE 3.19 - WARDELL/EYNON PLOT OF MAXIMUM SUBSIDENCE ABOVE ROOM AND PILLAR MINES
(Modified from Abel and Lee, 1980)

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$$(s/m)_{max}\% = 8.469 + 11.95 \ln [L_{max}(H/W)]$$

where $L_{max} = D/(1-R)$

D = mine depth
R = extraction ratio
H = mining height
W = pillar width
s = subsidence
m = extracted thickness of coal

Subsidence from both longwall and pillar retreat mining in the United States tends to be substantially less than in the United Kingdom, even when the latter has been adjusted to virgin conditions. The lesser subsidence in the United States is attributed primarily to the greater proportion of sandstone and limestone in the overburden.

Hunt (1980) has compared the magnitude of trough (sag) subsidence in Illinois resulting from room-and-pillar (partial extraction), pillar retreat and longwall mining (Figure 3.20). As would be expected, longwall, the most efficient mining method, causes the greatest subsidence for a particular mine panel width/depth ratio, followed by retreat mining and then room-and-pillar mining. The variability of subsidence at a given width/depth ratio for room and pillar mines is assumed to reflect the percent extraction, which ranged from 55 to 75 percent.

Very few measurements are available of subsidence troughs that have developed over abandoned partial extraction panels in the Northern Appalachian Coal Field. Cases measured over the years suggest maximum subsidence ranging from one-eighth to one-quarter the mine height. The percent extraction was probably on the order of 65 percent, although this cannot be confirmed.

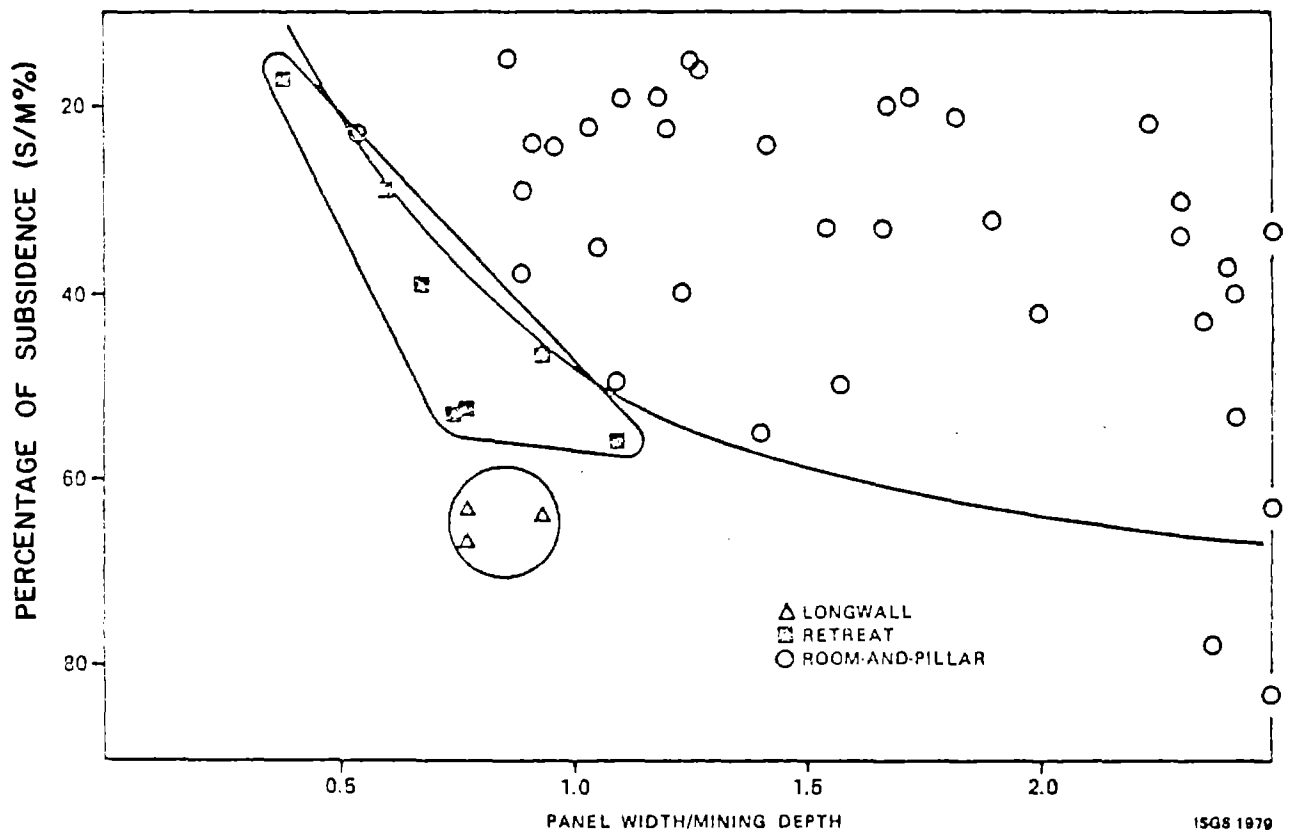
Able and Lee (1980), Peng and Cheng (1980) and Karmis et al. (1980) have all suggested the subsidence factor decreases with increase in the proportion of strong rock units in the overburden. Data presented by Abel and Lee (1980) can be interpreted as follows:

$$s/m\ \% \geq 80 - \%\ \text{shale}$$

$$s/m\ \% \geq \%\ \text{Sandstone} - 10$$

$$s/m\ \% \leq 90 - \%\ \text{limestone}$$

The triangular plot in Figure 3.21 prepared for the present study shows this information in graphical form. The data are much too variable to establish any more than broad bounds for expected subsidence on the basis of lithology.



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FIGURE 3.20 RELATIONSHIP BETWEEN PERCENTAGE OF SUBSIDENCE AND PANEL WIDTH/MINING DEPTH
(Hunt, 1980)

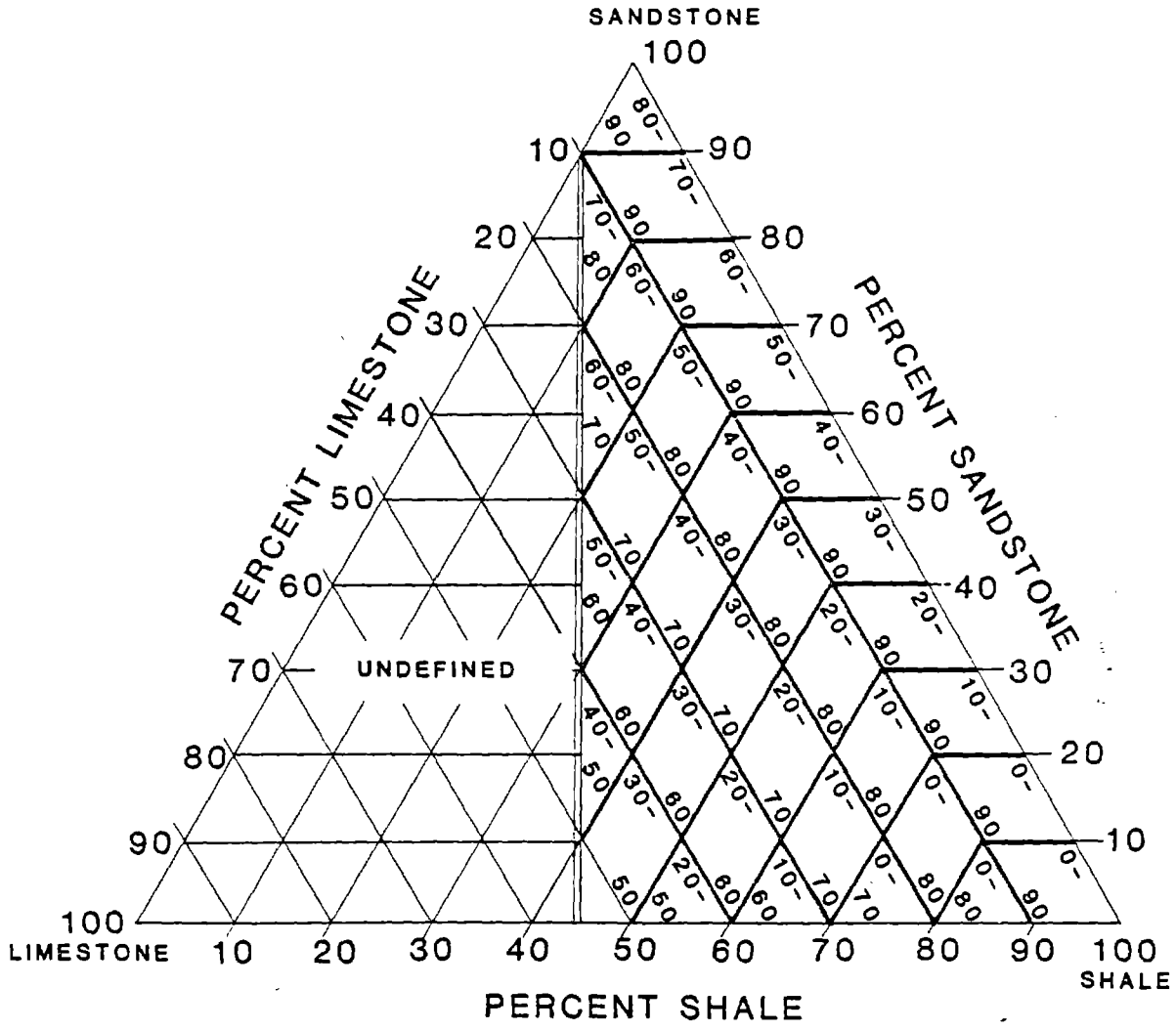


FIGURE 3.21 – INFERRED RANGES OF (S/m)%, MAXIMUM SUBSIDENCE AS A PERCENT OF MINING HEIGHT (PREPARED FROM DATA PRESENTED BY ABEL AND LEE, 1980).

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More recently, Tandanand and Powell (1982) have found that the subsidence factor 'a' (=s/m%) for longwall panels in the Northern Appalachian Coal Field is related to 'n', the proportion by thickness of sandstone and limestone in the overburden, according to: $a = 0.94 - 1.5n$. This expression is based on data from 16 longwall panels and has been found to yield estimates within 25 percent of measured values.

Dahl and Choi (1973) and Kohli et al. (1980) indicate that the amount of subsidence in the Northern Appalachian Coal Field seems not to depend on the natural ground surface slope. Ground surface slope is not a consideration in the Illinois Coal Field.

3. Critical Panel Dimensions

Maximum possible subsidence at a point is reached in the United States when both the width of the mine panel and the position of the point relative to the nearest panel edge both equal or exceed 1.2 h (Karmis, et al., 1981). Cheng and Peng (1981) suggest the value in the Northern Appalachian Coal Field is more nearly 1.11 h. This compares to 1.4 h in the United Kingdom. Critical panel dimensions have not been reported for the Interior Coal Province.

4. Transverse Subsidence Profile

Forty-two longwall subsidence profiles situated perpendicular to the direction of mining were analyzed by Cheng and Peng (1981). The surveys were conducted by coal operators in Ohio, Pennsylvania and West Virginia in the period 1962 to 1979 and in 1980 were supplemented by additional data. Twelve cases were of supercritical width; 30 were subcritical. Principal observations concerning the data are as follows:

1. Exponential and hyperbolic functions fit the observed profiles reasonably well.

2. Full subsidence profiles in virgin conditions are symmetrical about the panel centerline, but half profiles are not symmetrical about the ribside or inflection point.

3. Supercritical profiles exhibit "pan"-shaped profiles--points in the center interval of the panel subsiding uniformly and reaching the maximum possible for the panel. Subcritical profiles exhibit "bowl"-shaped profiles--the center point of the panel experiencing the largest subsidence, albeit less than the maximum under critical/supercritical conditions.

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4. The subcritical inflection point is situated toward the panel centerline rather than directly over the ribline and does not correspond to half the largest subsidence.

5. Where new panels are located adjacent to mined-out panels, the profile over the new panel tends to be asymmetrical relative to the panel centerline, with relatively greater settlements toward the mined-out side.

6. Qualitatively, transverse profiles over pillar retreat and longwall panels are similar, although profiles over pillar retreat panels may be somewhat more irregular owing to the presence of remnant pillar stumps. Data concerning ground slope, curvature, horizontal displacement and strain associated with pillar retreat mining are virtually nonexistent in the Northern Appalachian Coal Field, although at least rough estimates could be synthesized from available data.

Cheng and Peng (1981) conclude that an exponential function best fits the transverse subsidence profiles of longwall panels in the Northern Appalachian Coal Field. Munson and Eichfield (1980) conclude that a trigonometric function best fits the transverse subsidence profile of the Old Ben No. 24 longwall panel near Benton, Illinois.

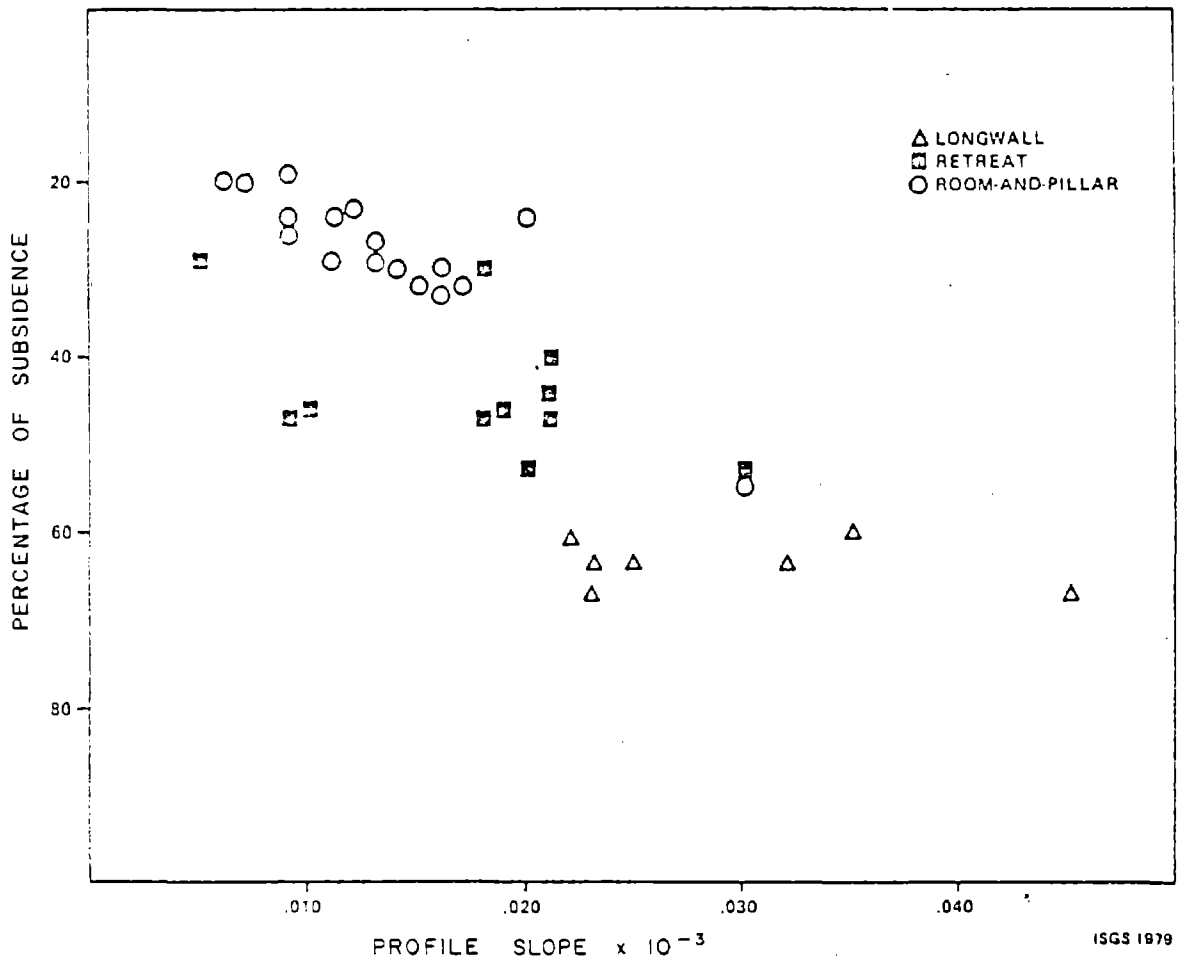
Maximum tensile horizontal strains of 2 percent have been measured at the Old Ben No. 24 Mine. O'Rourke and Turner (1979) point out that these strains are three to four times greater than predicted by NCB charts for comparable mining conditions in the United Kingdom. Ground strains in the Northern Appalachian Coal Field may be somewhat less than those in Illinois, but still more pronounced than those in the United Kingdom.

Hunt (1980) has investigated profile shapes for about 180 sag-type trough subsidence incidents in Illinois produced by longwall, pillar retreat and room-and-pillar (partial extraction) mining and has observed the following:

1. Profile slope (the tangent of the subsidence profile at the inflection point) increases in proportion to the percent subsidence (s/m) (Figure 3.22).

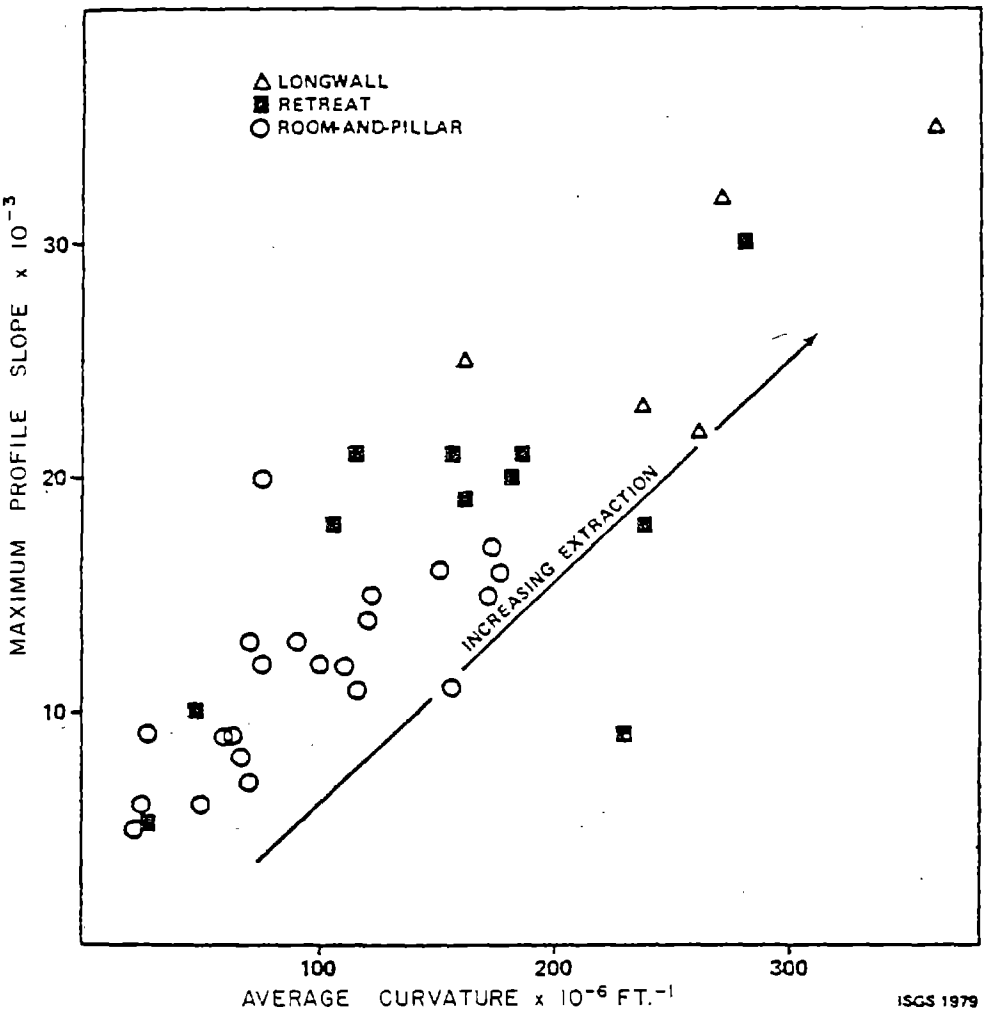
2. Average maximum curvature increases in proportion to the maximum profile slope (Figure 3.23).

3. Maximum tensile curvature, on the average, is equal but opposite to maximum tensile curvature (Figure 3.24).



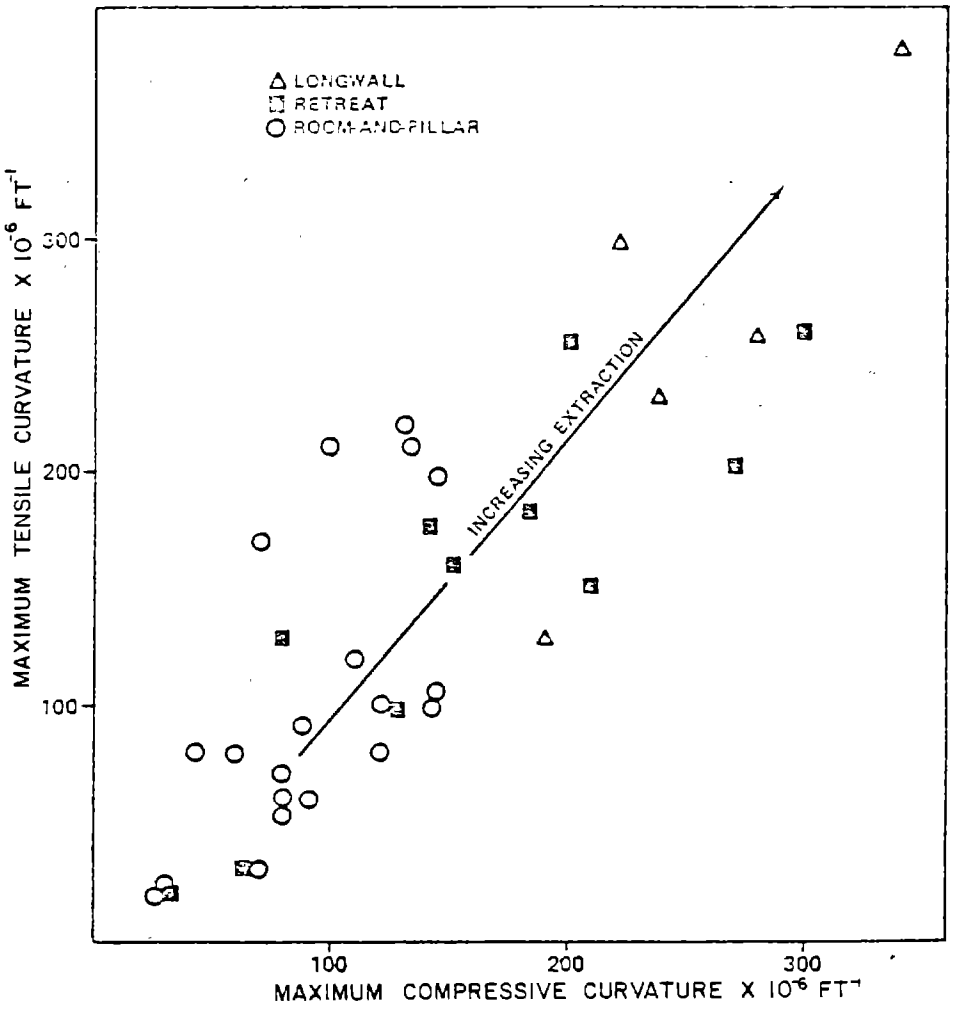
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FIGURE 3.22 RELATIONSHIP BETWEEN THE PROFILE AND THE PERCENTAGE OF SUBSIDENCE IN ILLINOIS
(Hunt, 1980)



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FIGURE 3.23 - RELATIONSHIP BETWEEN AVERAGE MAXIMUM CURVATURE AND MAXIMUM PROFILE SLOPE, ILLINOIS (Hunt, 1980)



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FIGURE 3.24 - RELATIONSHIP BETWEEN THE MAXIMUM TENSILE AND COMPRESSIVE CURVATURES SHOWING THE MINING METHODS
(Hunt, 1980)

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5. Angle Parameters

Angle parameters that characterize the subsidence profile include angle of draw (limit angle), angle of break, angle of complete mining and angle of critical deformation (Figure 3.25). The 'angle of draw' is defined as the angle with the vertical of a line connecting the edge of the panel to the point of zero subsidence at ground surface. The 'angle of break' is the angle with the vertical of a line connecting the edge of the panel to the point at ground surface where maximum tension occurs. The 'angle of complete mining' is the angle with the vertical of a line connecting the edge of the panel to the outermost point of maximum subsidence in the profile. The 'angle of critical deformation' is the angle with the vertical of a line connecting the edge of the panel 'with the point at ground surface at which a specified ground deformation has been exceeded.

The angle of draw, unless otherwise stated, is determined from stationary subsidence profiles, and in the United States ranges between 5 and 45 degrees, and most commonly between 20 and 28 degrees. The angle of draw for pillar retreat panels may be a few degrees higher than for longwall panels, but the data are not definitive. Moebs (1982) observed little consistency in the angles of draw (or other parameters) between four pillar retreat mines in southwestern Pennsylvania. He measured angles of draw between 9 degrees and 41 degrees and speculated that at least some of the variations were due to geologic conditions or surface effects, such as slope instability. Kohli, Peng and Thill (1980) report that angles of draw in excess of 28 degrees for longwall panels in the Northern Appalachian Coal Field were generally measured adjacent to gob areas mined by the room-and-pillar method. Adamek and Jeran (1980) suspect that locally the angle of draw in the Northern Appalachian Coal Field may be as little as 12 to 17 degrees owing to the bridging of competent strata in the upper overburden interval. This is based on a comparison of measured and predicted ground profiles using Bal's theory.

Data available world-wide broadly suggest that the angle of draw depends on overburden composition (Table 3.2). It is interesting to note, for example, that a representative angle of draw in the United States is about 22°, whereas in the United Kingdom it is 35° and in the Netherlands it is 45°. In the United States, overburden composition by rock type generally has not been found to bear a strong relationship to angle of draw (Figure 3.26) although Abel and Lee (1980) claim a relationship exists between angle of draw and percent sandstone and limestone. The apparent absence of a trend between angle of draw and rock type may be due non-representative or poor boring log data, poor survey data, the failure to account for the position of the strata in the rock column, failures to recognize that angle of draw can vary with position in a panel even without geologic variations, and other factors.

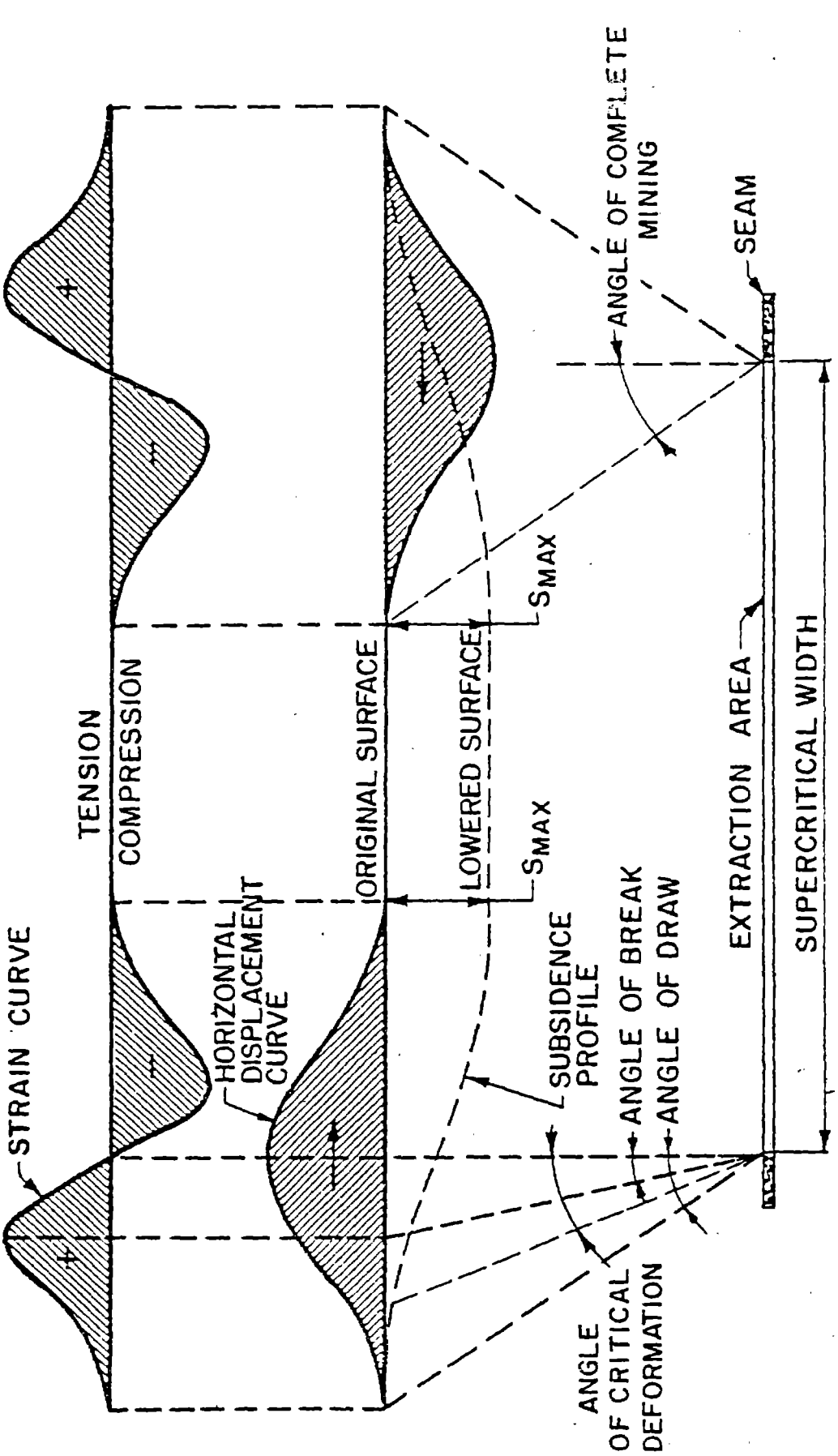


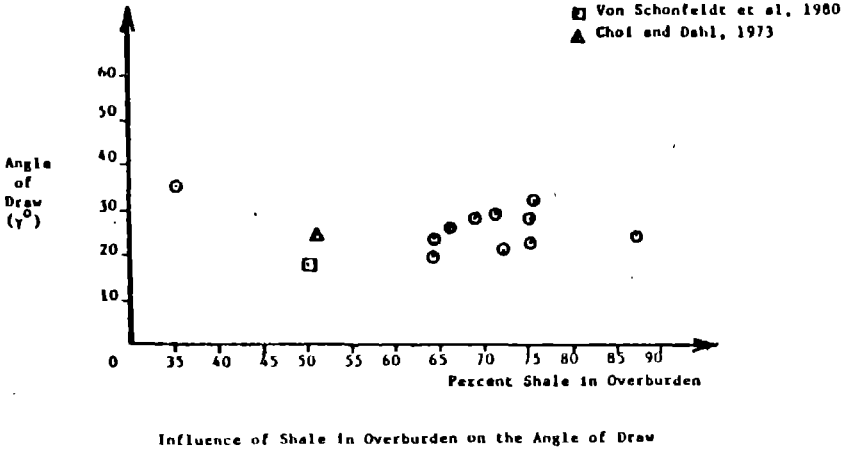
FIGURE 3.25 – DEFINITIONS OF ANGLE PARAMETERS OF SUBSIDENCE TROUGH.

TABLE 3.2

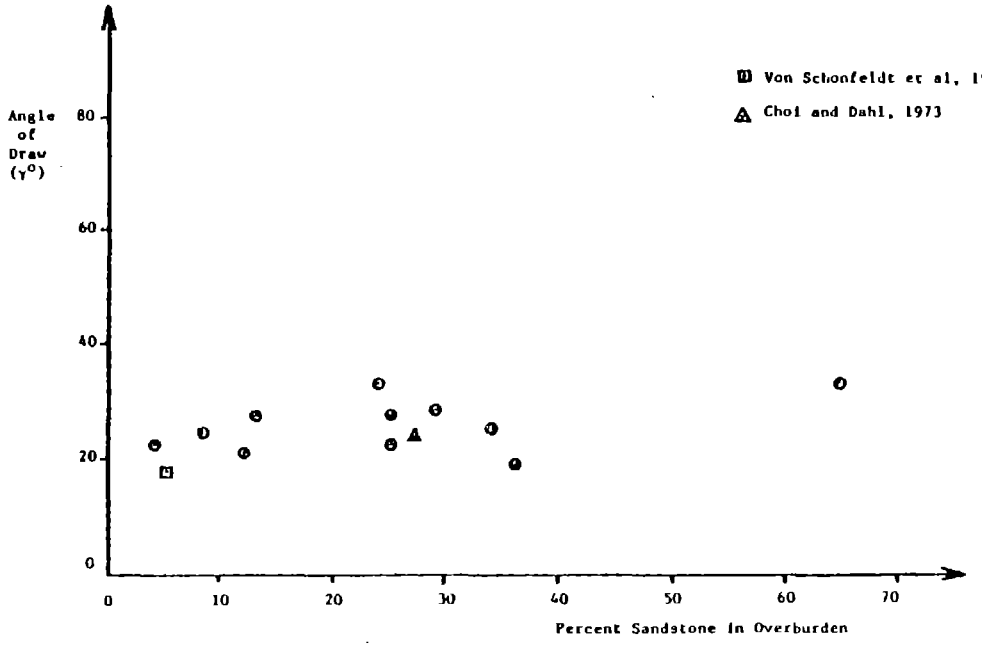
REPRESENTATIVE ANGLES OF DRAW
FROM COAL MINES WORLD WIDE HAVING
VARIOUS OVERBURDEN COMPOSITIONS

	Angle of Draw	Ground Conditions	Location
1.	45°	Deep Alluvial Sediments Overlying Rock Overburden	Limburg Coalfield, Netherlands
2.	41°	Deep Alluvial Sediments Overlying Rock Overburden	Trent River Valley, United Kingdom
3.	35°	Rock Overburden, Predominently Shales and Siltstones	Midlands Coalfield, United Kingdom
4.	23°-29°	Rock Overburden, Predominently Shales and Siltstone	Benton, Illinois
5.	26°	Rock Overburden, with Thick, Strong Sandstone Beds	Parts of the European Coalfields
6.	25%	Rock Overburden, with Thick, Strong Sandstone Beds	Scranton and Wilkes-Barre, Pennsylvania
7.	19°-37°	Rock Overburden, with 56% Shale, 26% Sandstone, 14% Limestone, Thin Soil	Green County, Pennsylvania
8.	16%	Rock Overburden, with Thick, Strong Sandstone Beds	Raton, New Mexico
9.	8°-15°	Rock Overburden, with 40% Siltstone/Shale 25% Sandstone, 30% Claystone, 5% Other. Thin Soil	Barbour County, West Virginia

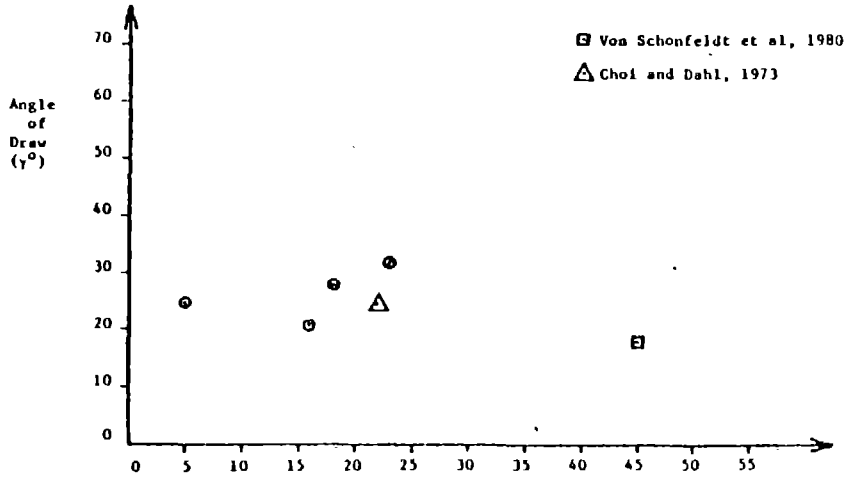
NOTE: Entries 1-6 and 8 were compiled from a variety of sources by O'Rourke and Turner, 1979. Entry 7 is from Moebs (1982). Entry 9 is from the writers' experience. Both longwall and pillar retreat are represented.



Influence of Shale in Overburden on the Angle of Draw



Influence of Sandstone in Overburden on the Angle of Draw



Influence of Limestone in Overburden on the Angle of Draw

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FIGURE 3.26 – RELATIONSHIPS BETWEEN ANGLE OF DRAW AND ROCK TYPE
(from Karmis et al, 1981)

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Little specific information concerning soil cover or near surface rock strata are available for most reported subsidence cases. Because soil is commonly less than twenty feet thick in the Northern Appalachian Coal Field, its presence is not expected to have been a major influence overall on angles of draw that have been reported. The range of angles of draw for the U.S.A. compiled by Hunt (1980), Figure 3.27, are believed to reflect variations in rock strata much more than soils. Specific cases where soil and near surface rock has influenced the angle of draw are discussed in a subsequent section.

In the United States, the angle of draw and angle of complete mining seem to be about equal. Cheng and Peng (1981) calculate that the angle of break for longwall panels in the Northern Appalachian Coal Field is about -3° (that is, over the mined panel), while the angle of critical deformation is about +4°. They base the angle of critical deformation on a critical ground surface curvature of 0.2 mm/m^2. This choice is arbitrary in that the level of critical deformation actually depends on the character of the surface structure.

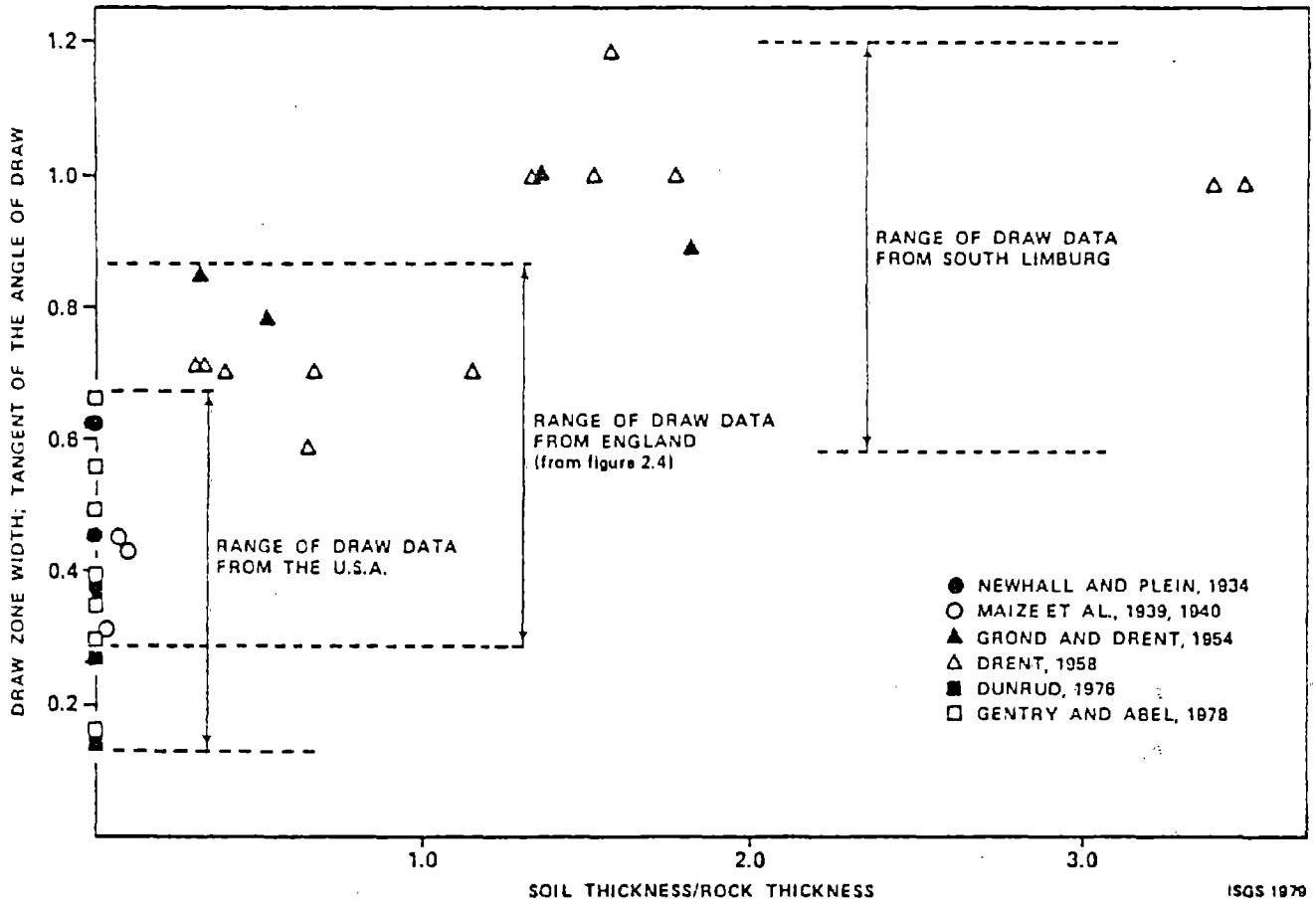
Data available at the present time are insufficient to determine if the angle of draw is greater on the dip side and smaller on the rise side of a dipping coal bed, as has been observed by Marr (1959) in the United Kingdom.

Conclusions Concerning Subsidence Parameters in the Northern Appalachian Coal Field and Illinois Basin

Significant progress has been made within the past few years in collecting and synthesizing available subsidence data in coal fields across the United States. By far most data are found in the Northern Appalachian Coal Field and the Illinois Basin.

The data demonstrate very clearly that subsidence above active mines is distinctly different from that over abandoned mines in terms of the plan and profile geometry, the time and place of occurrence and, to a certain extent, the mechanism. An understanding of one type of subsidence does not guarantee an understanding of the other. Conversely, it seems clear that the manner in which subsidence occurs above active or abandoned mines is much the same, whether it takes place in Illinois or in the Northern Appalachian Coal Field.

Distinct geologic differences have been recognized between and within the Appalachian and Illinois regions. To an extent not yet fully determined, these differences are reflected in variations in angles of draw, as well as in profiles of the subsidence development curves and other parameters. Unfortunately, the data are not sufficiently extensive or refined to establish a reliable relationship between subsidence parameters and mine site stratigraphy as given by one or more borehole geologic logs. The inability to relate reported subsidence parameters to basic



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FIGURE 3.27 - RELATIONSHIP BETWEEN THE TANGENT OF THE ANGLE OF DRAW AND RATIO OF SOIL TO ROCK THICKNESS (Hunt, 1980)

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geologic constraints makes impossible a separate identification of the influence of soil and near surface rock on these parameters.

Moreover, very few measurements exist of ground strain, slope and curvature that would be of use in relating mining to severity of damage to buildings at ground surface. Data plots that have been synthesized for longwall, room-and-pillar and pillar retreat in Illinois and the analytical expressions developed for longwall profiles in the Northern Appalachian Coal Field are noteworthy achievements. However, relatively few cases of subsidence damage to homes have been documented above longwall panels in the Northern Appalachian Coal Field, and virtually none have been documented above active longwall or room-and-pillar mines in Illinois. Similarly, the number of well documented cases of home damage above abandoned mines in Illinois is rather small at this time, although several cases are currently under investigation. Importantly, no systematic subsidence profile data have been collected and synthesized for the vast majority of home damage cases that have been documented above abandoned and active room-and-pillar and pillar retreat mines in the Northern Appalachian Coal Field. As a consequence, no method is available to identify the ground strains, slope and curvature prevailing at the time of damage, except for those damage cases above abandoned mines where the trough is still visible. The lack of data linking ground movement and home damage makes the identification of soil and near surface rock effects on damage severity nearly impossible in a quantitative sense. Even so, some insight into these effects can be gained by examining the mechanics of the subsidence process and by reviewing specific subsidence cases in the United States and abroad where soil and near surface rock have had a definable impact on the subsidence process and on surface structures. This is done in the following section.

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4. POTENTIAL INFLUENCE OF SOIL AND NEAR SURFACE ROCK STRATA IN MODIFYING SUBSIDENCE-RELATED GROUND MOVEMENTS

Soil and near surface rock strata effects on subsidence-related ground movements are perhaps of most interest when one is faced with having to determine why ground movements have taken place at a particular site or with having to forecast the character and probable severity of future ground movements. Questions sometimes posed are these:

1. Do certain types of soil or near surface rock at a site offer potential sources of ground movement that could be mistaken for subsidence or could intensify subsidence damage?

   McKay et al. (1980), for example, indicate that in the Belleville/Maryville area of southwestern Illinois, vertical ground movements of 0.2 to 0.4 inch result from the shrinking and swelling of loessal soils in response to alternating seasonal wet and dry periods. Although not harmful to undamaged structures, these ground movements appear to aggravate existing damage brought about by mine subsidence.

   From a similar perspective, Case Histories 1 and 2 (Appendix A) discuss the difficulties sometimes encountered in differentiating landslides from subsidence above abandoned underground coal mines. Section 2 of this report has already described a wide variety of ground movements, including consolidation settlement, collapse of the soil structure, heave and others, that might combine with, or perhaps be confused with, subsidence. Careful investigation, possibly involving the use of settlement plates, extensometers, inclinometers, and the like, may be required to sort out the individual sources of ground movement.

   Another question pertinent to soil and near-surface rock effects is the following.

2. Does the presence or absence of a particular type of soil or rock unit make one site more susceptible to subsidence than another?

   Wildanger (1980), for example, reports that in the St. David, Illinois, area, sinkholes tend to develop in locations where the St. David Limestone--the roof rock in many abandoned mines--is replaced by discontinuous concretions or fossiliferous shales. Ravelling and piping of the overlying loess through roof falls in the shale result in large voids in the loess that collapse to form sinkholes. This is discussed further in Case History 3.

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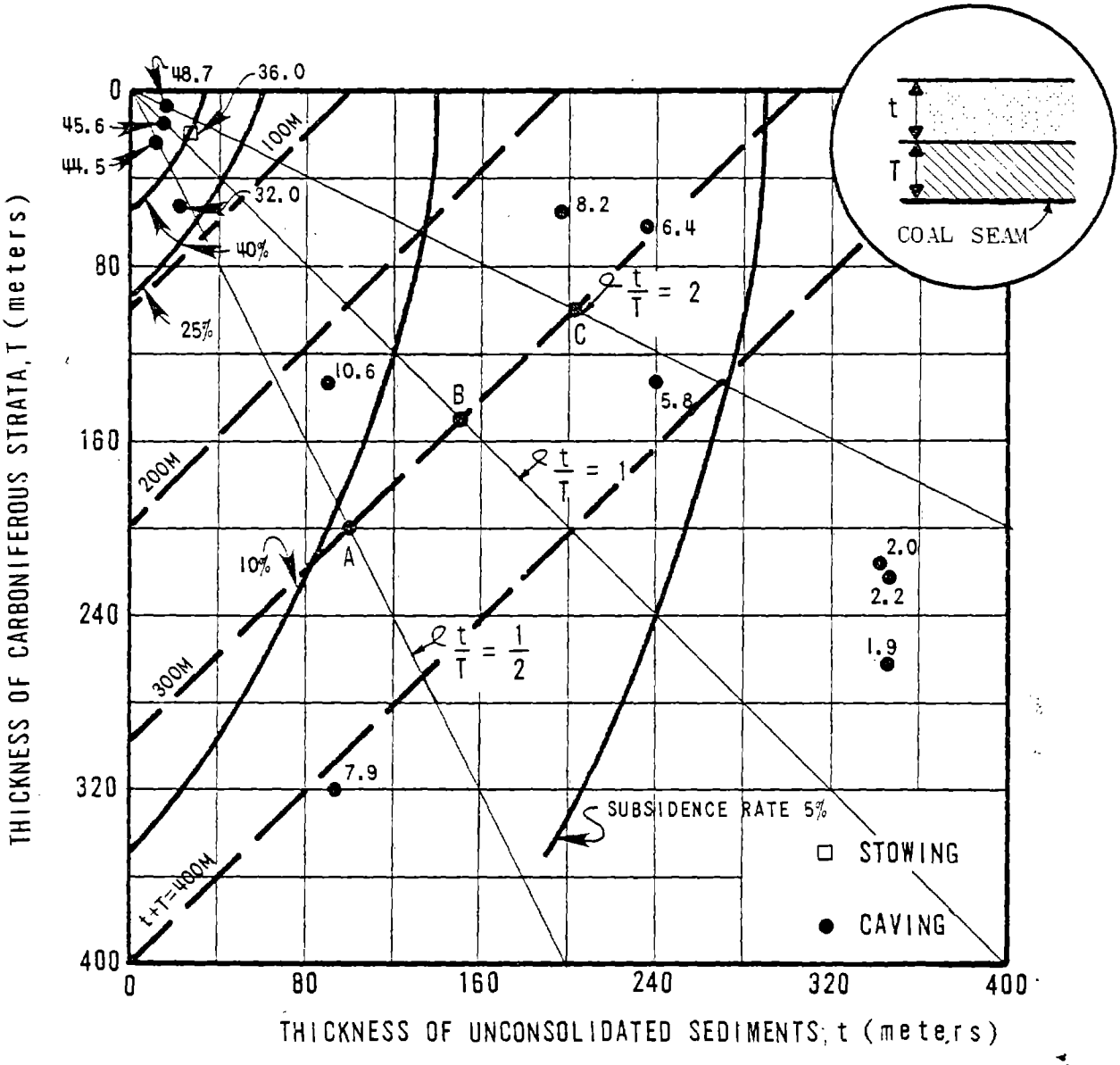
Elsewhere in Illinois, Hunt (1980) observes that several incidents of subsidence associated with severe heave of the mine floor were located above buried bedrock valleys infilled with Quaternary deposits more than 100 feet thick--soil thicknesses equal to or greater than the thickness of underlying rock strata. The implication of the buried valley scenario has not been resolved--perhaps it is associated with increased ground water seepage to mine level near the bedrock lows. Circumstances suggest, in any case, that adverse soil/rock conditions may have influenced the outcome.

A third often relevant question is the following.

3. Might the soil and near surface rock strata influence mining-induced ground movements or cause them to depart from the norm? Is the departure predictable or unpredictable?

Drent (1957), for example, observed in his work in the South Limburg Coalfield, Netherlands, that the rate of subsidence during mining is dependent on the thickness of unconsolidated sands and marl overlying the Carboniferous shale and sandstone strata above the mine. He cites as evidence a plot showing the relationship between thicknesses of rock strata and unconsolidated material (Figure 4.1), on which are shown lines of equal subsidence rate expressed as a percentage of total subsidence for a constant daily advance of the mine face. The plot indicates that for any total overburden thickness (t+T), the rate of subsidence decreases as the proportion of unconsolidated overburden increases. (Compare Points A, B and C). The total overburden thickness in the cases investigated by Drent ranges from 38 to 598 m (125 to 2000 ft.). The unconsolidated overburden represents from 17 to 57 percent of the total overburden thickness. The nominal dip of the coal seam is about 11 degrees. Drent did not attempt to explain his observations.

Turchaninov's (1979) experience in the USSR indicates that sediment thickness also has a bearing on the ground displacements associated with dipping rock strata undergoing subsidence. He finds that where the thickness of Tertiary and Quaternary sediments is less than 20 meters and less than 10 percent the thickness of the rock strata, the sediments have practically no influence on the subsidence process. But where the sediment thickness is 10 to 50 percent the thickness of the rock strata, the sediments modify ground movements as if to reduce the dip of the rock strata α to a value α', given by Arctan [tan α - h/H], where h is sediment thickness and H is overburden thickness, negative values of α' being taken as zero. Where the thickness of sediment is over 100 m and exceeds 50 percent the thickness of the



SUBSIDENCE RATE = Percentage of maximum subsidence over a five-day period with 2 meter advance of longwall face.

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FIGURE 4.1 - SUBSIDENCE RATE AS A FUNCTION OF UNCONSOLIDATED SEDIMENTS, SOUTH LIMBURG COALFIELD , NETHERLANDS (AFTER DRENT,1957)

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rock strata, observed ground movements are equivalent to those associated with horizontal rock strata, regardless of the dip of the strata.

Medyanstev et al. (1962, 1963) present equations appropriate to the USSR for maximum displacement and maximum horizontal strain that include the effect of sediment thickness. Maximum horizontal displacement is given by:

$$\xi_{max} = S_{max}(0.3 + 0.8\ [\tan \alpha - h/H])$$

Maximum horizontal strain is given by:

$$\varepsilon_{max} = (0.6 + 0.9\ \alpha/[\tan \alpha - h/H])\cos \alpha$$

Of interest is the fact that the influence of sediment thickness on horizontal displacement and strain is minimal when h/H and the dip of the rock strata are small, and is altogether nil when the dip angle is less than or equal to Arctan (h/H). Of further interest is the fact that soil properties do not enter into the calculations of displacement or strain.

Soil thicknesses in the United States are often modest--generally a few feet to a few tens of feet--somewhat similar to those in the United Kingdom. The influence on subsidence of near surface jointed rock strata in these situations can become significant. This has been confirmed by Shadbolt and Mabe's study (1974) of National Coal Board records, which has revealed that, because of near surface effects, about one out of every four subsidence cases in the United Kingdom fails to conform to conventional trends.

As is evident from these examples, numerous factors prompt the study of how soil and near surface rock strata affect the subsidence process. Still, relatively little work has been done on the subject for reasons such as the following:

1. The complexity of the subsidence process and the fragmentary nature of many subsidence records, both of which have promoted an empirical approach to subsidence prediction wherein soil characteristics are lumped with other geologic parameters in an implicit manner;

2. The predominence of mining engineers in the practice of subsidence engineering and their general lack of familiarity with methods of soil classification and analysis; and

3. The feeling among some engineers that near surface soil and rock are not ordinarily major determinants of the character of ground movements or the severity of damage to structures.

Whatever the reasons, there is a need to clarify the issue for the benefit of those whose who are designing structures in areas scheduled for future undermining. Available information is sketchy. The balance of this section is devoted to a review of basic principles and examples that illustrate influences that have been reported to date.

Influence of Near Surface Materials on the Distribution of Ground Strain

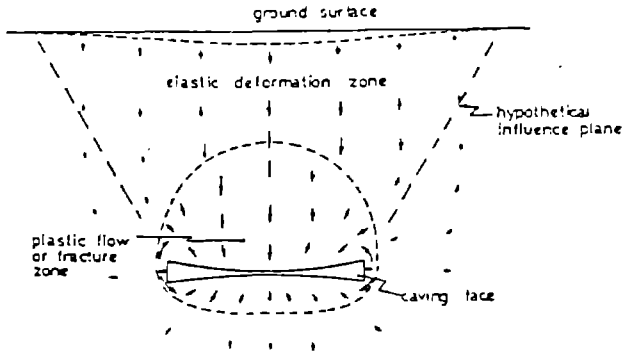
The similarity of the subsiding area to a downward deflecting beam (or plate) clamped at the perimeter can be recognized from Figure 4.2 which depicts one of the more widely held conceptions of how overburden subsides above a longwall panel (Denkhaus, 1964). Similar comparisons can be drawn for subsidence above a room-and-pillar panel. A more detailed consideration of classical beam behavior sheds some light on how near surface materials may act in such situations.

Beam Analogy. One of the basic precepts of structural engineering is that the longitudinal strain at any point on the top surface of an elastic beam is proportional to the radius of curvature at that point. Furthermore, the strain is independent of the physical dimensions or material properties of the beam. For example, two beams of identical size--one of wood, the other of steel--both deflected the same amount (Figure 4.3), exhibit the same distribution of longitudinal strain along their top surfaces. This is similarly true of a composite beam--one fabricated of lengthwise layers of different materials. The longitudinal strains on the top surface of the composite beam are the same as those of either single-material beam, as long as the radii of curvature of the beams are the same. By this reasoning, near surface ground strains in a subsiding area are expected to depend more on the curvature of the ground surface than on the types of soil and near surface rocks, their properties or their thicknesses.

Of course, the material properties and the geometry of a beam, as well as the individual layer thicknesses, govern the beam's stiffness and thereby its deflection under a particular set of boundary conditions. In situations where the topmost layer of a composite beam is much thinner and more compliant than the layers beneath, the curvature is controlled by the stiffer underlying layers. Because the soil mantle is often much thinner and compliant than the rock strata beneath--and, in fact, might be compared to a coat of paint on a beam--the rock strata rather than the soil govern the curvature of the ground surface and thereby the magnitude of the ground strains.

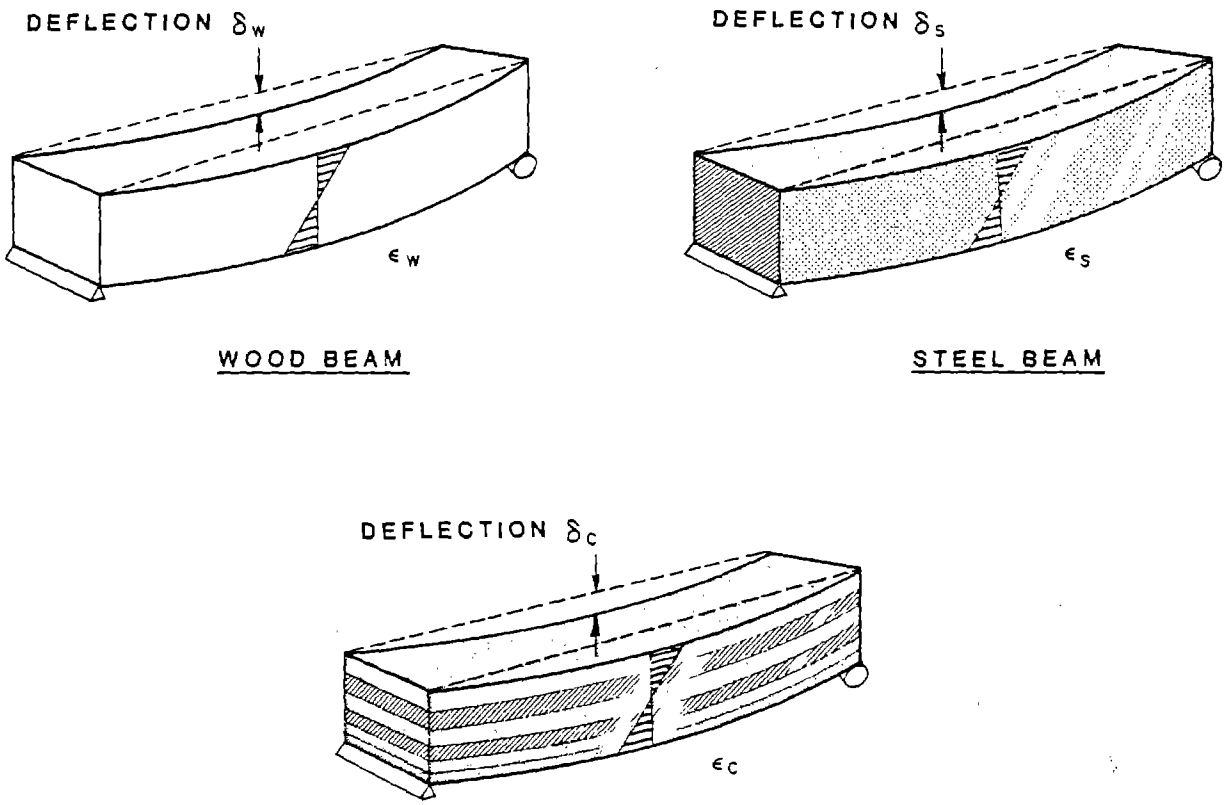
These concepts suggest the following:

1. In two subsiding areas identical in all respects except for properties of the rock strata, the distribution and magnitude of ground strains might be dissimilar due to differences in curvature assumed by the rock strata, even



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FIGURE 4.2 - CONCEPTUAL REPRESENTATION OF OVERBURDEN ADJUSTING TO UNDERGROUND MINING (from Wiggil,1963).



DEFLECTION $\delta_w = \delta_s = \delta_c$

LONGITUDINAL STRAIN $\epsilon_w = \epsilon_s = \epsilon_c$

FIGURE 4.3 - COMPARISON OF LONGITUDINAL STRAINS AND DEFLECTIONS OF THREE BEAMS OF DIFFERENT MATERIALS

though the properties, thicknesses and types of soil in both areas are the same; and

2. In two subsiding areas having soils of different types, properties or thicknesses, but identical in all other respects, including ground surface curvature, the distribution and magnitude of ground strains in both areas could well be the same. Differences in strain, if any, would be expected in locations where each soil has responded in its own way after the onset of the plastic state.

Plastic Behavior. The term 'plastic' pertains to a material that has yielded and for which strain is no longer directly proportional to stress. Plastic behavior is commonly manifested as shearing, cracking, buckling or crushing.

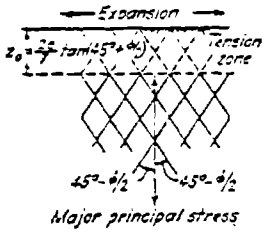
Should a soil deform uniformly after assuming a plastic state, the distribution of ground strain would continue to be proportional to ground surface curvature. (The reader is directed to Timoshenko and Gere, 1972 [Chapter 9] for a discussion of strains associated with the inelastic bending of beams.) Many sands, silts or clays might be expected to behave this way in a subsiding area, each soil adjusting to the new curvature of the rock strata through mechanisms described by Terzaghi (1943):

1. In the tensile zone, by shearing along a series of very closely spaced surfaces inclined at approximately $45°-\phi/2$ to the vertical (Figure 4.4A).

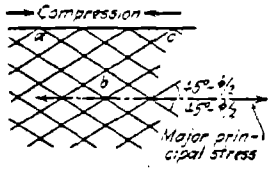
2. In the compressive zone, by shearing along a series of very closely spaced surfaces inclined at approximately $45°-\phi/2$ to the horizontal (Figure 4.4B).

The first mechanism describes what is commonly called the 'active' condition, because the weight of the soil assists in the shearing process, and the second, the 'passive' condition, because the weight of the soil opposes the shearing process. The shear surfaces change orientation slightly when the ground surface is sloped, and curve somewhat when the soils are clayey (Figures 4.4C, D, E). These details not withstanding, the direct relationship between ground strain and ground surface curvature is maintained because the soil deforms uniformly. Should the soil or near surface rock shear non-uniformly, or should it crack, buckle or crush, then the ground strain distribution may assume a dependence on the type of soil or near surface rock, its properties or its thickness.

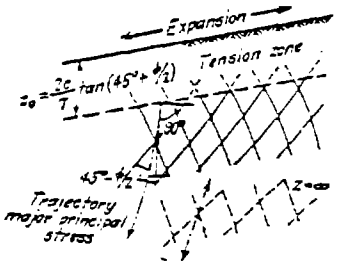
1. Buckling and Crushing. Compression features have been reported by Rockaway and Elifrits (1979) in connection with subsidence effects in the vicinity of Rend Lake Dam, Illinois; by Dunrud (1975) in connection with subsidence in Colorado; and by Tilton (1966) in connection with natural gas pipelines in Pennsylvania and West Virginia. Tilton reports pipelines being heaved several feet out of



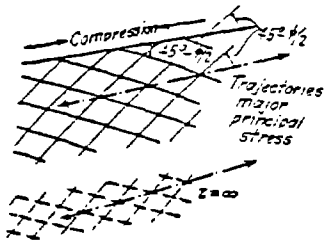
A.



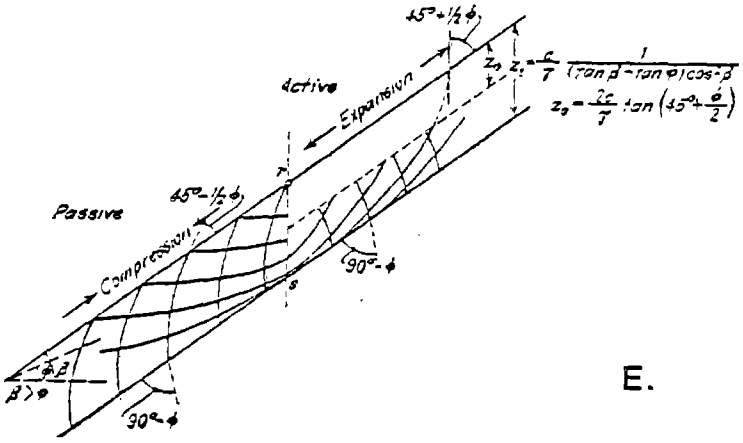
B.



C.



D.



E.

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FIGURE 4.4 – PLASTIC DEFORMATION OF SOIL BY SHEARING. LEVEL GROUND SURFACE: A. ACTIVE CONDITION: B. PASSIVE CONDITION: SLOPING GROUND SURFACE: C. ACTIVE CONDITION: D. PASSIVE CONDITION: E. DOWNHILL MOVEMENT OF SOIL WHERE THE SLOPE INCLINATION EXCEEDS THE FRICTION ANGLE (from TERZAGHI, 1943)

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the ground as a result of subsidence, a phenomenon that also took place in association with Case History 14 (Appendix A). Figure 5.13 in the next section of the report shows a buckled pavement in Beaver County, Pennsylvania.

2. Preferential Shearing. Because of local weak zones in the soil and rock, or other factors, soils may shear preferentially along one or more surfaces (Figure 4.5) rather than uniformly in the manner shown in Figure 4.4. The potential effects of preferential shearing are threefold:

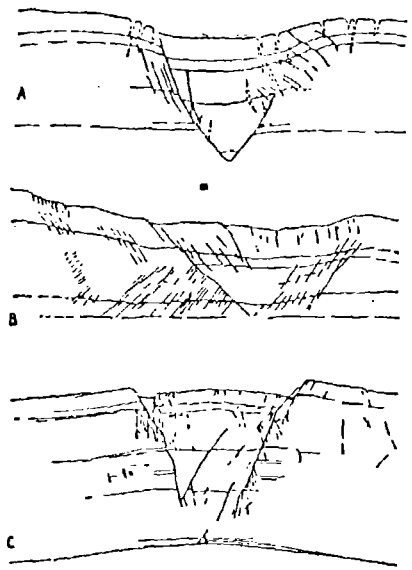
   a. Shearing may result in creep or slippage of hillside soils and a consequent distortion of the ground strain distribution; and

   b. The presence of slippage surfaces at depth may control the positions at ground surface of zones of pronounced differential settlement.

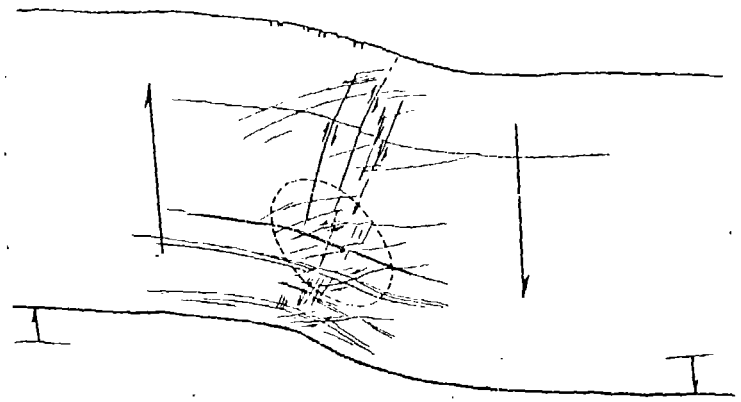
   c. Shearing associated with passage of the travelling wave may result in a net loosening of the soil and a change in soil properties.

Kapp (1973) has recognized that where the natural ground surface rises in the direction of mining, the relative displacement between two points at ground surface will be greater than if the natural ground surface were horizontal. Conversely, the relative movement between these two points will be less if the natural ground surface falls in the direction of mining (Figure 4.6). These deductions are based solely on geometrical considerations. Where natural ground slopes are steep enough to be destabilized by mining, the topographic effect becomes more dependent on the properties of the slope materials--that is, subsidence may increase the inclination of a hillslope to the extent that the component of soil weight directed downhill exceeds the shear resistance of the soil, causing the topmost several feet of soil to become unstable and move downhill. The width of the affected area could be several hundreds of feet.

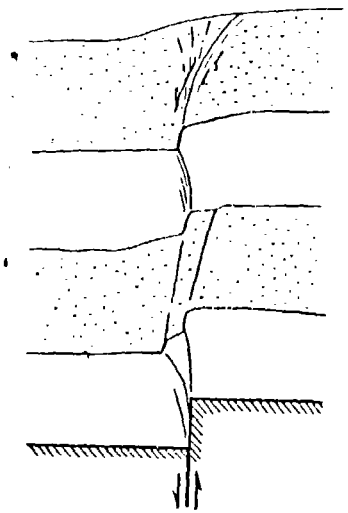
Hillslope instability resulting from mining is a well-recognized phenomenon in the Appalachian Coal Fields. In the early 1920's near Springdale (Allegheny County), Pennsylvania, while conducting a study that is acknowledged to have yielded the first precise record of subsidence over systematic pillar retreat work in the United States, Rayburn (1930) reported the occurrence of extensive landsliding in association with ground breaking during pillar extraction. One of the larger slides measured over 400 feet in an uphill direction and took place on a slope inclined at only 10 degrees. Besides



A.



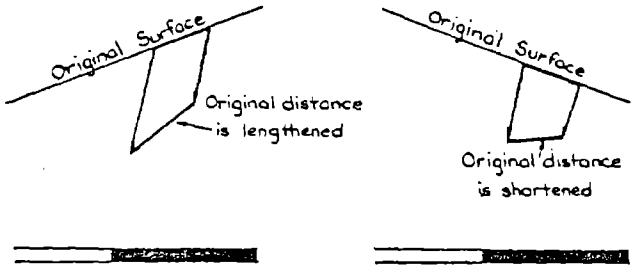
B.



C.

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FIGURE 4.5 - VARIOUS MANIFESTATIONS OF PREFERENTIAL SHEARING IN SOIL
A. GRABEN-LIKE STEP STRUCTURES IN TENSILE ZONE (after H. Cloos, 1930)
B. HIGH ANGLE REVERSE SHEARS IN ZONE OF MONOCLINAL FLEXURE (after E. Cloos, 1955). C. NORMAL SHEARS IN ZONE OF MONOCLINAL FLEXURE OVERLYING ROCK STRATA DISPLACED AT DEPTH (after Wunderlich, 1957) (A and B are laboratory clay models).



(a) Relative movement of surface points

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FIGURE 4.6 – EFFECT OF GROUND SLOPE ON STRAIN IN ABSENCE OF SLOPE INSTABILITY
(after KAPP, 1973)

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foiling attempts to measure horizontal displacements, the landslides in effect increased the subsidence as well as induced ground movements a considerable distance beyond the limits of mining. The topography at the mine site is typical of the Northern Appalachian Coal Field, where slopes are characteristically mantled by ten to twenty feet of residual soil and colluvium.

In the USSR, Turchaninov (1963) has reported subsidence-induced landslides at the Angren lignite deposit that reached "an order of magnitude or more" beyond the originally anticipated limits of subsidence.

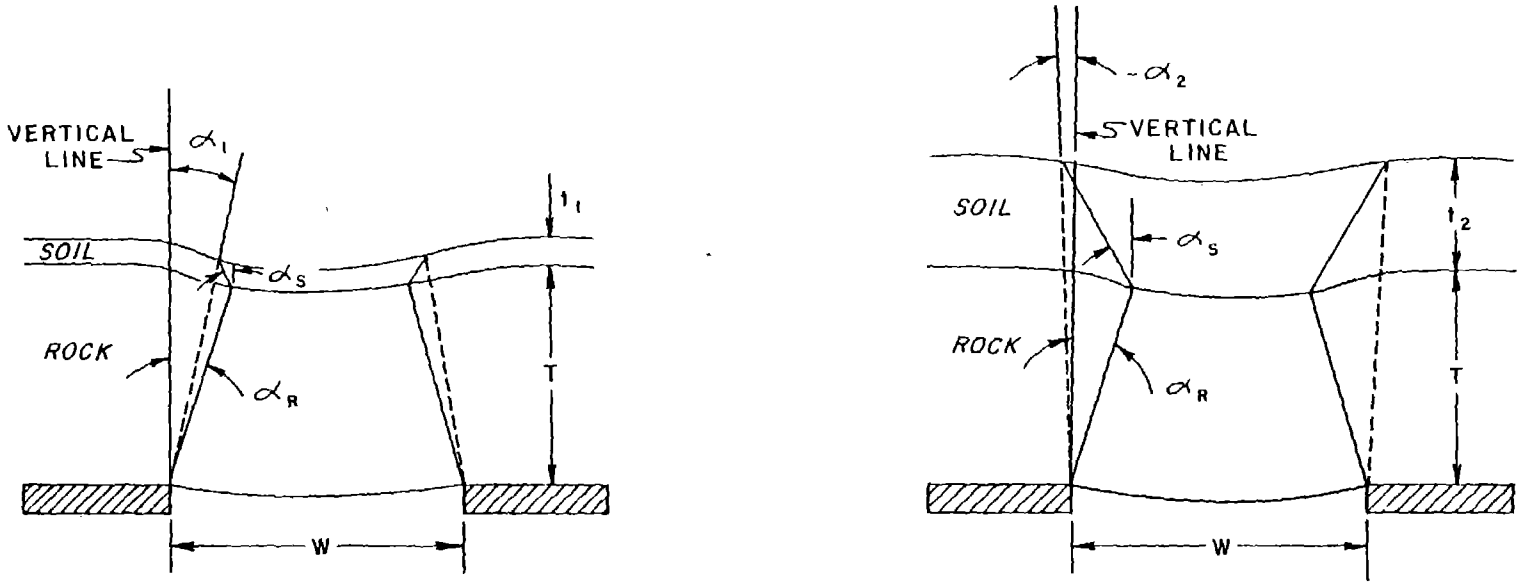
Experience with subsidence in hilly areas has prompted the Pennsylvania Office of Mine Subsidence Regulation to require an extended pillar support area in the downhill direction for homes that are to be permanently supported by pillars and are located on slopes steeper than 5 percent.

Forrester and Whittaker (1976) discuss the problem of subsidence-induced slope instability associated with colliery spoil heaps.

Wardell and Piggott (1969) postulate that subsidence-induced fracturing of ground water aquifers and the creation of hillside springs may have been a significant factor in failure of the spoil pile at Aberfan, Wales.

Preferential shearing in soils along surfaces oriented at $45°-\phi/2$ to the vertical causes "steps" (differential displacements) at ground surface that are offset laterally from the causative displacement within the rock mass (Figure 4.7). The amount of the offset is proportional to the thickness of the soil and the effective friction angle ϕ. The offset phenomenon has been confirmed by model studies (Singh et al., 1976). Steps have sometimes been interpreted in terms of the angle of break which is defined as the inclination to the vertical of a line extending between mine level and the point of maximum tension, where one would expect ground fractures. If this interpretation is accepted, it leads one to conclude that the angle of break can be influenced significantly by the character of the soils at ground surface. A negative angle of break might be found, for example, if the soil cover is thin, whereas a positive angle might be found if the soil cover is thick (Figure 4.7).

Turchaninov (1979) reports the occurrence of steps and ground cracks above steeply dipping coal beds in the Donets Coalfield of the USSR where mines are 3000 feet below the ground surface. He further reports that in regions of synclinal folding, mining-induced ground



W, T, α_R and α_S are the same for cases A and B.
However, $t_2 > t_1$.
$\alpha_R = 45 - \phi/2$

FIGURE 4.7 - INFLUENCE OF SOIL THICKNESS ON THE COMPUTED ANGLE OF FRACTURE A. THIN SOIL ZONE - POSITIVE ANGLE OF FRACTURE: B. THICK SOIL ZONE - NEGATIVE ANGLE OF FRACTURE

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movements frequently concentrate along a 250-foot wide zone that follows the intersection of the axial plane of the fold with ground surface. This is reportedly most prevalent where the lateral distance between the outcrop of the mined seams and the axial plane of the fold is no more than 1.5 to 10 times the length of the subsidence trough. The height of the resulting step at ground surface averages about 0.2 times the maximum subsidence. Figure 4.8 compares observed ground movements with those calculated both by refined methods and by conventional methods that assume planar strata. Ground movements above a synclinal basin in the anthracite fields of eastern Pennsylvania, described by Foster (1912), may be compatible with Turchaninov's observations in the USSR.

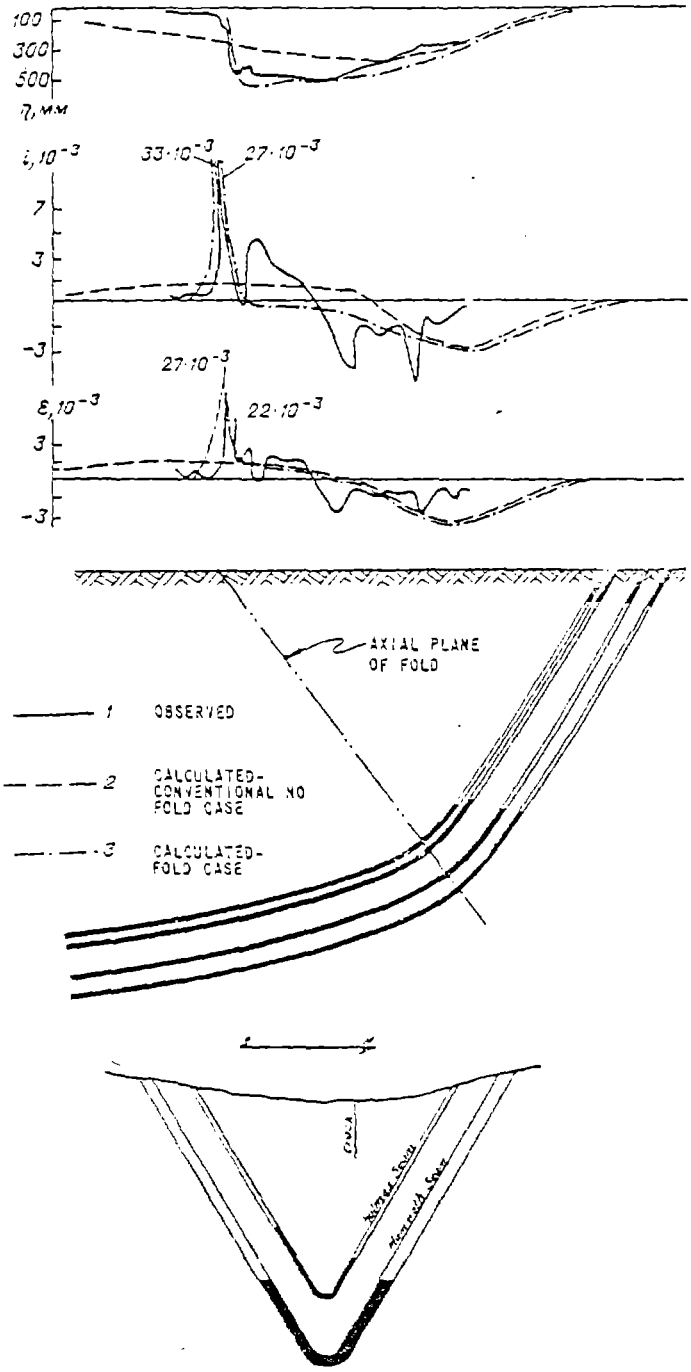
Though not often discussed in the context of subsidence in the United States, faults are quite prevalent in the coal fields of the United Kingdom and Europe. These, too, produce steps and distortions in the subsidence profile.

Lee (1966) reports that the reactivation of an existing fault by underground mining in the United Kingdom may produce a step at ground surface whose height is as much as 30 percent the expected maximum subsidence. The effect of fault-related differential settlements on a home is illustrated by a photograph taken by the writers in West Yorkshire, England (Figure 4.9).

Faults in the United Kingdom typically trend northwest-southeast, with a secondary set oriented perpendicular, in this way bounding areas in which individual mines operate. Normal faults are common in England. Thrust faults occur primarily in Scotland and Wales.

Shadbolt (personal communication) indicates that pre-Permian faults in the United Kingdom that are overlain by more recent strata generally have not influenced the subsidence profile, whereas post-Carboniferous faults that extend to ground surface have produced ground steps in about 80 percent of the cases where the mine workings are under the hade of the fault and in about 5 percent of the cases where mine workings are above the hade.

According to Wardell (personal communication), the traces of major faults can be estimated to within 100 yards by mapping at mine level and then located more precisely by trenching and boring at ground surface. Where more than one seam is to be mined, level surveys conducted during the mining of one seam can delineate fault locations along which preferential movements will probably take place during mining of overlying or underlying seams.



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FIGURE 4.8 - MINING-INDUCED GROUND MOVEMENTS IN PROXIMITY TO AXIAL PLANE OF SYNCLINAL TROUGH
(A-D after TURCHANINOV, 1979) (E after FOSTER, 1912)

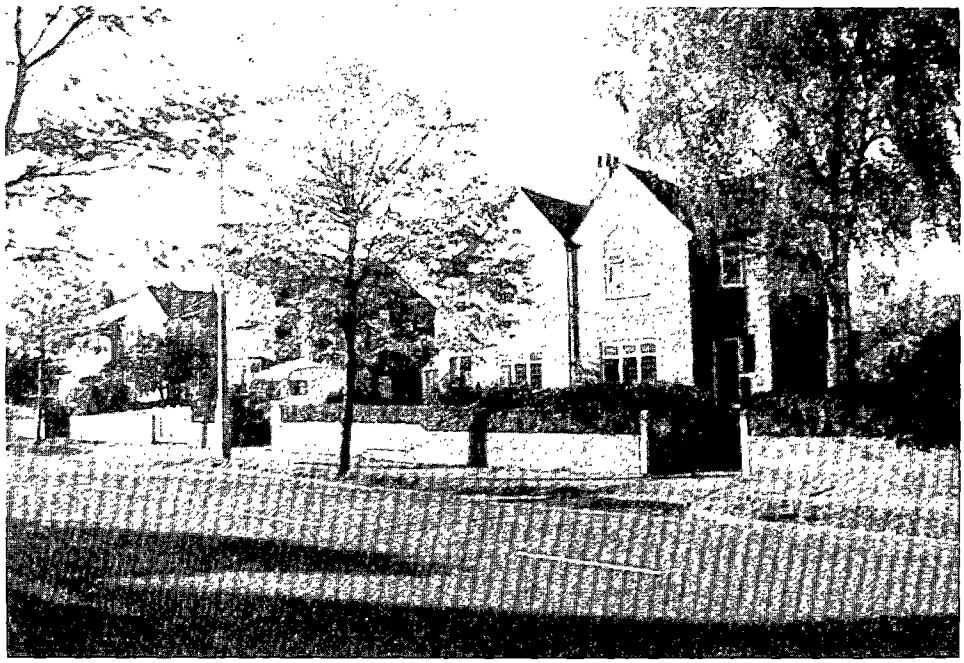


FIGURE 4.9 – OFFSET IN GROUND SURFACE RESULTING FROM PREFERENTIAL SUBSIDENCE-RELATED DISPLACEMENT ALONG FAULT (WAKEFIELD, ENGLAND)

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Wardell finds that faults locally distort the subsidence profile by accentuating vertical displacements and locally modifying ground strains, sometimes by a factor of 3 or 4. In the Netherlands, the effects of faults on ground strain have been recognized where rock is overlain by 700 feet of running sand.

Bräuner (personal communication) observes in the West German coal fields that a pit often develops at the juncture of the fault with ground surface, in effect creating a trough within a trough.

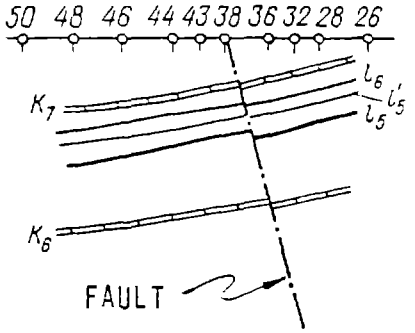
Turchaninov (1979) finds that ground surface displacements associated with a fault are much the same as those where the axial plane of a synclinal fold reaches ground surface. Ground movement observations in proximity to a fault are summarized in Figure 4.10 and are compared to those that are expected without a fault. Similar trends have been observed by Niemczyk (1965) in Germany and Marr (1969) in the United Kingdom.

Mining-induced fault movement data have not been reported in the United States. Rockaway and Elifrits (1979) discuss possible ramifications of the Rend Lake Fault System on mining in the vicinity of Rend Lake Dam in southern Illinois.

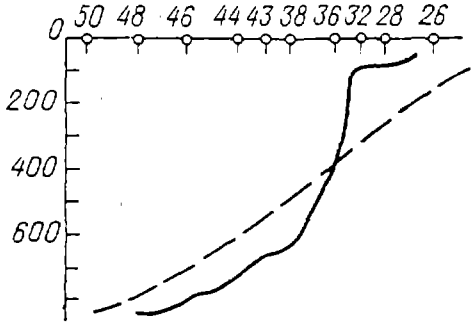
The third basic effect of preferential shearing -- loosening of the soil and modification of the soil properties--appears to be associated with the transient monoclinal flexure of the overburden that accompanies passage of the travelling wave from one end of the mine panel to the other. Although not studied in the context of subsidence, the pattern of shearing near ground surface probably resembles that shown in Figure 4.5 B. The shearing, in combination with the opening and closing of ground cracks (discussed below) imparts a hummocky character to the soil. An example is cited in Case History 15.

One of the very few studies where soil properties have been measured before and after undermining supports the loosening concept. Klezhev et al. (1980) identified a 20 to 50 percent reduction of the soil modulus and a 30 to 40 percent reduction of the unit cohesion in a 50 foot thick sequence of low strength silty clays and sandy clays undermined by longwall in a 4.5 foot coal seam at a depth of 1350 feet.

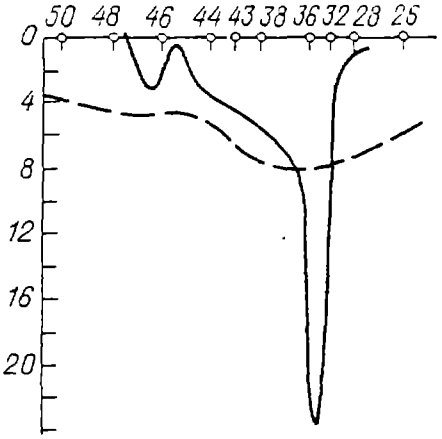
Decreases in soil strength and modulus are almost certainly non-uniform and very possibly foster differential settlements, increased earth pressures and other adverse effects on structures. Which soils might be affected by loosening is not certain. Gradational thin



A.
CROSS-SECTION



B.
SUBSIDENCE mm

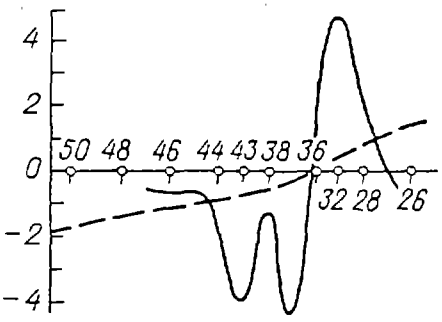


C.
GROUND SLOPE X 10^{-3}

KEY:

—— OBSERVED WITH FAULT

– – – ESTIMATED-NO FAULT



D.
GROUND STRAIN X 10^{-3}

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FIGURE 4.10 – MINING-INDUCED GROUND MOVEMENTS IN PROXIMITY TO A TECTONIC FAULT
(after TURCHANINOV, 1979)

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residual soils with rock fragments, such as found in western Pennsylvania, are possibly susceptible to this phenomenon.

3. Ground Cracks. A series of ground surface cracks and shears often develop along the perimeter of the mine panel as coal extraction progresses (Figure 4.11). Some ground cracks are formed by the fissuring of intact rock, while others simply represent the opening of existing joints in the near surface rock. At some sites no ground cracks can be seen. This has been the writers' experience in monitoring subsidence above a pillar extraction mine at 600-foot depth in northern West Virginia, where soil is less than five feet thick.

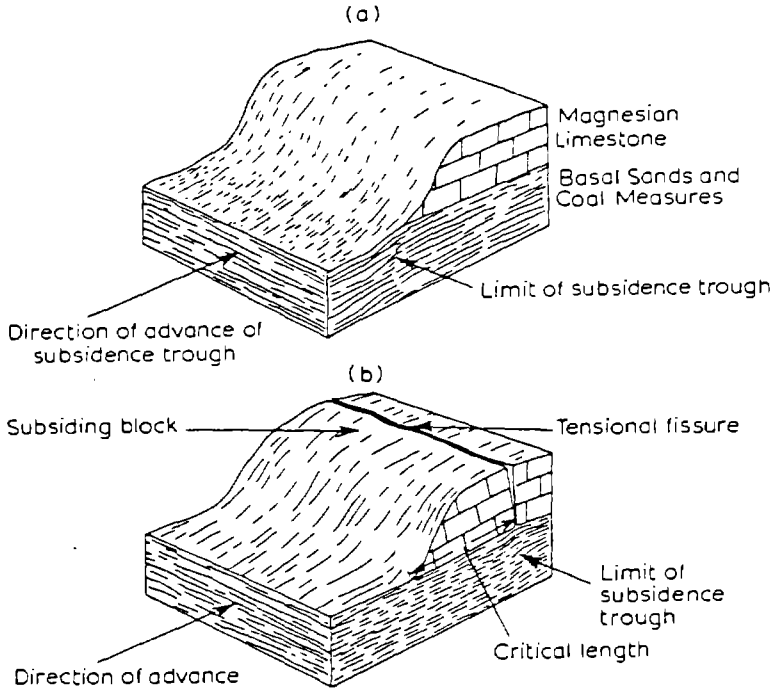
Turner (1967) has found while studying roadcut stability in Durham, England, that there is a critical advance distance of the mine face at which the Magnesian Limestone, cantilevering over the mined-out area, fractures in tension along its upper fiber producing a series of tension gashes parallel to the limestone outcrop (Figure 4.12).

Kapp (1973) finds in the Southern Coalfield of New South Wales, Australia, 50 miles south of Sydney, that steep natural ground slopes (some approaching 45 degrees) and relief of 350 feet intensify tensile strains over the ribs and contribute to the development of significant ground cracking. The cracks trend in the same direction as the ridges and cliffs in the area and conform to the general orientations of the joint sets.

Orchard (1969) has associated ground cracks a few centimeters wide in the United Kingdom with tensile strains on the order of 4 mm/m (0.4 percent). Field observations of earth dams and other embankments under ordinary service conditions indicate that cracks initiate in clay soils where the tensile strain exceeds 0.25 percent (2.5 mm/m). This should also hold true in subsidence applications. Many newly formed ground cracks appear to be about an inch wide. Excavations sometimes reveal the presence of pre-existing fissures that are much larger.

Generally, cracks that occur directly above the mine panel open and then close in conformance with the position of the mine face, many closing so tightly they cannot be located after the mine face has passed. Even so, soil may experience a net loosening during the mining process, as discussed above.

Open cracks visible at ground surface develop where the tensile strength of the soil has been exceeded. The shear strength of the soil theoretically limits open



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FIGURE 4.11 – MINING-INDUCED DEVELOPMENT OF TENSIONAL FISSURE IN MAGNESIAN LIMESTONE RIDGE, COUNTY DURHAM,ENGLAND
(after TURNER,1967)

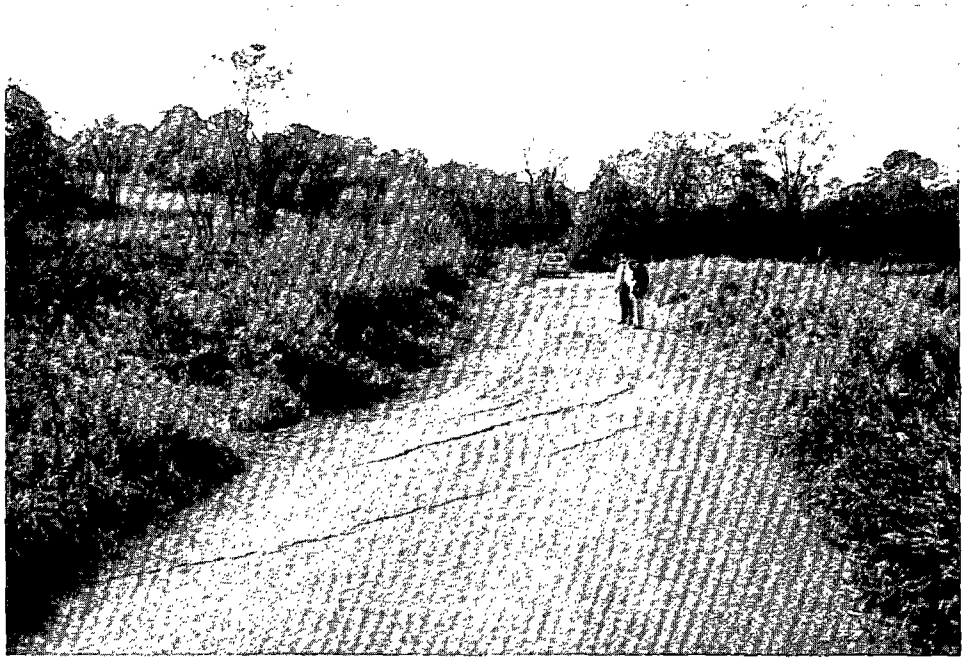


FIGURE 4.12 - GROUND CRACKS IN ROAD ABOVE MINE, ALLEGHENY COUNTY, PENNSYLVANIA

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cracks to a depth Z_o (Figure 4.4A). Soils below the maximum crack depth accommodate elongation (stretching) of the ground surface by assuming the "active" condition described previously in this section.

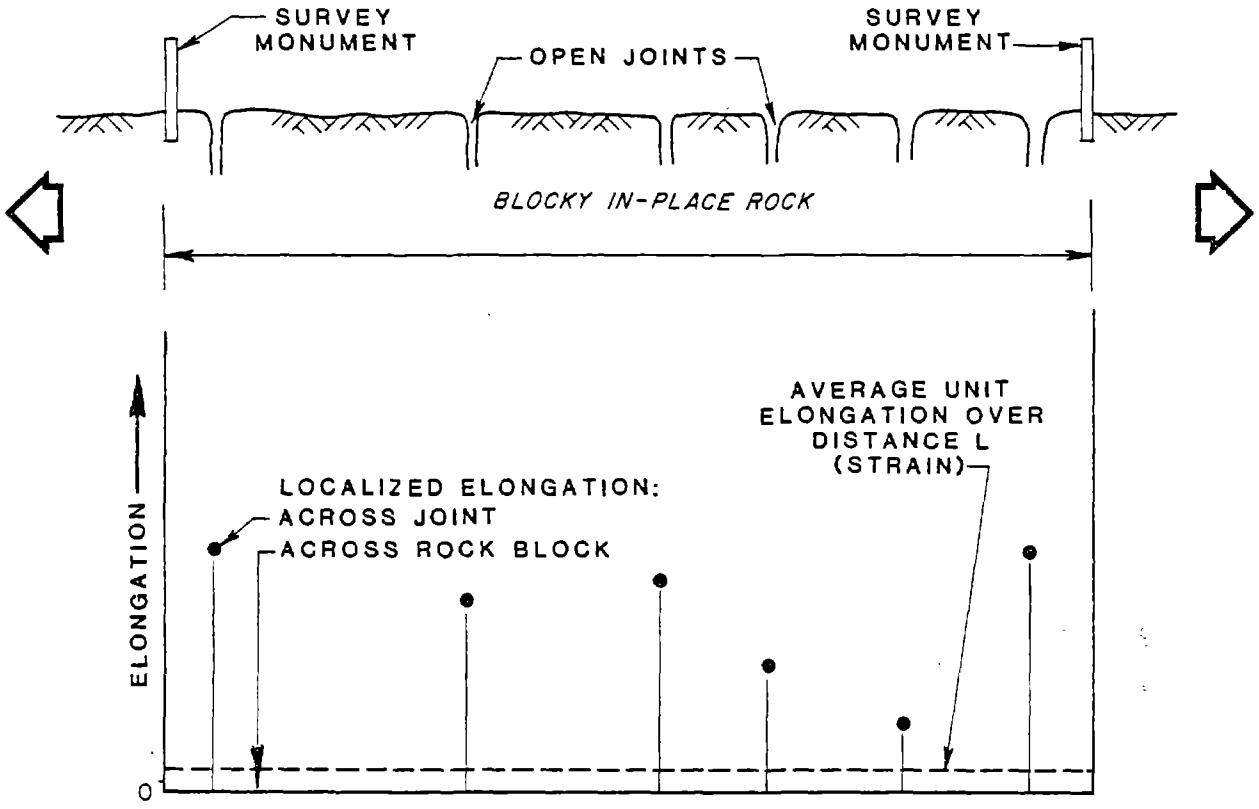
Cracks may develop in sandy, silty and clayey soils, as well as in gravelly soils with a clay binder. The ability of the cracks to stand open depends on surface tension in the soil porewater. The maximum crack depth is reduced by rainfall or surface water.

The effect of cracks is to concentrate ground surface displacements in the manner illustrated by Figure 4.13. The effect of concentrated movements at joints is likely to be most pronounced where the thickness of soil is less than the maximum crack depth Z_o. Here, the displacements of blocks of strata are transferred directly to ground surface. Where rock is below depth Z_o, a zone of shearing soil theoretically develops above the displacing bedrock joints as shown in Figure 4.4A, negating the effect on ground strains of the underlying bedrock joints. The latter phenomenon may be responsible for the "masking" or "cushioning" effect sometimes attributed to the thicker zones of soil and weathered sandstone overlying the blocky, jointed Bunter Sandstone of Great Britain. This rock unit is associated with highly irregular ground strains where the cover of soil and weathered rock is thin.

Whittaker and Breeds (1978) have studied the influence of near surface rock types on mining-related ground movements in the United Kingdom's East Midlands Coalfield. Surface rocks there fall into three main categories: (1) Triassic pebble beds, marls and massive sandstones, including the Bunter Sandstone; (2) Permian limestones; and (3) Carboniferous Upper and Middle Coal Measures that consist mainly of mudstones and sandstones with several coal-seams. Shadbolt (1972) refers to the sandstones and limestones as being brittle, and the siltstones, mudstones and coal seams as being more plastic. Joints markedly influence the response of the sandstones and limestones to subsidence.

The average mining depth in the East Midlands Coalfield is 400 to 500 meters and the usual face width is 200 meters. The panel length averages 800 meters and may reach 3000 meters where geologic conditions are favorable. Faults tend to parallel the main joint systems and the ribsides.

Whittaker and Breeds are of the opinion that subsidence profiles from localities where the Triassic Bunter Sandstone is near ground surface differ from the National Coal Board norms more often than profiles from



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FIGURE 4.13 - CONCEPTUAL EFFECT OF BLOCKY NEAR-SURFACE ROCK ON THE DISTRIBUTION OF GROUND STRAIN

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localities having other rock units near ground surface. This they attribute to the sandstone acting as a line of blocks for which displacements are concentrated at joints. Whittaker and Breeds make the following two observations concerning undermined areas where Bunter Sandstone is found near ground surface:

1. Less subsidence than predicted by the NCB curve, particularly where w/h > 1.0 (Figure 4.14A). (The rock blocks are assumed not to return to their original positions after the subsidence wave has passed).

2. Maximum tensile strain greater than the norm, and positioned farther from the extracted region (Figures 4.14B and C).

Whittaker and Breeds conclude that the character of the rock near ground surface appears not to influence:

1. Amount of subsidence where w/h < 1.0 (Figure 4.14A);

2. Magnitude of maximum compressive strain (Figure 4.14D).

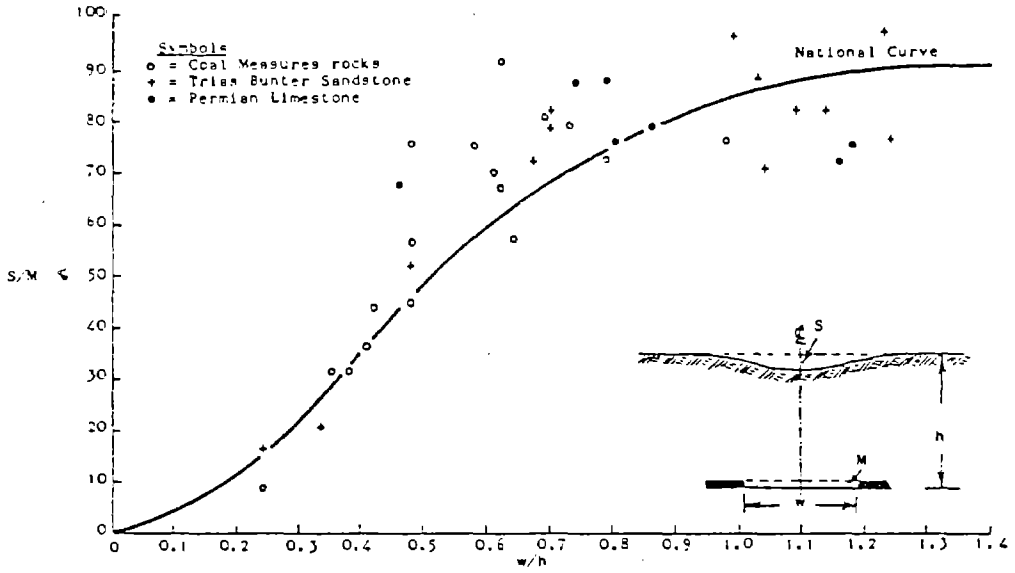
3. Magnitude or position of maximum ground slope (Figures 4.14E and F);

4. Angle of draw.

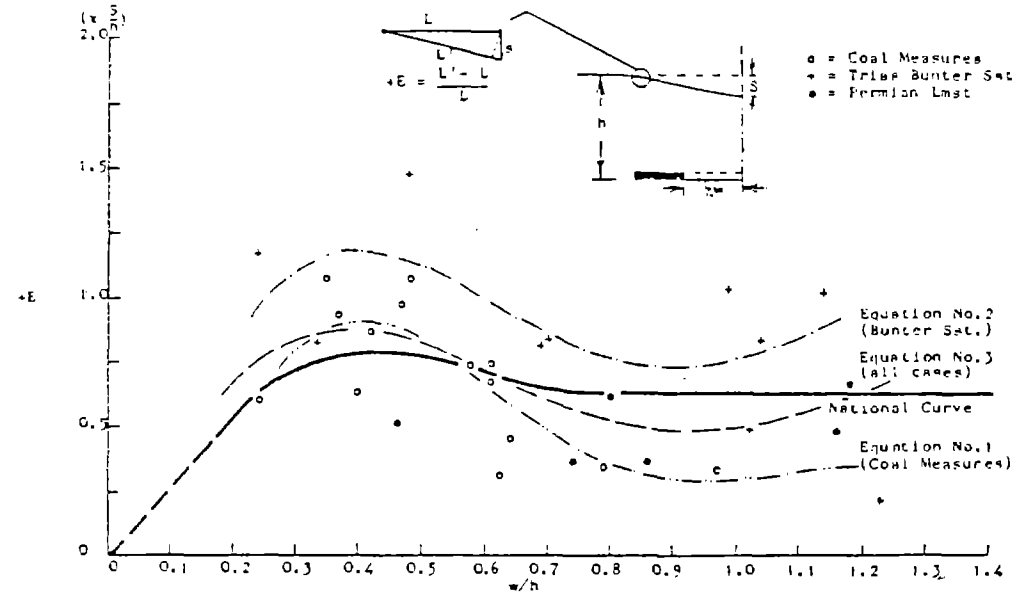
In their summary, Whittaker and Breeds further state that surface subsidence is dependent upon the thickness of surface soils overlying the bedrock. They do not elaborate.

Whittaker and Breeds' study is significant in being directed specifically towards interpreting ground movements in terms of rock type. Although the writers find it difficult to reconcile certain of Whittaker and Breeds' conclusions to the data presented, they fully appreciate how block-wise behavior of the sandstone can alter ground movements, and in Case Histories 11 and 12 (Appendix A) describe two such occurrences in western Pennsylvania--the first, where the sandstone appeared to be associated with an inordinately large angle of draw, and the second, where the sandstone appeared to be associated with a cantilevering action that damaged a series of homes.

The writers consider the most significant aspect of Whittaker and Breeds' data to be the scatter. Most of the data depart measurably from the NCB norms <u>regardless</u> of rock type. While this is not unexpected, it underscores the stochastic nature of subsidence as well as the



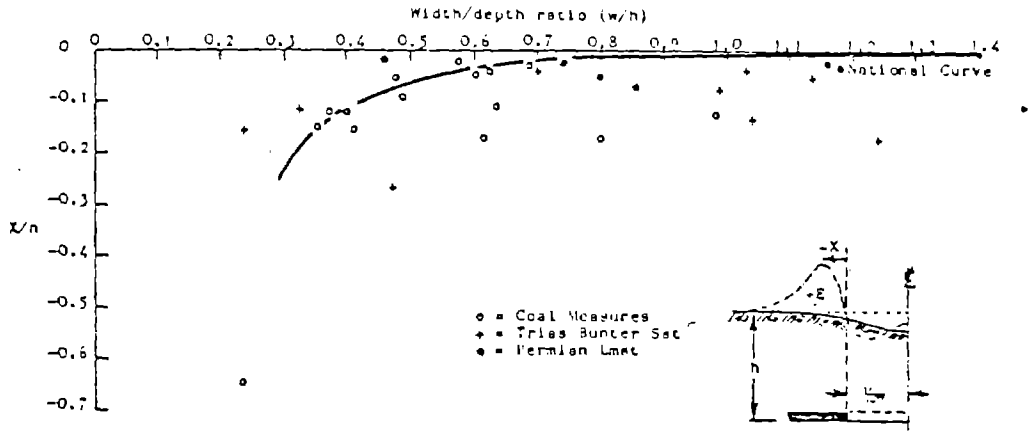
A RELATIONSHIP BETWEEN PARTIAL SUBSIDENCE (S/M) AND EXTRATION WIDTH IN TERMS OF DEPTH (W/h)



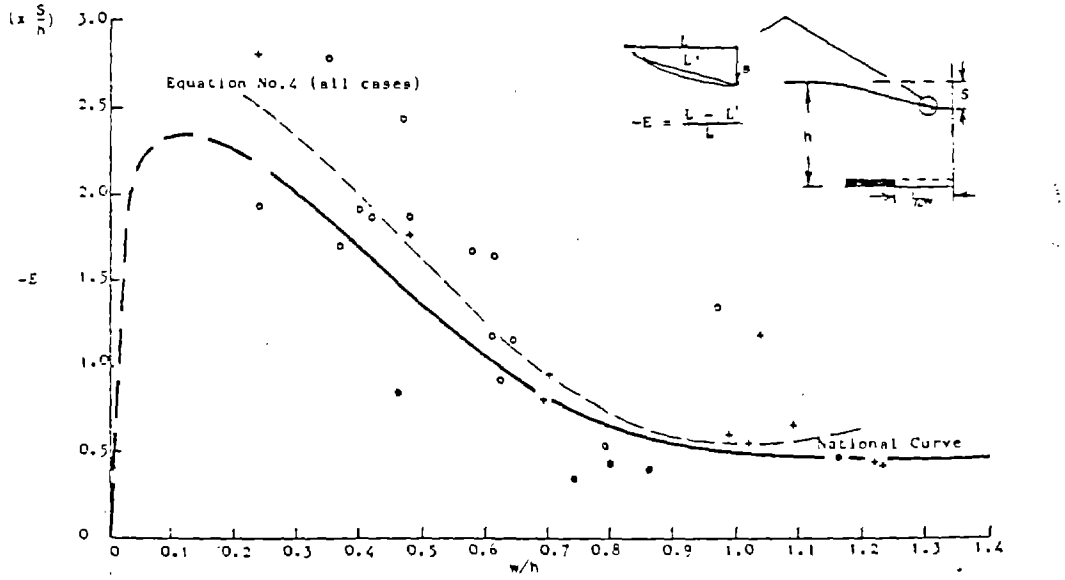
B RELATIONSHIP BETWEEN THE S/h COEFFICIENT FOR MAXIMUM TENSILE STRAIN (E) AND WIDTH/DEPTH

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FIGURE 4.14 – INFLUENCE OF NEAR SURFACE ROCK STRATA ON THE CHARACTER OF SUBSIDENCE IN THE EAST MIDLANDS COALFIELD, UNITED KINGDOM
(Whittaker and Breeds, 1978)



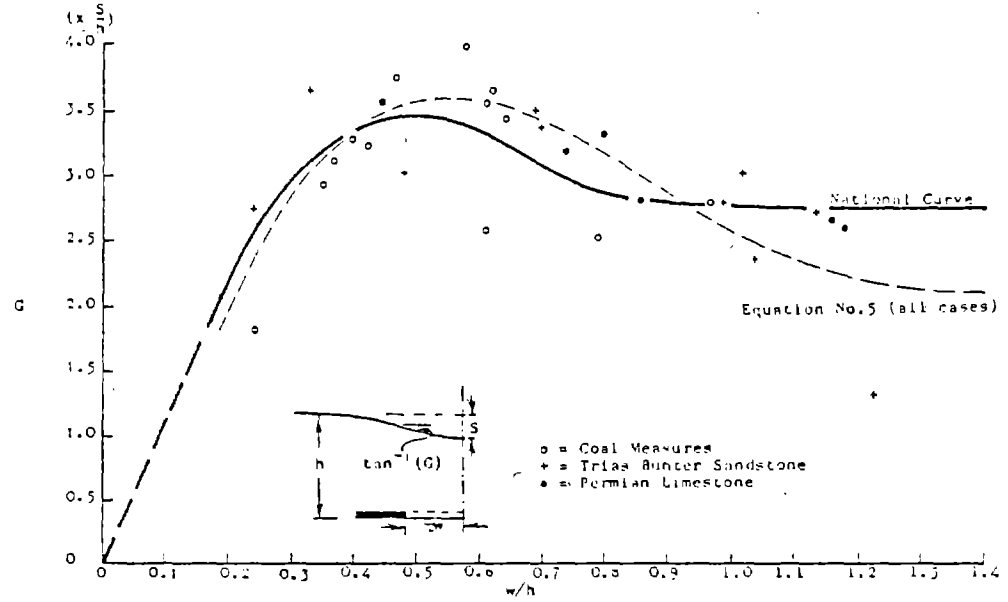
C POSITION OF MAXIMUM TENSILE STRAIN (e) RELATIVE TO RIBSIDE VERSUS WIDTH/DEPTH RATIO (W/h)



D RELATIONSHIP BETWEEN THE S/h COEFFICIENT FOR MAXIMUM COMPRESSIVE STRAIN (-E) AND THE WIDTH/DEPTH RATIO (W/h)

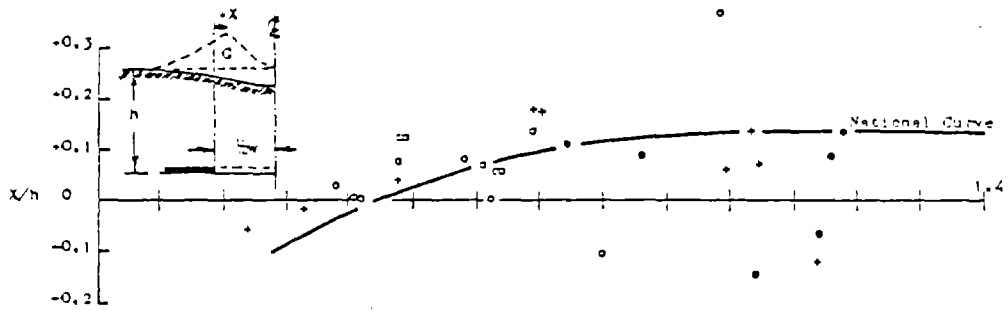
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FIGURE 4.14 - INFLUENCE OF NEAR SURFACE ROCK STRATA ON THE CHARACTER OF SUBSIDENCE IN THE EAST MIDLANDS COALFIELD, UNITED KINGDOM (CONTINUED)
(Whittaker and Breeds, 1978)



E RELATIONSHIP BETWEEN THE S/h COEFFICIENT FOR MAXIMUM SLOPE (G) AND WIDTH/DEPTH RATIO (W/h)

Fig. 3 Position of maximum tensile strain (+E) relative to ribside versus width/depth ratio(w/h)



F POSITION OF HALF MAXIMUM SUBSIDENCE (S) RELATIVE TO RIBSIDE VERSUS WIDTH/DEPTH RATIO (W/h)

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FIGURE 4.14 - INFLUENCE OF NEAR SURFACE ROCK STRATA ON THE CHARACTER OF SUBSIDENCE IN THE EAST MIDLANDS COALFIELD, UNITED KINGDOM (CONTINUED)
(Whittaker and Breeds, 1978)

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necessity for providing a margin of safety in design calculations involving subsidence, just as for any other type of ground movement.

The difficulty in identifying unarguable trends unique to rock type in Whittaker and Breeds' formidable collection of data gives some indication of the problem in determining comparable trends, including soil and near surface rock effects, in the United States, where no norm has yet been established and where reliable subsidence data are only now being collected and synthesized.

Peripheral Phenomena Associated with the Subsidence Process

Apart from the subsidence-related ground movements described above, additional displacements may take place due to phenomena, such as consolidation of the soil or loss of soil into rock fractures.

Consolidation, as has been discussed in Section 2, is a soil volume reduction process associated with the expulsion of pore water in response to a modification in ground loading. In a tensile zone, soils may experience limited consolidation from a reduction in lateral effective stress. In a compressive zone, the same may result from an increase in lateral effective stress. Some amount of reconsolidation of subsidence loosened soil might be expected. Overall, consolidation from ground strains is probably small. Consolidation might also result from water table lowering due to fracturing of an aquifer during mining or to mining through a water-bearing fault zone or other such geologic feature. The process would be conceptually identical to ground surface movements that have resulted from lowering ground water levels during tunnel driving in Sweden and Norway. (Refer to Section 2). Although such settlement could be significant where soil thicknesses are large, formally documented cases are not known. Case History 14 (Appendix A) discusses reported damage to a series of homes in western Pennsylvania that may have been associated with mining-induced dewatering of an alluvial terrace deposit.

Another phenomenon that could lead to ground settlement is loss of ground. Some buildings on shallow foundations settle differentially after the material beneath the footings drops away. In the extreme case, a sinkhole develops. More commonly, the soil simply erodes into joints or subsidence fractures that opened in the underlying rock strata. Neither appears to be common, but both have been observed.

Shadbolt (personal communication) described to the writers the damage suffered by a 900-year old brick church in Treversal, England, that resulted from the gradual erosion of soil from beneath footings into open joints in the underlying rock strata. Five or 6 coal seams had been mined beneath the site over the preceding 150 years at depths in excess of 750 feet. Last mining took place in the mid-1960's. Subsidence damage followed five

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years later. The hill on which the church is located is capped by limestone one to three feet below ground surface, which is transected by occasional soil-filled natural fissures about 18-inches in width. (Shadbolt reports having seen a 28-foot wide natural fissure at another site). Subsidence presumably opened the joints in the rock strata, loosening the soil infilling, allowing it to drop away or increasing its susceptibility to erosion. Over the 5-year period intervening between last mining and subsidence, enough soil winnowed away to develop settlement cracks in several walls of the church. As part of the damage repair, a grouting program was conducted beneath the structure in areas of soil loss.

In the United States, at a trough subsidence site above an abandoned mine in the Pittsburgh area, a split barrel drive sampling investigation conducted by state personnel discovered what appeared to be alluvial material at mine level--possible evidence that soil had eroded through open joints in the rock overburden.

In the same geologic environment and same part of the state, the Commonwealth of Pennsylvania conducted extensive grouting programs at the sites of two schools that were damaged by subsidence over abandoned coal mines located within 100 feet of ground surface. The sites were overlain by 15 to 50 feet of alluvial terrace material that was considered susceptible to erosion into open joints in the underlying rock strata, some of which may have opened as a result of subsidence. Cement grout was injected beneath the school buildings through patterns of closely spaced vertical and inclined drill holes to consolidate the alluvial material. Deeper grouting was conducted to backfill the mine and to close open joints in the rock strata. At one of the sites, described in Case History 9, ground cracks that developed in proximity to, and beneath, the school building--cracks that were discovered when the building was dismantled for repairs--made necessary the removal of existing footings and the regrading and compaction of the subgrade with a vibratory roller to restore the soil to an acceptable density and obviate the possibility of future soil erosion.

The likelihood that one or more of the peripheral phenomena described above will occur at a particular site depends on the near-surface soil properties. The amount of consolidation that might take place, for example, depends on the thickness of soil, its initial void ratio (the ratio of volume of voids to volume of solids), and its compression index (the rate of change of void ratio with change in effective stress, which, in turn, is a function of the elastic modulus and Poisson's ratio of the soil). The rate of consolidation depends on the permeability of the soil and the rate of change of void ratio with change in effective stress (Taylor, 1948). Both granular and clayey soils are subject to consolidation, although the phenomenon is more often associated with clayey soils due to the time-dependent nature of the process resulting from their low permeability. Whereas consolidation depends on the deformability of the soil, loss of ground depends primarily on soil strength and plasticity. The higher the values

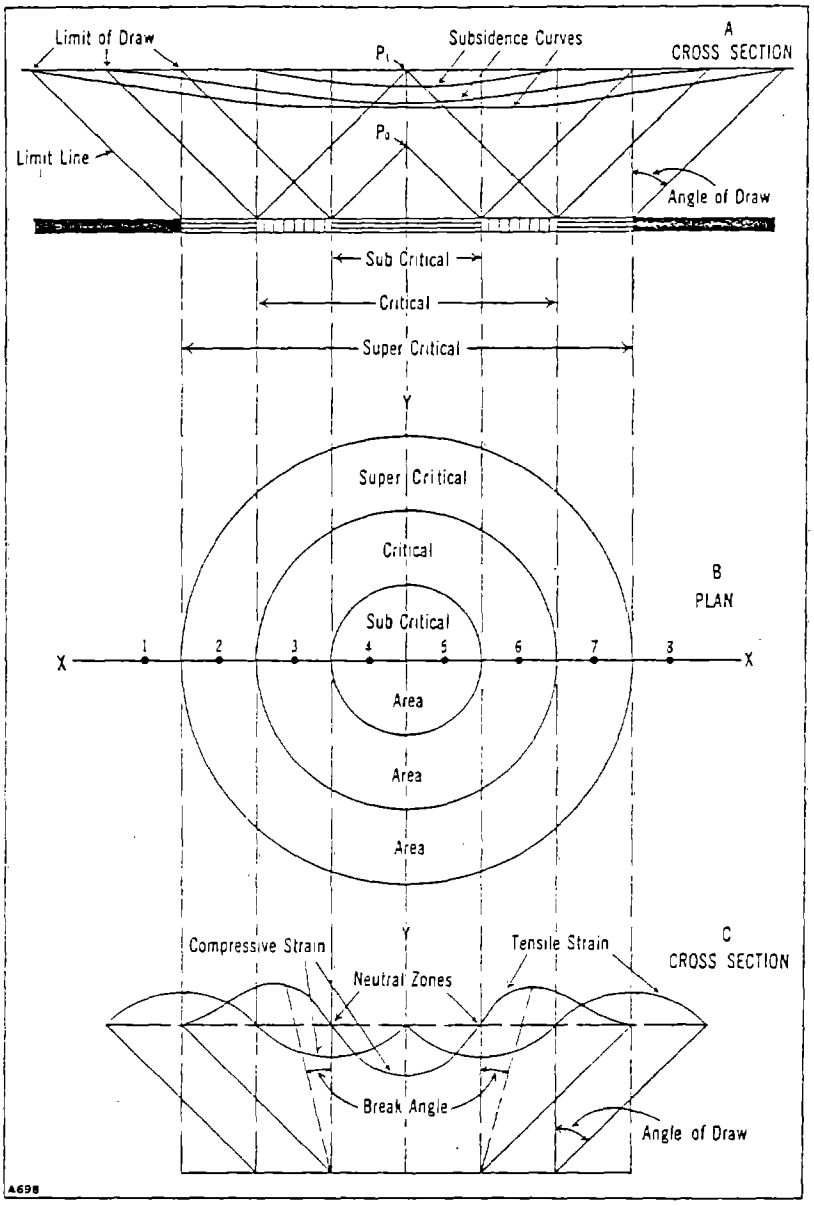
of ϕ, c and plasticity, the less is the tendency for loss of ground. This is attributable to the greater ability of high strength plastic soils, such as stiff clays, to bridge over areas of lost support, whether they be open joints or caved mine openings, compared to lower strength plastic silts or clays, or non plastic silts, sands or gravels. (Nishida, 1972, using a finite element model, discusses the effect of soil properties on the ability of soil to bridge over mine openings at shallow depth.) Estimation of the probable amount of ground movement that might be associated with these peripheral phenomena is difficult, if not impossible, to accomplish in a rational manner.

The effects described above pertain to the relationships between the soil and near surface rock and expected ground movements. The following paragraphs deal with the manner in which the soils might influence the transfer of subsidence-induced ground movements to the undersides of building foundations and to subgrade walls.

Transfer of Horizontal Strain

Horizontal ground strains are typically compressive near the center of a subsiding area and tensile near the perimeter (Figure 4.15). An increase in size of a subsiding area due to pillar extraction in an active mine or to progressive collapse of an abandoned mine redefines the magnitudes and areal extent of the horizontal ground strains and changes the positions of the tensile and compressive zones relative to structures founded at ground surface. Friction and adhesion at the interface between foundation and subsoil result in structural deformations equalling at least a fraction of the horizontal displacement of the subsiding ground (Gardner and Hibbard, 1961). Owing to the eccentricity at which the subgrade shear reaction acts, it is capable of producing significant bending moments in the structure as well as direct tensile forces. Strengths of structural components are generally sufficient to preclude damage when ground strains are compressive, but may be inadequate when ground strains are tensile.

During subsidence, the horizontal displacement of the soil and foundation located in the tensile zone will be compatible until the shear stress along the soil/foundation interface (Figure 4.16) reaches the shear strength of the interface, whereupon the soil will slide along the underside of the foundation, imparting a shear stress commensurate with the prevailing shear resistance along the soil/foundation interface (Geddes and Cooper, 1962, Geddes, 1978). The shear stress at the interface will remain constant as long as the shear resistance along the interface remains constant and ground movements remain tensile. In actuality, ground movements are cyclic, increasing to some maximum at each point on the interface and then decreasing. When, at a point on the interface, the direction of displacement reverses in accordance with passage of the traveling wave, the force-displacement relationship does not retrace the original curve, but rather follows a loop, such as shown in Figure 4.17. The fact that local slippage occurs at



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FIGURE 4.15- MINE SUBSIDENCE RELATED GROUND MOVEMENTS. SUBSIDENCE TERMINOLOGY (from NCB, 1952).



FIGURE 4.16- TRANSFER OF SHEAR STRESS TO UNDERSIDE OF FOUNDATION AS SUBSIDING GROUND STRETCHES

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critical levels of displacement and the force-displacement relationship is not reversible are principal reasons why the terms 'ground strain' and 'structural strain' are not synonymous (Geddes, 1980). Contrary to traditional thinking, a subsidence wave consisting simply of an extensional phase followed by a compressional phase may, at a point in the structure, first produce a tensile force, then a compressive force, and finally leave the structure with a net tensile force (Figure 4.18). The soil structure interaction problem is further complicated by the fact that structures are not perfectly flexible, so the magnitude of the vertical support reaction may vary during passage of the subsidence wave. These complexities notwithstanding, the foregoing considerations lead to the conclusion that, because the properties of the soil beneath a structure influence the shear strength along the interface, they also must influence the magnitude of force transferred to the building.

Should the soil overburden be thin or absent and should the near-surface rock be blocky, ground movements may concentrate along pre-existing joints, creating localized high strain gradients, as discussed earlier in this section. A structure founded on such material might experience damage even when the average horizontal strain over the length of the structure is rather modest. [Shadbolt (personal communication) observes in the United Kingdom that the tensile displacement concentrated at a single joint may exceed an inch, but has not been known to exceed 0.0625 S_{max}.]

Were a structure isolated from the subsoil by a frictionless, adhesionless membrane, the structure would then experience no damage from lateral soil deformation regardless of the intensity or the unevenness of horizontal strain in the subsoil. This is the principle underlying the British practice of placing structures on reinforced concrete mat foundations separated from the subsoil by a polyethylene sheet and bed of compacted sand (NCB, 1975).

Unlike structures in Britain, most structures in the Eastern and Interior Coal Provinces of the United States include basements or crawl spaces and are founded on strip footings. The interface mechanism by which tensile ground movements affect these buildings is much the same as for buildings on shallow mats, although footings or other protrusions extending below the floor slab may couple the structure more effectively to the subsoil and thereby impede slippage between the structure and subsoil. Also, in the United States, the vertical component of ground movement can be as important in causing building deformation as the horizontal component--evidently more so than in the United Kingdom where mining is at much greater depths.

Although compressive strains are ordinarily not of concern to structures founded at ground surface, they can be of concern to structures with basements. The concern arises, at least partly, from increased earth pressures on the subgrade walls. These earth pressures are discussed below.

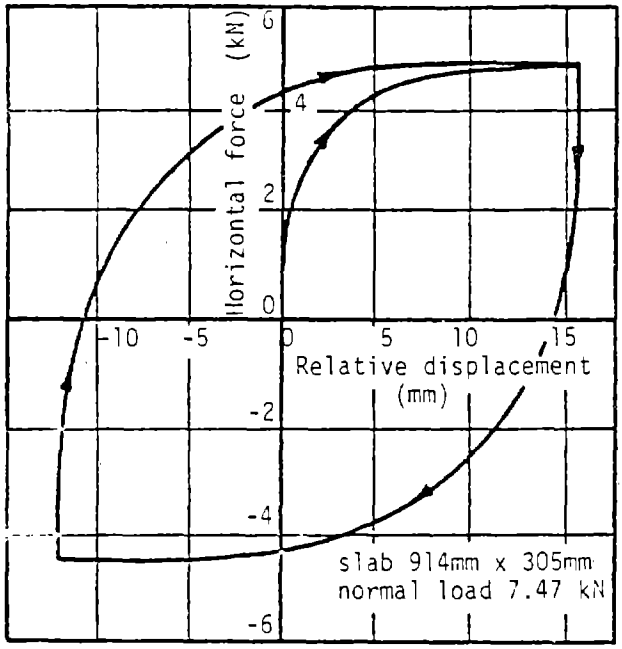
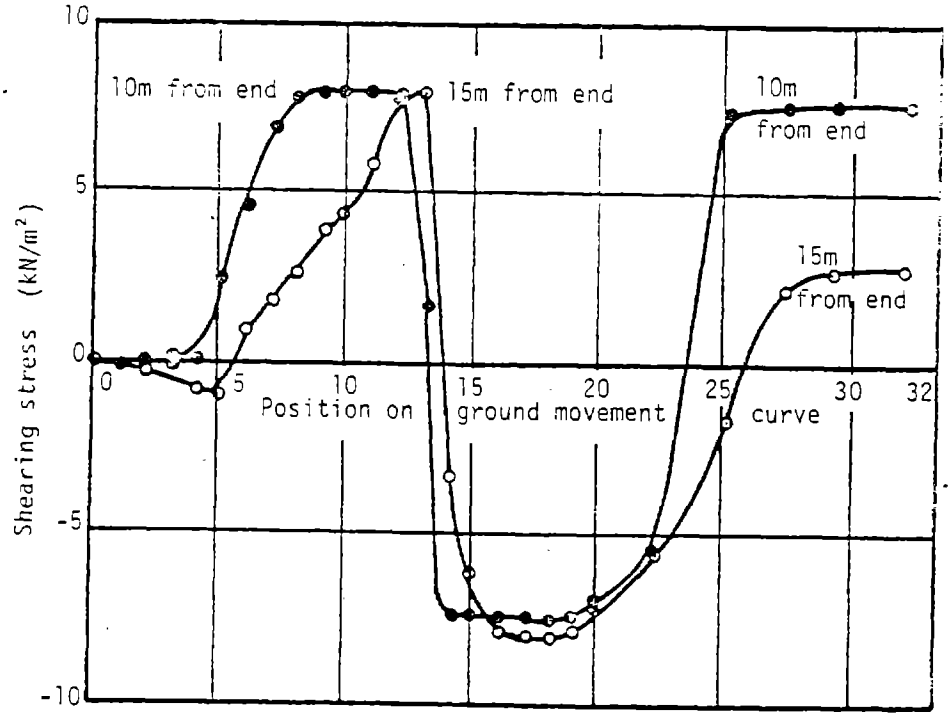


FIGURE 4.17 – CHARACTERISTIC FORCE-DISPLACEMENT RELATIONSHIP AT SOIL/STRUCTURE INTERFACE
(from GEDDES,1980)



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FIGURE 4.18 – STRESS DEVELOPED AT CENTER OF STRUCTURE IN RESPONSE TO PASSAGE OF TENSION AND COMPRESSION PHASES OF SUBSIDENCE WAVE
(from GEDDES,1980)

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Earth Pressures

Walls below ground surface are subjected to pressure from the soil behind. The distribution and intensity of pressure depend upon the type of soil, method of soil placement behind wall, wall height, wall rigidity, wall friction, position of the water table, and other factors (Packshaw, 1946).

Basement walls on the uphill sides of homes in subsiding areas have failed in flexure, as shown in Figure 4.19, and in shear, as shown in Figure 4.20. In these situations, subsidence may have intensified earth pressures by: 1) steepening the slope, thereby directing a greater component of the soil weight toward the wall; 2) weakening the fabric of the soil; or 3) creating new passageways for groundwater to exit the hillside. Both items 2 and 3 promote a reduction in shear strength along the surfaces on which movement takes place. Alternatively, subsidence may have simply weakened the wall, making it susceptible to failure under the existing earth pressure, whether in a tensile or a compressive subsiding zone.

Subgrade walls oppose the lateral soil deformation that accompanies subsidence. In a compressive subsidence zone, the restraining action will result in an increase in lateral earth pressure proportionate to the deformation modulus of the soil--the stiffer the soil, the higher the pressure increase for a particular amount of ground movement.

Theoretically, the earth pressure due to ground compression can increase until the soil ruptures along an arcuate surface extending from the base of the wall to ground surface (Terzaghi, 1943), as shown in Figure 4.19B. The earth pressure prevailing when the soil ruptures is commonly termed the "passive" pressure owing to the relative motion of the wall toward the adjoining soil. Under most circumstances, the wall fails well before the passive pressure is reached. Trenches have sometimes been excavated to footing depth around structures to minimize the lateral transfer of ground strains that could potentially cause severe, if not irreparable, damage to structures (NCB, 1975; Breeds, 1978).

Inasmuch as a structure is invariably subjected to different combinations of compressive and tensile forces that change in direction and magnitude as mining progresses and the subsiding area enlarges (Archibald and Weir, 1969; Figure 4.21), deformation history should be considered when evaluating the relationship between severity of damage to a structure and subsidence. Observed damage to homes above active mines in western Pennsylvania is discussed in the next section.

Conclusions

The foregoing discussion suggests that as long as soils and near surface rocks deform in a uniform manner, ground strains will be proportional to the curvature of the ground surface and independent of material type, properties or thickness--comparable to the

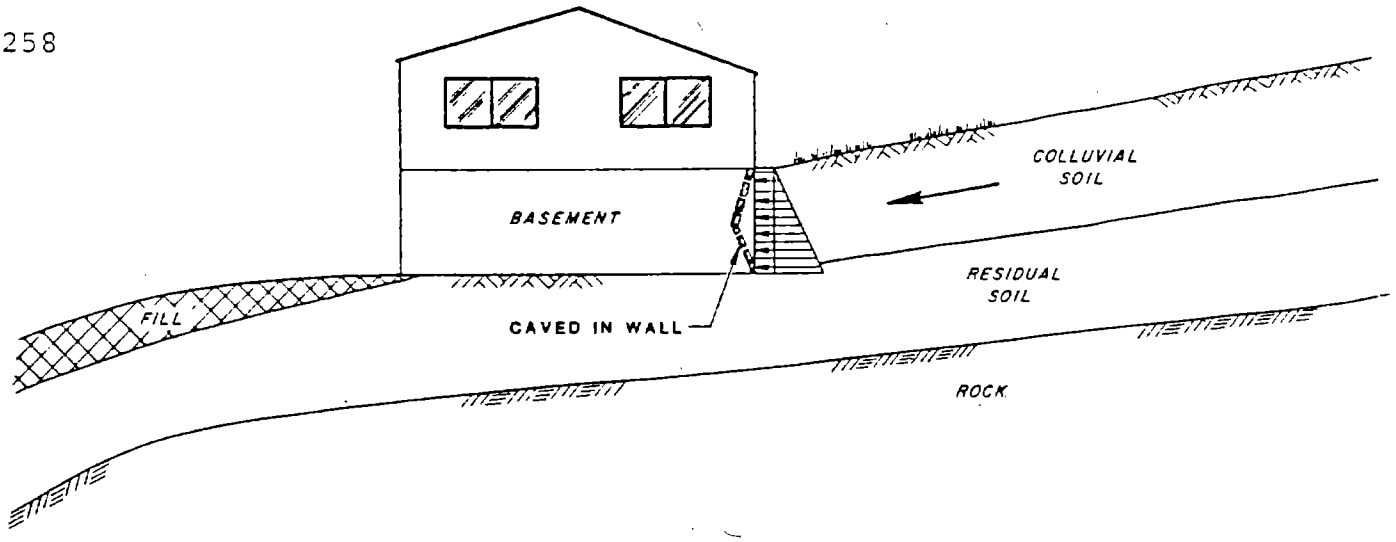


FIGURE 4.19A - EARTH PRESSURE ON UPHILL SUBGRADE WALL OF BUILDING INTENSIFIED BY SUBSIDENCE - INDUCED HILLSLOPE INSTABILITY

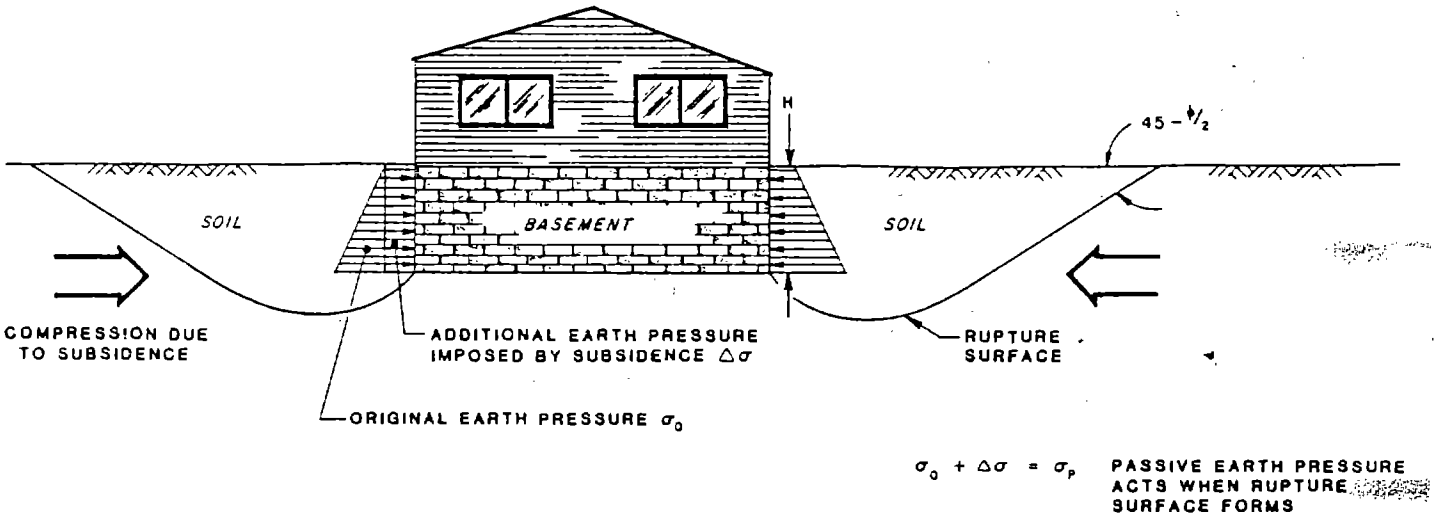


FIGURE 4.19B - EARTH PRESSURE ON SUBGRADE WALLS OF BUILDING LOCATED IN COMPRESSIVE SUBSIDING ZONE

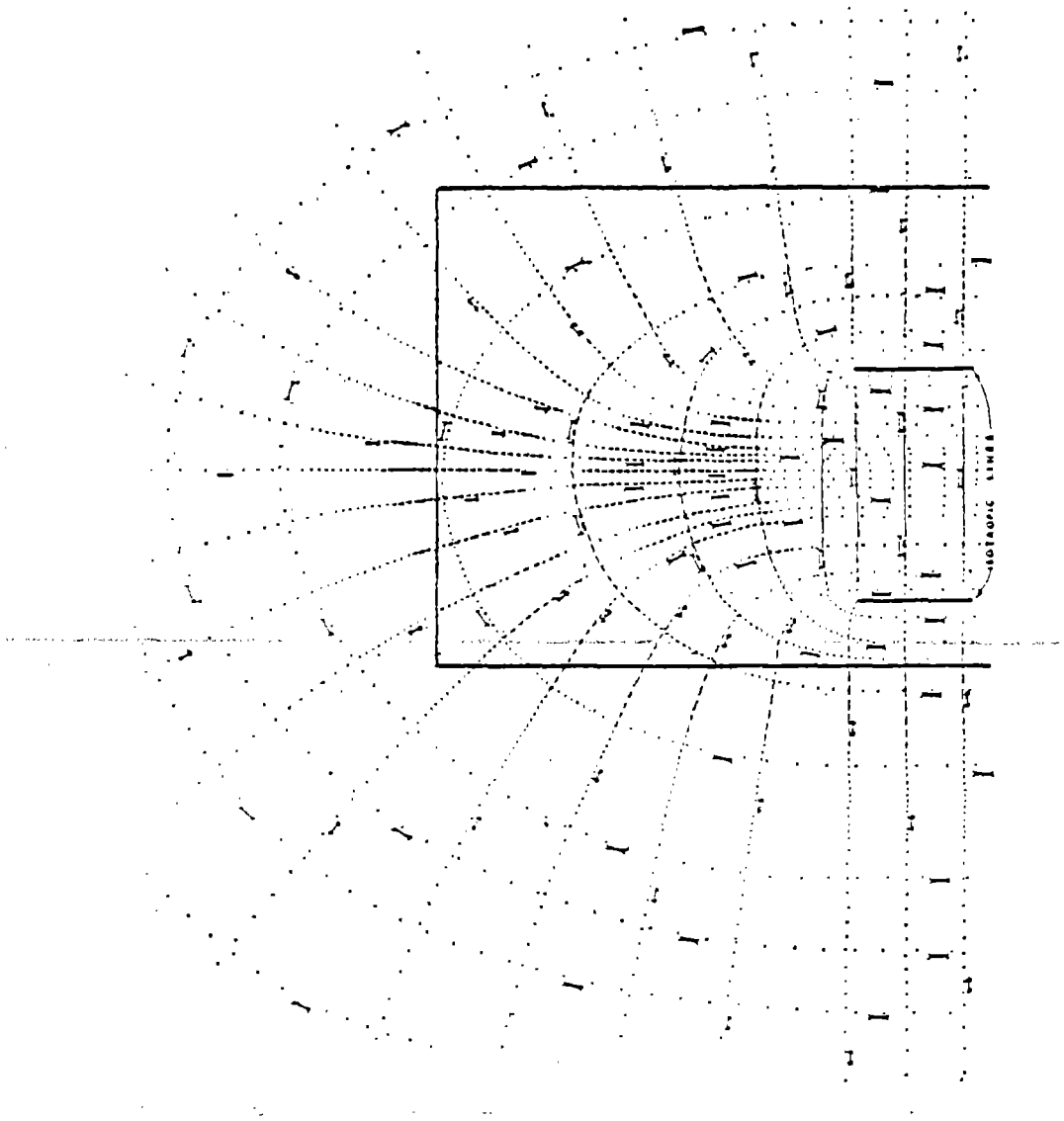


FIGURE 4.20 - BASEMENT WALL ON UPHILL SIDE OF HOME FAILED IN SHEAR AND OUT-OF-PLANE FLEXURE, WASHINGTON COUNTY,PENNSYLVANIA

CARNHILL COLLIERY

SUBSIDENCE INVESTIGATION SITE

MAGNITUDE AND DIRECTION OF PRINCIPAL STRAINS



CODE			
0-05 mm/m	. . . . .	50-10 mm/m	.....
05-10	. . . . .	10-20	----------
10-25	. . . . .	20-30	- - - - - -
25-50	.....	30-40	———

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FIGURE 4.21 - MAGNITUDE AND DIRECTION OF PRINCIPAL STRAINS ABOVE LONGWALL PANEL, AYRSHIRE, SCOTLAND
(ARCHIBALD AND WEIR, 1969)

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way that longitudinal strains on the top surface of a beam are proportional solely to the curvature of the beam. Often there are locations in a subsiding area (most often the zones of highest tension or highest compression) where the soil and near surface rock behave brittley, shear nonuniformly, buckle or crush after the onset of plastic behavior. Where such types of plastic behavior take place, ground strains cannot be estimated by the curvature principle or be determined by any other means than field measurements. To what extent such behavior occurs is a function of the types, properties and thicknesses of the near surface materials.

Because irregular plastic deformations can be recognized in many subsiding areas, and because the extent to which these types of deformations influence the behavior of structures at ground surface has not been documented, there is merit in further study of the problem through field studies and case histories. A mathematical analysis of 50 case histories, discussed in Section 6 of the report, furnishes certain preliminary insights into the effects of near surface materials on the severity of damage to homes.

5. GENERAL DAMAGE EVALUATION AND DAMAGE CRITERIA

Introduction

The objective of this section is to discuss damage to structures from a variety of causes in terms of the types of evidence that are normally available in the form of crack patterns and other observable features. This accomplishes two purposes:

1. Demonstrates the difficulties in evaluating the causes of damage to structures, particularly that visual evidence is not always sufficient to differentiate mine subsidence damage from that due to other causes; and

2. Gives perspective to the term "damage".

Damage Classifications

Because damage to a structure can range from cosmetic flaws that do not hamper the structure's usefulness or performance to complete structural failure or collapse, it is helpful to have some scale or measure for discriminating degree of damage. The simplest scale is based on three degrees of damage:

1. Cosmetic damage, in which only the visual appearance of a structure is affected;

2. Functional damage, in which the usefulness of a structure is impaired or disrupted; and

3. Structural damage, in which the stability or safety of a structure is impaired.

These gradations are highly subjective and depend on the function of the structure, construction details and finishes, building materials, reactions of owners and users to damage, and a host of other factors that do not necessarily relate to performance. Recognizing this shortcoming, numerous researchers have sought more definitive classifications. Burland, Broms and deMello (1977) propose the classification shown in Table 5.1 for visible damage to walls, with particular reference to ease of repair of plaster and brickwork or masonry. This classification is a synthesis of earlier work by Jennings and Kerrich (1962) for heave of buildings on swelling clay and the original United Kingdom National Coal Board Classification for subsidence damage. It is appropriate for application to damage caused by most ground movements.

In consideration of the broad spectrum of possible building damage sources, it is instructive to examine damage classifications based on other causes of damage. The Modified Mercalli Intensity Scale of 1931, devised for rating earthquake intensity (Table 5.2), is based largely on building damage. The middle range of the scale (V to IX), in particular, contains descriptions such as "cracked

Degree of Damage	Description of Typical Damage[+] (Ease of Repair is Underlined)	Approximate Crack Width mm
	Hairline cracks of less than about 0.1 mm are classed as negligible.	
1. Very Slight	Fine cracks which can easily be treated during normal decoration. Perhaps isolated slight fracture in building. Cracks in external brickwork visible on close inspection.	≤1*
2. Slight	Cracks easily filled. Re-decoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some re-pointing may be required externally to ensure weathertightness. Doors and windows may stick slightly.	≤5*
3. Moderate	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked to suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5 to 15* or a number of cracks >3
4. Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15 to 25* but also depends on number of cracks
5. Very Severe	This requires a major repair job involving partial or complete re-building. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	usually >25* but depends on number of cracks

[+]In assessing the degree of damage account must be taken of its location in the building or structure.
*Crack width is only one aspect of damage and should be used on its own as a direct measure of it.

TABLE 5.1 - CLASSIFICATION OF VISIBLE DAMAGE TO WALLS WITH PARTICULAR REFERENCE TO EASE OF REPAIR OF PLASTER AND BRICKWORK OR MASONRY (AFTER BURLAND, BROMS AND DEMELLO, 1977)

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I.	Not felt except by a very few under especially favorable circumstances.
II.	Felt by only a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III.	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing truck. Duration estimated.
IV.	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls made creaking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V.	Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.
VI.	Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII.	Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.
VIII.	Damage slight in specially designed structures; considerable in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed.
IX.	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.
X.	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.
XI.	Few if any (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII.	Damage total. Waves seen on ground surface. Lines of sight and level distorted. Objects thrown upward into the air.

TABLE 5.2 - MODIFIED MERCALLI INTENSITY SCALE
OF 1931 FOR RATING EARTHQUAKE INTENSITY

plaster", "chimneys broken", and "frame structure thrown out of plumb", which are quite similar to the types of damage described in the classifications based on ground movements (Table 5.1).

Similar classifications have been proposed for blasting damage. Duvall, Atchison, and Fogelson (1967) classify major damage from blasting vibrations as "serious cracking and fall of plaster," minor damage as "opening of old cracks and creating new fine cracks." Although the damages caused by the transient movements of earthquakes and blasting occur in an entirely different manner than those due to subsidence or settlement, the damages that can result are sometimes similar. The same could be said for damage caused by snow loads or excessive floor loads.

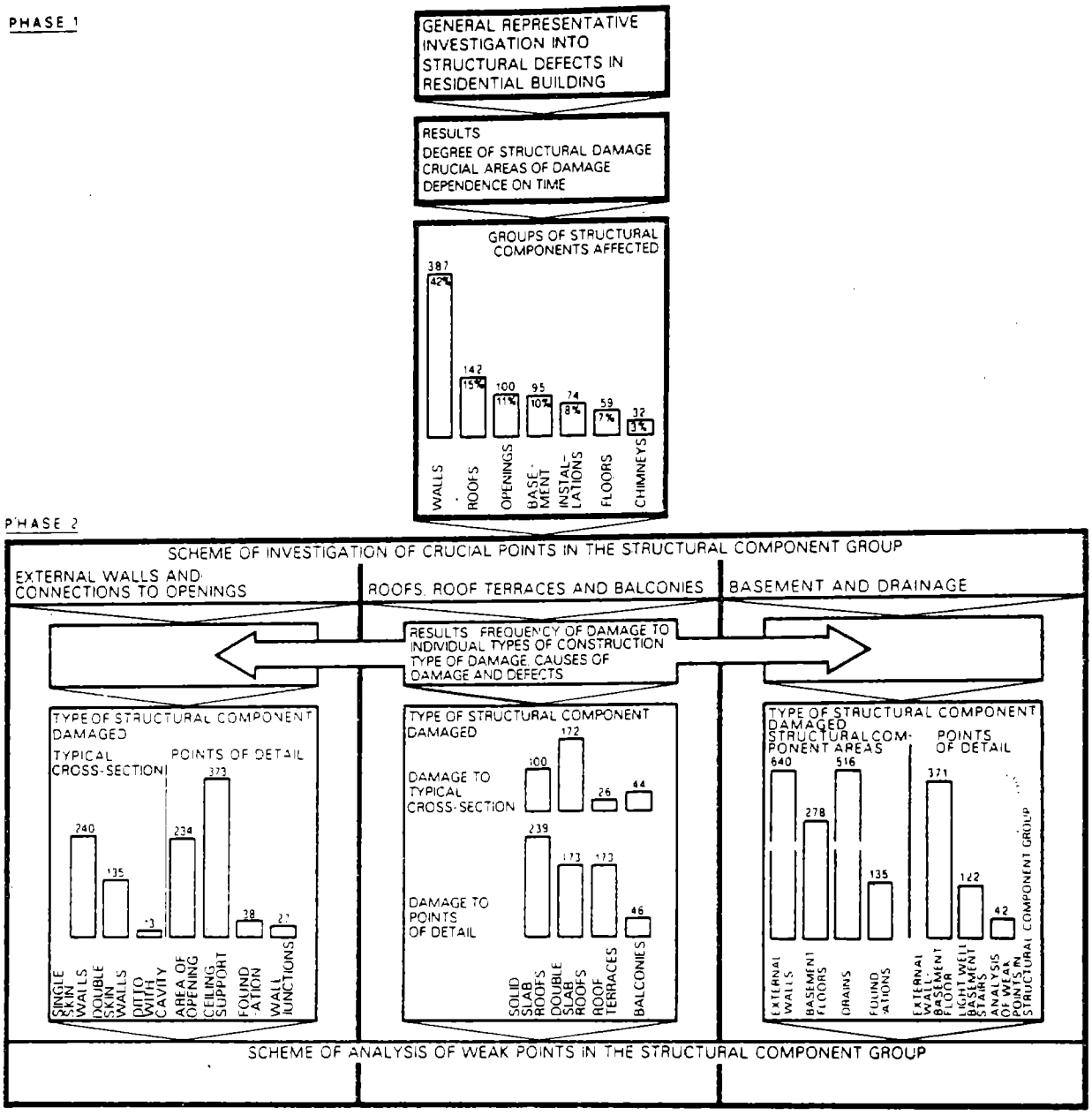
Potential Causes of Damage to Structures

General Considerations. In the most general sense, damage to a structure occurs either as a result of overstressing the structural materials, each of which possesses a particular strength, or weakening these materials while they are under stress. The former is analagous to pulling a rope until its tensile strength is exceeded. The latter is analagous to hanging a weight from the rope and slowly cutting the strands until the rope's capacity is exceeded. Ground movement-induced damage very often, but not always, comes from the second source--structural support is partially or totally removed while the loading remains unchanged. It is important to realize, however, that damage to structures can be caused by other than ground movements.

An enlightening review of common structural problems experienced by residential buildings is presented in a report commissioned by the Ministry of the Interior in the State of North Rhein-Westfalia, German Federal Republic (Schild et al., 1979). The findings of this group of structural specialists, based on investigations of several hundred buildings, revealed that 60 to 75 percent of reported problems to buildings resulted from defects of detail in such components as foundation and wall connections, supports at roof and ceiling, framing around window and door openings, damp-proofing, etc. Their summary of damage frequency to various structural components is presented in Figure 5.1. Notably, ground movement was not the major cause of problems. Instead, most could be attributed to unsatisfactory engineering, poor construction, deterioration, neglect and misuse. These are discussed further in the following paragraphs.

Poor Engineering. A number of serious problems, which may lead to "built-in" overstressing or weakening of a structure can arise during engineering and design.

1. Inadequate design loads--omitting an important design load; underestimating magnitude of design load; improperly placing design load on structure.



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FIGURE 5.1 - SUMMARY OF COMMON STRUCTURAL PROBLEMS EXPERIENCED BY ORDINARY RESIDENTIAL BUILDINGS (after Schild et.al., 1977)

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2. Faulty foundation/structural analysis and design--incorrectly assessing behavior of structure, subsurface conditions, or ground movements, including erring in calculations.

3. Faulty choice of structural materials--selecting materials too soft, too brittle, or incapable of withstanding environmental conditions to which they will be subjected.

4. Communications failures between owner, architect, and designers, including changes in design without evaluation by the designer.

5. Poor specifications and drawings--failure to clearly specify important materials or procedures; improper or inadequate construction practices; improper or inadequate materials testing and construction inspection.

Poor Construction. Poor construction arises from unsatisfactory construction methods and unsatisfactry construction materials. Although many problems arise simply from a failure to follow design or construction drawings and specifications, others arise from practices that lie outside the responsibility of the designer.

1. Poor Construction Methods:

   - o Incorrectly sequencing construction operations, such as completing frame erection without permanent connections or installation of the floor system.
   - o Inadequate shoring, bracing, and other temporary support, or prematurely removing such support.
   - o Omitting or misplacing reinforcing steel in concrete.
   - o Poor workmanship in vibrating concrete, compacting soil, tightening structural connections, etc.
   - o Intentional corner-cutting to reduce costs, including: using incomplete welds; using smaller structural members than required; failing to drive foundation piling to required resistance or depth; using insufficient reinforcing steel in concrete; placing concrete slabs thinner than required; improperly curing concrete; substituting construction materials without authorization, such as using corrugated metal culvert where reinforced concrete is required.
   - o Working under adverse weather conditions--freezing weather, rain, snow, etc.

- o Failing to remove mud, water, or construction refuse from critical areas (concrete forms, bearing areas).

2. Poor Construction Materials:

   - o Worn-out second-hand materials for bracing, shoring, or support.
   - o Low strength concrete, or concrete without required additives.
   - o Inferior grade steel.
   - o Concrete at consistency greater or less than required.
   - o Poor fill or backfill material (swelling, poorly draining or unstable soil or rock).

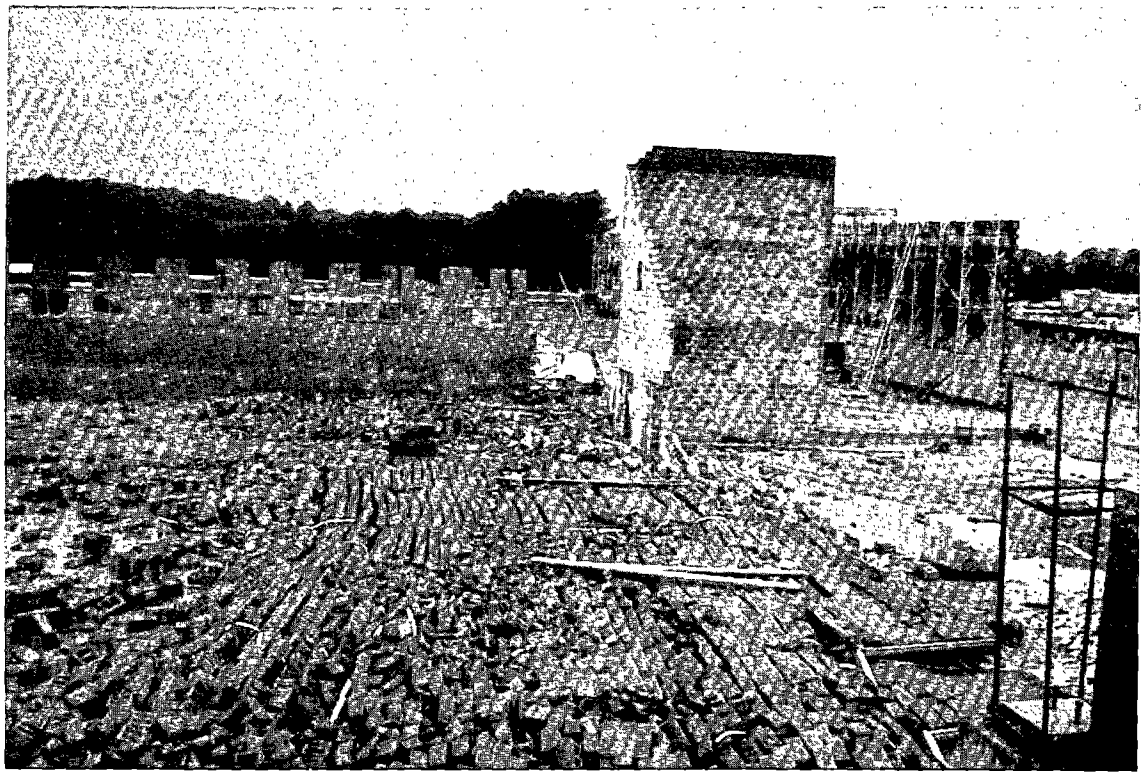
This list represents only a limited sampling of the more well known causes of construction problems. Figures 5.2A and 5.2B show a few examples. Obviously most of these problems could be avoided by close supervision, inspection, and testing. Incompetence, apathy, and dishonesty are unfortunately not unknown among those entrusted with these duties. Good construction is as much a problem of caring as knowing. Achieving the desired result necessitates that the foundation engineer confirm that subsurface conditions exposed during construction are not significantly different than those assumed in design. If conditions are different, it is his responsibility to recommend to the owner those changes in design and/or construction that need to be made.

Normal Deterioration and Neglect of Maintenance. Feld (1968) points out that not many structures collapse from normal deterioration. Damage by deterioration is common nevertheless. Johnson (1965) states that "the principal cause of deterioration of structures is some relatively small carelessness or oversight in the details of design or in the construction." This point is open to argument, as practically all man-made objects deteriorate with time--some are even designed to do so. Many different forms of deterioration lead to serious structural damage. Several are listed below for the four most common building construction materials: steel, concrete, timber, and masonry.

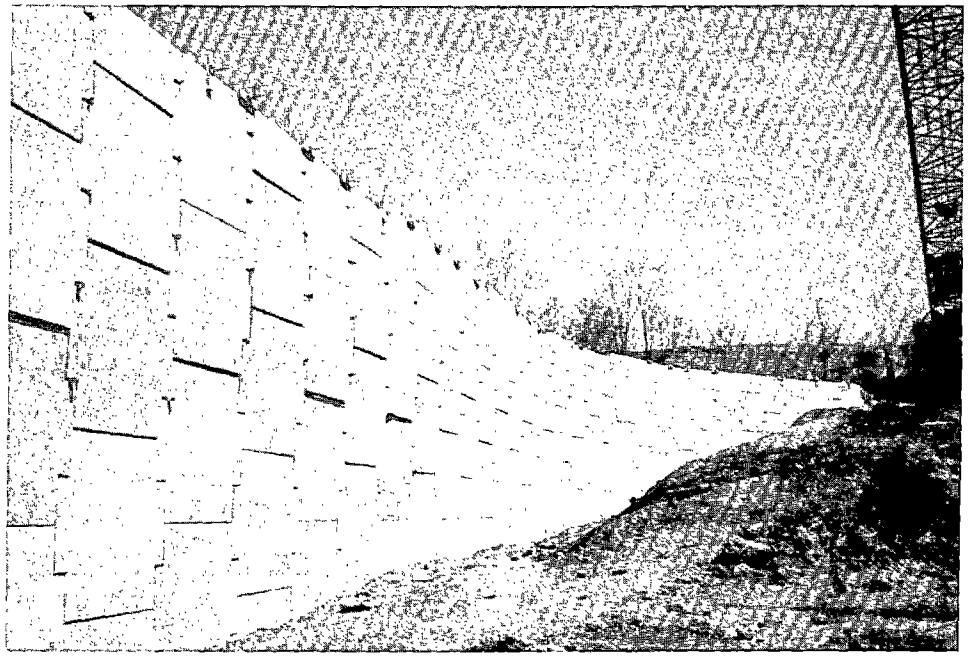
1. Steel

   Basic causes of deterioration, including severe oxidation, cracking and distortion, are:

   - o Corrosion due to water, air, salt, other chemicals--decreases cross-section member; accompanying stress concentrations reduce load-carrying capacity.



A.



B.

FIGURE 5.2 - EXAMPLES OF DAMAGE ASSOCIATED WITH CONSTRUCTION PROBLEMS: A. IMPROPERLY BRACED WALL BLOWN OVER DURING CONSTRUCTION, B. ERRATIC DEFLECTIONS OF REINFORCED EARTH RETAINING WALL RESULTING FROM IMPROPER BACKFILL.

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- Abrasion (erosion)--due to moving parts in contact and to moving fluids (especially those containing solid particles)--creates same problems as corrosion.

- Loosening of connections due to shock or impact loading--induces slip in joints; causes distortion of the structure; creates areas of high stress concentration; increases vulnerability of structure to fatigue failure.

- Fatigue due to repetitive or fluctuating load--causes small fractures; may result in eventual collapse without warning.

- Impact from moving objects--may cause local distortion; can be difficult to differentiate from buckling due to overstress.

2. Concrete

Basic causes of deterioration, including cracking, spalling and disintegration, are:

- Actions incident to construction operations, such as settlement of subgrade, movement of formwork, vibrations, internal settlement of concrete suspension, setting shrinkage, premature removal of shoring.

- Shrinkage with drying

- Temperature stresses

- Absorption of moisture

- Chemical or electrolytic corrosion of reinforcement

- Chemical reactions with cement aggregate

- Weathering

- Shock waves

- Erosion and abrasion

3. Timber

Basic causes of deterioration are:

- Decay or rot

- Insect attack

- Excessive deflection, checking and splitting due to shrinkage and continued seasoning after assembly

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- o Corrosion of hardware (nails, bolts, screws)

4. Masonry (including brick, stone, tile, plaster)

   Basic causes of deterioration, including cracking, spalling and disintegration are the same as for concrete.

It is apparent that some of these symptoms of deterioration are a result of neglect in design or construction, but many are due to ordinary wear and tear, and tend to occur in spite of good design and construction practices.

Misuse or Mistreatment of Structure. Any loading, operation, or modification that subjects a structure to stresses, displacements or environmental conditions beyond those for which it was designed may be regarded as misuse or mistreatment of the structure.

Structural overloading is a well known example. A well designed, well built structure can normally withstand loads significantly greater than those for which it was designed. Feneron and Rodin (1976) have found that individual structural components sometimes exceed code requirements by factors as high as ten. Levels of safety vary erratically from building to building and from component to component. An evaluation of two specific structures is shown in Figure 5.3. Variations in safety levels between components can be attributed to simplified design assumptions, the use of standardized components, the strength contributions of non-structural elements, the influence of non-structural design considerations in planning and architecture, and other factors. Combining the ordinary safety factor concept with the fact that engineers have a tendency to overestimate the actual loadings on a structure (Peck, 1965), it is probable that many structures can withstand loads two to three times those actually experienced during normal operating conditions. Most modern structures in the United States are so conservatively designed that damage by overloading is only a remote possibility. Older structures, and a few modern structures, do not enjoy such a measure of protection. Older structures, in particular, run the greatest risk of overloading for the simple reason that no one knows the loads they were designed to carry.

Overloading in buildings seems most likely to occur as a result of the storage of excessively heavy articles on floors or overoccupancy. The floor in a private residence in Erie, Pennsylvania, collapsed in 1911 when 50 people congregated in the kitchen during a funeral (Feld, 1968). Old warehouses furnish numerous cases of structural overloads. Common liquid overloads include drums of chemicals, ponded rainwater and water tanks. Roof collapse under excessive snow load is relatively common.

An operational overload sometimes occurs in storage bins or silos that are to be filled in a certain sequence. If loading occurs out of sequence, or in a manner not anticipated in design, damage or failure may result, even though the total loads are less than the full service load.

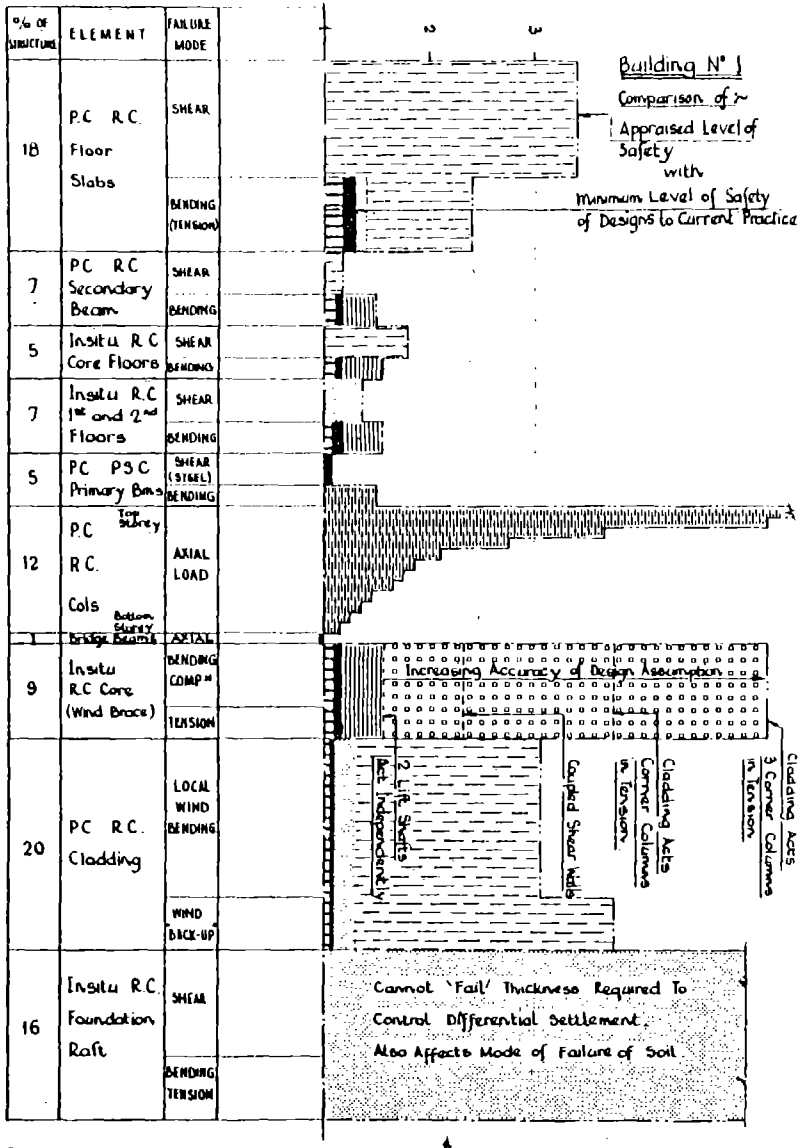


FIGURE 1 : 14 STOREY PRECAST CONCRETE OFFICE BUILDING

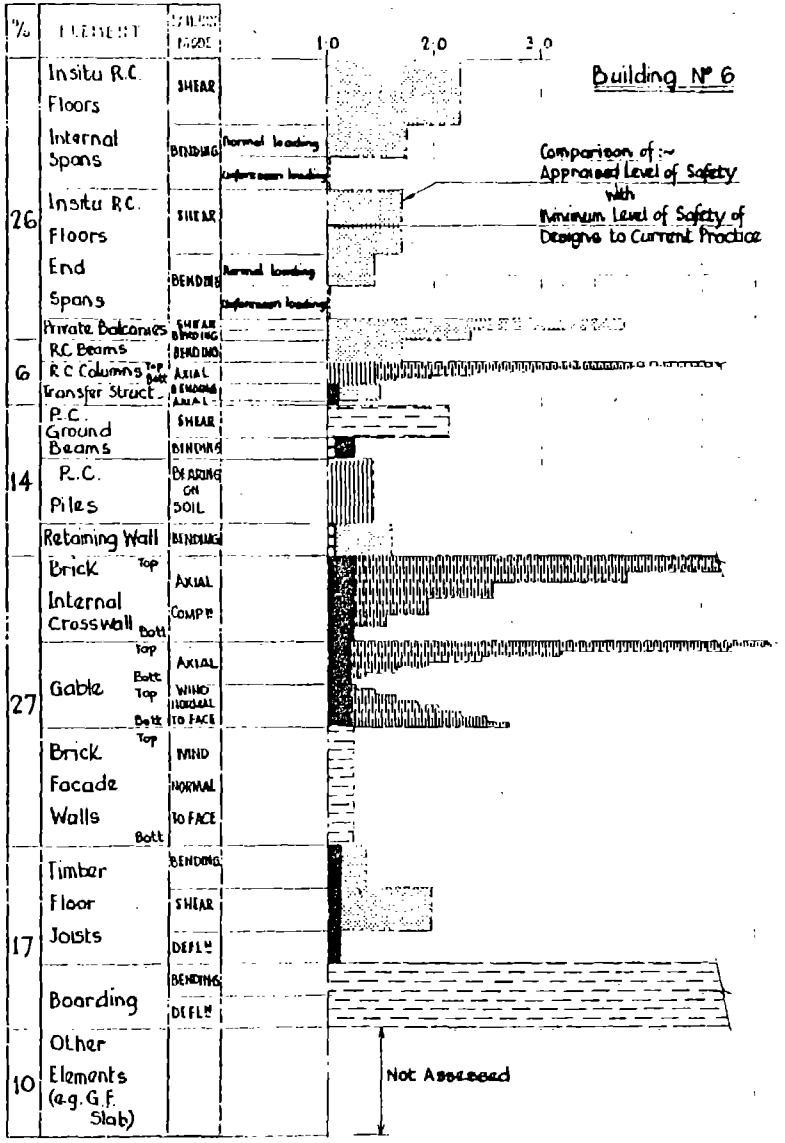


FIGURE 6 : 8 STOREY BRICK CROSSWALL MAISONETTES

FIGURE 5.3 – EVALUATION OF LEVELS OF STRUCTURAL COMPONENTS IN TWO BUILDINGS

(after Feneron and Rodin, 1976)

Level of Safety higher than minimum level proposed using γ_c factors

Level of safety lower than minimum level proposed using γ_c factors

Previous code requirements greater than present (e.g. reinforcement to CP114 greater than required by CP110)

Previous code requirements less than present (e.g. shear resistance lower in designs to CP114 than to CP110)

Rounding (e.g. Allocation of bars to given reinforcement areas)

Logistic Reasons (e.g. maintaining standard size of columns or prefabricated units)

Simple - safe assumption (e.g. neglect of cladding action in resisting wind)

Over-Design

Additional strength provided for non-structural reasons (e.g. Thermal or sound insulation, minimum practical dimensions etc.)

Additional strength required to meet other structural criteria (e.g. stiffness)

Minimum level of safety of designs to current practice

Assessed level of safety in existing building

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FIGURE 5.3 - CONTINUED

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Modifications to existing buildings have occasionally resulted in spectacular building damages. Some of the more common modifications include: added floors or stories; subdivision of a large building spaces into smaller spaces; adding structural loads; removal of walls and other load-carrying members during enlargement or remodeling operations; and cutting doorways or windows in load-carrying members.

Man-Induced and Natural Disasters. Few forms of disaster are routinely assumed in structural design. It is simply too expensive to design for extreme forces that might occur on an unpredictable schedule and contribute loads and operating conditions of unknown and unpredictable magnitude.

Disasters result from fires, explosions, collisions, vibrations, floods, tornadoes and other severe storms, volcanoes, tsunamis, avalanches and earthquakes. Of these, only earthquakes are routinely accounted for in design, and then only in specified regions of the country where they are relatively common.

Concluding Remarks on Potential Causes of Damage to Structures. Building damages can be linked to many causes, some of them subtle. A design engineer's oversight of a potential source of ground movement could lead to serious structural damages in the completed project, as could ground movements associated with poor construction methods and materials. Subsurface erosion of foundation subsoils brought about by a leaking sewer line, for example, could bring about a deterioration and maintenance problem, while overloading could lead to bearing capacity failure. These same causes might ultimately lead to a landslide at some time during the life of a structure.

Numerous case histories of building damage and deterioration caused by one or more of the factors discussed in this section are given in two excellent references by Feld (1968) and Johnson (1965).

Visual Evidence of Damage

General Considerations. One of the prime difficulties in evaluating building damage results from the common absence of visual evidence that points conclusively to the cause. Except for storms and similar phenomena, one must usually determine the cause from the visual evidence of damage--the physical after-effects. This is particularly true for building damage caused by ground movements.

The visual evidence that is normally available includes:

1. Distribution and orientation of debris (from a structural collapse).

2. Repaired building components.

3. Ground displacements.

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4. Building displacements and distortions.

5. Distorted, displaced, or failed structural members and construction materials.

The more common forms of damage evidence resulting from ground movements are discussed in the following paragraphs.

Collapse Debris. The distribution and orientation of debris from a complete structural collapse is frequently so diffuse as to prohibit identifying the cause. An example of total collapse is shown in Figure 5.4. If ground movements were the cause, the nature of the movements would generally become evident by the time collapse had occurred. Burland and Wroth (1974) state that, "Apart from a few notable exceptions, buildings will usually become unserviceable before there is a danger of structural collapse."

Repaired Building Components. Damaged structures that have been expertly repaired may offer the least evidence of earlier ground movement. An example of expert repair to a brick building that had undergone earlier differential settlement is shown in Figure 5.5. In cases such as this, the only visual evidence may be construction materials having a slightly different appearance.

Ground Displacements. Normally, in the absence of precise measurements, there are only two types of evidence that attest to the occurrence of past ground movements: 1) an actual rupture or flexure of the ground, such as might accompany a landslide or bearing capacity failure (Figure 2.2); and 2) the relative displacement of the ground relative to an unattached reference object, such as a tree, sidewalk, wall or building. Accurate pre- and post-damage level survey data may better define the magnitude and extent of movements, but the movement pattern may not necessarily be so unique as to fully identify its source.

Building Displacements and Distortions. Displacement of a building relative to its foundation could have resulted from a cause other than ground movement, such as thermal expansion (Figure 5.6). Purely horizontal displacements of a building and its foundation are not common and are usually a sign of horizontal translational ground movement, such as a landslide or tectonic movement (fault). A purely vertical building displacement could occur by virtually any ground movement source, including those that involve ground distortions. Rotational building displacements (tilt) with significant horizontal and vertical displacement are often indicative of a landslide, bearing capacity failure, or tectonic movement. Rotational building displacements (tilt) without significant horizontal or vertical displacement may result from many sources of ground movement. Only a few special types of building displacement might therefore point to specific sources of ground movement.

Partial distortions of a structure occur as a result of localized overstressing or weakening (Figure 5.7A). A heavy snow load on the roof of a building may distort the roof but not affect



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FIGURE 5.4 - EXAMPLES OF COLLAPSE DEBRIS. FARGO GRAIN ELEVATOR BEARING CAPACITY FAILURE (after Nordlund and Deere, 1970).

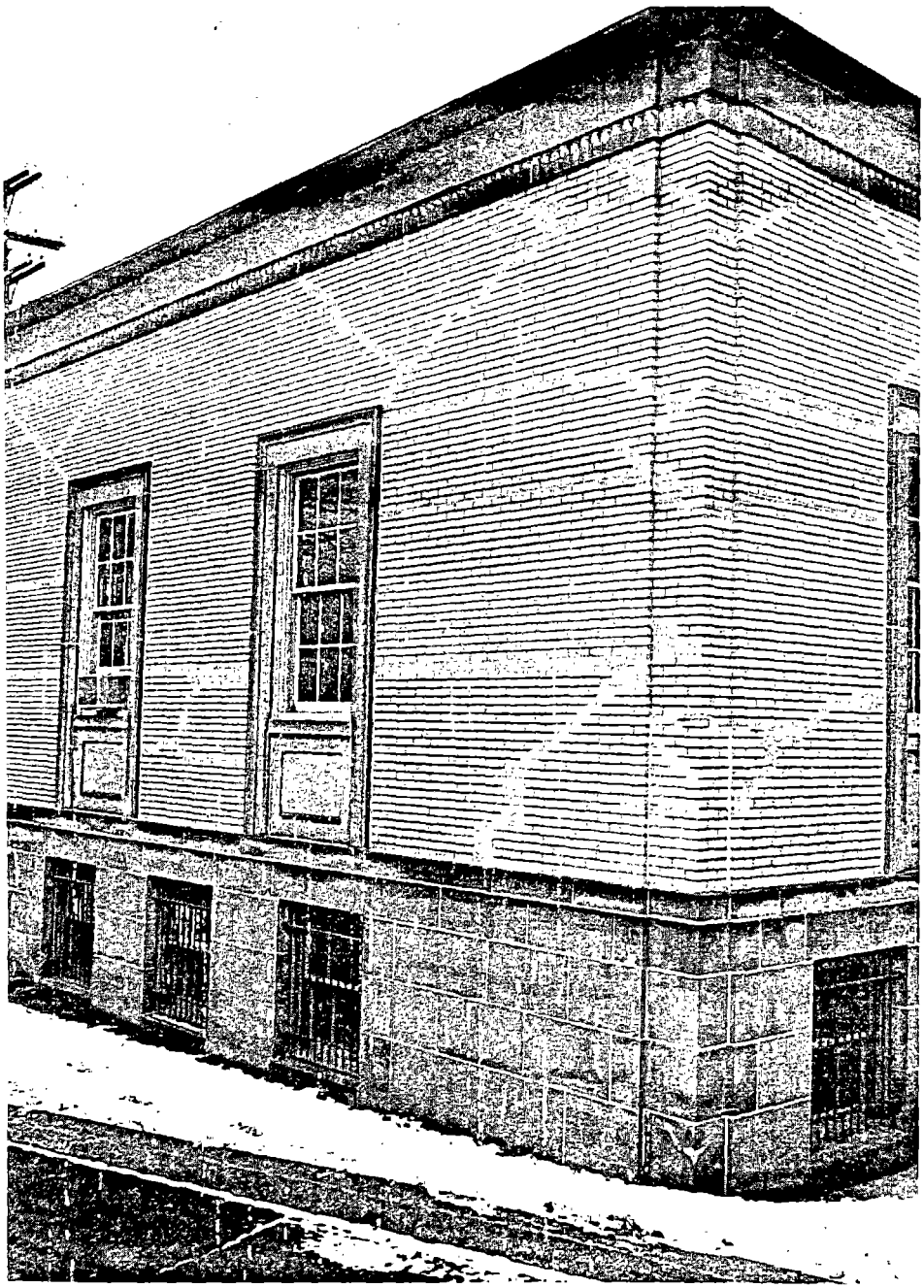
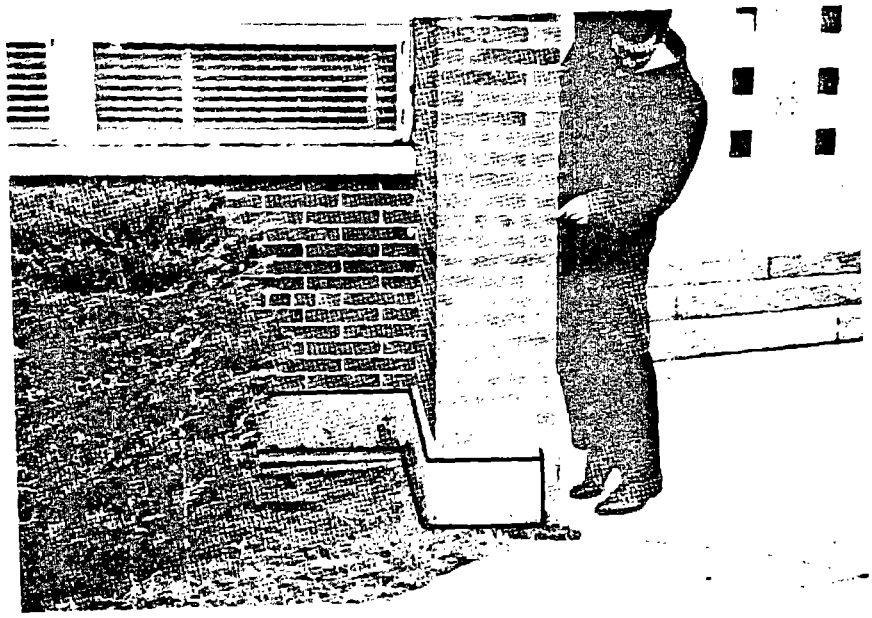


FIGURE 5.5 - EXAMPLE OF DAMAGED BUILDING THAT HAS BEEN REPAIRED. CAUSE OF DAMAGE IS UNKNOWN.

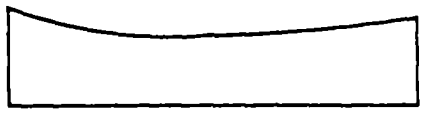


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FIGURE 5.6 - BUILDING DISPLACEMENT ON FOUNDATION CAUSED BY THERMAL EXPANSION
(after Feld, 1968).

Cause of Distortion:

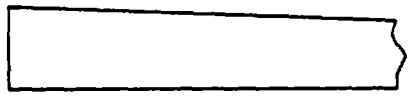
Heavy snow load on roof



Thermal expansion of roof



Columns buckled during fire



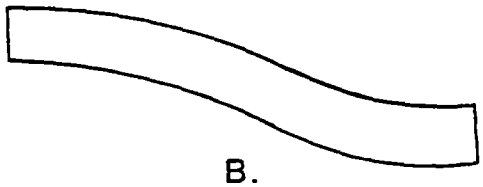
A.

Cause of Distortion:

Subsidence due to undermining



Landslide under one end of building



B.

FIGURE 5.7 - TYPES OF BUILDING DISTORTIONS A. EXAMPLES OF PARTIAL OR LOCAL DISTORTION B. EXAMPLES OF TOTAL DISTORTION

other levels of the building. Fire, collision or thermal gradients may similarly influence only limited portions of a structure. Total distortions affect the entire structure, as shown in Figure 5.7B.

Obviously, the degree and type of distortion depends on the rigidity of the structure, its foundation, and the nature of the overstressing or weakening that causes distortion.

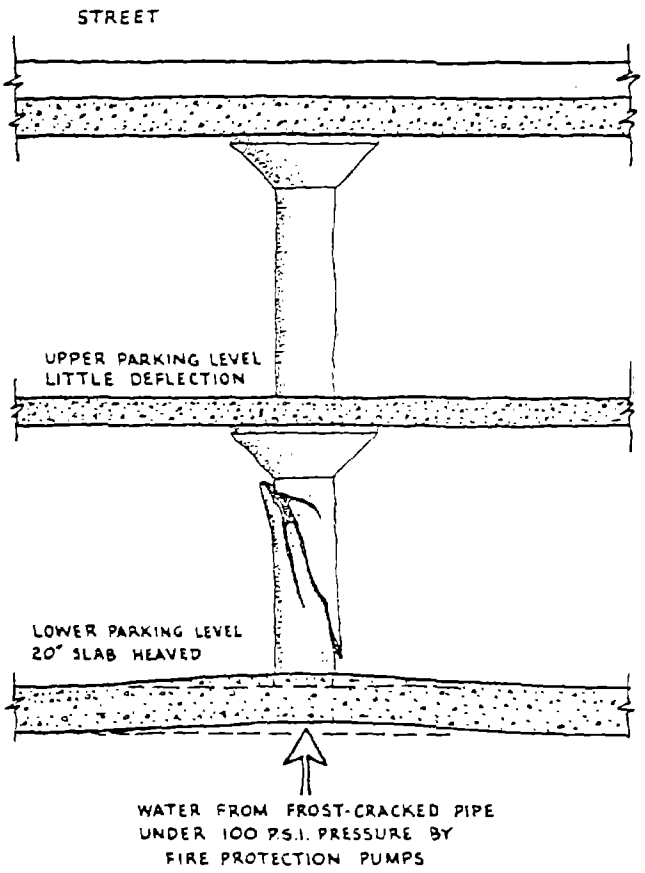
Localized ground movements, such as those caused by frost heave, sinkhole collapse or highly erratic soil conditions, may result in local distortions of a structure. Feld (1968) reports that thirteen lower columns in a two-story flat slab parking garage sheared as the roof and upper floor resisted upward movement caused by the build-up of ice when a waterline cracked (Figure 5.8). Lukas and Gnaedinger (1972) discuss a case where the subsoils were affected by chemical attack. A statement made by these investigators relative to interpretation of building damage and ground movements is repeated below because of its general applicability to damage evaluation:

> "In multi-story buildings one occasionally sees a floor in which damage occurs and the members situated above or below this level are undamaged on adjacent floors. When one tries to analyze why cracks occur, it soon becomes apparent that if anything is unpredictable, it is the way the structure will behave under different loads and deflections."

The variety of building distortions that can result from all sources of damage is endless. No attempt is made here to discuss the distortions from non-ground movement sources, which vary so significantly from site to site that few typical forms emerge. Distortions from ground movements tend to be more amenable to categorization and are discussed below.

Five common modes of building distortion are shown in Figure 5.9 and have been exaggerated for the sake of clarity. Examples of ground movements that could cause each distorted shape are also shown. Although these sketches of building distortion might aid in the identification of causative ground movements, there is clearly no one-to-one association between a specific type of building distortion and a specific type of ground movement. Additional ground movement possibilities leading to each distorted shape in Figure 5.9 are certainly possible. Almost any ground movement under one portion of a structure can lead to one of the modes of distortion shown in Figure 5.10. Furthermore, the mode of distortion depends to a large degree on the structural rigidity of a building. A landslide under one end of a building, for example, might cause either unsymmetrical hogging or monoclinal flexure, depending on the flexibility of the building.

Unless a structure, or a portion of it, has experienced substantial displacements, distortion may not be apparent by visual observation alone. Instrument surveys and careful length



NOTE: UPPER LEVEL OF PARKING GARAGE
IS NOT AFFECTED BY ICE HEAVE

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FIGURE 5.8 - EXAMPLE OF LOCAL DISTORTION. (after Feld, 1968)

measurements are required before and after distortion occurs. One common indicator of building distortion that does not require instrument survey or measurement is the jamming of doors and windows. Another well known indicator is the appearance of cracks on the inside and outside of a structure, a subject treated in the following section.

Distorted, Displaced, or Failed Structural Members and Construction Materials. Associated with building distortions are displacements, flexures and failures of structural members, and other materials within a building. Evidence of damage may become noticeable long before building distortion becomes significant. If patterns of damage can be observed, it is sometimes possible to establish overall building distortions and thereby identify possible causes of damage.

The initial and most common evidence of building distortion is cracking of walls or surface finishes, such as interior plaster or exterior cladding. Burland and Wroth (1974) point out that building damage due to ground settlement under surface loads is almost always confined to the finishes and does not extend to the structural members. A great deal of attention often focuses on crack morphology--the interpretation of cracks in structures, their width, patterns, orientation, location and origin. Although not definitive, crack morphology can occasionally suggest the mode of ground movement responsible for damage. Tschebotarioff, for example, makes a few suggestions shown in Figure 5.11.

In reference to concrete, the American Concrete Institute (ACI, 1968) defines a crack as an incomplete separation [of a material] into one or more parts, with or without space between, and suggests that cracks in concrete be classified by the descriptors: direction, width, and depth. The adjectives 'longitudinal', 'transverse', 'vertical', 'diagonal', and 'random' have been suggested to describe directionality. The ACI classifies crack width as 'fine' (generally less than 1 mm), 'medium' (1 to 2 mm) and 'wide' (over 2 mm). The British building damage classification shown in Table 5.1 attempts to link crack width to level of damage.

Some types of local cracking frequently observed in concrete, such as pattern cracking, checking and D-cracking, are not related to building distortions and must be distinguished from the more continuous, longer cracks associated with structural distortion. The ACI (1968), Johnson (1965) and Feld (1965 and 1968) give some indication of the types and causes of cracking observed in concrete.

Crack Morphology - Ground Movements. Some crack patterns associated with the more common types of ground movements are shown in Figure 5.12. Several features are to be noted:

1. Cracks are often tapered, increasing in width with distance from the tip.

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2. Prominent diagonal crack sets are present in sagging and hogging load-bearing walls (A, B, and G). These are caused by shear deformations and the development of diagonal tensile strains.

3. Prominent vertical crack sets also develop in sagging and hogging load bearing walls. They are caused by bending deformations and the development of direct tensile strains.

4. Cracks due to ground movements may occur in any direction. Horizontal cracks appear to be somewhat less common than other types.

5. Cracks due to ground movement may appear at any location in a building--walls, ceiling, floor; basement, living quarters, etc.

6. Cracks may follow major structural components, as in (C).

7. Cracks may radiate from doors, windows, and other areas of stress concentration (E, F, G, I and K). Diagonal cracks connecting nearby stress concentrations are common (G, J, K).

The pattern of cracking is obviously a function of numerous factors, many of which are interdependent: the magnitude and distribution of ground movements; building shape; foundation type; structural design of building (particularly features affecting relative stiffness in shear and bending); and the stress/deformation characteristics of the building materials. Whereas simple structures subjected to simple modes of deformation may yield characteristic crack patterns, more complex structures subjected to the same modes of deformation may yield crack patterns that defy interpretation. Cracks do not necessarily display signatures that would allow identification of the ground movement source. Examples of structural cracking caused by various ground movements are shown in Figure 5.13. Crack patterns associated with subsidence damage above active and abandoned mines in the Eastern and Interior Coal Provinces are discussed in more detail in Section 6.

The occurrence of horizontal cracks in structures affected by ground movements is particularly noteworthy because such cracks are frequently attributed to above-ground causes. According to Sowers (1962), the horizontal crack shown in Figure 5.12A can occur in walls that are partially or completely supported by spandrel beams. Such cracks may form below the spandrels in the middle of the building, probably as a result of direct tension due to the weight of the lower portions of the wall, as is the case in Figure 5.12D. Any horizontal weakness in a load-bearing wall could serve as a plane of separation or failure during sagging or hogging of the wall.

Attewell (1977b) reports cases of horizontal cracking in the brick walls of a factory building in England that were distorted by ground movements above a soft ground sewer tunnel. The cracks

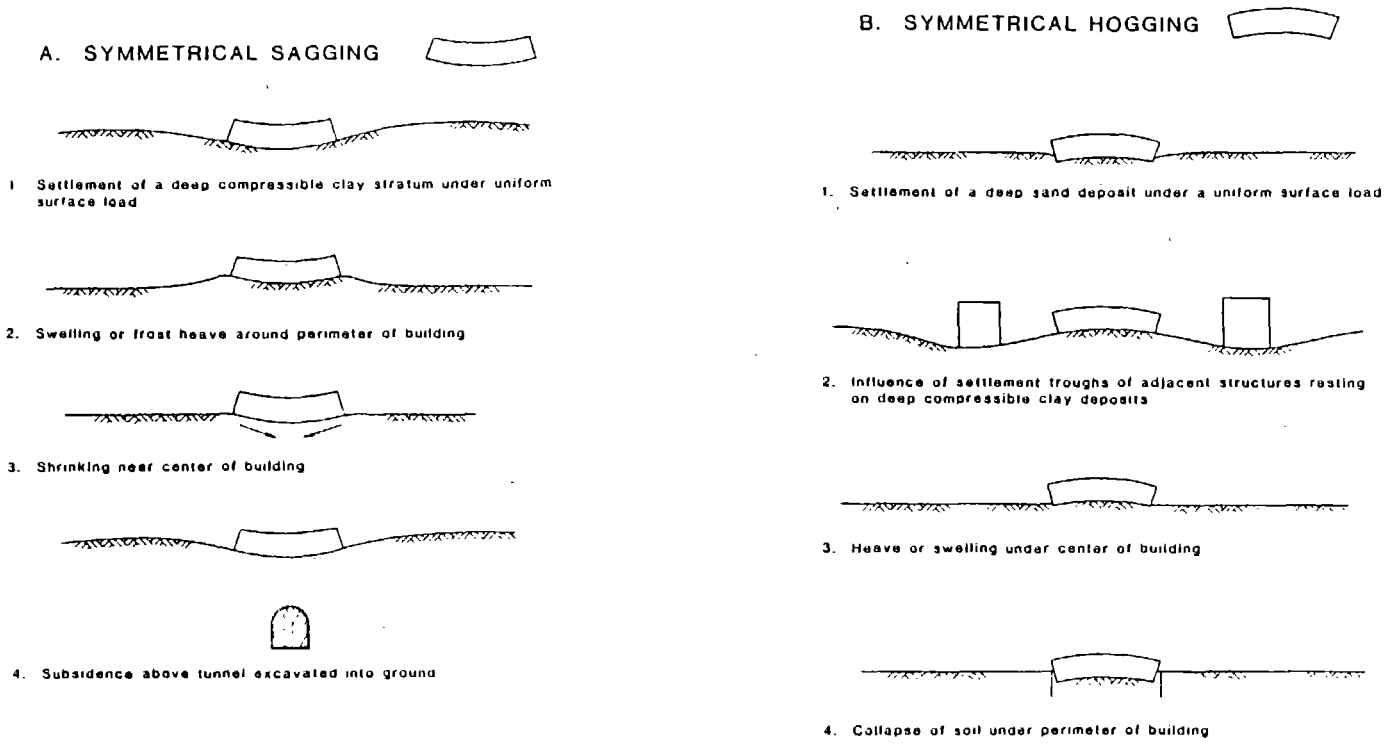


FIGURE 5.9 - FIVE COMMON MODES OF TOTAL BUILDING DISTORTION CAUSED BY GROUND MOVEMENTS.

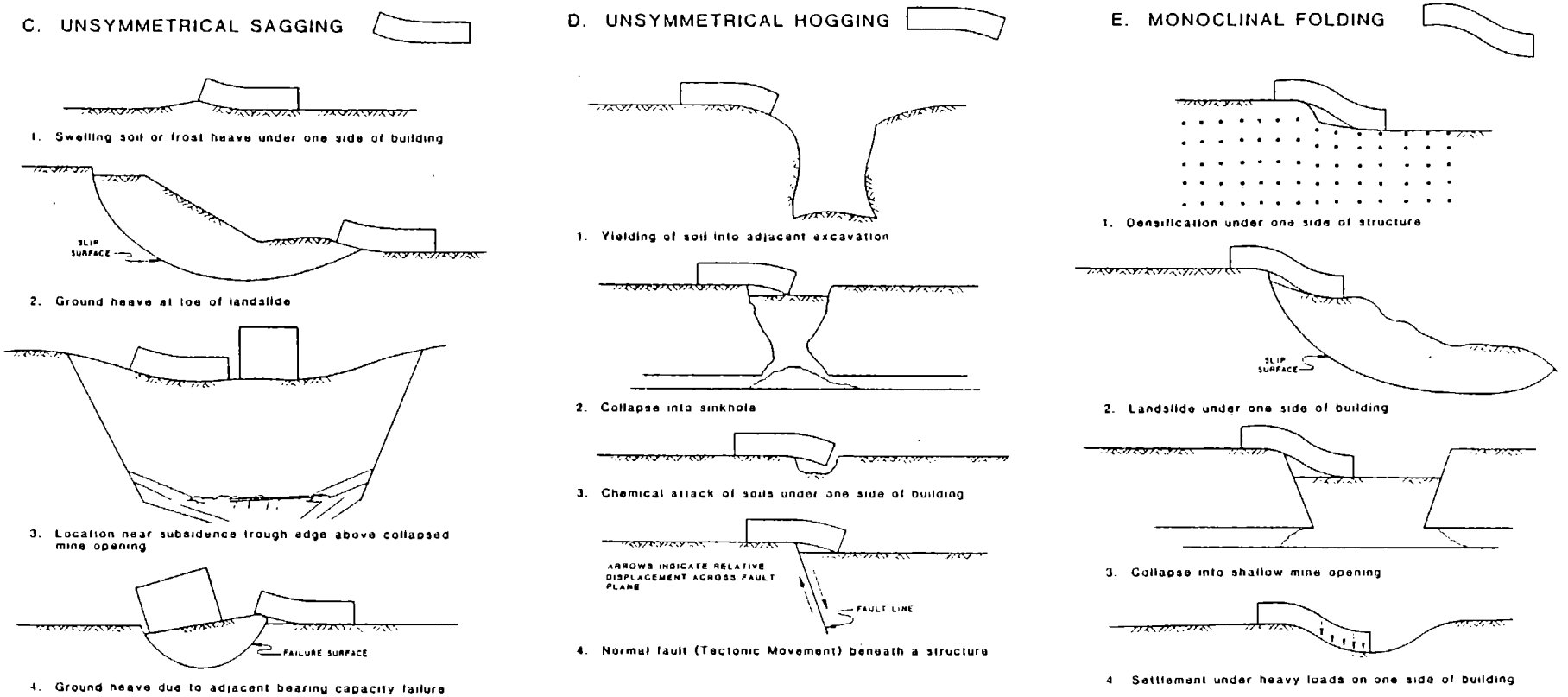


FIGURE 5.9 – CONTINUED

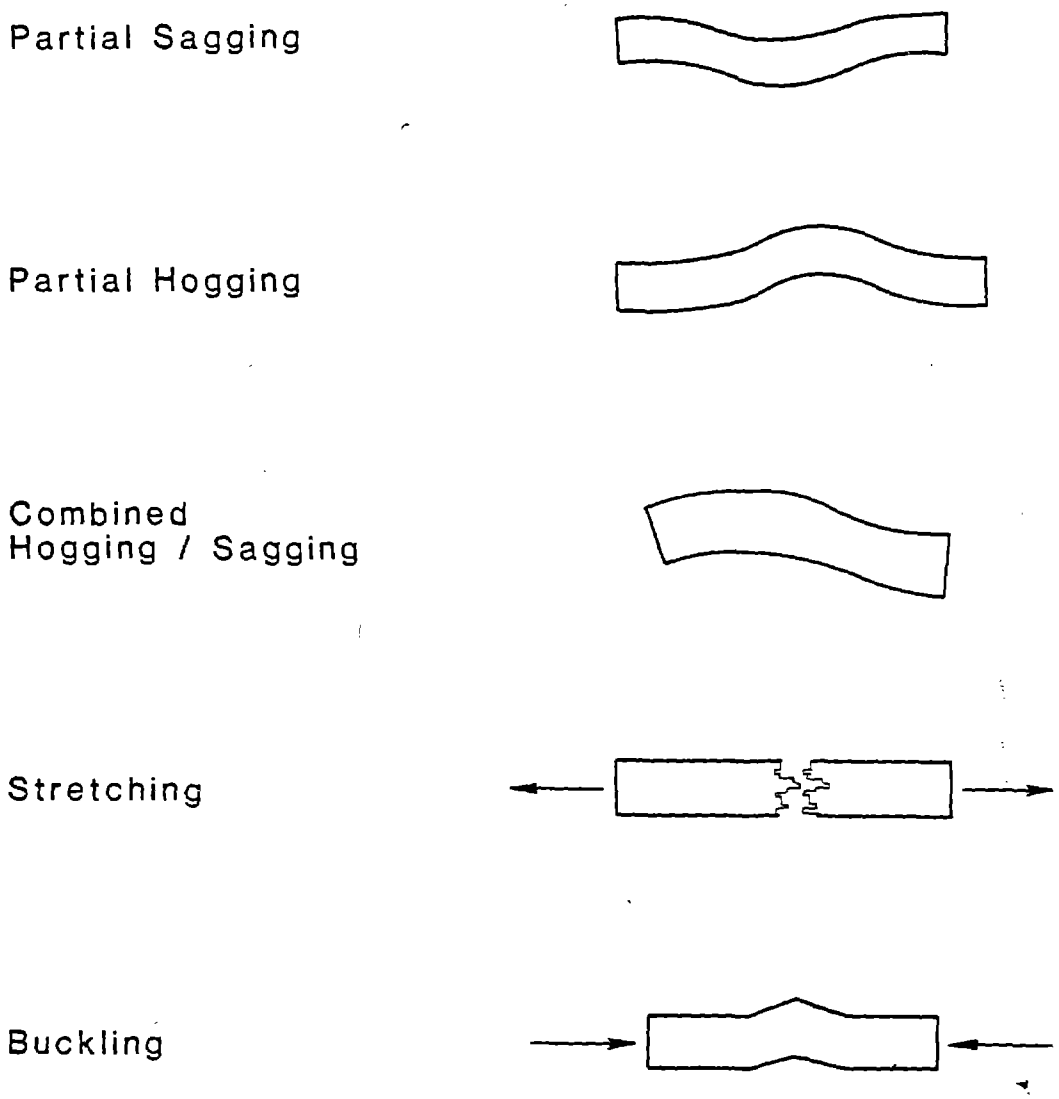
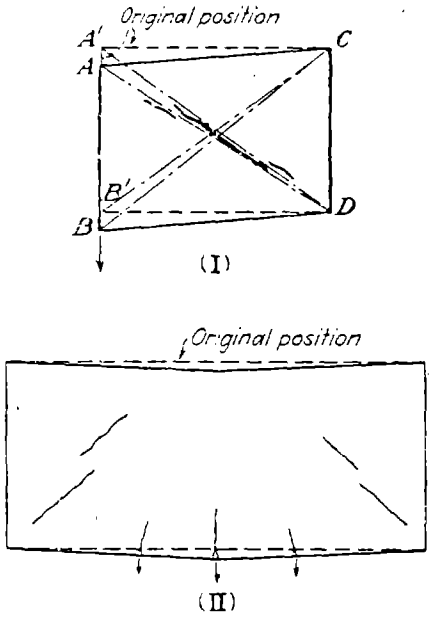


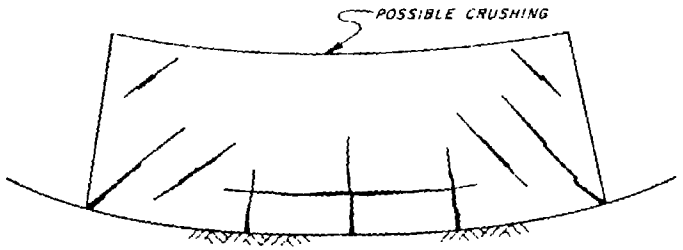
FIGURE 5.10 - CERTAIN COMMON MODES OF TOTAL BUILDING DISTORTION CAUSED BY GROUND MOVEMENT



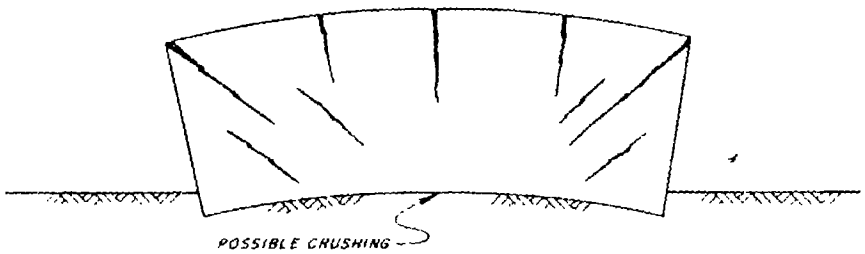
(I) Cracks in a cross wall, caused by the settlement of one of its ends from $A'B'$ to AB. (II) Cracks caused by the deflection of the central part of a front wall.

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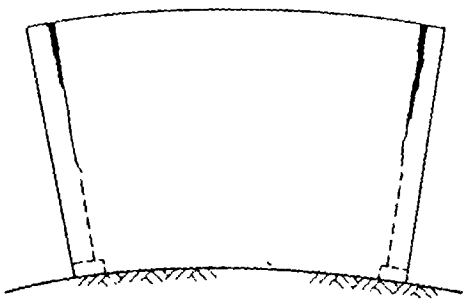
FIGURE 5.11 - POSSIBLE MANNER OF ESTIMATING THE DIRECTION OF DIFFERENTIAL SETTLEMENTS OF A BUILDING FROM THE INCLINATION OF CRACKS IN ITS WALLS (Tschebotarioff, 1973)



A. CRACK PATTERN ASSOCIATED WITH SAGGING MODE OF DISTORTION - LOAD BEARING WALL



B. CRACK PATTERN ASSOCIATED WITH HOGGING MODE OF DISTORTION - LOAD BEARING WALL



C. CRACK PATTERN OF STRUCTURE WHERE OUTSIDE WALLS CARRY HEAVY LOADS OR WHERE FOOTINGS ECCENTRICALLY LOADED.

A-C. AFTER SOWERS (1962)

D.

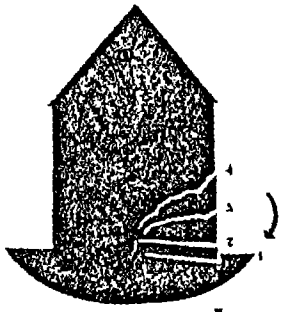


Fig. 3. Settlement of the corner of a house following withdrawal of support from below the foundation

E.



Fig. 4. Cardboard cut-out illustrating crack pattern arising from settlement at the centre of a wall with window and door openings

F.

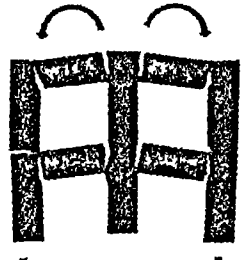
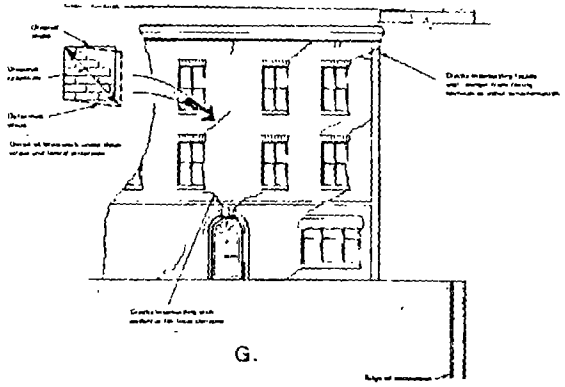


Fig. 4 Cardboard cut out illustrating crack pattern arising from settlement at the ends of a wall with window and door openings. Note that the cracks appear in the same positions but that they all taper in a different direction

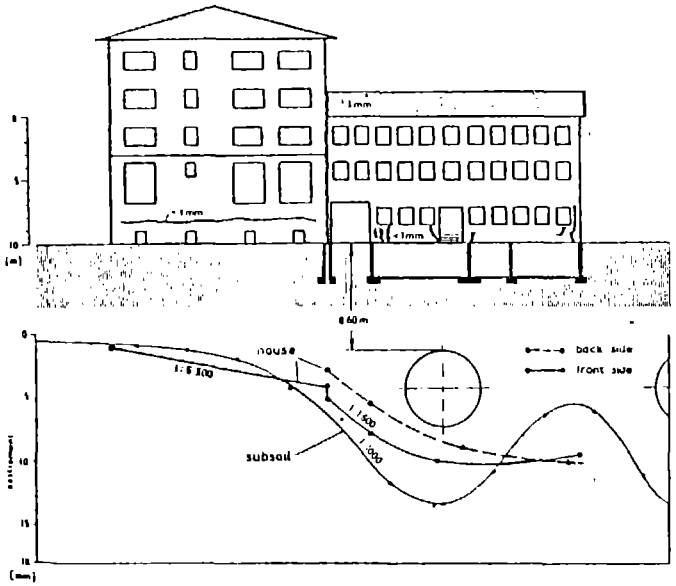


G.

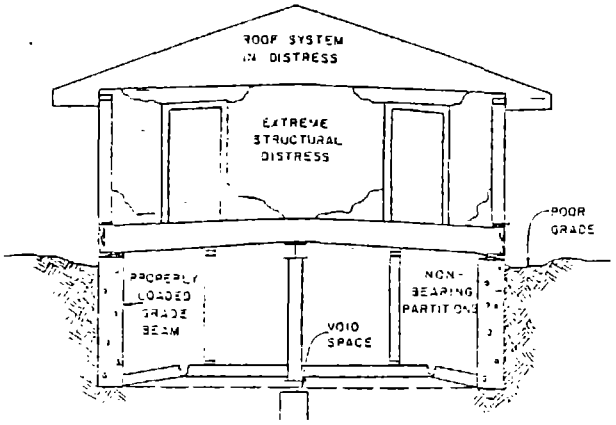
D-F. AFTER PRYKE (1974)
G. AFTER O'ROURKE, CORDING AND BOSCARDIN (1977)

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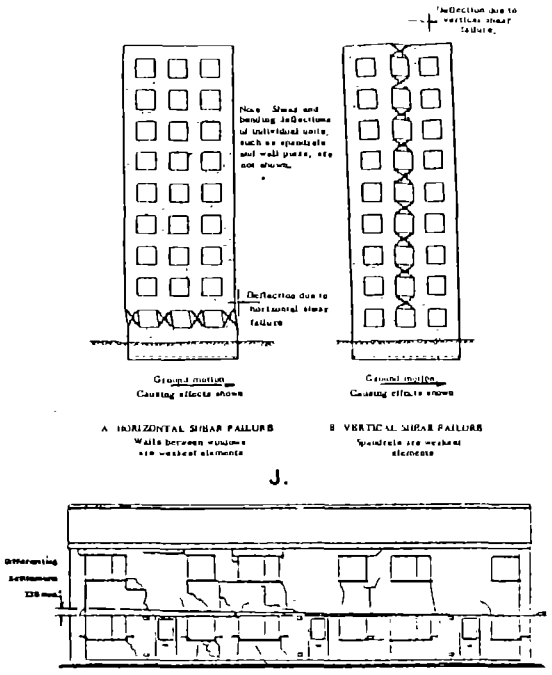
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H.



I.



J.

K.

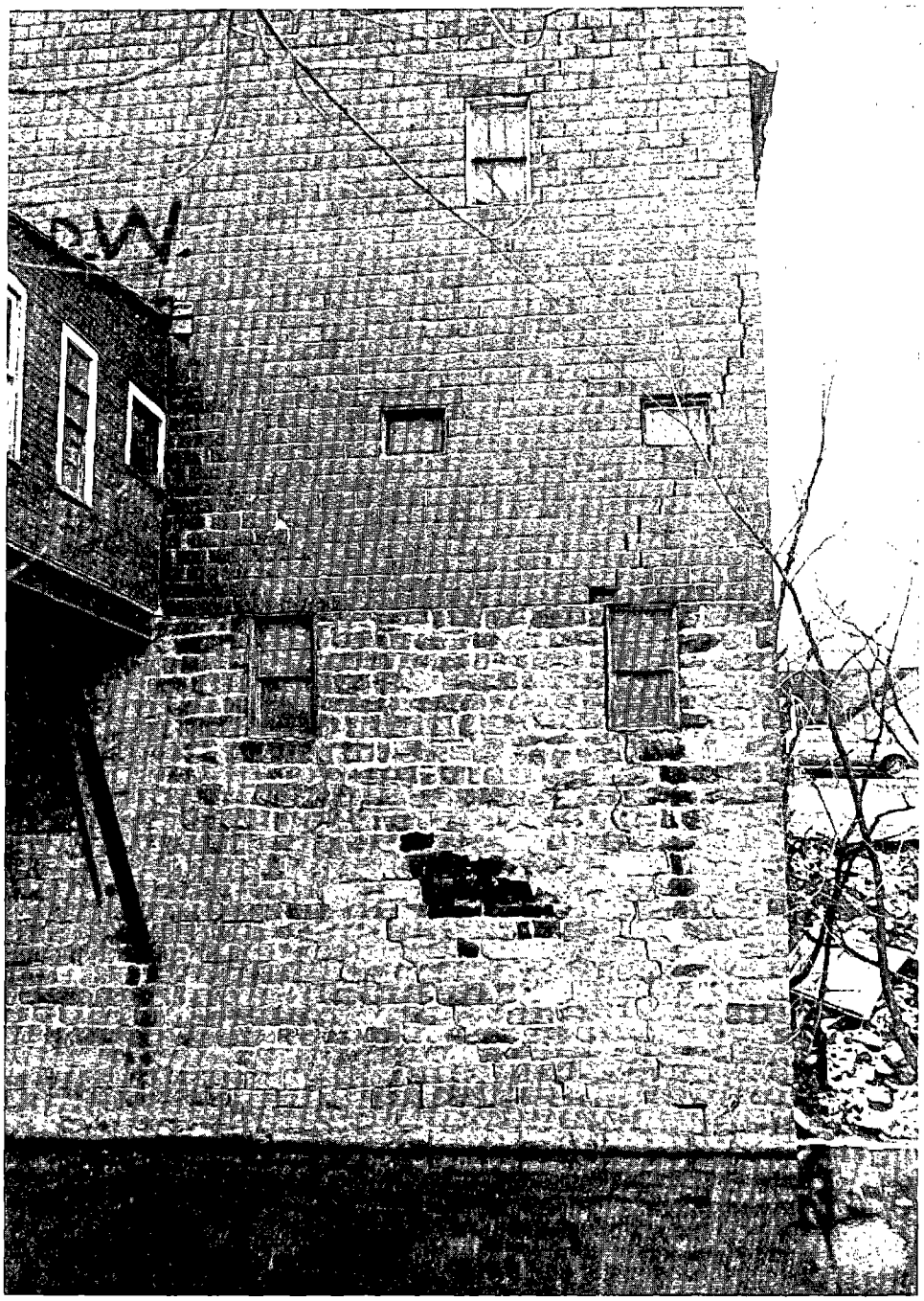
H. AFTER BRETH AND CHAMBOSSE (1974b)

I. AFTER HOLTZ AND HART (1978)

J. AFTER STEINBRUGGE (1970)

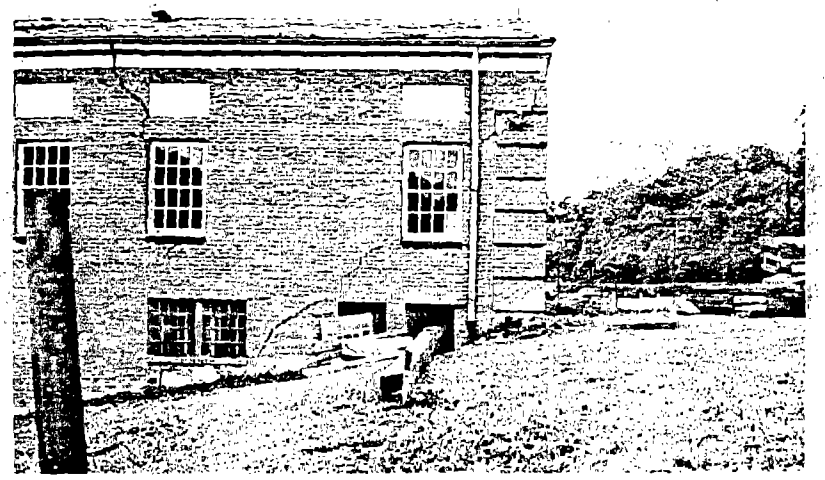
K. AFTER THORBURN AND REID (1977)

FIGURE 5.12 - CONTINUED

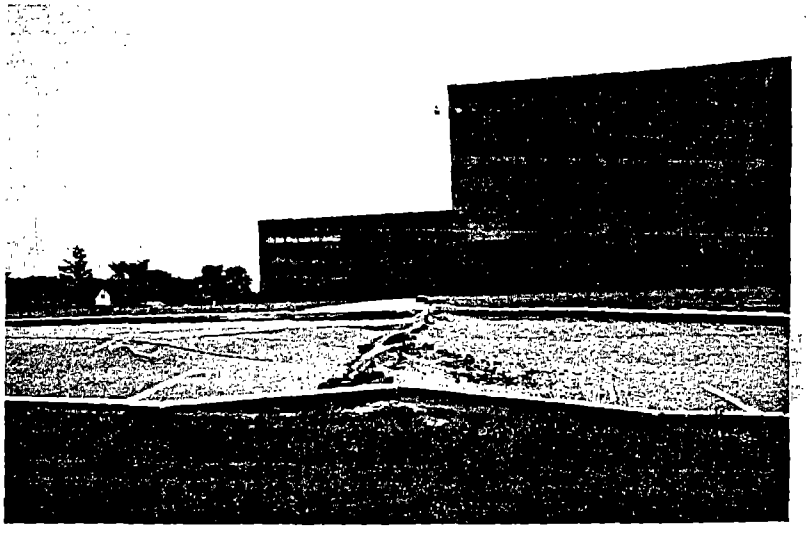


A. SETTLEMENT CRACKS IN REAR WALL OF WAREHOUSE FOUNDED IN CREEK BOTTOM

FIGURE 5.13- EXAMPLES OF CRACK DAMAGE CAUSED BY GROUND MOVEMENTS.

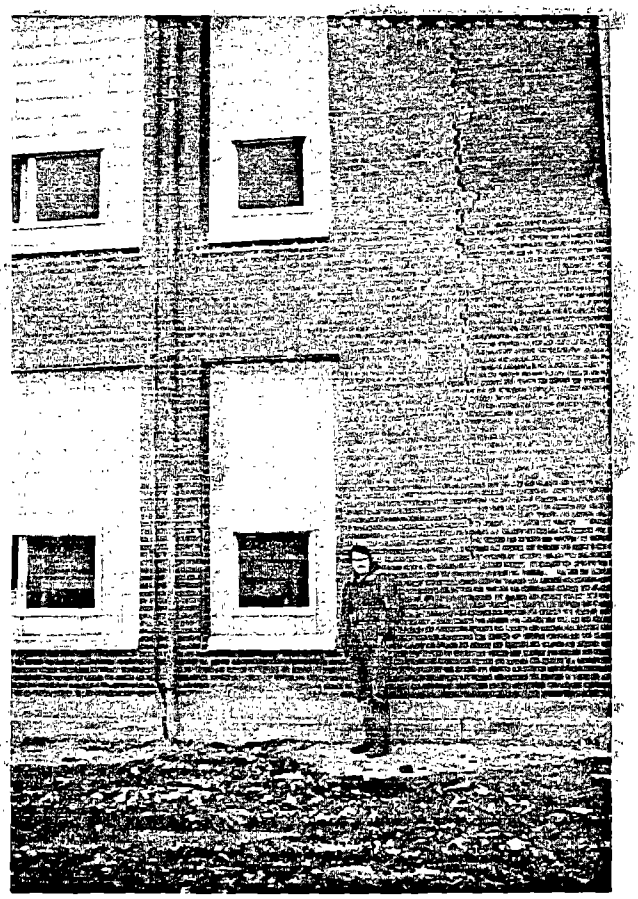


B. MINE SUBSIDENCE DAMAGE TO BRICK OFFICE BUILDING

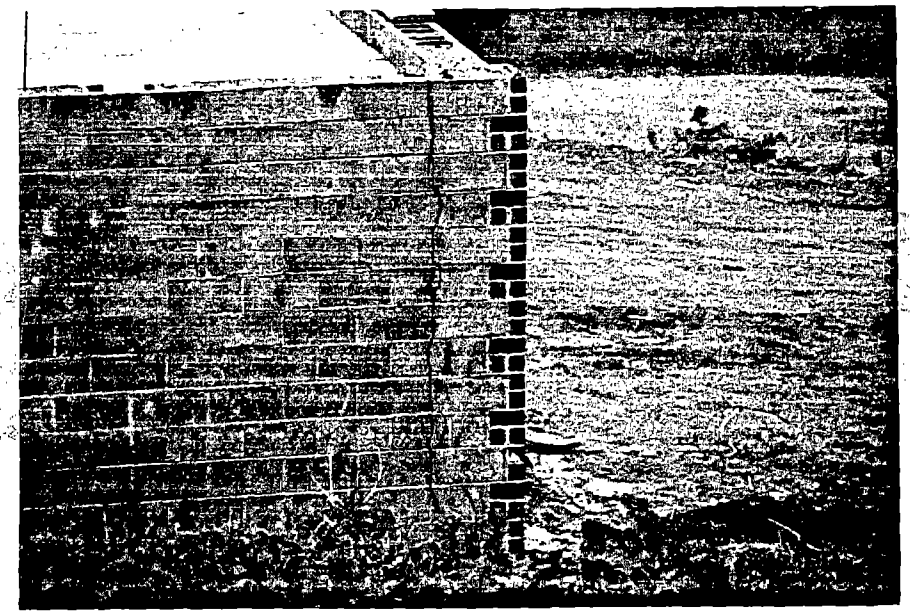


C. BUCKLED CONCRETE ROADWAY DUE TO HORIZONTAL COMPRESSION STRAINS IN SUBSIDING ZONE ABOVE COAL MINE, BEAVER COUNTY, PENNSYLVANIA

FIGURE 5.13 (CONTINUED)



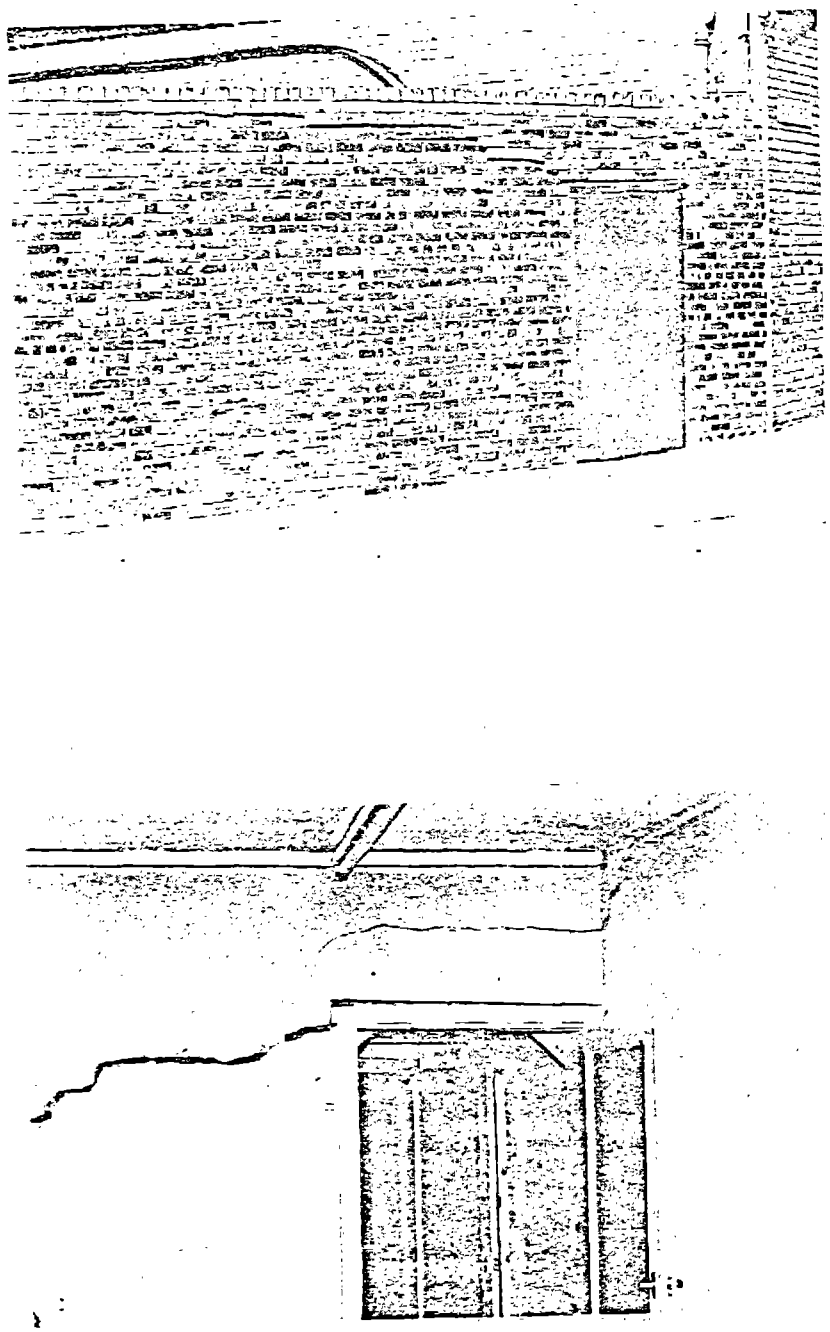
D. VERTICAL CRACK IN BRICK SCHOOL CAUSED BY MINE SUBSIDENCE



E. VERTICAL CRACK IN REAR WALL OF STOREROOM CAUSED BY LANDSLIDE PUSHING AGAINST WALL

FIGURE 5.13 (CONTINUED)

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F. HORIZONTAL CRACKING IN BRICK FACTORY CAUSED BY NEAR-SURFACE SOFT-GROUND TUNNELING

(Attewell, 1977b)

FIGURE 5.13 (CONTINUED)

occurred in three locations: 1) along the brick mortar/ damp course at the base of a wall, where the presence of a strong concrete footing to the brickwork inhibited the upward migration of direct tensile cracks under a sagging mode of distortion; 2) in a brickwall just below a flat roof, where forward ground movements associated with the passing of the tunnel excavation resulted in tensile and shearing stresses along a mortar joint; and 3) in interior brick and mortar walls subjected to out-of-phase longitudinal and transverse ground motions caused by the tunnel passing obliquely beneath the wall (Figure 5.13F).

Horizontal cracks are common in subgrade walls as a result of earth and water pressures. Frost heave can also cause horizontal cracks in subgrade walls to depths of 12 to 24 inches below ground surface.

Crack Morphology--Causes Not Related to Ground Movements. A brief review is given here of some typical examples of cracking that can be attributed to causes other than ground movements. The reader is encouraged to compare these examples with the cases presented in the previous paragraphs.

Cracking in masonry, mortar and plaster resembling that due to ground movements is commonly caused by the following: thermal expansion and contraction; differential movement between two parts of a structure; differential stiffnesses between interconnected building elements; drying shrinkage; and abrupt changes in cross-section.

Some common cracking failures in concrete and masonry have been outlined in a technical note from the Structural Clay Products Institute (1963). These failures are particularly pertinent to the assessment of building damage and are presented verbatim in the following paragraphs:

> Long Walls. During expansion, long walls with openings develop shearing stresses in areas of minimum cross-section. Diagonal cracks often occur in piers between window or door openings. Such cracks usually extend from the head or sill at the jamb of the opening, depending upon the direction of movement and the path of least resistance (Figure 5.14A).
>
> Foundations. When masonry walls are built on concrete foundations that extend above grade, thermal expansion of the masonry may work against the drying shrinkage of the concrete, causing extension of the masonry wall beyond the foundation. This action often cracks a section of the concrete foundation. When the masonry wall contracts with lowering of temperature, the tensile strength of the masonry is not sufficient to move the masonry wall back with it, thereby causing cracks in the masonry near the corners (Figure 5.14B).
>
> Parapet Walls. Parapet walls are particularly troublesome. With two exposed surfaces, parapets are subjected to extremes

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of moisture and temperature which may be substantially different from those in the wall below. Although throughwall flashing is often necessary, it creates planes of weakness. Expansion of roof systems can aggravate the problem. Furthermore, parapets lack the weight necessary to maintain integrity. Expansion can cause parapets to bow, to crack horizontally at the roof line and to overhang corners (Figure 5.14C).

Incased Columns. Where structural elements are incased with masonry, any movement of the column is transferred to the masonry, subsequently causing vertical cracks. These cracks occur on the exterior as well as the interior of the building (Figure 5.14D).

Curling of Concrete. When concrete floor and roof slabs are poured directly on masonry walls, curling of the slab often occurs due to shrinkage, deflection, and plastic flow of the concrete. If the slab warps, it can cause ruptures in the masonry, particularly at the corners. Horizontal cracks often occur below the slabs. It is often noted that portions of the wall below such a crack are not in the same vertical plane with the wall above the crack (Figure 5.14E).

Corners. In construction utilizing a structural frame, cracking of the masonry at the corners has been a serious problem in recent years. This cracking is primarily caused by thermal movement of the outside wythe of the wall. It is particularly prevalent in insulated cavity wall construction (Figure 5.14F).

Offsets and Setbacks. Vertical cracks are quite common at wall setbacks or offsets. When one or more walls expand toward an offset, the movement produces bending in the offset, causing vertical cracks (Figure 5.14G).

Shortening of Structural Frames. In high-rise buildings, vertical shortening of the structural frame frequently imposes very high stresses in the masonry facing. Under these conditions, masonry failures are characterized by bowing, by horizontal cracks at angles, by vertical cracks near corners, and by spalling of masonry units at window heads, shelf angles and other points where stresses are concentrated. Excessive shortening may occur in either steel or concrete frames. It is concrete, however, which is the greatest offender, particularly lightweight concrete (Figure 5.14H).

Additional examples of cracking due to common causes other than ground movements can occur in the following building elements.

Plaster. Plaster is likely to shrink a different amount than the wood or masonry base which supports it. Shrinkage cracks in plaster walls and ceilings are usually vertical and horizontal, and are of uniform width or become narrow at both ends (Sowers and Sowers, 1970). Examples of such cracks are shown in Figure 5.14I. Short shrinkage cracks may also propagate

diagonally from the corners of doors and windows. Other common cracks in plaster, caused by the deflection of relatively flexible framing, such as wood studs, rafters, or floor joists that support walls or ceilings, may follow the outline of the supporting members. Thermal expansion cracks can occur in interior plaster walls as well as in exterior masonry walls.

Grade Beams. If a foundation element such as a pile, pile cap, pier, or footing pedestal fails due to deterioration or poor construction or is omitted from construction, grade beams, cap beams, and other major load carrying structural elements may crack and even collapse. Such failures are likely to be ascribed to ground movements. One case is shown in Figure 5.14J, where failure occurred because a foundation pile was omitted. The cracking of the brick wall could easily be misjudged as due to settlement. Similar cracking can result from the omission of reinforcing stirrups during construction, from the deterioration of foundation piles by rotting (timber) or sulfate attack (concrete), etc.

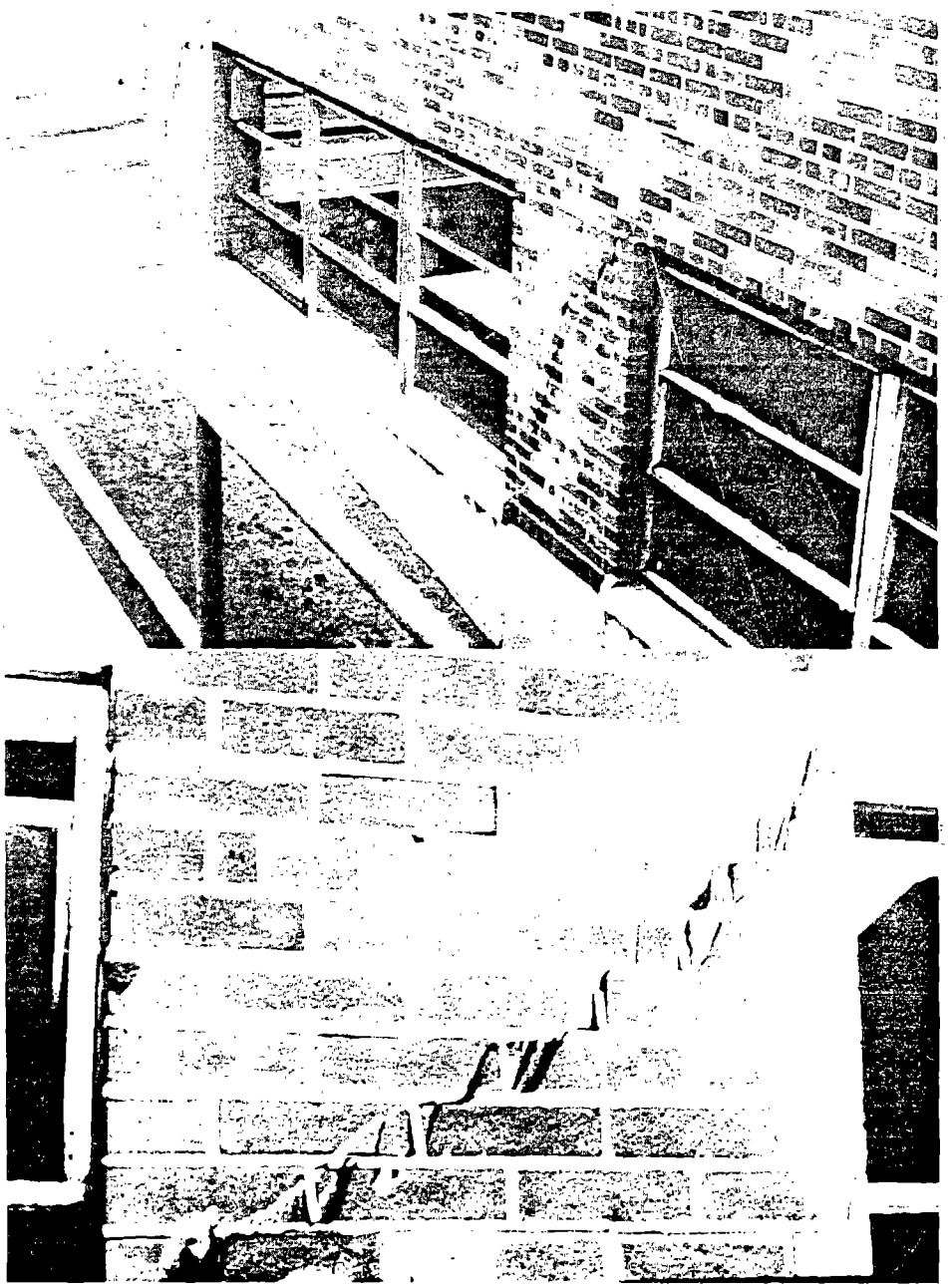
Partitions and Interior Walls. If the stiffnesses of partitions and interior walls that bear on or are rigidly connected to floors are not compatible with floor deflections, crack patterns may develop in the partitions and walls. Because the deflected floor may sag in a dish-shaped profile, crack patterns may resemble those for the sagging mode of distortion shown in Figure 5.9A caused by ground movement. Openings in partitions and walls can create the same type of problem, as shown in Figure 5.14K.

Floor Slabs. Cracking of floor slabs can be caused by numerous factors: settlement of subgrade during construction (Figure 5.14L); drying shrinkage; temperature changes (Figure 5.14M); abrupt changes in cross-section (Figure 5.14N); and floor overload.

Crack Morphology - Concluding Remarks. The following conclusions can be drawn from the foregoing discussion of crack morphology:

1. Observed cracks may result from a variety of causes and should not automatically be attributed to ground movements. In fact, among the variety of cracks found in a typical building, some almost certainly will not have been caused by ground movements. (Inspection during or immediately after an episode of ground movement may permit the differentiation of old cracks from new. Old, (pre-existing) cracks are often paint-filled and exhibit dirty, weathered faces; new cracks generally exhibit clean, fresh faces.

2. Ground movements typically cause a series of cracks rather than a single one, although the presence of a series of cracks is not sufficient reason to conclude

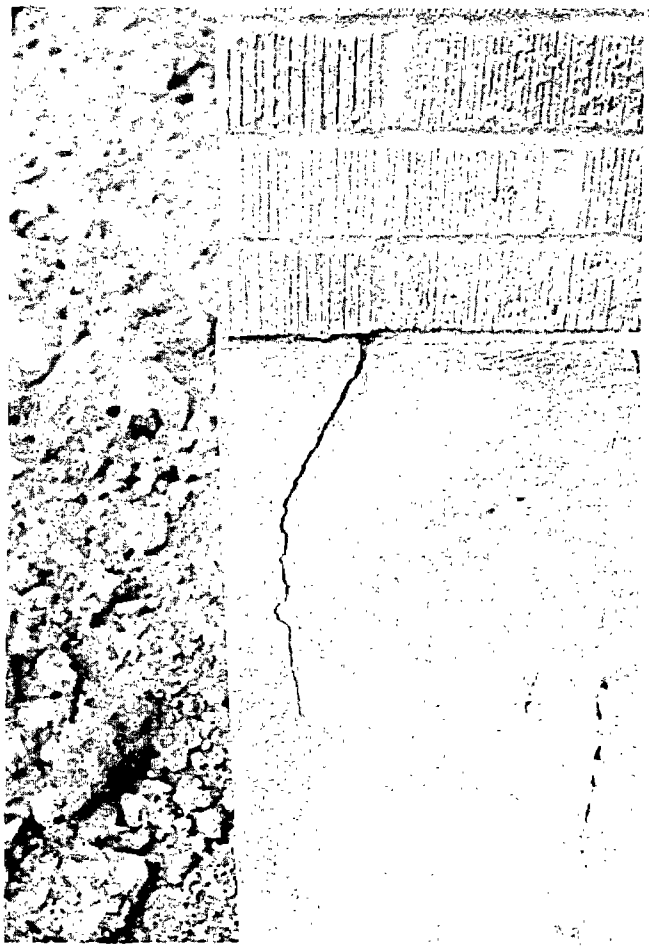


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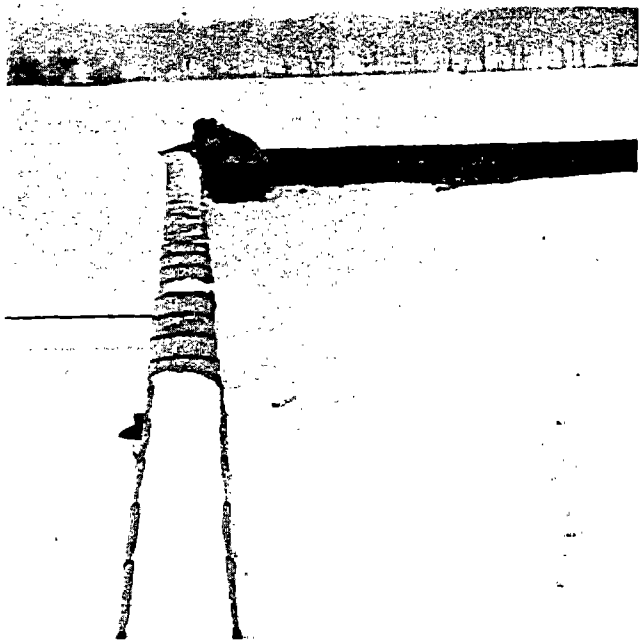
A. THERMAL EXPANSION CRACKS IN LONG WALL.

FIGURE 5.14 - EXAMPLES OF CRACK DAMAGE CAUSED BY SOURCES OTHER THAN GROUND MOVEMENTS

(A-H. After Structural Clay Products Institute (1963): J. After Feld (1968); K-N. After Johnson (1965)).

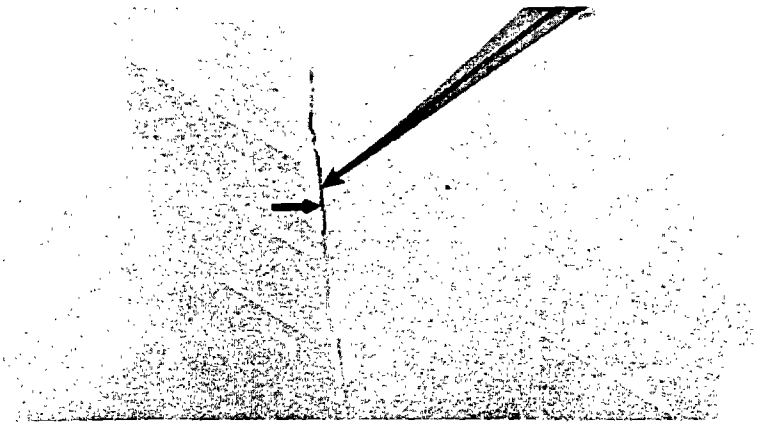


B. CRACKING AT FOUNDATION CORNER DUE TO SLIDING OF BRICKWALL DURING THERMAL EXPANSION.



C. BOWING OF PARAPET DUE TO THERMAL EXPANSION.

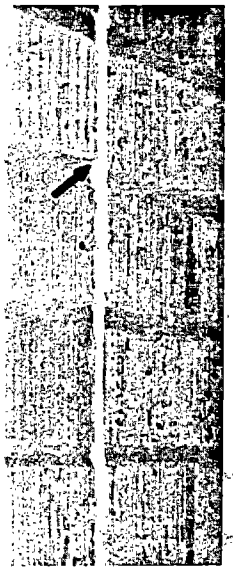
FIGURE 5.14 (CONTINUED)



D. CRACKING DUE TO STRUCTURAL DEFLECTION

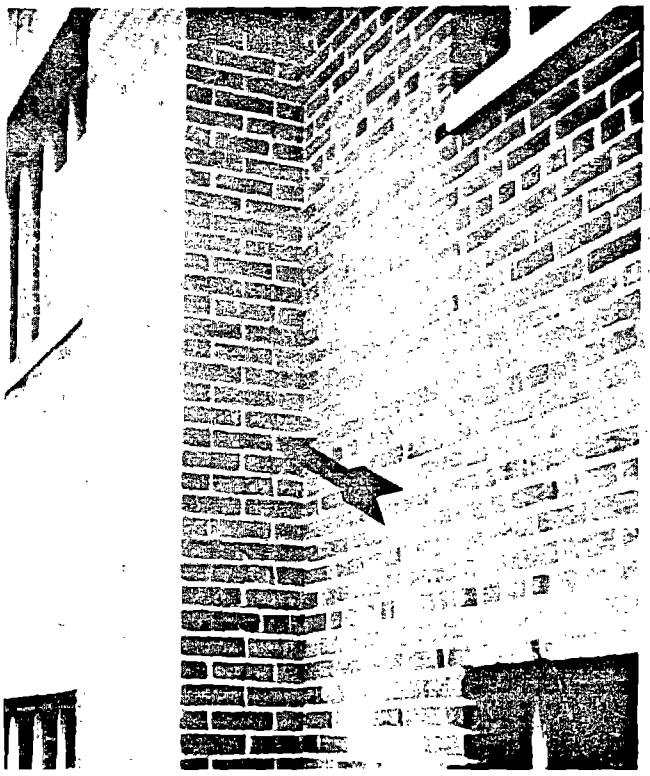


E. CRACKING DUE TO CURLING OF CONCRETE SLAB.

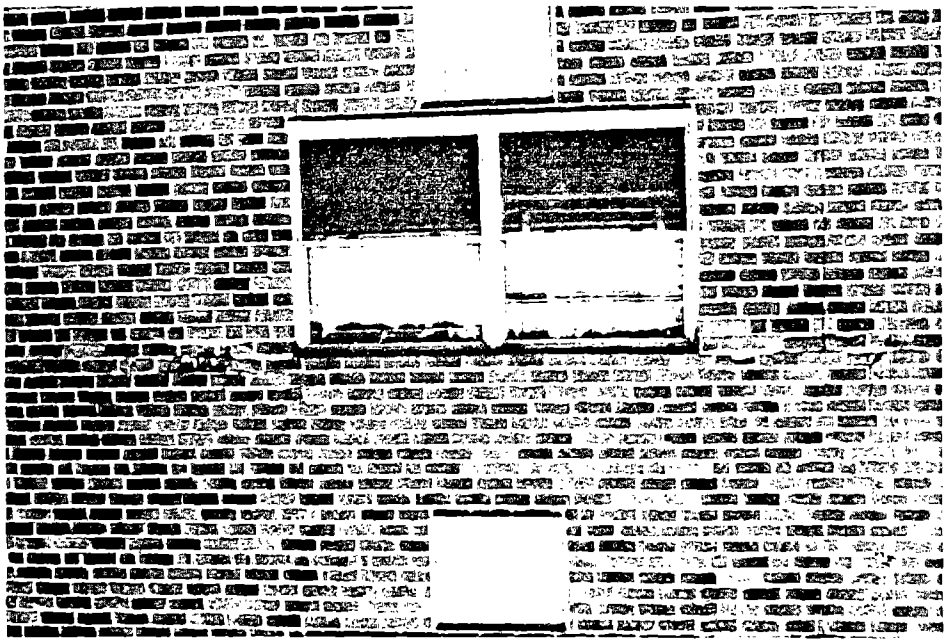


F. CRACKING DUE TO THERMAL EXPANSION

FIGURE 5.14 (CONTINUED)

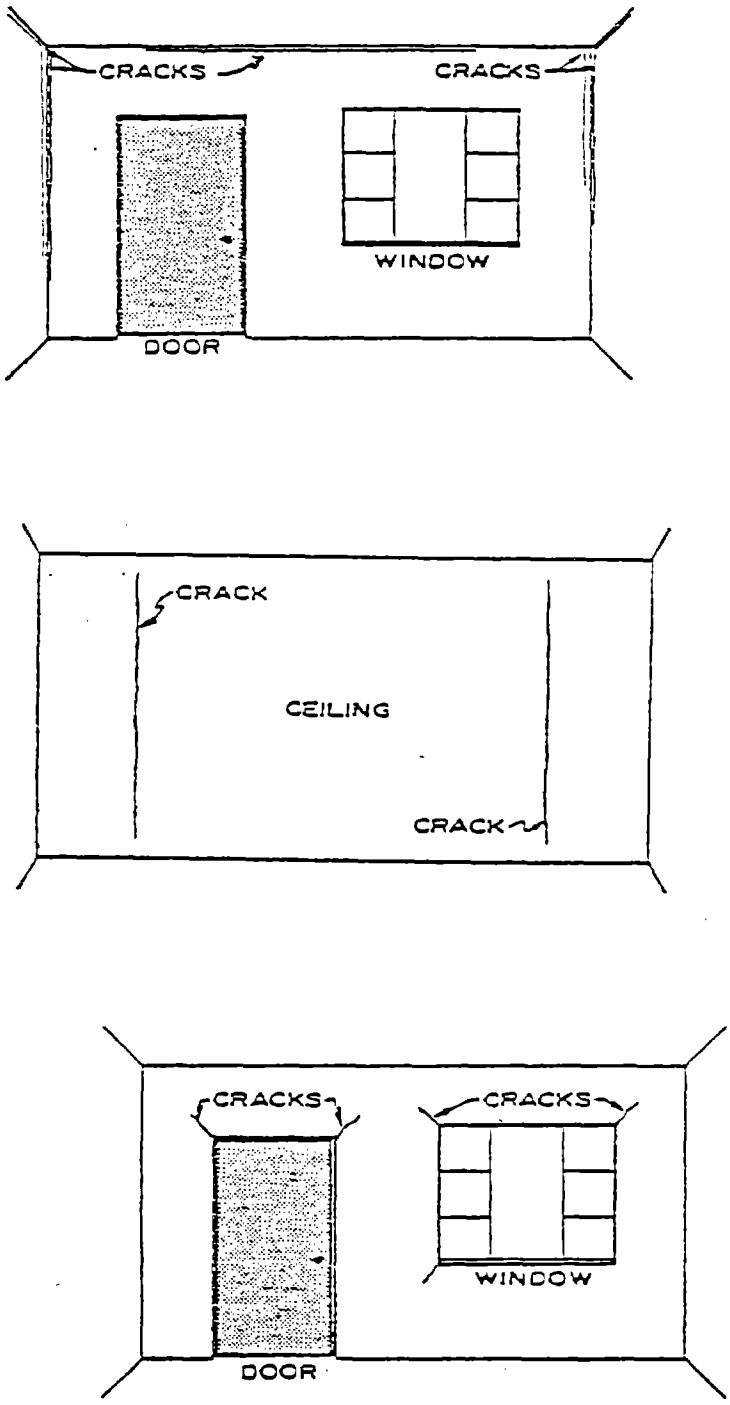


G. CRACKING DUE TO THERMAL EXPANSION OF WALL.



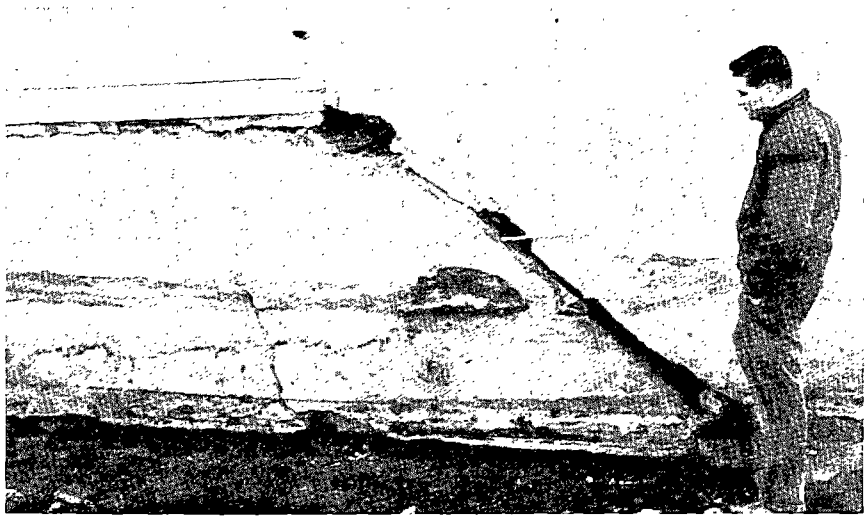
H. CRACKS DUE TO SHORTENING OF STRUCTURAL FRAME.

FIGURE 5.14 (CONTINUED)



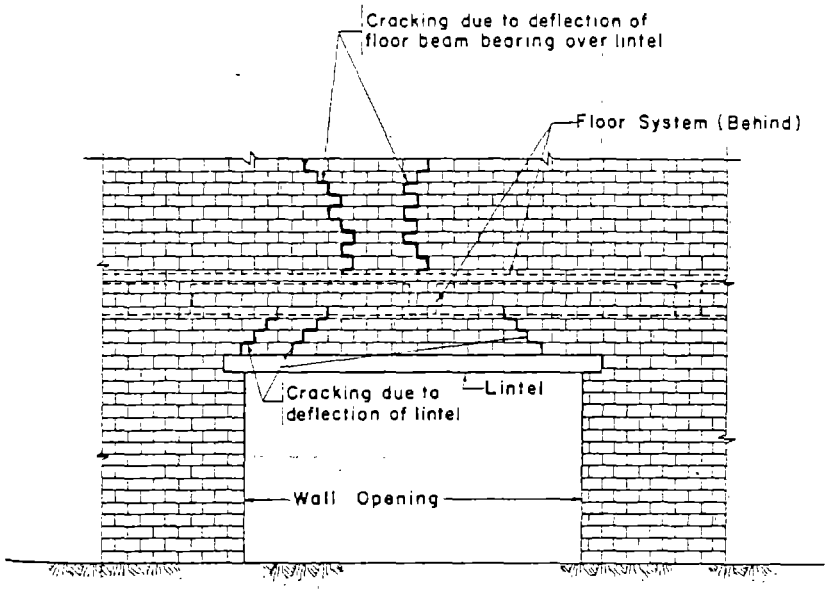
1. TYPICAL PLASTER SHRINKAGE CRACKS.

FIGURE 5.14 (CONTINUED)

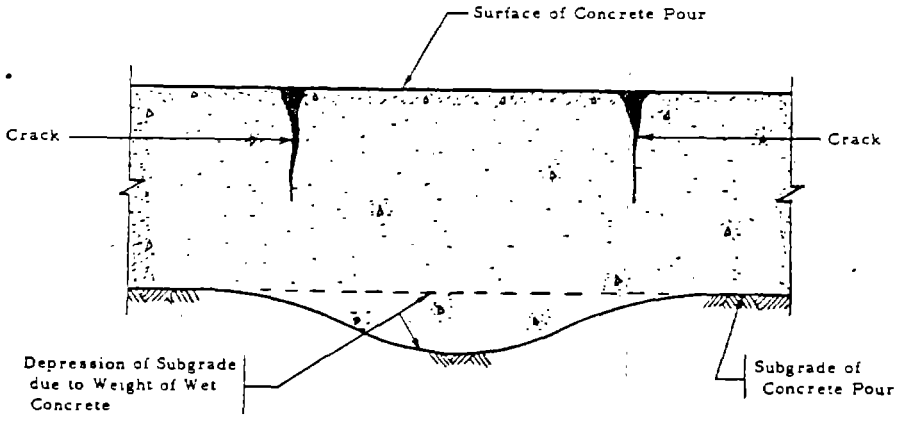


J. CRACKS IN GRADE BEAM AND BRICKWALL CAUSED BY OMISSION OF FOUNDATION PILE.

FIGURE 5.14 (CONTINUED)

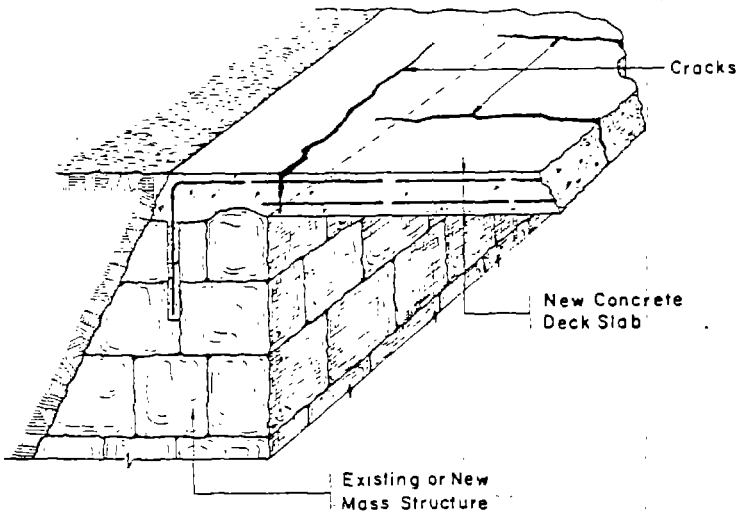


K. CRACKING DUE TO DEFLECTION. NOTE THE SIMILARITY OF THE CRACK PATTERN TO THAT PRODUCED BY SETTLEMENT OF A FOUNDATION.

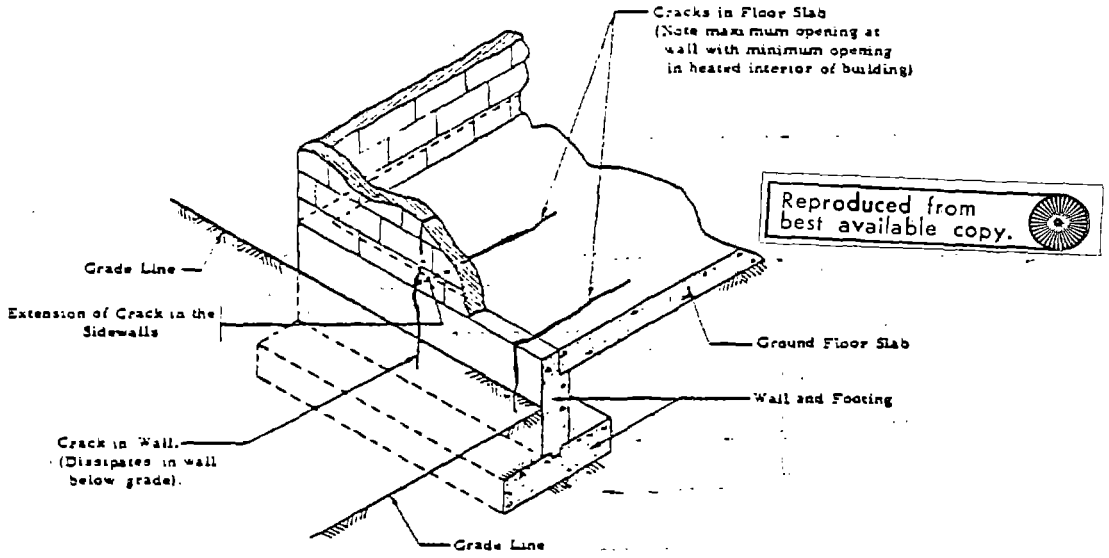


L. CRACKING DUE TO SETTLEMENT OF SUBGRADE DURING CONSTRUCTION.

FIGURE 5.14 (CONTINUED)



M. TEMPERATURE CRACKING IN A BUILDING.



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N. CRACKING DUE TO AN ABRUPT CHANGE IN SECTION OF CONCRETE STRUCTURE.

FIGURE 5.14 (CONTINUED)

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that ground movements have taken place. It is true, however, that a single crack often is not due to ground movement.

3. Crack direction is sometimes a reasonable indicator of cause and has led to such generalizations as: "cracks in walls due to foundation settlements usually ran diagonally," (Johnson, 1965); "cracking that is not diagonal will rarely be related to the foundations" (Legget, 1973) and "diagonal cracks are caused by settlement". Care must obviously be exercised in applying such truisms.

4. Crack width is not a reliable indicator of cause of damage, but crack extension and width changes can indicate continuing movement.

5. Thermal expansion cracks can frequently be identified by their opening and closing with temperature changes (Sowers and Sowers, 1970). Tschebotarioff (1973) warns, however, that "once a crack has been formed because of excessive structural deformations (distortion), it immediately becomes a temperature expansion joint."

6. Crack patterns in foundation walls and footings can sometimes be useful indicators of source of ground movement. Crack patterns in brick facings can also be important insofar as facings are tied to the footings. Crack patterns within the living quarters are often influenced by internal structural and cosmetic details and are therefore more difficult to interpret and less useful for assessing the probable source of ground movements.

7. Crack morphology should not be relied upon exclusively to indicate the cause of structural damage.

Evaluation of Cause of Damage

Visual information is only one form of evidence that might be available in evaluating the cause of damage to a structure.

Its value frequently can be enhanced by additional information. The age and historical development of cracks, the physical and topographic layout of the area surrounding the damaged structure, records from building construction, reports or observations of the physical condition of adjacent structures, streets, and utilities; and details of recent construction activities, disasters, or unusual climatic events are but a few examples of valuable evidence that may give credence to a particular damage hypothesis (Janney, 1979). Sometimes these extra pieces of evidence, in combination with the observed features, are sufficient to reliably evaluate the cause of damage. At other times, more comprehensive and time-consuming level surveys or subsurface exploration programs are required to reach a conclusion. There always exists

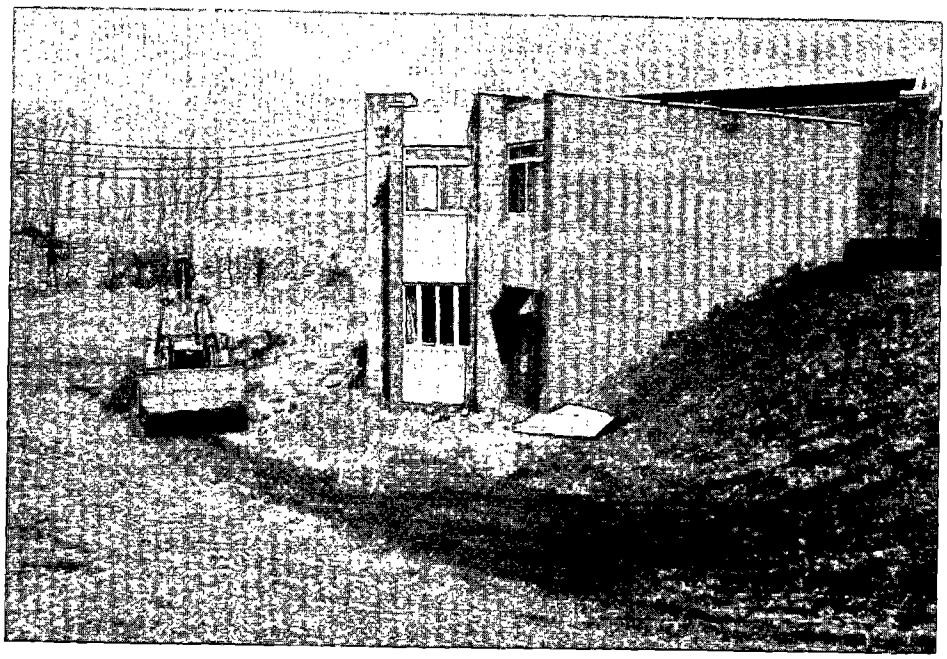
the possibility that no reasonable amount of investigation will lead to a positive conclusion. Mr. Richard Driscoll of the Building Research Station stated, in personal communication, that uncertainties in determining causes of minor (architectural) damage to homes in the United Kingdom has led the government to conduct hearings for the purpose of limiting the National Coal Board's liability for mine subsidence damage to cases of functional or structural damage (Categories 3 and above in Table 5.1). Johnson (1965) emphasizes that "there are no rules or set procedures for determining the cause or causes of building deterioration. Each case is an individual problem and must be individually diagnosed."

The following is a case in point. The first writer was requested to determine the cause of cracks that were developing in a number of interior walls of a small brick church located in western Pennsylvania. Church members indicated the walls had been cracking perceptibly over a period of three months on an almost continuous, week-by-week basis. Although inspection proved the damage to be no more than cosmetic, church members anticipated potentially serious consequences in the event the phenomenon continued unabated. For this reason, a more thorough investigation was intiated.

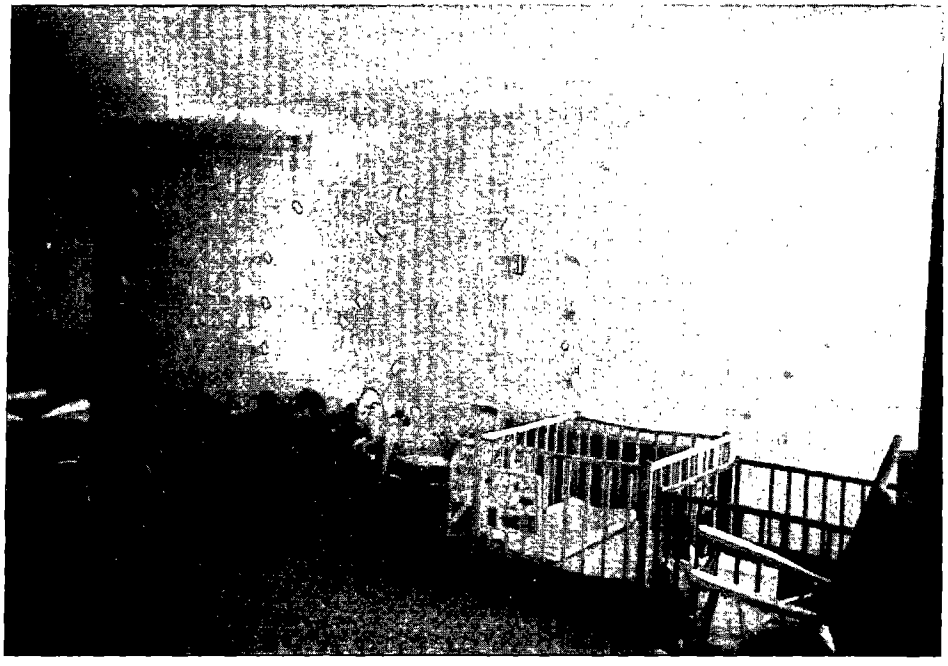
Many of the more pronounced cracks were located in the interior walls of the upper of the two building levels and were oriented near vertically in broad arcs concave toward one end of the building (Figure 5.15). These cracks were indicative of a hogging type deformation, such as might be associated with either settlement of one end of the building or heaving of the interior. (Recall Figures 5.9B and D).

Because the wall cracks began appearing almost 20 years after construction of the church edifice, changed conditions at the church site or within the edifice itself were thought responsible for the damage. The church site, however, exhibited no signs of ground movements, and a level survey conducted inside and outside the church distinguished no abnormalities in the floor profile. Relative movements within the church edifice were evidently less than 0.01 foot. The church had undergone no renovations that might have altered floor loadings or changed heating or humidity levels. There appeared to be no structural problems associated with the superstructure of the church. Test pit excavations revealed that the footings were not cracked or otherwise degraded; they were of the dimensions and in the locations required by design; they were at sufficient depth below ground surface to avoid frost heaving. The footings rested upon in place silty shale or on two to three feet of hard to very hard residual silty clay underlain by in-place silty shale. No evidence was found of pyrites or other expansive minerals that might be capable of heaving. The problem was quite perplexing.

During this period, a second, independent investigation was conducted to determine the cause of deterioration and spalling of sections of the exterior brick facing of the church, a problem wholly separate from the building movements and interior wall



A.



B.

FIGURE 5.15 - DAMAGED CHURCH BUILDING
A. EXTERIOR WALL UNCRACKED,
B. INTERIOR WALL WITH CRACKS

cracks that were the focus of our attention. For reasons of safety, the deterioration of the brick facing prompted the removal of the church steeple, a simple structure of minimal weight composed of three steel members arranged in an open pyramidal form.

Quite fortuitously, removal of the steeple revealed that the brick-faced columns that had supported the three legs of the steeple were hollow from top to bottom (contrary to the architects drawings), and that inadequate flashing at the column/steeple juncture permitted rainwater to enter the columns and pass unrestricted to the footings. Localized water stains on the three columns suggested the head of water in the columns may at times have reached 10 or more feet.

Once the steeple was removed and the column tops were capped, the progressive wall cracking stopped. It was speculated but could not be confirmed, that the interior wall cracks had been caused by minor ground settlement associated with compression and readjustment of the rock strata beneath the footings. The church edifice is located approximately 100 feet above an abandoned coal mine that had been backfilled with grout by a reputable contractor under the inspection of another engineer prior to construction of the church. Core borings made prior to church construction revealed that the rock strata were rather weathered and fractured. Some fracturing may have been the result of mining activity. The rock strata between mine level and ground surface were not grouted during mine stabilization.

This case history illustrates several points that had been made earlier: (1) subtle, virtually imperceptible ground movements can cause visible, unsightly damage to structures that may be alarming to occupants, if not to the engineers who investigate them; (2) crack morphology, by itself, may not furnish all necessary information to identify the source of movement; and (3) the nature of the movements may defy positive investigation even with considerable expenditure of time and money, although occasionally enough can be learned about the problem to eliminate the immediate cause--in this case adverse drainage.

Damage Criteria

If, in the design of a structure, it has been determined that the structure will experience ground movements, and if the amount of movement has been estimated, there remains the task of determining the effect of these movements on the integrity of the structure. A slightly different version of this problem faces the engineer who is told that no damages can be tolerated and is requested to identify the maximum allowable ground movement that corresponds to this requirement. The basic problem in these two cases is one of relating ground movements to structural damage. This same problem is also faced by the individual who attempts to determine if observed damage was due to excessive ground movements.

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Traditionally, foundation engineers are told by structural engineers or owners of the facilities whose foundations they are designing that foundation movements must not exceed a certain value. This value is typically rather small by comparison to the magnitudes of movement described in the preceeding pages and often does not exceed an inch. Peck (1965) points out that most structures can tolerate greater settlements (total and differential) than their designers realize or are prepared to admit.

A great deal of research, argument, and discussion over the past 20 years has centered around the questions of: (1) who should place the limitations on movement; and (2) what exactly are realistic allowable movements (Burland, Broms, and deMello, 1977). There are no clear cut answers. Although the development of realistic damage criteria has progressed slowly, owing to the traditional separation between structural engineers and foundation engineers, a great deal of progress has been made in certain areas, as reviewed below.

Fundamental Problem

The problem of relating ground movements to structural damage should theoretically be a straight-forward one to solve. The properties and engineering behavior of the materials from which a building is constructed are ordinarily known. They can be determined in the laboratory, and relationships can be established between stress, strain and failure. If ground movements are known, or can be estimated, then it should be possible to analyze the structure and determine the stresses and strains at every point in the structure. By comparing the computed stresses and strains with the stress-strain-failure behavior determined from laboratory tests on the individual building materials, it should be possible to determine if any part of the building would experience excessively high deformations or excessively high loads. In principle, this same operation could be carried out for every material and member in a building, from the concrete floor slab to the plaster walls.

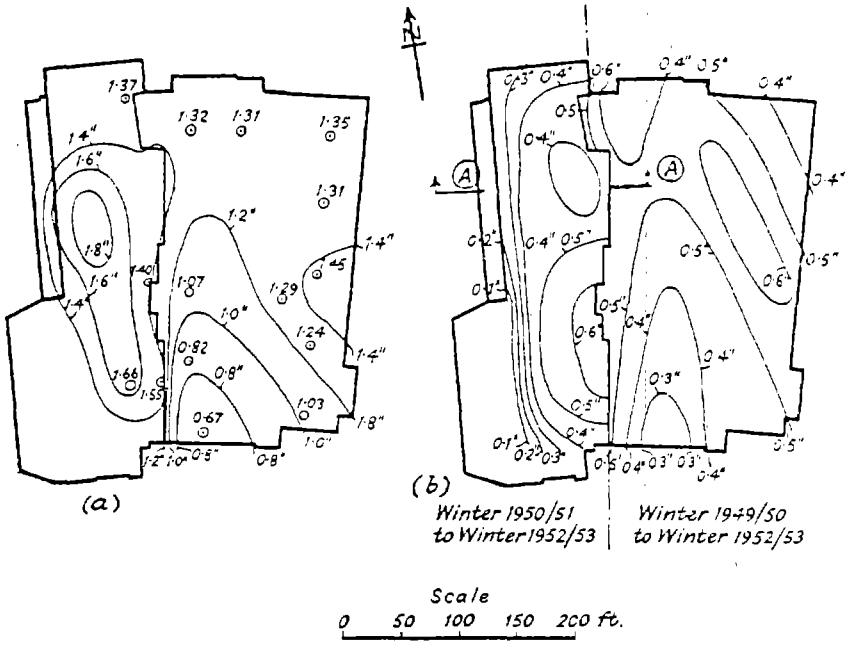
The Winkler (1867) elastic spring foundation was one of the first soil-structure interaction models developed for structures on compressible soil that settle under their own weight. It is still employed on occasion to analyze plates and beams resting on a compliant foundation. Unfortunately, the key parameter, the coefficient of subgrade reaction of the foundation soil, is difficult to determine in practice, and displacements of the subgrade are assumed to occur only beneath the loaded area. Owing to these deficiencies, soil-structure interaction methods have evolved to the use of isotropic elastic half space models (Gorbunov-Possadov, 1940) or layered cross anisotropic half space models (Fraser and Wardle, 1975). Elastic continuum subgrade models are attractive in that the subgrade parameters are rather easily determined and ground displacements adjacent to the loaded area can be computed, as can stresses and strains in the subgrade. Moreover, the layered subgrade system, which incorporates an increasing modulus with depth, leads to a narrower settlement profile, which is more in

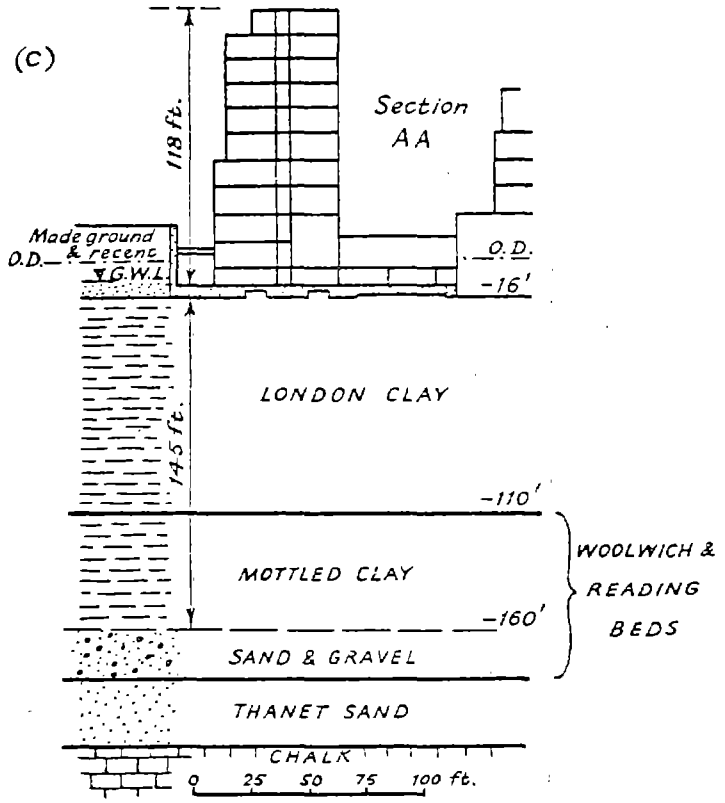
keeping with field measurements than profiles based on isotropic behavior (Gorbunov-Possadov and Davydov, 1973).

A basic soil-structure interaction analysis of a steel frame building settling under its own weight might encompass the following steps, or variations thereof: (1) compute the load delivered to the subgrade (foundations soil) by each footing, assuming no settlement, using conventional structural analysis techniques that account for the stiffness of the steel frame; (2) calculate the foundation settlement resulting from these imposed loads, assuming the structure is perfectly flexible; (3) recompute the footing loads due to these settlements using moment distribution techniques; and (4) recompute the settlements on the basis of the revised footing loads. Repeat steps (3) and (4) until the footing loads and settlements are compatible. Resulting will be estimates of loads and moments for each structural member, which must be evaluated on the basis of permissible stresses and deflections to ascertain the suitability of the design in view of soil conditions at the site.

Meyerhof (1947, 1953) used such an approach to estimate the settlement-induced bending stresses in the structural framework of the Whitehall Gardens Building in London, which is founded on a thick stratum of stiff, fissured London Clay (Figure 5.16). Meyerhof found from his analysis that the measured stresses in the framework varied considerably from those calculated and were generally no more than half the calculated values. He attributed this disparity to the cladding on the building, a factor which is not conveniently incorporated into the analysis. Inasmuch as soil conditions for that study were unusually favorable--a thick deposit of homogenous clay whose properties had been extensively studied--structural designers have tended to turn to empirical methods (Little, 1961) and leave analytical methods to researchers, except in very exceptional cases (such as dynamic building movements associated with an earthquake, or the investigation of innovative structures in areas to be undermined [e.g., Jones and Spencer, 1977; Kawulok, 1977; Klepikov, et al., 1980]). Through empirical methods, engineers have by-passed the soil-structure interaction problem and attempted to relate the magnitude of ground movements or some measure of building distortion directly to a threshold at which damage of a particular type is first observed.

Brown (1979) and Yao (1980) have begun exploring uses of the calculus of fuzzy sets in assessing damage to existing structures, particularly damage resulting from earthquakes. This technique permits subjective, as well as the usual quantitative, factors to be uncorporated into evaluations of the probability of building failure. Linguistic variables, such as 'severely damaged' and 'moderately damaged' serve as classifiers that reflect the gravity and consequences of certain damage characteristics and can be manipulated in a logical manner using matrix methods. The inclusion of subjective factors into damage assessments is claimed by Brown to have yielded estimates of failure probability that are





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FIGURE 5.16 – WHITEHALL GARDENS BUILDING
a. OBSERVED TOTAL SETTLEMENT,
b. OBSERVED DIFFERENTIAL SETTLEMENT,
c. TYPICAL CROSS SECTION (after Meyerhof, 1953)

much closer to observed values than had been predicted from quantitative factors alone. Although the technique shows potential, it is relatively new and has not yet been applied to situations of general interest to the engineering profession. The more conventional, commonly used damage criteria are reviewed in the following pages and deal with: (1) maximum differential and total settlement under surface loads; (2) various measures of building distortion; (3) critical tensile strain; and (4) horizontal strain, tilt and radius of curvature of the ground surface.

Criteria Based on Settlements

General Considerations. Foundation engineers have long been concerned about damage to a structure that might result from settlement under surface loads, usually the self-weight of the structure. Yet, as noted previously, numerous buildings have experienced large vertical ground movements without being imperiled. In Mexico City, for example, many buildings have moved downward 5 to 10 feet, and have remained serviceable and sometimes even undamaged. This is because it is the *differential* vertical movement that causes most damage (Figures 5.17A and 5.17B). It is notable both that subsidence troughs above abandoned mines much more resemble settlement depressions caused by surface loadings than subsidence troughs above operating total extraction mines, and that subsidence troughs above abandoned mines produce the types of damage associated with differential settlement.

Because the behavior of a foundation due to settlement under surface loads is significantly dependent on the nature of the subsurface materials, damage criteria for building settlements have generally been specified on the basis of two major soil types: sand (cohesionless soil) and clay (cohesive soil).

Settlement Criteria for Sandy Soil. Terzaghi and Peck (1948) stated that, "most ordinary structures, such as office buildings, apartment houses, or factories can withstand a differential settlement between adjacent columns of three-quarters of an inch." Their experience is that, even under extreme conditions involving individual footings of different sizes and at different depths, the differential settlement between any two adjacent footings is unlikely to exceed 75 percent of the maximum settlement. (Refer to Figure 5.18 for notation of settlement). On this basis, they recommend that footing pressures be selected so as to limit the largest total settlement to one inch. For buildings on raft foundations, the maximum allowable settlement is two inches, a criterion that reflects the stiffening effect of the raft and building frame.

Similar criteria have been proposed by Skempton and MacDonald (1956) for load-bearing walls and for panels in traditional-type frame buildings--damage avoidance limits of 1.25 inches for greatest differential settlement, 2 inches maximum settlement for isolated foundations and 2 to 3 inches for raft foundations. Burland, Broms and deMello (1977) note the following:



A

HORIZONTAL COMPRESSIONAL AND EXTENSIONAL STRAINS DUE TO SUBSIDENCE CAUSED BY FLUID EXTRACTION OR COAL MINING

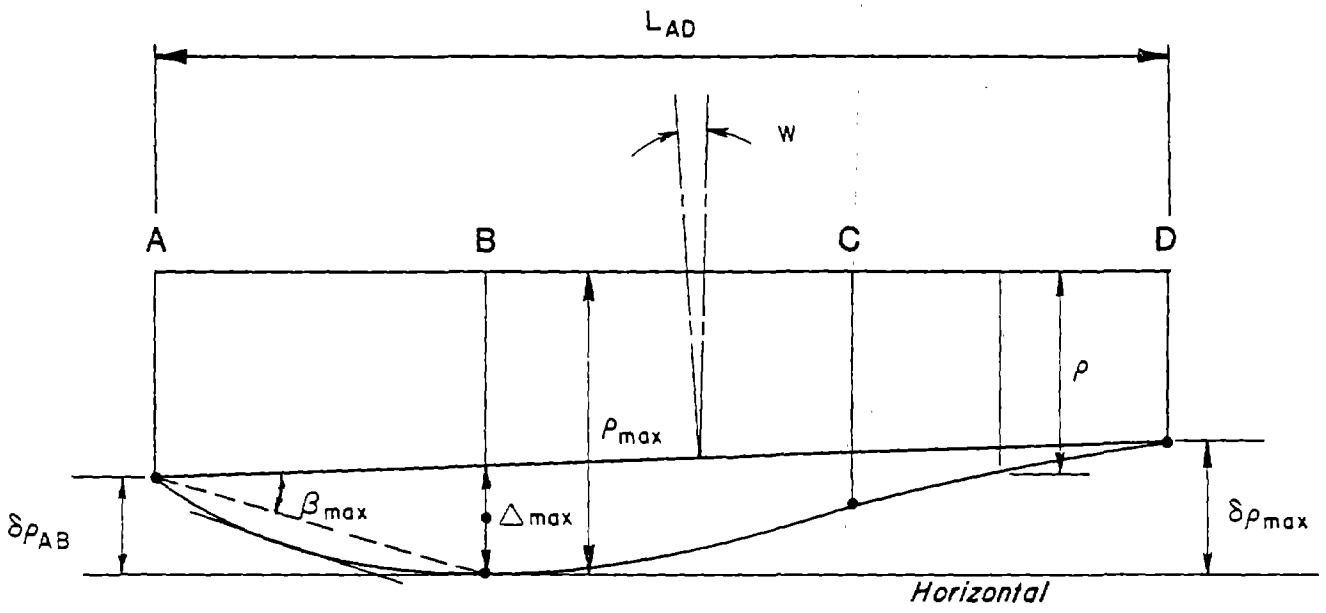


SHARP DISTORTION

B

DIAGONAL TENSILE STRAINS RESULTING FROM DIFFERENTIAL VERTICAL SETTLEMENT UNDER SELF WEIGHT

FIGURE 5.17 - DAMAGING STRAINS IN TWO DIFFERENT CASES OF VERTICAL GROUND MOVEMENTS.



ρ	TOTAL SETTLEMENT
ρ_{max}	MAXIMUM (TOTAL) SETTLEMENT
$\delta\rho$	DIFFERENTIAL SETTLEMENT
$\delta\rho_{max}$	MAXIMUM OR GREATEST DIFFERENTIAL SETTLEMENT
$\delta\rho_{AB}$	DIFFERENTIAL SETTLEMENT BETWEEN POINTS A AND B
β	ANGULAR DISTORTION OR RELATIVE ROTATION, ROTATION OF LINE JOINING TWO REFERENCE POINTS RELATIVE TO THE TILT
Δ	RELATIVE DEFLECTION (RELATIVE SAG OR RELATIVE HOG)
$\frac{\Delta}{L}$	DEFLECTION RATIO (SAGGING OR HOGGING RATIO)
$\left(\frac{\Delta}{L}\right)_{max} = \frac{\Delta_{max}}{L_{AD}}$	
w	TILT

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(Definitions as per Burland, Broms and deMello, 1977)

FIGURE 5.18 - NOTATION FOR SETTLEMENT AND DISTORTION PARAMETERS

1. The foregoing criteria may be conservative for buildings in which the cladding or finish is placed after most of the load is applied, and thus after most of the settlement has occurred. This degree of conservatism is amplified by Terzaghi and Peck's statement that normally the differential settlement between adjacent footings is much less than 75 percent of the maximum settlement.

2. In the major reviews of settlements of buildings founded on sand, no cases of building damage have been reported.

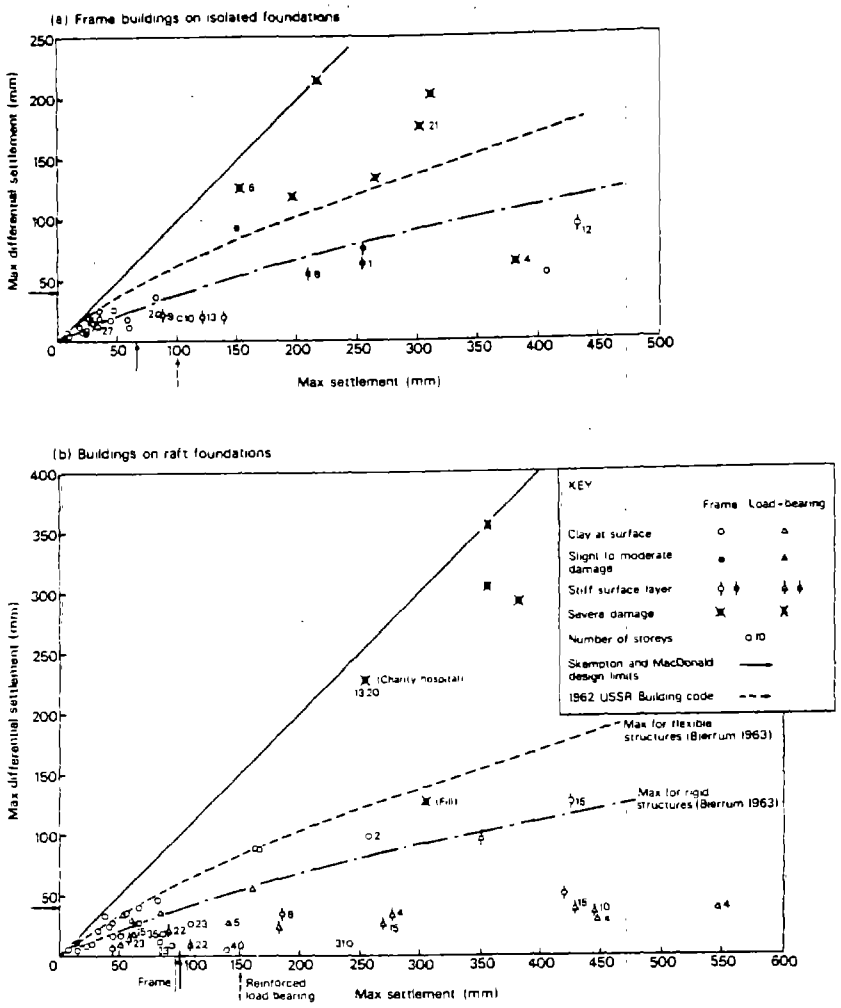
3. Most reported cases of settlement on sand do not exceed 40 millimeters (about 1.5 inches).

Settlement Criteria for Clayey Soils. Skempton and MacDonald (1956) suggest that to avoid building damage the greatest differential settlement of foundations on clay should not exceed 1.75 inches. The recommended design limits for maximum settlements are 3 inches for isolated footings and 3 to 5 inches for rafts. Burland, Broms and deMello (1977) have collected the settlement data shown in Figure 5.19 for buildings founded on uniform deposits of clayey soils, some of which include a stiff upper layer overlying a deeper more compressible stratum. Separate plots of maximum differential settlement versus total settlement are given for frame buildings on isolated foundations and for buildings on raft foundations. Open symbols represent cases of no damage; solid symbols represent cases of slight to moderate damage; and solid symbols with crosses represent cases of severe damage. The Skempton/MacDonald limits are shown as arrows on the axes, together with limits of maximum total settlement suggested by the 1962 USSR building code. The maximum tolerable differential settlement curves suggested by Bjerrum (1963) for flexible and rigid structures are also shown. The figures illustrate the following:

1. A number of undamaged buildings, particularly those founded on rafts, have experienced maximum differential settlements greater than the limits suggested by Skempton and MacDonald and the USSR building code. If differential settlement is a valid criterion for damage, the limits are certainly higher than those suggested by a single value approach.

2. There appears to be no relationship whatsoever between damage and total or maximum settlement, a fact that was clear from earlier discussions.

Conclusions Concerning Settlement Criteria. The suggested criteria based on limiting differential and total settlement commonly provide safe and conservative guidelines for foundation design when movements are due to surface loads. Although settlement appears to be an important factor in determining damage, other factors also contribute. Aside from the fact that total settlement may jeopardize services entering the building or the serviceability of adjacent structures, it appears no more important when resulting from



5.19 - SETTLEMENTS OF BUILDINGS ON CLAYEY SOILS.
A. FRAMED BUILDINGS ON ISOLATED FOUNDATIONS:
B. BUILDINGS ON RAFT FOUNDATIONS.
OPEN SYMBOLS REPRESENT CASES OF NO DAMAGE SOLID SYMBOLS SLIGHT TO MODERATE DAMAGE SOLID SYMBOLS WITH CROSSES SEVERE DAMAGE
(after Burland Broms and deMello 1977).

surface loads than when resulting from mining or fluid extraction subsidence.

Criteria Based on Building Distortions

Terminology. Most foundation engineers have recognized the importance of differential foundation settlements in causing building distortions and damage. Yet, the amount of distortion produced by a given differential settlement between two adjacent isolated footings or between two points on a raft or continuous footing clearly depends on the distance between the two points, as well as on the differential settlement itself. Accordingly, a great deal of attention has been focused on a parameter known as the angular distortion or relative rotation, defined by the symbol β, in Figure 5.18. Burland, Broms and deMello (1977) prefer the term 'relative rotation', because the term 'angular distortion' implies shear distortion within a structure, which they say may prevail in frame buildings but not necessarily in all structures.

The definitions of terms in Figure 5.18 are based on the known settlement of a number of discrete points on the foundation of a structure. These foundation movements are not the same as ground movement except in the special case of an extremely flexible foundation; and even in that case, the foundation movement may be substantially different than ground movement if a break or discontinuity occurs in the ground, as in a landslide or sinkhole collapse. It should be apparent that the distorted foundation shown in Figure 5.18 could be caused by any type of ground movement--translational, rotational, distortional or volumetric.

Building Distortion Criteria. Burland and Wroth (1974); Burland, Broms and deMello (1977); and Geddes (1977a) have reviewed various distortion criteria suggested by Meyerhof (1953); Skempton and MacDonald (1956); Polshin and Tokar (1957); and Grant, Christian and Vanmarcke (1974). A summary of the criteria is given in Table 5.3. The suggested norm is that given in Figure 5.18 after Burland, Broms and deMello, who have also interpreted the criteria. Burland and Wroth point out that the term 'angular distortion' has been used to describe at least four different modes of deformation. When building tilt does not occur and a smooth profile cannot be drawn between reference points (which is the usual case), the angular distortion simply becomes the differential settlement between two points divided by the distance between the two points.

Burland, Broms and deMello (1977); Peck, Deere and Capacete (1956); Terzaghi (1956) and others have pointed out a number of weaknesses in the foregoing criteria, particularly in the work of Skempton and MacDonald, which is regarded as a milestone in the development of the subject.

The principal weakness lies in the nonuniformity and imprecision with which the degree and extent of damage has been described by the sources from which the data were drawn. Skempton-MacDonald and Grant, Christian, and Vanmarcke (1974) used the broad damage

Table 5.3

SUMMARY OF DAMAGE CRITERIA BASED ON BUILDING DISTORTIONS - DERIVED FROM CIVIL ENGINEERING EXPERIENCE

Author	Suggested Limiting Criterion	Notes
Meyeroff (1953)[1,2]	$\beta_{max} = 1/300$[3]	Differential settlement between adjacent columns of open frames of encased steel and reinforced concrete. At cracking.
		Suggested for Design
	$\beta = 1/1000$	Infilled frames
	$\frac{\Delta}{L} = 1/200$[4]	Load bearing walls or continuous brick cladding
Skempton and MacDonald (1956)	$\beta = 1/150$	Structural damage to stanchions and beams occurs.
	$\beta = 1/300$	Limit of damage to loadbearing walls and panels in frame buildings.
	$\beta = 1/500$	Suggested design limit to avoid cracking.
	$\beta = 1/1000$	Limit to avoid any settlement damage.
Polshin and Tokar (1957)		To avoid damage to:
	$\beta = 1/200$	Unfilled or unclad steel and concrete frames.
	$\beta = 1/500$	Infilled steel and concrete frame structures.
		To limit damage in load-bearing brick walls to very slight:
	$\frac{\Delta}{L} = 1/3300$	L/H <3 sand[5]
	$\frac{\Delta}{L} = 1/2500$	L/H <3 soft clay
	$\frac{\Delta}{L} = 1/2000$	L/H <5 sand
	$\frac{\Delta}{L} = 1/4000$	L/H <5 soft clay

Table 5.3
(continued)

Author	Suggested Limiting Criterion	Notes
Grant, Christian, and Vanmarcke (1974)	β = 1/300	Damage limit for load-bearing walls and frame buildings.
Burland and Wroth (1974)	Critical Tensile Strain (ε = 0.00075% for brick-work)	Criteria can be applied to any building materials subjected to any mode of distortion.

[1]Referenced in Burland, Broms and de Mello (1977).

[2]Referenced in Geddes (1978).

[3]β = Angular Distortion = Rotation of line joining two reference points, relative to tilt.

[4]Δ = Relative Deflection between two points separated by distance L.

[5]L = Building Length, H = Building Height.

classifications of 'architectural', 'functional', and 'structural'. Furthermore, despite the almost 200 case histories, the data for load-bearing walls are limited. Burland, Broms and deMello suggest that the damage limit of $\beta = 1/300$ be used with caution in application to such structures. In addition, the quoted values of relative rotation, or angular distortion are total values and not necessarily those which occurred subsequent to construction of the infilling or application of the finishes. Because some settlement and relative rotation probably occurs prior to placement of cladding and finishes, the critical value of β may be significantly lower than the reported or total value on which the criteria are based. This is not a problem with load bearing walls. Finally, the limiting values of relative rotation for structural damage to open frame buildings have been derived from "average" buildings in which structural members are of common materials and dimensions. The criteria do not necessarily apply to buildings having exceptionally large or stiff structural members.

A few special cases of damage criteria related to building distortion have been derived for specific causes of ground movement. O'Rourke, Cording and Boscardin (1977) and Boscardin (1980) propose the criteria shown in Table 5.4 and Figure 5.20 for damage related to building distortion for brick-bearing wall structures located within 10 to 15 meters of shallow tunnels and open cuts. The angular and lateral distortions are assumed to be of approximately the same magnitude.

Conclusions Concerning Building Distortion Criteria. Criteria based on building distortions may provide useful guidelines for routine design as do the criteria based on limiting settlements, but as Burland, Broms and deMello (1977) point out, these criteria are essentially empirical and offer little insight into the real cause of damage. Furthermore, available criteria do not account for all types of building deformation. For example, the buildings shown in Figure 5.17A are obviously not subjected to angular distortion yet may suffer considerable damage due to compressional or extensional horizontal strains. Thus, a more general basis for relating building deformation to damage would be preferable.

Criteria Based on Critical Tensile Strain

General Considerations. Burland and Wroth (1974) have suggested an approach to building damage criteria that avoids some of the shortcomings of several of the criteria reviewed in the previous pages. In particular, they examine the fundamental cause of damage in a structure, the modes of deformation that are likely to cause damage, and the effect of building shape on damage.

Basic Concept. In view of the fact that most building damage due to movements manifests itself as tensile cracking, Burland and Wroth (1974) have proposed the concept of a limiting or critical tensile strain as a criterion for the onset of visible damage to building cladding and finishes. The concept is based on the work of British researchers, who have conducted laboratory experiments

Table 5.4

DAMAGE RELATE TO BUILDING DISTORTION
FOR BRICK-BEARING WALL STRUCTURES

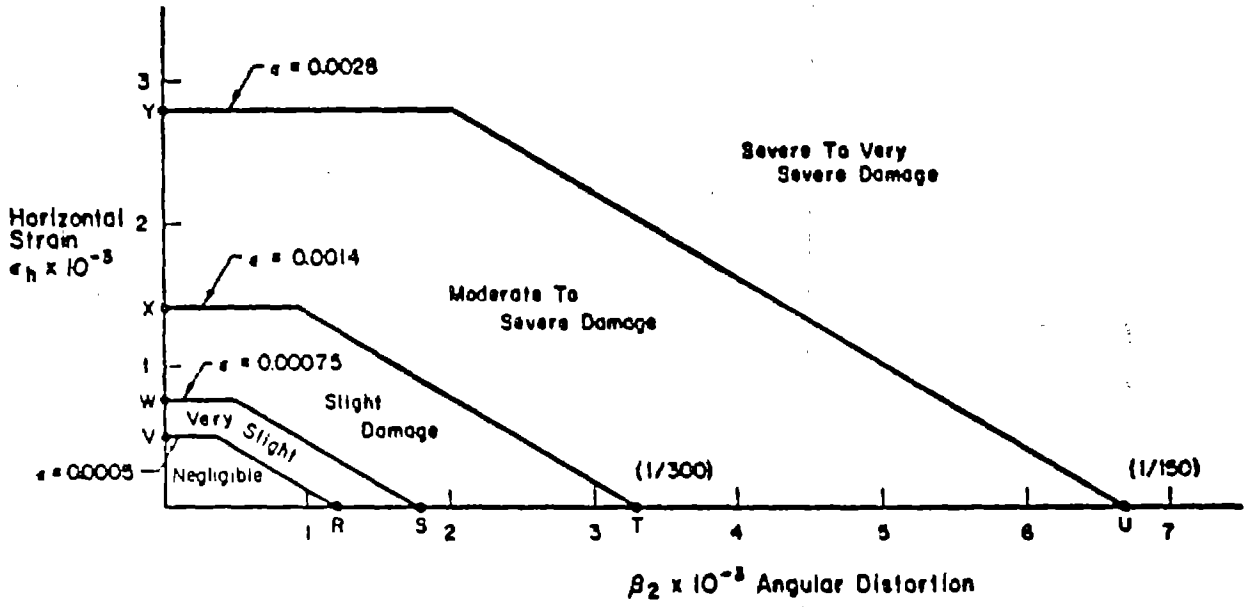
Description of Damage	*Angular, δ_V, and lateral, δ_H, distortion at the ground surface
Threshold of architectural damage	1.0×10^{-3}
Architectural damage, sticking doors; may be conspicuous concentrations of cracks, cracks and separations as large as 0.3 to 0.6 cm wide	1.0×1.0^{-3} to 3.0×10^{-3}
Damage is an inconvenience to building occupants; jammed doors and windows; broken window panes; building services may be restricted. Cracks and separations may be as large as 1.5 to 5.0 cm wide	3.0×10^{-3} to 7.0×10^{-3}
Spalling of stone cladding and possible collapse of cornices along the facade wall (differential movements parallel to brick-bearing walls)	7.0×10^{-3} to 8.0×10^{-3}

*Note:

1) Angular and lateral distortion are assumed to be approximately equal.

2) Criterial derived from ground Movements adjacent to braced cuts and tunnels.

3) After O'Rourke, Cording, and Boscardin, 1977.



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FIGURE 5.20 - DAMAGE CRITERIA FOR BEARING WALL STRUCTURES (Boscardin, 1980)

on the cracking of brick walls, and on the work of Polshin and Tokar (1957) in the USSR. Results of these studies, as well as British standards for permissible shrinkage and expansion strains in brick and concrete products, suggest that the onset of visible damage in these materials occurs at tensile strains ranging from 0.05 to 0.10 percent. The lower end of this range agrees with findings from studies of reinforced concrete beams that show visible cracking during bending at tensile strains in the range of 0.035 to 0.05 percent.

Burland and Wroth point out that the critical tensile strain for the onset of visible cracking bears no relationship to the usually much smaller strain at which loss of tensile strength occurs. Critical tensile strain is essentially a measure of serviceability related to visible cracking, although the occurrence of visible cracks in a structure does not imply failure of the structure or even necessarily a constraint on serviceability.

The critical tensile strain concept is attractive in that it can be applied to any form of building distortion caused by horizontal ground strains, differential settlements, or other forms of ground movement. Like the building distortion criteria discussed above, the critical tensile strain concept does not make any assumptions about soil-structure interaction and, by itself, does not relate ground movements to damage.

Application of Critical Tensile Strain Concept. Burland and Wroth (1974) have demonstrated the concept of critical tensile strain in the context of bending of a simple beam. As shown in Figure 5.8A, the beam may experience cracking due to diagonal tensile strains (shear distortion) or direct tensile strains. Appealing to the theory of elasticity, they develop relationships between the L/H ratio (length/height) of the beam and the ratio Δ/L $(1/\varepsilon crit)$, where Δ/L is the deflection ratio (Figure 5.18) and $\varepsilon crit$ is the critical tensile strain of the material out of which the beam is fabricated. Relationships are obtained for various values of the ratio of elastic modulus to shear modulus E/G for different modes of bending (or positions of the beam's neutral axis--the longitudinal line along which the bending stress is zero).

Depending on the value of E/G and the position of the neutral axis, either the diagonal strain or the bending strain may first reach the critical level. Burland and Wroth test the theoretical relationships against settlement case histories for three different conditions:

1. Sagging load-bearing walls, with E/G = 2.5 and the neutral axis at midheight, corresponding to an isotropic beam;

2. Sagging frame buildings, with E/G = 12.5 (low shear stiffness) and neutral axis at midheight; and

3. Hogging load-bearing walls, with E/G = 0.5 (low bending stiffness) and neutral axis at bottom.

By assuming the critical tensile strain in all cases is 0.075 percent (the average value for brickwork and blockwork set in cement mortar), the curves shown in Figure 5.21 are obtained. Theoretically, if the deflection ratio (Δ/L) lies above the curves labeled 1, 2, and 3 for any given L/H ratio, visible cracking should occur, provided the buildings act as simple beams. Case histories for which deflection ratios (Δ/L), L/H ratios and degree of damage are known have been plotted in the figure. Also shown are the relative rotation and deflection ratio criteria proposed by Skempton/MacDonald and Polshin and Tokar, as well as some of these investigators' original data and more recent data from papers presented at the 1974 British Geotechnical Conference on Settlement of Structures.

The following conclusions can be drawn from Figure 5.21.

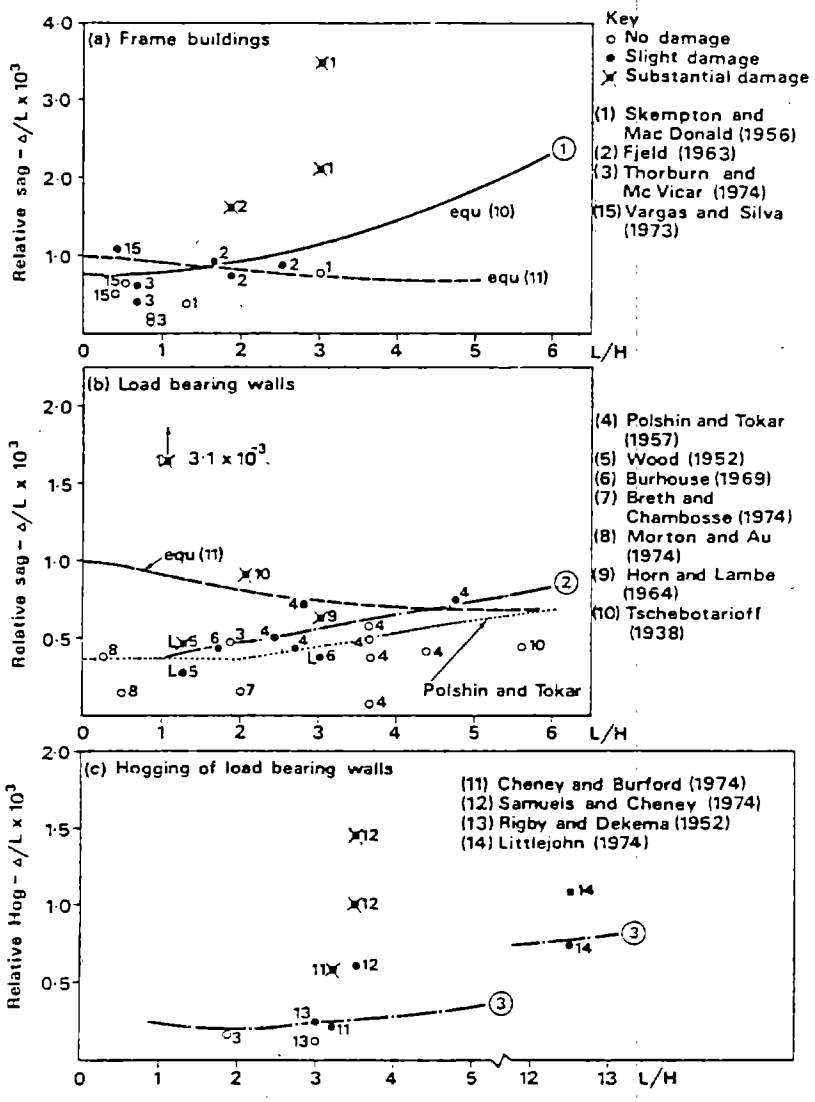
1. Despite the simple assumption that buildings behave like beams, the analysis appears to reflect the general behavior of actual structures. Most cases of severe damage (black points with crosses) lie above the full curves; most cases of no damage (open points) lie below; and most cases of slight damage (solid points) cluster around the curves.

2. The angular distortion cracking limit of β = 1/300 proposed by Skempton and MacDonald appears to be conservative for frame buildings. The criterion proposed by Polshin and Tokar appears more satisfactory for load-bearing walls. The limiting sagging deflection ratio for load-bearing walls is obviously much lower than for sagging frame buildings. This is substantiated by a number of case histories in which slight to severe damage occurred at angular distortions less than 1/300.

3. Cracking in the hogging mode is known to occur at deflection ratios approximately one-half those that cause cracking in sagging walls, Figure 5.9. This finding is particularly significant in light of the many forms of ground movement that may result in hogging (including subsidence) and is substantiated by numerous observations of damaged load-bearing walls subjected to the hogging modes of distortion.

Conclusions Regarding Application of Critical Tensile Strain Concept. The simple beam analysis described above demonstrates that the critical tensile strain concept holds promise as a fundamental damage criterion. The critical tensile strain concept is attractive for the following reasons:

1. Many structures have rather simple geometries that are amenable to this type of characterization. Structures of



NOTE: CRITICAL TENSILE STRAIN TAKEN AS 0.075 PERCENT

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FIGURE 5.21 - RELATIONSHIPS BETWEEN L/H AND DEFLECTION RATIO (△/L) FOR VARIOUS DEGREES OF DAMAGE. A. FRAME BUILDINGS: B. SAGGING OF LOAD -BEARING WALLS. (after Burland and Wroth, 1974).

different sizes, and more complex shapes and types of construction can be dealt with, provided that a stress analysis is performed.

2. Different modes of building deformation can be accommodated. Deflections producing critical tensile strains are easily derived for a simple elastic beam and can take into account three-dimensional effects, such as racking.

3. A variety of construction materials can be incorporated into the analysis. It is not necessary to use the same value of critical tensile strain for all structures. Several materials can be incorporated into more sophisticated analyses.

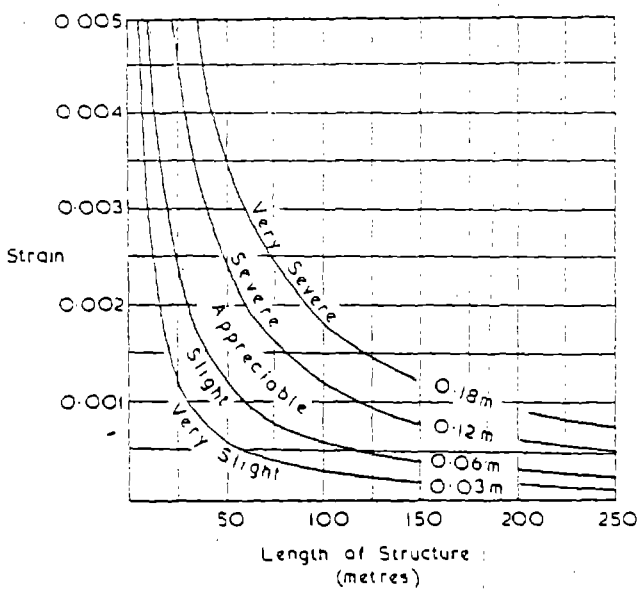
4. Finally, the critical tensile strain concept has applications to mine subsidence situations above both active and abandoned mines as an indicator of the onset of slight damage. Naturally, it must be augmented by other damage criteria at higher levels of damage. Three somewhat different classification systems developed specifically for mine subsidence applications are described below.

Criteria Based on Ground Strain, Tilt and Curvature

Along with vertical and horizontal displacement components, mine subsidence produces four types of differential movements that might adversely affect surface structures. They are direct strain ("ground strain") ε, shear strain γ, tilt I and curvature K (Salamon, 1977). These parameters vary in magnitude directionally and areally. Sometimes, where the ground surface profiles are very broad and shallow relative to structures at ground surface--and where little or no curvature or bending deformation is imposed on structures, damage to structures is considered to be primarily the result of the horizontal extensional or compressional strains at the ground surface.

The United Kingdom National Coal Board (NCB) has developed, through a synthesis of numerous case records, a relationship between degree of damage, horizontal ground strain and length of structure (NCB, 1975), which is shown in Figure 5.22. The Coal Board's damage classification is shown in Table 5.5. Most cases from which these criteria were developed are claimed to be conventional, brick or stone structures without basements, ranging in length from 50 to 450 feet, in which damage was due primarily to horizontal strain. Damage consisted of cracks and deformation of brickwork and claddings, with partial loss of beam bearing length in some structures. Continuous framed structures were not included (Geddes, 1977c).

Geddes (1977a and 1977b) points out an underlying fallacy in the relationship. The curves of constant length change in Figure 5.22 imply that the change in ground length due to subsidence is equal to the change in length of the structure, a condition which will not be realized if slip occurs between the



COMPARE WITH TABLE 5.5

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FIGURE 5.22 - U.K. NATIONAL COAL BOARD DAMAGE CRITERIA FOR BUILDINGS SUBJECT TO MINING SUBSIDENCE. HORIZONTAL GROUND STRAIN IS BASIS FOR CRITERIA. (After NCB, 1975)

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

TABLE 5.5 - U.K. National Coal Board Classification for Mining Subsidence Damage (after NCB, 1975)

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ground and a building's foundation during the extensional and compressional phases of ground movement. This has been discussed in Section 4. Geddes believes the NCB relationships are of limited use in assessing individual damage cases. Nonetheless, the curves in Figure 5.22 serve as a rough, pragmatic correlation between level of damage and ground strain for conventional structures in the United Kingdom.

It is worth noting that both Ryncarz (1963) and Zhuo Guoquan (1982) have reported differences between ground movements due to subsidence and building movements. Ryncarz suggests on the basis of model tests on sands that building weight, its stiffness and its position within the subsidence trough all influence the degree of tilt a building will assume. Zhuo Guoquan presents field observation-based empirical relationships between ground surface curvature, tilt and strain and corresponding deformations of one story homes, as well as mixed story brick, stone and reinforced concrete buildings, in mainland China. As might be expected, the relationships vary depending on whether the ground strain is tensile or compressive, the ground slope positive or negative, and the ground curvature concave up or down.

In the Netherlands, structural tilt resulting from mine subsidence is used to establish the permanent loss of value of a structure (Pöttgens, 1979). The mining company must reimburse the owner for this loss as well as for direct damage. A tilt of 0.025 is considered to reduce a structure's market value by about 10 percent; a tilt of 0.050 by 30 percent. In Germany, a similar criterion indicates a reduction in market value of 20 percent for a tilt of 0.025 and 75 percent for a tilt of 0.050. Tilt is measured across the building floor on a corner to corner diagonal.

Subsidence damage in the Peoples Republic of China is categorized in much the same manner as in the United Kingdom. However, homes (Table 5.6) and larger buildings (Table 5.7) each have their own classification. Homes represented by Table 5.6 are one floor (20 x 40 feet in plan), wood frame, tile roofed structures with brick walls, stone/mortar foundations and no basements or crawl spaces. The larger structures represented by Table 5.7 are two or three story brick bearing wall structures with reinforced concrete floor slabs and stone/mortar foundations. Ground surface deformations associated with each damage class are specified in terms of slope, curvature and strain. Those for homes are somewhat different from those for larger buildings.

The Central Mining Institute, Katowice, Poland, has taken a somewhat different tack concerning damage prediction. The Institute has devised a classification system for evaluating the potential susceptibility to subsidence damage of existing structures above active mines in the Upper Silesian Coal Field based on specific attributes of the structure and the ground movements it will experience (Skinderowicz, 1978). The classification system was developed in response to a post-World War II energy demand in

Table 5.6

SUBSIDENCE DAMAGE CLASSIFICATION SYSTEM FOR HOMES
PEOPLES REPUBLIC OF CHINA

Class of Damage	Description of Structure Damage	Nominal Wall Crack Width, mm	Deformation of Ground Surface			Degree of Repair
			Slope (mm/m)	Curvature (10^{-3}/m)	Strain (mm/m)	
I	Several slight cracks show on the walls.	<4	<3	<0.2	<2	No Repair
II	Cracks show on the walls; doors and windows crooked, wall cladding falling off; bearing of beams deformed slightly.	4-10	3-8	0.2-0.5	2-4	Slight Repair
III	Cracks show on the walls. Deformation of doors and windows is serious; walls slope; roof beam ends beginning to draw out; floor slopes noticeably and exhibits cracks.	10-20	8-14	0.5-1.0	4-6	Medium Repair
IV	Horizontal and shear cracks show on the walls. Walls are far out of plumb, distorted and partially damaged. Roof beam ends drawn out noticeably. Roof raised. Walls partially damaged due to pressure.	>20	>14	>1.0	>6	Heavy

NOTE:

1. The criteria are based on subsidence measurement data collected from Zaozhuang, Fengfeng and Fuxin Coal Mining Administration in China. All the buildings are one story tile-roofed houses with brick walls, wooden frames, random-sized stone and mortar foundations. Zhu Jiayu (personal communication) notes that the houses are generally about 20 x 40 feet in plan and have no basements or crawl spaces. Interior walls are often classed with lime/sand/fiber plaster 0.5 inch thick. Mines are commonly 200-300 meters below ground surface. Many are longwall, some are room and pillar.

2. Data from Zhuo Guoquan et al. (1982). Table translated by Zhu Jiayu.

Table 5.7

SUBSIDENCE DAMAGE CLASSIFICATION SYSTEM FOR LARGE BUILDINGS
PEOPLES REPUBLIC OF CHINA

Class of Damage	Description of Structure Damage	Nominal Crack Width, mm	Deformation of Ground Surface: Slope (mm/m)	Deformation of Ground Surface: Curvature (10^{-3}/m)	Deformation of Ground Surface: Strain (mm/m)	Degree of Repair
I	Several cracks (A) show on the first floor walls below and above windows; very few raks (B) show on the walls between windows; cracks (C) show on ceilings.	A. 1-2 B. <4 C. <2	<4	<0.5	<2	No Repair
II	Doors and windows stick; cracks (A) show on several walls between windows and longitudinal partition walls; cracks (B) show on floors.	A. 4-6 B. <10	4-6	0.5-1.0	2-3	Slight Repair
III	Cracks (A) run through most of the first floor walls between windows; cracks (B) show on the longitudinal partition walls; windows and doors distorted; cross walls slope seriously.	A. >10-15 B. >10	6-10	1.0-2.0	3-6	Medium Repair
IV	Cracks (A) show on the first and second floor walls between windows; longitudinal and cross walls slope seriously; noticeable horizontal shearing in walls.	A. >10	>10	>2.0	>6	Heavy Repair

NOTE:

1. The criteria are based on subsidence measurement data collected from Benxi Coal Mining Administration in China. All buildings are two or three floor buildings with brick walls, reinforced concrete floorslabs, and random-sized stone and mortar foundations.

2. Data from Zhuo Guoquan et al. (1982). Table translated by Zhu Jiayu.

Poland that necessitated mining within the reserves of coal ("safety pillars") that had been left to protect towns, industrial plants, power stations and other structures sensitive to ground movements. Extraction from safety pillars represents about 40 percent of Poland's total coal production.

Figure 5.23 presents the Polish classification system in concise form. The damage susceptibility of a structure is assessed by tallying the points from each of seven categories that characterize the geometry and structural attributes of the structure. The resistance class to which the structure is assigned carries with it allowable limits of ground surface tilt, horizontal strain and radius of curvature, which if exceeded, will result in damage to the structure. It will be noted that the classification system does not specify the probable damage severity in the event the allowable ground deformation limits are exceeded.

In practical application, the assignment of point scores is the responsibility of personnel in Poland whose judgment is tempered with years of experience under the conditions for which the system was developed. The system is not directly transferrable to U.S. conditions. Yet it is noteworthy, in that it attempts to account for the geometry of the building, type of construction, type of foundation and subgrade elements, soil type, building conditions and structural safeguards incorporated to avert subsidence damage.

Figure 5.24 presents a somewhat similar, though less detailed, system developed for predicting the susceptibility to damage of buildings located above active mines of the Donets Coalfield in the U.S.S.R. The number of stories and height index the building to a protection category, with which are associated values of ground surface tilt, horizontal ground strain and radius of curvature that must not be exceeded to avoid building damage. The Coefficient of Safety, defined in Figure 5.24, identifies the minimum depth to the mine that is necessary to achieve non-damaging ground movements for each Building Protection Category. In a refinement of allowable deformation criteria for buildings to be undermined in the Donets Basin, Belyaev (1968) has coupled strain, curvature and building height and length in a single expression that bears some resemblance to the previously cited expressions of Burland and Wroth (1975):

$L(\varepsilon + H/\rho) \leq 100$ mm: brick bearing wall buildings

≤ 325 mm: single story frame buildings

Figure 5.25 compares several damage classifications in terms of allowable ground strain, tilt and radius of curvature. As discussed above, the Chinese classifications indicate the increase in damage severity to be expected for a given building type with increase in ground surface deformation. This is similar to the NCB classification, which, however, is based solely on ground strain. The Polish and Soviet classifications, in contrast, indicate the limiting

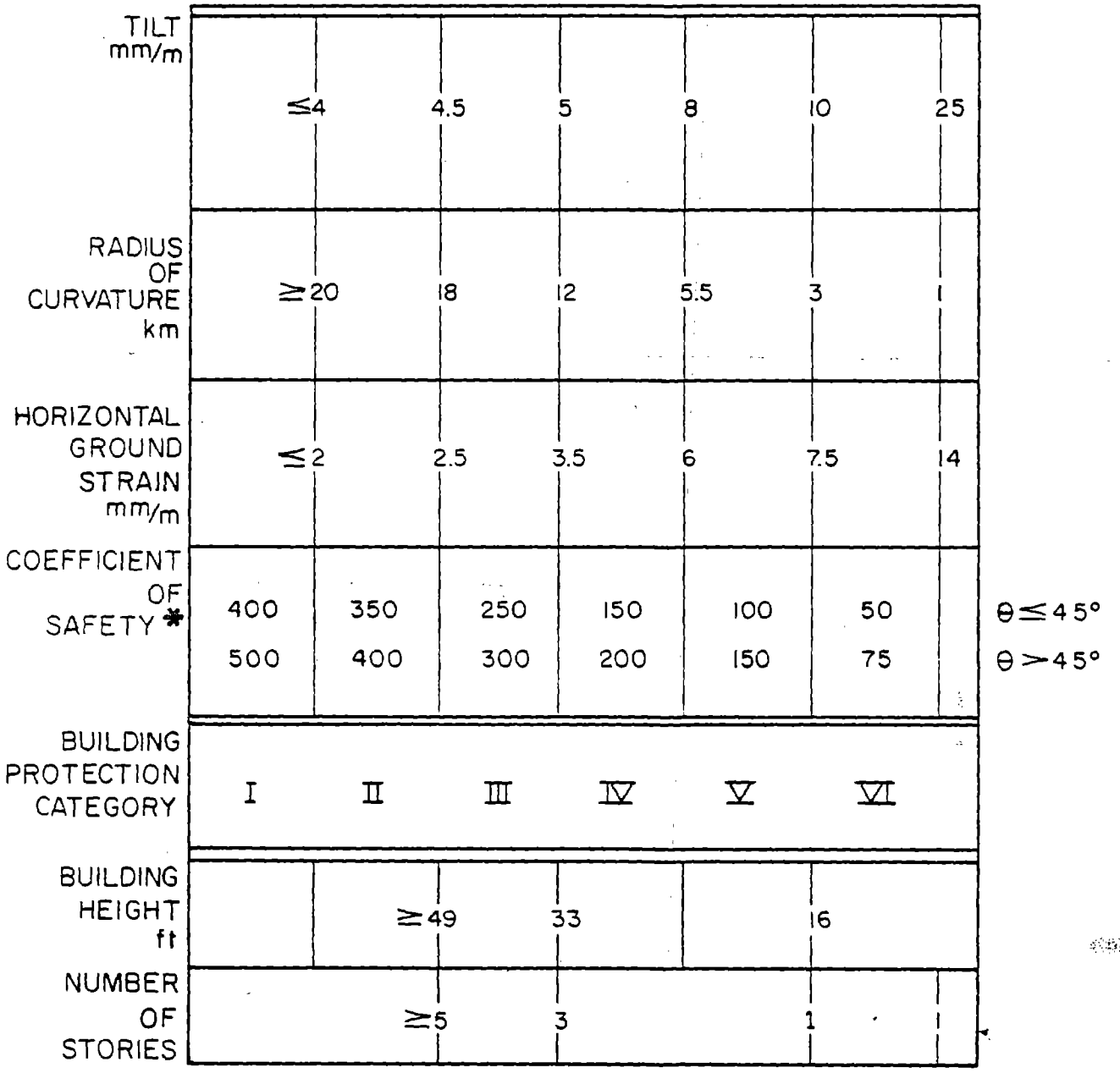
		POINTS
1. LENGTH OF STRUCTURE		
	≤10m	4
	11-15m	7
	16-20m	11
	21-25m	16
	26-30m	22
	31-35m	29
	36-40m	37
	≥40m	42
2. STRUCTURE SHAPE		
	Simple, compact	0
	Somewhat articulated	3
	Substantially articulated	6
	Simple, extensive	6
	Articulated, extensive	8
3. STRUCTURE FOUNDATION		
	Single level w/ or w/o cellars	0
	Multiple level	3
	Multiple level with cellars	6
	Multiple level with garage	8
4. SUBSOILS		
	Compressible	0
	Somewhat compressible	4
	Incompressible	12
5. STRUCTURE CONSTRUCTION		
	Rigid	0
	Somewhat rigid	4
	Flexible	8
6. SUBSIDENCE PROTECTION		
	Full protection	0
	Partial protection	4
	No protection	
7. CONDITION OF STRUCTURE		
	Good	0
	Medium	6
	Poor	12
	TOTAL POINTS____	

(Data from Skinderowicz, 1978)

PERMISSIBLE GROUND SURFACE DEFORMATION AS PER BUILDING RESISTANCE CLASS

TILT mm/m	0	2-1/2	5	10	15	
HORIZONTAL GROUND STRAIN mm/m	0	1-1/2	3	6	9	
RADIUS OF CURVATURE km	∞	20	12	6	4	
RESISTANCE CLASS	0	1	2	3	4	5
TOTAL POINTS	≥48	37 - 47	28 - 36	21 - 27	≤20	<<20

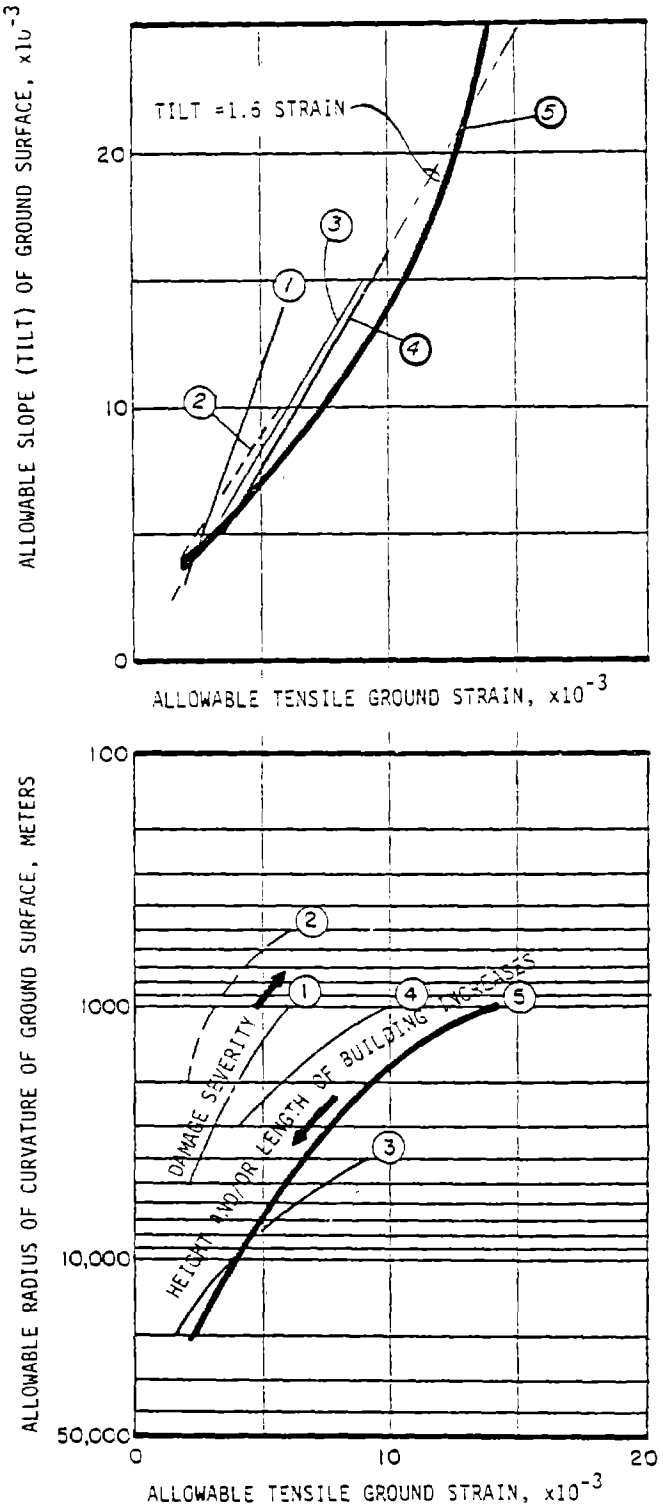
FIGURE 5.23 - POLISH CENTRAL MINING INSTITUTE CLASSIFICATION - SUSCEPTIBILITY OF BUILDINGS TO SUBSIDENCE DAMAGE



* SAFE DEPTH/MINED THICKNESS OF COAL, WHERE SAFE DEPTH IS DEFINED AS MINIMUM OVERBURDEN THICKNESS BELOW WHICH COAL SEAM OF SPECIFIED THICKNESS CAN BE MINED WITHOUT EXCEEDING GROUND SURFACE DEFORMATION LIMITS.

(Data from VNIMI, 1958)

FIGURE 5.24 - SOVIET CLASSIFICATION - SUSCEPTIBILITY OF BUILDINGS TO SUBSIDENCE DAMAGE, DONETS COALFIELD, USSR.



CURVES

1 ZAOAHUANG, FENGFENG AND FUXIN COAL MINING ADMIN. PEOPLES REPUBLIC OF CHINA

2 BENXI COAL MINING ADMIN. PEOPLES REPUBLIC OF CHINA

3 UPPER SILISIAN COALFIELD, POLAND

4 KARAGANDA DISTRICT, USSR

5 DONETS DISTRICT USSR

NOTE:

1. Curves 1 & 2: Damage Severity increases with increase in allowable deformation. Building type and size held constant. refer to tables 5,6 & 7.

2. Curves 3-5: Allowable deformation decreases as building height and/or length increases. Damage becomes "Appreciable" if allowable deformation is exceeded. refer to figures 5.23 and 5.24 for details; as well as Brauner (1973).

FIGURE 5.25 - COMPARISON OF SEVERAL ALLOWABLE DEFORMATION CRITERIA FOR MINE SUBSIDENCE

ground surface deformations which, if exceeded, would result in 'appreciable' damage to buildings of certain sizes and types. It will be noted for both classification types that the limiting value of ground tilt has customarily been taken as about 1.6 times the limiting value of ground strain.

Conclusions

The practical obstacles to conducting formal soil-structure interaction analyses to predict building damage under a specified set of ground movements are two-fold:

1. The technical problem of analyzing the interaction between ground movement and structures, which has not been reduced to a routine procedure in any but the most elementary cases; and

2. The expense of analyzing even such ordinary structures as houses, apartments, and conventional business and industrial buildings, if the analysis is refined to account for cladding, and other details.

For these reasons, formal soil-structure interactions analyses are usually reserved for unusual or expensive structures, where either the consequences of damage are severe or the potential savings in cost warrant the additional analytical effort. Few structures have been analyzed in the context of mining subsidence.

It is to be recognized that buildings may sustain damage from poor engineering, poor construction, normal deterioration, neglect of maintenance and misuse, as well as from ground movements. Care must therefore be exercised in collecting evidence of damage and interpreting it. Considerable attention has been paid to crack morphology. Cracks may or may not provide the information needed to assign a cause to the observed damage.

Considering the wide variety of ground movement sources, damage criteria have evolved from relatively few, the most prominent being settlement under surface loads, movements adjacent to open cuts and tunnels, and subsidence due to coal mining. The criteria range from simple empirical rules to more sophisticated relationships that include the effects of building shape and mode of distortion.

To properly assess the prospects for damage in a specific situation, it is necessary to account for all likely modes of building deformation. Use of more than one damage criterion may be necessary. O'Rourke (1977), for example, indicates that buildings subjected to mining subsidence (a ground movement that is often considered to induce most damage through horizontal strains) may be subjected to significant bending distortion in the hogging mode for relatively shallow mining depths. In a similar vein, Attewell (1977a) warns that the damage criteria formulated on the basis of self-weight settlements are not generally applicable to ground

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movements caused by the yield of soil into soft-ground tunnels. The lateral compressive ground strains near the center of the subsidence trough (above the tunnel centerline) act to resist the development of direct tensile cracking at the base of a load-bearing wall. Near the edges of the trough, lateral ground strains in combination with angular distortions caused by differential settlement can cause damage at lower distortions than might be indicated by criteria developed from settlement analyses. Directional and areal variations of the ground movement necessitate a three-dimensional consideration of any problem.

A rather ingeneous approach to damage criteria lies in the critical tensile strain concept. It seems applicable to any mode of distortion caused either by a single source of ground movement or by two or more sources acting together. For specific building types and specific modes of distortion arising from ground movements other than settlement under surface weight, it should be possible to develop new curves similar to those in Figure 5.21 that relate building shape, stiffness, and other important parameters to an index of building distortion and critical tensile strain for homes with and without basements. This approach seems particularly attractive to the problem of combined bending/shear distortion and horizontal strain to which buildings in a narrow subsidence trough may be subjected. It is also applicable to buildings located above broad troughs typical of active mines. Burland and Wroth (1974) suggest that efforts be concentrated on a simple approach, taking into account properties of the structure. Where necessary, soil-structure interaction analysis can be used to determine strains throughout a structure, and these can be compared to appropriate critical tensile strains. At the same time, it is necessary for mine subsidence applications to extend damage criteria beyond the onset of the slight damage associated with the cracking of interior walls to the more severe types of damage, such as tilting and buckling of walls, failure of utilities, etc. Establishment of criteria encompassing the broad spectrum of damages associated with subsidence requires that damages be catalogued at individual subsidence sites, and that during the subsidence process, building deformation parameters, such as angular distortion, be measured in a three dimensional context along with ground surface deformation parameters, such as strain, tilt and radius of curvature. In the next section of the report, the types of subsidence damage sustained by homes above active mines in western Pennsylvania are reviewed and specific recommendations are presented for developing a subsidence damage classification system for conditions in the United States.

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6. SUBSIDENCE DAMAGE TO HOMES ABOVE ACTIVE MINES IN WESTERN PENNSYLVANIA

Introduction

One of the most important objectives of studying mine subsidence is to develop a capability for predicting the level of damage that will be sustained by structures constructed at ground surface. It is well accepted that the severity of damage derives from a combination of factors related to the characteristics of the structure, the site, geology, mining, and so on (Shadbolt, 1972).

The few preliminary studies that have been conducted to date in the United States have attempted to relate damage to structures to such factors as overburden thickness and proximity of mining to the structure. Unfortunately, the term 'damage severity' is often not carefully defined or carries different meanings from one study to the next. As was pointed out in the previous section, this shortcoming is shared with several studies of building damage associated with settlements of soft ground.

The National Coal Board and Building Research Establishment classifications provide a good starting point for developing a damage classification system, but are not directly applicable to conditions in the United States owing to certain differences in building construction. In particular, homes in the United States generally have basements; those in the United Kingdom generally do not.

The purpose of this section of the report is to present: 1) a tentative method for systematically classifying the severity of subsidence damage to homes based on the description of damage, the nature of required repairs and the repair costs, and 2) tentative relationships between damage severity and geologic, mining, site and structure parameters.

The following bounds are placed on the damage classification and the relationships that will be discussed:

1. All structures are conventional homes--from one to 2.5 story, one or two family dwellings--most with basements.

2. All structures are located above active mines--some over panels, some over barrier pillars, and some over pillar support areas within panels.

3. Overburden ranges in thickness from 150 to 795 feet.

4. The geologic setting is characteristic of the Northern Appalachian Coal Field, particularly western Pennsylvania. Soils are primarily residual, colluvial or alluvial. Rock strata are Pennsylvanian and Permian-age cyclothemic sedimentary units with significant lateral and vertical variability.

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Data Availability

The data used in the analyses were dictated by availability. The only significant body of reasonably well documented cases of subsidence damage to structures above active mines in the United States is maintained by the Pennsylvania Office of Mine Subsidence Regulation and pertains to homes in western Pennsylvania. These data include cost settlements, estimates for the repair of damage and, commonly, descriptions of damage.

Two hundred eighty two (282) cases of mine subsidence over active mines were examined. Figure 6.1 summarizes the availability within this group of cases having damage description, damage repair estimates and cost settlements, description of the structure and so on. (We note that these data are more amendable to the development of damage classification than are documented cases above abandoned mines, which generally do not include cost data.) Fifty-four of the 282 cases over active mines involved cases that were pending and therefore unavailable for study. Eliminating these, as well as cases with limited damage descriptions or insufficient information to estimate home replacement value, left 1134 cases involving 1134 homes. The volume of data was quite large, prompting the use of a computerized data base.

Data Base And System of Analysis

The data base employed was designed to permit entry of several hundred cases, each with as many as 105 parameters. The data file structure and definitions of the parameters are presented in Appendix B. Data concerning the 134 cases are listed in Appendix C. Functions of the basic parameters are presented in Appendix D. Variable definitions and abbreviated list titles should be self-explanatory.

Programs CREATE and LIST were written for the purpose of creating, maintaining and presenting the data base. Both are user interactive. A third program, DEFLIS, provides a listing of the variables, their formats and definitions.

To perform the statistical analysis, three programs were adopted from IBM's scientific subroutine package for the 1130 computer:

MULTR1-multiple linear regression with outputs:

- Standard error of regression
- Multiple correlation coefficient
- Fstatistic
- Variable:
  - Mean
  - Standard deviation
  - Regression coefficient
  - Standard error or regression coefficient statistic

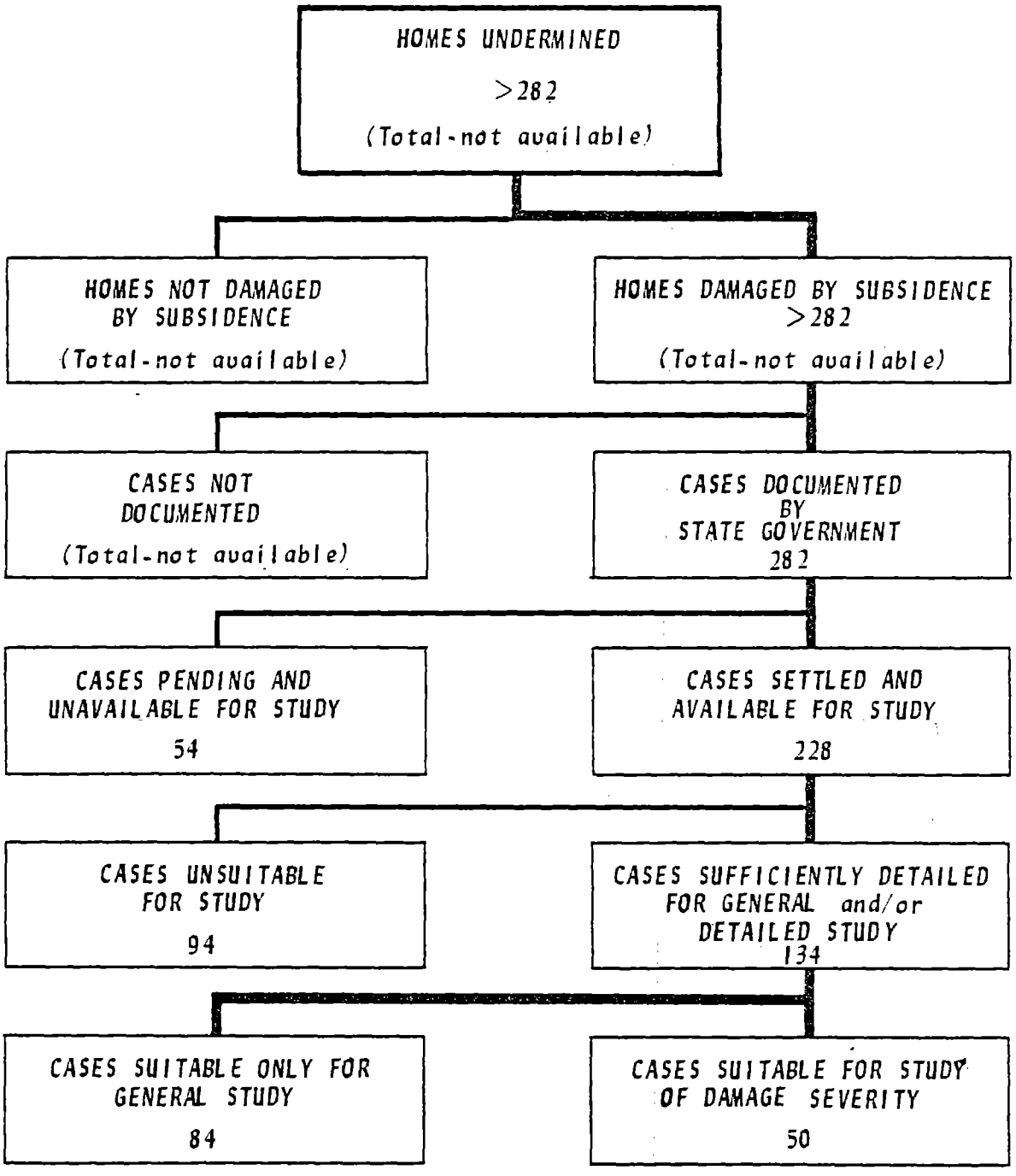


FIGURE 6.1 - CASE AVAILABILITY - SUBSIDENCE OVER ACTIVE MINES IN WESTERN PENNSYLVANIA, 1966-1981

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Intercept
Plot of single independent variable regressions
Residuals
Estimates of the dependent variable

TABONE--statistics of a single variable:

Histogram
Mean
Standard deviation
Range

TABTWO--cross tabulation of statistics of two variables:

Mean
Standard deviation
Ranges
Frequency matrix

All are user interactive and permit selection of records by specifying an exclusion matrix (S matrix) of 0's or 1's, one per record, where 0 identifies an excluded record.

MULTR1 was modified to permit a regression analysis of any group as large as 40 of the 141 variables of 94 functions defined for each site.

Three supplementary, user interactive programs written to process data include:

SUM1-- computes the frequency of specific values of a single variable

SUMSIX-- computes the frequency of damage associated with:

1. Foundation wall damage (specified as walls, doors or windows);

2. Foundation slab damage;

3. Exterior wall damage (specified as walls, doors or windows of the superstructure, excluding attached garages); and

4. Interior damage (specified as walls, ceilings or doors).

DAMAG1-- computes the frequency of distinct combinations of damaged home components and severity statistics per combination; and the frequencies and statistics for all combinations of damaged home components.

Data and programs were stored on a magnetic disk. Calculations were performed on a Digital PDP-11/40 computer.

Regression analyses were performed to identify principal structure, site, geologic and mining parameters that might govern severity. A model of severity was developed using a stepwise elimination regression analysis--the variable of lowest t-statistic being eliminated until a 90% confidence level was achieved for all remaining variables. Several methods of defining severity were investigated and numerous parameters were added and deleted during the program before achieving what was considered to be the most satisfactory interpretation of the data. Recommendations for improving the data base are presented at the conclusion of this section of the report.

Time of Occurrence of Subsidence

All reported incidents of subsidence above active mines that involved damage to a home took place during the period 1966-1981 (Figure 6.2A). The distribution of reported cases by month is more or less uniform (Figure 6.2B).

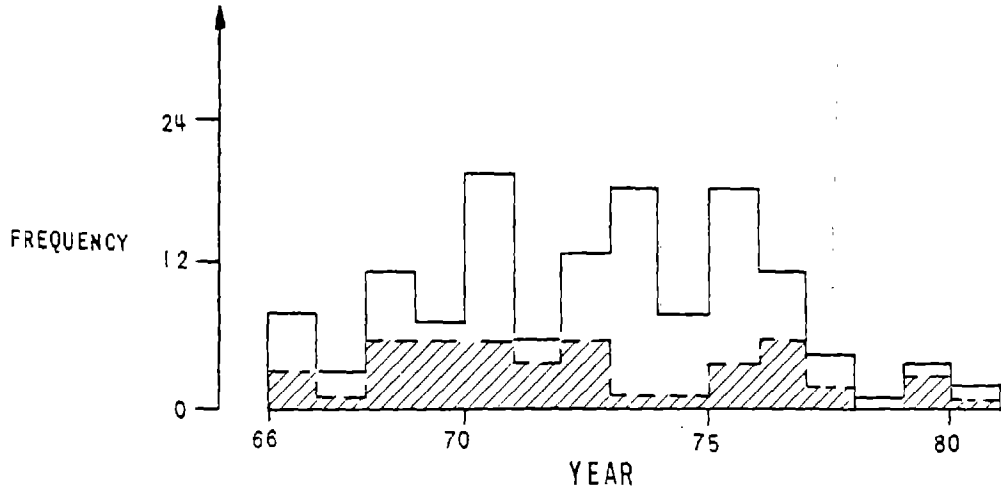
Types of Homes Damaged by Subsidence

Slightly over sixty percent of the homes that have been reported damaged by subsidence above active mines in western Pennsylvania are conventional one-story ranch-type dwellings. The remaining forty percent are divided almost equally between conventional 1.5-story dwellings, such as those of Cape Cod style, and 2-story dwellings (Figure 6.3A). The few out-of-the-ordinary homes include a basement with no superstructure and two 2.5-story homes. A selection of representative homes are pictured in Figures 6.4A through 6.4H. Table 6.1 presents the frequencies of occurrence of the various types of homes with regard to exterior facing type, number of stories, foundation type and typical basement wall components.

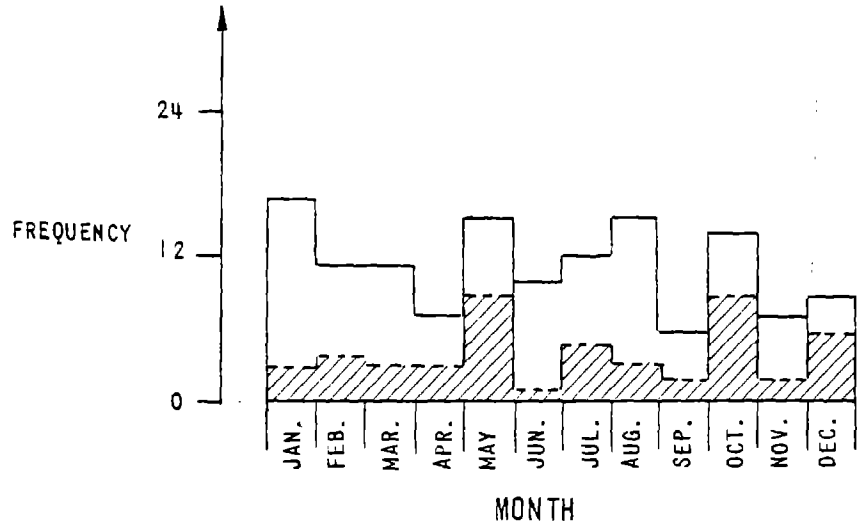
Most homes (about two-thirds) are of average quality and are less than 45 years old (Figure 6.3G). Replacement values generally range between $45 and $75 thousand (1981 dollars) (Figure 6.3H). Homes in rather exclusive suburbs, as well as certain large old homes, are valued in excess of $100 thousand, while small, rather simply constructed dwellings built in outlying areas during World War II are valued at less than $45 thousand.

Nearly fifty percent of the homes are rectangular in plan, and forty percent are L-shaped (Figure 6.3B). The remainder, which do not conform to a simple geometry, are termed 'irregular'.

Living areas range from about 1,000 square feet to 7,000 square feet and typically range between 1,300 and 2,500 square feet (Figure 6.3I).



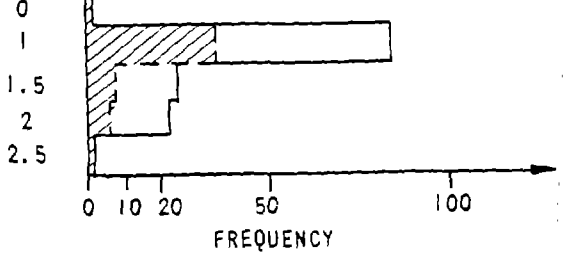
A.



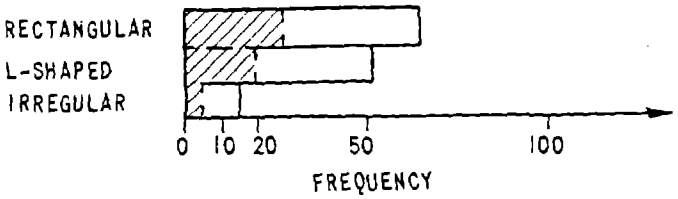
B.

NOTE: Hatching denotes cases in damage severity model

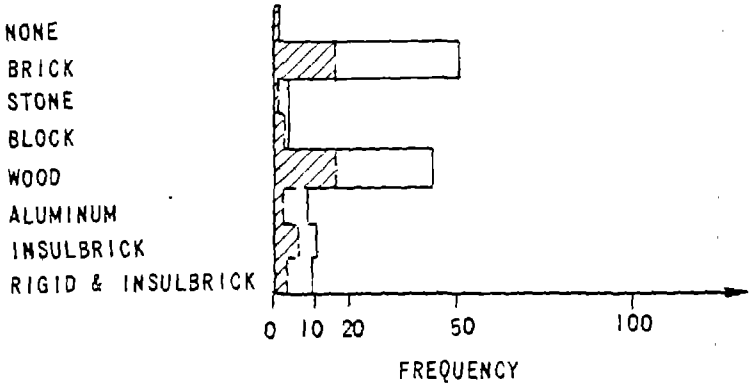
FIGURE 6.2 - TIME OF OCCURRENCE OF REPORTED SUBSIDENCE INVOLVING HOME DAMAGE ABOVE AN ACTIVE MINE IN WESTERN PENNSYLVANIA



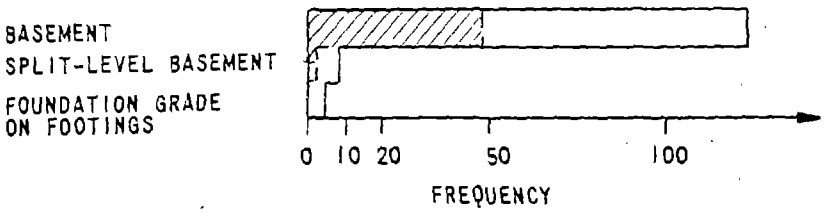
A.
NUMBER OF STORIES



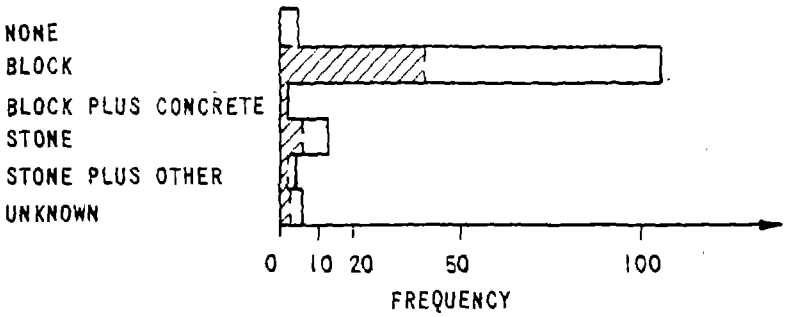
B.
SHAPE OF SUPERSTRUCTURE



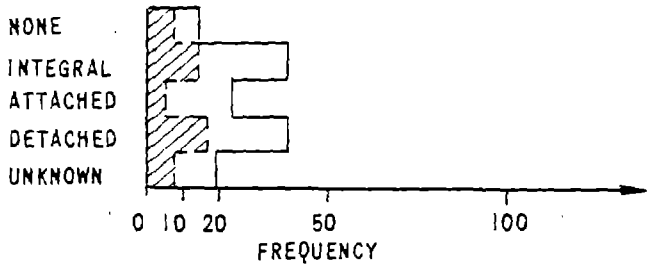
C.
EXTERIOR WALL FACING



D.
FOUNDATION TYPE



E.
BASEMENT BEARING WALL
COMPONENTS



F.
GARAGE TYPE

NOTE: (1) Hatching denotes cases
in damage severity mode
(2) Frequency=Number of cas

FIGURE 6.3 - CHARACTERISTICS OF HOMES IN WESTERN PENNSYLVANIA DAMAGED BY SUBSIDENCE ABOVE ACTIVE MINES

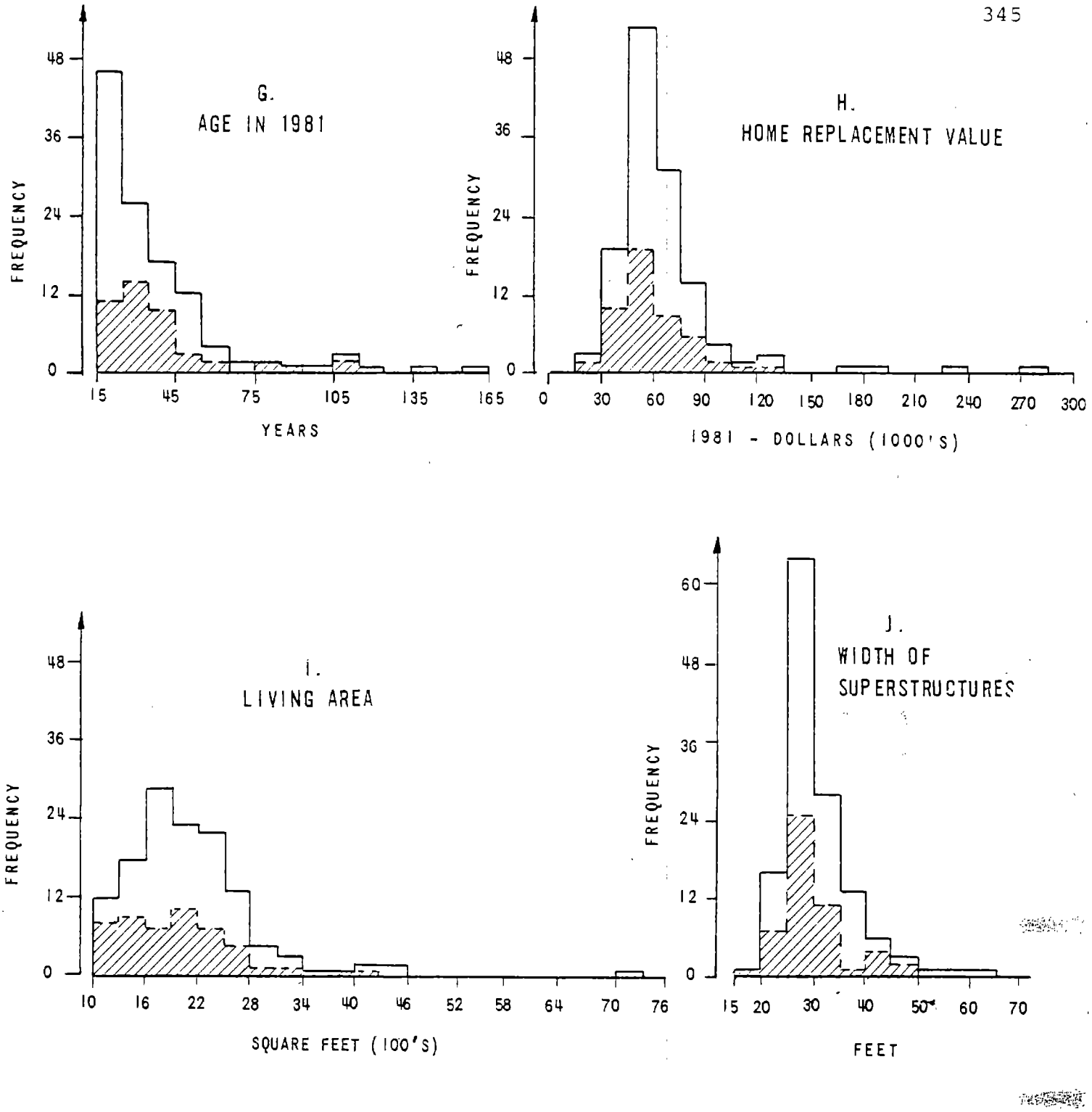
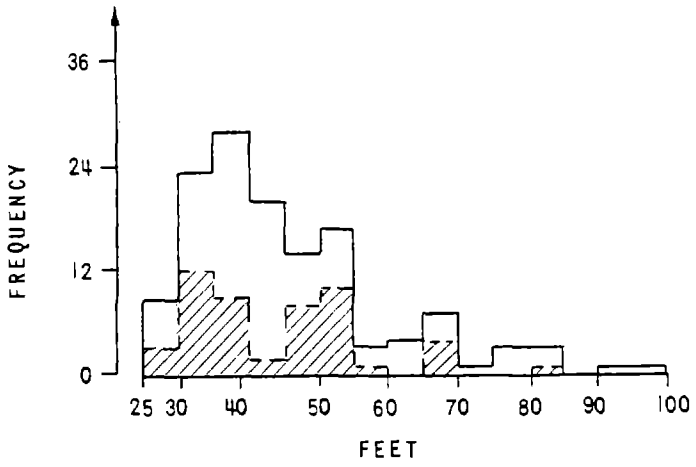
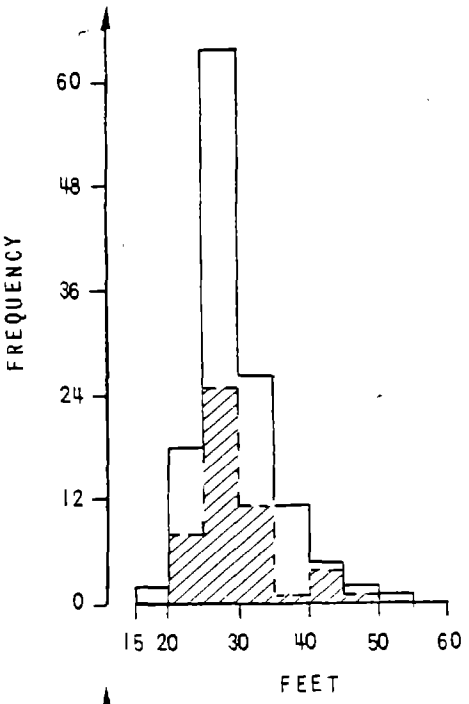


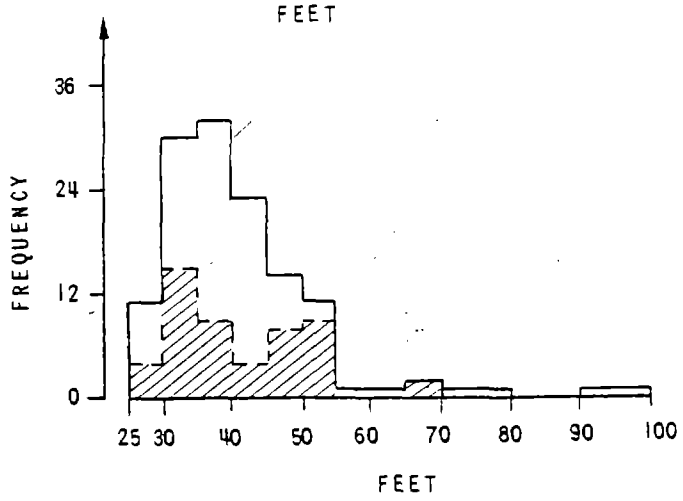
FIGURE 6.3 - CONTINUED



K.
LENGTH OF
SUPERSTRUCTURE



L.
WIDTH OF BASEMENT TYPE
FOUNDATIONS (INCLUDES SPLIT-
LEVEL)



M.
LENGTH OF BASEMENT TYPE
FOUNDATIONS (INCLUDES SPLIT-
LEVEL)

FIGURE 6.3 - CONTINUED

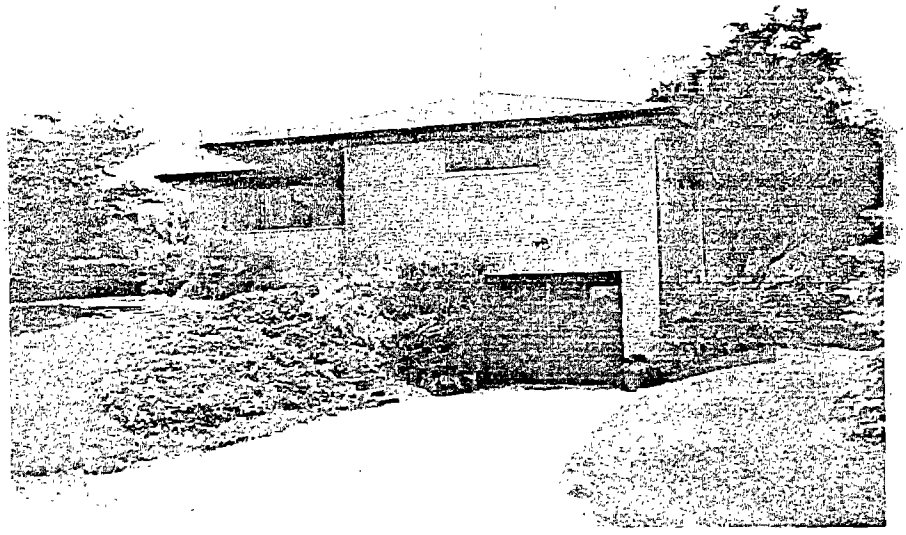


FIGURE 6.4A- ONE STORY RIGIDLY-FACED HOME WITH INTEGRAL GARAGE (CASE NUMBER 166)

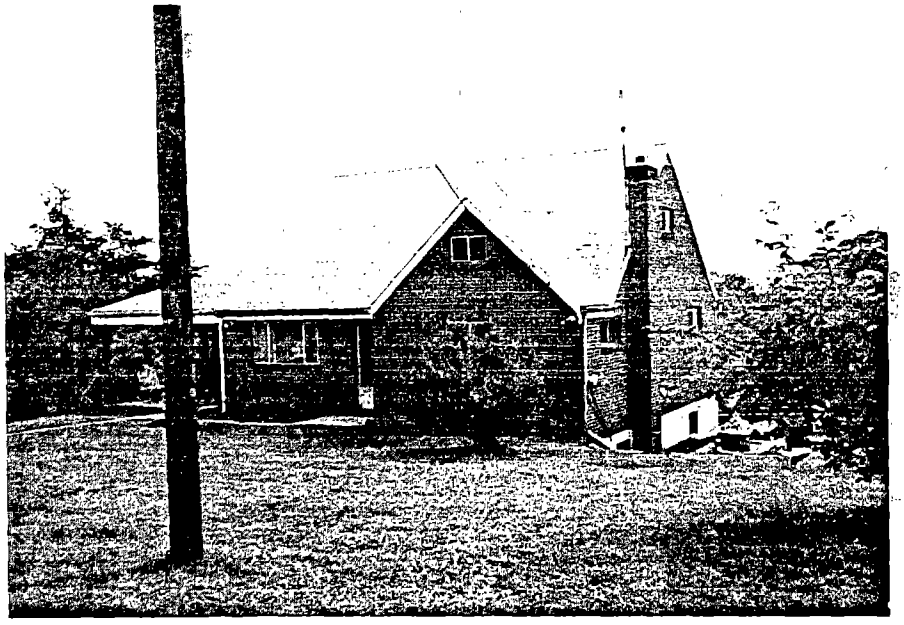


FIGURE 6.4B- ONE AND ONE-HALF STORY RIGIDLY-FACED HOME (CASE NUMBER 142)

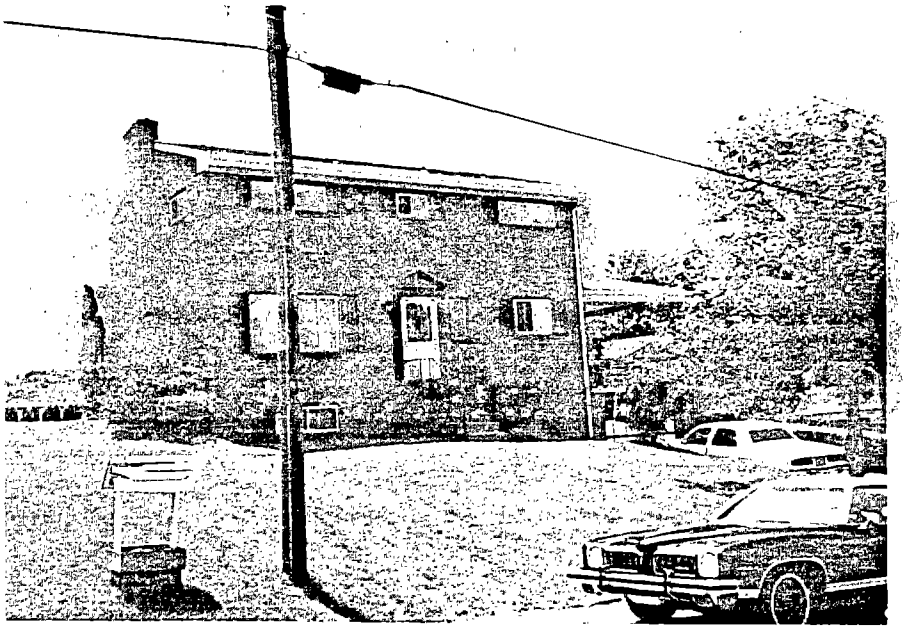


FIGURE 6.4C- TWO STORY RIGIDLY-FACED HOME WITH ATTACHED GARAGE (CASE NUMBER 433)

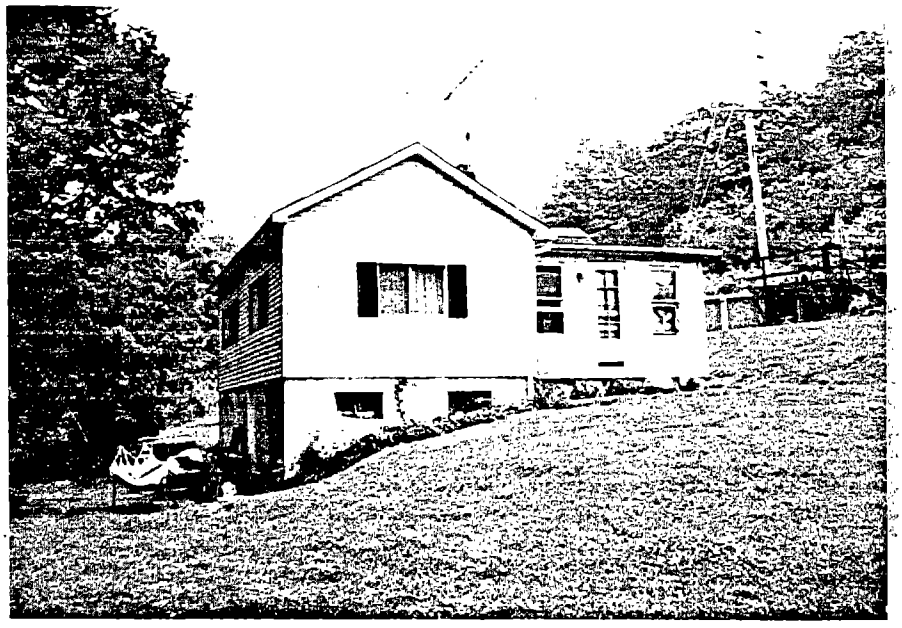


FIGURE 6.4D- ONE STORY FLEXIBLY-FACED HOME (CASE NUMBER 83)



FIGURE 6.4E- ONE AND ONE-HALF STORY FLEXIBLY-FACED HOME WITH INTEGRAL GARAGE (CASE NUMBER 35)



FIGURE 6.4F- TWO STORY FLEXIBLY-FACED HOME (CASE NUMBER 82)

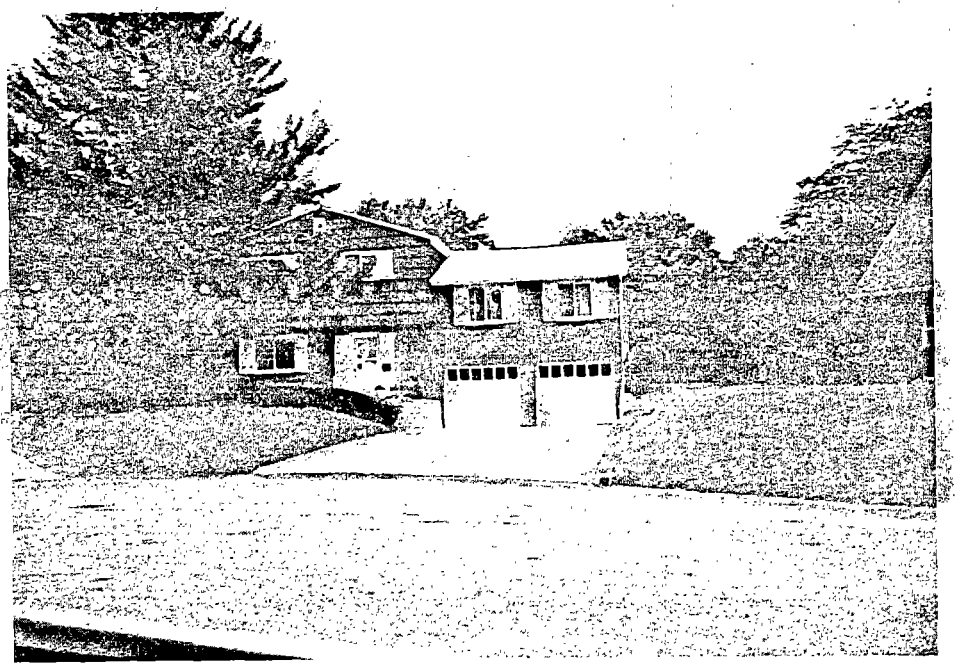


FIGURE 6.4G- ONE STORY COMBINATION FACED SPLIT-LEVEL HOME WITH INTEGRAL GARAGE (CASE NUMBER 354)

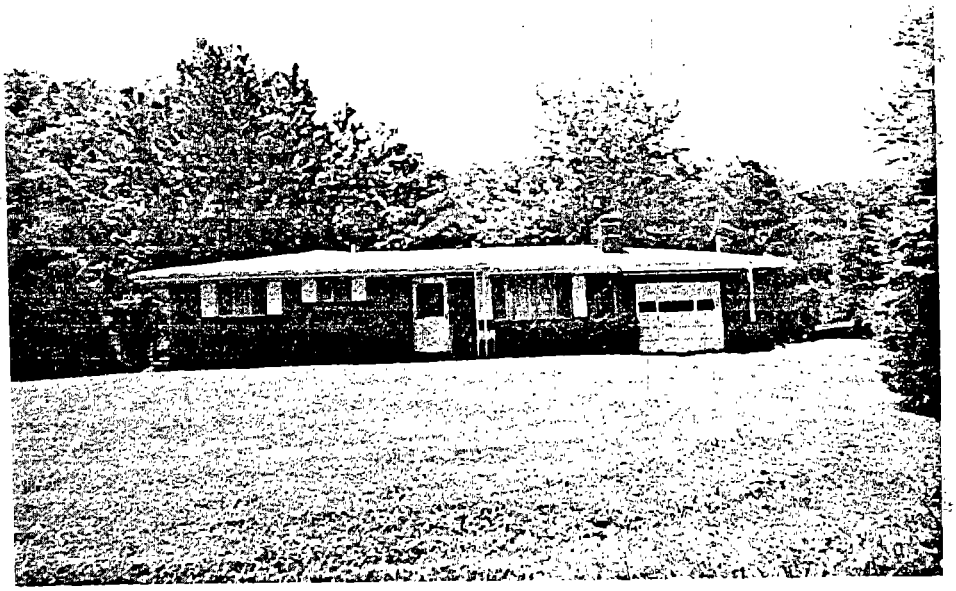


FIGURE 6.4H- ONE STORY RIGIDLY-FACED HOME WITH ATTACHED GARAGE (CASE NUMBER 121)

Table 6.1

TYPICAL HOMES

Type Number	Exterior Facing Type	Number of Stories	Foundation Type	Typical Basement Wall Components	Frequency	Age (1981) Mean	Standard Deviation	Range	Number
1	Rigid	1.0	I	Block	37	25.8	7.6	15- 41	31
2	Rigid	1.5	I	Block	4	23.3	6.7	19- 31	3
3	Rigid	1.5	I	Stone	1	138.0	0.0	138	1
4	Rigid	2.0	I	Block	6	24.4	7.1	16- 34	5
5	Rigid	2.0	I	Stone	4	115.0	37.1	72-162	4
6	Rigid	1.0	II	Block	2	19.5	0.7	19- 20	2
7	Rigid	1.5	II	Block	2	20.0	0.0	20	2
8	Rigid	1.0	III	None	3	18.5	2.1	17- 20	2
9	Flexible	1.0	I	Block	31	35.3	13.3	16- 63	6
10	Flexible	1.0	I	Stone	2	49.0	1.4	48- 50	2
11	Flexible	1.5	I	Block	14	39.5	9.2	23- 52	2
12	Flexible	1.5	I	Stone	2	71.0	48.1	37-105	2
13	Flexible	2.0	I	Block	4	54.3	16.2	41- 74	4
14	Flexible	2.0	I	Stone	7	75.7	25.3	40-107	7
15	Flexible	2.5	I	Stone	1	79.0	0.0	79	1
16	Flexible	1.0	II	Block	1	-	-	-	0
17	Flexible	1.5	II	Block	1	26.0	0.0	26	1
18	Flexible	2.0	III	None	1	22.0	0.0	22	1
19	Combination	1.0	I	Block	5	18.0	3.4	16- 24	5
20	Combination	2.0	I	Block	2	18.5	0.7	18- 19	2
21	Combination	1.0	II	Block	2	19.5	2.1	18- 21	2
22	Combination	1.0	III	None	1	28.0	0.0	28	1
23	None	0.0	I	Block	1	24.0	0.0	24	1

Note: 1) Foundation types are: I-single level basement; II-split level basement; and III-slab-on-grade or a crawl space beneath at least two-thirds of the superstructure.

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All but four percent of the homes have full or partial basements (Figure 6.3D). The remainder are of slab-on-grade construction or have a shallow crawl space. Figure 6.5 depicts the three typical foundation types.

The maximum superstructure length of the homes ranges from 25 feet to 99 feet and is typically 35 to 60 feet. The width ranges from 18 to 63 feet and is typically 25 to 30 feet (Figures 6.3J and 6.3K).

Basement dimensions generally conform to those of the superstructure, although the basement does not usually extend beneath an attached garage at grade (Figures 6.3L and 6.3M). Basement walls extend a uniform depth below grade in about half the homes and in the others vary in depth from front to back (26 percent), diagonally (15 percent) or laterally from one side of the home to the other (9 percent). The nominal depth of the basement wall below grade (embedment depth) is about 4.5 feet and commonly ranges from 6 feet along one face of the home to nil along all or part of the opposite face. Figure 6.6 presents examples of the embedment configurations.

Superstructures of homes are generally wood frame.

About half the homes possess rigid exterior facings (brick, stone or block), and the other half possess flexible facings (wood, aluminum or insulbrick) or combinations of rigid and flexible facings (Figure 6.3C).

Basement walls (bearing walls) are composed of concrete block in more than 80 percent of the homes (Figure 6.3E) and in the remainder are cast-in-place concrete or stone, occasionally in combination with other materials. Perhaps twenty percent of the basements built since 1965 are reinforced as per recommendations of the National Concrete Masonry Association (1975). None of the basements are designed to resist subsidence. Basement floors are almost always concrete and only rarely dirt or asphalt.

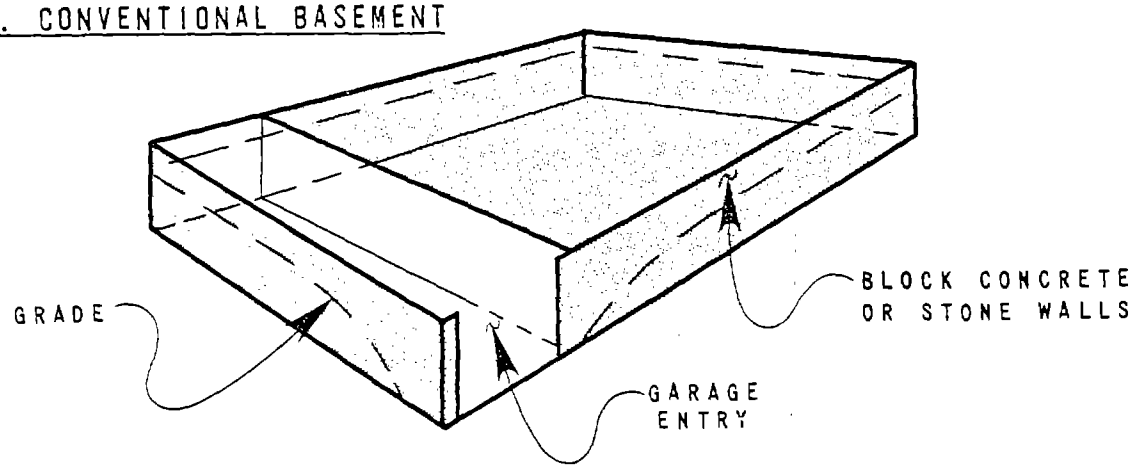
Homes built prior to 1955 generally have plaster interior wall facings. Those built more recently generally have plaster board (dry wall) interior wall facings.

Approximately eighty-five percent of the homes have garages. Of these, nearly forty percent are integral, about the same number are detached, and the remainder are attached (Figure 6.3F). An integral garage is incorporated within the basement of the home; an attached garage has a wall and possibly a roof in common with the home; a detached garage is completely separate from the home.

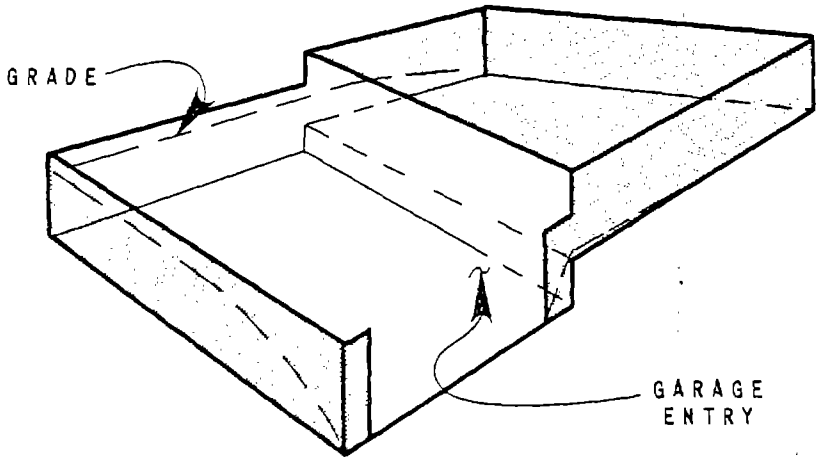
Site, Mining and Geologic Characteristics

Topographically, about half the sites are naturally sloping (48 percent), more than 80 percent of these at grades between 5 and 20 percent. About one quarter of the sites lie on alluvial

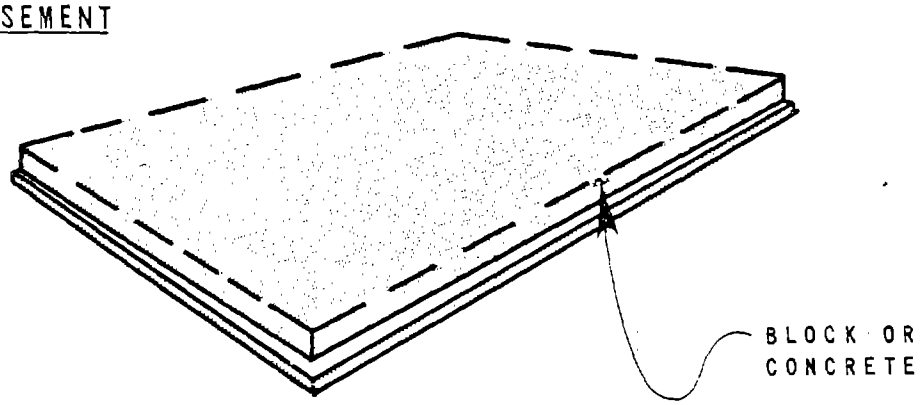
A. CONVENTIONAL BASEMENT



B. SPLIT-LEVEL BASEMENT



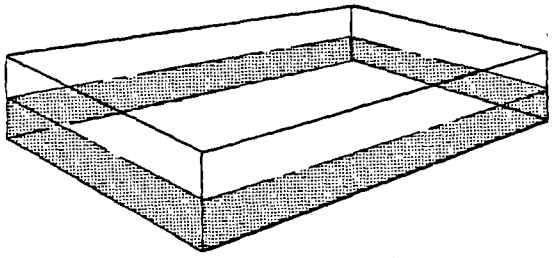
C. NO BASEMENT



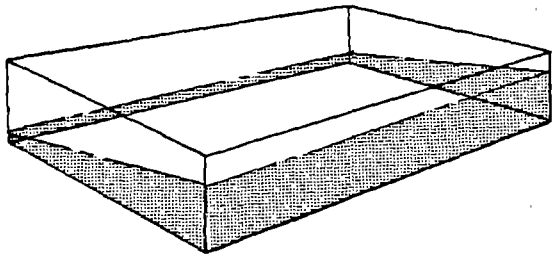
NOTE: FOOTINGS NOT SHOWN
FOR TYPES I & II

FIGURE 6.5 - TYPICAL TYPES OF SUBSTRUCTURAL ELEMENTS

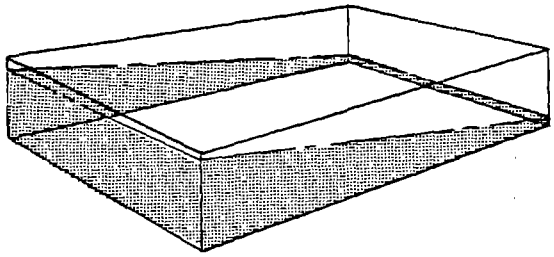
LEVEL:



LENGTH TO LENGTH:



WIDTH TO WIDTH:



DIAGONAL:

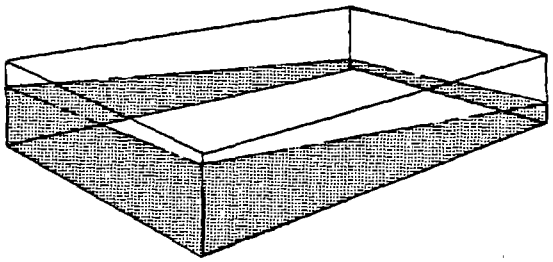


FIGURE 6.6 – CONVENTIONAL EMBEDMENT CONFIGURATION TYPES

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terraces; twenty percent are located on naturally elevated areas such as ridges, plateaus and slope crests; and the remaining 8 percent are located on valley floors or at the toes of slopes (Figure 6.7B). Figure 6.8 presents examples of these topographic features. All are typical of western Pennsylvania.

Residual and/or colluvial soils are characteristic of most sites of subsidence (72 percent). Recent alluvium and alluvial terrace deposits are characteristic of the remainder (Figure 6.7A).

Mined coal seams beneath the sites are the Pittsburgh, the Upper Freeport (known in certain locations as the Thick Freeport), the Lower Kittanning and the Lower Freeport in descending order of prominence. The Pittsburgh Coal is associated with 59 percent of the cases of home damage above active mines, while the Upper Freeport accounts for 19 percent, the Lower Freeport for 4 percent and the Lower Kittanning for 18 percent (Figure 6.7D).

Mined heights as determined from mine maps and available references ranged from 3 to 8 feet with an average of about 6.25 feet (Figure 6.7E). The Pittsburgh and Upper Freeport Coals both average about 7 feet, the Lower Freeport about 3.75 feet and the Lower Kittanning about 3.5 feet (Table 6.2).

Overburden thicknesses at the subsidence sites range from 150 to 795 feet (Figure 6.7C) and average about 460 feet. Representative overburden thicknesses are 400 feet for sites above the Pittsburgh Coal, 500 feet for sites above the Upper Freeport, 360 feet for sites above the Lower Freeport and 600 feet for sites above the Lower Kittanning. Table 6.3 presents a summary of overburden thicknesses relative to soil type and seam mined.

The ratio of mined height to overburden thickness ranges from 0.006 to 0.018, the lowest values being associated with the Lower Kittanning Coal and the highest with the Pittsburgh Coal.

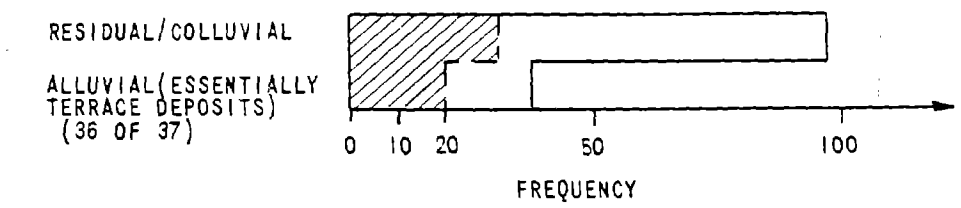
Mining types are categorized as follows:

A. Proximity to the subsidence site of mining that precipitated home damage:

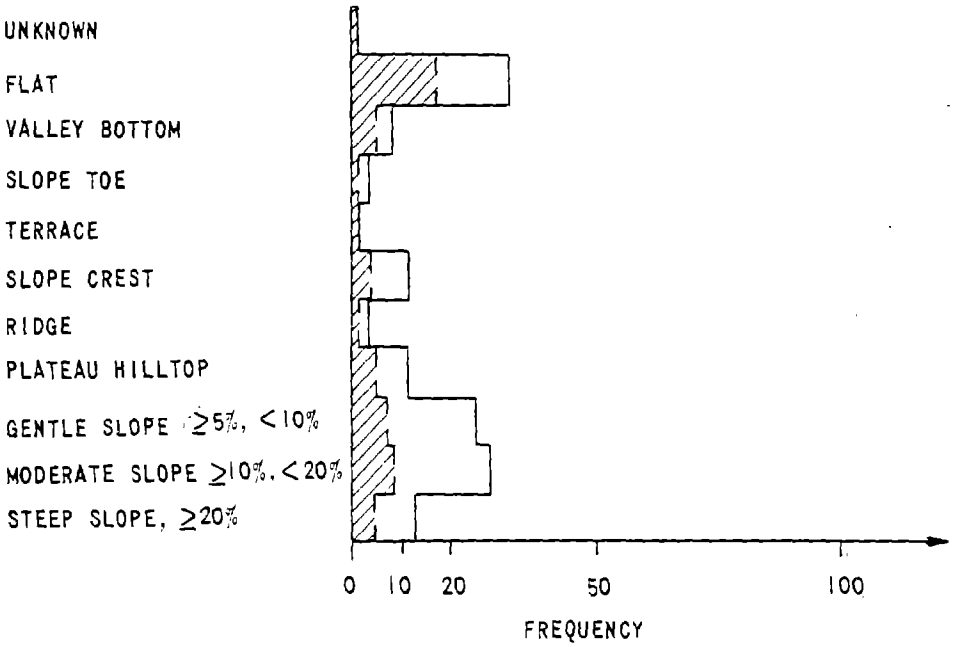
   1. Directly beneath site
   2. Adjacent to site

B. Type of mining and nominal extraction percentage:

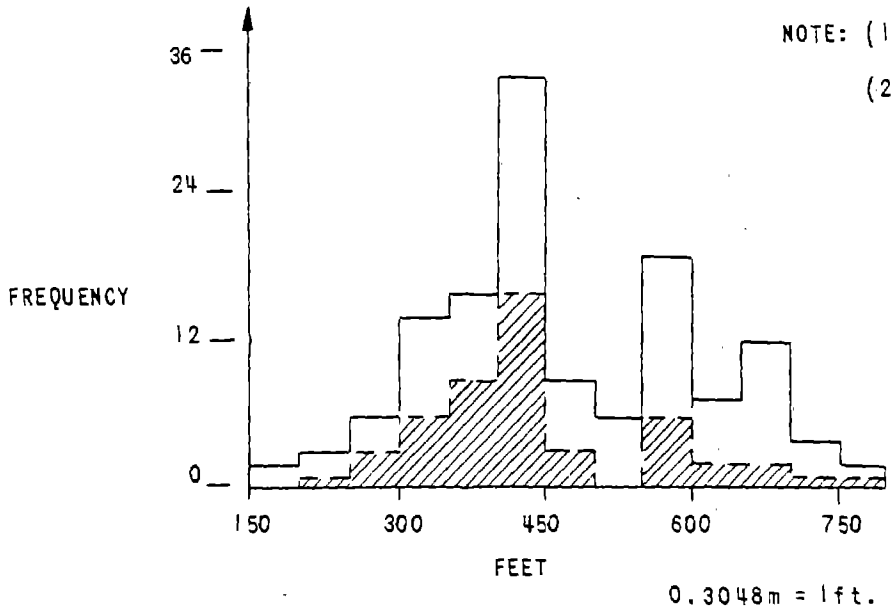
1.	Barrier pillar (solid coal)	0%
2.	Advance phase of retreat mining	35%
3.	Retreat, pillars split or slabbed	50%
4.	Retreat, pillars split and notched	70%
5.	Retreat, full extraction	85-90%
6.	Longwall	95%
7.	Unknown	-



A.
SOIL TYPES



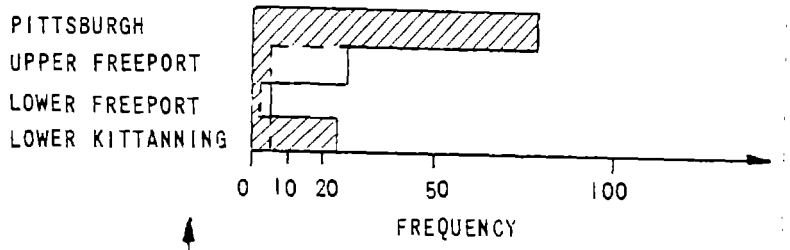
B.
TOPOGRAPHIC FEATURES



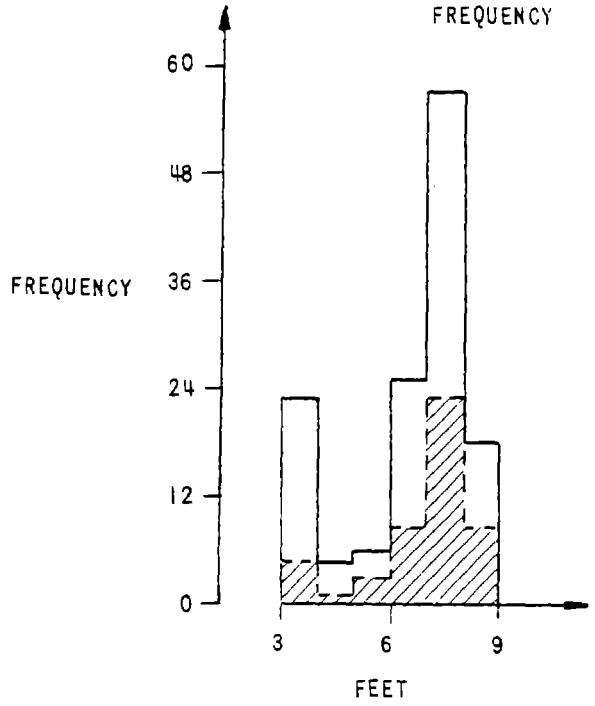
NOTE: (1) Hatching denotes cases in damage severity model
(2) Frequency=Number of cases

C.
MINE DEPTH

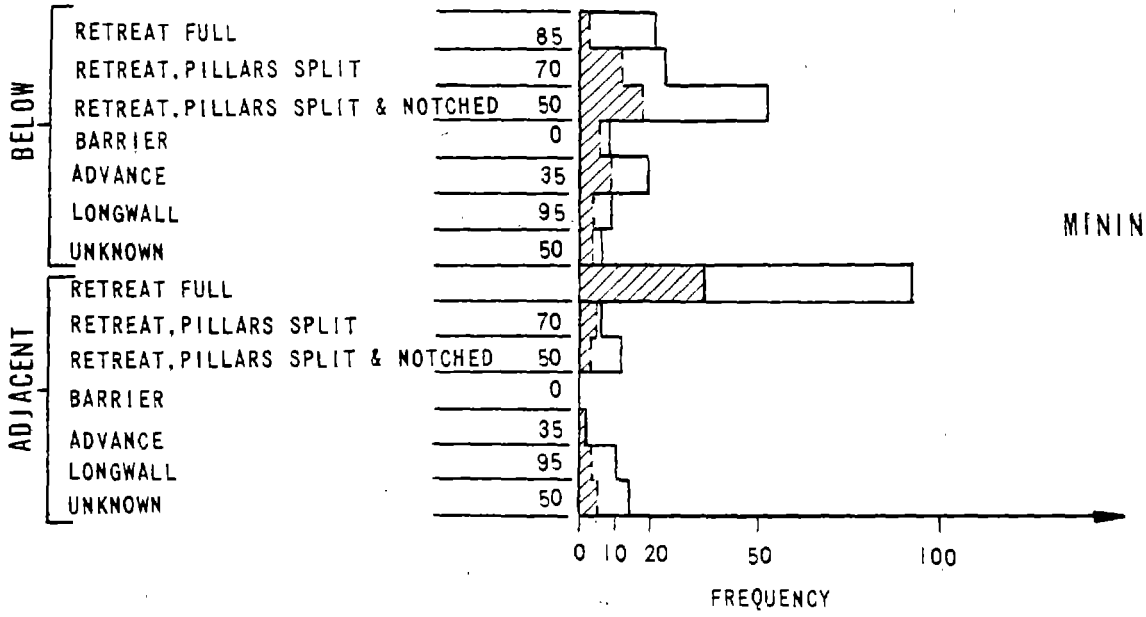
FIGURE 6.7 - SOIL, TOPOGRAPHIC AND MINING CHARACTERISTICS OF SITES OF HOME DAMAGE ABOVE MINES IN WESTERN PENNSYLVANIA



D.
COAL SEAM



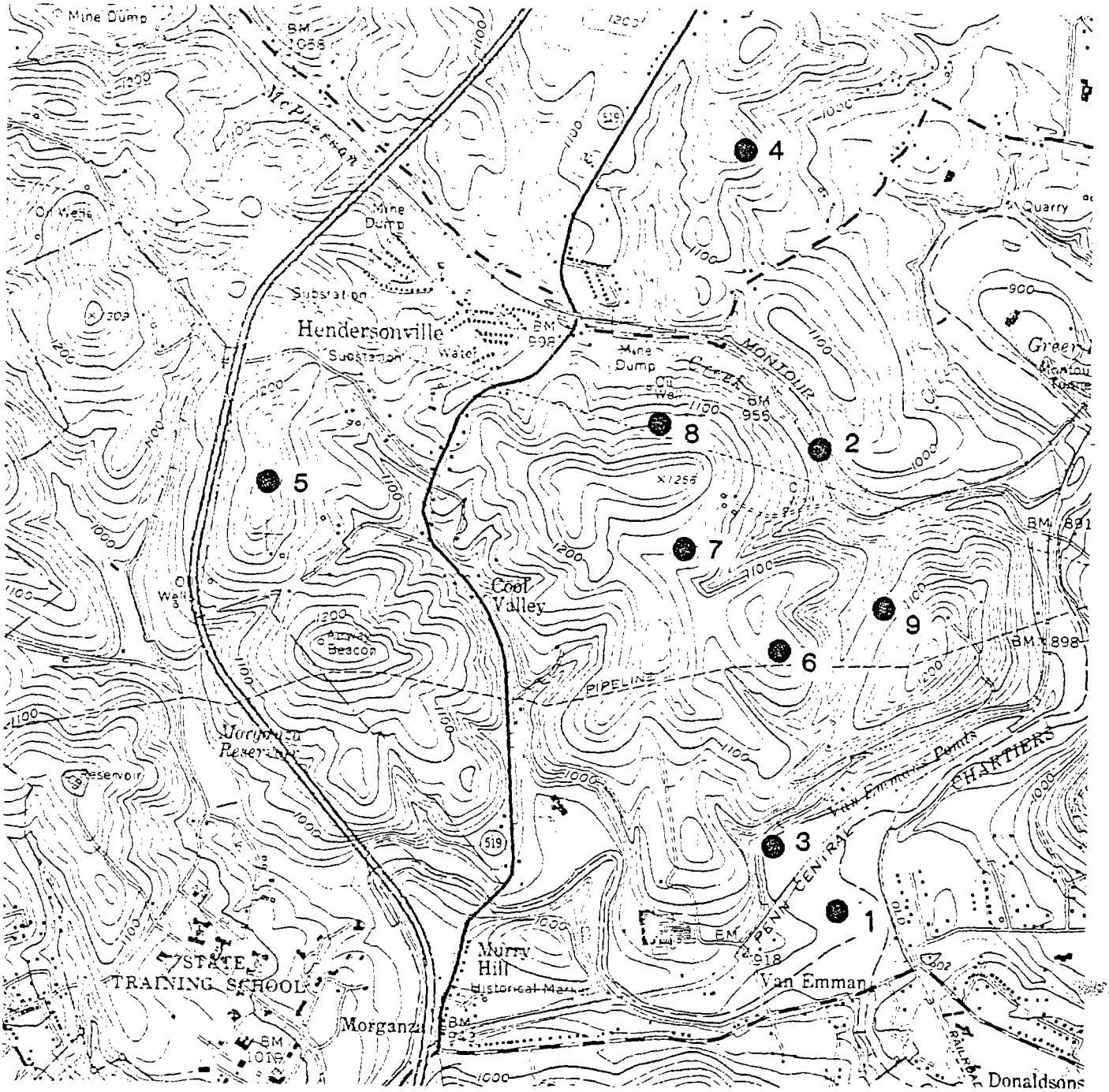
E.
NOMINAL MINED COAL THICKNESSES



F.
MINING TYPES

FIGURE 6.7 - CONTINUED

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GENERAL TOPOGRAPHY KEY:

1. FLAT; 2. VALLEY FLOOR; 3. SLOPE TOE; 4. SLOPE CREST; 5. RIDGE;
6. PLATEAU OR HILLTOP; 7. GENTLE SLOPE; 8. MODERATE SLOPE; 9. SEVERE SLOPE.

FIGURE 6.8 - CHARACTERISTIC TOPOGRAPHIC SITE FEATURES, WESTERN PENNSYLVANIA

Table 6.2

MINED HEIGHTS OF COAL SEAMS
(feet)

Seam	Minimum	Maximum	Mean	Standard Deviation	Number of Cases
Pittsburgh	5.0	8.0	6.98	0.74	79
Upper Freeport	6.0	7.5	6.97	0.32	26
Lower Freeport	3.1	4.3	3.86	0.61	5
Lower Kittanning	3.0	5.7	3.53	0.67	24

Table 6.3

OVERBURDEN THICKNESS
RELATIVE TO SOIL TYPE AND SEAM MINED

		Total Overburden (Soil and Rock), feet				
Soil Type	Seam	Minimum	Maximum	Mean	Standard Deviation	Frequency
Residual/Colluvial	All	160	795	488.4	143.1	97
Alluvial	All	150	509	393.9	58.9	37
Residual/Colluvial	Pittsburgh	160	755	393.4	174.3	45
Residual/Colluvial	Upper Freeport	209	703	523.4	133.5	23
Residual/Colluvial	Lower Freeport	228	470	362.2	104.9	5
Residual/Colluvial	Lower Kittanning	267	795	592.6	114.8	24
Alluvial	Pittsburgh	150	509	397.5	58.1	34
Alluvial	Upper Freeport	284	410	353.0	63.9	3
All	Pittsburgh	150	755	395.2	136.3	79
All	Upper Freeport	209	703	503.7	138.2	26
All	Lower Freeport	228	470	362.2	104.8	5
All	Lower Kittanning	267	795	592.6	114.8	24

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Figure 6.7F presents the distribution of mining types. Table 6.4 lists mining types, by coal seam, beneath and adjacent to sites of subsidence.

Full extraction mining was practiced beneath 22 percent of the sites (full retreat, 16 percent; longwall, 6 percent); partial extraction mining, beneath 68 percent of the sites (advance, 13 percent; pillar splitting, 17 percent; pillar splitting and notching 38 percent); and no mining, beneath those 6 percent of the sites located above barrier pillars. The type of mining practiced beneath 4 percent of the sites is unknown. At 69 percent of the sites, adjacent mining was full retreat, at 13 percent, retreat with pillars split or split and notched and at 7 percent, longwall. The type of mining adjacent was uncertain at the remaining 11 percent of the sites. At only one site was subsidence damage associated with advance mining adjacent to the site. An angle comparable to the angle of draw was calculated for the shortest distance between damaged home and adjacent mining for sites where mine maps are available. These are presented on a site-by-site basis in Appendix C.

Subsidence Damage to Homes

Subsidence damage to homes is generally exemplified by cracks in walls and slabs and misalignments of doors and windows. Types of damage are discussed in more detail in the following paragraphs, along with an evaluation of damage severity viewed in terms of: 1) potential hazard to the occupants, 2) building deformation and attendant effects on structural integrity, and 3) magnitude of repair and associated cost to restore the building to its pre-subsidence condition. These gages of damage severity provide a basis for the damage classification system that is presented.

Locations of Damage to Homes

Reported damage to homes resulting from subsidence above active mines in western Pennsylvania has been catalogued in terms of eight basic building components:

1. Basement
2. Exterior
3. Interior
4. Roof
5. Chimmey
6. Garage
7. Utilities
8. Pavement

Damage to the basement, the building exterior and the garage have been identified more specifically by location as wall, door, window or floor. Damage to the building interior has been identified by location as wall, door, ceiling or floor. Exterior floors (pavements and porches) have been grouped under 'pavement'. The

Table 6.4

MINING TYPES, BY COAL SEAM, BENEATH AND ADJACENT TO SITES OF SUBSIDENCE

Mining Type		Seam				
Below	Adjacent	Pittsburgh	Upper Freeport	Lower Freeport	Lower Kittanning	Total
1. Retreat, Full	Retreat, Full	6		1	13	20
	Unknown				1	1
2. Retreat, Pillars Split or Slabbed	Retreat, Full	17				17
	Retreat, Pillars Split or Slabbed	3				3
	Unknown	3				3
3. Retreat, Pillars Split and Notched	Retreat, Full	24	11		1	36
	Retreat, Pillars Split or Slabbed		2			2
	Retreat, Pillars Split and Notched	2	8			10
	Unknown	2	1			3
4. Barrier	Retreat, Full	2	2			4
	Retreat, Pillars Split or Slabbed	1				1
	Retreat, Pillars Split and Notched	2				2
	Advance	1				1
5. Advance	Retreat, Full	9	2	2	2	15
	Longwall			1	1	2
	Unknown	1				1
6. Longwall	Longwall	1		1	6	8
7. Unknown	Unknown	5				5
						134

term 'basement' includes all substructure portions of a home with sufficient clearance for a person to stand upright, whether furnished as a living area or unfinished. Integral garages are evaluated separately from basements.

The observed frequency of damage to each of the basic building components is summarized in Table 6.5, and to specific components in Table 6.6. Tabulations of damage on a site-by-site basis are presented in Appendix C.

Two rather obvious conclusions are reached from the data:

1. Subsidence damage occurs in a wide variety of locations in homes. Sometimes it is confined to a single location (the basement, for example), while at other times it is more widespread, including the basement, interior, and exterior, etc. A total of 34 distinct combinations of the eight basic damage locations have been reported.

2. Some portions of homes tend to be damaged much more frequently than others, suggesting a progression of damage from foundation level upward:

   a. The basement, for example, is the single most frequent location for subsidence damage. All homes having basements sustained such damage. The high incidence of basement damage seems consistent with the fact that ground movements due to mining are transmitted to the dwelling by way of the foundation. Sensibly, the effects of these movements are felt there before anywhere else and sometimes no where else. Damage was reportedly confined to the basement in 19 percent of all cases.

   b. The interior of the living quarters is the second most prevalent damage location and was reported in 56 percent of the cases. In 28 percent of the cases, interior damage was reported without damage to the exterior of the home. Theoretically, pronounced hogging deformations could have damaged the upper levels of a home without concurrently damaging the basement. However, this seems not to have occurred. Damage to the superstructure appears to be mitigated by its own framing action and by its minimal mechanical attachment to the basement walls. This is illustrated in Case History 7.

   c. Damage to the exterior of a home is somewhat less common than interior and basement damage, but still was reported in 38 percent of the cases.

      Exterior damage was experienced by 59 percent of the rigidly faced homes and 16 percent of the flexibly faced homes. Homes with combination rigid

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Table 6.5

COMBINATIONS OF DAMAGED BASIC BUILDING COMPONENTS

Combination Number	Damaged Components								Frequency
1	B								25
2	B	E							3
3	B		I						17
4	B	E	I						10
5	B	E	I	R					1
6	B		I		C				2
7	B	E	I		C				2
8	B					G			11
9		E				G			1
10	B	E				G			6
11	B		I			G			5
12	B	E	I			G			3
13	B	E	I			G			1
14	B	E			C	G			1
15	B	E	I		C	G			2
16	B	E	I				U		1
17	B	E	I	R			U		1
18	B					G	U		1
19	B	E	I	R		G	U		1
20								P	1
21	B							P	8
22	B	E						P	2
23	B		I					P	6
24		E	I					P	2
25	B	E	I					P	6
26	B	E	I		C			P	1
27	B		I			G		P	4
28	B	E	I			G		P	4
29	B	E	I	R		G		P	1
30	B		I		C	G		P	1
31	B		I				U	P	1
32	B	E	I				U	P	1
33	B	E	I			G	U	P	1
34	B	E	I	R	C	G	U	P	1
									134

Key:
- B - Basement
- E - Exterior
- I - Interior
- R - Roof
- C - Chimney
- G - Garage
- U - Utility
- P - Pavement

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and flexible facings exhibited intermediate frequencies of damage. In only 10 percent of the cases did exterior damage occur without interior damage. Table 6.7 presents combinations of exterior and interior damage encountered relative to the type of exterior facing.

The brittleness of interior plaster walls and the ease with which plaster board separates along joints seems to account for the relatively greater number of reported incidents of damage to home interiors than to exteriors of either type of home.

d. Roof and chimney damage have been reported in only 11 percent of the cases, and in all but one have been associated with basement, exterior and/or interior damage.

e. Garage damage is reported for 42 percent of 101 homes having garages.

Sixty-seven percent of the integral garages and 42 percent of the attached garages sustained damage. Interior garage walls are generally unfinished, making garages less susceptible to damage than living quarters. Only 16 percent of detached garages were reported damaged. Because compensation is not provided for subsidence damage to detached garages, more may have been damaged than reported.

f. Utility damage was reported in only 8 percent of the cases. This number is probably low, since repair of utilities is often handled directly by utility companies and may not be documented in government records. Records do not include information on water wells.

g. Pavement damage including sidewalks, driveways, patios and porches was reported at 30 percent of the homes.

Items a through d are consistent with the proposition that building deformations propagate from the foundation upward to the superstructure--the basement is virtually always damaged, the interior and exterior are damaged less often, and the roof is damaged even less often. Distinct from the locations of damage, which has been the focus of the discussion thus far, the actual severity of damage to the structure is largely a function of the structures' ability to adjust to varying support conditions brought about by ground surface displacement without exhibiting signs of distress.

Table 6.6

SPECIFIC DAMAGED BUILDING COMPONENTS

Component	All (134)*			Rigid Facing			Flexible Facing (64)		
	Damaged	Percent	Population	Damaged	Percent	Population	Damaged	Percent	Population
Home									
Basement	129*	100	129	56	100	56	63	100	63
Unspecified	2	2		1	2		1	2	
Wall	121	94		51	91		60	95	
Door	17	13		8	14		4	6	
Window	6	5		3	5		3	5	
Floor	99	77		45	80		45	71	
Exterior	51*	38	133	35	59	59	10	16	64
Unspecified	0	0		0	0		0	0	
Wall	38	29		30	51		2	3	
Door	24	18		13	22		6	9	
Window	12	9		7	12		4	6	
Pavement	40	30	134						
Interior	75*	56	133	35	59	59	32	50	64
Unspecified	15	11		6	10		5	8	
Wall	48	36		23	39		21	33	
Door	22	17		10	17		7	11	
Ceiling	51	38		27	46		22	34	
Floor	7	5		5	8		2	3	
Roof	5	4	134	4	7	59	1	2	64
Chimney	11*	-	-	6	-	-	4	-	-
Garage									
Integral	26	67	39						
Unspecified	1	3							
Wall	19	49							
Door	3	6							
Window	1	3							
Floor	21	54							

Table 6.6
(continued)

Component	All Number	All Percent	Total
Attached	10	42	24
Unspecified	0	0	
Wall	5	21	
Door	1	4	
Window	0	0	
Floor	7	29	
Detached	6	16	38
Unspecified	1	3	
Wall	2	5	
Door	1	3	
Window	0	0	
Floor	2	5	
Utility	8	6	134

Note:

*Total number of homes is 134--64 with flexible facing; 59 with rigid facing; 10 with combination flexible and rigid facing and one with no facing (no superstructure).
Five homes are slab-on-grade; 129 have basements.

Table 6.7

COMBINATIONS OF EXTERIOR AND INTERIOR DAMAGE RELATED TO EXTERIOR FACING TYPE

Components Damaged	Frequency Per Exterior Facing Type				
	Rigid	Flexible	Combination	None*	Total
Neither Exterior nor Interior	14	29	2	1	46
Exterior only	10	3	0	0	13
Interior only	10	25	2	0	37
Both Exterior and Interior	25	7	6	0	38
Total	59	64	10	1	134

*Basement with no superstructure.

Severity of Damage

The amount of damage sustained by a structure can be expressed in both physical and monetary terms. The three measures of severity discussed below are considered by the writers to be the most meaningful descriptors of subsidence damage to dwellings. They include: 1) hazard to occupants posed by the damaged buildings; 2) deformation and reduced serviceability of the building; and 3) damage repair and attendant cost.

Due to data availability, it has been necessary to confine the discussion to 50 homes where the type and extent of repairs could be identified.

1. Hazard to Occupants

In most of the homes under study, the immediate hazard to occupants was little or none, and occupancy of the home was not often interrupted by subsidence. This is true even where repair costs ranged upwards of several thousand dollars and basement walls were extensively cracked. Sticking doors and windows may have posed an egression problem during an emergency, but few data are available to clarify the degree or probability of hazard posed by these situations.

Subsidence has sometimes presented a hazard to the occupants as a result of pronounced differential settlement, lateral extension or inward movement or bowing of basement walls. Any of these can produce a local reduction of superstructure support or break utility lines. Although the immediate hazard is generally limited hazard (except possibly for broken gas lines), the long term hazard can be more severe where homeowners opt to use damage settlement monies for other than repairing damage. This is a tendency where damage is to the basement or some other low occupancy part of the home.

Shoring has occasionally been installed in houses for temporary support until repairs could be made (Figure 4.20). No outright collapses have been reported.

2. Deformation and Reduced Serviceability of the Homes

Formal measurements of building deformation and associated ground movements are rare. Consequently, the differential and total settlements, extensions and tilts of the 134 damaged homes that have been studied are generally unknown, although observations of damage, including crack patterns and their dimensions, provide some indication of the extent and mode of deformation.

Basements were found to furnish the greatest amount of systematic crack information. Characteristic crack

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patterns for basement slabs and bearing walls are shown in Figures 6.9-6.11. Statistics of crack dimensions and relevant areas are presented in Figures 6.12A-E (slabs) and 6.13A-E (walls). The crack dimension data presented are related solely to 42 cases where basement damage repair requirements are fully known.

Cracking in basement slabs was manifested both as fresh fractures and widening of construction joints. Separations between slab and wall resulted from movements, such as rotation of walls and differential settlement of slabs. Slab cracks typically occurred in one of three patterns and often bisected the long dimension of the basement floor slab. Patterns include: (1) one or more parallel cracks aligned with the long or short dimension of the slab; (2) a grid of cracks parallel to both the long and short dimensions; and (3) random cracks (similar to the grid but with a greater proportion of diagonals) that appeared to exhibit no preferred orientation. About one-third of the cases exhibited a random pattern, while another third were divided equally between the aligned and grid types. The final third of the homes reported no cracking of the floor slab, possibly because it was overshadowed by other types of damage.

Cracking in basement walls was manifested primarily as: (1) vertical fractures along mortar joints and through concrete blocks; (3) diagonal (stair step) fractures along mortar joints; (2) lateral fractures along mortar joints; (3) and (4) combinations thereof. Vertical cracks were found in about 80 percent of the cases, diagonal cracks in about 60 percent, and lateral cracks in 30 percent. Lateral cracks followed mortar joints and were associated with the out-of-plane flexure of walls (bowing or bulging) or with differential settlement. Bowing of basement walls was reported in roughly 5 percent of the cases, but is suspected in at least 20 to 30 percent.

The predominant form of wall cracking is a combination of verticals and diagonals. Typically, these cracks extend diagonally downward in stair step fashion from the tops of walls or the corners of windows or door openings through mortar joints toward midspan of the wall at mid-height, then vertically downward through the blocks, ignoring mortar joints, and meeting if not passing through the footings. These cracks are frequently aligned with cracks that transect the greatest dimension of the slab.

Cracks that develop in the basement walls during each stage of the subsidence process progressively alter the mechanical properties of the wall sections, thereby influencing how the walls respond to successive stages of subsidence.

WALLS

SLAB	0	1	2	3	4	5	6	7	8	TOTALS
0	–	3	1	1	5	3	1	1	2	17
1	2	3	–	–	–	7	–	2	2	16
2	–	1	–	–	–	5	–	–	–	6
3	1	1	–	2	1	1	1	–	2	9
4	–	–	–	–	–	–	–	–	2*	2
TOTALS	3	8	1	3	6	16	2	3	8	50

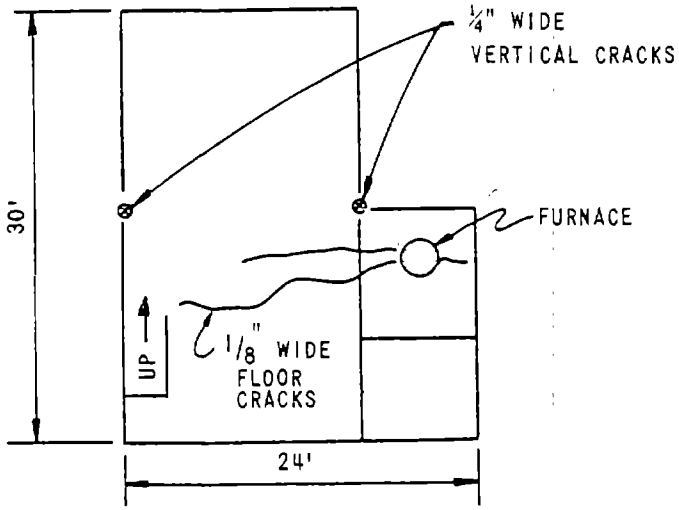
KEY

SLAB:
- 0 NONE
- 1 RANDOM
- 2 ALIGNED
- 3 GRID
- 4 UNKNOWN

WALL:
- 0 NONE
- 1 VERTICAL
- 2 HORIZONTAL
- 3 DIAGONAL
- 4 VERTICAL & HORIZONTAL
- 5 VERTICAL & DIAGONAL
- 6 HORIZONTAL & DIAGONAL
- 7 ALL
- 8 UNKNOWN

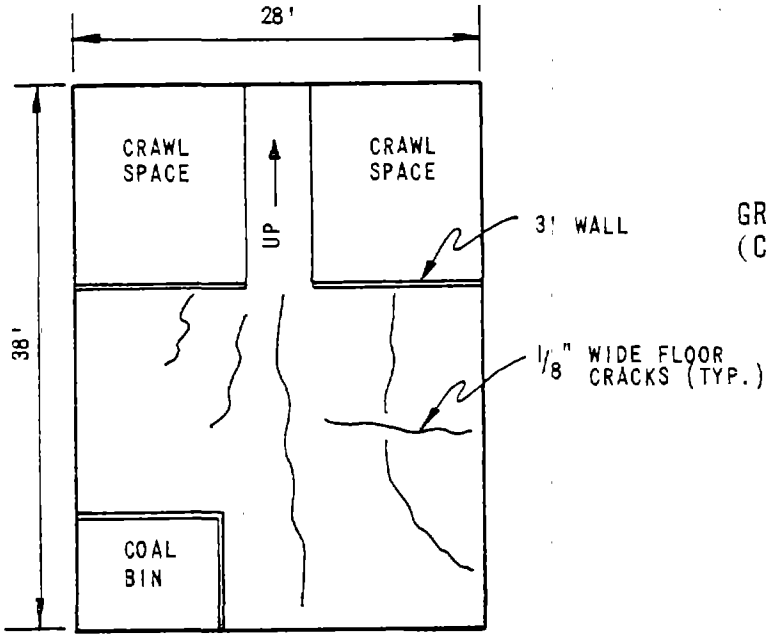
* NOTE:
INCIDENTS OF GENERAL TILTING

FIGURE 6.9 – BASEMENT CRACK PATTERNS, FREQUENCY OF OCCURRENCE



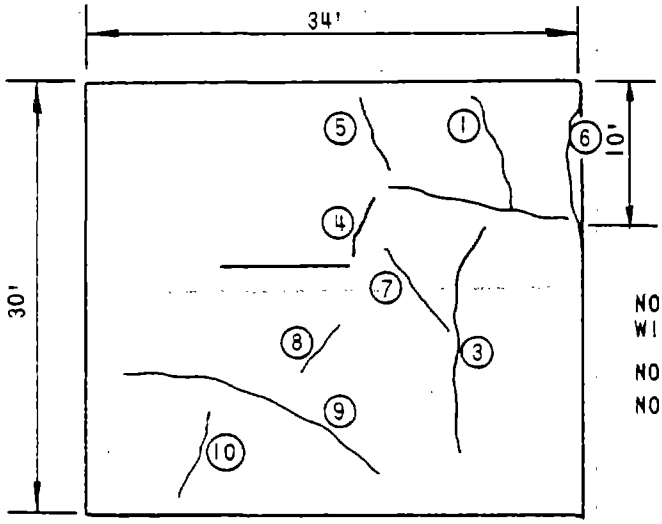
A.

ALIGNED CRACK PATTERN
(CASE NUMBER 441)



B.

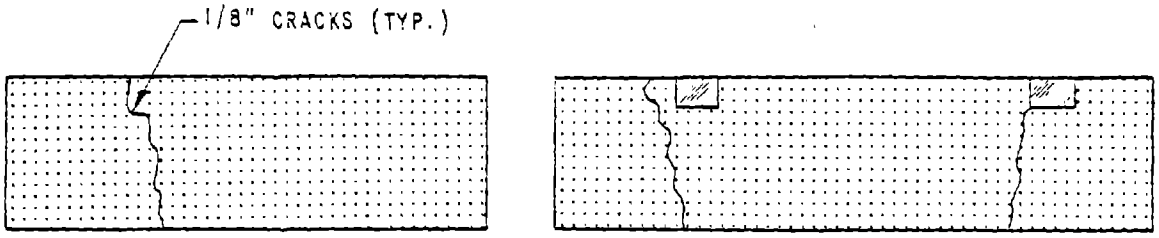
GRID CRACK PATTERN
(CASE NUMBER 152)



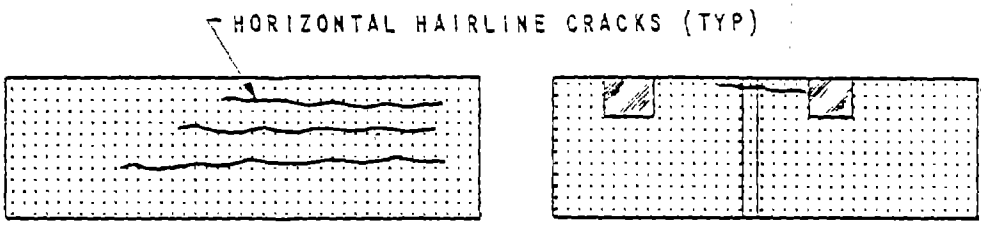
C.

RANDOM CRACK PATTERN
(CASE NUMBER 58)

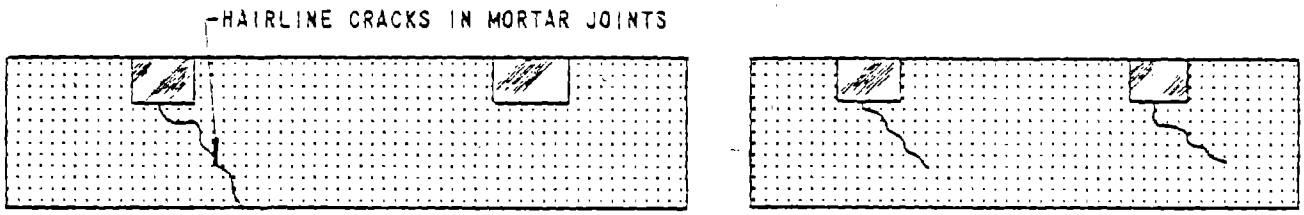
FIGURE 6.10- REPRESENTATIVE BASEMENT SLAB CRACK PATTERNS



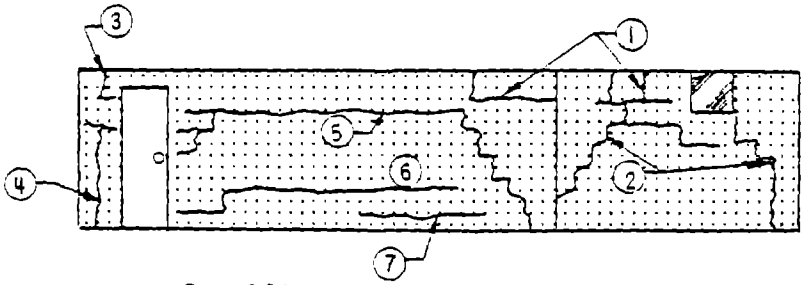
A. VERTICAL WALL CRACKS (CASE NUMBER 96)



B. LATERAL WALL CRACKS (CASE NUMBER 72)



C. DIAGONAL WALL CRACKS (CASE NUMBER 74)



D. COMBINATION (CASE NUMBER 30)

CRACK WIDTHS:

1	1/32"
2	HAIRLINE
3	1/4"
4	HAIRLINE
5	1/4"
6	1/8"
7	WALL BULGE

25.4mm = 1 inch

FIGURE 6.11- REPRESENTATIVE BASEMENT WALL CRACKS

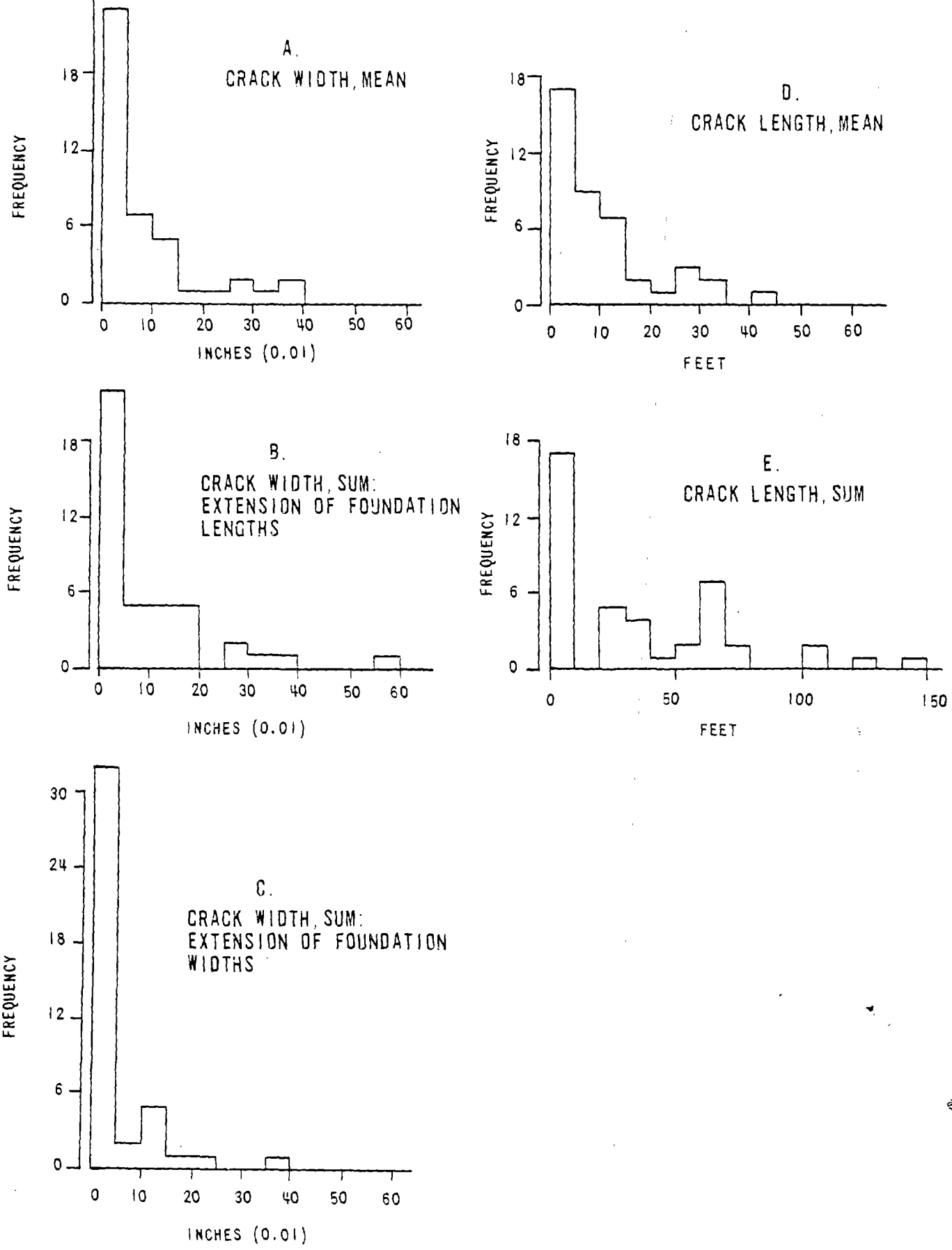


FIGURE 6.12 – CRACK DIMENSIONS, BASEMENT FLOOR SLAB FOLLOWING SUBSIDENCE

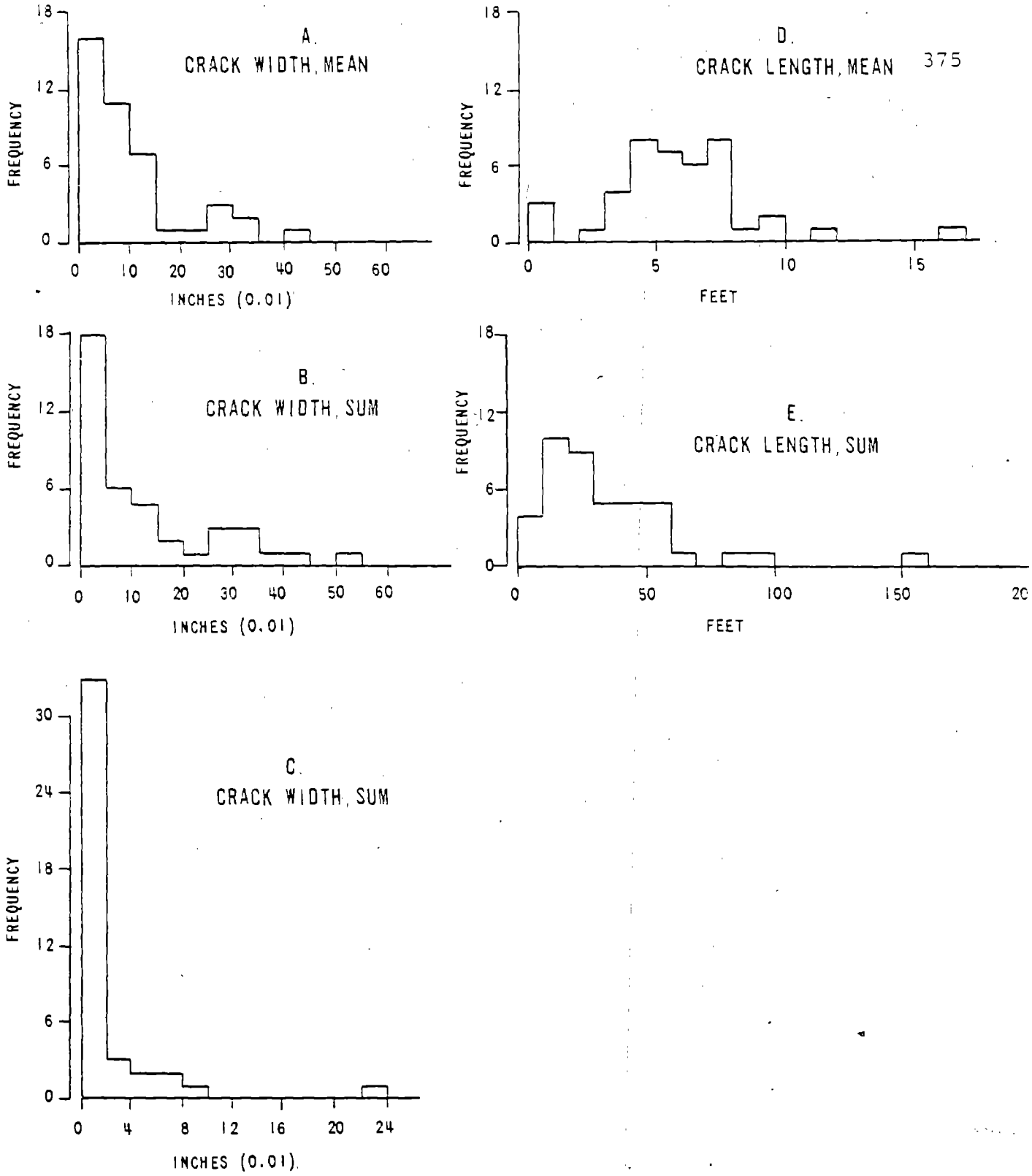


FIGURE 6.13 - CRACK DIMENSIONS, BASEMENT WALLS FOLLOWING SUBSIDENCE

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Generally, wall crack patterns appear to reflect a combination both of bending and shearing. In many cases, damage to walls that more or less parallel the direction of mining appears to be initiated by hogging and extension attendant with approach of the traveling wave. Shearing seems to become more important as the wave passes due to local loss of support for the footing. Then as the wave passes, additional cracking is produced through sagging and increased lateral earth pressure on basement perimeter walls in the compressive zone of the trough.

Bowing of walls may be associated with in-plane bending, with compressive ground movements, or with an interaction where the lower section of the wall, the footing and floor slab follow the curvature of the ground while the upper wall section is restrained by the superstructure (An example is presented in Case History 7). In addition, out-of-plane wall movements take the form of transverse translation displacements that are concentrated along particular masonry joints (Figure 4.20). Some out-of-plane wall movement is typical at moderate to high severity levels.

A net tilt after completion of mining was reported at 4 percent of the homes and could be inferred at several percent more where reference was made to interior doors that would swing to a particular angle, and so on.

Based on available documentation of damage, the following statements can be made regarding the effect of subsidence on building serviceability relative to the more important basic components:

a. Basement damage was more often functional or structural in nature than cosmetic. Functional damage occurred most often as: 1) loss of watertightness of basement walls due to cracking, or 2) differential settlement of the floor slab. Structural damage occurred primarily as loss of integrity of the bearing walls as a result of pronounced differential settlement and occasionally bowing of bearing walls, leading to local loss of support of the superstructure.

b. Interior damage most often took the form of cracking of the wall finish, slanting of floors, and misalignment of door frames between rooms, which occurred in nearly 55 percent of the damaged homes. Wallboard (drywall), commonly used in home construction since the early 1950's, seems to accomodate deformations somewhat better than conventional plaster and thereby reduces the tendency for extensive wall cracking. In drywalls the damage is essentially confined to the opening of joints.

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c. Exterior damage was generally cosmetic and took the form of cracking of joints between bricks or facing stone (reported at 50 percent of the homes having rigid facings) and separation of siding (reported at 3 percent of the homes having flexible facings). The slightly less common functional damage generally took the form of misalignments of windows and exterior doors (generally reported as broken windows or sticking windows and doors) and was reported at 33 percent of the homes having rigid exterior facings and 12 percent of those having flexible exterior facings.

d. In general, the types of damage observed appear to be the result of a three-dimensional interaction of the structure with the ground that cannot be explained simply in terms of extensions and compressions, as are often alluded to in the literature. Clearly, in-plane and out-of-plane flexure, shearing and racking can be significant modes of deformation in addition to extension and compression.

3. Damage Repair

The cost to restore a dwelling to its pre-subsidence condition is perhaps the most meaningful measure of severity from the viewpoints of the owner, the mining company, the government and insurance companies. It is certainly the most frequently quoted response to inquiries of damage severity.

Both the monetary settlement for damages and a damage estimate were available for the 134 homes. The distribution of monetary settlements is presented in Figure 6.14.

The amount of the monetary settlement in each case was influenced by a variety of factors apart from the actual amount of damage, among them:

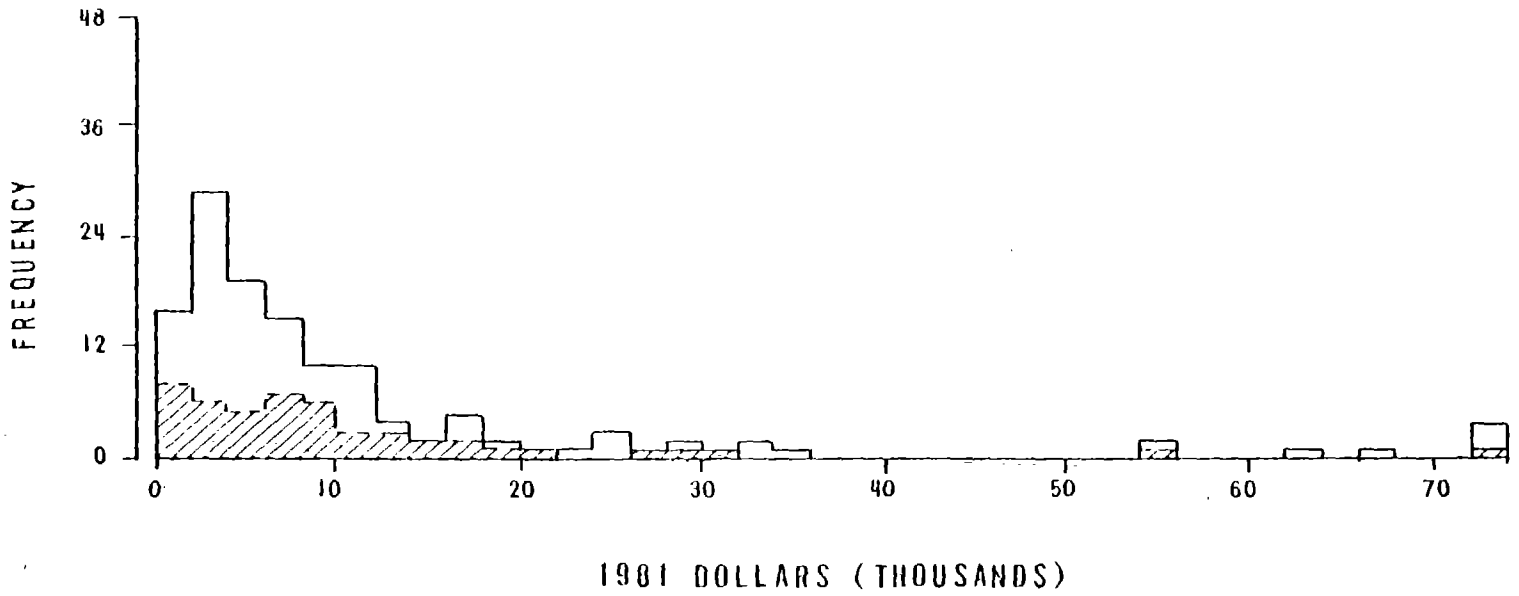
a. Intrinsic differences between homes,

b. The contractor's method of performing repairs and the conditions under which the repair contract was drawn up,

c. The economic climate and labor conditions at the time and place the work was performed,

d. The season of the year during which the work was performed; and

e. Site specific judgments concerning what was bona fide subsidence damage and what was not.



NOTE: Hatching denotes cases in damage severity model

FIGURE 6.14- SETTLEMENT COSTS OF MINE SUBSIDENCE DAMAGE TO HOMES, WESTERN PENNSYLVANIA

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Owing to these factors, reported monetary settlements appear to range more widely than observed variations in damage, thereby blurring distinctions between levels of damage severity. To better establish the costs of repairing the basement, the exterior and interior of the home, etc., independent repair cost estimates were, therefore, prepared by the writers.

To do so, the costs for returning the home to its pre-subsidence function and structural integrity were viewed from the contractor's perspective. It was recognized that the specific items of work a contractor must perform, including his labor and materials requirements, are more fundamental indicators of repair costs and hence damage severity than are detailed descriptions of each and every crack in the home. Further, it was recognized that basement repairs were necessary in virtually every case. Also, it was recognized that basement repairs generally represented the largest portion of the repair cost, and their implementation often lessened repairs to the superstructure by relieving strains in the superstructure, thereby closing major interior and exterior cracks and realigning window frames and doors.

To gain a realistic perspective several general contractors were posed a hypothetical question: "What criteria would govern repairs to the basement of their own home?" The general response was to maintain the basement corners within about 1/8 inch of square, and take corrective action if extension along a lineal dimension exceeds the limits summarized in the Basement Damage Repair Guidelines (Table 6.8). Typical types of repairs are summarized in Table 6.9 along with a few typical unit costs.

Distributions of estimated total, basement, interior and exterior repair costs for 38 homes having information sufficient for evaluation are shown in Figure 6.15. (The distribution of basement repair costs for all 50 homes is presented in Figure 6.16). An examination of these costs--that is, costs to repair principally the major functional and structural damage--reveals the following:

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Table 6.8

Basement Damage Repair Guidelines

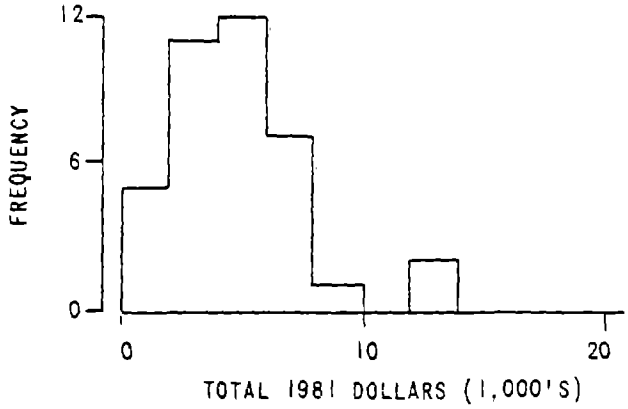
1. Examine the basement bearing walls and slab for damage.
   a. Record location, orientation and dimensions of cracks
   b. Record deformation
      (1) in-plane extensions and differentials
      (2) out-of-plane differentials
      (3) tilt
2. Evaluate damage with view toward potential replacement if:
   a. Extension ($\geq$.0005 length or height of wall, with continuous cracks extending half or none of the dimensions).
      (1) $\geq$0.25 inches, for slabs or lateral wall extension
      (2) $\geq$0.06 inches for vertical wall extension
   b. Differential settlement or tilt $\geq$0.001 span
   c. Excavate external perimeter of damaged wall if foregoing criteria are exceeded upon initial examination in order to further clarify damage and minimize repair costs
3. Minimize component replacement--utilize local cut-out technique
   a. replace walls by halves of straight unjointed sections
   b. replace slabs by quadrants of the major rectangle or sections adjoining thereto
4. Repointing/pointing and sealing
   a. wall(s); applies to non-replaced, straight, unjointed sections
      (1) when any wall or portion thereof is replaced, repoint the remainder of the walls.
      (2) when no wall or portion thereof is replaced, repoint the walls exhibiting cracks
   b. slab; applies to non-replaced quadrants of the major rectangle or sections adjoining thereto

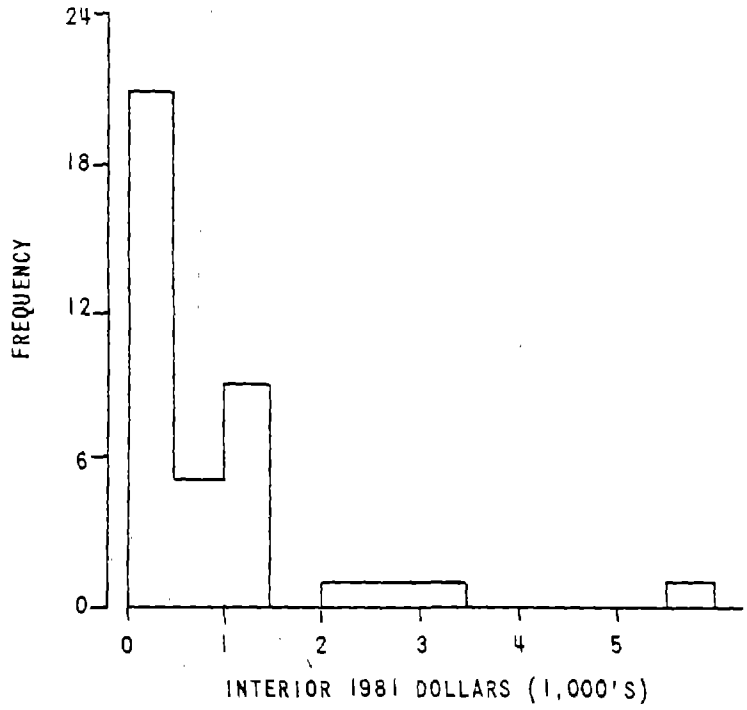
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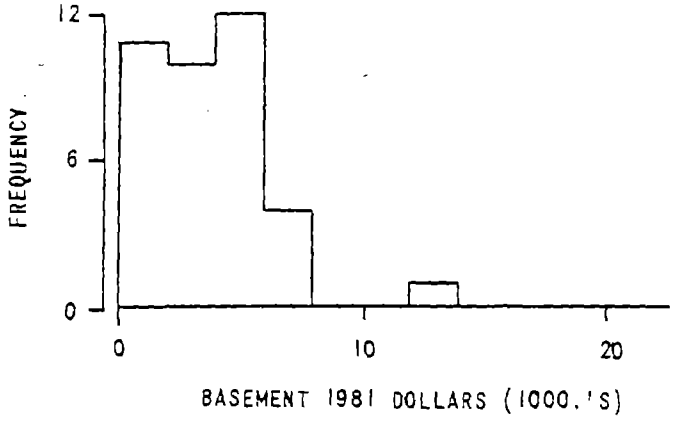
Table 6.9

Typical Basement Repairs and Unit Costs
(1981 Dollars; Englesman, 1981)

Excavation and backfilling - $272.00/day
Repointing - $0.99/ft^2
Sealing or water proofing foundation - $0.42/ft^2
Laying block - $2.73/ft^2
Cutting out block - $1.05/ft^2
Cutting out footing - $6.23/ft
Constructing footing - $4.09/ft
Cutting out 4" concrete slab - $2.80/ft^2
Constructing 4" concrete slab - $2.03/ft^2
Removing, fitting and reinstalling exterior door - $37.57
(Frames included $175.00)
Removing, fitting and reinstalling interior door - $22.29
Spackling drywall - $0.15/ft^2
Removing and reinstalling paneling - $1.57/ft^2
Adjusting trim - $.50/ft
Adjusting paneling - $0.50/ft^2
Painting wall (plus ceiling) - $0.25/ft^2
Painting ceiling only - $0.18/ft^2
Removing and replacing drywall - $0.46/ft^2 ($0.58/ft^2 ceiling)
Removing and reinstalling basement window - $29.36
Replacing basement floor drain - $19.60/ft
Hauling debris - $1.55/yd^3
Disconnecting/reconnecting outlets - $27.00
Refitting dry wall - $0.055/ft^2
Cutting out brick - $0.95/ft^2
Laying brick - $3.47/ft^2







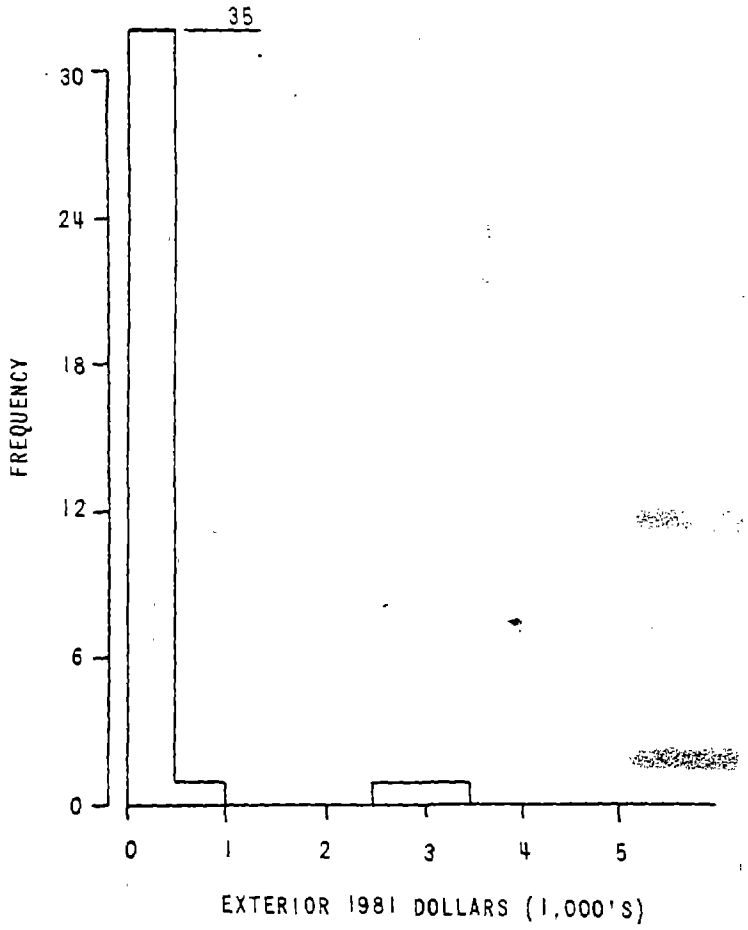


FIGURE 6.15 - TOTAL,BASEMENT,EXTERIOR AND INTERIOR REPAIR COSTS

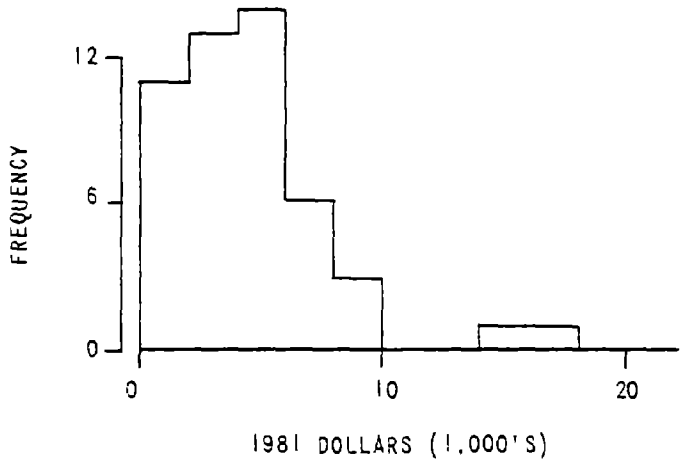


FIGURE 6.16 - BASEMENT REPAIR COST

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a. These 38 cases, although only approximately representative of the larger population of 134, indicate that the total repair cost of the home is generally less than 20 percent of its replacement value and is commonly about 10 percent (Figure 6.17). There is a paucity of data in the positive tail of the distribution. However, review of the 134 subsidence cases suggests that probably no more than 10 percent would fall in the total repair cost range of 20 to 100 percent of the replacement value of the home.

b. Basement repair costs ranged from $200 to $12,200 in the 38 cases under discussion and generally represented the major portion of total repair costs. They exceeded 90 percent of the total repair cost in two-thirds of the cases and half the total repair cost in almost 90 percent of the cases. Outside the immediate group of cases under discussion, basement repair costs reached as high as $20,200.

c. Interior repair costs were less than basement repair costs in nearly 90 percent of the cases; they were less than half of basement repair costs in about 80 percent and were negligible in approximately 40 percent. Homes where interior repairs equalled or exceeded basement repair costs were associated with comparatively small amounts of basement damage. Interior repair costs do not seem strongly related to the basement repair costs.

d. Exterior repair costs were less than 15 percent of basement repair costs in about 85 percent of the cases and in no case exceeded 70 percent. They are negligible in nearly 60 percent of the cases. Exterior repair costs do not seem strongly related to basement repair costs.

Damage reparations associated with restoring a home to an appearance free of subsidence effects and acceptable to the homeowner can be significant. These costs, which are separate from, and not closely aligned with, the functional/structural damage repair costs discussed above, can vary depending on style and quality of construction, accouterments, neighborhood, etc. They depend on a host of subjective, sometimes nebulous, site specific factors that cannot be accounted for as rationally as the repair of functional and structural damage. These aesthetic/cosmetic/decorative expenses not infrequently add another 15 to 30 percent to the total repair cost.

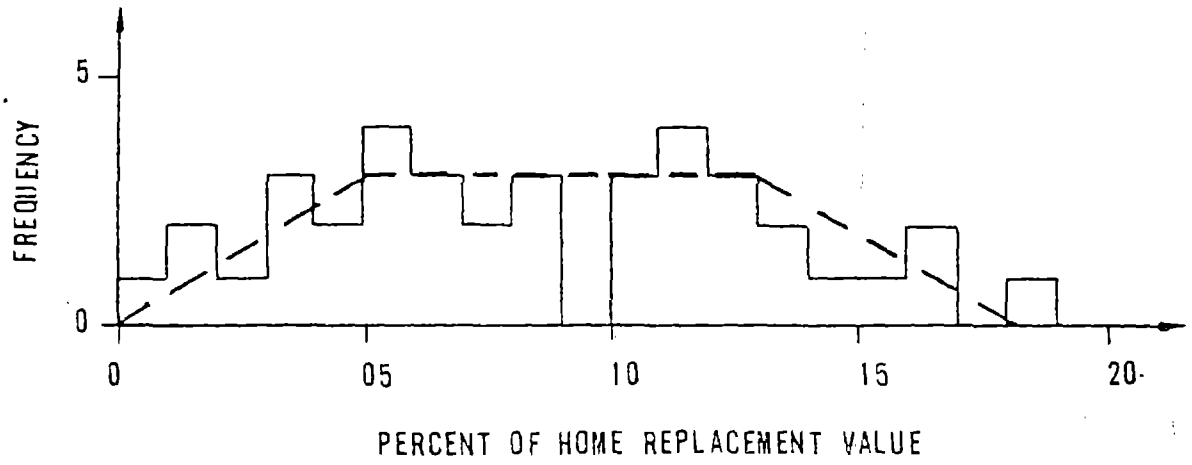


FIGURE 6.17- TOTAL ESTIMATED REPAIR COST AS A FRACTION OF HOME REPLACEMENT VALUE

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Severity Index

Attention thus far has been directed toward a description of the damages and repair costs of homes damaged by subsidence. The discussion has not yet concerned itself with quantifying the level of damage on a severity scale, which would afford the opportunity to statistically evaluate the relative importance of geologic, mining, site and structural factors that influence the severity of damage to homes.

As has been discussed, repair costs are almost always dominated by basement damage, as are most hazardous conditions that might develop. Basement construction is simple and generally varies little from one home to another. Size is the principal variant. For this reason, the writers have indexed the damage severity level against basement repairs, using the relationship:

$$\text{Severity Index} = R_P + 4\,[R_B A + R_S(1-A)],$$

where R_P = fraction of basement perimeter walls repointed. $R_p = 1$, if either R_B or R_s are greater than zero;

R_B = fraction of the basement bearing walls replaced ($0 \leq R_B \leq 1$);

R_S = fraction of the basement floor slab replaced ($0 \leq R_S \leq 1$);

$$A = \frac{\text{Cost of basement bearing wall replacement}}{\text{Cost of basement bearing wall and floor slab replacement}}$$

The Severity Index varies continuously between 0 and 5, the level of damage increasing toward the high end of the scale.

Figures 6.18A-C present the frequency distributions of: 1) the length of basement perimeters, 2) the length of bearing walls and 3) parameter 'A' (defined above) of 53 homes for which the Severity Index could be established. Figures 6.19A-C present the corresponding distributions of basement repair requirements in terms of percentages of: 1) basement perimeter wall repointing, 2) basement bearing wall replacement and 3) basement slab replacement.

Severity Index Model Sample Selection

Regression analyses of the Severity Index were conducted relative to structure, site, mining and geologic parameters pertaining to those 50 cases of the 53 above, for which necessary information could be obtained. Figures 6.2 and 6.5 (earlier in this section) present the distributions of these parameters for the sample of 50 (shaded), as well as for the general population (unshaded).

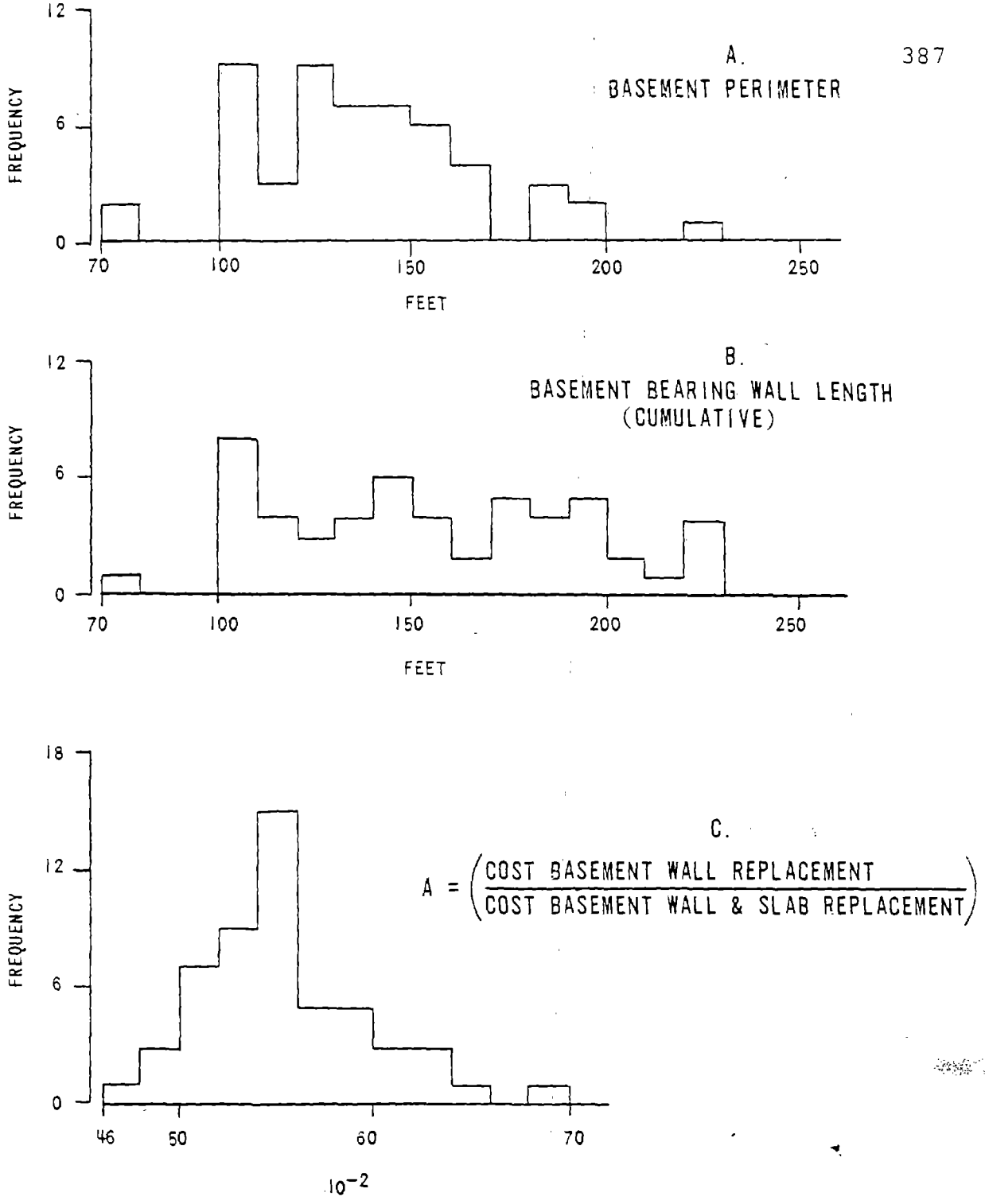
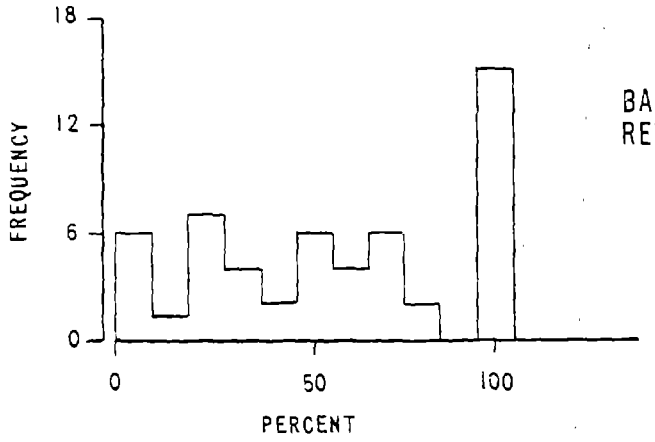
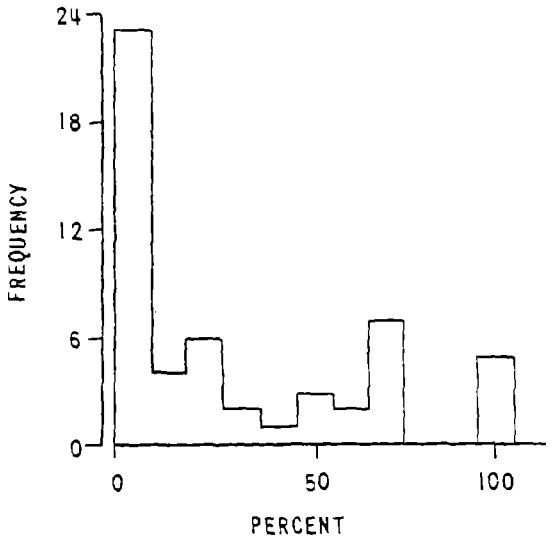


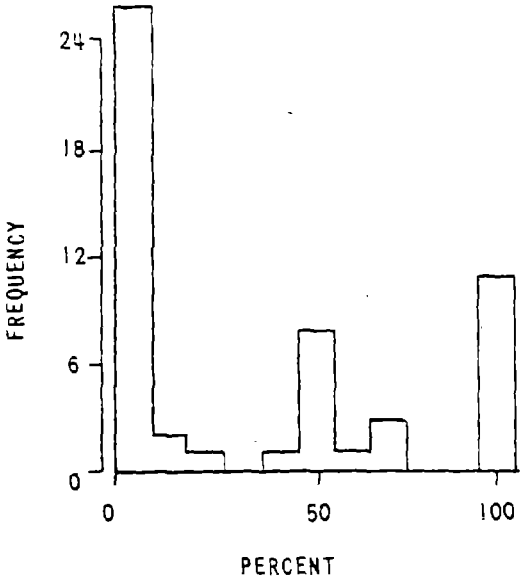
FIGURE 6.18 - BASEMENT PERIMETER, BEARING WALL LENGTH AND REPLACEMENT COST STATISTICS



A.
BASEMENT PERIMETER WALL
REPOINTING PERCENTAGE



B.
BASEMENT BEARING WALL
REPLACEMENT PERCENTAGE



C.
BASEMENT SLAB REPLACEMENT
PERCENTAGE

FIGURE 6.19 - BASEMENT WALL REPOINTING AND REPLACEMENT,SLAB REPLACEMENT

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To achieve a distribution of coal seams for the sample that was similar to that of the general population, it would have been necessary to reduce the sample to roughly 25 cases. Such a small sample would surely have reduced the attainable level of confidence in the model to far below an acceptable level. Therefore, the sample of fifty cases was accepted for development of the Severity Index model.

Damage Relative to the Severity Index Model

The following paragraphs present for the sample of 50 cases the relationships found between the Severity Index, described above, and 1) Locations of Damage within Homes, 2) Hazard to Occupants, 3) Deformation and Cracking and 4) Repair Costs.

1. Locations of Damage within Homes

Table 6.10 presents severity statistics of the combinations of damaged building components. The frequency distributions with respect to severity are presented in Figure 6.20 for the basement, interior and exterior as they relate to the rigidity of the exterior facing. Figure 6.21 presents cumulative probability distributions of these components. The data for the sample of 50 indicate that:

a. Damage to the interior of a home is reported somewhat more frequently than to the exterior at any particular level of severity, and interior and exterior damage are distributed similarly to basement damage, but at lesser frequencies.

b. Damage to the pavement and garage appear to be distributed similarly to basements, also at lesser frequencies.

c. Damage to the roof is reported rather infrequently and appears to be associated with Severity Indices in the neighborhood of 3.5 or higher.

Because no two homes or site conditions are exactly alike, the types and locations of damage in each home of a group of very similar homes could vary considerably. A higher level of severity can be associated with no more than moderate damage to many basic components or with severe damage to a few basic components. There is no reason to expect the same damage at each and every home at a given severity level.

2. Hazard to Occupants

The hazard to occupants of a subsidence-damaged home is almost nil when the Severity Index is less than one--when repairs are confined essentially to repointing mortar joints in basement walls. In contrast, when the Severity Index is close to five, any house is likely to pose some sort of hazard to its occupants--perhaps, one or more severely bowed basement walls nearing collapse. At a Severity Index of four, possibly

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Table 6.10

Distinct Combinations of Damaged Basic Building Components

Combination Number	Damaged Components								Total Population (134)	Model (50)	Severity Index Mean	Severity Index Standard Deviation
1	B								25	6	2.10	1.03
2	B	E							3	2	4.01	0.49
3	B		I						17	12	1.39	0.90
4	B	E	I						10	2	2.36	2.38
5	B	E	I	R					1			
6	B		I		C				2	1	2.25	
7	B	E	I		C				2	1	1.66	
8	B					G			11	6	1.72	1.22
9		E				G			1			
10	B	E				G			6	1	1.24	
11	B		I			G			5			
12	B	E	I			G			3	1	1.72	
13			I		C	G			1			
14	B	E			C	G			1			
15	B	E	I		C	G			2	1	0.53	
16	B	E	I				U		1			
17	B	E	I	R			U		1			
18	B					G	U		1	1	1.96	
19	B	E	I	R		G	U		1			
20								P	1			
21	B							P	8	3	1.94	1.25
22	B	E						P	2	1	2.68	
23	B		I					P	6	4	1.86	1.13
24		E	I					P	2			
25	B	E	I					P	6	3	4.18	1.41
26	B	E	I		C			P	1	1	1.88	
27	B		I			G		P	4	1	3.93	
28	B	E	I			G		P	4	1	1.79	
29	B	E	I	R		G		P	1	1	3.94	
30	B		I		C	G		P	1			
31	B		I				U	P	1			
32	B	E	I				U	P	1	1	2.36	
33	B	E	I			G	U	P	1			
34	B	E	I	R	C	G	U	P	1			
Basic Building Components Damaged Alone or With Others												
B - Basement									129	50	2.08	1.25
E - Exterior									51	16	2.69	1.47
I - Interior									75	30	2.05	1.33
R - Roof									5	1	3.94	
C - Chimney									11	4	1.58	0.74
G - Garage									42	13	1.96	1.23
U - Utility									8	2	2.16	0.28
P - Pavement									40	16	2.65	1.34

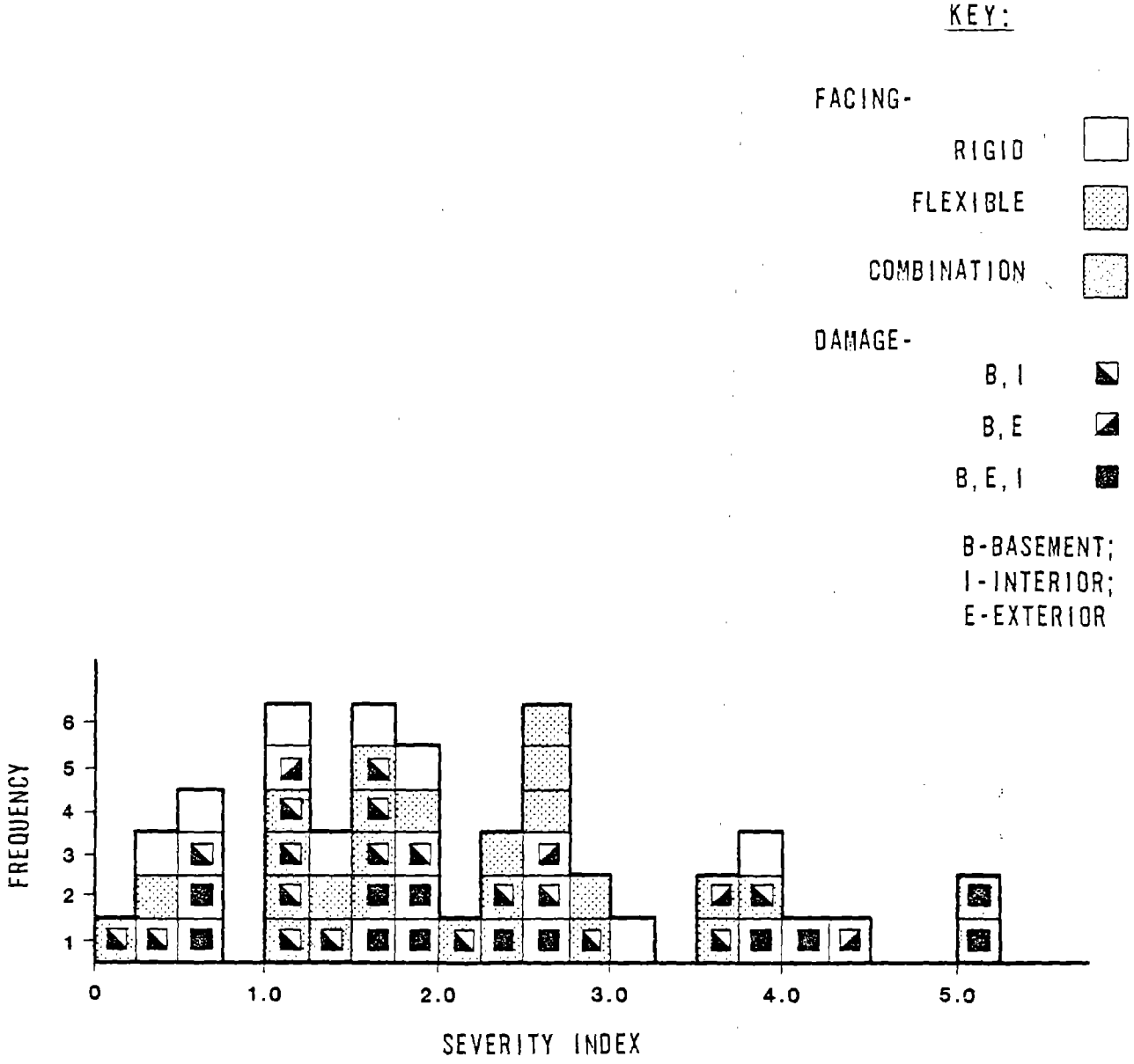


FIGURE 6.20 - COMBINATIONS OF HOME DAMAGE AND FACING IN RELATION TO SEVERITY INDEX

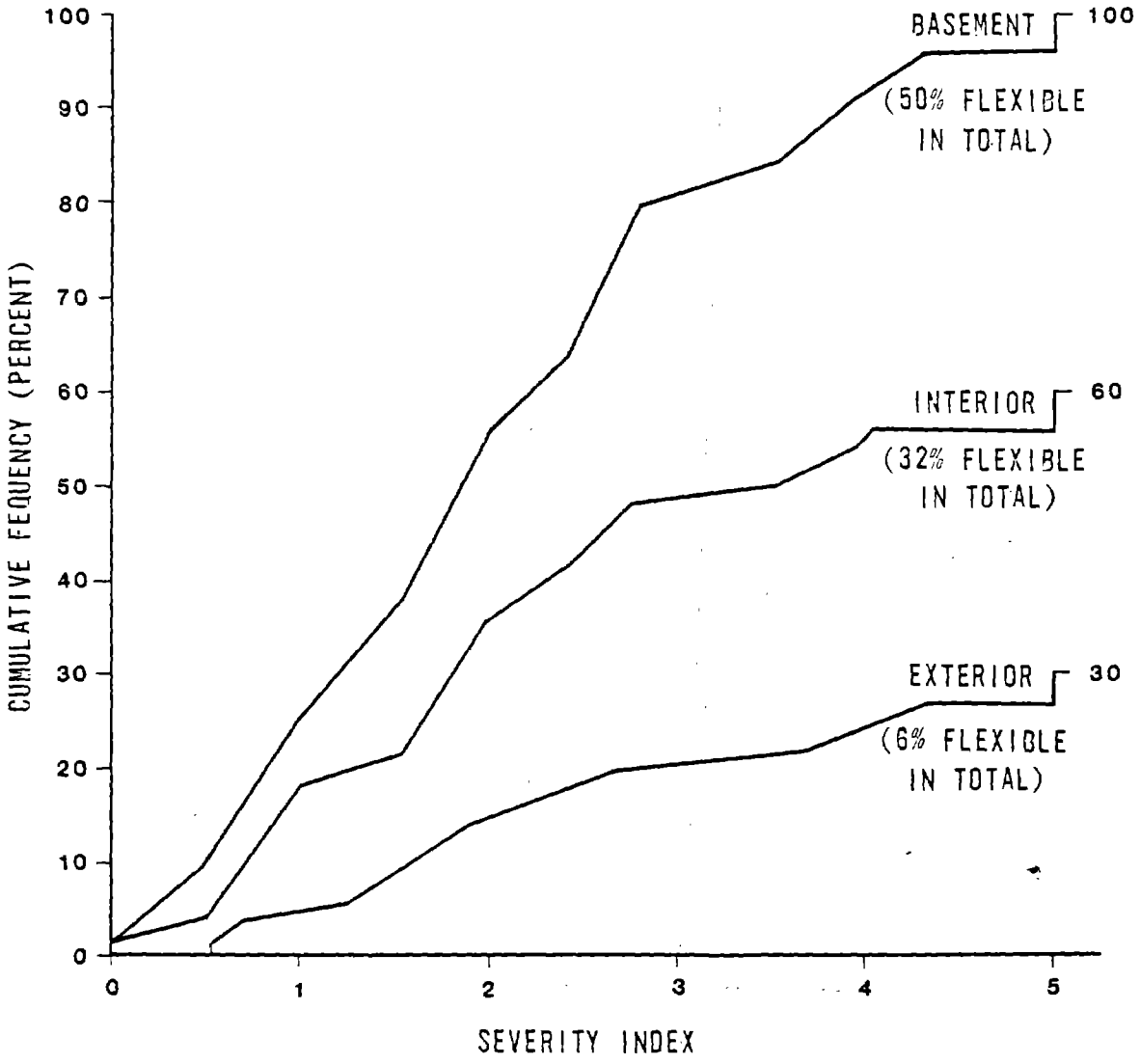


FIGURE 6.21 - CUMULATIVE FREQUENCY DISTRIBUTIONS OF THE MAJOR DAMAGED BUILDING COMPONENTS RELATIVE TO THE SEVERITY INDEX

only one out of every three homes poses a hazard to its occupants. Until a more comprehensive body of data is collected, it seems reasonable to assume that the frequency of hazardous conditions increases with the Severity Index and is distributed more or less linearly between these three points.

3. Deformation and Cracking

Crack dimensions together with crack patterns provide a rough estimate of the Severity Index through the empirical equation:

$$SI = 1.10 + 0.00058\ CA_S + 0.00076\ CA_W + CP$$

where CA_S is the crack area of the basement floor slab, in square inches;

CA_W is the crack area of the basement walls, in square inches;

CP is the crack pattern of the floor slab (random, 0.14; grid, 0.75 and aligned, 1.53)

Crack area is defined as the sum of products of mean crack widths and lengths. The Correlation Coefficient is 0.80; the standard error is 0.61, and the confidence level exceeds 99 percent.

4. Relationship Between Damage Severity and Repair Cost

The basement repair cost is more conveniently expressed in terms of parameters used to define the Severity Index (basement dimensions, type of basement repairs) than in terms of the Severity Index itself. On this basis, the estimated basement repair cost is given by the following empirical relationship:

$$C = 9.87\ PR_p + 39.45\ BR_B + 4.83\ SR_S + 544.E + M$$

where

C = basement repair cost, 1981 dollars

P = basement perimeter walls (sum of lengths), feet

B = basement bearing walls (sum of lengths), feet

S = slab area, square feet

R_P = fraction of basement perimeter repointed, not including perimeter walls reconstructed

R_B = fraction of basement bearing walls reconstructed

R_S = fraction of basement floor slab replaced

E = excavation requirement,

= 1. if R_P or R_B or R_S >0.

M = Miscellaneous costs such as door adjustments

Damage to the interior may add another $500 to $1500 to repair costs, and damage to the exterior, a few hundred dollars to perhaps $1000. Within these ranges, both exterior and interior repair costs tend to be higher for rigidly faced homes than for flexibly faced homes, commonly by a factor of 2 or 3. Miscellaneous costs, such as door adjustments, are generally less than a few hundred dollars.

Together these figures yield a rough estimate of total repair costs, with a standard error of $1,638, a Correlation Coefficient of 0.82 and an F-Statistic of 73.9, with a confidence level much better than 99 percent.

Additional costs associated with repair of a roof, porch, chimney, attached garage or utility must be evaluated on a site specific basis, since relatively few cases of such damage have been reported. Moreover, an allowance must be made for the aesthetic/cosmetic/decorative expenses. One might add another 20 percent to the repair cost.

The foregoing discussion has described subsidence damage in homes typical of the Northern Appalachian Coal Province (particularly western Pennsylvania), the frequency with which damage affects various portions of these homes, and the cost to repair damages. The concepts are summarized in the context of a damage classification system described next.

Subsidence Damage Classification System

Table 6.11 portrays a subsidence damage classification system applicable to homes affected by subsidence above active mines in the Northern Appalachian Coal Field, based on data collected during the present study. Its immediate utility lies in summarizing the types of subsidence damage to homes that are characteristic of the region. Its further utility lies in providing a uniform, consistent method for specifying the level of subsidence damage to a typical home when used in conjunction with other information provided in this section of the report.

Several comments are in order concerning the classification system:

1. The system is based on damage to the basement. Accompanying damage to the superstructure may vary depending on the actual

Table 6.11

SUBSIDENCE DAMAGE CLASSIFICATION
NORTHERN APPALACHIAN COAL FIELD

Class	Characteristic Basement Damage	Severity Index
I Slight	o Hairline cracks in one or more basement walls and possibly floor slab. o Some cracks in perimeter walls causing loss of water tightness. o Repointing required in some or all walls.	0
II Moderate	o Cracks in one or more basement walls and floor slab. o Some wall/footing reconstruction and floor slab replacement required, as well as local repointing.	1
III Severe	o Cracks in one or more basement walls and floor slab. o Possible wall instability and loss of superstructure support, requiring shoring and bracing. o Extensive repair work involving wall/footing reconstruction and floor slab replacement.	2
IV Very Severe	o Cracks typically in all basement walls, as well as floor slab. o Possible instability of several walls and loss of superstructure support, requiring extensive shoring and bracing. o Possible significant tilt to home. o General reconstruction of basement walls, footings and floor slab required.	4 5

displacements of the basement walls, the type of building construction and other factors. Available data suggest the probability of damage to a particular building component is more or less the same at any severity level, although the damage to that component may be more serious the higher the severity level. Generally, damage to the superstructure would be moderate at most, regardless of the severity of basement damage. Moderate damage might include cracks in interior walls; cracks in external brickwork that require repointing and possibly a small amount of brief replacement; sticking doors and windows; some loss of watertightness. Utility pipes leading to the home may rupture.

2. The four classes of severity in the present system are not necessarily comparable to classifications developed abroad. The latter focus on damage to the building interior and exterior--considerations that are clearly secondary to basement damage in western Pennsylvania and much of the United States. Because of such differences, it is generally inappropriate, if not meaningless, to discuss damage to U.S. homes in terms of a classification such as the NCB, or to attach much significance to predictions of severity based on ground movement/ damage relationships developed outside the United States.

3. Classes of severity cannot be specified according to particular crack widths or particular numbers of cracks. A rough relationship exists between Severity Index and crack area and pattern has been presented earlier in the text.

4. The four classes of damage severity are composites based on subsidence from a variety of mining types. The types, sequences and amounts of ground surface deformation that produce a particular level of damage are as yet undefined.

It is recommended that a series of homes be monitored during mining to establish more precisely the onset of important stages in the deformation and damage process. Damage should be correlated with ground surface deformation (principal displacements and strains) over a variety of mining types to establish severity relationships that can be used for predictive purposes and will enhance the usefulness of the classification system.

Evaluation of Structural, Site, Geologic and Mining Factors that Influence the Level of Subsidence Damage as Measured By the Severity Index

Given the measure of severity proposed above, and the best available sample of data (fifty cases of subsidence over active mines), a series of multiple linear regression analyses were conducted to identify the relative influence of particular structural, site, geologic and mining factors on the severity of subsidence damage to homes expressed in terms of the Severity Index. Each site was characterized by 22 parameters--13 pertaining to the structure, 3 to the site, 2 to geology and 4 to mining:

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Age of home when subsidence took place

Foundation type

Basement embedment configuration

L - Basement length

W - Basement width

LW - Basement length times width

RI - Cross sectional moment of inertia of basement bearing walls

Basement Slab Area

Deflection ratios, basement walls (four terms)

w - Lineal subgrade load due to weight of structure

SF - Shape factor, plan configuration of superstructure

Facing type, home exterior

Net tilt

D - Angle between house and adjacent mining

Topography

O - Overburden thickness

Soil Type

Coal seam

Mining type - beneath site and adjacent to site

M - Mined height

M/O

M/O^2

Through a stepwise regression analysis, variables were eliminated until the remainder achieved a confidence level of 90 percent. Table 6.12 presents the regression coefficients and other pertinent statistics of the models. Figure 6.22 compares the actual, post-subsidence Severity Indices for the 50 cases with those predicted from the pre-subsidence structure, site, geologic and mining parameters.

Table 6.12

THE SEVERITY INDEX MODEL

Function	Severity Index Increment	Regression Coefficient	t*[1] Statistic
Coal seam, if other than the Pittsburgh	+0.84		1.94
LW, rectangular area encompassing the basement (feet^2)		+0.0219	3.83
M, mined height (feet)		+0.5420	2.01
M/O, ratio of mined height to overburden thickness		-333.4	3.39
(M/O)E, above times the extraction percentage below the site		+2.094	1.79
Mining type below, iff:			
retreat, full	+0.00		
retreat, pillars split and notched	-1.69		1.68*[2]
retreat pillars split	+0.53		
barrier	+4.08		
advance or longwall	+1.28		
Mining type adjacent, iff:			
retreat, full	+0.00		
retreat, pillars split and notched	+1.26		1.68*[2]
retreat, pillars split	-1.02		
longwall, advance or unknown	-2.88		
O, overburden thickness (feet)		-0.009692	3.24
SF, shape factor (0.00 to 0.07 at irregular)		-16.58	1.69
Tan D, where D is the angle to adjacent mining		-0.05967	1.72
Topography, iff: other than flat, toe of slope or a valley bottom	-0.81		2.02
W, basement width (feet)		-1.569	3.84
$wL^3/[RI(E)]$ Deflection ratios		+357.7	3.00
$wL^3/[TT(G)8']$ due to bending		-257600.0	4.15
$wW^3/[RI(E)]$ or shear for		-1357.0	3.00
wW/[TT(G)8'] bearing walls of plan length or width (dimensionless)		+367400.0	4.13
Pertinent Statistics			
Intercept	+26.20		
Standard Error:	0.77		
Correlation Coefficient:	0.89		
F-statistic (23,26):	4.54		

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Table 6.12
(continued)

Confidence Level of Model: >>99%

*Notes: 1) The confidence level for individual independent variables is 90 percent at a t-statistic of 1.68.

2) The t-statistic exceeds 1.68 for all groups of mining types considered in the analysis (2.30 to 4.7 for mining type below and 2.04 to 2.52 for mining types adjacent).

3) iff = if and only if.

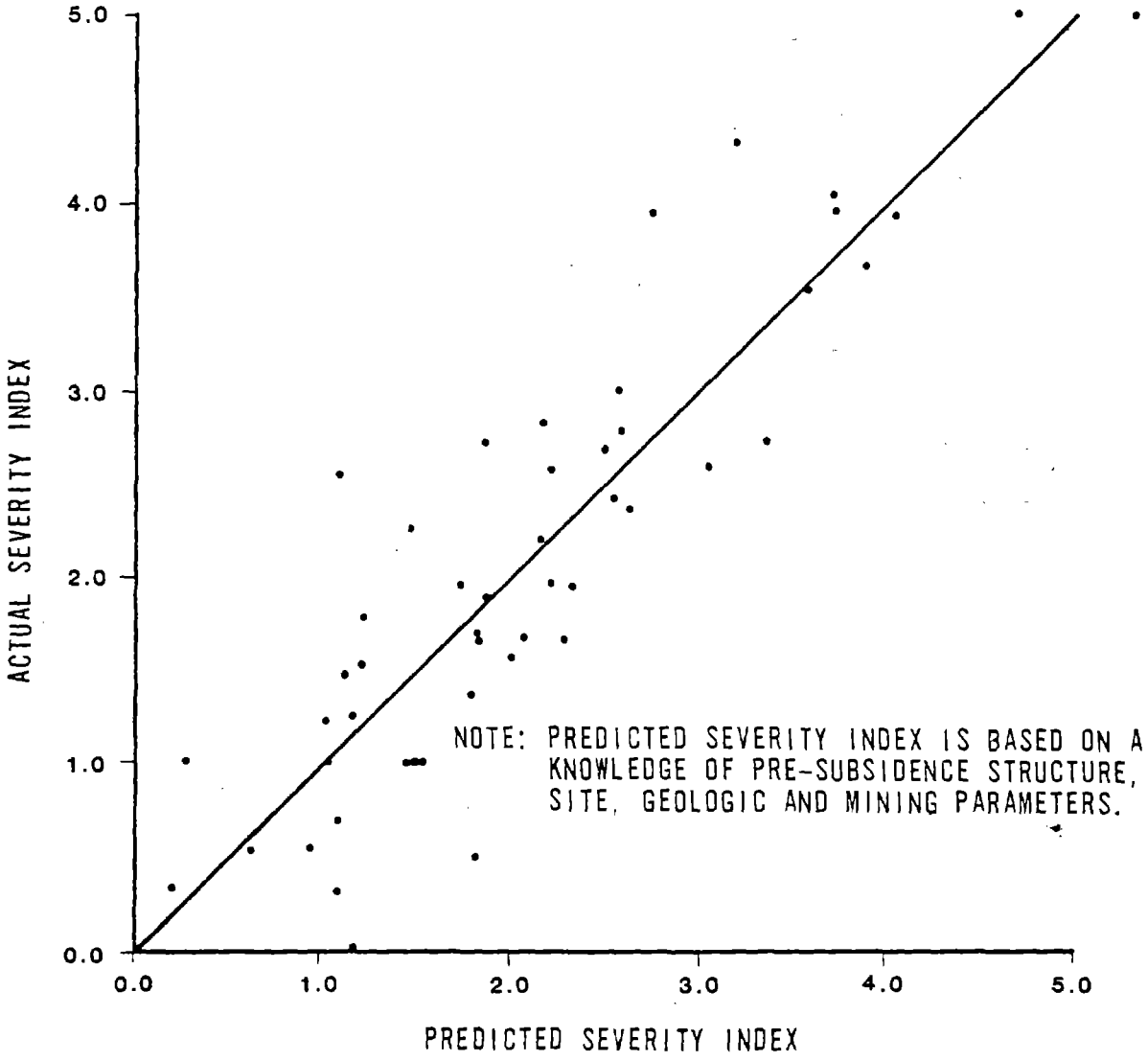


FIGURE 6.22 - COMPARISON OF ACTUAL AND PREDICTED SEVERITY INDEXES

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The regression analysis suggests the following trends for each of the four groups of parameters:

1. Structure--Collectively, the deflection ratios, shape factor, basement width and rectangular area encompassing the basement seem to exert only a moderate influence on the Severity Index for the homes included in the sample--the Severity Index increasing only 1.32 from a 25x45 foot brick-faced rectangular ranch to a 45x50 foot L-shaped, split level home with combination facing. The increase in Severity Index with plan area of home is as expected.

2. Site--The Severity Index decreases as the overburden thickness increases, also as expected. The rate of decrease is 0.0097 per foot over the range of 150 to 795 feet. The Severity Index is about 0.8 lower if the site is located on a ridge, a hillslope or some other elevated area, rather than on an alluvial flat, a valley floor or at the toe of a slope. This appears to reflect differences in soils (types, properties, thicknesses) as much as it does differences in topography. Ridges, hillslopes and other elevated areas tend to be covered with residual, and sometimes colluvial, soils. Ninety percent of the 28 such sites have residual or colluvial soils at ground surface. These soils are usually of limited thickness so that foundations for homes are placed directly on the underlying rock, which is commonly claystone or shale. Conversely, homes on flats (flood plains) are typically located on alluvial deposits, whose substantial thickness necessitates placement of the foundations on soil--soil that is commonly erratic in density and may be soft or loose a short distance below ground surface. Sites located on valley floors or at the toes of slopes may be founded partially or totally on thick residual soil, colluvium or alluvium. Indeed, eighty percent of the 22 "low" sites located on flats, valley floors or at toes of slopes contain alluvial soil, and the remainder, residual or colluvial soil.

3. Geology--The extremely limited availability of site specific stratigraphic information permits only an implicit consideration of possible geologic influences on the Severity Index expressed in terms of coal seam. The regression analysis indicates that the Severity Index is 0.84 higher for geologic scenarios involving other than the Pittsburgh Coal.

   A supplementary regression analysis based on eight cases above the Pittsburgh Coal for which drilling logs were available suggests that the presence of competent strata in the overburden leads to a lower Severity Index (Table 6.13). The very simple model does not account for position of the competent strata in the overburden. Cantilevering of competent strata can lead to permanent

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Table 6.13

THE SEVERITY INDEX RELATED
TO COMPETENT ROCK STRATA IN OVERBURDEN

Function	Regression Coefficient	t-Statistic
E, extraction percentage for mining beneath the site	0.007296	6.88
LW, Rectangular area encompassing the basement ($feet^2$)	-0.001385	2.09
O_{RH}, cumulative thickness of competent rock strata (feet)	-0.008083	2.89
M/O, ratio of mined thickness to overburden thickness	123.3	3.03
Intercept	= 4.53	
Correlation Coefficient	= 0.98	
Standard Error	= 0.36	
F-Statistic (4, 3)	= 18.94	
Confidence Level	> 95. percent	

Note, the 90 percent confidence level is achieved for each of the independent variables at a t-statistic of about 1.90.

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tilting of a home, which is expensive to repair and is characteristically associated with a high Severity Index. Geologic factors deserve closer scrutiny in future research.

Soil in combination with topography is believed to exert some influence on the Severity Index, as pointed out above. However, the influence appears to be much less than that of other parameters. Soil type, as a separate parameter, was eliminated from the regression analysis because of a low confidence level.

4. Mining--The combined effect of the mining parameters is for the Severity Index to increase with mined height and extraction ratio and to decrease with overburden thickness. This is as expected. For a limited range of cases pertaining to the Pittsburgh Coal at moderate depths, these trends are reversed. The effect is minor and can probably be attributed to the relatively small sample used for analysis and the coarseness of the analysis.

The regression analysis further suggests that subsidence damage associated with advance mining, retreat mining with pillars split or slabbed, longwall mining or movements above a barrier pillar will be more severe than damage associated with full extraction retreat mining. Damage promises to be less severe if associated with retreat mining where pillars are split and notched.

In interpreting these results, it is critical to appreciate that subsidence, in fact, does not occur above the majority of sites where partial extraction mining is practiced, and ordinarily structures at ground surface are not damaged. The regression relationship, on the other hand, considers cases only where subsidence damage has occurred. The increased severity associated with certain types of partial extraction mining is very probably linked to pronounced differential settlements that accompany failure of pillar support. Because the character of such subsidence is site-specific and does not follow a prescribed, predictable pattern, no further comment can be made on this issue. Quite possibly the comparative damage severities observed at this time for particular below-site mining types will change as more data are collected and analyzed. The same is probably true for adjacent mining. In the cases included in this analysis, mining below the site generally governed the severity, with adjacent mining exerting a lesser influence.

Total Repair Cost Prediction

Based on the foregoing considerations, a rough estimate of total repair costs excluding aesthetic/cosmetic/decorative expenses

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can be estimated for a home prior to subsidence, given the Severity Index that has been predicted from structure, site, mining and geologic parameters:

$$T = C_o + C1(SI)P + C_2P + C_3(SI)B + C_4B + C_5(SI)S + C_6S + C_7(EI) + C_8(SI),$$

where P = perimeter of basement walls

B = basement bearing walls, sum of lengths

S = slab area, basement floor

SI = Severity Index

EI = exterior and interior superstructure repair costs given the exterior facing type

C = regression coefficient as per Table 6.14

The model assumes that simple linear relationships exist between the Severity Index and the three parameters: perimeter wall repointing, basement slab replacement and basement bearing wall replacement. The parameters having low t-statistics were retained in the model because their removal would not significantly reduce the standard error. Clearly, both the model and the input data require refinement. This is recommended for future efforts.

Conclusions and Recommendations

Some headway has been made in the characterization of subsidence damage to homes above active mines in the United States and the prediction of its severity under a variety of conditions representative of U.S. building practice, building sizes, geology and mining--particularly those of western Pennsylvania.

A Subsidence Classification System, incorporating a damage Severity Index, has been devised that permits subsidence damage to be specified in a uniform and consistent manner. The Severity Index has been expressed in terms of basement damage. Justification for this approach is that basements are found in 96 percent of all homes in western Pennsylvania; basement construction is relatively simple and generally consistent from home to home; basements are prominent in virtually every case where damage is reported; and basement repair commonly represents the largest portion of repair costs.

Given this method of specifying subsidence damage, and lacking explicit measurements of ground movements at individual subsidence sites, the potential usefulness has been demonstrated of an empirical regression model for identifying many of the major factors that govern the severity of damage to homes in western Pennsylvania. The regression model has shown, as expected, that near surface

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Table 6.14

TOTAL REPAIR COST MODEL

Function	Regression Coefficient	t-statistic
Intercept	C_o = $10,009	
(SI)P	C_1 = 34.95	0.59
Perimeter of basement, P	C_2 = -156.9	1.24
(SI)B	C_3 = -1.705	0.05
Bearing Wall, sum of lengths of all sides in basement	C_4 = 17.74	0.29
(SI)S	C_5 = -2.175	0.57
Slab Area, S (basement floor)	C_6 = 10.78	0.99
Exterior and Interior repair costs, EI	C_7 = 0.01447	0.02
Severity Index	C_8 = -1219	0.33
Correlation Coefficient	0.50	
Standard Error	$ 2,737	
F-Statistic (8,29)	1.23	
Confidence Level of the Model	<90 percent	

Note: The 90 percent confidence level is reached at a t-statistic of about 1.68.

effects, collectively expressed in terms of the parameter 'topography' and including the effect of soil, do influence the severity level, but to a much lesser degree than do overburden thickness, type of mining, size of home and certain other factors.

In a practical application, the regression model can yield a pre-mining estimate of the potential severity of subsidence damage to a particular home. A complementary regression model yields a rough presubsidence estimate of potential repair costs to that home in advance of mining. Two additional regression models furnish post-subsidence assessments of the Severity Index at the home based on observed cracks in the basement walls and floor slab and of the potential repair costs based on the nature of damage to the basement. As has been emphasized, the pre-mining prediction of the details of damage to a particular home--in distinction to prediction of the Severity Index--is not possible. The types and numbers of building components affected and the individual degrees of damage can vary widely at a given Severity Index.

Despite their shortcomings, these analyses have yielded results that are quite encouraging, particularly in view of the fact that they are based on only 50 cases--little more than 1/6 of the cases of subsidence involving home damage above active mines that have been documented in southwestern Pennsylvania between 1966 and 1981 and encompassing a variety of mining types. There is reason to expect that with the collection and analysis of higher quality data, the models can be refined to much more satisfactorily explain the factors governing damage severity. Efforts in that direction should be considered equally as important, if not more important, than attempts to develop ground movement prediction models. This is for several reasons. The principal reason, of course, is that the fundamental objective of subsidence prediction is to evaluate potential damage to buildings, not to estimate the ground movements themselves. Secondly, even if reliable ground movement models are developed, the problem of relating ground movements to building damage still exists. Soil/structure interaction is clearly a complex three-dimensional phenomenon that has not yet been characterized in simple form. Transverse or longitudinal ground strains are recognized to be rough and probably inadequate predictors of damage, and whereas principal ground strains are better for this purpose, the soil/structure interaction problem remains even if they are used. Finally, ground movement models are virtually always directed towards total extraction, whereas fully 75 percent of the homes reported damaged to date in western Pennsylvania are located above partial extraction mining, where next to nothing is known about mining-related ground movements. From a practical point of view, then, there is much to recommend the statistical approach to damage prediction, by-passing an explicit statement of soil-structure interaction, but incorporating into the analysis in generalized form the major parameters that govern ground movements, soil/structure interaction and building deformation.

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Obviously, documentation of cases of mine subsidence damage must be improved if a more realistic predictive model is to be developed. This documentation must be more complete and consistent. A dimensioned plan of the basement and wall profiles with crack or separation dimensions and locations should be included in each case record, along with documentation concerning extensions and angular distortions, differential settlements and rigid body rotations for the individual wall segments and slabs. The wall components, concrete block or stone, should be noted with dimensions. Superstructure to basement connections should be identified. The location of grade should be shown on wall profiles. The type of home should be described as to the number of stories, exterior facing type and its age. The home must be located on a topographic map along with its plan orientation. A record of damage to specific components of a home should be made, and damage predating subsidence must be clearly documented. This documentation should be maintained for all homes whether damaged or not that are in proximity to some form of mining within a distance equal to the overburden thickness. Only in this way can reliable statistics be compiled concerning the probability of damage severity for a particular type structure, site, geologic and mining conditions.

Data must be collected concerning overburden characteristics. Existing well logs and borings from mining companies are of some use, but it is obvious that borings to the mine are necessary with soil classified according to currently accepted conventions of the American Society of Civil Engineers (Casagrande, 1948) and the International Society for Rock Mechanics (Brown, 1981). Competent rock units should be identified by type thickness and position in the overburden, at minimum. Incorporation into the model of soil strength and compressibility data would be also desirable in order to differentiate soils in a mechanistic rather that a descriptive manner. However, this refinement should probably be considered secondary to simply identifying the type of soil and estimating its thickness at each site.

Finally, consistent, clear mining information is requisite to identify the sequencing and types of mining below and adjacent to the home site. Inclusion of the progression of home damage with development of a subsidence profile is a desirable goal and would provide a link between the Severity Index and both differential and maximum subsidence.

Implementation of the recommended improvements should lead to a very satisfactory, interpretable model that can be tailored to particular regions for predicting damage severity to conventional homes located above active mines.

7. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

This report summarizes current views concerning soil and near surface rock strata effects on the severity of subsidence damage to structures above coal mines in the United States, particularly western Pennsylvania.

Two hundred eighty two incidents of subsidence damage to homes above active mines in western Pennsylvania have been reviewed through the cooperation of the Pennsylvania Office of Mine Subsidence Regulation. Numerous other subsidence cases above active and abandoned mines have been identified and reviewed through other sources.

Section 2 of the report examines fourteen different types of ground movements that might be confused with, or might complicate, ground movements associated with mine subsidence. It examines the conditions under which these ground movements occur and evaluates methods available for their identification and prediction. Section 3 of the report discusses the character of subsidence above active and abandoned coal mines in the Interior and Northern Appalachian Coal Fields and reviews the status of predicting such ground movements prior to mining. The role of soil and near surface rock effects in influencing the character of subsidence and soil-structure interaction are examined in Section 4. Section 5 summarizes existing criteria for assessing damage to surface structures and examines their potential application to mine subsidence situations. Section 6 documents the subsidence damage experienced by 134 homes above active mines in western Pennsylvania. It quantifies the level of damage of 50 representative homes in terms of a newly developed Severity Index and examines the relative contributions of particular structure, site, geologic and mining factors in dictating the severity of damage. Case histories from Pennsylvania and Illinois that illustrate particular soil and near surface rock effects above active and abandoned mines are presented in Appendix A. Details of concerning individual damaged homes, site characteristics, geologic setting and mining types at each of 134 locations of mine subsidence above active mines in western Pennsylvania are presented in Appendices B through D, along with the definitions of variables and functions that have been utilized in regression analyses to identify the factors that govern damage severity.

Conclusions and Recommendations

What is commonly termed "normal" or "ordinary" settlement encompasses a wide variety of ground movements that includes consolidation settlement, heave, erosion, piping and many others. Movements can originate from so many individual sources that identification of those acting at a given location may be difficult.

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Although each type of ground movement is controlled by certain unique factors, the outward nature of the movements determined from casual observation may be insufficient to identify their source.

Virtually all types of ground movement include some component of rigid body translation, rigid body rotation, distortion or volume change. The types of movement most commonly associated with mine subsidence can also be associated with at least six other types of ground movement. The identification of subsurface material types at a site can sometimes suggest specific sources of ground movement. But subsurface material identification is probably most useful for eliminating sources of ground surface movement that are not likely to be related to on-site soil and rock types. Subsurface exploration, ground surveys, instrumentation programs and testing may be required to make the necessary identifications.

Ordinary ground movements vary widely in magnitude, from fractions of an inch to feet, and in rate, from inches per year to inches per hour. The severity of damage to surface structures may range from slight to relatively severe for virtually any type of ground movement. Not all types of ground movement, of course, could totally destroy a surface structure. Of the fourteen common types of ground movement, the magnitude of movement can be predicted in only six, and, even then, the prediction is rarely better than ±25 percent. The accuracy of ground movement predictions is limited for all types of movement. Generally, but not always, settlement under surface loads and settlements associated with shrinking, swelling and collapsible soils can be predicted somewhat better than the others. The art of prediction of ground movements is rather primitive, not simply a lack of analytical techniques, but deficiencies in our ability to define subsurface soil, rock and ground water conditions, or the distribution, duration and intensity of stress changes. Highly accurate predictions will probably never be possible. In complex cases, the prospects are dim of ever distinguishing by calculation alone the individual contributions of several sources of ground movement that are acting together. Perhaps where only two sources operate and one is settlement under surface loads, the individual contributions can be identified.

Significant progress has been made in recent years in gathering and evaluating available subsidence data in U.S. coal fields, particularly in the Northern Appalachian Coal Field and the Illinois Basin. These data confirm that subsidence incidents above active mines and abandoned mines are distinctly different from one another in terms of geometry, time and place of occurrence, and (to a certain extent) mechanism, yet the manner in which subsidence occurs above an active or abandoned mine is much the same regardless of geographic location -- whether it takes place in Appalachia or in Illinois. Although geologic differences exist between the Appalachian and Illinois regions, available data are not sufficiently refined or extensive to identify reliable relationships between subsidence parameters and basic mine site stratigraphy determined from a few borehole logs. The influence of soil and

near surface rock on subsidence parameters cannot be determined from available data.

Very few measurements exist in the United States of ground strain, slope and curvature that are of use in relating mining to severity of damage to structures at ground surface. Furthermore, almost no systematic subsidence profile data have been collected and synthesized for the vast majority of home damage cases that have been documented above abandoned and active mines in the Northern Appalachian Coal Field and the Illinois Basin. For this reason, it is not possible to identify the prevailing ground strain, slope or curvature that may have caused damage for any but a few cases. The lack of data relating ground movement and home damage makes the identification of soil and near surface rock effects on damage severity nearly impossible. Case histories of mine subsidence make clear, however, that soil and near surface rock can influence the character of subsidence through landslides and slippage, block-wise movements of rock strata, reactivation of faults, buckling of strata, loss of soil into open joints, etc. Many of the phenomena reported in the British and European literature are recognized in the United States. As is typical of many forms of ground movement, near surface phenomena tend to be unpredictable in both magnitude and occurrence. To some degree they must be expected at every subsidence site.

Even without anomalous near surface effects, the soil-structure interaction problem is a difficult one whether subsidence is involved or not. This is because the analysis has not been reduced to a routine procedure except in very simple cases. In addition, realistic analyses that take into account cladding and other details can be expensive. For these reasons, guidance concerning allowable ground movements is generally obtained from empirical relationships. Unfortunately, the most common empirical relationships--those developed for self-weight settlements--are generally not applicable to mine subsidence situations, which commonly involve a significant horizontal component of ground movement as well as a vertical component. Further, most damage criteria are rather subjective in specifying levels of damage, and most, in fact, are limited to the onset of cosmetic, or superficial, damage and do not recognize the more pronounced damage commonly associated with mine subsidence. Establishment of sound, simple criteria encompassing the broad spectrum of mine subsidence damages requires that damage and repair requirements be catalogued at individual subsidence sites, and that during the subsidence process, building deformation parameters, such as angular distortion, be measured along with ground strain, tilt and radius of curvature.

In the present program, some progress has been made in this direction by reviewing the subsidence damage sustained by 134 typical homes in western Pennsylvania that were located 157 to 650 feet above active mines in which a variety of mining types have been utilized. Damage was expressed in terms of location within the home and severity. Basements were always found damaged, the

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other components being damaged less frequently in this order: interior, exterior and roof. Very few data were available to quantify ground movements and structure movements in relation to observed damage. Damage appeared to be due to a combination of hogging, sagging, extension and compression, rather than simply to extension and compression as suggested in the literature. It is clear that U.S. homes, a large proportion of which have basements and include superstructures with only limited mechanical connection to the basement, respond differently to subsidence than do buildings of slab-on-grade construction.

Inasmuch as (1) basements are virtually always damaged, (2) basement repair costs characteristically represent the largest proportion of total repair costs, and (3) basements are rather uniformly constructed from home to home, basement damage has been chosen as the basis for a newly developed subsidence damage classification system applicable to homes in the United States, particularly western Pennsylvania. The Severity Index, a continous function ranging from 0.0 to 5.0, is a quantitative gage of the severity of damage to a home. The range from 0.0 to 1.0 corresponds to slight damage, where repairs require little more than repointing the basement walls. The range from 1.0 to 2.0 corresponds to moderate damage, where repairs may include a relatively small amount of wall reconstruction. The range from 2.0 to 4.0 corresponds to severe damage, where extensive basement repairs are required, including major wall reconstruction and floor slab replacement. The range 4.0 to 5.0 corresponds to very severe damage where nearly total reconstruction of the basement is required.

Superstructure damage does not always conform to basement damage in a predictable manner due to the framing action of the superstructure and the limited mechanical connection between the superstructure and basement. Nonetheless, the more severe cases of superstructure damage are typically associated with relatively severe basement damage. In strict terms, the Severity Index is a measure of damage to the basement, but because basement damage dominates the cost of repair, the Severity Index in effect reflects the degree of damage to the entire home. Recognizing that the Severity Index represents a uniform, consistent measure of damage that can be compared from site to site, a predictive model of Severity Index was developed through a stepwise elimination regression analysis that provided a means for evaluating the relative importance of particular structure, site, geologic and mining factors in influencing the severity of damage to homes. The severity was found to be greater the larger and more square the home, the shallower the mine and the lower the topographic relief at the site. The last factor reflects a combined effect with soil. The types of mining below and adjacent to the site also have an effect on severity.

By and large, the results of the empirical model are as one would expect and are considered to be reasonably good considering the limited amount of data available. Specific recommendations for

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future data collection are presented in the last pages of Section 6.

Improved documentation of subsidence cases should in the future yield a very satisfactory analytical model for predicting damage severity. Eventual incorporation into the model of ground movement data should improve our understanding of the processes that control severity and should measurably improve our ability to predict it. More appropriate building designs might result, along with a better understanding of how to protect existing structures.

Further development of methods of damage prediction should be considered as important, if not more important, than the development of subsidence prediction models. The use in the United States of relationships between ground movements and damage severity that have been developed abroad is to be discouraged.

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CASE HISTORIES

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Case History 1

Hillslope Ground Movements above Abandoned Mine

Site Location

The site is located near Pittsburgh, Pennsylvania, on a hillside characterized by nearly flat-lying sedimentary rock strata and brush- and tree-covered slopes, as is typical of this region.

The homes are primarily one-story brick dwellings that were built in the early 1950's, 15 or more years after the completion of mining in the Pittsburgh Coal. The coal bed, which lies about 95 feet below basement level, underlies the entire site area and outcrops about 900 feet east and downhill of the homes.

The earth movement occupied a 100 by 115-foot area on a north facing 25 degree slope of a small valley that opens eastward toward a nearby river.

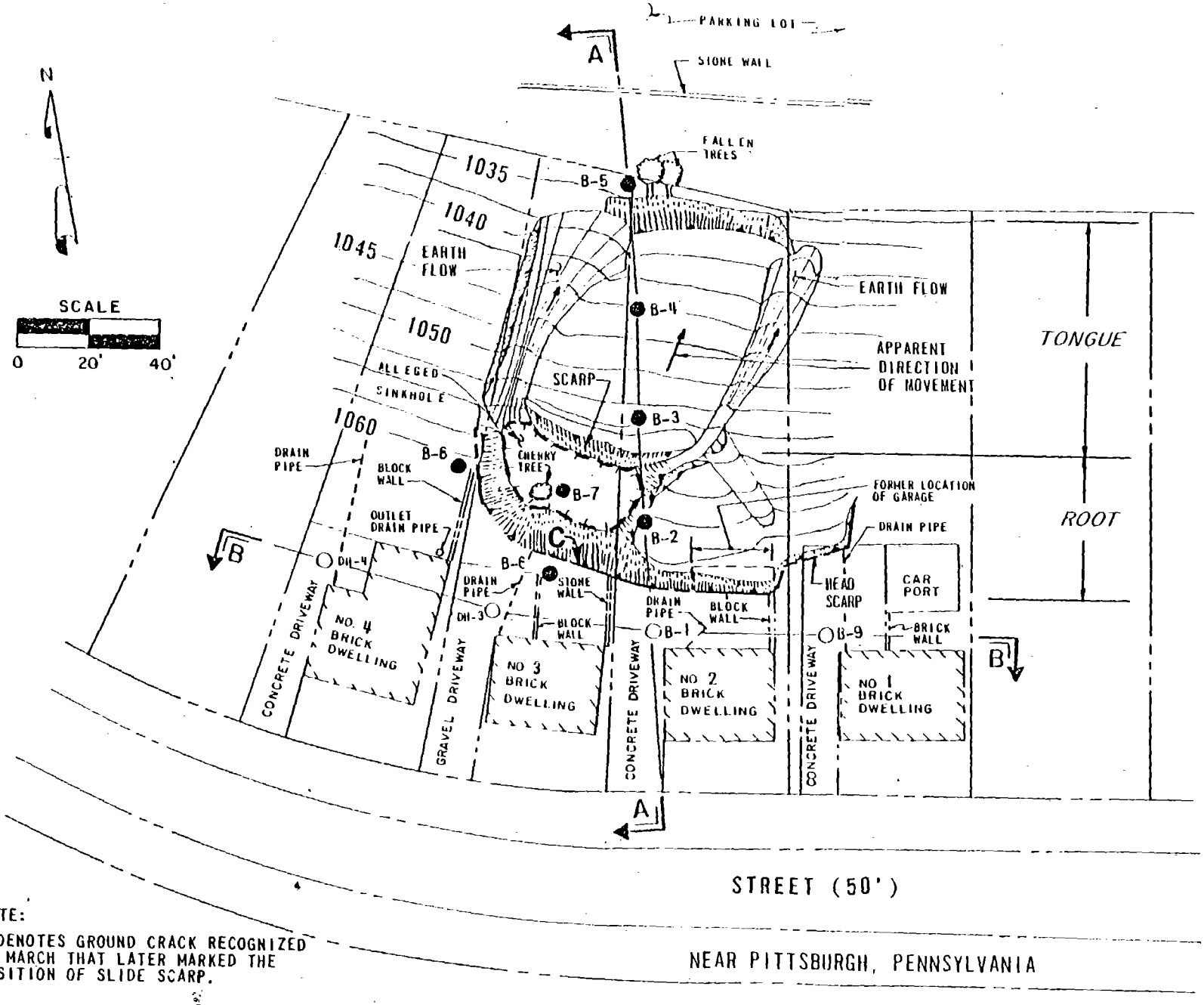
Character of the Movements

On March 2, several years ago, the residents of two adjacent homes observed that a 10-by 90-foot section of their backyards had settled 2 to 3 inches since the previous day. The settled area was part of a level bench that extended from the rear of the homes to the break in slope 35 feet to the north, and was circumscribed by a crack, denoted by "C" in Figure A.1, that swept in an arc across the yards--open end downhill--reaching to within 25 feet of the rear of the two homes. The face of the slope extending downhill from the level area appeared not to have changed.

From this time forward, settlement of the yard behind Homes 2 and 3 continued at a slow but steady rate so that by the end of March, a gap about two feet high had opened between the ground surface and the underside of the garage at Home 2. The crack extended outward from the back of the home to within 8 feet of the break in slope.

The residents, fearing that the garage was in jeopardy, secured a contractor to underpin it. The underpinning was accomplished by backfilling the gap between the underside of the garage and ground surface with 20 tons of slag, and then casting a concrete footing beneath the downhill wall of the garage into which steel pipes were embedded to meet the underside of the garage. The underpinning was completed by April 14.

Movements continued, and by May 1, the ground surface had settled another foot. At this point, one wall of the garage collapsed. Movements went on for the next several months and by January 12 of the next year, ground cracks had worked back to within 15 feet of the homes. Settlements in some parts of the yard had reached 12 feet. By this time, cracks had developed in the two



NOTE:
C DENOTES GROUND CRACK RECOGNIZED IN MARCH THAT LATER MARKED THE POSITION OF SLIDE SCARP.

FIGURE A.1— PLAN OF GROUND MOVEMENTS, CASE HISTORY 1

remaining walls of the garage, and on February 2, the floor slab of the garage dropped. A week later the two garage walls and patio located on the garage roof collapsed. Movements of the ground surface following collapse of the garage were not appreciable. The movements did not extend far enough to damage any of the homes. However, continued movements combined with erosion could have had serious consequences.

Previous Studies

Two investigations were conducted at the site. In August of the first year, a state agency investigated the site at the request of the residents, and as part of their study drilled four core holes, one immediately behind each of the four homes in the vicinity of the disturbance. From their investigation, the agency concluded that the movements were associated with an earth slide. As such, abatement of the movements and the repair of any damage were responsibilities of the residents.

In March of the following year, after collapse of the garage at Home 2, the borough engineer conducted a transit survey of the site and summarized his views regarding the origin of the movements based on observations made over the previous year. He concluded from his study of ground features that the earth movements were the result of mine subsidence rather than an earth slide, noting in particular that the cherry tree located within the depression near the crest of the hill remained in an upright position during and after movement. To him this indicated that movements of the ground surface within the depression had been principally downward, a type of movement he felt could only be associated with the winnowing of soil and rock into the abandoned mine below through a collapse in the mine roof. He claimed also that he could perceive no changes in the face of the slope downhill of the settled area and felt this confirmed that subsidence rather than sliding was the responsible mechanism of movement. These concepts are shown in Figure A.3, based on a sketch by the borough engineer.

As a result of the difference in interpretation of site features between state and borough officials, the writers were engaged by the state to conduct an independent study of the cause of the earth movements and to present recommendations for stabilizing the affected area.

Supplementary Study

The writers' study encompassed four tasks:

1. Discussions with residents of the affected area regarding the character and sequence of the movements;

2. Inspection of surficial site features including scarps, ground cracks, tree orientation and other indicators of the origin and direction of ground movements;

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3. Delineation of subsurface conditions by means of seven borings, six of which extended from ground surface to the floor of the mine; and

4. Collection of information regarding mining beneath the site and subsidence incidents that have taken place in the general vicinity of the site.

Identification of the Origin of the Movement

An examination of surface features at the site by the writers revealed that an earth slide had indeed taken place. The slide embraced approximately 2,600 cubic yards of material and had affected an area 115 feet wide that extended nearly 100 feet downslope from the depression at the crest of the hill. The features described above are labelled in the plan and section views (Figures A.1 and A.2). It will be noted that the outward displacement of a portion of the upper slide mass and the downward movement of the wedge of material adjoining the head scarp had created a substantial depression in the ground surface near the crest of the slope. The somewhat rounded appearance of this feature in plan and its sinking motion as it developed were apparently responsible for its misidentification as a sinkhole.

Had there actually been a sinkhole at this location, one would have expected top of rock near the center to have been lower than at the undisturbed perimeter, particularly if 12 feet of vertical displacement had taken place; and one would also have expected a lack of registry of the strata within the sinkhole relative to the material at the perimeter. Profiles of top of rock and correlations of strata between borings indicated that no sinkhole was located within the depression.

Factors Causing the Movement

The data obtained from the borings indicated that the slide was confined to the mantle of soil that covers the hillside to a depth of 5 to 30 feet. Much of this soil was a wedge of fill placed at the head of the slope to create a level yard area. The soil beneath the fill and mantling the decomposed rock to a thickness of 5 to 15 feet is colluvial, having been derived by the weathering of the underlying sedimentary rocks and gradually transported downhill by slow creep and periodic slides.

Colluvial slopes are at best only marginally stable. Several hundred slides have been recorded in Allegheny County, Pennsylvania, over the past 20 years in geographic circumstances similar to those at the subject site, particularly on slopes upon which fill has been placed.

Evidence at the subject site suggests that sliding was hastened if not caused by two particular factors:

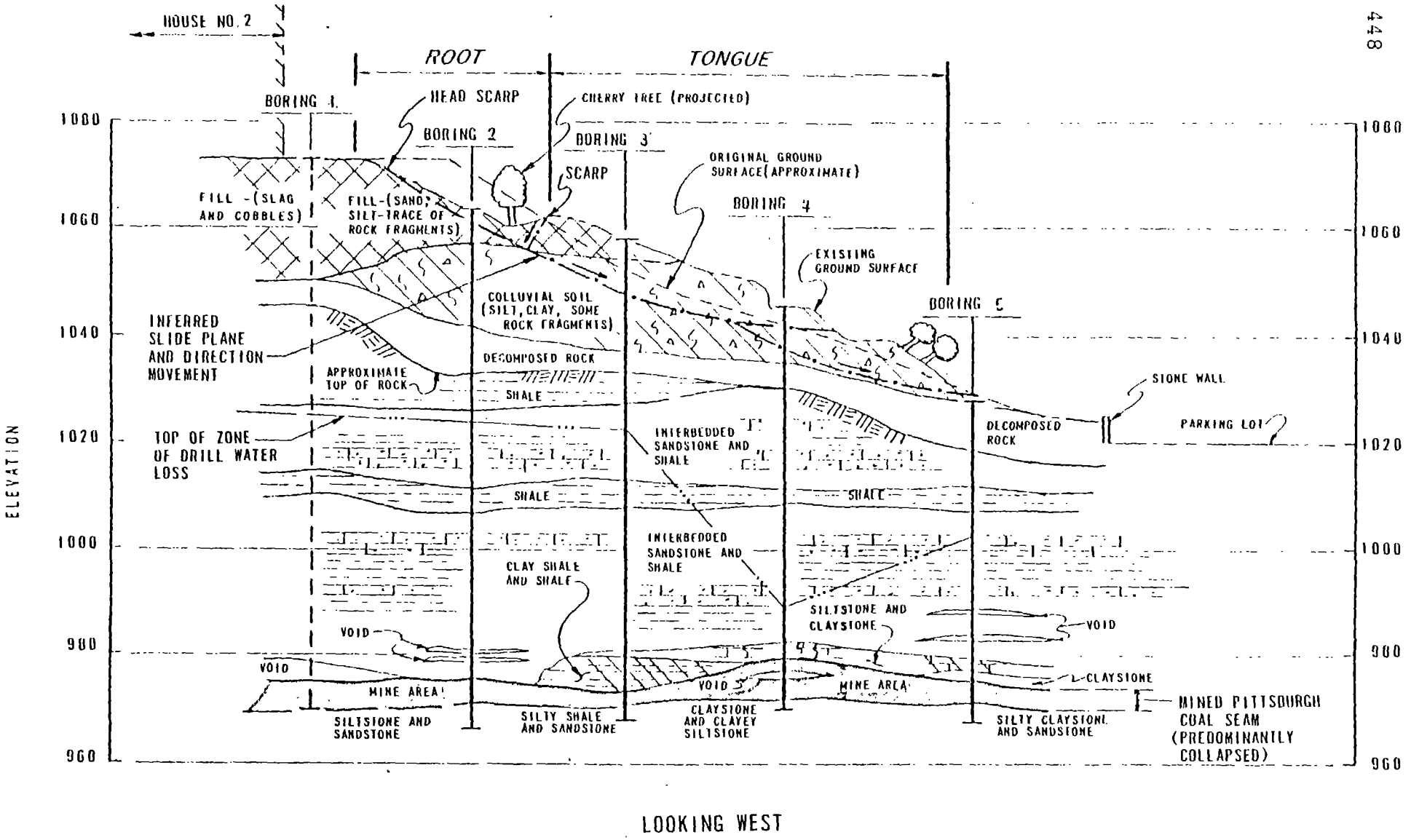


FIGURE A.2 SECTION A-A THROUGH GROUND MOVEMENT AREA, CASE HISTORY 1

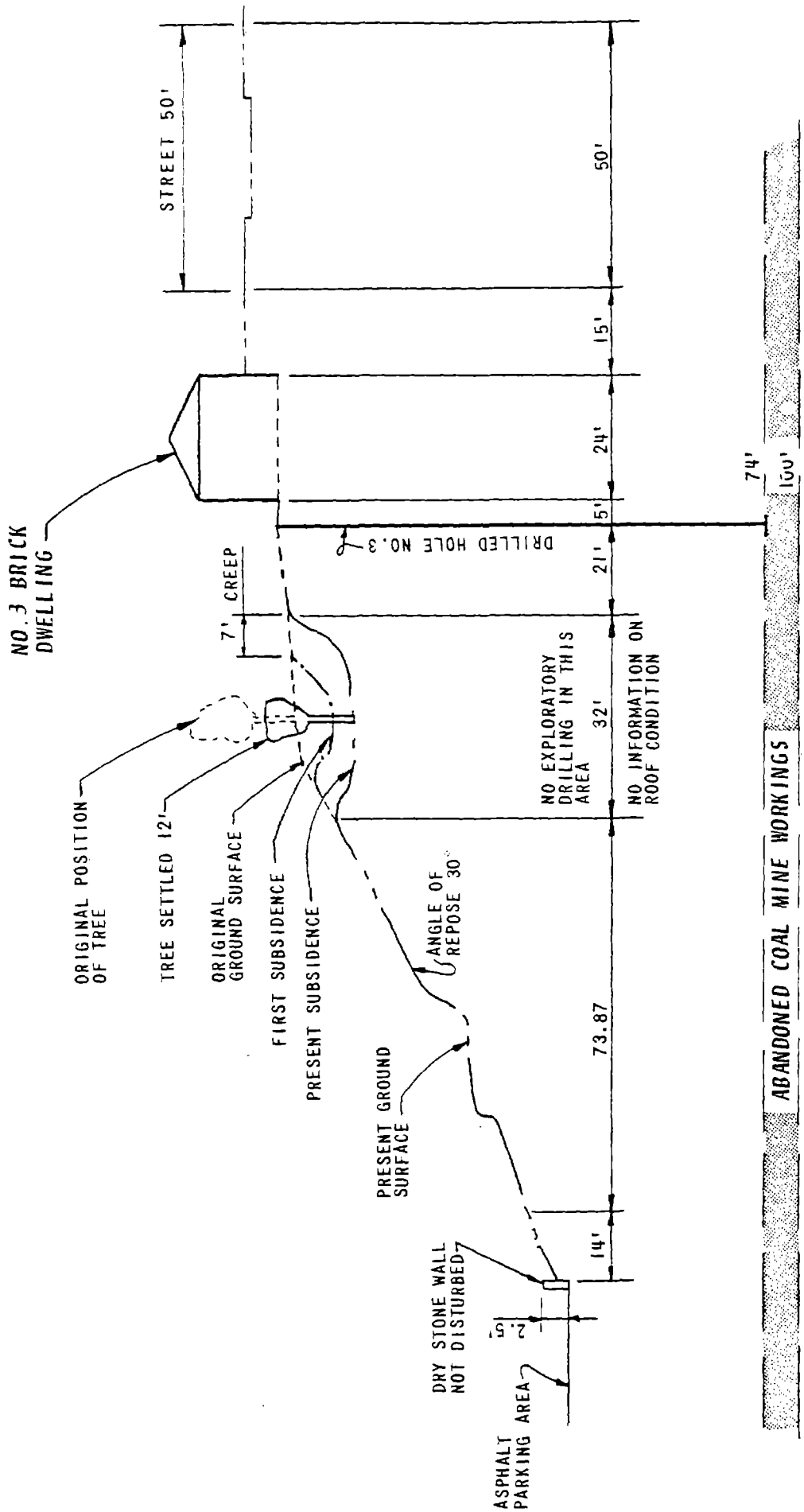


FIGURE A.3 – SECTION A-A THROUGH GROUND MOVEMENT AREA, ALTERNATE INTERPRETATION, CASE HISTORY 1

1. Discharging storm water and runoff onto the hillside at the crest of the slope; and

2. Overloading the slope by construction near the crest.

It is evident that water played a large role in the failure of the slope. Lobes of soil emanating from the depression near the head of the slope extended downhill along the east and west margins of the slide area. These lobes represent zones where the soil had apparently flowed downhill as a viscous fluid. It is probable that the soil was in a highly saturated condition when this took place and it is also probable that the soil became saturated from drainage at the crest of the hill.

Three drain pipes extended to the slide scarp from the backyards of the homes. In addition, runoff from the driveway and backyard areas of the four homes flowed onto the hillslope, channeled by the garage walls and stone walls. At the time of the slide, the slope was also possibly charged with water from the wedge-shaped body of slag fill located behind the four homes. Water percolating into the slag from the backyard and driveway may have collected behind the less pervious fill mantling the slope and decreased the shear strength of the soil in this vicinity.

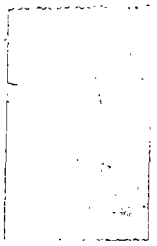
Loading the slope began by placing the wedge of fill at the head and constructing the garage. These loads were apparently not large enough to precipitate movements until the strength of the soil was reduced by the presence of sufficient quantities of water. Once the slope began to fail and displacements of several feet took place, the placement of 54 tons of slag and concrete beneath the garage undoubtedly caused the additional slope movements.

Interestingly, besides the effect of water and overloading, there is also the possibility that subsidence may have contributed to the instability of the slope. While the development of a sinkhole at the head of the slide had been ruled out by drilling information, it is not impossible that subsidence associated with either a sinkhole or trough near the base of the slope removed enough support to permit slope movements. The borings near the base of the slope did show that the mine was essentially collapsed, as it was in most areas on site, but there is no way to determine when collapse took place. Quite possibly it took place immediately after extraction of the coal, as is common during retreat mining, but this is not certain. In any case, subsidence is not believed to have been a necessary condition for slope failure, although it could have been a contributing factor.

This case is noteworthy in that it illustrates how ground movements are occasionally misinterpreted--in this case by a public official whose judgment was considered unimpeachable by local residents because he was a registered professional engineer, but who actually had little background in this type of work. Moreover, the case suggests that incorrect identification of the source of the ground movement could have led to wholly inappropriate remedial

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measures--backfilling the mine, for example, rather than directing efforts toward modifying surface drainage or reducing the loading at the head of the slope.



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Case History 2

Cause of Ground Movements Inconclusive above Abandoned Mine

In 1976, ground movements of unknown origin caused damage to four neighboring homes in Allegheny County, Pennsylvania (Figure A.4). Two of these homes were severely damaged.

All of the homes were located 90 to 110 feet above the abandoned Pittsburgh Terminal Coal Company Mines No. 1 and No. 2 in the Pittsburgh coal bed. Last mining was in the 1930's. Deep rotary borings made along the street in front of the four homes determined that overburden above the mine consisted of 5 to 15 feet of soil overlying 85 to 95 feet of rock.

The general ground slope in the community is 14 degrees. Local slope variations result in a much steeper 30 degree slope from the rear of the damaged homes to a highway below. Shallow drilling by the writers behind Home C (Figure A.5) indicates that part of this steeper slope was formed when earth fill was placed there prior to construction of the housing development. The rear portion of each of the damaged homes was constructed on at least a 12-foot thickness of the fill (Figure A.6). It is suspected, but could not be confirmed, that fill is also located below the front of the home.

A short chronology of the case is presented below. It is a typical situation that involves a complex inter-relationship of factors, which is difficult for government officials to act on rapidly and with certainty.

1930's	Pittsburgh Terminal Coal Company mine is abandoned
1953	Homes are built on sloping hillside and at least partially founded on fill and residual soil.
July, 1967	In an application for Pennsylvania mine subsidence insurance, Home B (Figure A.4) is found to have been damaged by ground movements sometime in the past. The damage is ruled to be other than mine subsidence by Pennsylvania State Officials.
September, 1971	New cracks develop in Home B; old cracks reopen in the basement, exterior brick, and interior plaster. A State Official judges the damage to have been caused by mine subsidence.

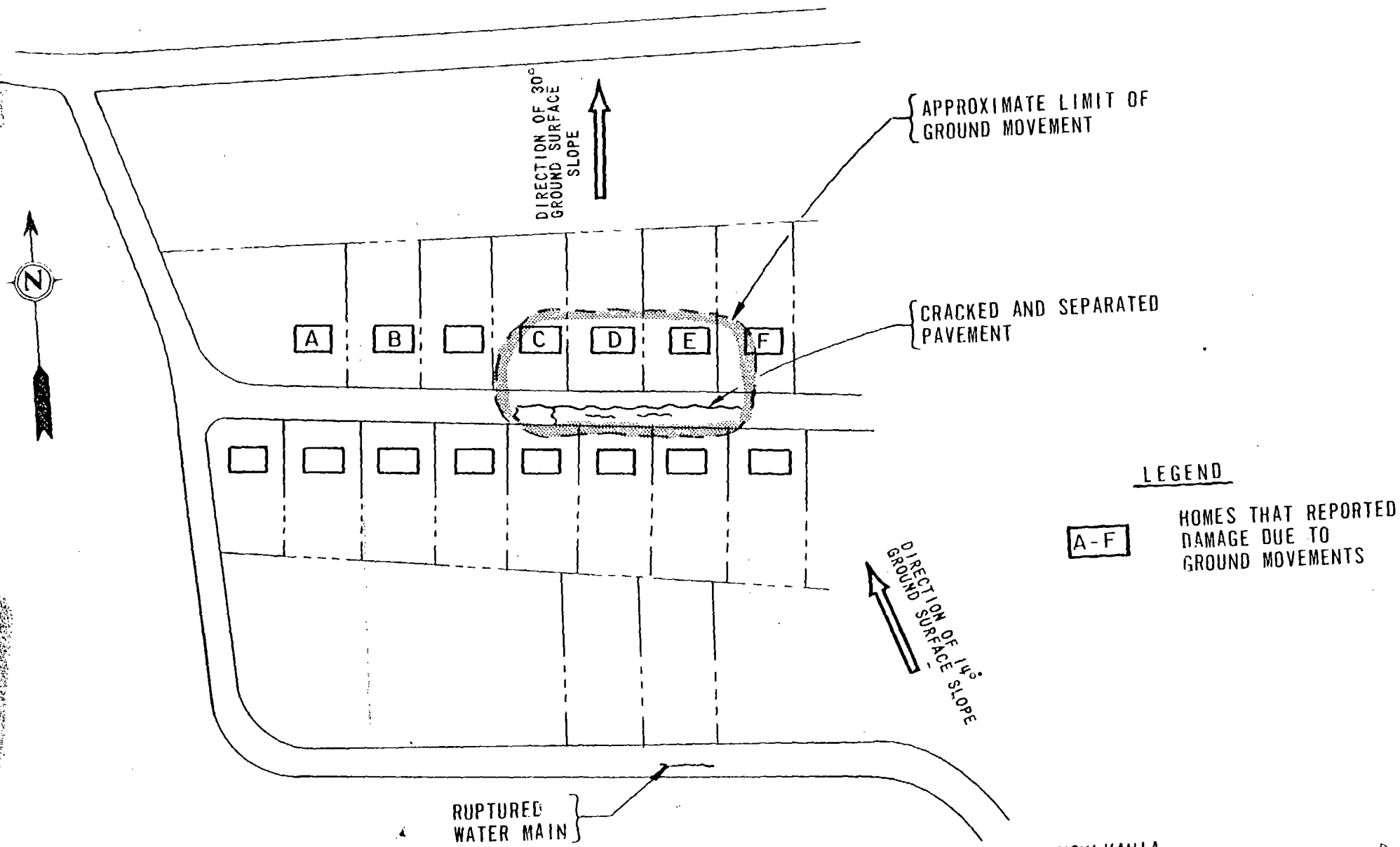
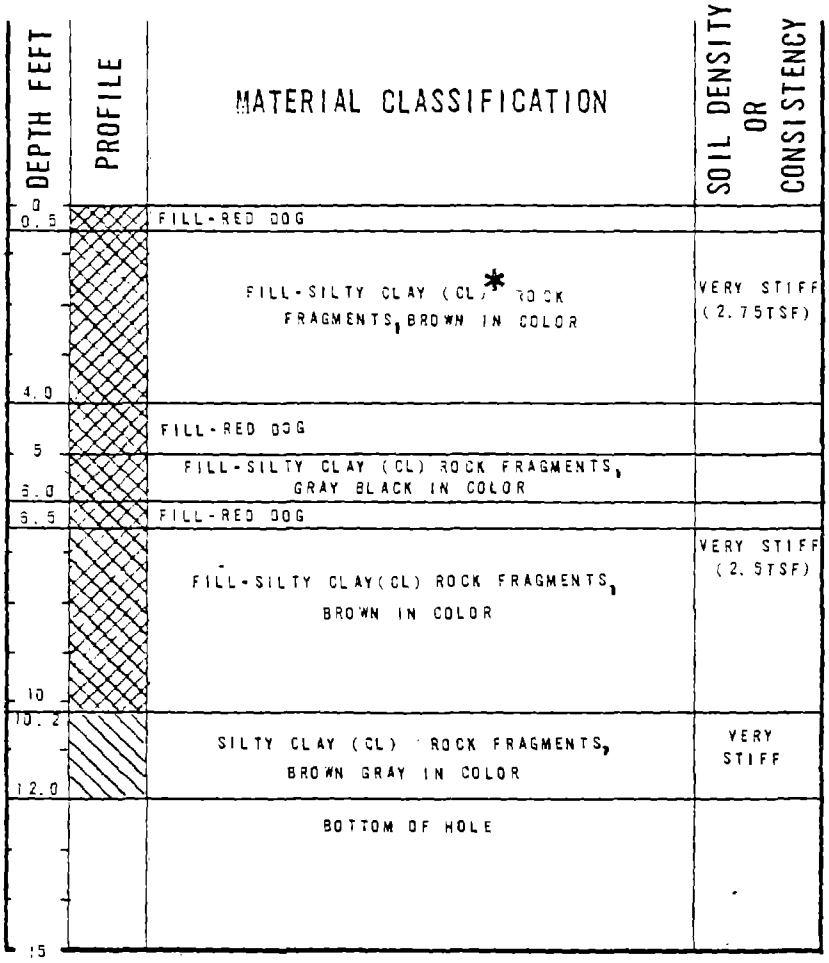


FIGURE A-4-PLAN VIEW OF THE NEIGHBORHOOD AND HOMES AFFECTED BY GROUND MOVEMENTS, CASE HISTORY 2



*AN EXPLANATION OF SOIL CLASSIFICATION SYMBOLS IS PRESENTED IN FIGURE A.61

FIGURE A.5 – SOIL BORING, CASE HISTORY 2

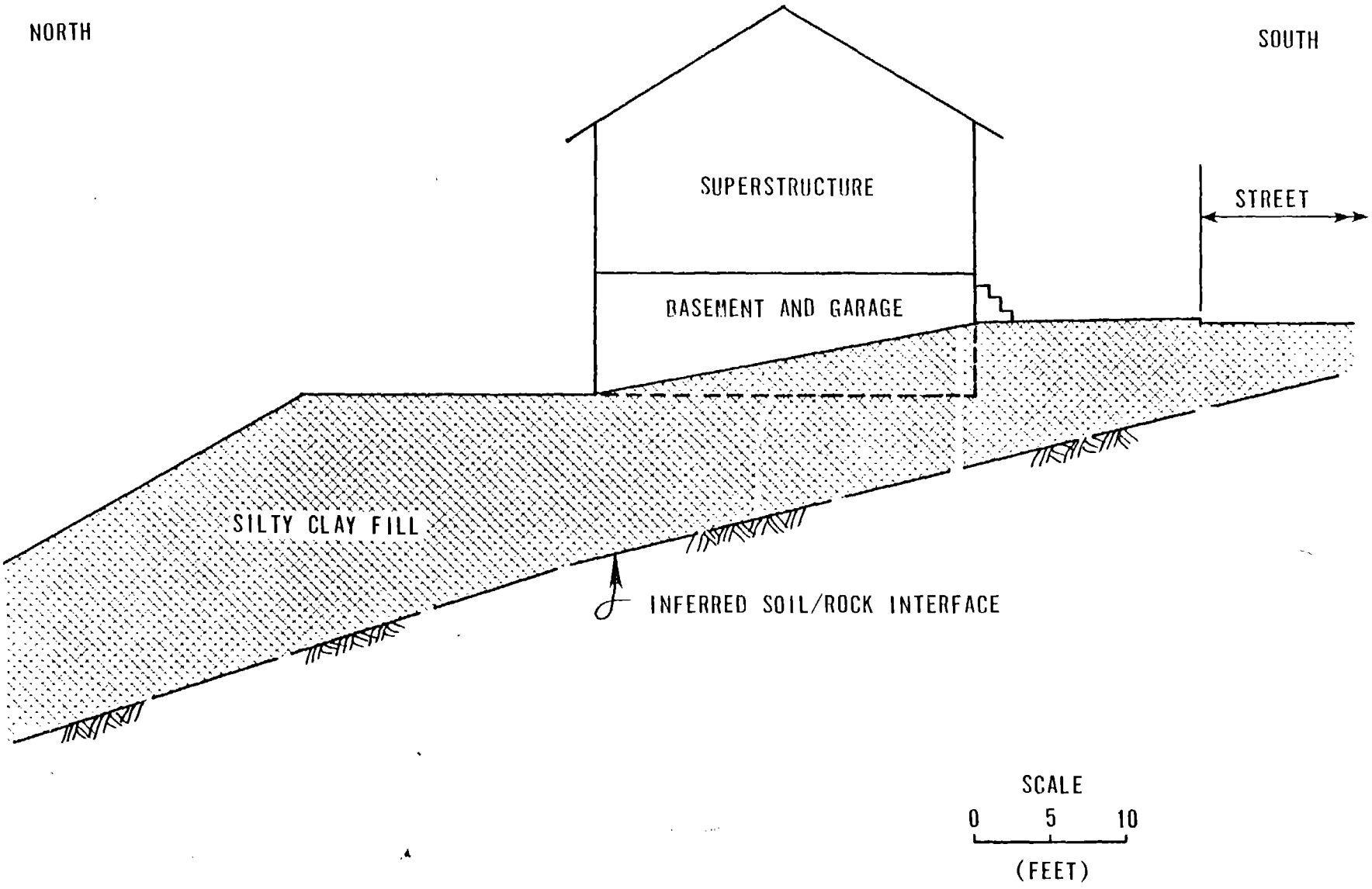


FIGURE A.6 CONCEPTUAL CROSS-SECTION OF HOME C AND SURROUNDING TERRAIN PRIOR TO GROUND MOVEMENTS, CASE HISTORY 2

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A short time before July 18, 1976	A water main break occurs one block directly up-slope from the subject site.
July 21, 1976	Damage due to ground movements is discovered by residents in Homes C, D, E, and F.
July 22, 1976	In the morning, running water is heard behind the front basement wall of Home E. By evening, the water begins to seep through the basement wall.
July 23, 1976	Water stops flowing into Home E.
July 24, 1976	The width of a crack in the center of the front road is found to have opened three-quarters of an inch from the previous day. By now, moderate to severe damage has occurred to Homes C and E. Homes D and F sustain only light damage. No further damage occurs after this day.
October 19-25, 1976	Exploratory drilling at the site is performed by the State government. No rock core is taken. Voids were encountered throughout the rock overburden. Subsidence is concluded to have occurred sometime in the past.
October 20, 1977	A state-sponsored subsidence abatement project is begun at the site. Ground disturbance due to drilling activities cause existing cracks in Homes A and B to widen.
November 3, 1977	Contractor changes drilling rigs. Cracking of nearby homes ceases.

Home C, which suffered the most severe damage, is a one-story, six room, 26-year old, brick-faced structure. Approximately one-third of the cinder-block walled basement is occupied by a garage. The rear of the home moved in a downhill direction, causing the home to break in tension and tilt slightly from front to back. The rear basement wall and the two walls adjacent to it were cracked. The concrete basement floor was also cracked. The rear and side

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plaster walls of the home's interior were damaged, with long cracks developing in the front room ceiling parallel to the front of the home. Door and window frames throughout the home became warped. The concrete slab that forms the front porch was cracked.

Although the claim of mine subsidence was finally allowed by state officials, conclusive proof of mine subsidence coincident with home damage was never obtained. Ground movement may have been due solely to slope failure initiated by the water from the broken water main, or may have been due to mine subsidence brought about by the excess water, which then precipitated downslope earth movements. An episode of mine subsidence prior to construction of the homes may have created a potentially unstable slope which did not move until triggered by the events of July, 1976. Although the broken rock strata and voids encountered in the overburden during drilling suggest that some degree of mine subsidence had actually occurred at the site, it is impossible to say when.

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Case History 3

Influence of Overburden Characteristics on Sinkhole Development*

In April, 1979, a 12-by-16 foot diameter sinkhole, 4 feet deep, developed in a street next to the Post Office in the Village of St. David, Illinois, about 40 miles southwest of Peoria. A water main was ruptured in the process. The Abandoned Mined Lands Reclamation Council (Illinois) determined that the subsided street posed an extreme danger to public health, safety, general welfare and property, and was successful in securing federal Office of Surface Mining funds for a study of the problem by the University of Illinois. Approximately 60 sinkholes were identified in the St. David area. Ten were located under or adjacent to structures; six caused damage to structures. The key findings of the study are presented below.

Coal Mines under St. David (Fulton County), Illinois, are located in the Springfield No. 5 Coal and were operated from the 1860's to the 1940's by room-and-pillar methods. Mining beneath the Village had been completed by about 1900.

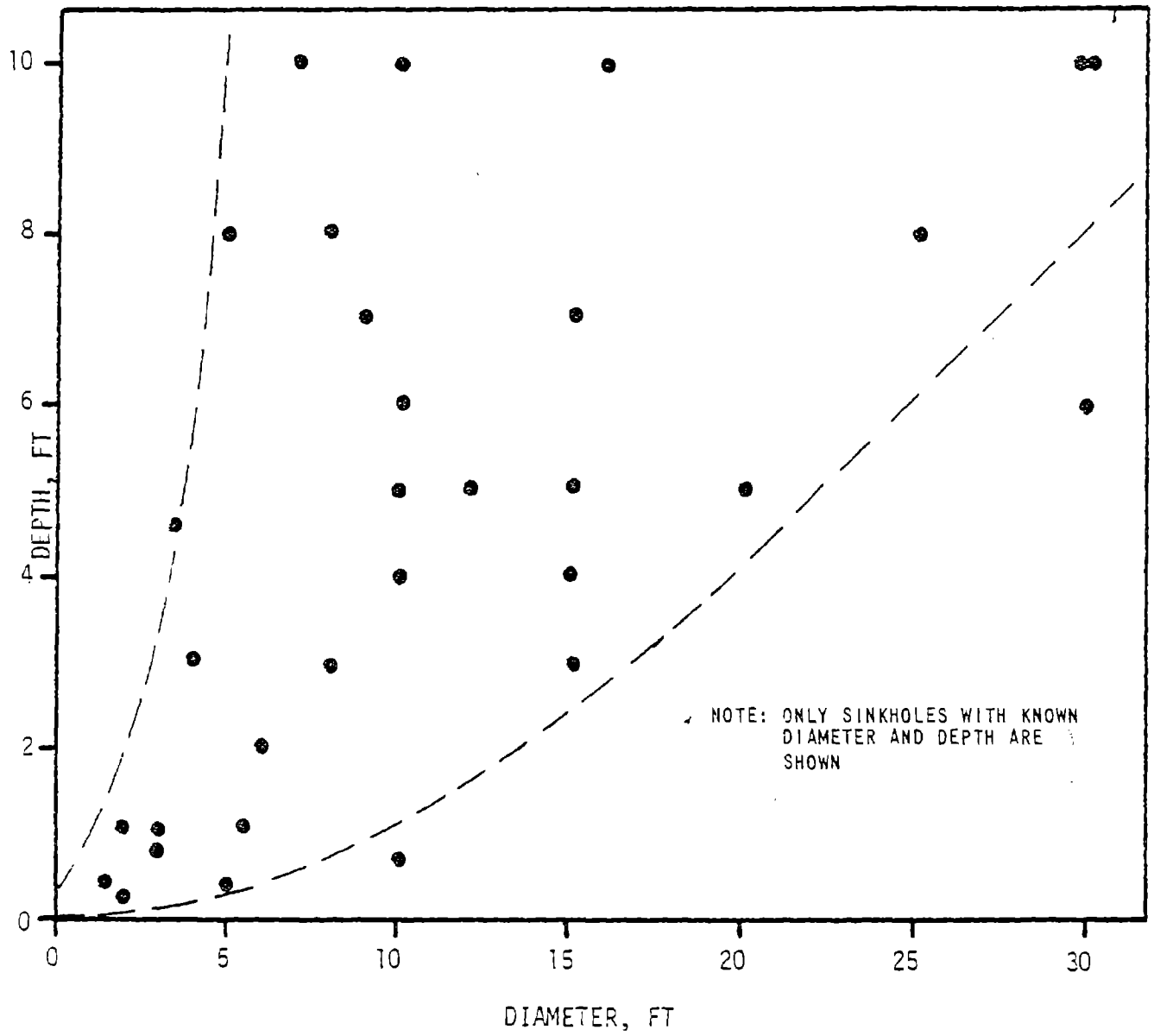
The coal is nearly flat-lying and 4.5- to 5-feet thick. The St. David Limestone forms the roof in many mines, but is discontinuous in those beneath the Village of St. David. In some places the limestone grades into discontinuous concretions or fossiliferous shales. The black shale is subject to slaking and other phenomena that sooner or later lead to its collapse.

Bedrock (primarily shale) is overlain by surficial deposits of Peorian loess reaching thicknesses of 25 feet. In some cases, little more than a foot of shale separates the mine opening from the overlying loess. The loess can be characterized as a brown and orange clayey silt with small rootholes, having a liquid limit of 33%, a plasticity index of 12% and an unconfined compressive strength of about 2 tons per square foot at a natural water content of 17%.

Sinkholes are the common form of subsidence owing to the relative widths of mine openings and depth to the mine. The coal mines are only 40 to 55 feet below ground surface and include entries ranging in width from 6-8 feet to 20-30 feet. Reported sinkholes range up to 30 feet in diameter and 10 feet in depth (Figure A.7). No relationship has been identified between diameter and depth of sinkholes. Most sinkholes appear not to deepen appreciably with time.

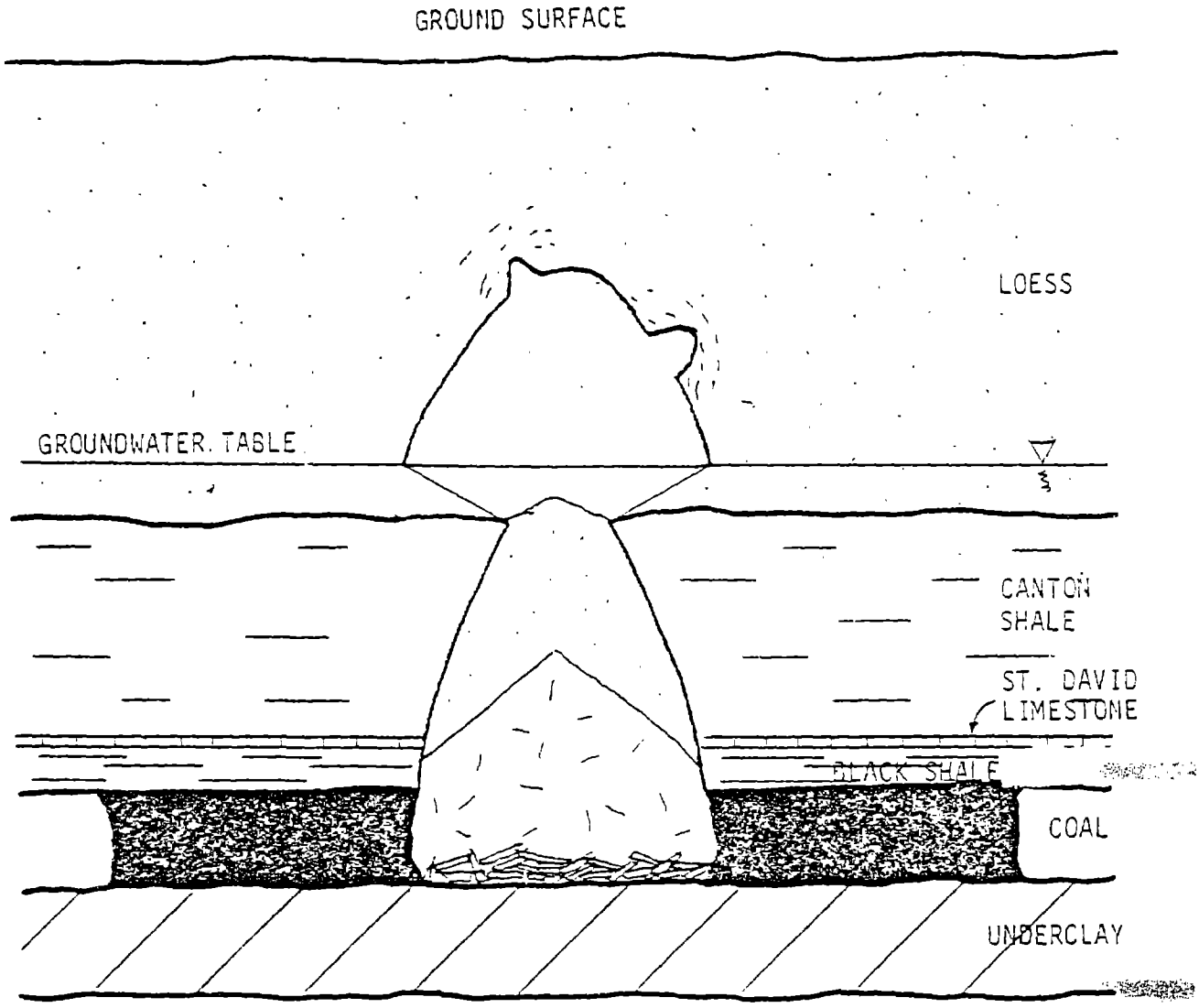
The migration of roof cave zones from mine level upward through the rock and overlying loess to ground surface is promoted by the seepage of ground water to the mine through fissures in the Canton Shale. Raveling and piping of the loess, whose cohesion is significantly reduced by saturation, results in near-surface

*This case history is a digest by the present writers of a report prepared for the Illinois Abandoned Mined Lands Reclamation Council by the University of Illinois (Wildanger, Mahar and Nieto, 1980).



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FIGURE A.7 - RELATIONSHIP BETWEEN DIAMETER AND DEPTH OF SINKHOLES, ST.DAVID, ILLINOIS, CASE HISTORY 3
(WILDANGER, et al, 1980)



Approximate Scale: 1" = 10'

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FIGURE A.8- UPWARD MIGRATION OF THE MINE VOID THROUGH THE LOESS ; ST. DAVID, ILLINOIS, CASE HISTORY 3 (WILDANGER, et al, 1980)

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collapse-prone voids in the loess that occasionally are larger than the caved opening at top of rock (Figure A.8). Many sinkholes appear after rainstorms.

The present writers observe from this work that the development of mine sinkholes in Illinois and the Appalachian region is much the same.

Case History 4

Subsidence above Abandoned Mine Attributed to Failure of Mine Floor*

The O'Fallon subsidence incident took place in the summer of 1978 on West Fifth Street (Old Route 50) in O'Fallon, Illinois, fifteen miles east of St. Louis, Missouri. Two homes were damaged.

Subsidence first became apparent during the period August 13-18, and intensified during a 36-hour period on August 19 and 20. Ground movements were accompanied by several water line breaks and some cracking and distortion of the homes. In the period of accelerated movement, subsidence increased to a maximum about 2.9 feet, accompanied by further movements of the homes. Over the next 10 months, subsidence increased to about 3.25 feet.

The subsidence trough ultimately measured about 450 feet in an east-west direction and 350 feet in a north-south direction (Figure A.9). The maximum subsidence of 3.25 feet is slightly in excess of half the mine height. The reported trough profile is shown in Figure A.10. The geometric characteristics of the profile (Table A.1) are typical of those found above other room-and-pillar mines in Illinois.

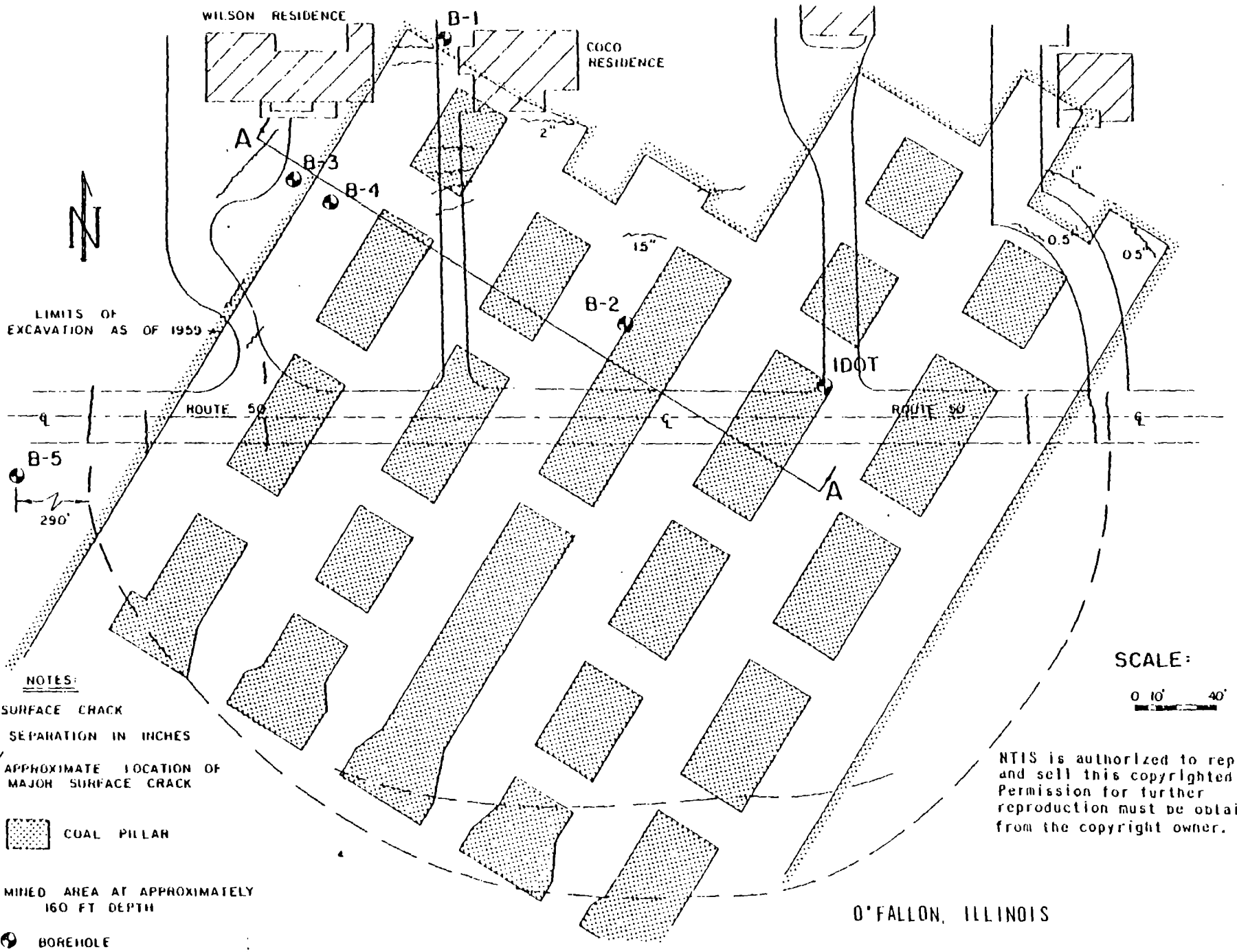
Subsidence was attributed to a bearing capacity failure of the underclay mine floor at the periphery of one or more coal pillars. This is thought to have reduced the cross-sectional areas of the pillars, causing them to crush under the weight of the overburden. Transfer of the overburden load to adjacent pillars led to crushing of the adjacent pillars and ultimately to development of the trough at ground surface.

The St. Ellen Mine is located in the 6-foot Herrin No. 6 Coal, 160 to 170 feet below ground surface, and is thought to have last been mined in the late 1950's. Rooms are generally 30 to 32 feet wide; pillars are slightly narrower. Panel width is about 320 feet.

Overburden at the site consists of about 90 feet of soil overlying about 80 feet of interbedded shale, sandstone and limestone. Soil consists of 15 to 30 feet of medium stiff loess; 30 to 40 feet of medium dense sand, silty clay and gravel alluvium; and 15 to 45 feet of glacial till composed of medium dense to dense sand and clayey silt/silty clay.

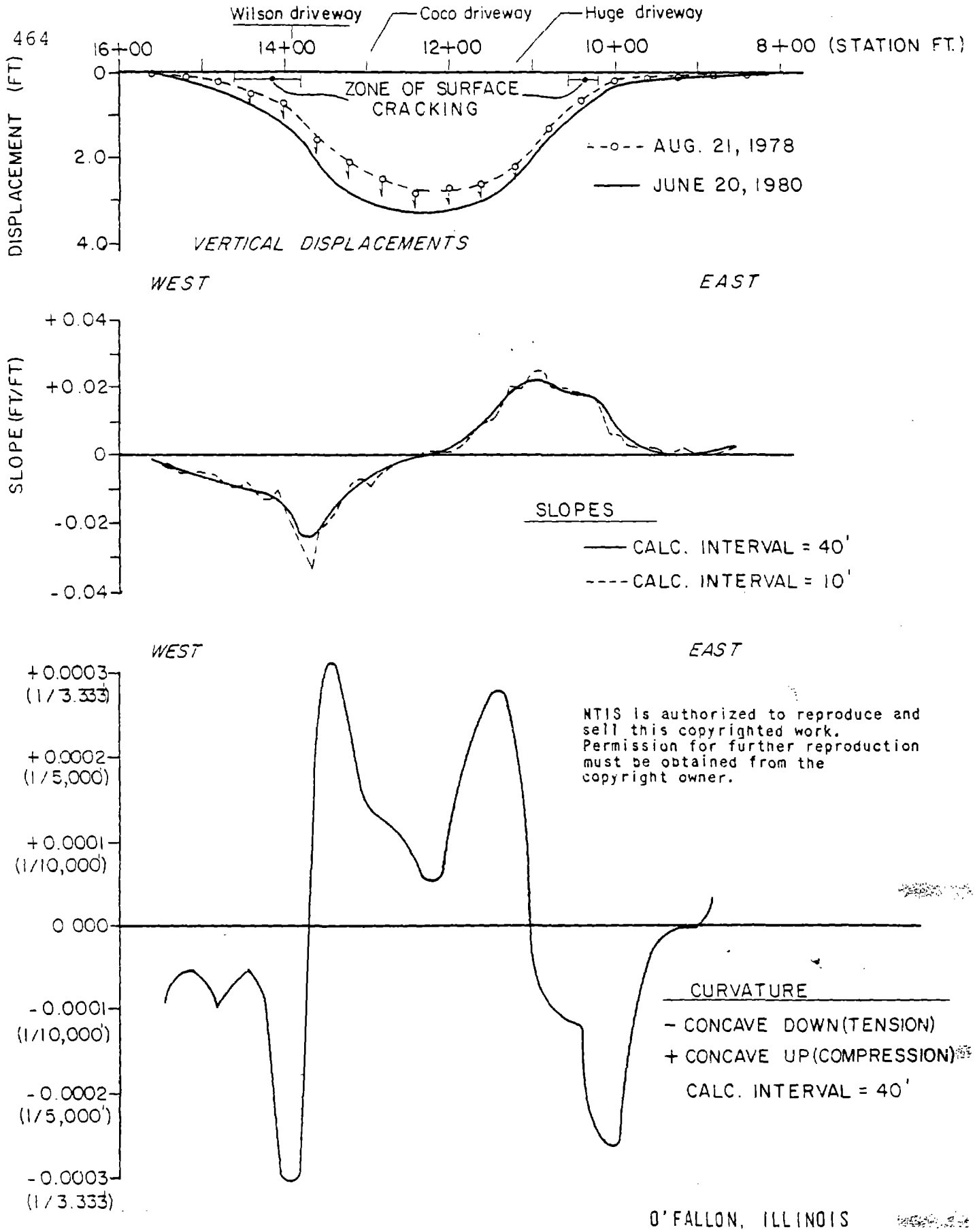
The two homes damaged by subsidence are positioned over the tensile zone on the northwest edge of the subsidence trough. They are single story, wood-framed structures measuring roughly 40- by 60-feet in plan, built in the late 1940's and enlarged since that time. The original sections of the homes have basements. One home is covered with asbestos siding, the other with stucco.

*This case history is a digest by the present writers of two reports prepared for the Illinois Abandoned Land Reclamation Council by the University of Illinois (Mahar, et al, 1979; Marino et al, 1980).



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FIGURE A.9 – PLAN OF SUBSIDENCE FEATURES AND MINED AREAS, CASE HISTORY 4 (MARINO et al, 1980)



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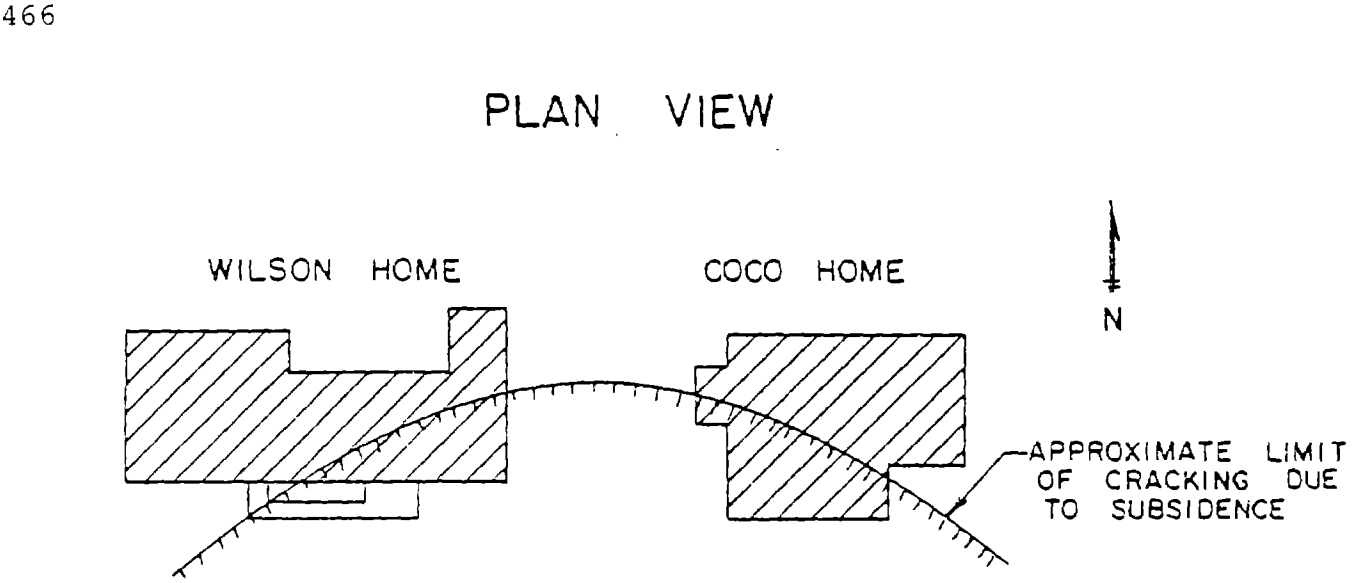
FIGURE A.10-ADJUSTED PROFILE, SLOPE, AND CURVATURE ALONG EAST-WEST SECTION, CASE HISTORY 4 (MARINO et al, 1980)

TABLE A.1

SUMMARY OF SUBSIDENCE TROUGH CHARACTERISTICS AT THE STUDY SITE

(MARINO et al,1980)

MAXIMUM DIAMETER		450 ft 735 ft	(Crack Limit) (Affected Surface)
MINIMUM DIAMETER		∿ 350 ft ∿ 410 ft	(Crack Limit) (Affected Surface)
MAXIMUM SETTLEMENT		3.25 ft	
MAXIMUM SLOPE		0.024 ft/ft	(Measured Length = 40 ft)
MAXIMUM CURVATURE			
	COMPRESSION	3.13×10^{-4} ft^{-1}	(Measured Length = 40 ft)
	TENSION	3.00×10^{-4} ft^{-1}	
ANGLE OF BREAK		10°	
ANGLE OF DRAW		45°	
$\frac{\text{SETTLEMENT}}{\text{SEAM HEIGHT}}$		0.54	
$\frac{\text{PANEL WIDTH}}{\text{PANEL DEPTH}}$		1.9 - 2.0	



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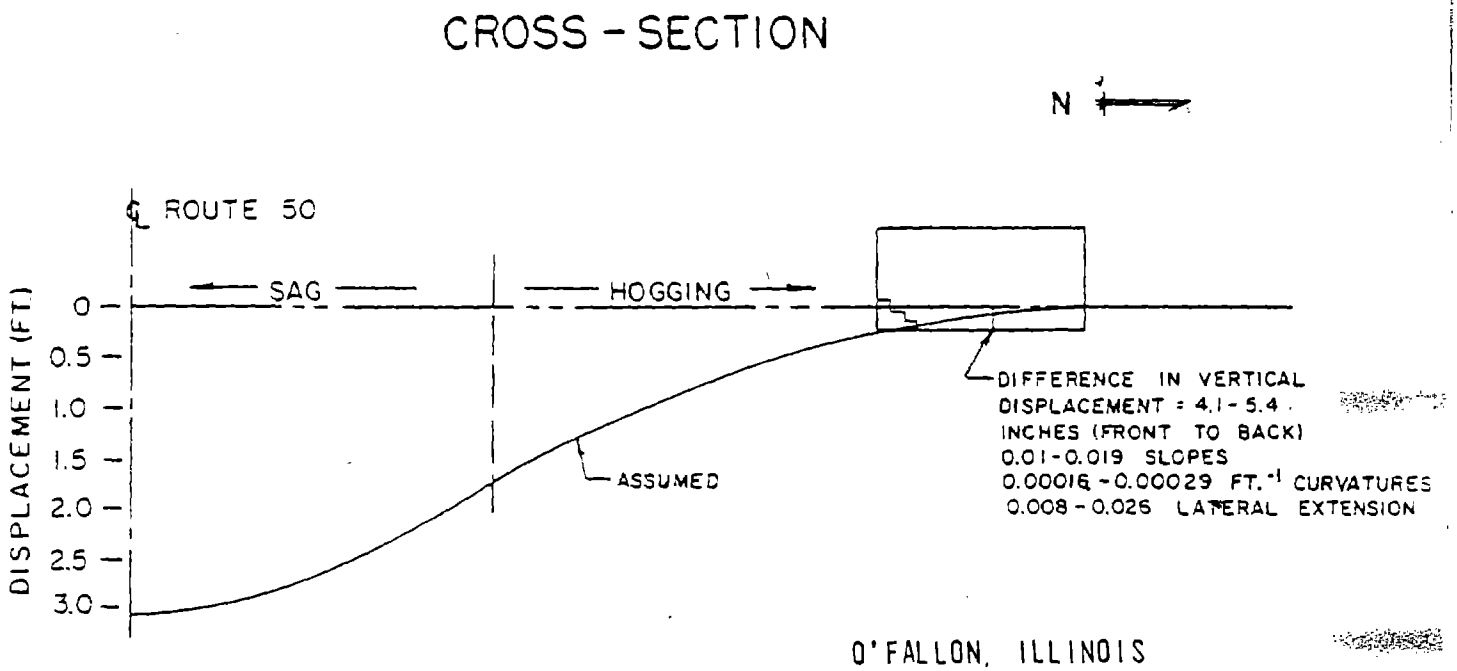


FIGURE A.11- PLAN AND CROSS-SECTION VIEWS OF COCO AND WILSON HOMES RELATIVE TO THE SUBSIDENCE TROUGH, CASE HISTORY 4 (MARINO et al, 1980)

TABLE A.2
ESTIMATES OF GROUND INDUCED STRAINS
O'FALLEN, ILLINOIS
(MARINO et al,1980)

LOCATION	DATA	CALCULATED ANGULAR DISTORTION
In vicinity of cracking on Route 50	Vertical displacements (Fig. 5.5)	max. 0.010 min. 0.0025
Wilson Home:		
south basement wall	Levels on common mortar joint	0.0056
east basement wall	Levels on common mortar joint	0.0137
southeast basement walls	Levels on common mortar joint	0.0152
Coco Home:		
west basement wall	Levels on common mortar joint	0.0036
west basement wall + porch slab	Levels on common mortar joint and across porch slab	0.0036
east basement wall	Levels on common mortar joint	0.0036
southeast across porch slab	Levels on common mortar joint and across porch slab	0.0068
exterior-south-block wall	Levels on common mortar joint	0.0083

		CALCULATED EXTENSION
In vicinity of cracking on Route 50	1/2 to 1 times the change in slope (O'Rourke et al., 1976)	max. 0.010 min. 0.0013
Wilson Home:		
south basement wall	cracks at base (Fig. 6.15)	0.0008
east basement wall	cracks at base (Fig. 6.15)	0.0125
southeast across basement floor	floor cracks (Fig. 6.15)	0.026
Coco Home:		
east basement wall	cracks at base (Fig. 6.7)	0.0007
west basement wall	cracks at base (Fig. 6.7)	0.001
basement floor	floor cracks (Fig. 6.7)	0.001
exterior east wall	cracks in wall (Fig. 6.6)	0.0032
porch-front addition	block-wall separation (Fig. 5.6)	0.025

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During subsidence, the homes deformed by hogging and were subjected to horizontal as well as vertical ground movements. The most severe home damage occurred within the area circumscribed by ground cracks. Differential settlements across the short (40-foot) dimensions of the homes ranged from 4 to 5 inches. Ground slopes ranged from 0.010 to 0.019 and curvatures from 1.6 to 2.9 x 10^{-4} foot (Figure A.11).

Only the addition to the easterly home experienced structural damage, which was associated with tilting and lateral movements. Lateral movements in the basement measured 1 to 2 inches and a separation developed between the addition and the original structure. Cracks in the basement tended not to extend upward into the superstructure owing to the poor connection between superstructure and foundation. The superstructure simply followed the downward movements of the foundation. Damage to the exterior was mild.

Damage to the westerly home was similarly confined to the trough-ward side and was manifested largely as diagonal cracks in and slight tilt of basement walls, hairline cracks in the ceiling of the superstructure, and sticking doors and windows. The frame of the home did not tilt measurably. The superstructure deflected downward as the basement walls on the trough-ward corner of the home separated from the superstructure and moved outward. Most of the damage to the superstructure occurred where it was left unsupported. Leaks developed in the roof. Damage to the exterior of the home was minimal.

Overall, damage to the homes was considered to be severe, using the Burland/Wroth classification. A summary of lateral strains and angular distortions is presented in Table A.2. For both homes, there was very little difference in the displacement of the foundation and the adjoining soil. Marino et al. (1980) report that investigations in O'Fallon show soils within and at the edges of a subsiding area are usually less compacted than the soils outside the affected area. The recompaction of the soils results in the uneven downward displacements of the ground surface. The possible implication of this phenomenon on observed movements or home damage at O'Fallon was not made known. It is understood that monitoring of home damage, groundwater levels, and overburden movements at and below ground surface continue at the time of this writing (1982).

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Case History 5

Subsidence Above an Abandoned Mine Accompanied by Dilatency, Compaction and Piping of Poorly Consolidated Overburden Material*

The State Police Headquarters Building at Maryville, Illinois, is suspected to have first experienced subsidence on May 20, 1980. The headquarters building is a one-story structure, built in 1964, with reinforced cinderblock bearing walls and a brick facade, founded on a reinforced concrete, wall footing bearing four feet below grade. The floors are slab-on-grade, underlain by two feet of sand. A four foot square utility tunnel is located inside most of the exterior foundation walls. The roof is supported on steel joists. Twenty-five feet south of the building is a 350-foot high communications tower founded on a two-foot by two-foot reinforced concrete pier extending twelve feet below ground surface and supported laterally by guy wires at third points around the perimeter.

Subsidence took the form of a subtle ground feature of elliptical shape, approximately 400 feet long in a north-south direction and 240 feet long in an east-west direction (Figure A.12). Only about one-quarter of the trough was circumscribed by ground cracks. Subsidence was time dependent. During the period of reporting in mid- to late-1980, the center of the sag dropped at a rate of about 0.25 inch per week. Occasional shallow depressions a foot or so in diameter developed in the parking lot as a result of piping from infiltrating surface water. Some ground movement was also attributed to the dilatency and subsequent compaction of the shallow unconsolidated or poorly consolidated overburden in response to subsidence. No attempt was made to identify the individual contributions of the three mechanisms of ground movements. Overburden at the site consists of 85 feet of soil (about 60 percent, layers of loess, the remainder, layers of glacial till), overlying 150 feet of shale and claystone interbedded with lesser amounts of limestone and siltstone (Figure A.13).

The site was undermined some time prior to the mid-1920's by room and pillar methods that recovered about 57 percent of the Herrin No. 6 Coal Seam (Figure A.14). The mine is located 230 to 240 feet below ground surface. Mine height was 6.5 to 7.5 feet. The mine reportedly experienced some floor heave while in operation.

The subsidence problem was not originally considered serious. As ground movements progressed, this view was revised.

The headquarters building is situated on the northeast edge in the zone of maximum tension. The south and southwestern portions of the building experienced extensive damage. The south exterior wall settled differentially over its 77-foot length. Hogging led

*This case history is a digest by the present writers of a report prepared for the Illinois Abandoned Mined Land Reclamation Council by the University of Illinois (Wildanger et.al, 1980).

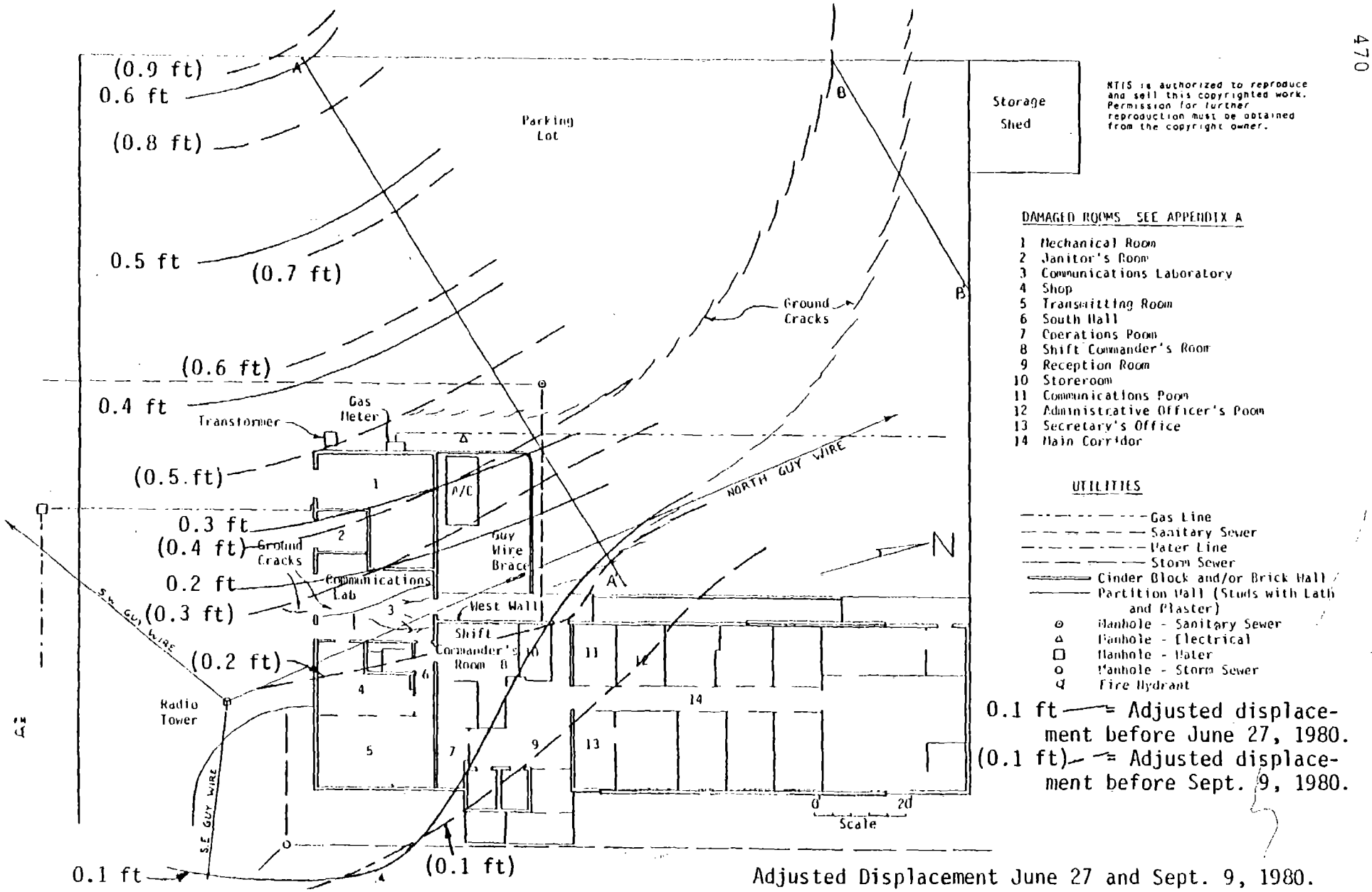


FIGURE A.12- PLAN OF SUBSIDENCE FEATURE, STATE POLICE HEADQUARTERS BUILDING MARYVILLE, ILL., CASE HISTORY 5
(from WILDANGER, et al. 1980)

GEOLOGIC UNIT	% Clay	% Silt	% Sand	Shear Strength (TSF)	% Moisture/RQD	NOTES
PECRIA LOESS (thickness = 15.3')	12.9	86.4	0.7	*0.7 - >1.65	w_p = 24 - 25% w_L = 28 - 32% w = 26 - 32%	Coarse wind blown silt; yellowish brown to brownish gray in color; calcareous below modern soil development
ROXANA SILT (thickness=14.3')	16.3	81.5	2.0	**0.1-1.25	w = 16.6 - 30.6%	Coarse wind blown silt; four color zones pink (uppermost), gray/brown pink, and gray/brown (lowermost); contains some dolomite in upper zone gray/brown zone
FORT RUSSELL TILL (thickness=21.7')	24.0	42.0	34.0	**0.8->4.5	w = 11.8 - 23.6%	Till, massive, loam texture, calcareous; color is gray and oxidizes to gray/brown
CHINATOWN SILT (3.0' - 8.2')						Loess grading into colluvium or a pedisediment; brown silty clay loam
OMPHGHENT TILL (11.8' - 14.8')	37.0	45.0	18.0	**1.5 - >4.5	w = 12.9 - 21.8%	Till, massive, silty clay loam, calcareous; olive brown in color
BURDICK LOESS (3.2' - 17.7')	31.6	66.9	1.5	**1.4-3.7	w = 17 - 25%	Leached, silt loam to silty clay loam
SHALE (42' - 56')					RQD = 85%	
POCK BRANCH CLAYSTONE (0.2')						
CLAYSTONE (26')					RQD = 35%	
PIASA LIMESTONE (2.5')					RQD > 90%	
SHALE AND CLAYSTONE (8.0 - 9.6')					RQD > 90%	
CLAYSTONE AND SILTSTONE (0.7-0.8')					Claystone RQD = 75% Siltstone RQD = 90%	
BRERETON LIMESTONE (3.3 - 13.6')	ANNA SHALE (thickness = 0.0')				RQD = 75% 90% 85%	
HERRIN (No.6) COAL (2.0')						
CLAYSTONE (1.2 - 9.2')					RQD <10%; 50%; 25%	

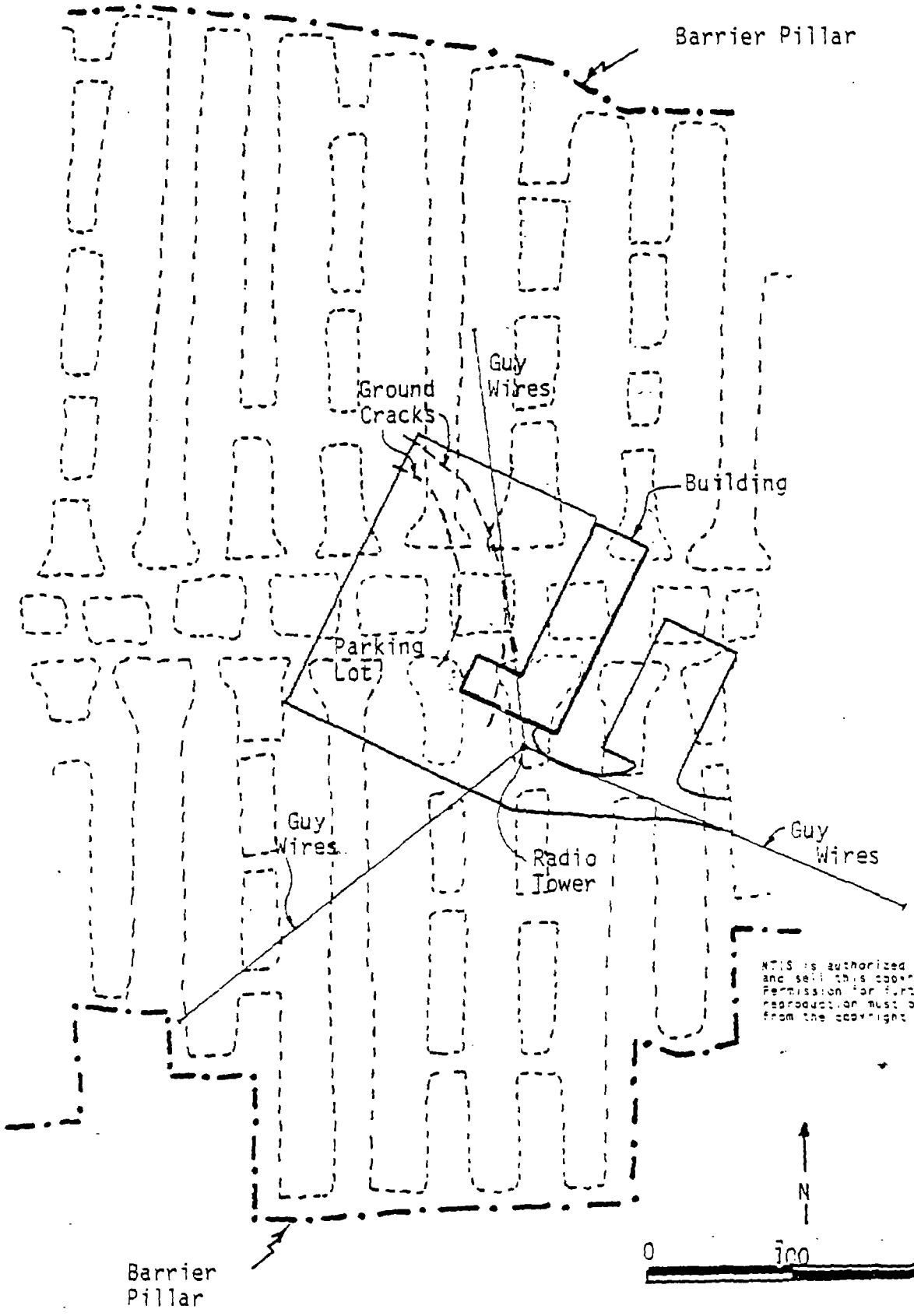
1. *Field vane shear strength test
   **Standard field penetrometer test
2. w_p = plastic limit
   w_L = liquid limit
   w = natural moisture content
3. Grain size composition:
   sand 2mm-62μ; silt 62 - 2μ; clay <2μ
4. Field vane shear measurements and moisture content were sampled below the modern soil.
5. RQD = Rock Quality Designation
6. This data represents selected values from three diamond drill cores and does not include all of the data presented in the ISGS paper.

Physical properties of bedrock cores after McKay, et al. (1980).

Physical properties of soil (Quaternary) deposits from McKay (1979), McKay, et al (1980) and Gibson (1979).

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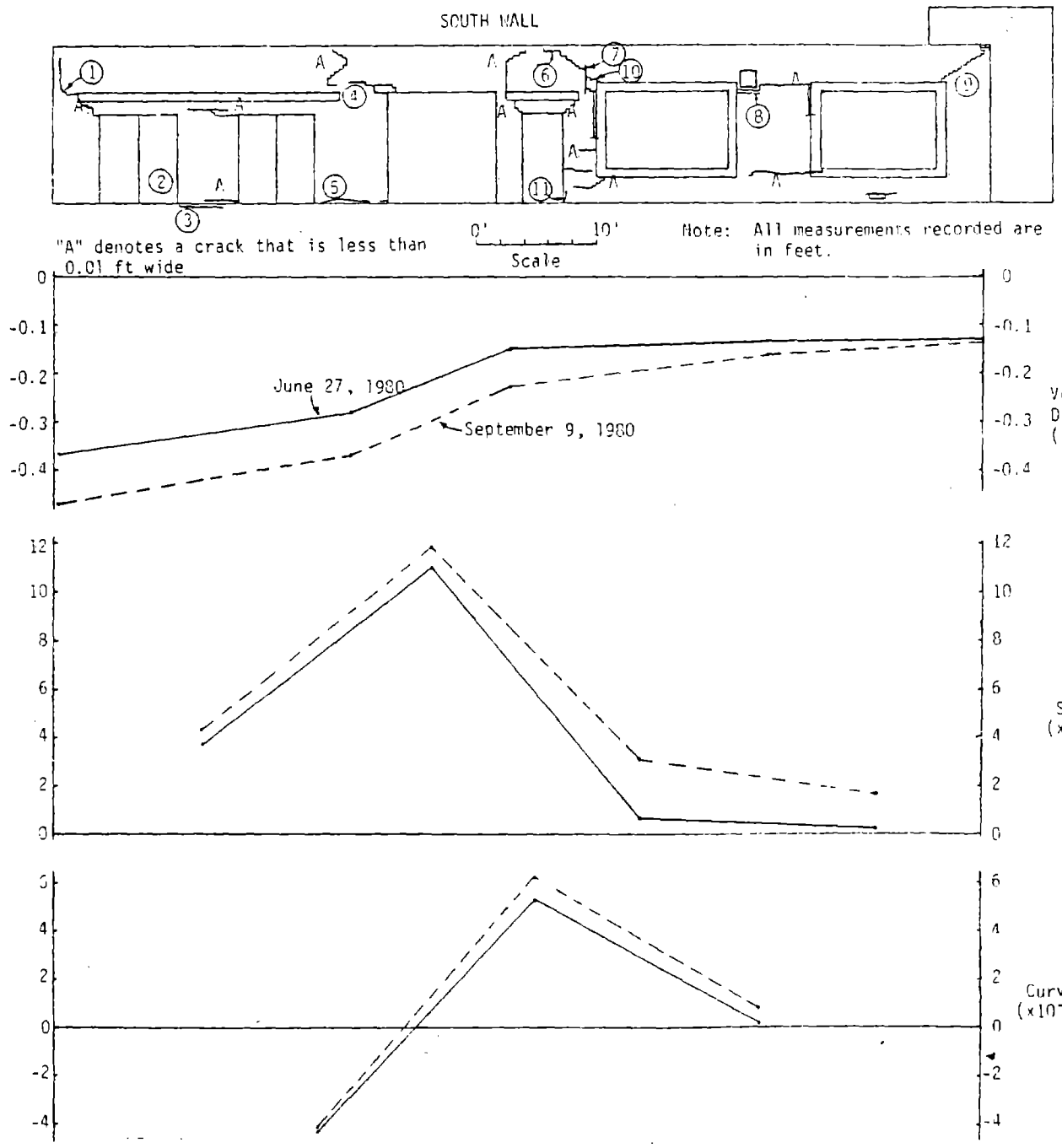
FIGURE A.13 - GEOLOGIC COLUMN, STATE POLICE HEADQUARTERS BUILDING MARYVILLE, ILL. (from WILDANGER et al, 1980)



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FIGURE A.14-LOCATION OF STATE POLICE BUILDING IN RELATION TO THE MINED OUT AREA, CASE HISTORY 5 (from WILDANGER et al. 1980)

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FIGURE A.15- DAMAGE AND MOVEMENTS ALONG SOUTH WALL, STATE POLICE HEADQUARTERS BUILDING, MARYVILLE, ILL., CASE HISTORY 5 (from WILDANGER, 1980)

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to more severe distortions at midlength of the wall (Figure A.15). Damage appeared as stairstep diagonal cracks and horizontal cracks at mortar joints, often emanating from corners of windows and doors. Uneven tilting induced bending and torsion in the west wall, accompanied by vertical tension cracks and horizontal tension and shear cracks. Outward movement of the walls removed support from a section of the roof. The only utility damage was a leak where the gas line connected with the gas meter.

The radio tower, located in the tension zone of the sag, has experienced primarily horizontal movement. As of September 1980, tilt was less than one-tenth of a degree. Movements of the guy wire anchor blocks have necessitated periodic adjustments of guy wire tension.

Other emergency measures implemented to mitigate the effects of subsidence included the installation of temporary adjustable support bracings, a special roof membrane, natural gas detectors and alarm systems. Ground cracks were filled with bentonite or other suitable materials to reduce infiltration and reduce the possibility for subsurface erosion. Limited site grading was performed to direct drainage away from the building. A structural analysis was conducted of the radio tower.

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Case History 6

Similar Overburden Characteristics, Dissimilar Ground Settlement above two Abandoned Mines

Two subsidence incidents occurred within a mile of one another in Natrona Heights, Pennsylvania. Physical features of the two sites are much the same but surface ground movements resulting from failure of the overburden were quite dissimilar (Table A.3). Both were undermined by the Pennsylvania Salt Manufacturing Company's Brackenridge Mine more than 70 years ago. (Figure A.16 illustrates the distribution of narrow pillars left). The mined Upper Freeport coal is 4 to 5 feet thick in both cases. Depth to mine-level is 150 to 175 feet, about 90 feet of the overburden is rock, 40 to 75 feet is unconsolidated sands, silts and gravels, and 600 to 700 foot diameter area was affected by ground movements at both sites. Although subsidence was less than a foot in both incidents, that at Site A was several times more than that at Site B.

Incident A. Ground movements began on January 13, 1964, and continued steadily for three days. Six homes were damaged to varying degrees--two seriously. On January 20, further subsidence movements were reported when several water lines were ruptured along the alley north of Homes A, B, and C (Figure A.17).

Ground cracks were evident along much of the boundary of the affected area and apparently coincided with basement cracks in Homes A, D, E, and F.

Table A.3

COMPARISON OF SITE CHARACTERISTICS
NATRONA HEIGHTS, PENNSYLVANIA

Similarities	Incident A	Incident B
Overburden Thickness	150-170 ft.	175 ft.
Soil Type (thickness)	Loose sand, silty sand some gravel (unknown-probably 40-75 ft.)	Loose sand, sandy silt, and some gravel (88 ft.)
Bedrock Thickness	Unknown-probably 80-100 ft.	87 ft.
Name and (thickness) of Coal Mined	Upper Freeport (4-5 ft.)	Upper Freeport (4-5 ft.)
Approximate Area Affected by Ground Movements	410,000 sq. ft.	500,000 sq. ft.

Similarities	Incident A	Incident B
Dissimilarities		
Number of Homes Damages	6	15
Topographic Relief	15-20 ft.	2-3 ft.
Date of Last Mining	1910	1850
Date of Surface Subsidence	January 13-20, 1964	September 6-17, 1969
Duration of Surface Subsidence	January 13, 1964-6 homes damaged-1 week January 20, 1964-several water lines-1 day	September 6-8 homes damaged-1 day September 14-6 homes damaged-1 day September 17-1 home damaged-1 day
Approximate Downward Movement	5-8" first episode Approx. 2" second episode	1" to 2" Total
Ground cracks	Observed	Not observed

Homes B and C, located closest to the center of the trough, suffered the most severe damage. Both are 1-story, wood-frame structures built in the early 1950's and have cinderblock walled basements with concrete slab floors. All four walls in the basements of each home were extensively cracked and two of the walls in each basement bulged inwards as much as 3 inches along horizontal cracks that extended 1/3 to 1/2 the length of the longest walls. Both basement floors were also slightly cracked. Home B suffered little damage to the superstructure--the chimney tilted slightly, a few interior plaster walls suffered minor cracking, and the front door would not close. Home C experienced more extensive damage to the superstructure--the plaster walls in the center of the structure were cracked, as were all of the ceilings; the roof bowed slightly and leaked around the chimney; doors and windows would not function properly, and the exterior brick facing above the bowed basement walls separated along mortar joints 0.25 to 0.5 inches. The gas pipe leading into the home was pushed 8 inches into the basement and the walls and floor of the detached garage were badly cracked. Both Homes B and C were tilted about 1.5 inches toward the center of the trough.

Damage to the other homes at the site consisted of minor cracks in the basement walls. The home immediately adjacent to Home C reported no damage.

Sampling of near-surface soils during the present study revealed that most of the homes within the subsided area are probably founded on an unknown thickness loose to medium dense, fine sand (Figure A.18). Total overburden thickness is 120 to 175 feet. Unconsolidated sediment thickness is 40 to 75 feet. Topographic



FIGURE A.16 – DISTRIBUTION OF COAL PILLARS BELOW HOMES REPORTING SUBSIDENC DAMAGE, CASE HISTORY 6

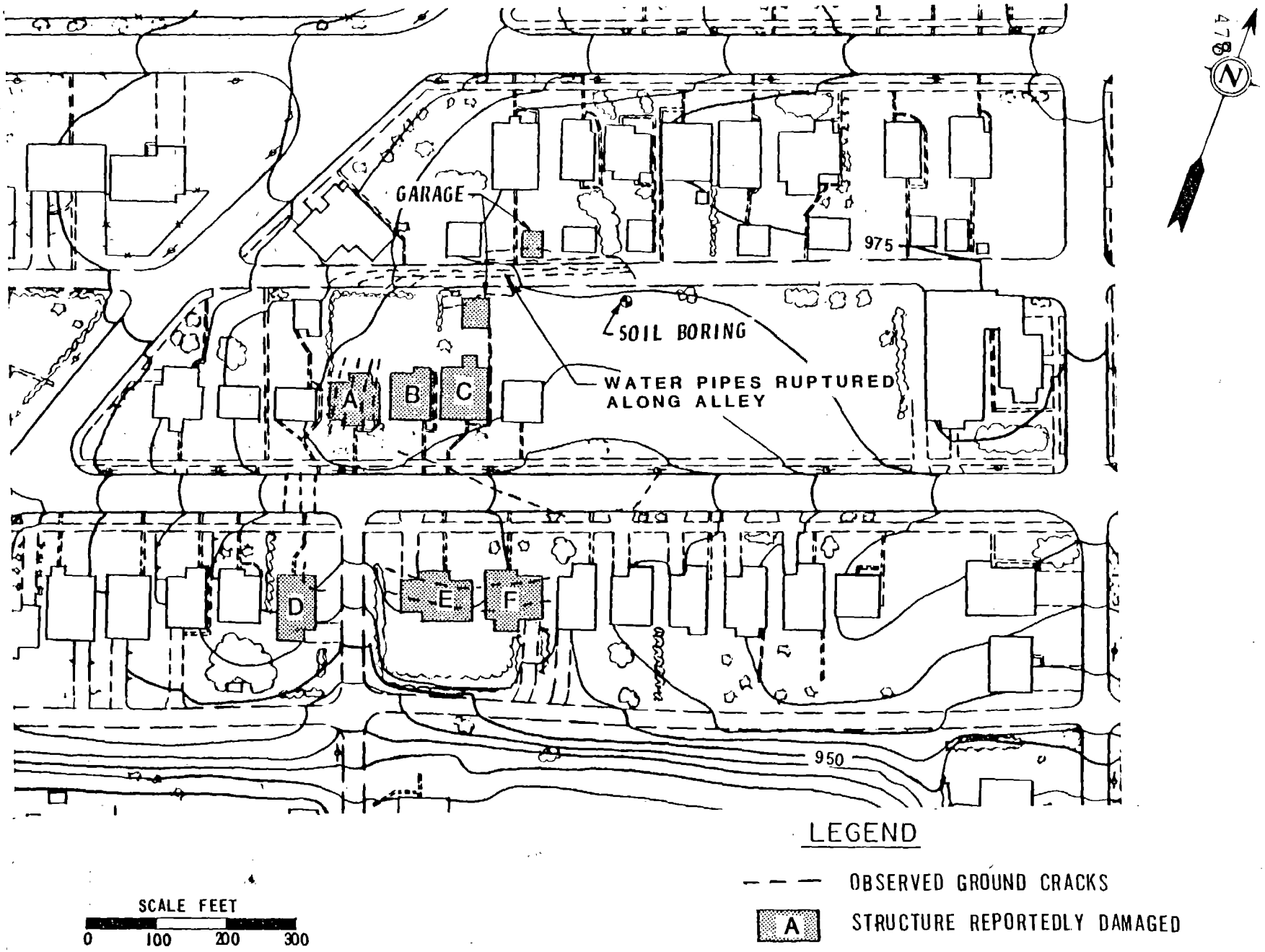


FIGURE A.17– SITE OF 1964 SUBSIDENCE INCIDENT, NATRONA HEIGHTS, PENNSYLVANIA, CASE HISTORY 6

INCIDENT A

DEPTH (FEET)	PROFILE	MATERIAL CLASSIFICATION*	SOIL DENSITY OR CONSISTENCY	STANDARD PENETRATION RESISTANCE (BLOWS/FOOT
0		FILL-SOME BRICK FRAGMENTS, DARK BROWN IN COLOR	LOOSE	6
2.0		SILTY SAND (SM)* – SOME ROCK FRAGMENTS AND TRACE OF WOOD, BROWN IN COLOR WITH BLACK MOTTLING, PROBABLY FILL	LOOSE	5
4.1		SANDS & GRAVELS – SOME SILT , ORANGE BROWN WITH SOME GRAY IN COLOR	LOOSE	6
5			MEDIUM DENSE	21
				12
10				20
			LOOSE	6
15				9
16		BOTTOM OF HOLE		

INCIDENT B

DEPTH (FEET)	PROFILE	MATERIAL CLASSIFICATION	SOIL DENSITY OR CONSISTENCY	STANDARD PENETRATION RESISTANCE (BLOWS/FOOT
0		SANDY SILT AND COAL/ASH FRAGMENTS (SM), DARK BROWN IN COLOR, PROBABLY FILL.	VERY LOOSE	2
2.25		CLAYEY SAND-TRACE SILT(SC), YELLOW BROWN AND GRAY IN COLOR	LOOSE	9
5			MEDIUM DENSE	21
				14
8.25		SILTY FINE SAND (SM), YELLOW BROWN IN COLOR		17
9.10		SAND, SOME GRAVEL (SP), GRAY IN COLOR	MEDIUM DENSE	
10				24
				30
14.0		BOTTOM OF HOLE		

*SEE FIGURE A.61 FOR UNIFIED SOIL CLASSIFICATION SYMBOLS AND DEFINITIONS

FIGURE A.18–SOIL BORINGS, NATRONA HEIGHTS, PENNSYLVANIA, CASE HISTORY 6

relief at the site is 15 to 20 feet. Coal mining below the site was last conducted in 1910.

Incident B. The first documented episode of subsidence at this location took place on the morning of September 6, 1969, when the owners of eight homes (Figure A.19) noticed cracking of their basement walls. Eight days later, on September 14, 1969, ground movements again damaged eight homes, two of which had also been damaged on September 6. On September 17, an additional home reported damage. Upon examining each of the 15 homes, state investigators surmised that minor amounts of subsidence had probably occurred in the past, since screw jacks had been placed in many of the basements as early as 15 years before the incident under investigation. Most of the homes in this community were built in the 1920's and 1930's.

Other than a very shallow 10-foot diameter trough, 2 to 5 inches deep, that formed in front of Home F, no changes in ground elevations were noted by the owners of the damaged homes. However, judging from the areal extent of damage, as many as 500,000 square feet may have been involved in subtle ground movements imperceptible to the homeowners. The size of the affected area apparently increased with each successive subsidence episode. In general, the ground movements and accompanying home damage progressed from southwest to northeast. The resident of Home F claims that very slight movements still occurred as of 1981, as evidenced by movement along existing basement cracks. State inspectors in 1969 estimated, from an examination of building deformations, that approximately 1 to 2 inches of downward movement occurred within the affected area.

As a result of the first subsidence episode (September 6) that affected eight homes, three homeowners reported a cracked cellar floor, and six claimed that one to four of their basement walls had been cracked. The cumulative width of cracks in any one wall varied from 1 to 1.5 inches. Gas pipelines entering two of the homes were ruptured. The building most seriously damaged during the September 6 episode was Home F, a two-story, wood-frame structure clad with aluminum siding that was built in 1928. The basement consists of cinderblock walls and a concrete slab floor. Its construction is similar to that of the other homes on this street. Damage was most severe in the basement where all four walls were cracked and the western wall was bowed inward about two inches. A horizontal crack formed as a result of the bowing and extended almost the entire length of the wall. The plaster walls in the center of the superstructure were slightly cracked and some water leakage around the chimney occurred as a result of slight bowing of the roof. Windows and doors would not open or close properly and several windows had to be replaced. The gas line from the street was pushed eight inches into the home.

Neighboring Home E was less damaged, but all of the basement walls and the floor were cracked and two of the walls bulged slightly inwards along horizontal cracks that extended 1/3 to 1/2



FIGURE A.19 - SITE OF 1969 SUBSIDENCE INCIDENT, NATRONA HEIGHTS, PENNSYLVANIA, CASE HISTORY 6

the length of the basement. The chimney tilted a few degrees and one exterior door would not close.

The second subsidence episode (September 14) affected eight homes, two of which had been damaged by the first episode. Four homes experienced minor cracking to the basement walls; two homes suffered ruptured gas lines; and two homes experienced hairline cracking of several interior plaster walls.

Only one structure was affected by the third episode (September 17). Damage was very slight.

Coal mining was last conducted below the site in 1850 by the Pennsylvania Salt Manufacturing Company. The site is essentially flat, with a topographic relief of 2 to 3 feet. Shallow drilling shows the soil to be 8.25 feet of very loose to medium dense sandy silt overlying an undetermined thickness of medium grained sand. None of the homes was founded on rock. At least 88 feet of the overburden consists of unconsolidated sediments. In 1971, the portion of the mine below this group of homes was backfilled by the Commonwealth.

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Case History 7

Dissimilar Overburden Characteristics, Similar Ground Movements and Resulting Damage above Two Abandoned Mines

In September, 1980, and in October, 1981, separate subsidence incidents occurred above abandoned mines in Baldwin (Pittsburgh), Pennsylvania and 120 miles south in Shinnston, West Virginia. Comparison of the two cases reveals that, even though the geologic features of the two sites are rather dissimilar (Table A.4), the resultant subsidence troughs and damage to the surface structures are much the same (Table A.5).

Although located in communities where the homes are relatively closely spaced, only a single home was damaged at each site. The basements of both homes were severely damaged, the wood frame superstructures were badly distorted and the brick/ stone facings were severely cracked. The cases are discussed below.

Baldwin, Pennsylvania - Subsidence began in the early morning of September 22, 1980, while the owners were away on vacation. A passing neighbor thought he noticed something "different" about the home, but wasn't certain what it was. The neighbor's wife periodically checked the home and by mid-morning reported that it seemed to be moving downwards. By the time the neighbor returned home from work at noontime, large horizontal cracks had developed in the front brick facing. The neighbors, fearing eventual collapse of the structure, notified the owners and the local authorities and began removing furniture. Perceptible movement of the structure continued through the next day. Very slow, sporadic movements occurred over the following six months.

Adverse effects on the home began when the front portion of the integral garage and adjoining basement started settling more rapidly than the rest of the basement (Figure A.20). At first, displacements of the brick-clad, wood-frame superstructure conformed with those of the basement, but as settlement continued, the basement separated locally from the stiffer superstructure. In the process, long, horizontal cracks up to 5 inches wide formed in the front and side brick facing. In the basement, three of the walls and the floor were severly cracked. Significant tilting was confined to the front basement wall.

The shapes and positions of the subsidence trough and the damaged building are illustrated in Figure A.21. The subsidence trough, as defined by the ground cracks, was almost circular in shape, 75 feet in diameter, and nearly encompassed the affected home. As defined by the limit of ground movements, the diameter of the trough is estimated to be 80 feet. The greatest downward movement took place below the front southeast quarter of the home. Ground cracks up to 2 inches wide formed where the change in slope of the ground was most pronounced. A total of 22 inches of downward differential ground movement was measured between the northwest rear corner of the basement and the center of the trough.

Table A.4

DISSIMILARITIES OF SITE CHARACTERISTICS
BALDWIN, PENNSYLVANIA AND SHINNSTON, WEST VIRGINIA

Particulars	Baldwin, Pennsylvania	Shinnston, West Virginia
Date of last mining	1936	1913-last mine development, early 1950's-indiscriminant pillar robbing
Depth to mine	77 feet	117 feet
Soil overburden thickness	25 feet	17 feet
Soil description	Colluvium/residual soil (silts and clays) overlain by fill of unknown composition	Raised terrace deposit-silts, sands, clay, with some gravel
Rock overburden thickness	52 feet	100 feet
Rock description Pennsylvanian Age sedimentary strata	Sandstone with some shale	Approximately equal amounts of shale and limestone
Mode of building deformation	Unsymmetric hogging	Unsymmetric sagging

Table A.5

SIMILARITIES OF SITE CHARACTERISTICS
BALDWIN, PENNSYLVANIA AND SHINNSTON, WEST VIRGINIA

Particulars	Baldwin, Pennsylvania	Shinnston, West Virginia
General Data		
Number of homes damaged	One	One
Distance to nearest adjacent home	approximately 40 feet	approximately 40 feet
Site Topographic relief	10 feet	10 feet
Building Description		
Height	One story	One story
Construction superstructure	wood frame	wood frame
basement	cinderblock walls and concrete slab floor	cinderblock walls and concrete slab floor
exterior	brick facing	stone facing in front, aluminum siding on other three sides.
Area of basement	1580 ft^2	1080 ft^2
Subsidence Damage		
Superstructure interior	severe cracking of two or three plaster walls, few cracks in other plaster walls; tilted floors, doors and windows out of square	severe cracking of two or three plaster walls; few cracks in other plaster walls; tilted floors; doors and windows out of square
Basement	two walls severely cracked and buckled; two walls moderately cracked; floor distorted	all walls severely cracked and buckled; two walls moderately cracked; floor distorted

Table A.5
(Continued)

Particulars	Baldwin, Pennsylvania	Shinnston, West Virginia
Superstructure Exterior	severe horizontal and diagonal cracking of two walls, minor cracking of others	severe horizontal cracking of front stone facing; warped aluminum siding
Subsidence Data		
Date and duration of ground movement	September 22 and 23, 1980-rapid movement, next 3 months - very slow movement	October 29, 30, 31, 1981-rapid movement, next 6 months - very slow movement
Approximate amount of ultimate downward movement	22 inches	18 inches
Diameter of subsidence trough	approximately 80 feet	approximately 83 feet
Approximate area of subsidence trough	5000 square feet	5000 square feet
Geologic and Mining Data		
Name of mined coal	Pittsburgh Coal	Pittsburgh Coal
Thickness of mined coal	9 feet	8 feet
Status of mine	abandoned	abandoned

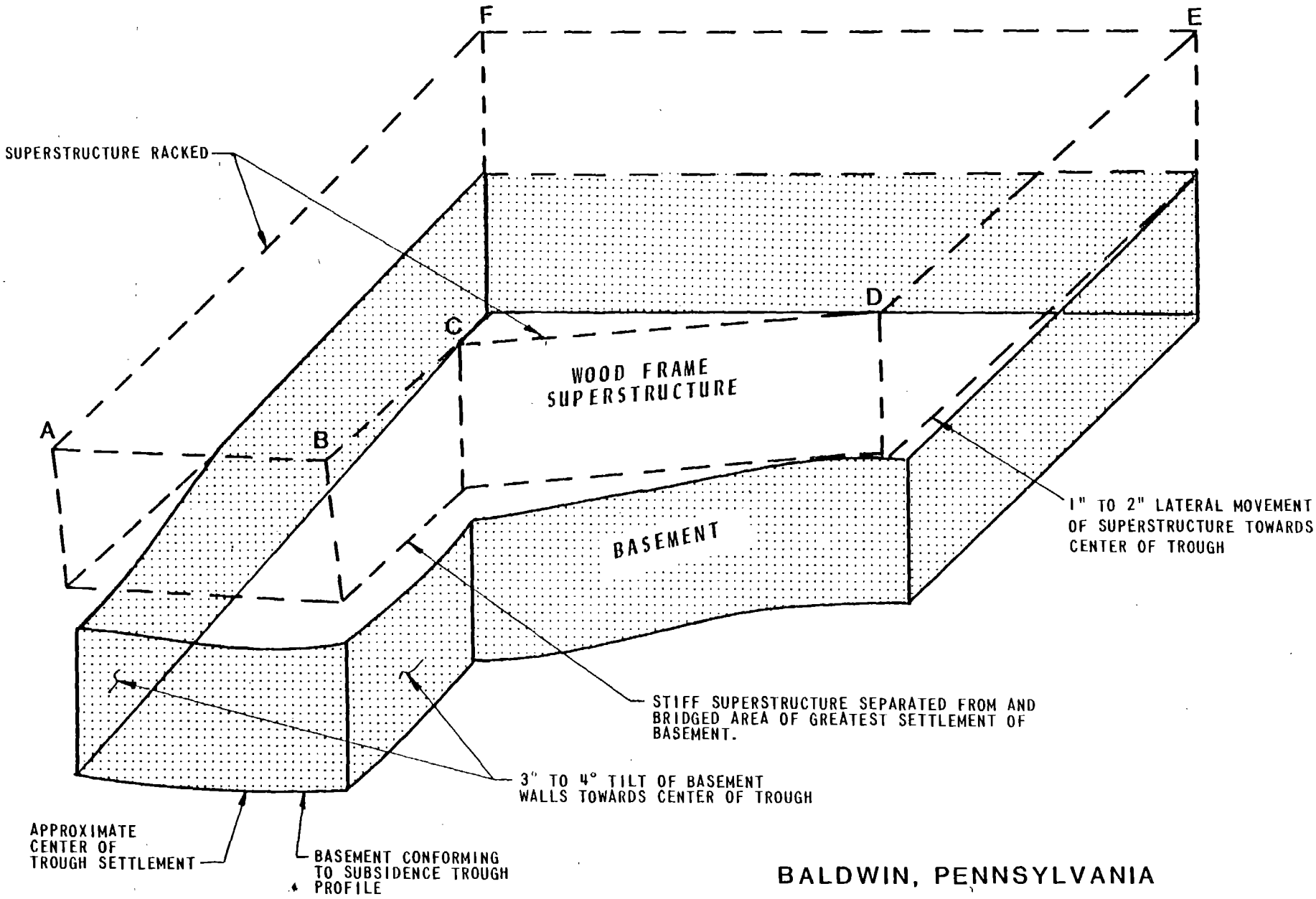
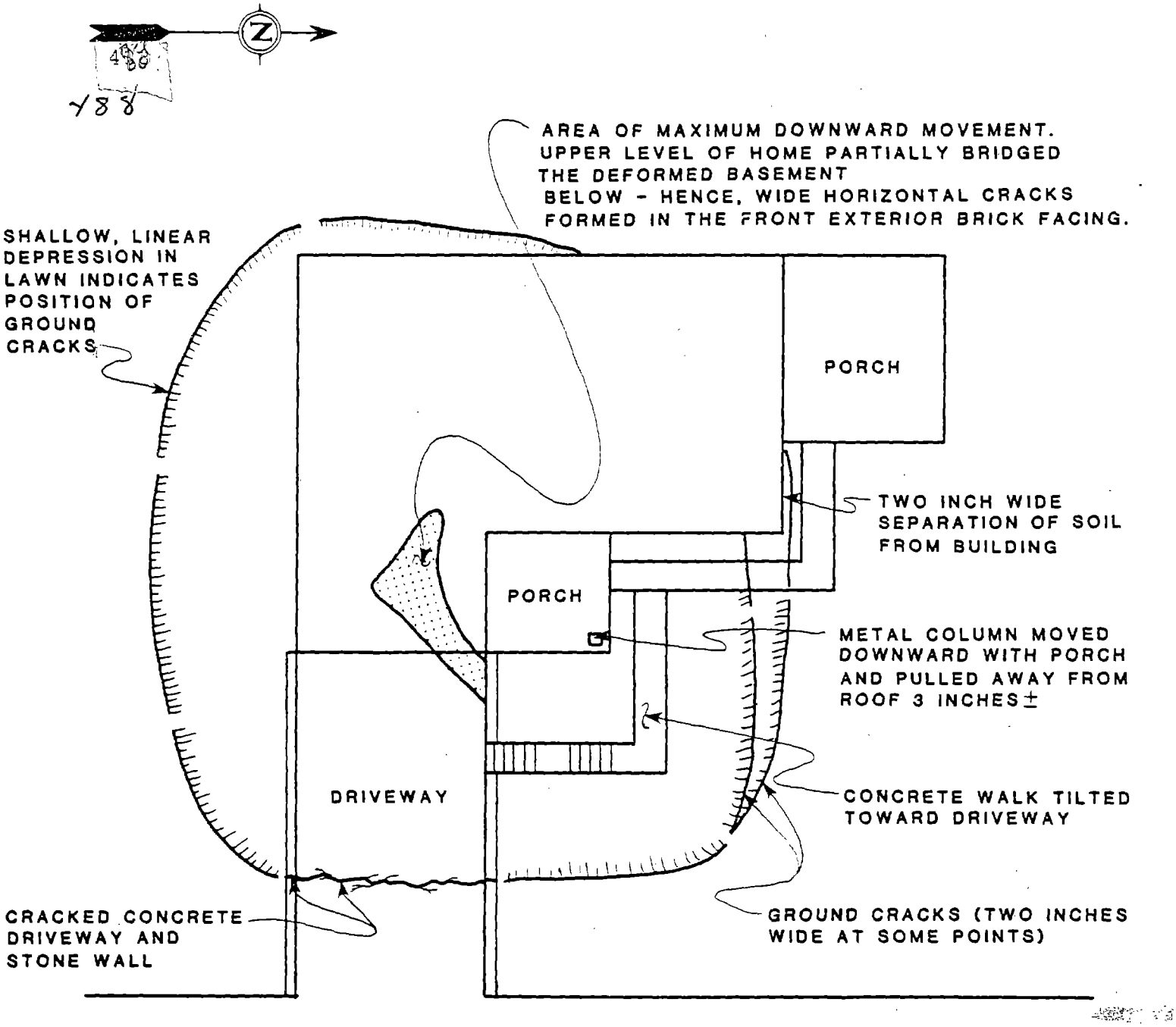


FIGURE A.20–SCHEMATIC REPRESENTATION OF DEFORMED SUPERSTRUCTURE RELATIVE TO BASEMENT DEFORMATION, CASE HISTORY 7



NOTE: THE SUBSIDENCE TROUGH AS DEFINED BY GROUND CRACKS IS NOMINALLY 75 FEET IN DIAMETER. FULL TROUGH EXTENDING TO THE LIMIT OF NO GROUND MOVEMENT IS SOMEWHAT LARGER.

SCALE
0 5 10 15
(FEET)

BALDWIN, PENNSYLVANIA

FIGURE A.21- SHAPE AND POSITION OF SUBSIDENCE TROUGH RELATIVE TO THE DAMAGED HOME, CASE HISTORY 7

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A contour map of the deformed basement floor reflects the irregularity of the subsided ground surface (Figure A.22). A profile is presented in Figure A.23. Approximately one inch of subsidence occurred after construction of the contour map and profiles.

To repair the home, the brick facing was removed from the exterior of the building, the superstructure was leveled by jacking (Figure A.24) and the basement concrete-slab floor and two of the cinderblock walls were replaced (Figure A.25). Although the interior plaster walls were severely cracked and misaligned, most major cracks closed upon releveling the superstructure. Releveling also returned doors and windows to square and corrected most of the problems associated with tilt of the superstructure.

Last mining below the damaged property was conducted in 1936 by the Becks Run Coal Mine. No mine map was available to indicate the size and location of coal pillars in the 8 to 9-foot thick Pittsburgh Coal. Eleven rotary borings to mine-level drilled by the U.S. Office of Surface Mining indicate that the mine is about 80 feet below the ground surface and a 6 to 16-foot height of rock rubble (which includes 1 to 2-foot high voids) now exists over much of the area at mine level. (One boring encountered a 10-foot high void). The boring logs also indicate that soil occupies the upper 22 to 28 feet of overburden. Experience at other nearby sites indicates that the lower 4 to 6 feet of soil is residual silty clay or colluvium and the remainder is probably fill material.

Shinnston, West Virginia - The damaged home is a one story frame structure with basement and attached garage. The basement does not extend beneath the garage. The front of the home is clad largely by stone and the other three sides by aluminum siding.

The subsidence trough that developed at the home was nominally 83 feet in diameter (Figure A.26). No ground cracks were visible. Contours on the basement floor (Figure A.27) show it to have distorted in an irregular manner partially controlled by a pre-existing expansion joint located in the center of the floor paralleling the long axis of the basement. Two months after the onset of subsidence, the maximum differential downward movement was about 1.3 feet. Approximately 0.2 feet of additional subsidence occurred after this measurement. Maximum downward movement took place near the front wall of the home.

Serious damage to the home was mainly confined to the basement, although distortion resulted in some damage to the superstructure as well. As the subsidence trough developed beneath the home, the front basement wall sagged downward relative to the adjoining basement walls, as did the floor slab (Figure A.28). The footings and adjoining basement walls attempted to conform to the profile of the subsidence trough and rotated inward toward the center of the trough. Constraint offered to the walls by their attachment to the stiff superstructure caused them to crack horizontally at or somewhat above mid-height (Figure A.29).

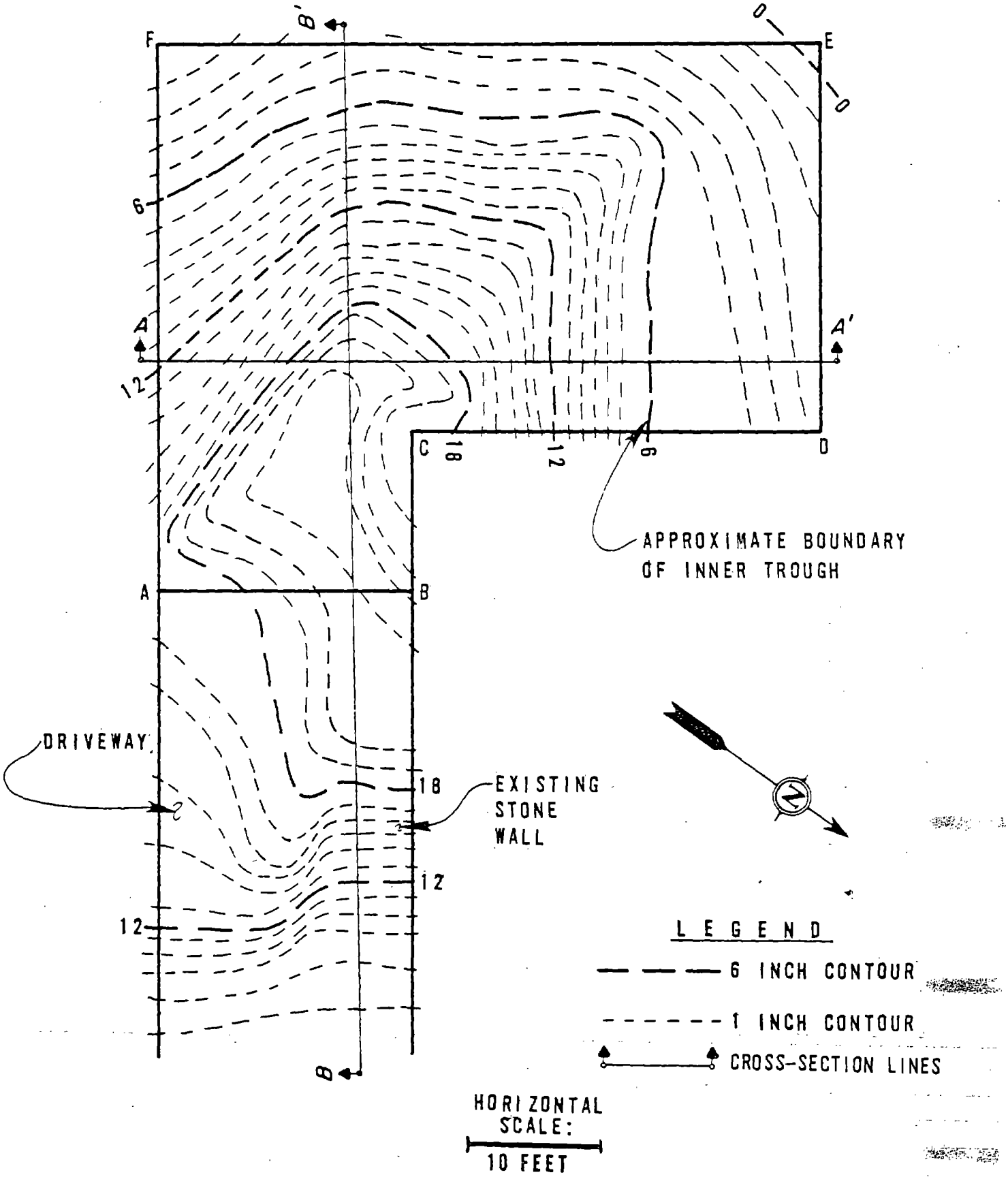


FIGURE A.22 CONTOUR MAP OF THE DEFORMED BASEMENT FLOOR AND DRIVEWAY, CASE HISTORY 7.

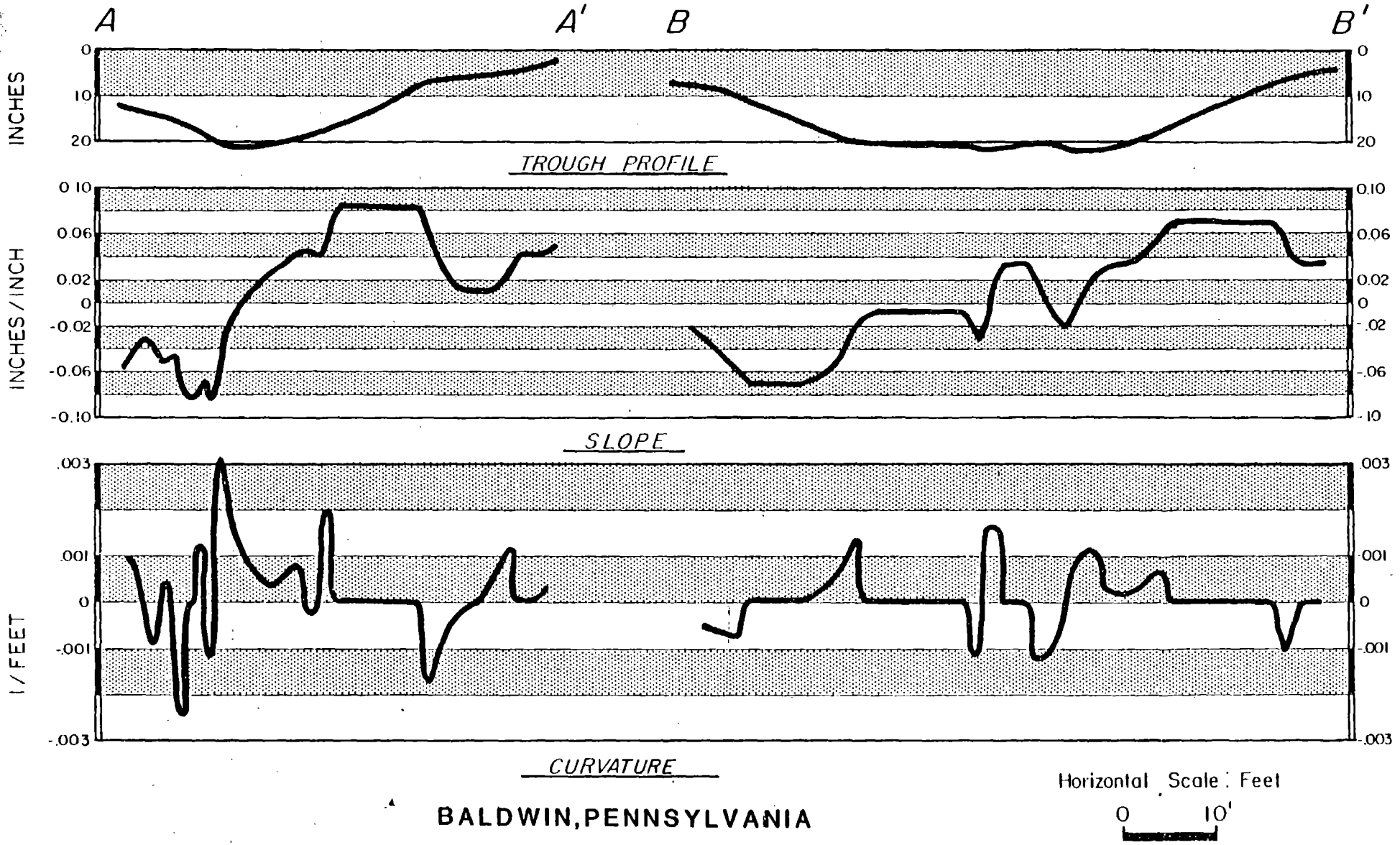
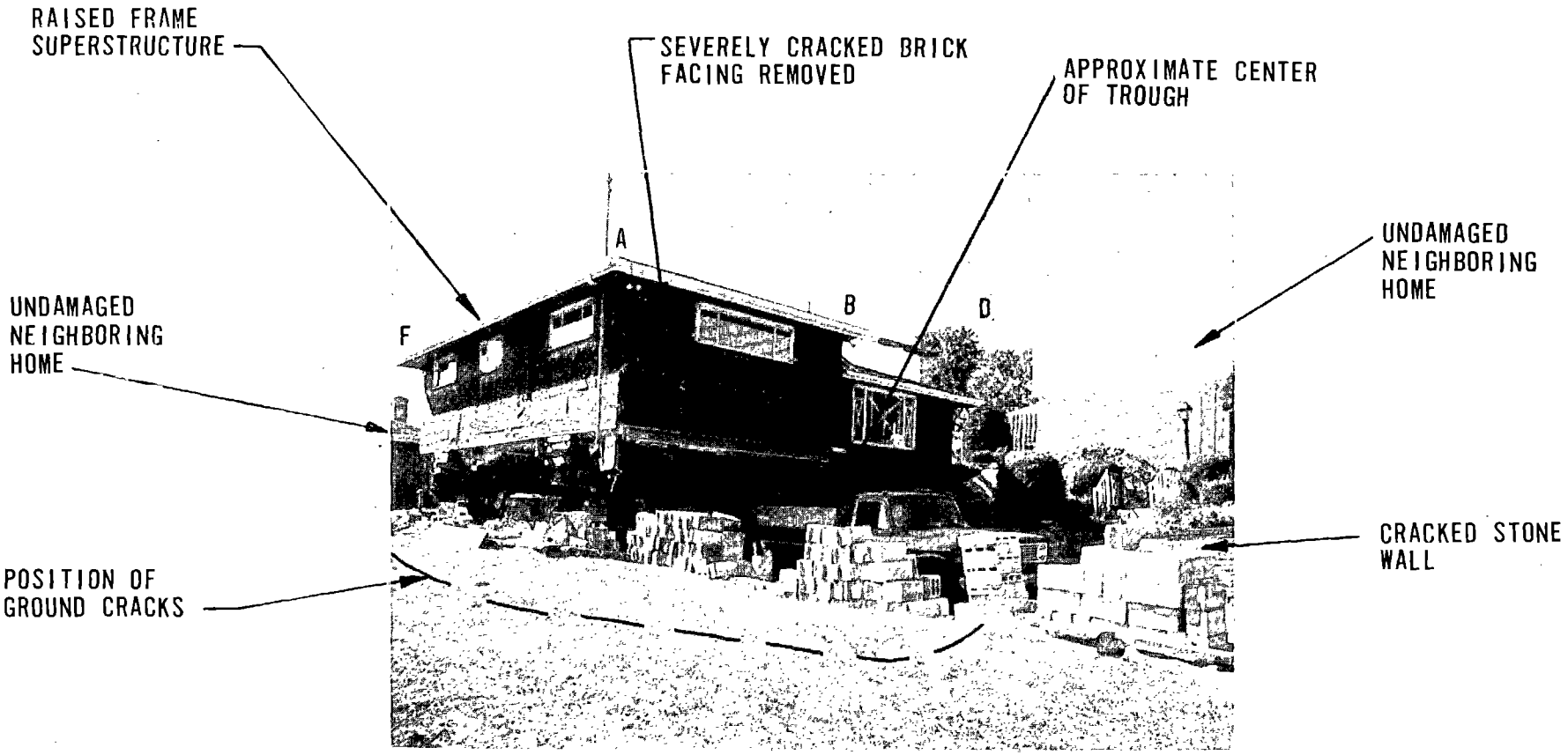
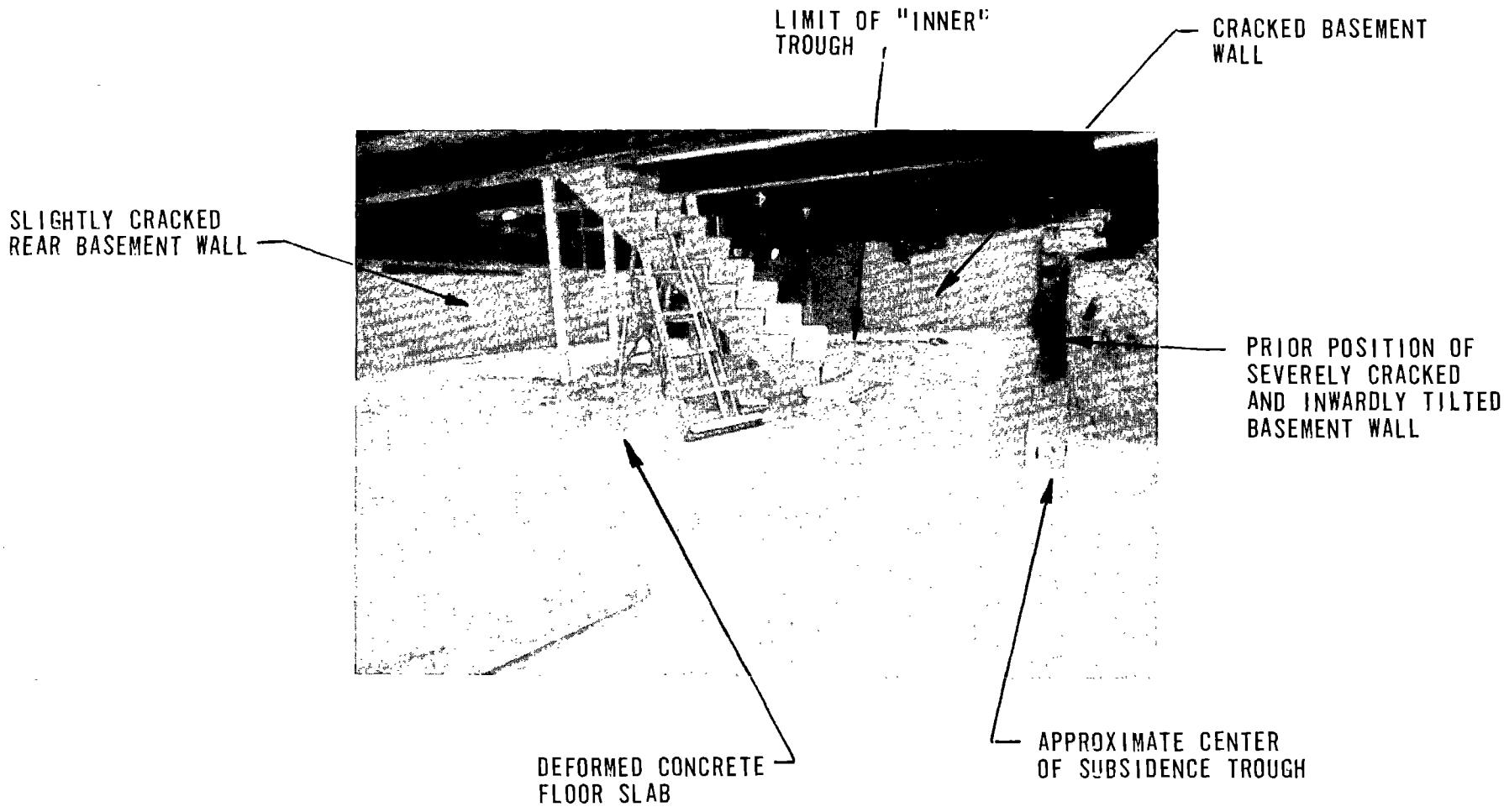


FIGURE A.23 – SUBSIDENCE TROUGH PROFILE, SLOPE AND CURVATURE ALONG LINES A-A' & B-B', CASE HISTORY 7



BALDWIN, PENNSYLVANIA

FIGURE A.24- RAISED AND LEVELED WOOD FRAME SUPERSTRUCTURE, CASE HISTORY 7



FIGUREA.25- SEVERELY DAMAGED BASEMENT, CASE HISTORY 7

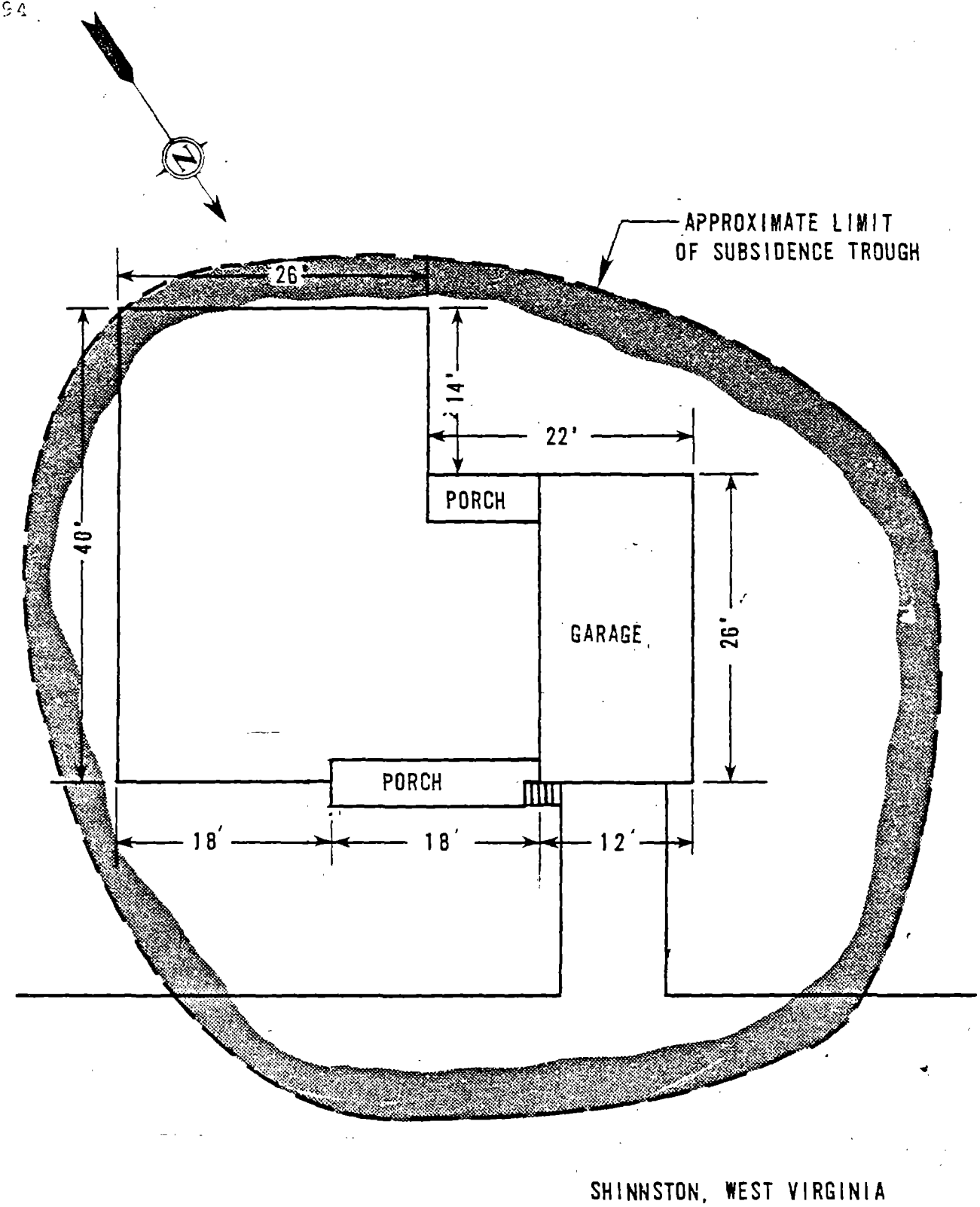
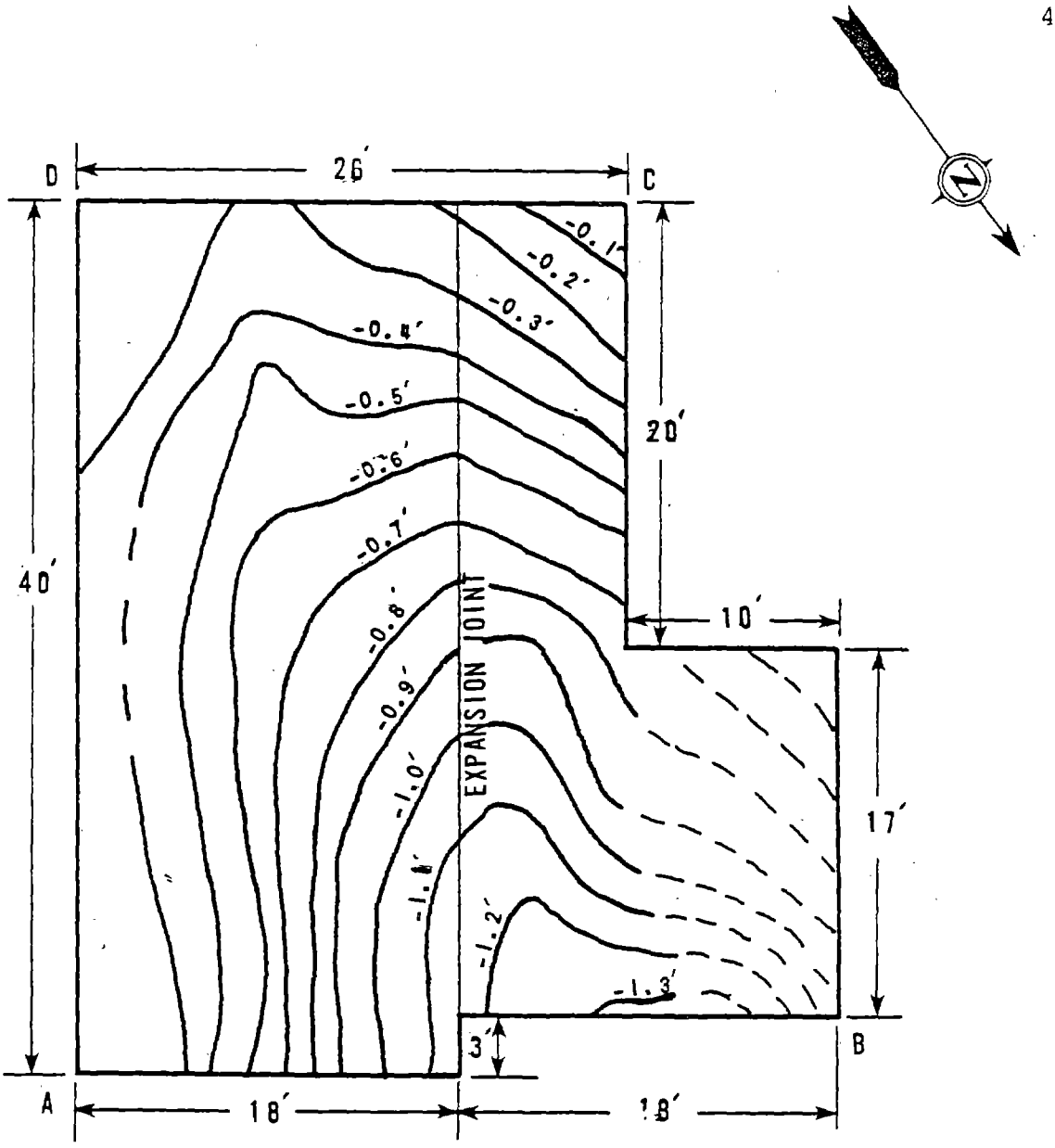
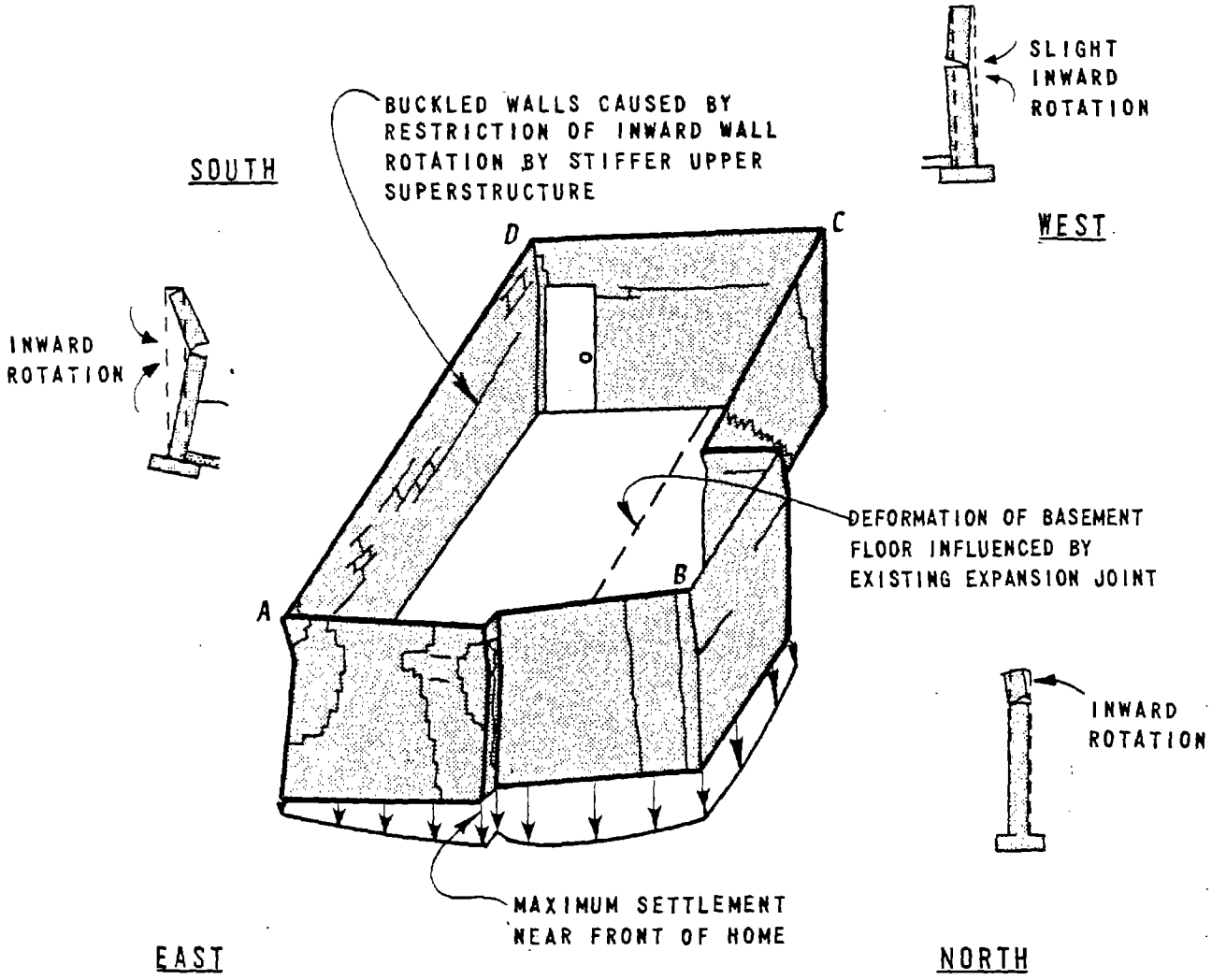


FIGURE A.26-SHAPE AND DIMENSIONS OF STRUCTURE AND SUBSIDENCE TROUGH, CASE HISTORY 7



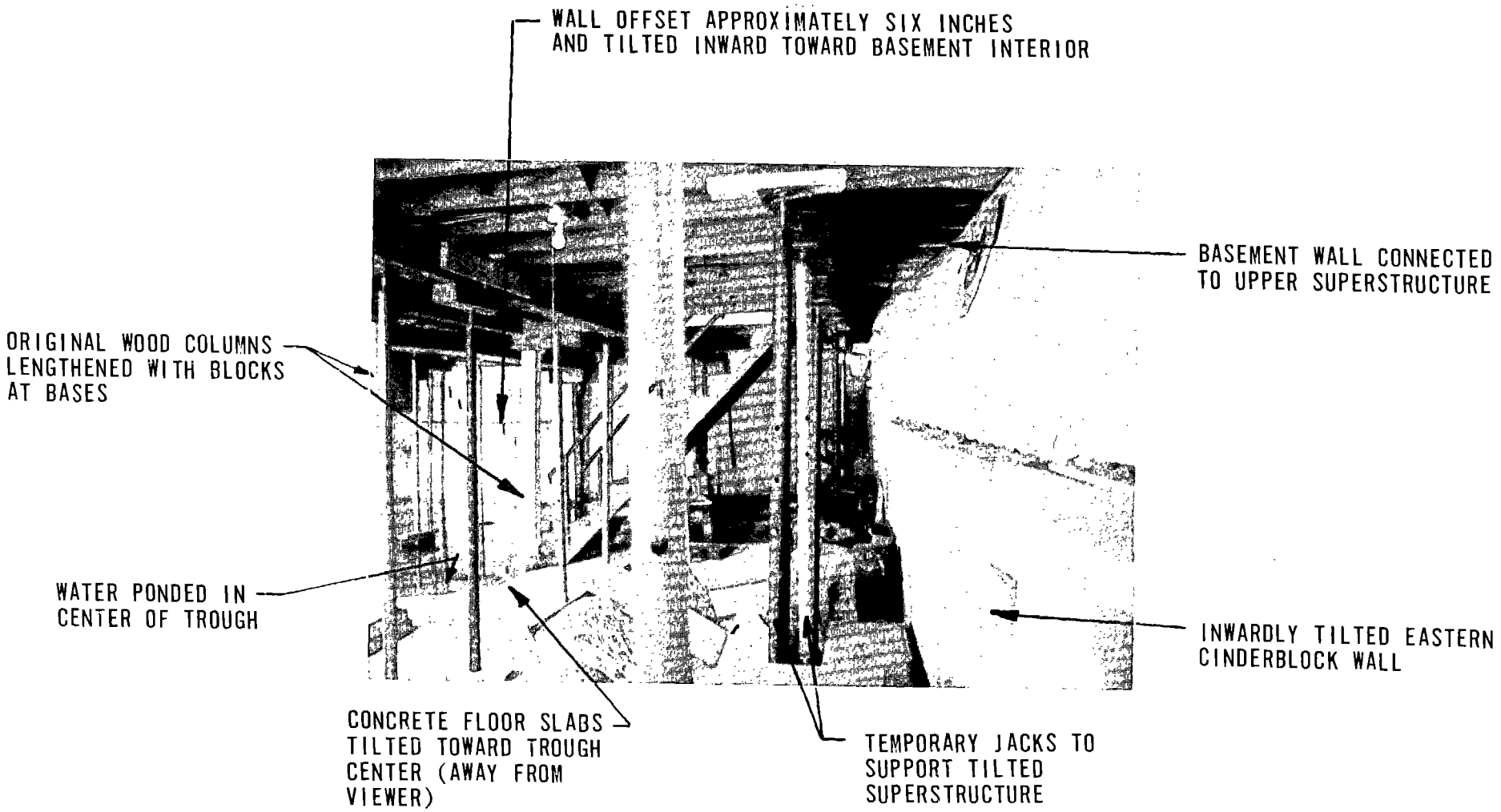
SHINNSTON, WEST VIRGINIA

FIGURE A.27—CONTOURS AND DIMENSIONS OF DEFORMED BASEMENT FLOOR, CASE HISTORY 7



SHINNSTON, WEST VIRGINIA

FIGURE A.28 – DEFORMATION OF BASEMENT WALLS AND FLOOR OF HOME, CASE HISTORY 7

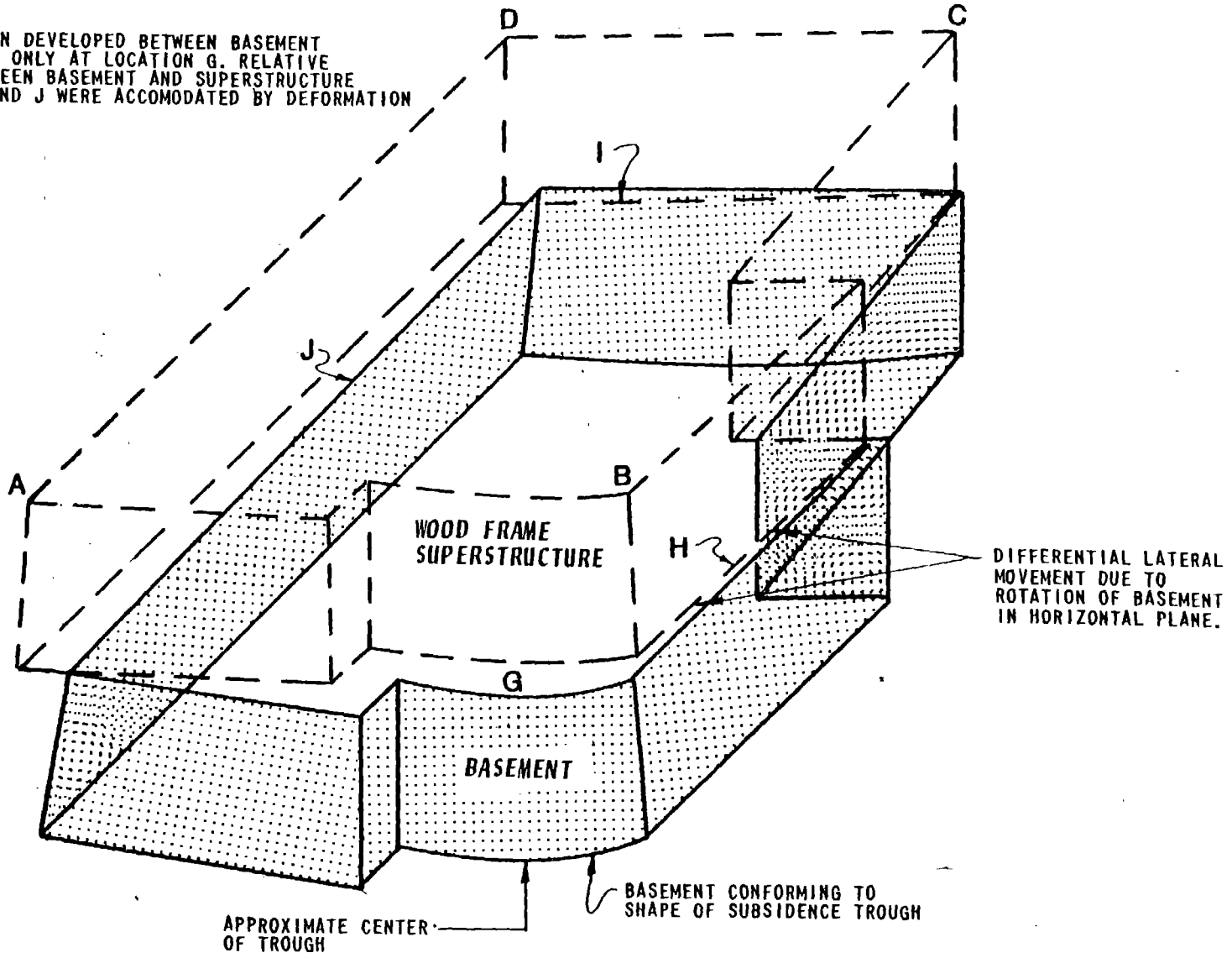


SHINNSTON, WEST VIRGINIA

VIEW LOOKING NORTH

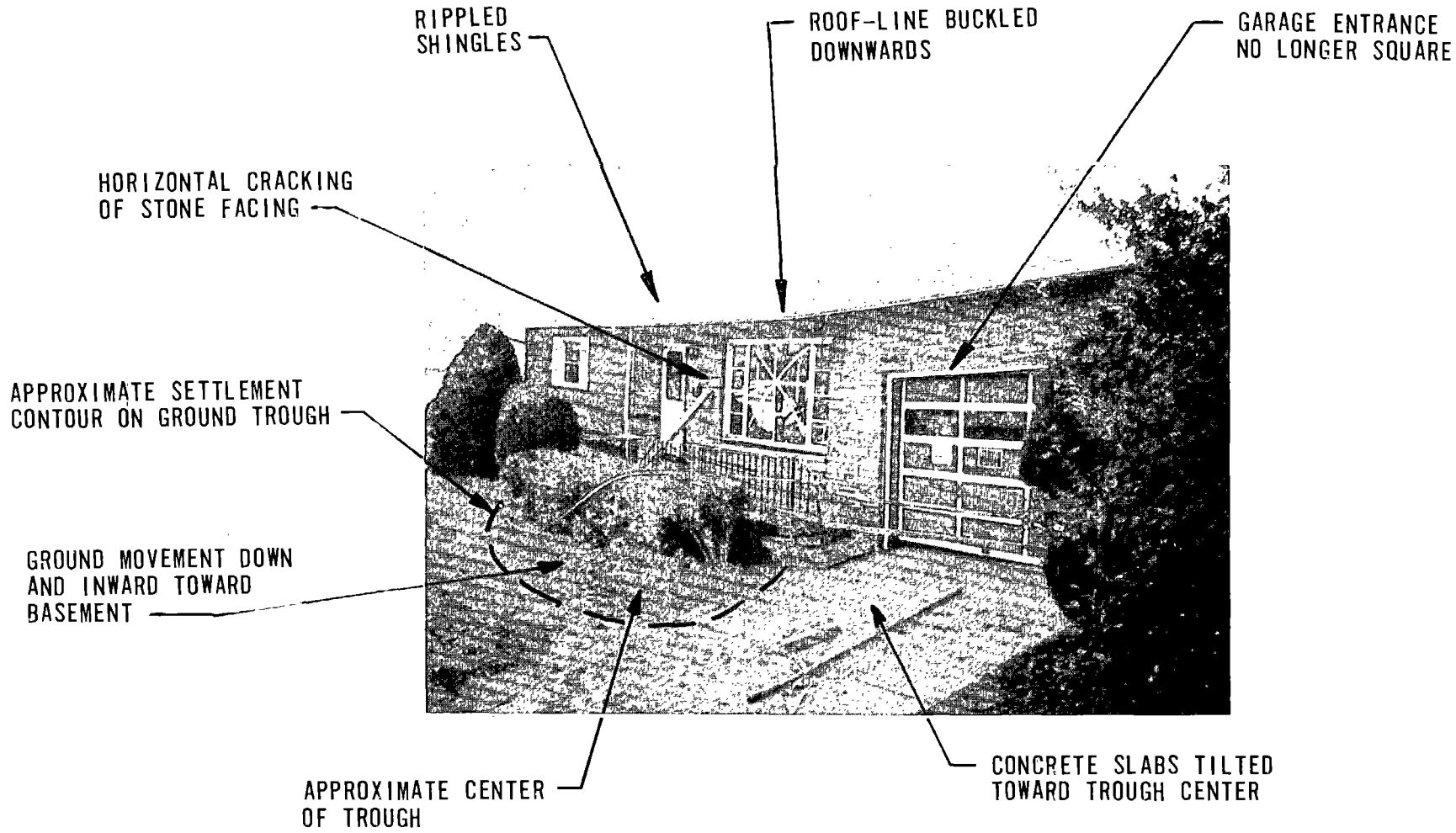
FIGURE A.29- PORTION OF DAMAGED BASEMENT, CASE HISTORY 7

NOTE: VERTICAL SEPARATION DEVELOPED BETWEEN BASEMENT AND SUPERSTRUCTURE ONLY AT LOCATION G. RELATIVE DISPLACEMENTS BETWEEN BASEMENT AND SUPERSTRUCTURE AT LOCATIONS H, I, AND J WERE ACCOMODATED BY DEFORMATION OF BASEMENT WALLS.



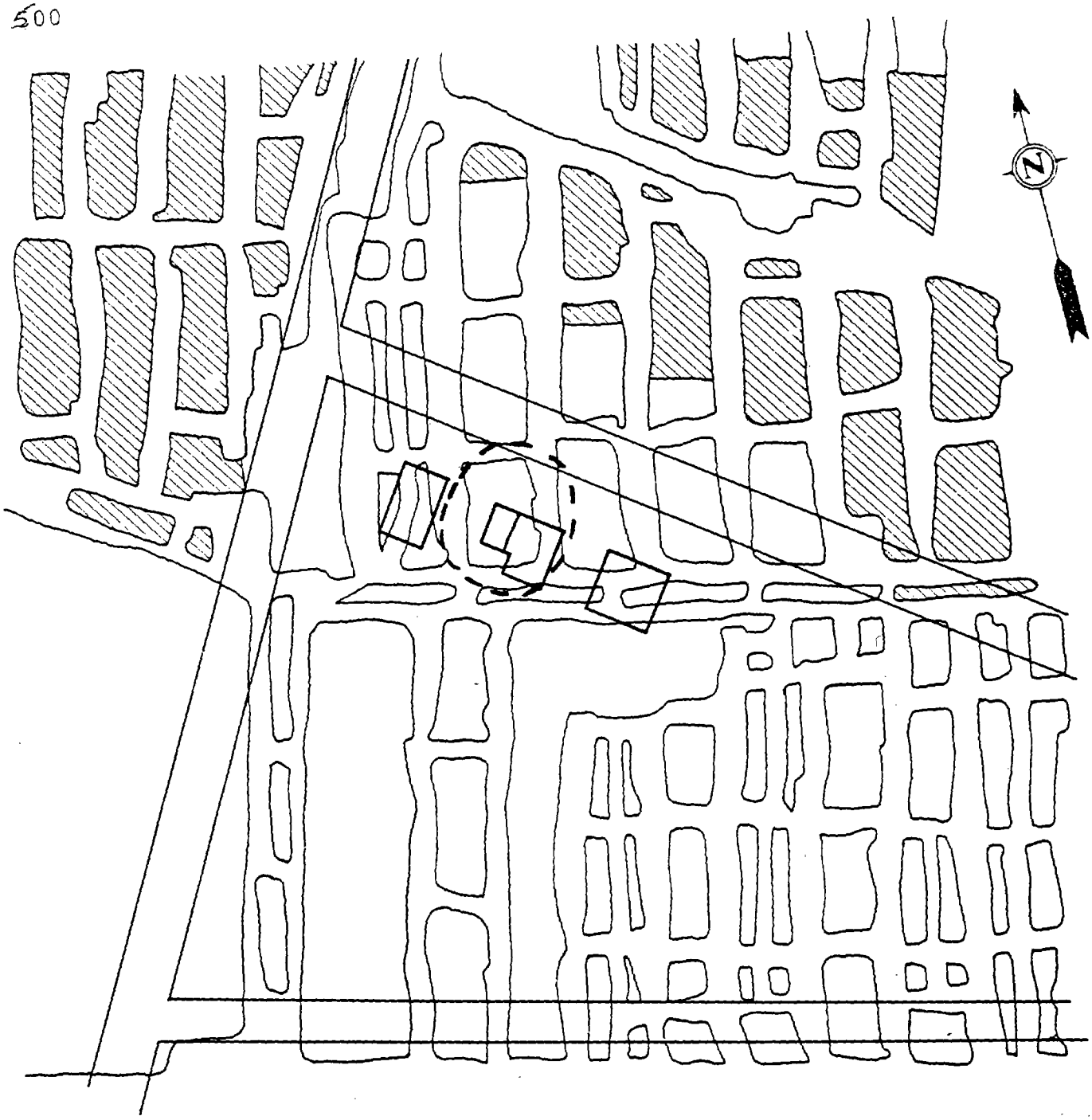
SHINNSTON, WEST VIRGINIA

FIGURE A.30- GENERALIZED REPRESENTATION OF DEFORMED SUPERSTRUCTURE RELATIVE TO BASEMENT DEFORMATION, CASE HISTORY 7



REFER TO FIGUREA 27 FOR SETTLEMENT CONTOURS ON BASEMENT FLOOR

FIGURE A.31—DAMAGE TO FRONT PORTION OF SUBSIDED HOME, CASE HISTORY 7



— HOME

— STREET

— LIMIT OF GROUND MOVEMENT

— COAL PILLAR (SHADED AREA - PORTION REMOVED DURING RETREAT MINING)

SCALE
0 50 100 200
(FEET)

SHINNSTON, WEST VIRGINIA

FIGURE A.32—EXTENT OF SUBSIDENCE AND PATTERN OF REMAINING COAL PILLARS, CASE HISTORY 7

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As the basement deformed, the frame superstructure sagged downwards, most noticeably near the front of the home. Due to its greater stiffness, the superstructure was not able to move vertically downward as much as the basement below and a gap formed between the two units (Figure A.30). As the basement moved downwards and slightly counterclockwise, a wood beam supporting the first floor warped in the direction of relative movement and a portion of aluminum siding sprung loose near a rear corner of the superstructure.

Although damage to the basement was severe, only minor amounts of plaster cracked within the superstructure. The hardwood floors sloped and door and window frames became distorted.

Damage to the exterior of the structure (Figure A.31) consisted of a distorted garage doorway, a buckled roof, distorted aluminum siding, and horizontal cracks in the front stone facing that developed as the superstructure began to bridge the more rapidly subsiding basement.

Interviews with personnel from the U.S. Office of Surface Mining (OSM) revealed that the abandoned mine below the site was located in the 8-foot thick Pittsburgh Coal. A mine map (Figure A.32) shows that coal pillars were completely extracted in 1913 west and south of the site and were split to the north. Complete coal pillars were left below the site. Reports by nearby residents indicate, however, that pillar robbing within this area of the mine occurred in the early 1950's. Borings by the U.S. OSM found the mine to be 117 feet below ground surface. Thirteen to 22 feet of the overburden is unconsolidated silts, sands, and some gravels. Rock overburden is reported to be approximately equal amounts of limestone and shale.

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Case History 8

Multiple Episodes of Subsidence above Abandoned Mine - All Episodes Occurring at a Single Site

Between 1971 and 1981, ground movements occurred sporadically at the site and ultimately caused significant damage to two homes in eastern Allegheny County, Pennsylvania. From 1971 to 1975, minor hairline cracks appeared in the front exterior brick facing of the east home, although the ground movements themselves were not perceptible. In 1975, severe damage occurred to the east home (Figure A.33), the sudden ground movements mistakenly being attributed by the residents to upward heave of the structure due to expansive minerals in the rock strata. In 1976, a second rapid episode of ground movement occurred and two subsidence troughs became distinct. This time the west home was severly damaged. In 1978, the Commonwealth of Pennsylvania backfilled the voids at mine level and in the overburden with fly ash. The east home experienced no further movement after the backfilling operations. Slight cracking of repaired areas in the west home were noticed until 1981, when all movement apparently ceased.

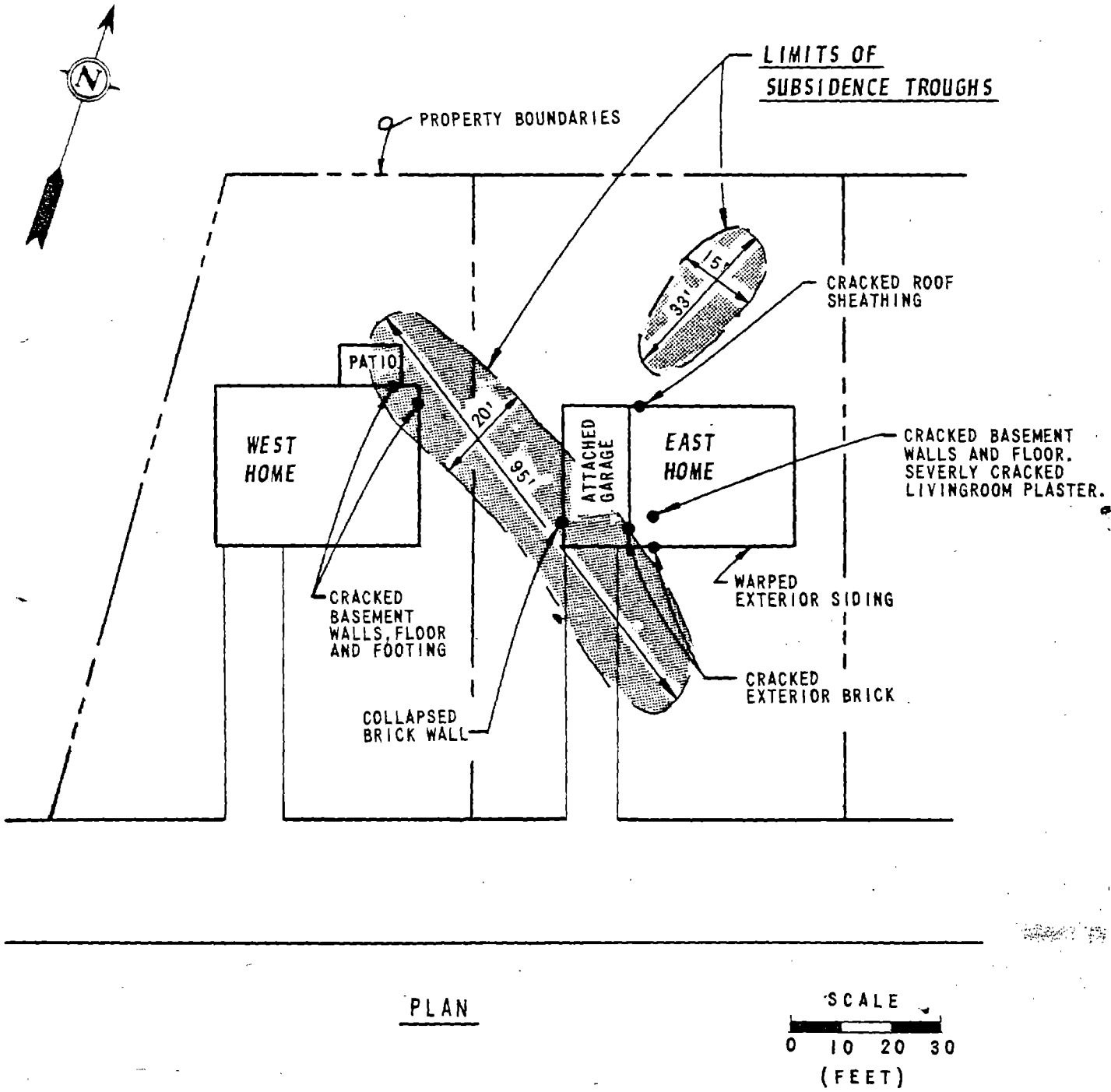
Forty feet below the site is the Oak Hill Mine, which was abandoned in 1924. It is located in the 6-foot thick Pittsburgh Coal. The 5 to 10.5 feet of soil overlying the rock strata is a brown sandy clay earth fill of medium stiff consistancy. Drilling by the Commonwealth of Pennsylvania revealed that the original mine opening had collapsed, and voids or zones of very loose rock rubble 0.5 to 8 inches high existed at depths of 13, 20, 23, 29 and 33 feet.

Of the two subsidence troughs that formed, only the larger actually caused damage to the two neighboring homes. The larger trough was elongate (Figure A.33) and measured approximately 95 feet in length, 20 feet in width, and at least 7 inches in depth. The smaller trough was oval in shape, with long axis oriented perpendicular to the larger trough, and measured 30 feet in length and 14 feet in width. Maps indicate troughs are oriented in the principal directions of rooms and entries within the mine.

Both of the damaged homes are frame structures with cinder block basements, and are largely brick-faced. The east home is splitlevel; the west home is single story. Damage to the homes is as follows:

East Home

Basement - The basement cinderblock walls and concrete basement floor were badly cracked near the front corner of the basement, rupturing a gas utility pipe and allowing groundwater seepage into the basement.



ALLEGHENY COUNTY, PENNSYLVANIA

FIGURE A.33—PLAN VIEW OF TWO HOMES DAMAGED BY PROGRESSIVE GROUND MOVEMENTS THAT OCCURRED OVER A PERIOD OF SIX YEARS, CASE HISTORY 8

Exterior - The outside wall of the attached brick garage first tilted inward, then cracked, and finally collapsed. A portion of the brick facing on the front of the house also collapsed. The second floor wood-siding warped; the brick-facing on the side wall cracked; and the wood roof in the rear of the house distorted and allowed water to leak into the attic.

Interior - All plaster walls and ceilings were reportedly cracked, although the front living room cathedral ceiling/ walls were most extensively damaged. All door and window frames were distorted.

Total repair cost to the east home was $8,000 (1975) and included reconstruction of the collapsed portion of the garage and replacement of the basement floor and exterior brick in the front corner of the home. All interior and exterior wall cracks were repaired and doors and windows were realigned.

West Home

Basement - The two cinder block walls, the concrete basement floor, and the foundation footing within the sub-sided area were cracked as a result of of differential vertical movement of at least 1.5 inches.

Exterior - The brick facing at the rear corner of the superstructure was extensively cracked, although only minor hairline cracks appeared in the brick facing elsewhere around the home.

Interior - Aside from one short, hairline crack in a front living room wall, no damage to the interior was reported. No door or window frames were distorted. Replacement of the basement walls, the floor and footing and exterior brick cost approximately $8,700 (1976).

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Case History 9

Multiple Episodes of Subsidence above Abandoned Mine Causing Significant Damage to Several School Buildings

In early February, 1976, mine subsidence in Washington County, Pennsylvania, resulted in severe structural damage to the one-story gymnasium, industrial arts and home economics classrooms of Bellmar Junior High School and to a portion of the newest addition to the nearby Marion Elementary School (Figures A.34, A,35). The affected portions of both schools were condemned as unsafe and withdrawn from use. After mine-level openings were backfilled with fly ash and the overburden pressure grouted, the elementary school was repaired and the affected areas of the junior high school were reconstructed.

The original Marion Elementary School was built in 1939. Repointing of brick work and reinforcement of some walls in the 1940's indicates that mine subsidence probably occurred at that time. Additions to the elementary school were built in 1957 and 1968. The Bellmar Junior High School was constructed in 1962. In 1968, several residential structures located within 0.25 miles northeast of the schools experienced approximately three feet of vertical ground movement and were severely damaged.

Figure A.36 illustrates the distribution and shape of mine pillars below the two schools as indicated by old maps of the Fremont Mine. First mining, which produced a series of long, narrow coal pillars, was carried out until 1903 when the mine was closed. Probably 65 to 75 percent of the coal was extracted. From the early 1940's to 1966, sporadic mining reduced the size of many of the remaining coal pillars. Since 1968, the mine has been flooded.

In 1976, after subsidence had severly damaged the two school buildings, subsurface investigations were conducted by the Commonwealth of Pennsylvania and several consulting firms. Numerous borings were drilled at the site.

Figure A.37A is a cross-section of the overburden along line A-A' of Figure A.36. There is almost no topographic relief at the site and the overburden is virtually a constant 125 feet thick. Below the school foundations, the upper 20 to 30 feet of overburden consists of unconsolidated, river terrace sands, silts, clays and gravels. Figure A.37B is a cross-section of the soil profile along line B-B' on Figure A.36. It illustrates the complex interrelationship of the unconsolidated terrace sediments described in terms of the Unified Soil Classification System. Contours of the in situ soil density as determined by the Standard Penetration Test are also shown in the figure. Shales, siltstones, sandstones and a prominent coal unit make up the remainder of the overburden.

In almost all boreholes, voids up to one foot in height were encountered in the rock overburden. In general, only about four

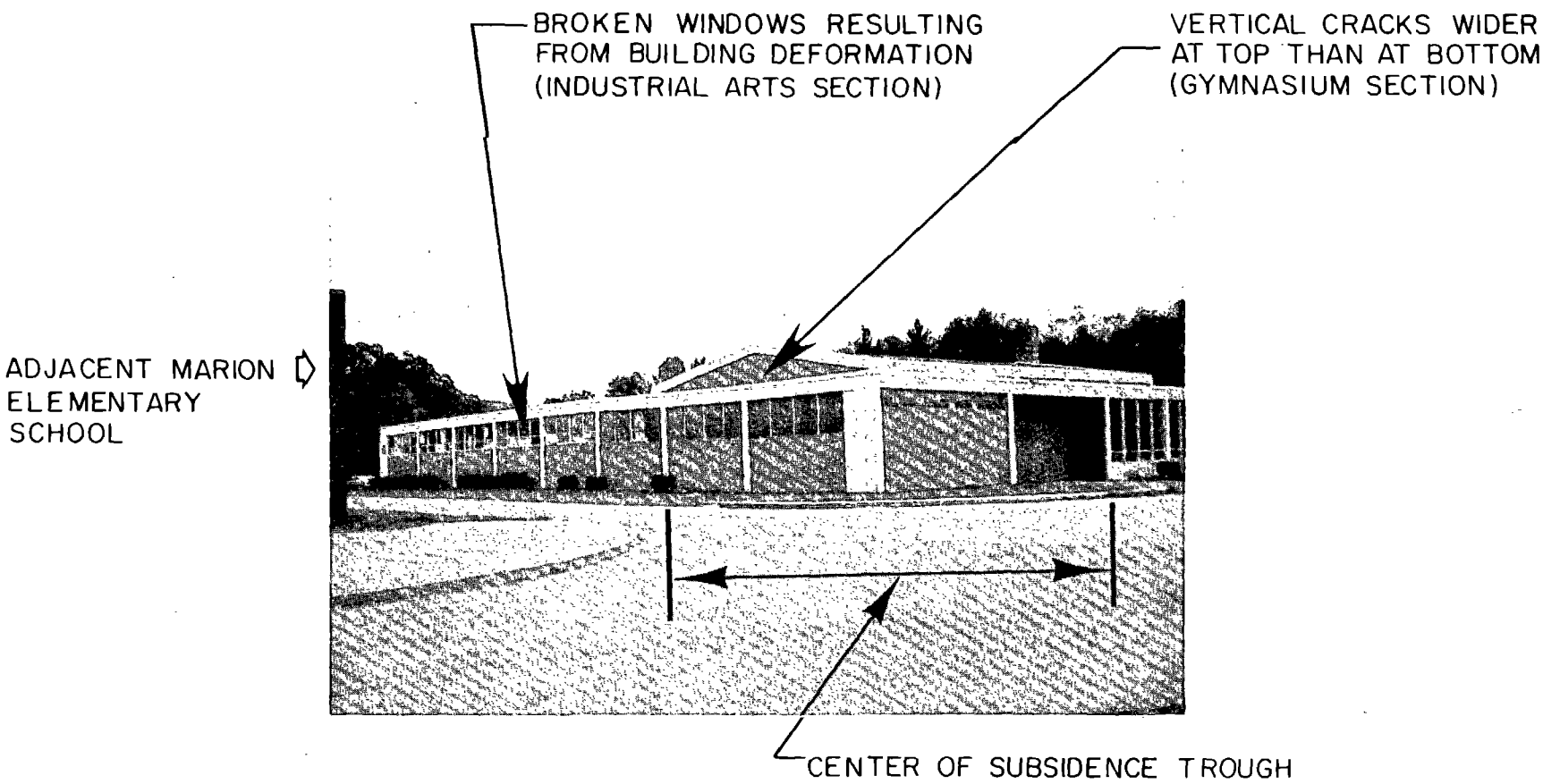


FIGURE A.34 - EXTERIOR OF DAMAGED PORTION OF BELLMAR JUNIOR HIGH SCHOOL CASE HISTORY 9

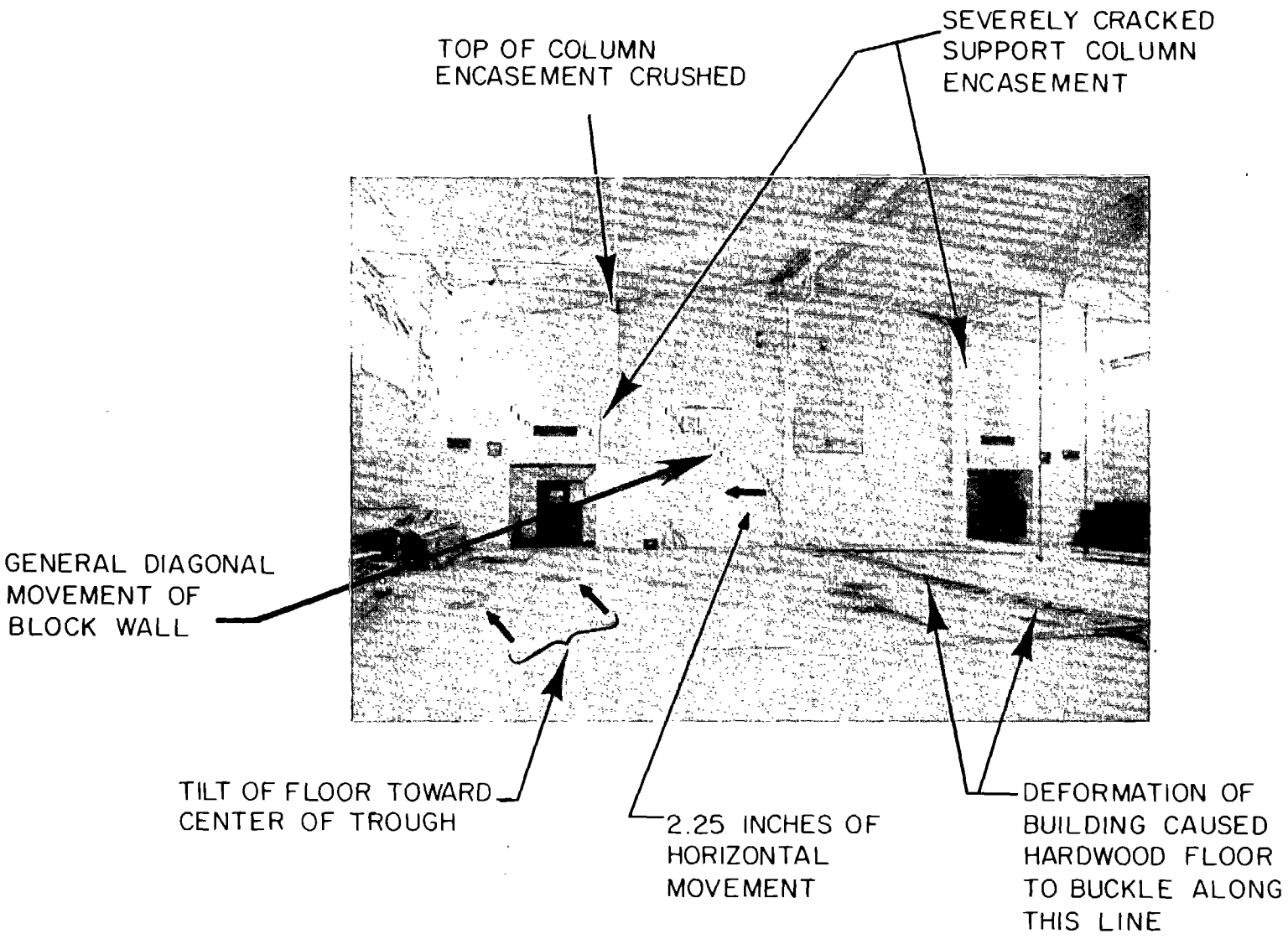
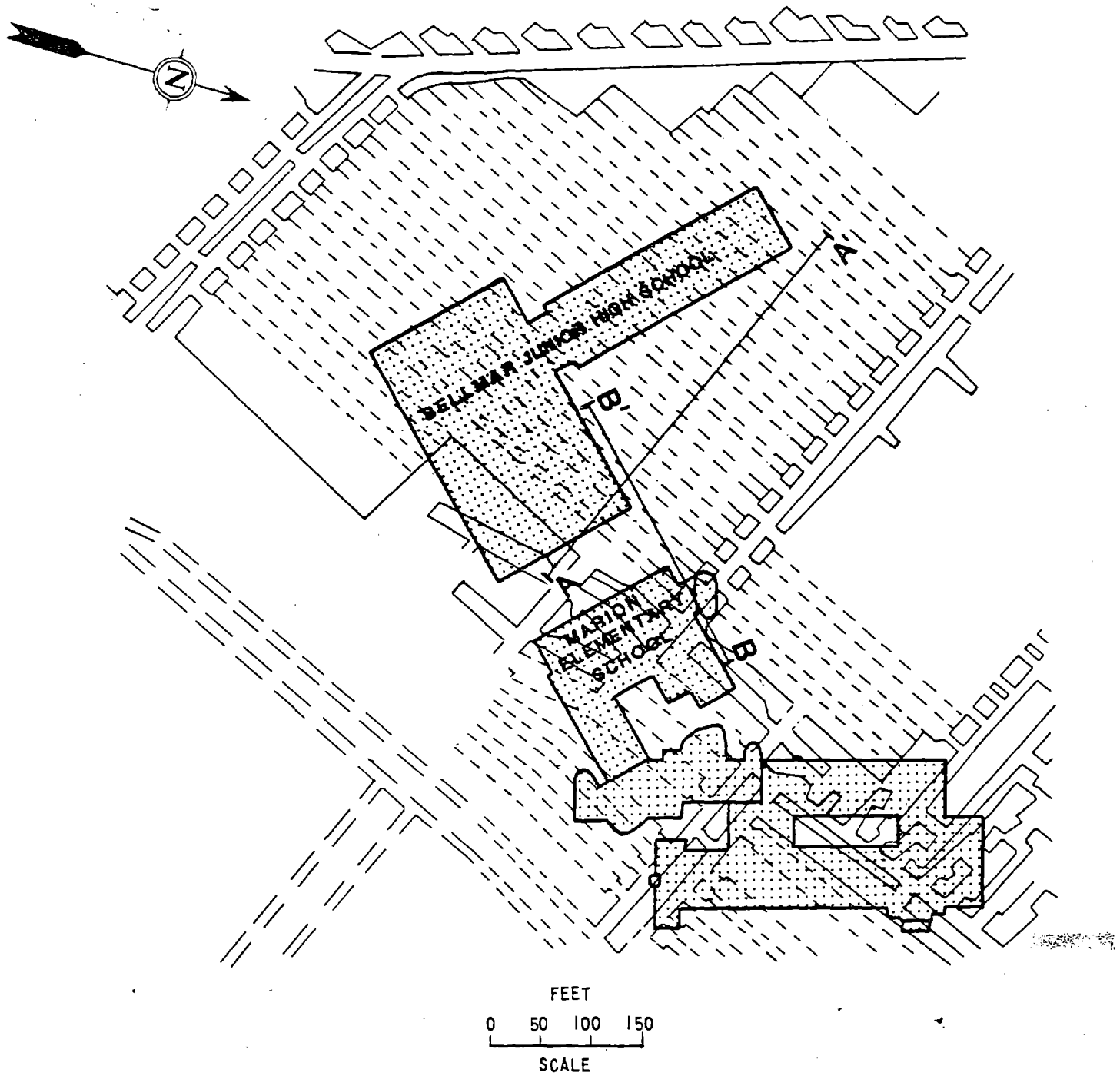


FIGURE A.35 – NORTHWESTERN END OF DAMAGED GYMNASIUM, BELLMAR JUNIOR HIGH SCHOOL



A —— A' LINE OF OVERBURDEN CROSS-SECTION (FIGURE A.37A)

B —— B' LINE OF SOIL CROSS-SECTION (FIGURE A.37B)

FIGURE A.36-POSITION OF BELLMAR AND MARION SCHOOLS RELATIVE TO REPORTED MINE-LEVEL COAL PILLAR CONFIGURATION, CASE HISTORY 9
(MODIFIED FROM GAI CONSULTANTS,INC., 1976)

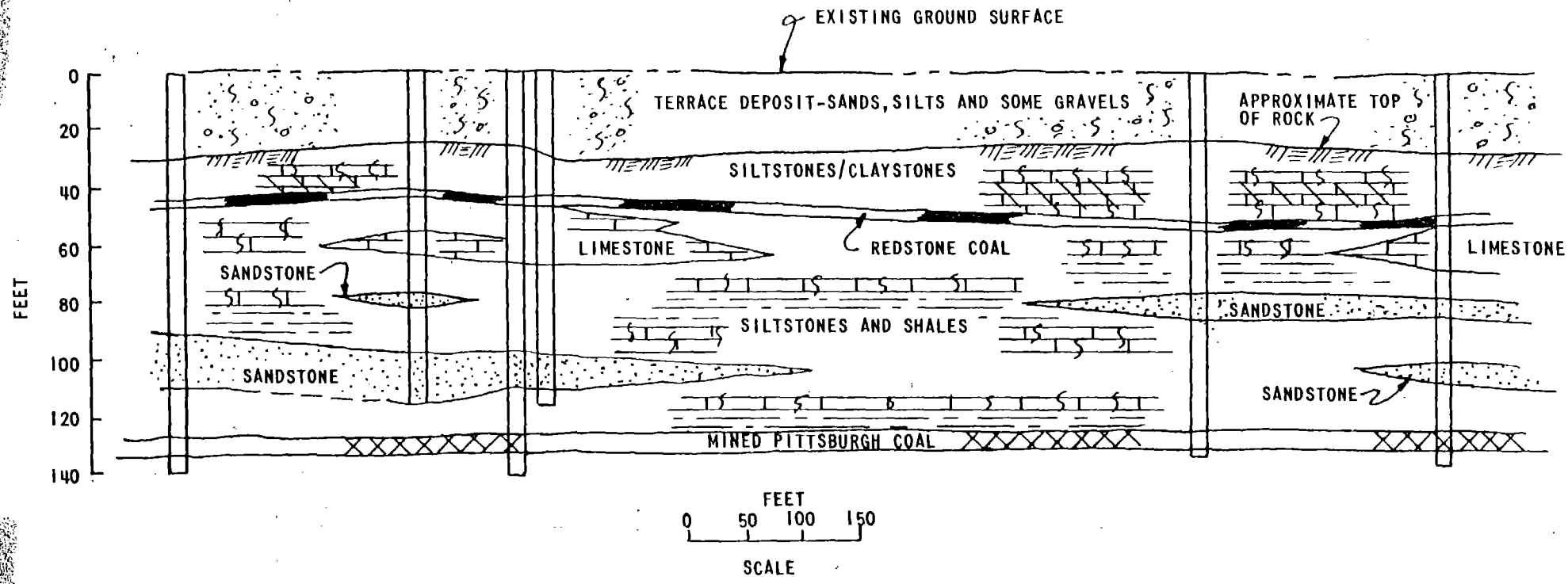
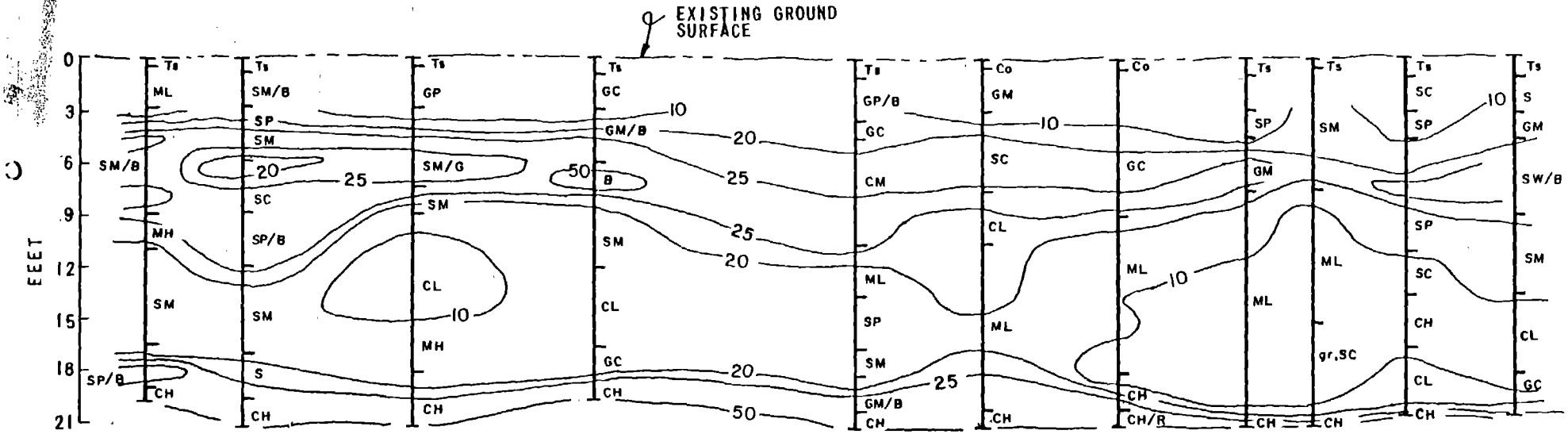


FIGURE A.37A - CROSS-SECTION A-A' OF SOIL AND ROCK OVERBURDEN, CASE HISTORY 9 (MODIFIED FROM BRANTHOOVER & RICHARDS, 1981)

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LEGEND

—20— STANDARD PENETRATION NUMBER (BLOWS/FT.)

ML UNIFIED SOIL CLASSIFICATION SYMBOL

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FIGURE A.37B- SOIL CROSS-SECTION B-B' INDICATING DISTRIBUTION OF SOIL TYPES AND CONTOURS OF INSITU SOIL DENSITY VALUES AS DETERMINED BY THE STANDARD PENETRATION TEST, CASE HISTORY 9
(MODIFIED FROM BRANTHOOVER & RICHARDS,1981)

feet of the original eight-foot mine opening was found to be open. Crushed coal pillars at mine-level were also detected.

Survey elevation measurements of the ground surface in and around the two schools not only determined the size and shape of the 1976 subsidence trough, but also detected two additional smaller troughs that apparently formed sometime during the fourteen years prior to the present trough. Downward movement of 0.3 feet was found to have occurred below the south corner of the junior high school, and 0.25 feet of downward movement was found to have occurred below the northeastern corner of the same building. Both smaller troughs caused a limited amount of cosmetic damage to the building.

Development of the large trough below the junior high school (shown in Figure A.38) began sometime between noon Saturday, February 7, 1976 and early Monday morning, February 9th. Inspection of the building by school officials revealed severe cracking of brick and masonry block walls and concrete floors in the industrial arts and gymnasium sections. For fear of partial building collapse, classes were cancelled and the entire building was closed for several days. Damage to the adjacent Marion elementary school did not occur until the next day, February 10, 1976, when the subsidence trough broadened, and slight cracks developed in the concrete floor of the elementary school. Meanwhile, some cracks in the junior high school increased to 2-inches in width and estimated downward vertical movement was 4 to 6 inches. The shape of the trough as of June, 1978, is shown in Figure A.38. At that time, maximum downward movement was 1.3 feet, and the oval-shaped trough varied from about 250 to 350 feet in diameter.

The interior concrete floor slab of the school conformed almost completely to the shape of the large subsidence trough. Differential vertical movement between it and the ground surface were not noted. Although major ground movements were not observed visually after the first two or three days of the incident, subtle, but potentially harmful, movements continued. In order to prevent serious damage to the elementary school, adjustable props were placed under all structural elements in the affected portion of the building. Damage was already so severe in the junior high school that no effort was made to prevent a worsening of that condition.

In order to reduce the risk of future subsidence movements, the Commonwealth of Pennsylvania, from 1978 to early 1979, pumped fly ash into mine-level voids and injected cement grout into the soil and rock overburden. Post-stabilization monitoring of long term strata movements from mine-level to ground surface with multi-position borehole extensometers indicated that overburden movement had apparently ceased as of August, 1979. Survey measurements by the writers in November, 1980, of the damaged interior of the junior high school (Figure A.39) revealed that the dimensions of the subsidence trough remained essentially the same as those measured in June, 1978 (Figure A.38). The slight differences between the two surveys are probably due to a difference in the

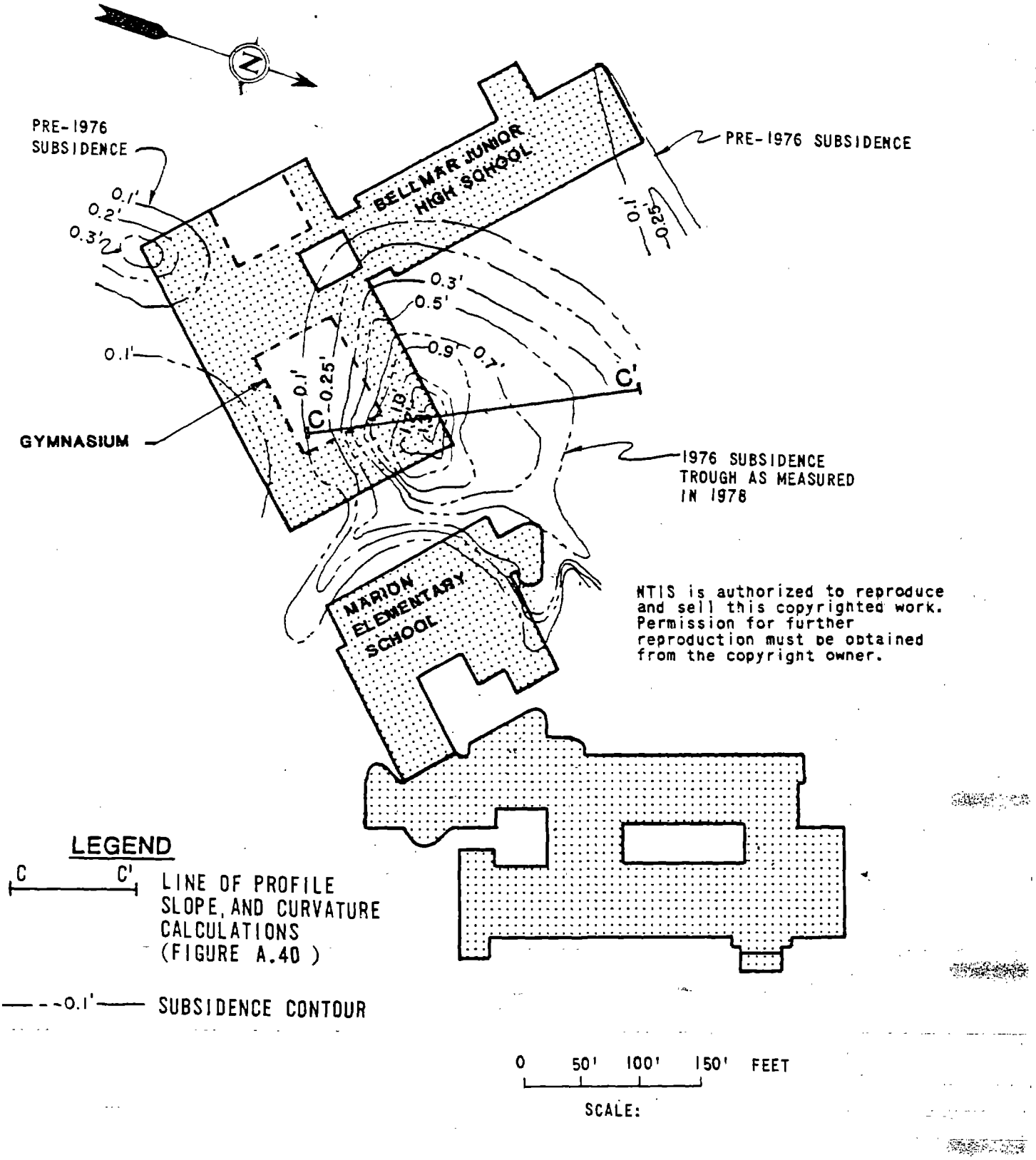
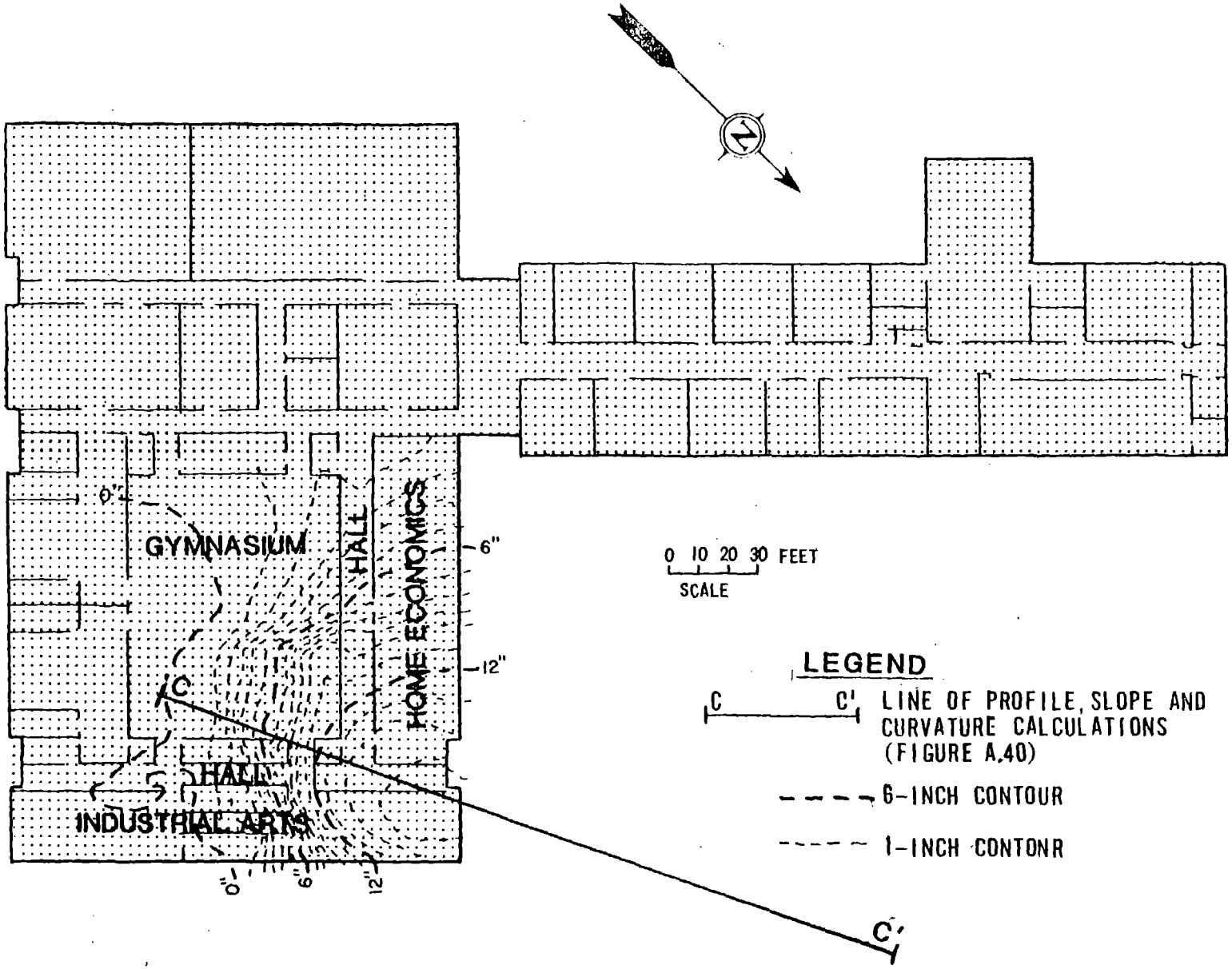


FIGURE A.38- SUBSIDENCE CONTOURS AND THE RELATIVE POSITIONS OF THE BELLMAR JUNIOR HIGH AND MARION ELEMENTARY SCHOOL, CASE HISTORY 9 (MODIFIED FROM BRANTHOOVER & RICHARDS, 1981)



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FIGURE A.39- SUBSIDENCE CONTOURS WITHIN THE JUNIOR HIGH SCHOOL AS MEASURED IN NOVEMBER, 1981, CASE HISTORY 9

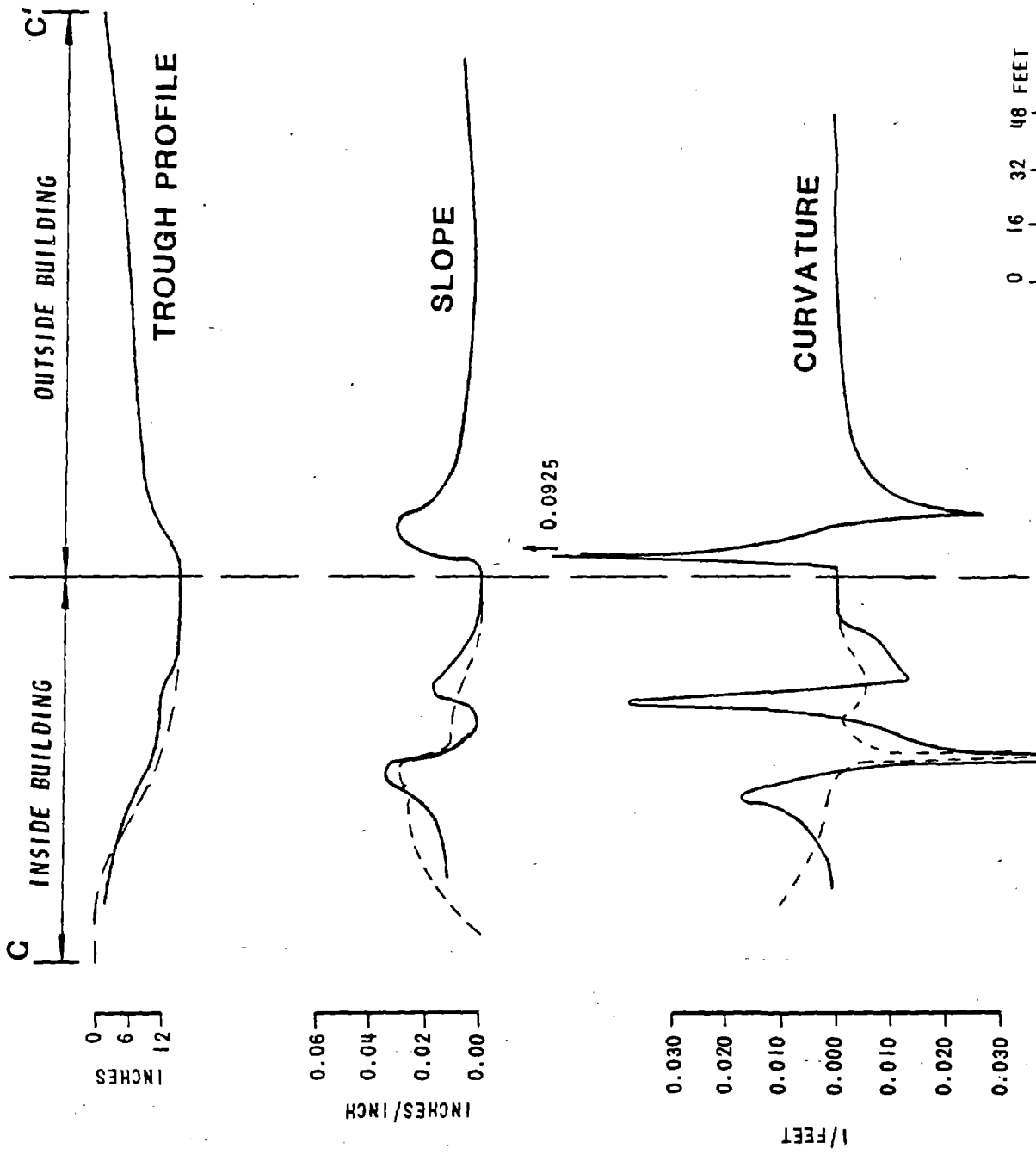


FIGURE A.40 SUBSIDENCE TROUGH PROFILE, SLOPE, AND CURVATURE ALONG LINE C - C' (FIGURES A.38 AND A.39), CASE HISTORY 9

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spacing of the survey grids. Comparisons of the profile, slope, and curvature of the 1978 and 1980 measured troughs are shown in Figure A.40. In mid-1982, repair and reconstruction of the two schools began.

Damage to the elementary school was not nearly as great as to the junior high school. Although cracking of the elementary school exterior brick walls was minimal, a 4-inch wide crack opened in one wall, and the wall closest to the junior high school rotated outward slightly, necessitating the installation of adjustable roof props to ensure continuous support of roof joists. Some cracking and vertical offset of interior concrete floor slabs also occurred.

Settlement of the northeast corner of the junior high school into the subsidence trough is visible in Figure A.35A. As the trough developed below the corner of the building, the steel building frame pulled horizontally toward and vertically downward into the center of the trough. Since movement was not uniform throughout the building, rooms tended to change shape three dimensionally. In general, the change in shape was more intense closest to the center of the trough.

Although damage to the interior of the Bellmar school was extensive, cracking of the exterior brick facing was minimal, perhaps due to its segmented design. Even so, many of the windows located above the brick panels broke. At the peak of the gymnasium roof, a vertical crack formed in the brickwork, which was wider at the top than at the bottom. The shape and location of this crack possibly indicated a slight change in shape of the gym roof due to differential movement of the side walls.

Severe cracking of brick and other masonry occurred throughout the interior of the deformed portion of the building. Figure A.35B shows the damage to the northeast end of the gymnasium. Block surrounding steel columns supporting the longitudinal steel roof joists were severely cracked and somewhat crushed as they were forced to rotate slightly and move at their base toward the center of the subsidence trough. Crushing of the block surrounding one column is visible in the figure. Diagonal movement of the block wall between the columns is also visible. Measurement of one crack alone indicated 2.25 inches of horizontal movement. The steel frame itself was also deformed. At least one of the large steel columns in the gym rotated six degrees outward. This movement may have been responsible for the exterior crack at the peak of the roof referred to earlier and for some of the cracking of the block columns noted above. Large scale warping and buckling of the hardwood gym floor occurred as the corner of the gym moved downward into the subsidence trough.

In hallways surrounding the gym, stair-stepped cracks up to three inches wide were numerous, particularly where the slope of the trough profile was greatest. Cracking of walls in the classrooms was most severe in the industrial arts section.

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During excavation of the Bellmar school foundation as part of reconstruction activities, at least four 1 to 1.5 inch wide cracks were found to extend completely through the strip footing. Cracks in the soil were locally filled by vibrating backfill material into them. The building's steel frame was realigned by jacking individual columns to their original elevations.

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Case History 10

Multiple Episodes of Subsidence - Each Incident Occurring at Different, but Nearby Sites

Between July 1, 1971, and November, 1980, four incidents of subsidence took place within a small section of a Pittsburgh, Pennsylvania suburb (Figure A.41). Three of them resulted in damage to several one and two-story homes, while the fourth caused damage to a 24-unit apartment complex. All of the buildings were located above the Montour Number 8 Mine, which was abandoned in the 1940's. Seventy-seven to 140 feet of rock and soil overburden separates the surface structures from the mine.

First Incident

Subsidence began slowly during the first week of July, 1971, when the residents of the home in this group of three that eventually became the most heavily damaged (Figure A.41) noted that the stairs leading from the first floor to the basement garage were beginning to warp. Two weeks after this initial observation, rapid ground movements began, and a total of three homes on adjoining properties became affected by the incident. The middle home, situated in the center of the developing subsidence trough, was severely damaged while the homes on either side, which extended only partially into the subsidence trough, suffered damage only to the sides that faced the middle home. Rapid ground movements continued for one to two weeks. By the end of this period, the rear corners of the middle home had subsided 15 to 18 inches relative to the front. Subsidence of portions of the neighboring homes totalled about 4 inches. Slight ground adjustments were noticed throughout the trough for several months. From September to November of 1971, the Commonwealth of Pennsylvania drilled exploratory holes and pumped fly ash into all voids encountered between ground surface and mine level.

Damage to the basement of the middle home was severe and consisted of intensely cracked and inwardly bowed cinderblock walls, broken and warped wood paneling, a cracked concrete floor, ruptured water and sewage lines, and badly distorted doors and windows. On the home's exterior, all brick facing, the chimney, all porches and a large concrete swimming pool were badly cracked. The swimming pool was eventually filled with earth and is currently used as a vegetable garden. Damage to the living quarters was relatively minor and was confined to cracked plaster walls and ceilings in the front living and dining rooms.

To repair the middle home, steel beams were placed under the superstructure, which was then raised and leveled. The cost was $5,000. Reconstruction of the entire basement and footings, replacement of the exterior brick and chimney and repair of the interior cracked plaster cost $38,000 (1971).

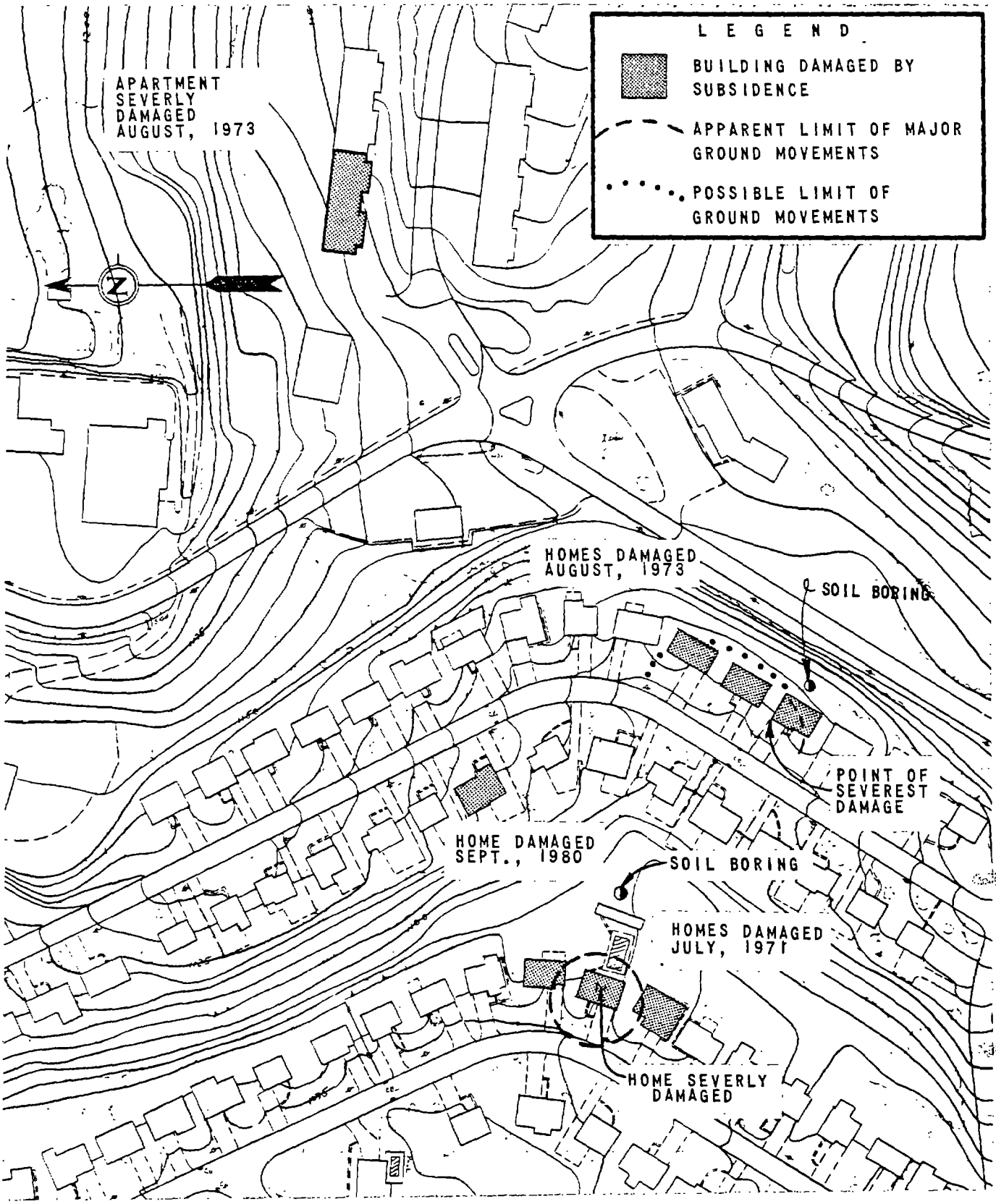


FIGURE A.41- LOCATION AND DATES OF SUBSIDENCE INCIDENTS, CASE HISTORY 10

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In the east home, an unusually placed bearing wall separated the two halves of the two-car basement garage. This wall, in combination with jacks positioned below all subsiding portions of the superstructure, tended to prevent translation of the main portion of the building toward the center of the subsidence trough during the ground movements and therefore only the part of the basement garage and exterior brick facing on the side of the bearing wall nearest the middle home suffered severe damage. Damage to the remainder of the structure was minor.

Damage to the west home took the form of cracked basement walls and exterior brick facing on the front corner nearest the middle home.

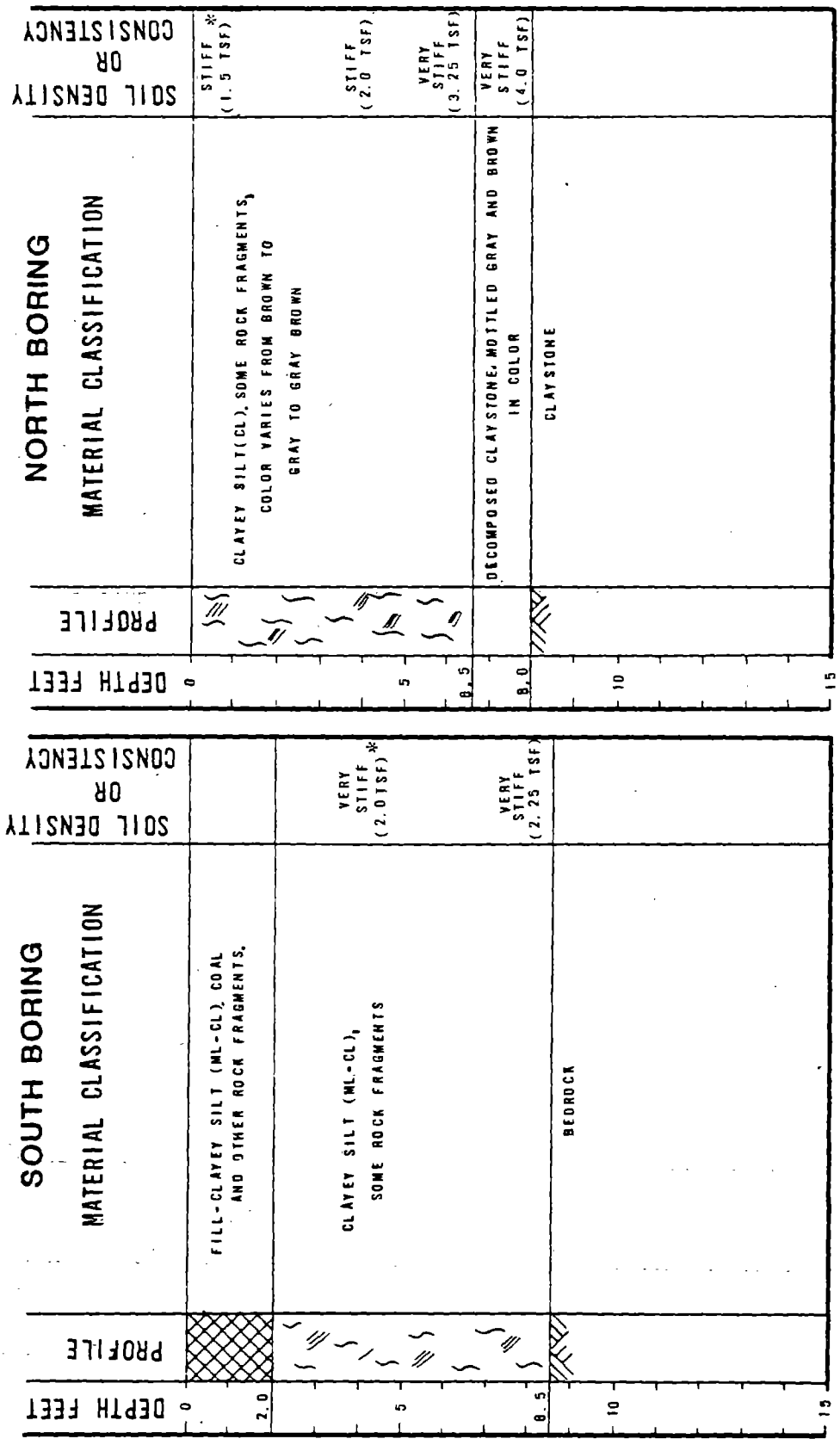
Second Incident

The second subsidence incident also ultimately involved three homes and was located only 700 feet north of the first incident. Ground movement began in February, 1973, when the front corner of the basement garage of the west-most of the three homes was damaged. Approximately $2,200 was spent by the homeowner to replace the severly cracked cinderblock wall and brick facing and to install an 8-inch I-beam directly over the footing across the damaged portion of the garage. Neither of the two neighboring homes to the east were damaged at this time.

In August of the same year (1973), ground movements suddenly and dramatically resumed. While the exact boundaries of the subsidence trough can only be inferred from the reports of damage, it is evident that downward movements were greatest in the west end of the trough and caused the most serious damage to the previously damaged home located there. The other two homes within the trough sustained relatively minor crack damage to their basement floors and exterior brick facing. The rear cinderblock basement wall of the center home was bowed inward, causing nearby doors and windows to stick. As with the February subsidence episode at this site, the most serious damage to the west-most home occurred at the front. Repair costs of the home this time amounted to approximately $19,000 (1973) and included placement of another steel I-beam below the front of the distorted superstructure to provide leveling and support; demolition and replacement of the entire front basement wall, footing, and exterior brick facing (which moved as a unit from 2 to 6 inches outward from the building); replacement and/or repointing of all cracked brick facing on all sides of the home; and patching and repainting of the cracked or warped interior woodwork and plaster walls and ceilings.

The Commonwealth of Pennsylvania backfilled the mine with fly ash in the fall of 1973. Slight ground movements continued until February, 1974.

A shallow boring made at the site by the writers indicates that bedrock is located approximately 8.5 feet below the ground



*Pocket penetrometer readings (tons/square foot)

FIGURE A,42—SOIL BORINGS, CASE HISTORY 10

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surface and is overlain by 2 feet of clayey silt fill above 6.5 feet of a very stiff, residual/colluvial clayey silt (Figure A.42). The footings located at the base of the excavated basement walls of the damaged homes are probably founded on rock (claystone).

Third Incident

In August, 1973, during the same period the three homes in the second incident were damaged, a third subsidence incident occurred a mile north of the second involving a $300,000, 24-unit, apartment complex which was in the final phase of construction. Over a period of two weeks, minor cracks that formed during the onset of ground movement continued to open and became major structural defects. Despite attempts to repair the building, it became so unsafe that it had to be razed. At least nine inches of settlement occurred across a circular area about 120 feet in diameter.

Fourth Incident

The fourth and most recent subsidence incident took place in September, 1980, in the general vicinity of the first and second incidents and damaged only one home. Ground movements were so slight that a subsidence trough was not perceptible and took place gradually over a period of about a month. No distinct period of rapid movement occurred. Damage included cracking of the basement floor, the cinderblock walls in the integral garage and a portion of the exterior brick facing.

All of the homes damaged in this community were two-story, split-level, frame structures with a combination of brick facing and wood siding. The homes were built between 1957 and 1966.

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Case History 11

Large Effective Angle of Draw Associated with Active Mining

In early September, 1966, residents of 22 of the 29 homes in a small community in western Pennsylvania began discovering cracks in their basement floors and walls. During the last two weeks of September, dull, rumbling and explosion-like noises emanated from the ground below their homes. At least once, the noises were accompanied by shaking of the ground and the homes.

Since 18 of the affected homes were located over yet unmined coal and since only preliminary room-and-pillar mining had been conducted below four of the homes (Figure A.43), state officials concluded that the ground movements originated from an area of total extraction mining which, at the time, was taking place west of the community in the Lower Kittanning Coal, approximately 700 feet below ground surface. The nearest home was 160 feet laterally from the mining, while the farthest was 840 feet. Whereas an angle of draw of only 13 degrees would have been required to affect the nearest home, an angle of draw of at least 50 degrees would have been required to affect the furthest (Figure A.44). The latter angle of draw is much greater than is normally observed. Mine maps indicate that coal nominally 40-inches in thickness was removed.

All damaged homes were wood-frame structures with basements. Reported damage included (or was limited to) cracking of the basement walls and floors. Five of the homeowners noted cracking of interior plaster walls. No tilting of any home was reported and damage seemed to have been caused mainly by horizontal ground movements.

The most severely damaged home was also the one closest to the pillar-extraction mining. Damage to this structure (Home A on Figure A.43) consisted of several 0.75-to 1-inch wide cracks in the cinder block basement walls and in the concrete basement floor, a picture window rotated 0.5 inch out of the plane of the wall, and several cracks in the brick facing of one of the exterior walls. The monetary settlement awarded through legal proceedings was $15,000. Figure A.45 illustrates the distribution of settlement costs for all damaged homes.

Figure A.46 is a plot of the monetary settlement for each home versus the distance of each home to the total extraction mining area. In general, the least damaged homes are located furthest from the areas of total coal extraction. The most damaged are the closest. Interestingly, there are also homes with no claimed damage scattered throughout the site.

A few possible reasons for the large angle of draw have been offered (Sossong, 1973):

- The entire community is underlain by a massive sandstone unit which is broken by widely spaced, somewhat irregular

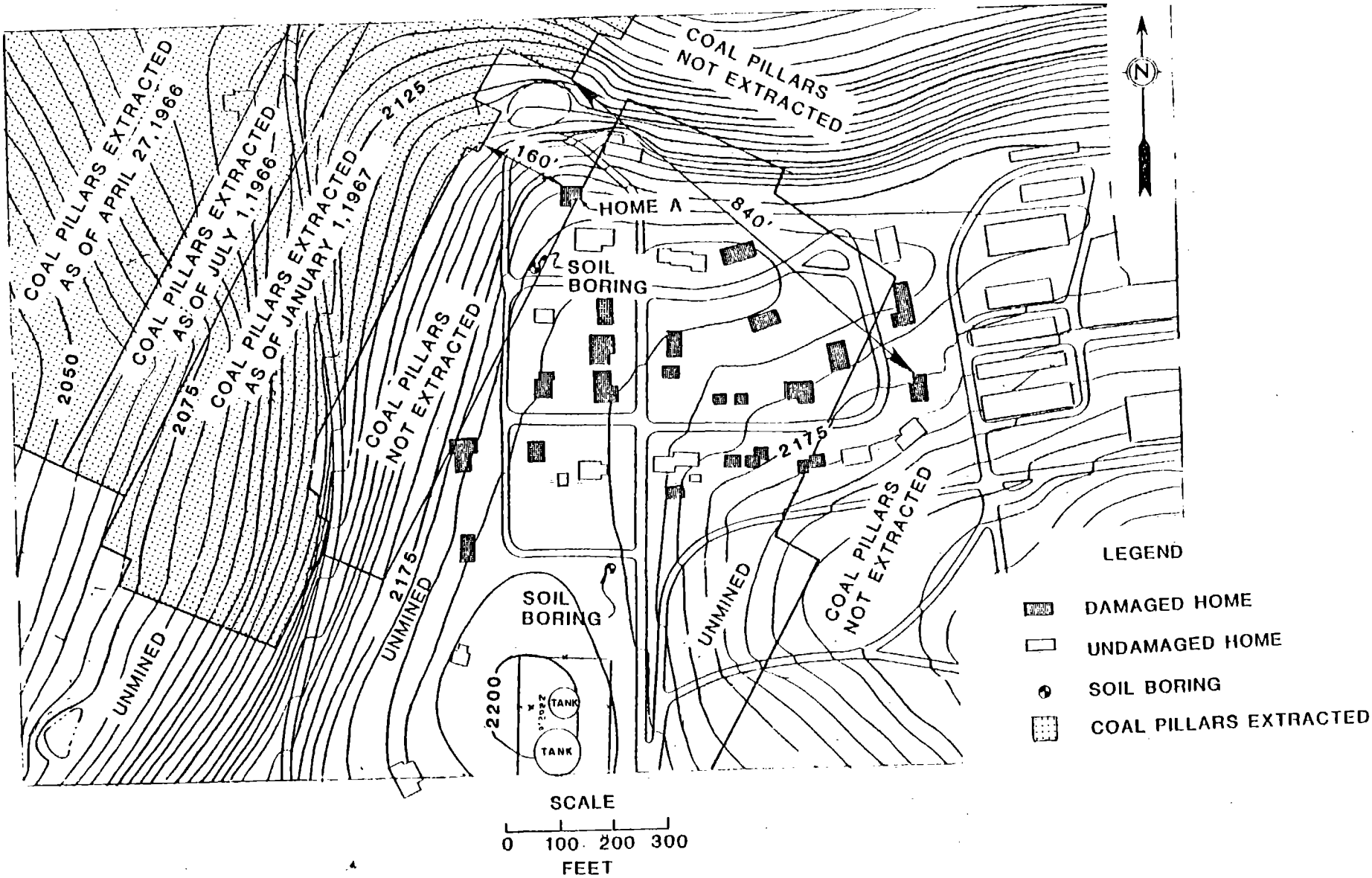


FIGURE A.43–RELATIONSHIP OF DAMAGED HOMES AND SURFACE FEATURES TO UNDERGROUND MINING ACTIVITIES, CASE HISTORY 11

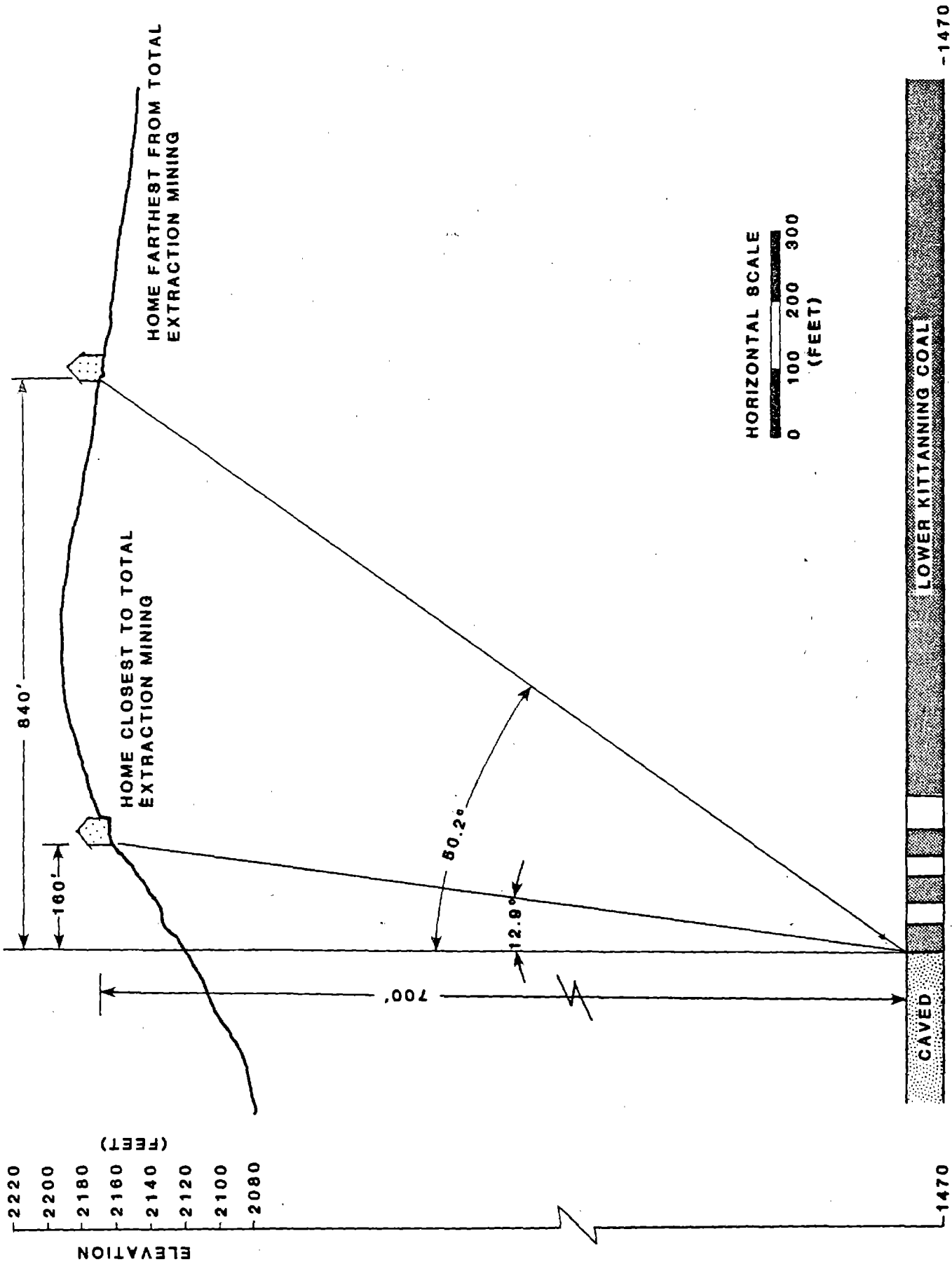


FIGURE A.44 – COMPOSITE DRAWING SHOWING COMPUTED ANGLES OF DRAW, CASE HISTORY 11

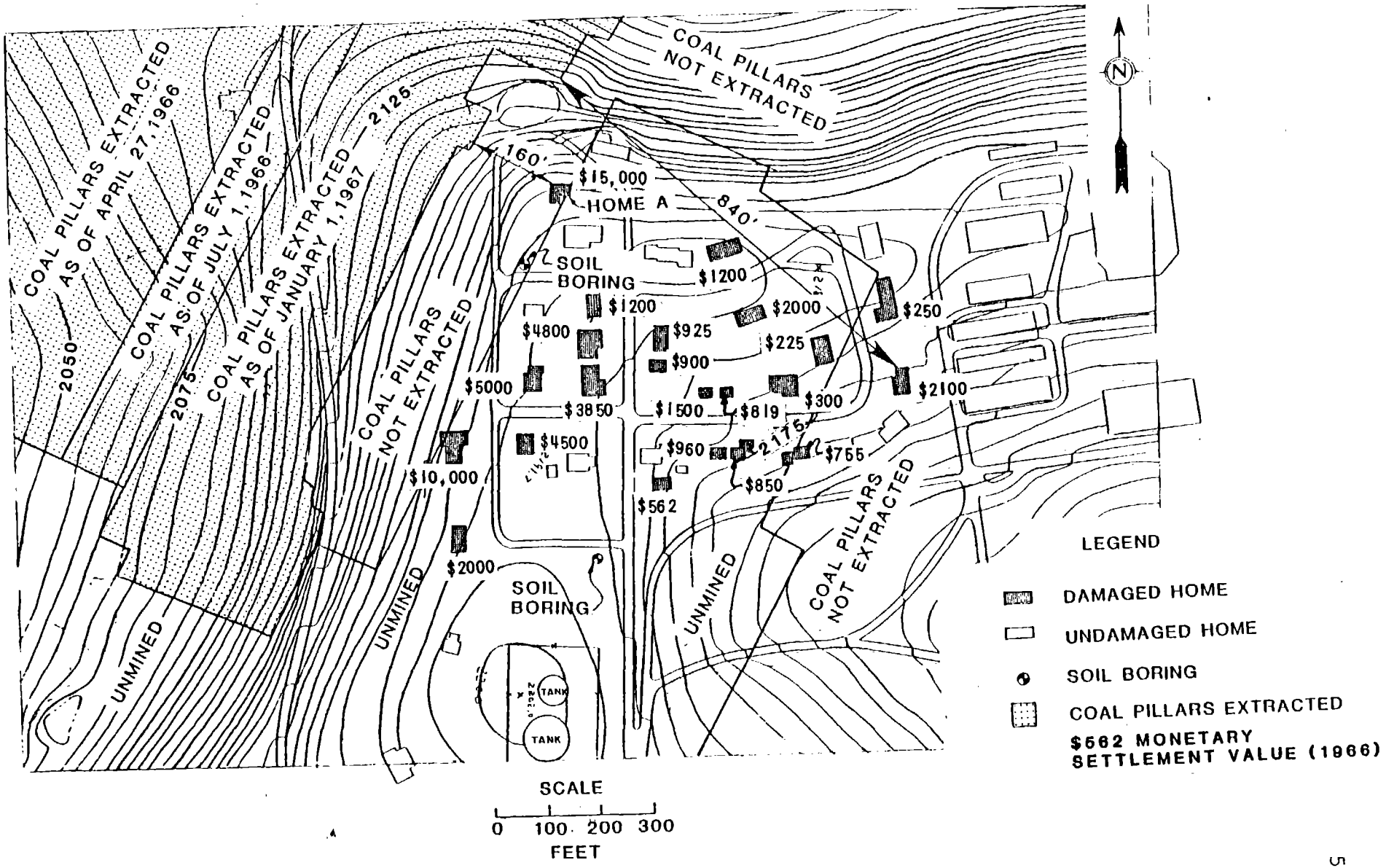


FIGURE A.45–MONETARY SETTLEMENT VALUE (1966) OF DAMAGE FOR EACH DAMAGED STRUCTURE, CASE HISTORY 11

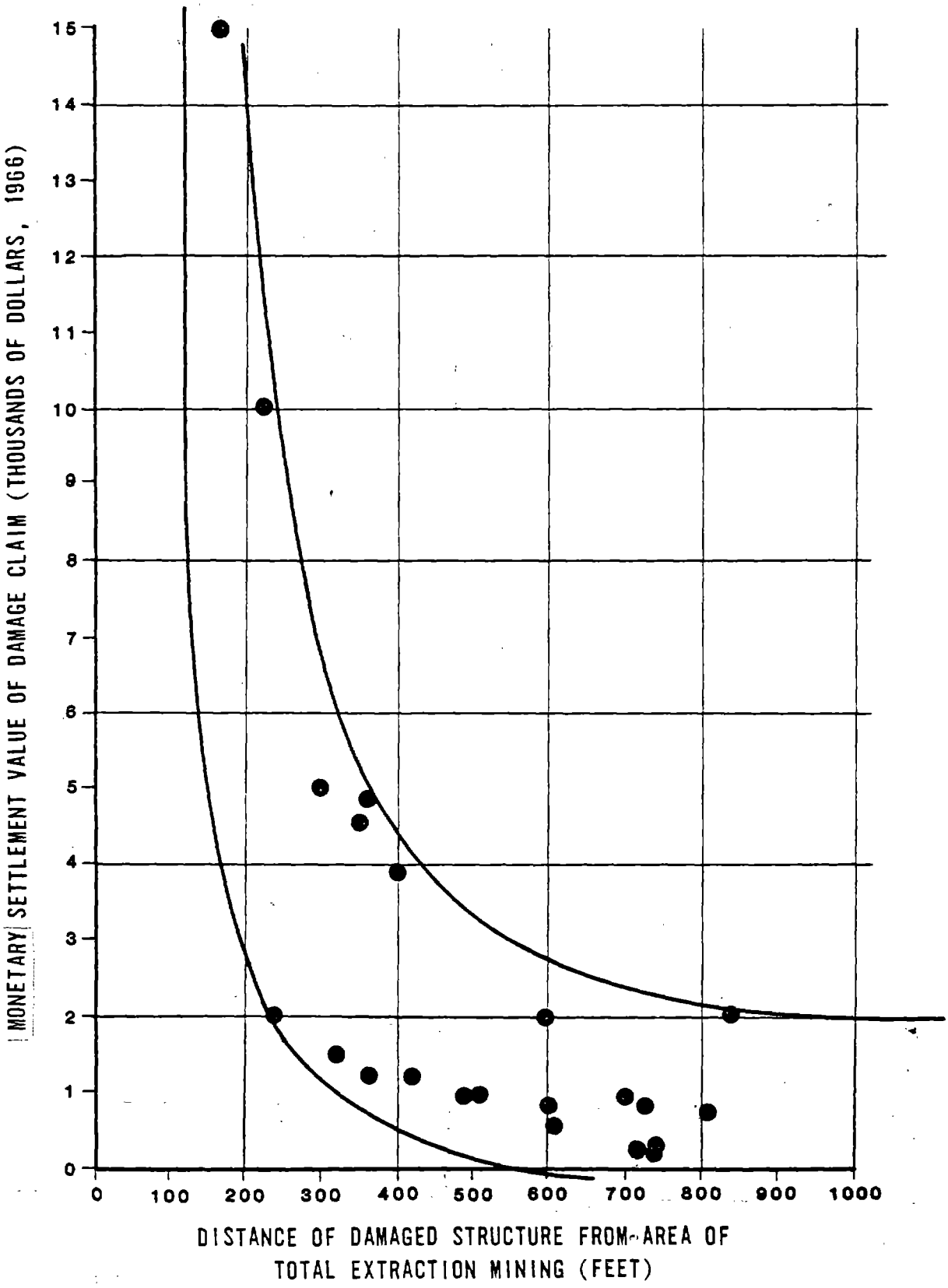


FIGURE A.46–RELATIONSHIP OF MONETARY SETTLEMENT VALUE OF DAMAGE CLAIM VERSUS DISTANCE OF DAMAGED STRUCTURE FROM AREA OF TOTAL EXTRACTION MINING, CASE HISTORY 11

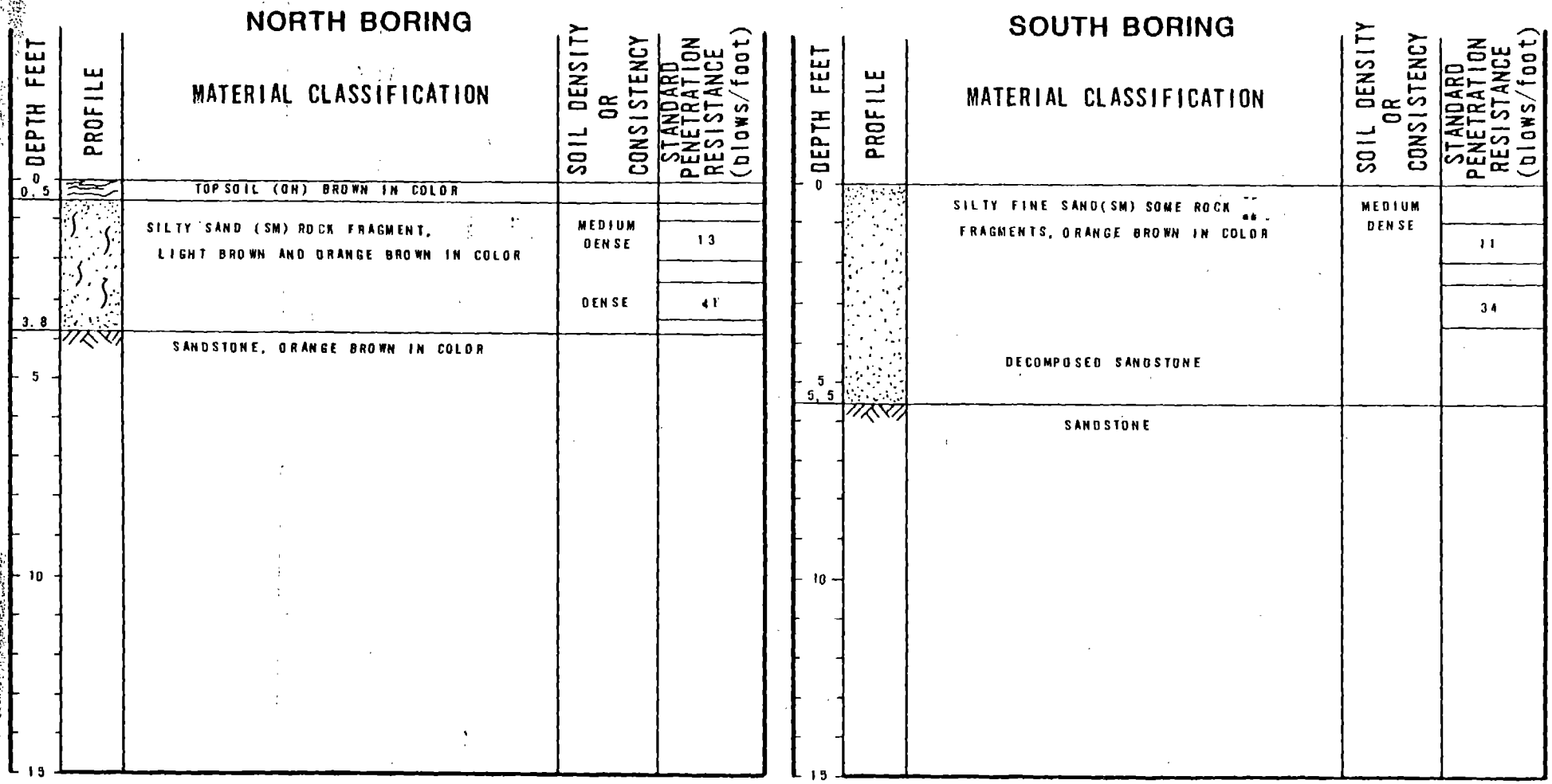


FIGURE A.47 – SOIL BORINGS, CASE HISTORY 11

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joints. In response to the initiating subsidence ground movements directly above the total extraction mine panels, the sandstone unit moved as a mass of interlocked blocks toward those panels. As a result, non-uniform horizontal ground movements were apparently widespread, but limited to areas of greater rock joint concentrations. Homes in immediate proximity to joints were damaged. Homes not underlain by joints presumably moved entirely with the sandstone blocks below them and thus sustained no damage. The fact that the site is directly underlain by sandstone was confirmed by shallow test borings conducted during the present study (Figure A.47). Soil cover, consisting of medium dense to dense silty sand with rock fragments, was found to be no greater than 5.5 feet thick. Most homes are apparently founded directly on bedrock.

- From a topographically high point near the center of the community, the ground surface slopes downward at 10 degrees toward the total extraction mine areas and at 3 degrees in the opposite direction (Figure A.44). The steeper slope toward the mined areas may have led to increased ground movements in the downslope direction and fostered the development of the greater than usual angle of draw.

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Case History 12

Presumed Heave of Overburden Between Two Total Extraction Mining Panels

At this site in Western Pennsylvania, twenty-one homes are evenly distributed along both sides of a 1400-foot section of north-south oriented road (Figure A.48). In March, April and May, 1975, ground trembling was periodically felt and noises heard by residents of this community. Eventually owners of nine homes reported their homes damaged.

All of the homes are located along the crest of a ridge with shallow valleys on either side to the east and west. The residual/colluvial soil at the site is 2 to 7 feet in thickness and all of the homes are founded on bedrock. Some of the damaged homes have basements while others do not. In general, damages due to the ground movements consisted of cracked basement floor slabs, cracks in the basement and interior plaster walls (primarily on the north and south sides of the homes), sticking doors and windows, pronounced tilt of most homes and several swimming pools, and cracked driveway pavement and patio slabs. Measured differential vertical movements varied between 1.2 and 9 inches across the widths of the homes.

The ground movements were attributed by homeowners and state officials to mining activities, which, in 1972, removed approximately 44 percent of the coal directly below the site (leaving the remainder as coal pillars) and 100 percent of the coal in two large areas 525 feet apart located southwest and northeast of the site (Figure A.48). In 1973, additional coal was taken from the coal pillars below half the site increasing the extraction there to about 53 percent. All mining took place in the flat-lying, 7-foot thick Pittsburgh Coal located from 370 to 430 feet below ground surface. Due to a series of factors ([1] the damaged homes were not directly undermined by the total extraction mining; [2] they were not within a 15 degree angle of draw zone around the total extraction mine panels where ground movements might have been expected, and [3] the amount of money requested by many of the residents to repair their homes was substantial, the average amount ultimately paid to the homeowners for damages being about $37,500 [1977]), the mining company retained a consultant to investigate all possible causes of ground movement that might have been responsible for the ground movement. The following conclusions resulted from that study:

o Although the ground surface slopes away from either side of the road, no evidence of sudden slope movement such as ground cracks, scarps or bulges were observed either by direct field inspection or from studies of aerial photographs. Slope failure was not the likely cause of damage.

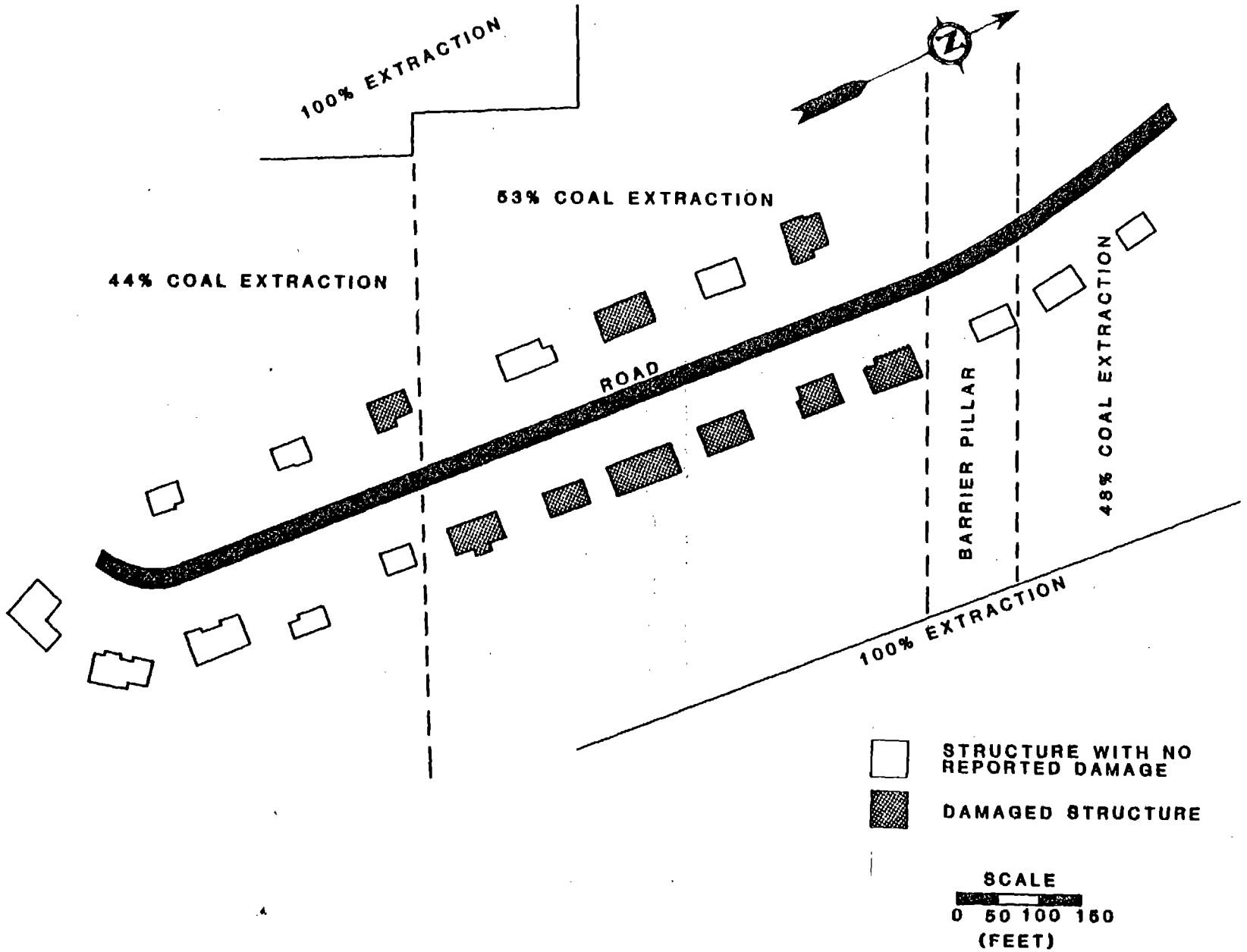


FIGURE A.48–DISTRIBUTION OF DAMAGED HOMES OVER PARTIAL AND TOTAL EXTRACTION MINE PANELS, CASE HISTORY 12

o Although the weathering of pyritic rock strata has been known to cause heave of the ground surface and construction of sewer lines at the site may have exposed fresh pyrite to the weathering process, several years of exposure is usually needed to produce significant ground movements. It was unlikely, therefore, that this mechanism was responsible for the relatively sudden and widespread movements that occurred.

o Moisture changes can cause swelling or shrinkage of certain clay minerals in soil or rock. The community's sewer system could conceivably have been responsible for some ground dewatering, but at the time of the ground movements, wet spring weather was the norm and the extreme drying of earth materials necessary for large scale ground movements was unlikely.

o Frost heave, because of the time of the year when the movements were reported, was also ruled out as a factor.

o Since many of the damaged structures were located far from sewage drains, underground erosion (piping) could not have been the source of ground movements.

o Damages similar to those observed in these homes could have been due to poor design, construction procedures, or gradual deterioration of materials. All of the homes, however, were relatively new and well built. It is unlikely that this type of damage would appear simultaneously in all nine homes in the same, short period of time.

o Mining activities can, under some circumstances, cause regional dewatering of aquifers which, in turn, can cause ground subsidence. Underground mining in this area, however, has been taking place for many years. No sudden change of groundwater conditions was observed. No large influx of water into the mine occurred.

o Although there are no records of mining in the coal seams above the Pittsburgh Coal in this area, the remote possibility exists that ground movements resulted from an abandoned mine located in one of these seams.

o Inadequate pillar support or total extraction mining below or in close proximity to surface structure can cause damaging ground movements. At this site, however, coal extraction below the damaged homes was not excessive according to accepted practice; no damaged structure was located within a 15 degree angle of draw zone around the total extraction mine panels; and the most serious damage was suffered by the homes furthest away from the total extraction areas. A visit into the mine below some of the damaged homes was made by state officials. A full inspection of the mine below all of the homes was impossible, however, due to roof falls and water. Since floor heave was noticed in at least some of the areas inspected, this mechanism could not be ruled out as the cause of the ground movements.

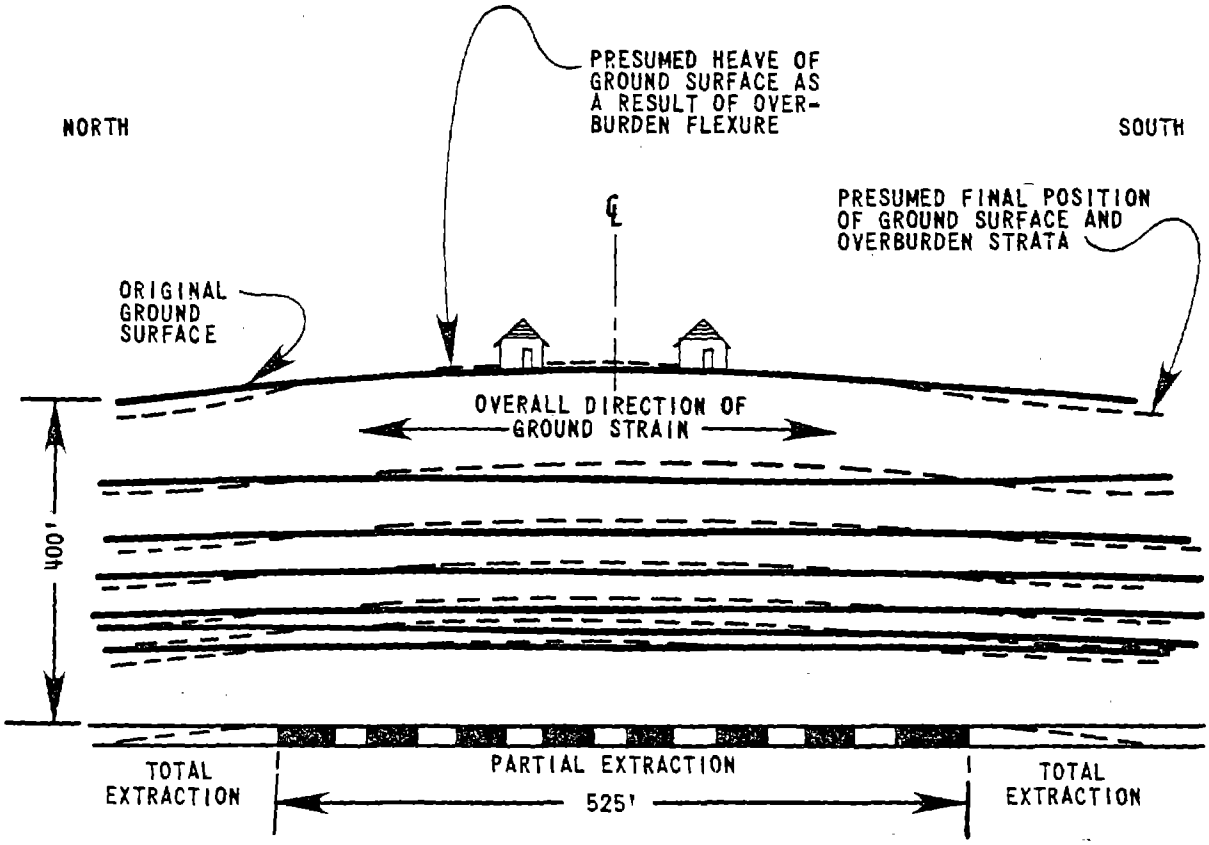


FIGURE A.49—CONCEPTUALIZED CROSS-SECTION ILLUSTRATING PRESUMED MECHANISM OF GROUND HEAVE, CASE HISTORY 12

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o Significant thicknesses of limestone and sandstone strata exist in the overburden. Conceivably, such stiff strata could have acted as a large beam which, when the coal was removed in the total extraction panels east and west of the damaged homes, deformed as shown in Figure A.49. Attendant heaving of the ground could have caused the type of crack damage and pronounced tilting observed in most of the homes. Unfortunately no survey measurements had been made and the direction of vertical ground movements is unknown.

The conclusions presented are those reached by D'Appolonia Consulting Engineers, Inc. The title of the report and name of the client are withheld for purposes of confidentiality.

Case History 13

Homes Damaged by Subsidence Resulting from Pillar-Extraction Mining

From mid-1972 to late-1974, twenty five homes located along or immediately adjacent to a road that passes through a western Pennsylvania community were gradually undermined and subsided by pillar extraction mining (Figure A.50). Removal of the 31- to 40-inch thick, Lower Kittanning Coal at a depth of 665 feet took place in distinct mine panels where 30-foot by 120-foot, rectangular-shaped, coal pillars were mined in pairs. Pillar extraction within each mine panel progressed from south to north and from west to east as one panel after another was mined.

All of the affected homes are situated on terrain that exhibits very little relief. The road along which most of the homes are located rises a gentle two degrees from east to west. Overburden at the site is almost entirely rock. Shallow soil borings found the overlying soil to be a clayey silt, 1.75 to 3.25 feet in thickness, and residual in origin (Figures A.51). All homes in the community are founded directly on rock.

Although twenty-five homes over the four mine panels were undermined, only fifteen reported subsidence damage. None of the homes that are located outside the boundaries of the panels reported damage. Thirteen of the fifteen damaged homes are 1-story, wood-frame structures. Two are 1.5 stories. All but one of the homes have basements. Six of the homes are brick-faced, six clad with wood siding, and three covered with asbestos shingles. The most seriously damaged are Homes B and C.

Although the exterior wood siding on Home B was not damaged, two of the basement walls, as well as the concrete-slab floor and a footing below one of the damaged walls were cracked as the structure experienced a hogging-type of deformation. The owner measured 1.25 inches of differential settlement along a fourteen-foot length of basement wall. Except for a few hairline cracks in the kitchen and a bedroom plaster wall, no other interior damage was reported. Approximately $4,500 (1975) was paid to the homeowner by the mining company for damage repairs. Slight ground movements were detected by the homeowner for about a year after subsidence first began.

Almost $16,000 (1978) was paid to the owner of Home C for subsidence damage. Damage to this one-story home consisted of 0.5 inch-wide cracks in two of the cinder block basement walls and the concrete-slab basement floor, two one-sixteenth inch wide cracks in the exterior brick facing, an extensively cracked, 5-foot high retaining wall (which eventually collapsed), and a one-inch separation of a large, brick porch from the main home. Except for hairline cracks in the plaster walls above certain doors and windows, no damage was done to the home's interior. The originially flat yard around the home became somewhat hummocky in appearance as a result of the ground movements, but the water well did not go dry.

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The only home with no basement (Home A) reported the least damage. One exterior brick wall suffered minor cracking which, in time, was reported to have closed enough to be almost indistinguishable to the eye.

Of the other twelve homes, nine reported moderate cracking of one or more basement walls; six reported cracks in the basement floor; five, minor cracking of interior plaster walls; two, cracking of plaster ceilings; and three of the four brick-faced homes, minor exterior cracks. One of the few homes not reporting basement damage suffered severe damage to the garage when its concrete roof collapsed. An average of about $1,900 (1975) was paid to these homeowners for subsidence damage.

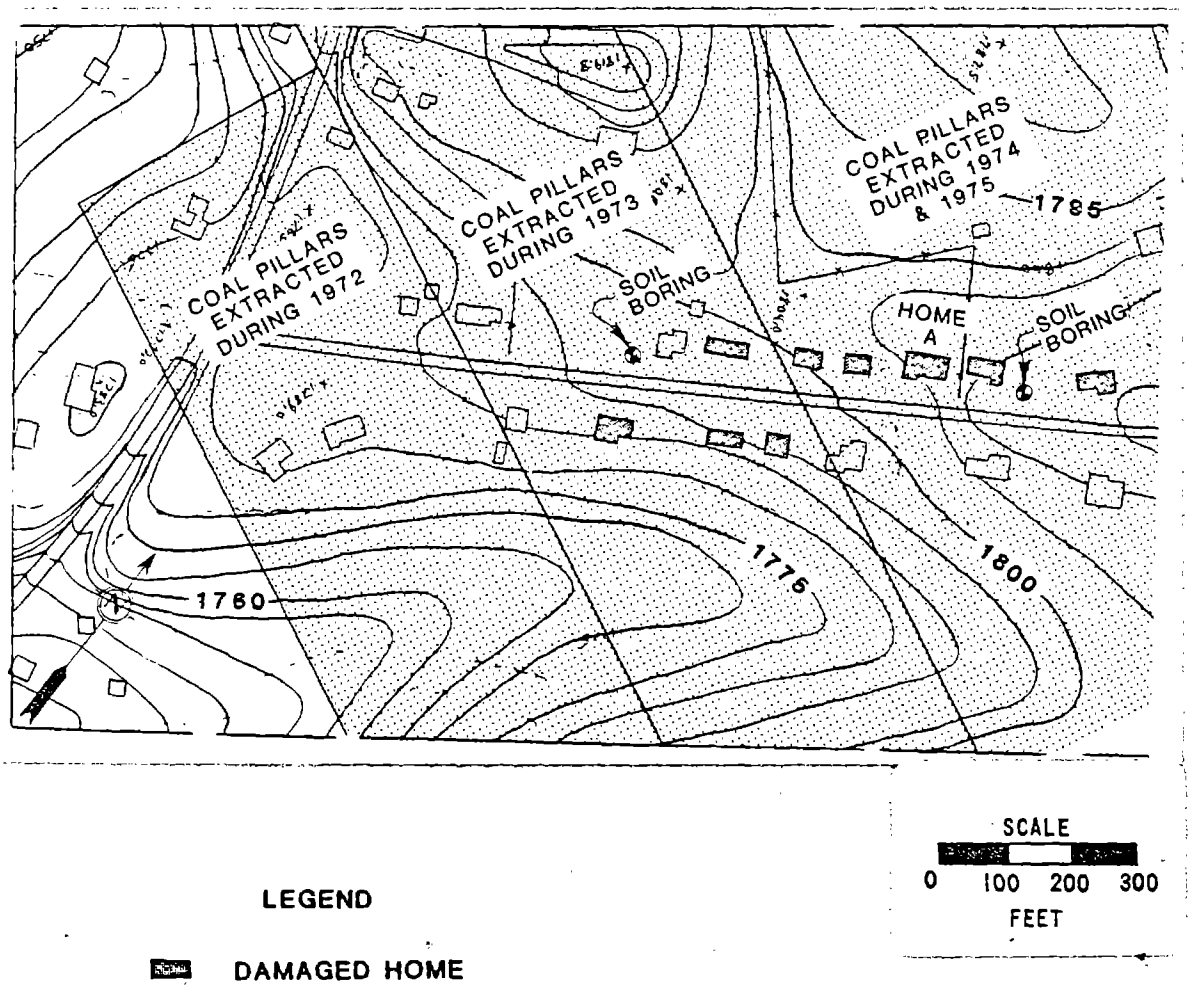
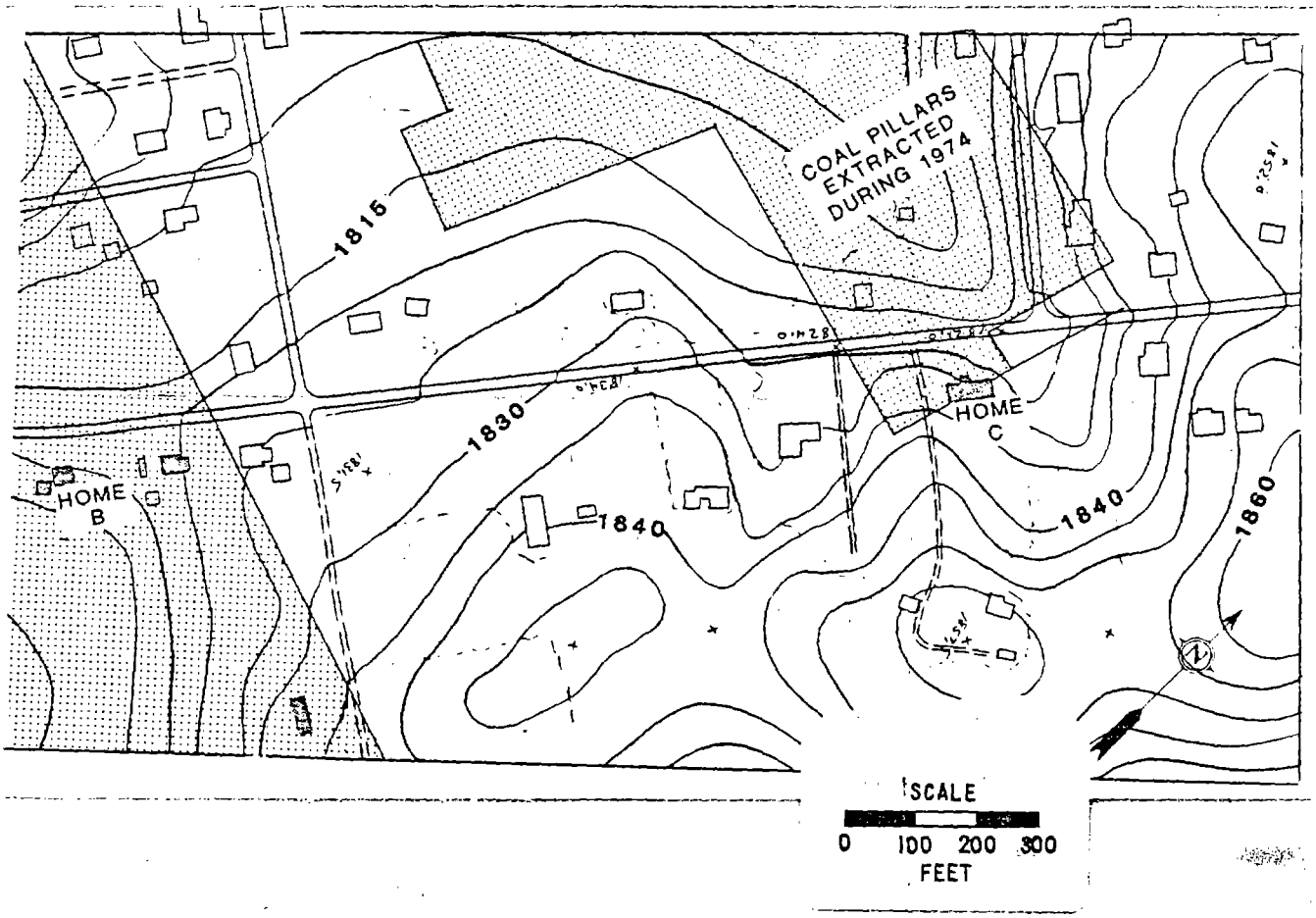


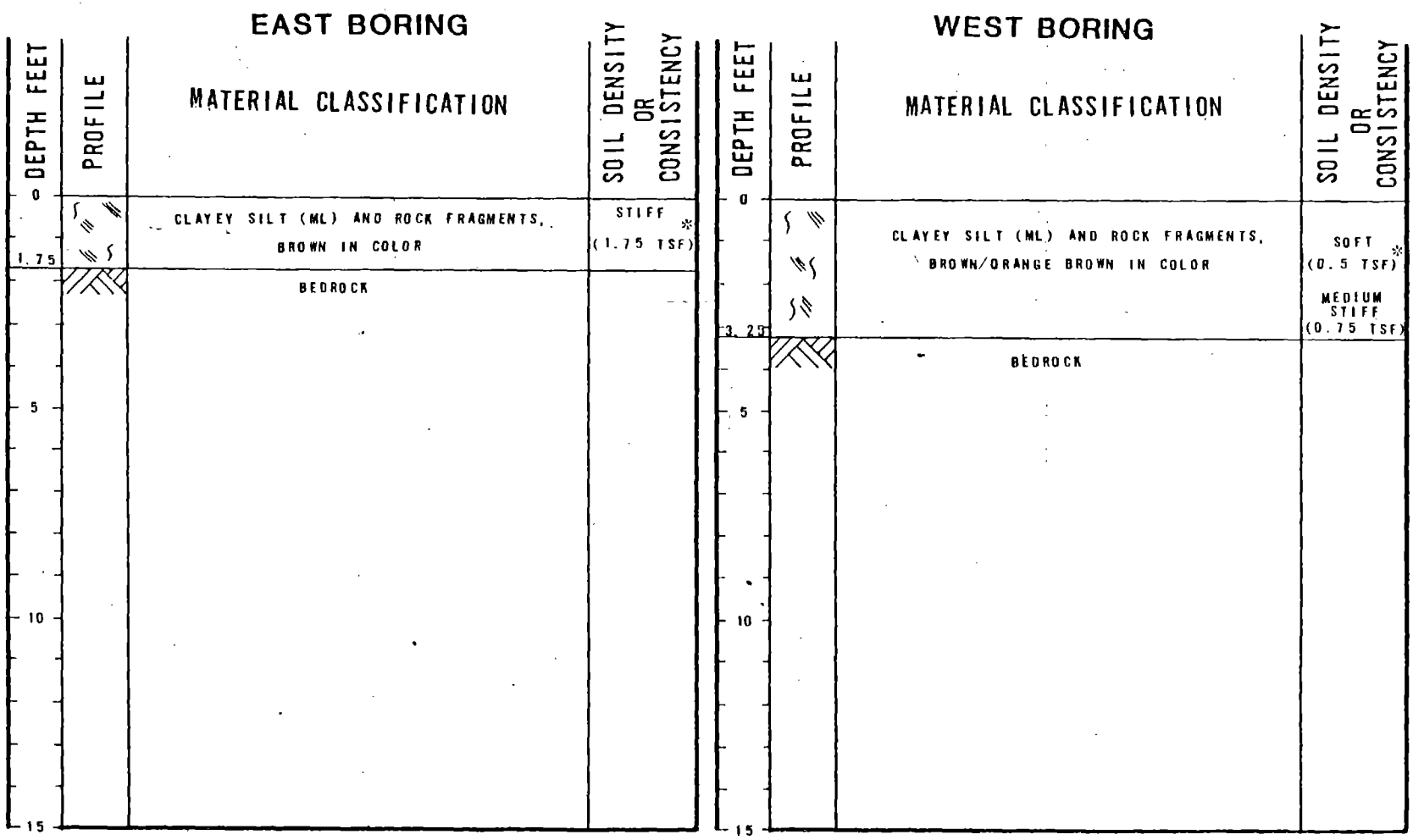
FIGURE A.50–SITE TOPOGRAPHY AND DISTRIBUTION OF DAMAGED HOMES RELATIVE TO MINING PROGRESS, CASE HISTORY 13



LEGEND

- DAMAGED HOME
- UNDAMAGED HOME
- SOIL BORING
- COAL PILLARS EXTRACTED

FIGURE A.50 – CONTINUED



*Pocket penetrometer reading (tons/square foot)

FIGURE A.51 - SOIL BORINGS, CASE HISTORY 13

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Case History 14

Multiple Incidents of Ground Movement Caused by Diverse Mining Events in an Active Mine

According to mine officials, approximately 500 homes in and around this small town have been undermined. At one time or another over a 50-year period of mining, about 100 of these structures have been directly affected by subsidence-related ground movements. Mining, however, was probably not responsible for damage to all 100 of the undermined and damaged buildings. In one part of the area, damage claims were made at the same time that sewer trenches were being excavated through the neighborhood. The excavations may have been the sole cause of unstable ground in that area.

In the following paragraphs a total of 33 damaged homes are discussed at four sites where mining was responsible for ground movements. The locations of the four sites relative to the town, the mine, and a significant soil deposit are illustrated on Figure A.52.

Sites 1, 2 and 3 are directly underlain by a sequence of Pleistocene-age, unconsolidated, river terrace deposits that were deposited in a now abandoned channel loop of the pre-glacial Monongahela River. These deposits reach tens of feet in thickness and form a complex of interlayered and interfingered sands, silts, clays and gravels. Site 4, except for one or two feet of topsoil, is underlain by rock strata. Mine operations below all sites are, or have been, conducted in the 5 to 8-foot thick Pittsburgh Coal, located 380 to 425 feet below ground surface.

Ground movements at Site 1 seem to have been caused mainly by bearing capacity failure of the mine floor, which resulted in uneven downward movement of the coal pillars and overburden. At Site 2, ground movements are attributed to the removal of slices of coal from the sides of coal pillars, which resulted in failure of unstable mine-level strata (roof, floor). Surface damage at Site 3 may have resulted from a combination of total extraction mining in a mine panel adjacent to the site and associated adjustments in loose near-surface soils directly below the damaged structures. At Site 4, pillar failure, or "squeeze", at mine-level caused the damaging ground movements. All four sites are discussed in more detail below:

Site 1 - The damaged homes at this site are part of a compact 75-home community located above a northern edge of the coal mine. All of the homes were built at the same time during 1944/1945 and are one-story in height. Each has a basement with cinderblock walls and a concrete slab floor. The wood frame superstructure of each home is enclosed with red clay, tile block walls. The homes were originally built by the federal government as two family dwellings, but most have since been converted to single family units.

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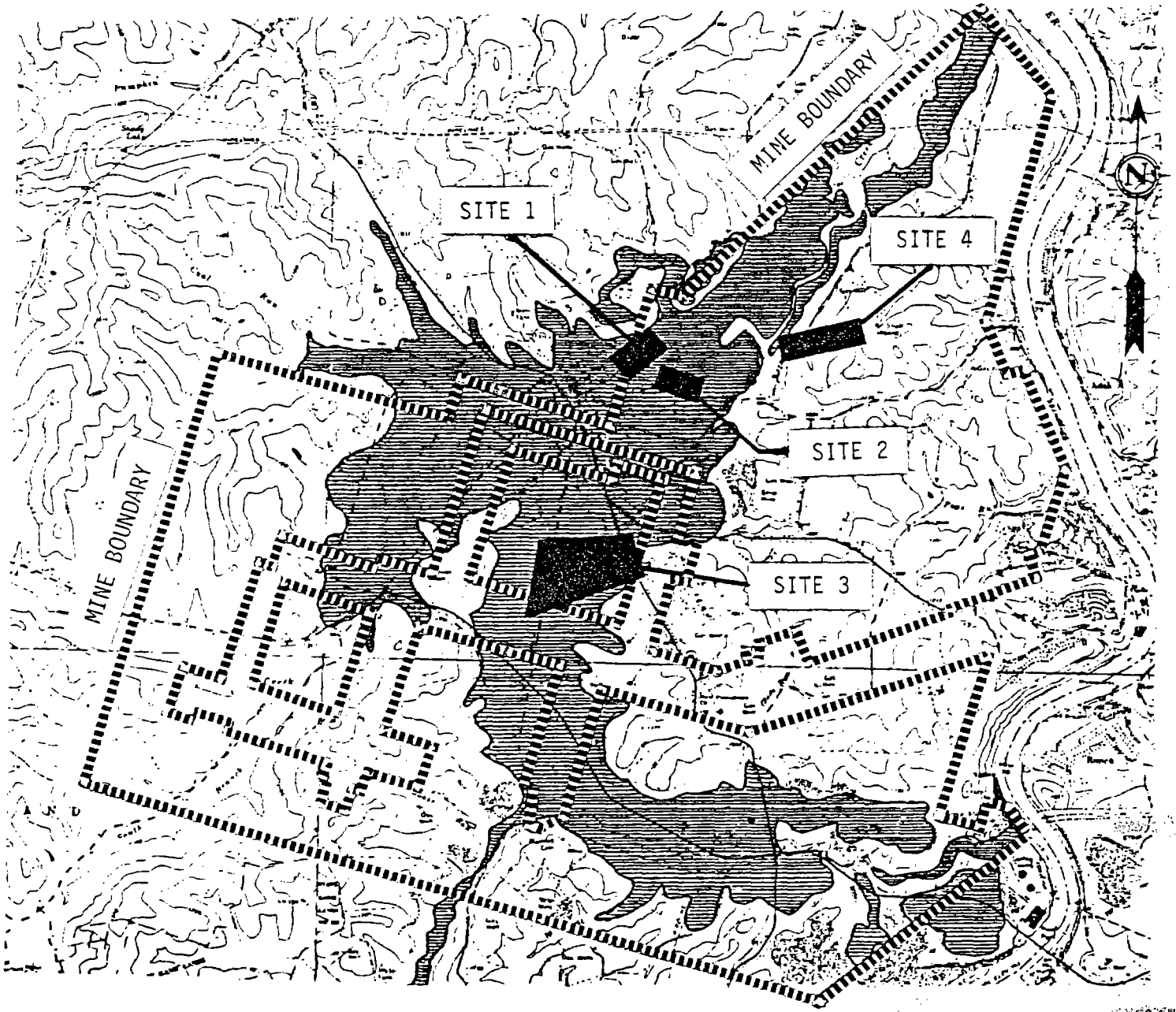


FIGURE A.52 - SITE LOCATIONS, CASE HISTORY 14

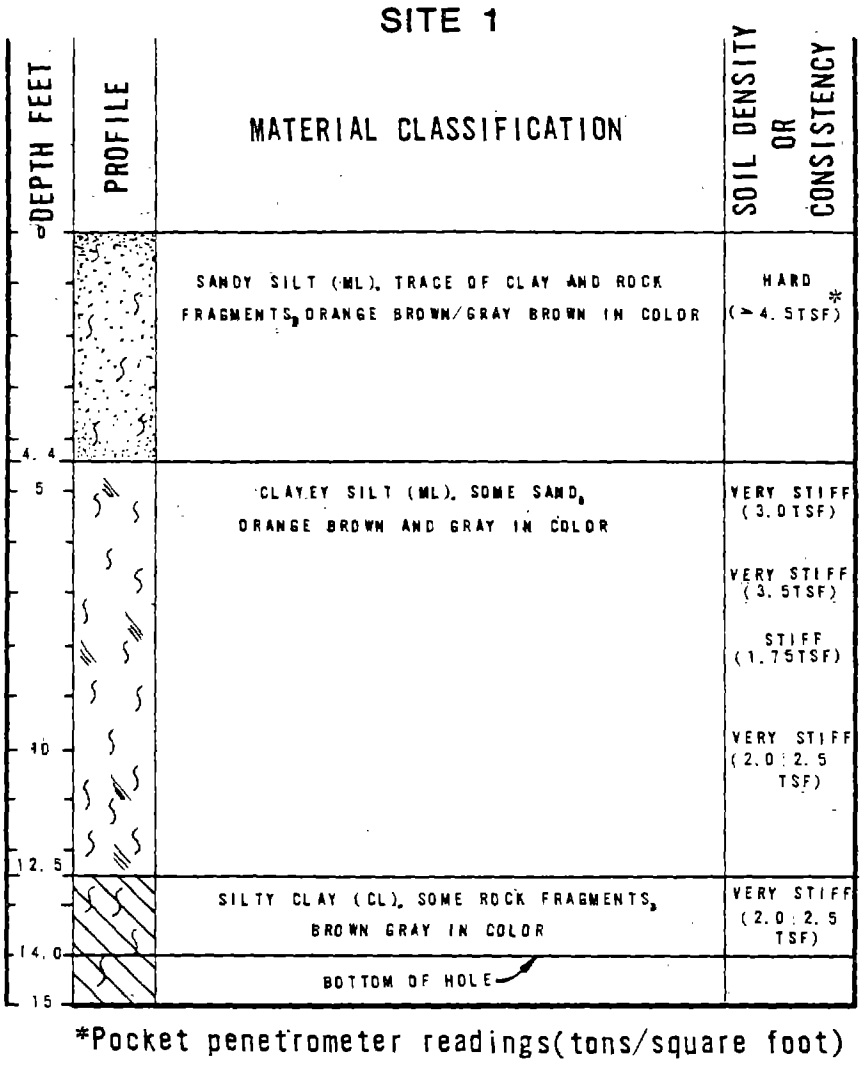
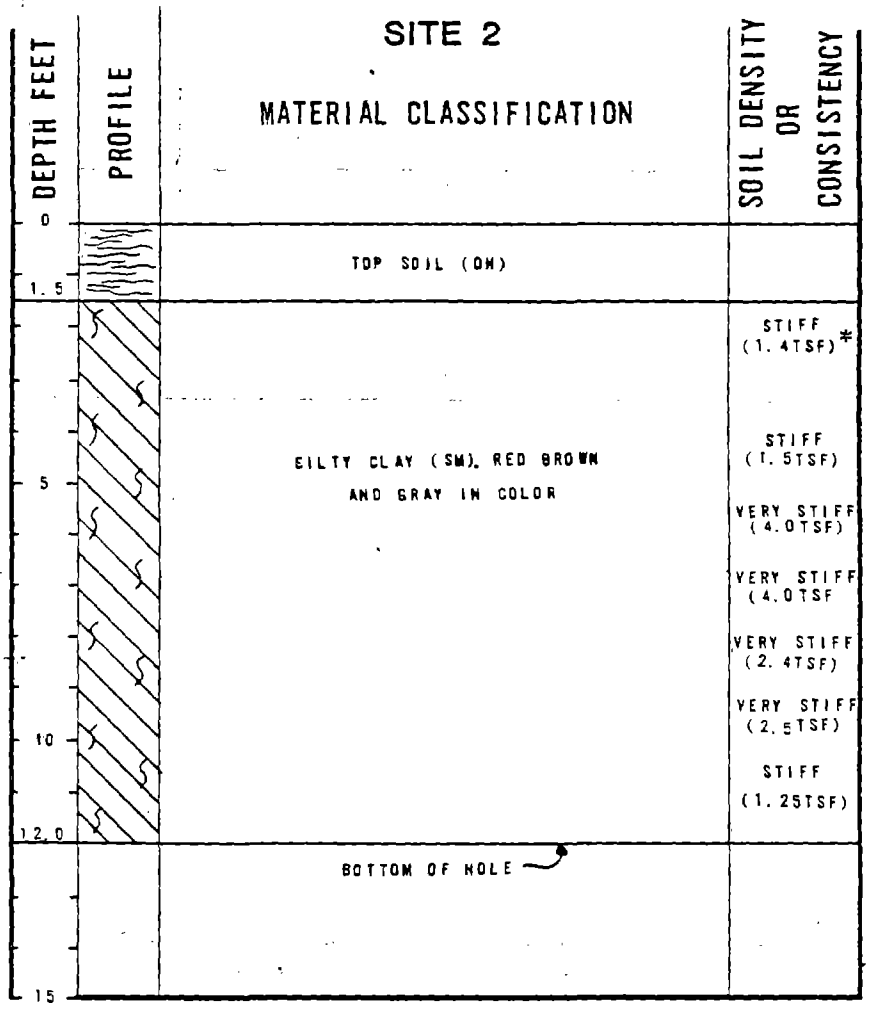


FIGURE A.53 – SOIL BORINGS, CASE HISTORY 14



*Pocket penetrometer reading (tons/square foot)

FIGURE A.53-(CONTINUED)

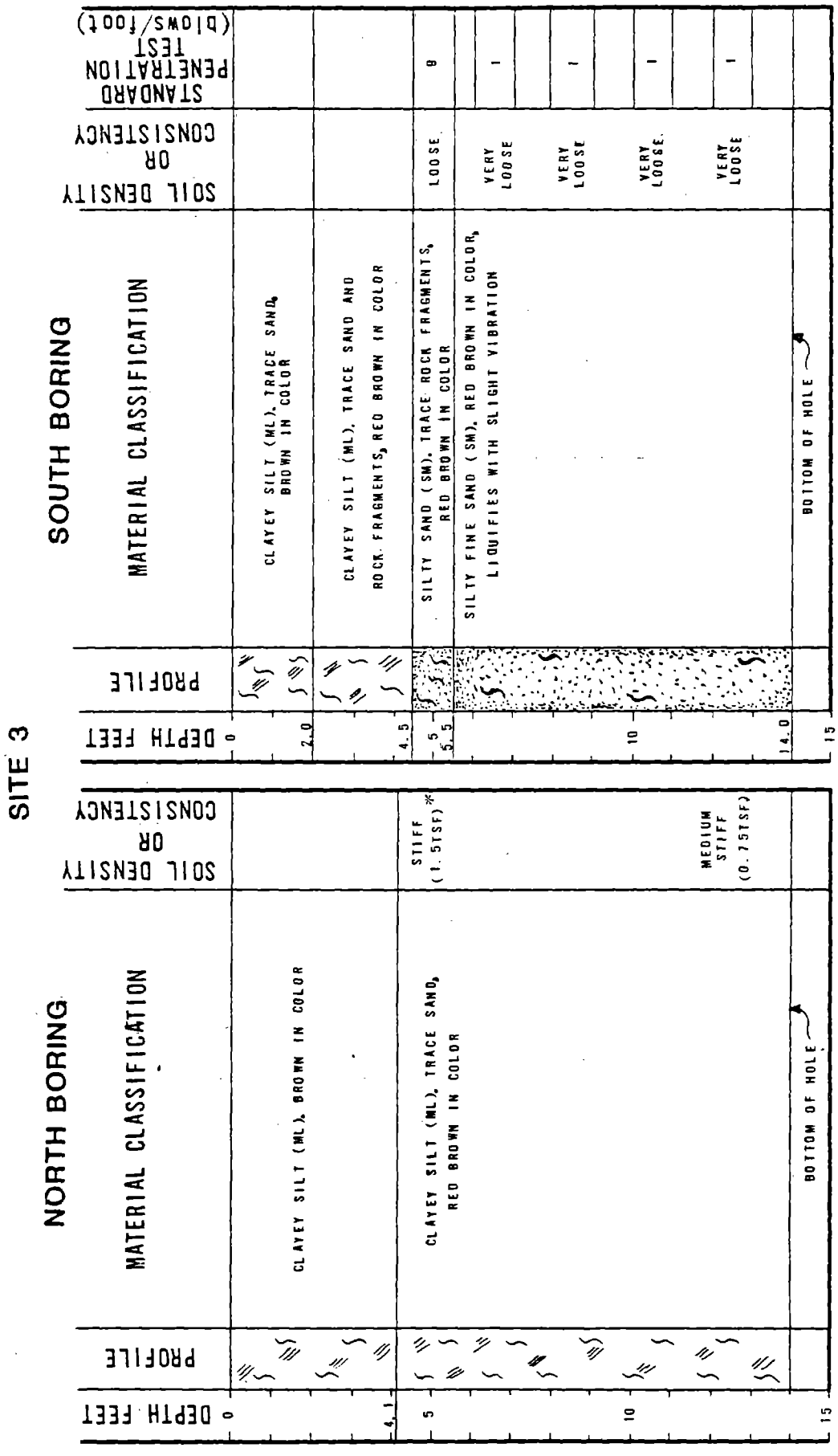


FIGURE A.53-(CONTINUED)

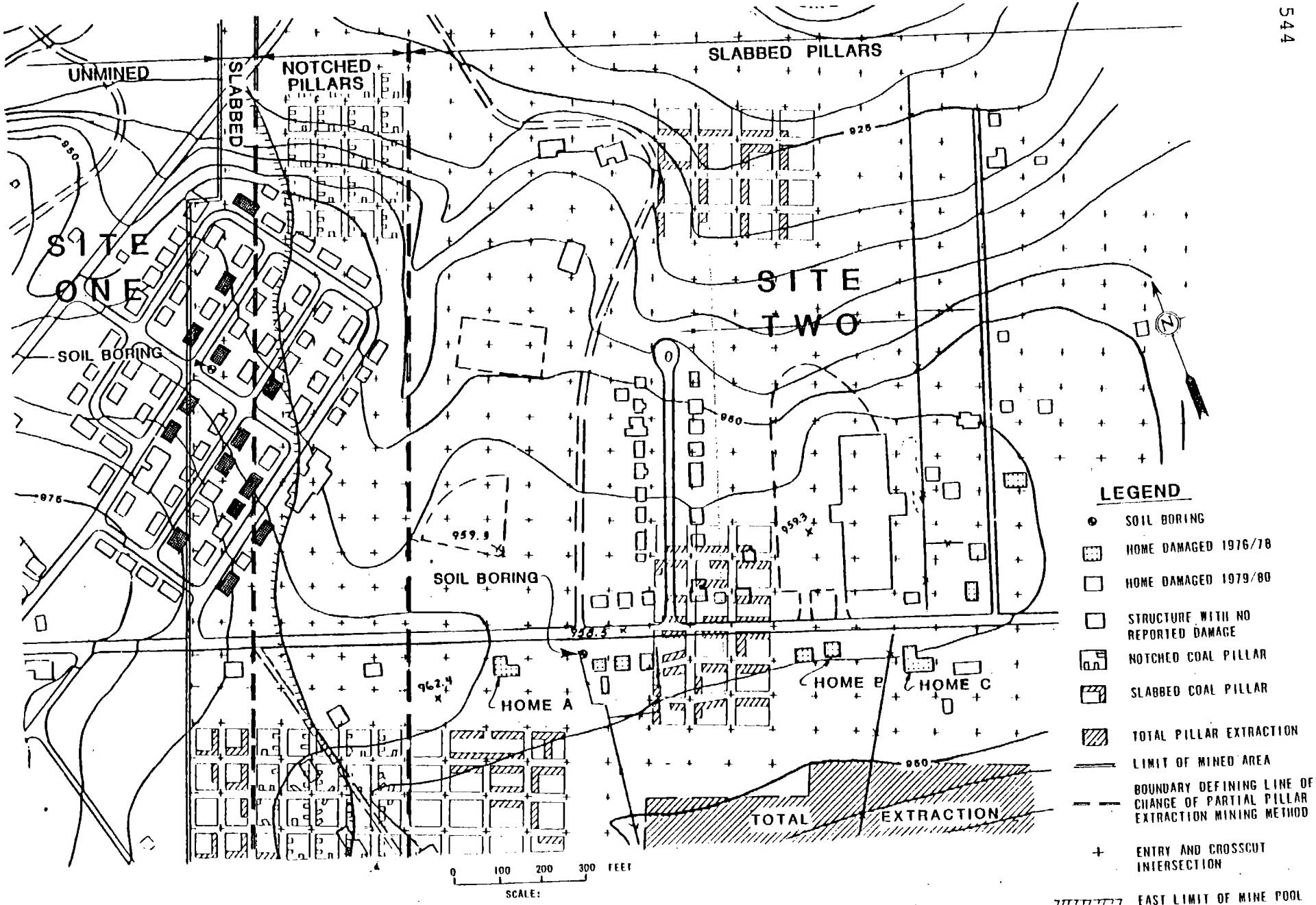


FIGURE A.54 – DISTRIBUTION OF DAMAGED HOMES ABOVE COAL PILLARS–SITES ONE AND TWO, CASE HISTORY 14

Approximately 400 feet of soil and rock separate the homes from the coal mine below. The exact depth to bedrock is not known, although the Pleistocene-age unconsolidated river terrace deposit is probably thicker than 25 feet. Shallow soil borings at the site found that the upper 12.5 feet of soil is stiff to hard sandy and clayey silts (Figure A.53). The soil directly below the silt is very stiff, silty clay. All buildings in the community are founded on soil. The terrain throughout the community is essentially flat.

As shown in Figure A.54, about a third of the homes in this community are located above a mine barrier-pillar that separates the active mine from an abandoned mine west of the community. Another third of the homes are located above a narrow zone of coal pillars from which slabs of coal were removed between mid-February and March, 1979. The easternmost third of homes lie above coal pillars that were notched by a continuous miner sometime before 1979. Of the 15 homes that reported damage due to ground movements, 13 are at least partially above the mine area occupied by the zone of slabbed mine pillars. The two homes not directly above this area are only 25 feet from it.

Mine officials indicate that water has percolated through the solid barrier pillar from the abandoned and flooded neighboring mine. The area of mine floor covered by water is shown in Figure A.54. A mine inspection showed many of the mine openings to be greatly reduced in height due to heave of the floor. Mine officials speculate that a combination of diminished mine floor bearing capacity caused by the water and reduced coal pillar cross-section caused by mining led the pillars to punch into the floor and the ground surface to move in response.

A second possible cause of ground movements may have been lowering of the groundwater table attendant with the development of subsidence-induced fractures in the overburden. The fractures may have extended upwards to the thick sequence of unconsolidated and permeable sand, silts, and gravels, thereby allowing groundwater to flow into the mine.

Because periodic survey measurements of the ground surface taken within the community over a year's time indicated upward as well as downward movement of the ground surface, it was also speculated that at least some ground movements occurred in response to seasonal weather changes. Had this been a major determinant of home damage, however, many homes in addition to those directly above the slabbed pillars would presumably have reported damage.

Of the 13 homes for which detailed damage information is available, 12 reportedly suffered some cracking of basement walls. Four experienced cracking of basement floors. Eight experienced separations (up to 1-inch) of the concrete floor slab from the basement walls. Twelve suffered cracking of the interior plaster walls and ceilings or warping of interior wood paneling. The door and window frames were warped in eight of the homes; the mortar between the exterior blocks was cracked in five; the roofs of two

homes leaked adjacent to the chimney, and one home tilted slightly. Only one home reported a utility pipe break. Between $5000 and $12,000 (1980) was paid by the mining company for damage repairs to each of eight homes. The average settlement repair cost for these homes was $8100 (1980).

All of the homeowners interviewed felt that ground movements began suddenly during the last week of July, 1979, and continued for at least 3 or 4 days. Some of the residents observed no further movement after that period, while others believed movements continued to mid-1981. Surveys by the mining company show downward ground surface movements occurring at least until mid-1980. Mining operations below the site terminated in March, 1979.

Site 2 - This site is located about 600 feet east of Site 1 (Figure A.52). Nine of approximately 20 homes located there were reportedly damaged by subsidence.

Mining of the Pittsburgh Coal, 380 feet below the site, began in early 1973. Extraction of additional coal by slabbing coal pillars began in mid-1975 and was completed in February, 1976. Reports of damage were filed by eight homeowners in February, 1976 and by one homeowner in August, 1978.

The initial room-and-pillar mining extracted about 35 percent of the coal. Secondary retreat mining varied from section to section in the mine, but, below Site 2, increased extraction to as much as 61 percent. The variability of the secondary mining below Site 2 and at two other mine sections short distances away is illustrated in Figure A.54. Where slabs of coal were cut from the facing sides of adjacent pillars, roof spans up to 55 feet were created at intersections. It is speculated that undersized coal pillars or excessive roof spans produced unstable mine floor and/or roof strata, which led to closure of the mine opening and, in turn, subsidence of the ground surface. The area of total coal extraction approximately 250 feet south of the damage area may also have contributed to the unstable condition of the overburden. (Controlled yielding of smaller pillars in high extraction areas is made difficult by the absence of adjacent support areas for the resulting pressure arch).

Of the nine homes that reported damage, seven suffered moderate to severely cracked basement walls and floors. Seepage of water into the cracked basements during periods of wet weather was reported at six homes. The cinderblock wall in the basement of Home B (Figure A.52) bulged inwards slightly along a long horizontal crack. The basement floor in this home also bowed upwards slightly. The owner of Home A claimed additional cracking of basement walls three years after his original February, 1976, damage claim.

The exteriors of two of the damaged homes were faced with brick. Neither exterior was damaged by ground movements, although the owner of Home C reported that his home tilted forward as a

front corner of his home moved downward 6 to 8 inches. Eventually, compensating ground movements releveled the structure. None of the homes clad with wood or aluminum siding suffered damage.

Seven homes experienced cracking of interior plaster walls, and six reported cracked ceilings. At Home B, the interior damage occurred six months after repairs were made to the damaged basement. Door and window frames in five of the homes warped and the cistern and sewer line at Home B were ruptured.

The ground surface at Site 2 is essentially flat. All homes were founded on soil. Shallow borings indicate that the upper 12 feet of soil is composed of stiff to very stiff, red-brown and gray, silty clay (Figure A.53). The depth to bedrock is undetermined.

Site 3 - By early 1970, several mining activities below this site resulted in the coal pillar configurations illustrated in Figure A.55. Advance mining produced 80- to 85-foot square, coal pillars on 95- to 100-foot centers. Retreat mining produced three separate areas of total pillar extraction and widespread partial pillar extraction wherein coal pillars were reduced in size by splitting, quartering, slabbing and notching. Between January, 1969, and September, 1972, twelve of twenty-three owners of homes along Road Y and along the southern portion of Road X claimed damage to their homes from ground movements resulting from the retreat mining activities (Figure A.55). In March, 1970, five of thirteen homes along the northern portion of Road X were also reported damaged. The actual cause of ground movements is not known, but failure of remnant coal pillars or the mine floor or adjustments brought about by adjacent total pillar extraction may have been responsible.

Deep exploration borings by the mining company show that the entire site is underlain by 40 to 90 feet of unconsolidated sands, gravels and silts. Supplementary shallow borings at two locations indicate near-surface unconsolidated sediments composed of very loose, silty, fine sand that readily liquefies when disturbed (Figure A.53). This sand may have enlarged the subsided area or worsened the effects of rock overburden instability created by retreat.

All but one of the damaged homes experienced cracking of basement walls. Eight basement floors were also cracked and the sewer lines from two of the homes ceased to function properly. All damaged basement walls and floors were repaired by repointing, although a wall in one home had to be realigned and a portion of the footing and concrete basement floor of another home had to be replaced.

Cracks, hairline to one-eighth inch wide, appeared in the interior plaster walls in ten of the homes and in the ceilings of seven.

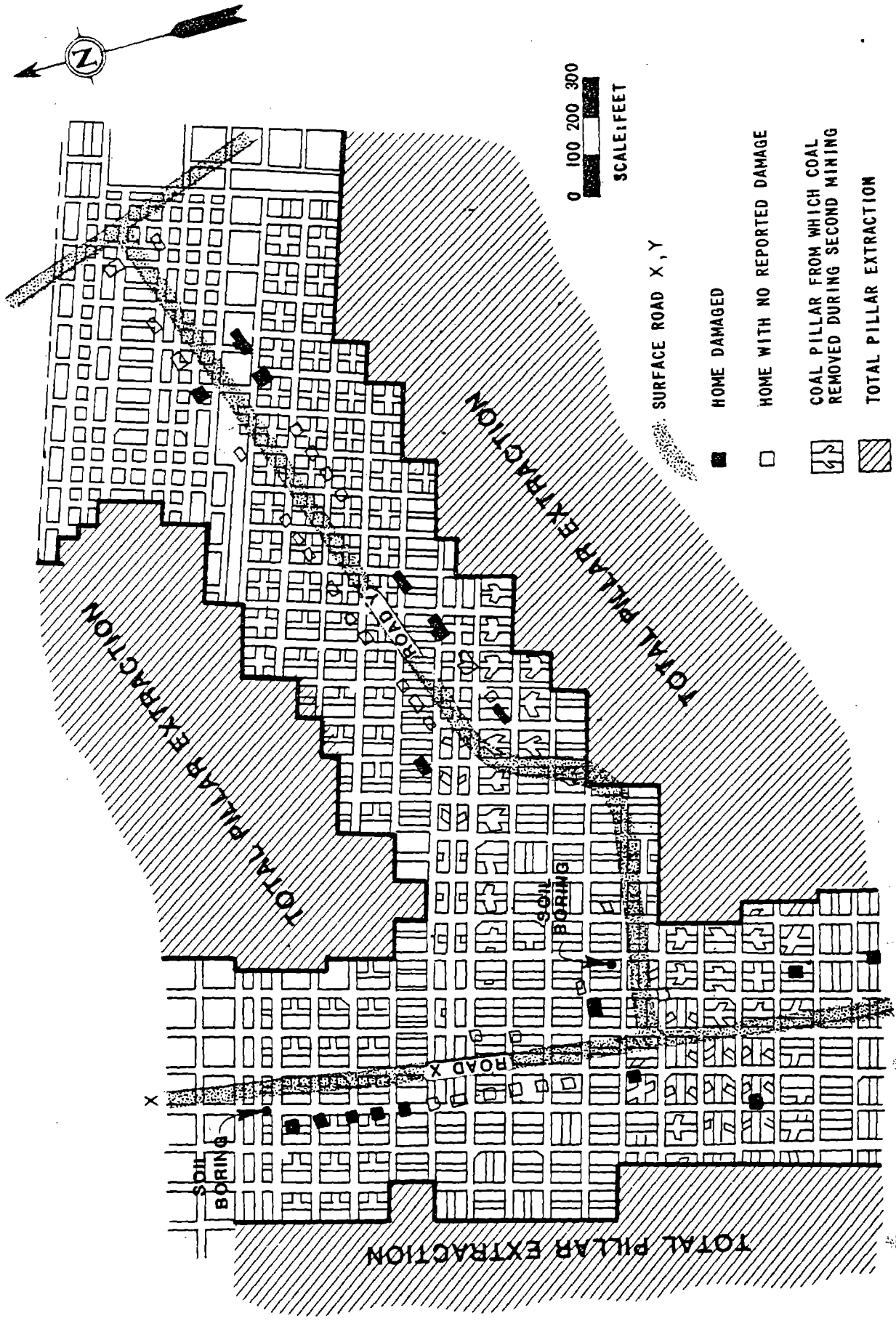


FIGURE A.55 —SITE 3 –LOCATION OF DAMAGED HOMES RELATIVE TO MINING BELOW, [illegible] SE HISTORY 14

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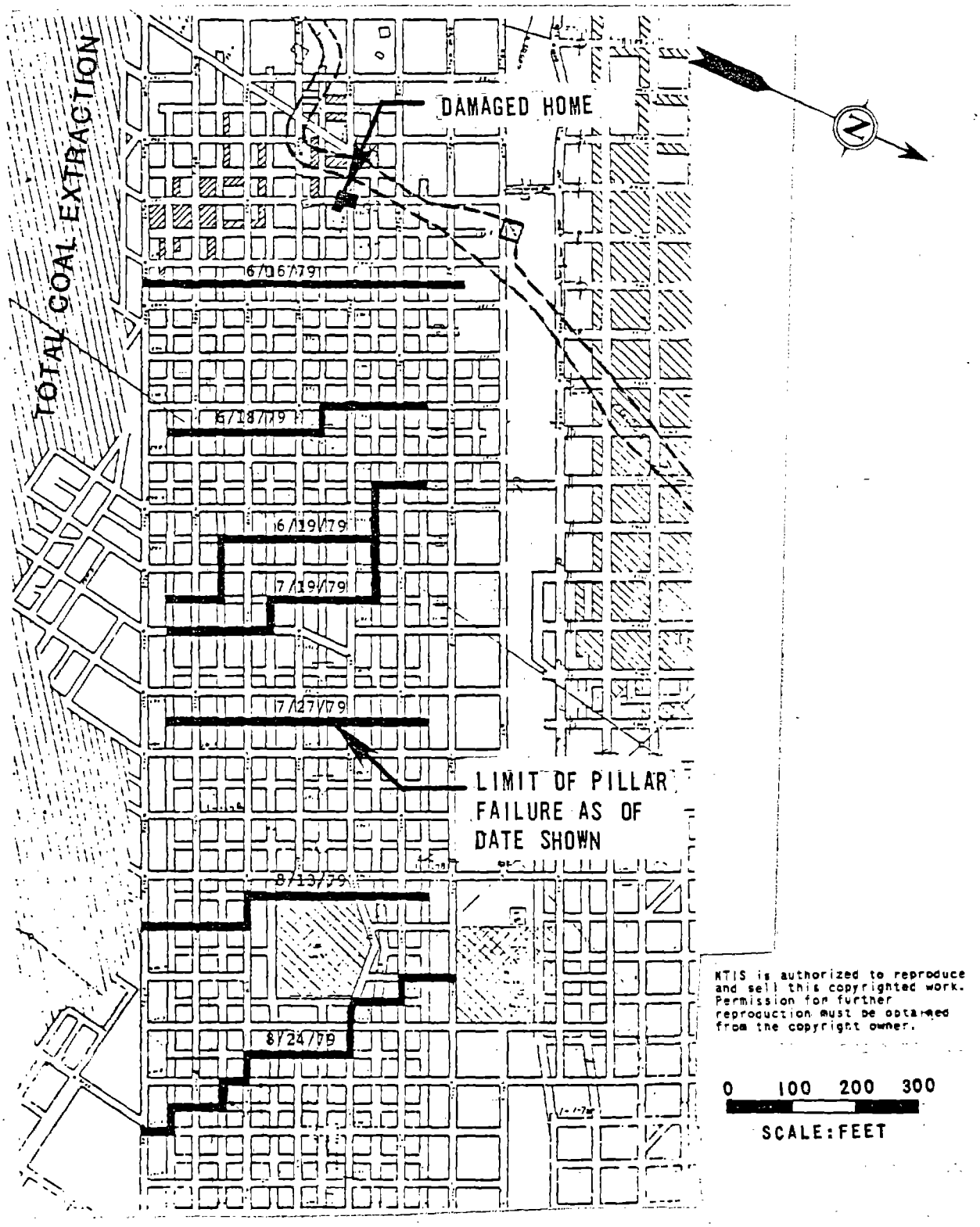


FIGURE A.56 – WEST TO EAST PROGRESS OF PILLAR FAILURE FROM JUNE 16, 1979 TO AUGUST 24, 1979, CASE HISTORY 14
(Modified from Mishra and Grayson, 1981)

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The exteriors of 7 of the 17 damaged homes are faced with brick or stone. Four of these exteriors were cracked and the entire front of one home had to be replaced. No homes with aluminum or wood siding experienced exterior damage.

The cost to the mining company for damage compensation to each homeowner varied from $525 to $12,923 (1972-1976). The average payment was $2540.

Site 4 - Ground surface movements at this site were caused by large-scale failure of coal pillars at mine-level. Fortunately, only one home was damaged. The mine-panel within which the failure took place is approximately 1800 feet long and 600 feet wide (Figure A.56). Overburden thickness varies from 450 to 575 feet. Only one or two feet of soil mantles the rock overburden (at least in the immediate vicinity of the damaged home). The height of the mine opening is 6 feet.

Originally in the late 1940's, 85-foot square coal pillars on 100-foot centers were developed within the mine panel. For a period beginning in 1976, the pillars were reduced in size by quartering. On June 4, 1979, total and partial pillar extraction retreat mining was initiated in the western end of the mine panel. At first the pillar extraction operations created overburden stresses which only caused local crushing of pillars and closure of mine openings. By June 16, however, the crushing of pillars had increased to such an extent that all mine openings across the entire width of the mine panel had closed ("squeezed") and a line of pillar failure began rapidly advancing eastward toward the other end of the mine panel. During the last week of August, 1979, after almost all of mine openings in the panel had closed, the progress of the squeeze slowed and stopped as barrier pillars in the west end of the panel apparently assumed much of the overburden stress.

Movements of the ground surface above the mine panel accompanied closure of the mine openings. The owner of a 150-year old, two-story, stone and brick, farmhouse located over the western end of the mine panel (Figure A.56), reported that, from June 13 to 15, 1979, he and his wife felt several periods of "tremendous shaking". From July 18 to 26 of that same year, several more strong vibrations shook their home and on January 13, 1980, and again on April 28, 1980, loud cracking noises were heard just outside the building.

As a result of the ground movements, cracks from 0.5 to 2-inches in width developed in the farm fields and in the ground yard immediately surrounding the above described house. Several of the cracks interrupted the function of the septic field. One large crack passed below the southern rear corner of the house and the entire structure leaned slightly towards the corner. While cracks in the brick exterior only formed around windows, the prevously horizontal line of bricks was distorted downward toward the right rear corner. Water in the roof gutters ran toward the rear of the house rather than toward the front as designed. The chimney within

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the house was out-of-plumb and concrete patio slabs tilted toward the rear of the house.

Due to the fact that the basement walls were built of stone and were two feet thick, cracking there was not evident, nor was it visible in the stone basement floor. Movement of furnace pipes, however, indicated at least one-inch of vertical differential movement between parts of the superstructure and the basement.

Throughout the interior of the house, cracks developed in the plaster walls and ceilings. Some outside doorways warped, although no window frames were reported distorted.

In addition to the ground movements resulting from the mine-level pillar failure, total pillar extraction in the northern adjacent mine panel also produced large ground cracks in farm fields. A buried gas transmission pipe there was so distorted that it buckled out of the ground.

Case History 15

Subsidence Damage Caused by a Single Episode of Longwall Mining

Between mid-1971 and mid-1972, two longwall panels were developed below ten homes along a 150-foot section of country road in central Pennsylvania (Figure A.57). All of the undermined homes were damaged from resulting ground movements although the degree and type of damage suffered by each home did not necessarily correspond to their position over the theoretically symetrically-subsided mine panels.

Longwall mining was carried out in the 550- to 566-foot deep, 4.5-foot thick, Lower Kittanning Coal. Mine maps indicate that only 3.75 feet of coal was actually removed.

Although hilly terrain is located northeast and west of the site, the ground where the damaged homes are situated is essentially flat. Shallow boreholes drilled during the present study indicate that bedrock lies between 6 and 12 feet below the ground surface (Figures A.58). The soil sampled in the north part of the site was thickest and composed mainly of stiff to very stiff silty clays. Soils sampled in the south portion of the site were predominantly soft to very stiff sandy silts. All sampled soils are probably colluvial or residual in origin. All homes along the road were founded on soil.

Only those homes that were located directly over the caved portions of the longwall panels reported subsidence damage. No reports of damage were filed for homes located above adjacent room and pillar mine areas although at least four of these homes are within 150 feet (equivalent to 15 degree angle of draw) of the boundaries of the longwall panels.

Damage suffered by homes along this road occurred during two distinctly different periods, each period corresponding to mining in one of the two mine panels. The first period in June and July, 1971, involved Homes I and J along the southern portion of the road. The second period, which occurred during the first half of 1972, involved eight homes. Estimates by homeowners of the duration of major ground movements vary from 6 weeks to 6 months. Occasional, minor ground adjustments are still reported by some residents (1981). Half of the homes were permanently tilted as a result of the ground movements. Differential vertical movements of foundations as great as 6 inches were reported, and all but one structure were tilted toward the center of the underlying panel being mined. The previously flat terrain surrounding some of the homes became extremely hummocky in response to subsidence, possibly reflecting a non-uniformity in ground movements that characterized the character and distribution of damage to structures--particularly those located over the center of the mine panels. Home D in the center of the western-most panel (Figure A.57) reported the most tilt and was tilted away from the panel centerline.

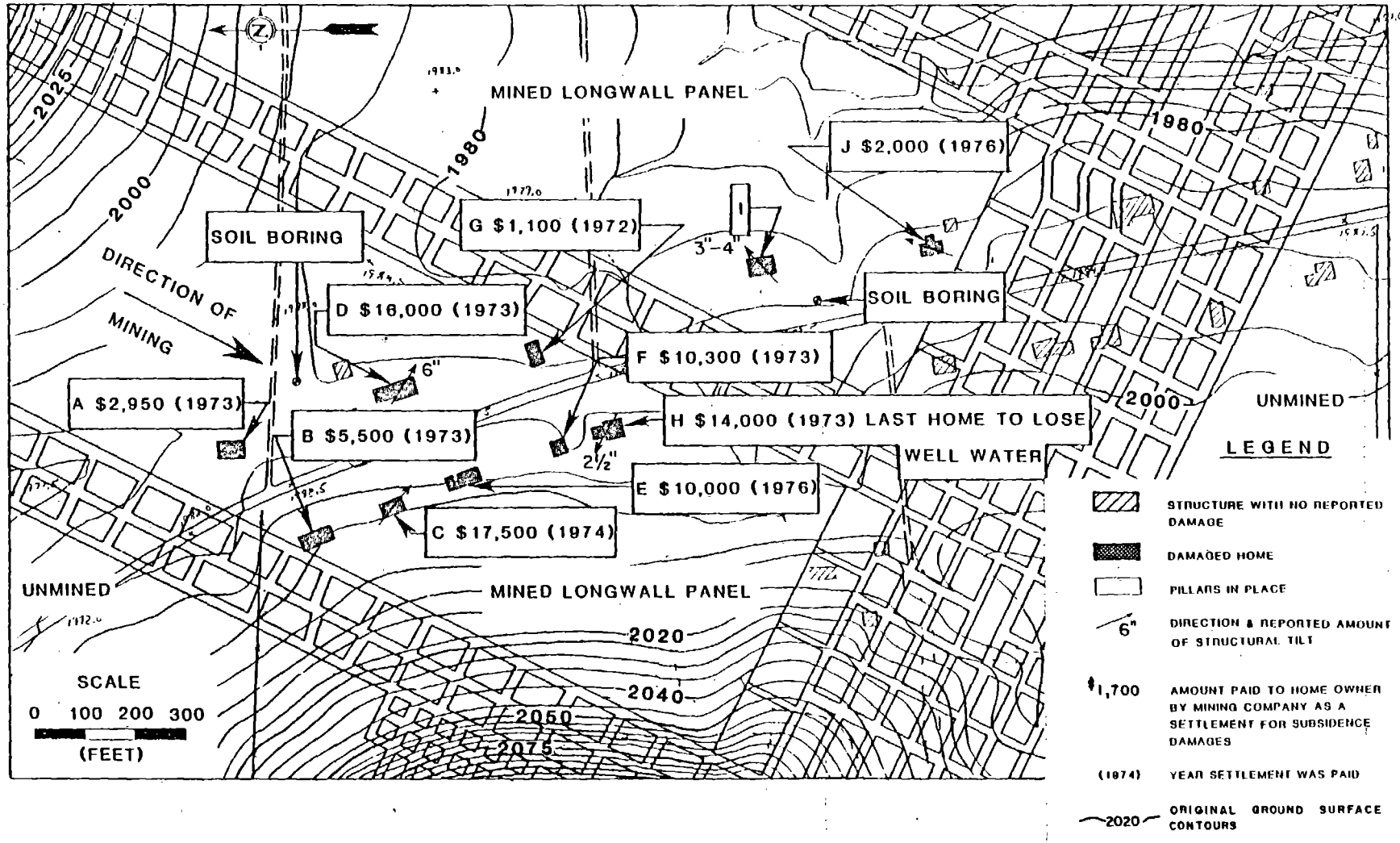


FIGURE A.57- DISTRIBUTION OF HOMES DAMAGED BY SUBSIDENCE RELATED TO LONGWALL MINING, CASE HISTORY 15

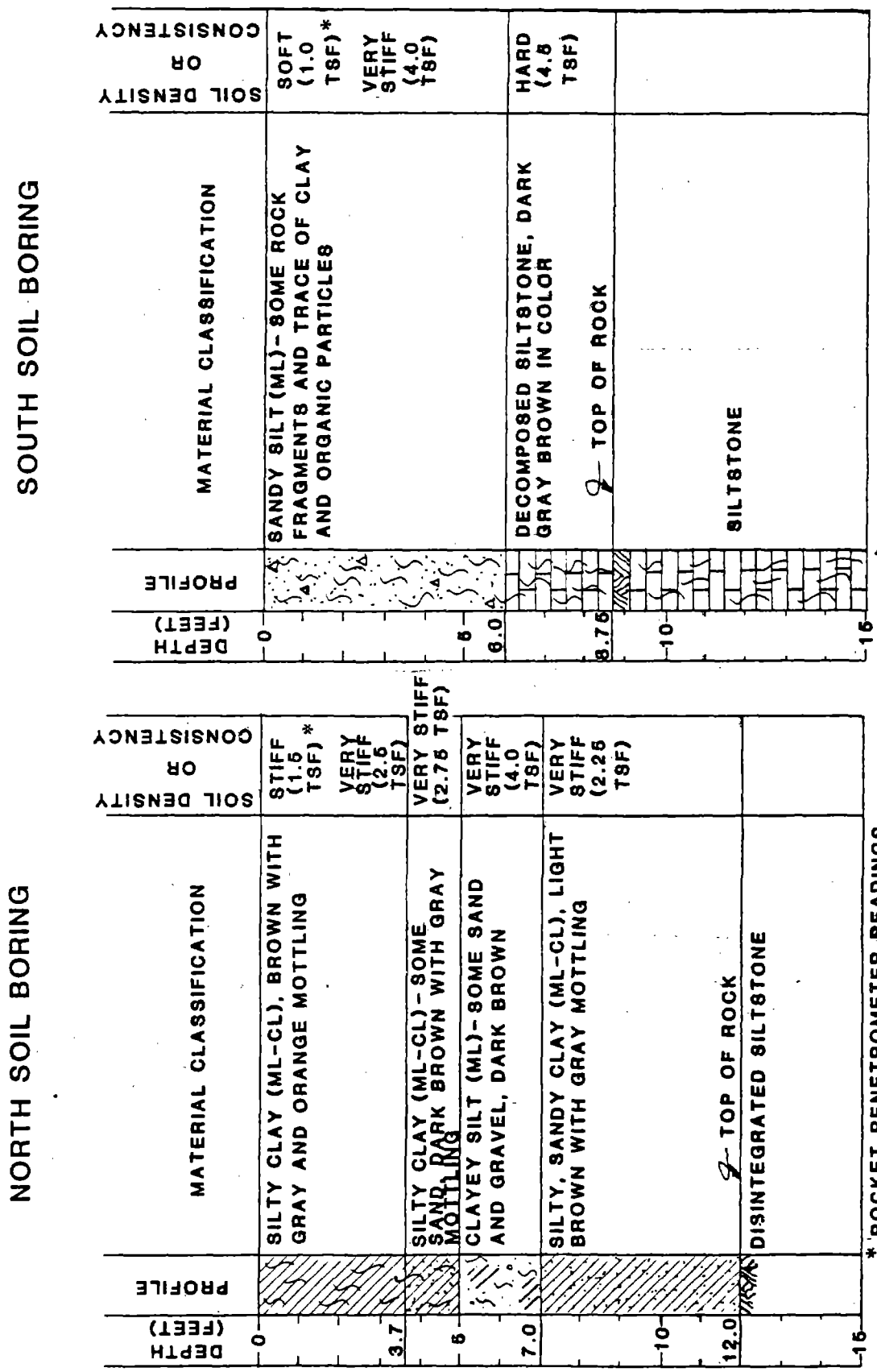


FIGURE A.58–SOIL BORINGS, LOWER KITTANNING LONGWALL SITE, PENNSYLVANIA, CASE HISTORY 15

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Available data do not permit correlating the occurrence of damage to homes with progress of the mining.

Homes that theoretically would have experienced ground strains of approximately equal magnitude and direction were not damaged to equal degrees.

All of the homes in this community are 1 or 1.5 story wood-frame structures with exterior brick facing or wood siding. All have basements. Damage common to each of the homes consists of moderate to severe cracking of basement cinderblock walls and concrete-slab floors, slight cracking of interior plaster walls and ceiling, and warped doorways and window frames.

Damage attributable specifically to compressive or tensile ground movements is difficult to differentiate and, when evident, seems not to be common to other homes that presumably experienced similar ground movements. One home close to the edge of the mine panel in a location principally associated with tensile ground movements suffered a bulging and badly cracked basement floor (Home H, Figure A.57) typical of a compressive ground movement. No other homes reported this type of basement damage. The roof of Home E, located in the center of the mine panel, buckled and leaked so severely that it had to be replaced. In no case was damage to exterior wood siding detected. Although exterior brick facing was usually cracked, only that of Home 5 was damaged severely. The owner of Home A reported that the chimney of his home moved differentially relative to the rest of the house, and cracks formed in the interior plaster wall immediately adjacent to the exterior position of the chimney. All garages within the community suffered moderate to severe cracking of the concrete-slab floors.

All of the water wells and springs in this community went dry as the longwall face approached or passed below. Some residents complained that methane escaped in large quantities from their dry wells. These wells had to be plugged. One homeowner deepened his dry 100-foot deep well to 200 feet. The new, deeper well produced water for another year until it too went dry. Home H was the last home in the community to be mined under and the last home to lose its well water. The mining company eventually financed a pipeline to this community from the water supply system of a nearby town.

Case History 16

Structures Damaged by Repeat Episodes of Longwall Mining

Longwall mining twice occurred below two homes at this site (Figure A.59). Damage caused by mining the deeper of two coal seams was relatively minor, while that caused by mining the shallower seam was severe.

The first episode of mining was carried out in the 800-foot deep Lower Kittanning Coal from late 1971 to early 1972. A thickness of approximately 48 inches of coal was removed. The second epsiode of mining was carried out from mid-1979 to early 1980 in the 700-foot deep Upper Kittanning coal. The approximate mined height of this seam was 40 inches.

Although the entire undermined area is hilly, both damaged homes were located on modest 6 degree slopes. Approximately 9.25 feet of soil overlies bedrock in the area of Home A and 4 feet near Home B. Both homes were founded on rock. Shallow borings indicate that the soil at both properties is sandy and clayey silt of residual or colluvial origin (Figures A.60).

Both homes are one-story, wood frame structures with basements. Both are clad with wood or aluminum siding. Neither sustained much damage from mining of the first longwall panel. The cinderblock basement wall of Home A bulged slightly inward and developed a few minor cracks. Door and window frames warped slightly. The water well servicing Home B went dry and a few hairline cracks appeared in the east and west basement walls.

The second episode of longwall mining took place in mine panels positioned almost directly over the longwall panels developed during the earlier, deeper mining (Figure A.59). Now, however, mining took place at shallower depths in strata already deformed from first mining.

Although less coal was removed during second mining, surface structures suffered much more severe damage. Horizontal cracks developed along the full lengths of the east and west walls of the basement in Home A. Those same walls bowed inwards approximately 3 to 4 inches; the cinderblocks in the northwest corner of the basement were severely cracked; and the east footing rose about 5 inches higher than the west footing, causing the entire structure to tilt noticeably. Since the interior of the wood-frame superstructure was paneled rather than plastered, damage was limited to severe warping of door and window frames. In addition, the sewer pipe leading away from the house broke and the water well dried up.

Major damage to Home B similarly consisted of severe cracking of basement walls, particularly those on the north and west sides of the house, which bulged slightly inward. One-half inch wide cracks developed in the concrete slab cellar floor. Cracks also formed in the plaster around doors and windows within the

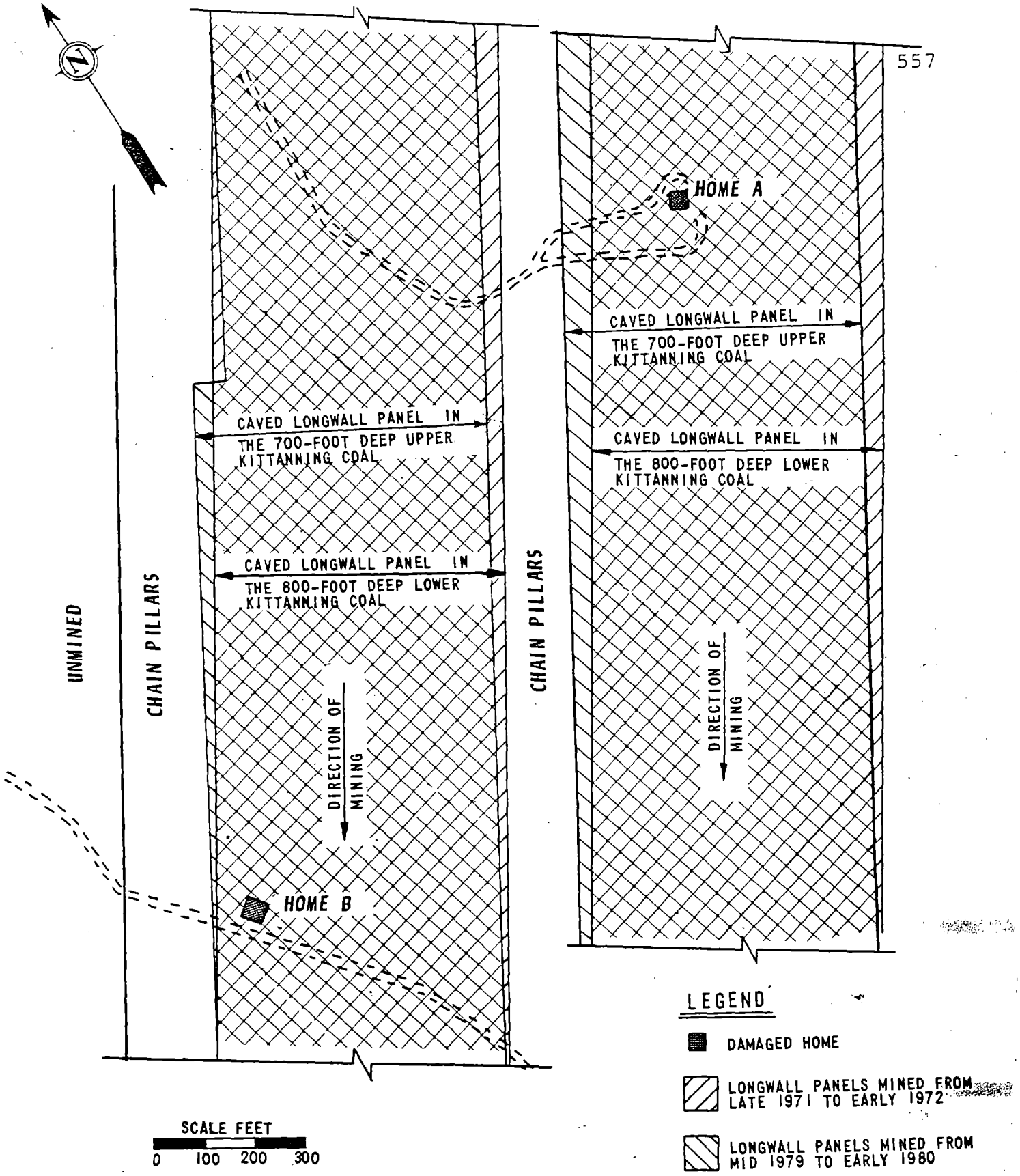
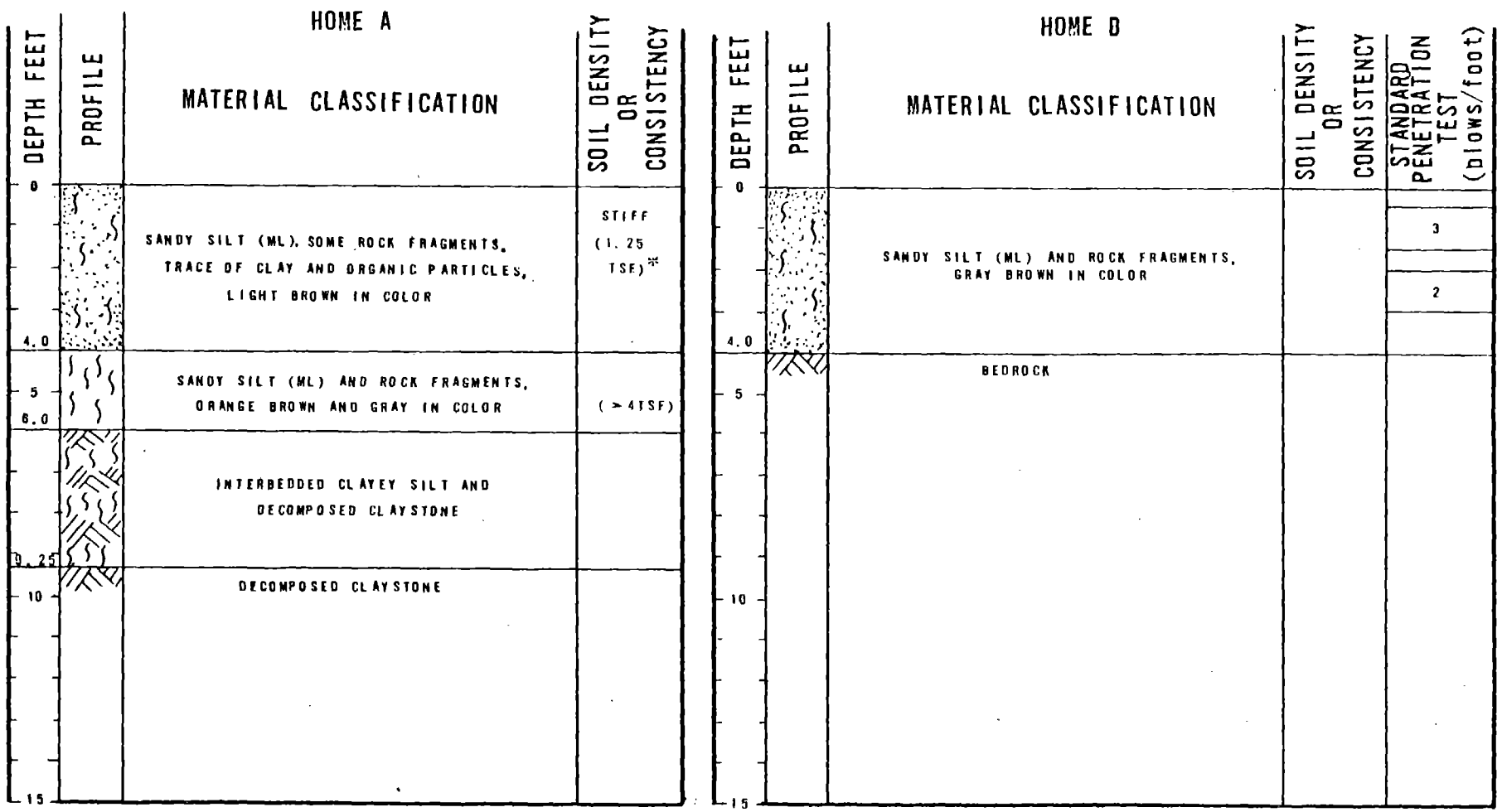


FIGURE A.59- LOCATIONS OF DAMAGED HOMES RELATIVE TO CAVED LONGWALL PANELS, CASE HISTORY 16



*Pocket penetrometer readings (tons/square foot)

FIGURE A.60 – SOIL BORINGS, CASE HISTORY 16

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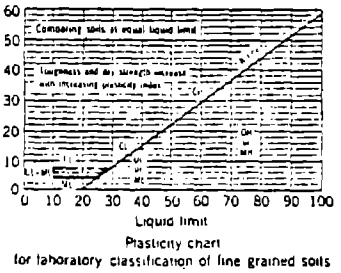
superstructure. Interior doors would not remain open due to a slight tilt of the entire home. While both homeowners reported that major damage to the homes occurred within a one or two day period, the owner of Home A noted continued ground movement for 3 to 4 months.

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Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)				Group Symbols	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils: More than half of material is larger than No. 200 sieve size[b]	Gravels: More than half of coarse fraction is larger than No. 4 sieve size (For visual classification, the 1/4 in. size may be used as equivalent to the No. 4 sieve size)	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel, maximum size, angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20%, hard, angular gravel particles 1/2-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3 Not meeting all gradation requirements for GW Atterberg limits below "A" line, or PI less than 4 — Above "A" line with PI between 4 and 7 are *borderline* cases requiring use of dual symbols Atterberg limits above "A" line, with PI greater than 7
			Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures		
			Plastic fines (for identification procedures, see *CL* below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures		
	Sands: More than half of coarse fraction is smaller than No. 4 sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well-graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3 Not meeting all gradation requirements for SW Atterberg limits below "A" line or PI less than 5 — Above "A" line with PI between 4 and 7 are *borderline* cases requiring use of dual symbols Atterberg limits below "A" line with PI greater than 7
			Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands, little or no fines		
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	SM	Silty sands, poorly graded sand-silt mixtures		
			Plastic fines (for identification procedures, see *CL* below)	SC	Clayey sands, poorly graded sand-clay mixtures		
Fine-grained soils: More than half of material is smaller than No. 200 sieve size (The No. 200 sieve size is about the smallest particle visible to naked eye)	Identification Procedures on Fraction Smaller than No. 40 Sieve Size						
		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)			
	Silts and clays: liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions Example: *Clayey silt*, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes; firm and dry in place; loess; (ML)
		Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		Slight to medium	Slow	Slight	OL	Organic silts and organic silty clays of low plasticity	
	Silts and clays: liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			Pt	Peat and other highly organic soils	

Use grain size curve in identifying the fractions as given under field identification

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows:
Less than 5%: GW, GP, SW, SP
More than 12%: GM, GC, SM, SC
5% to 12%: Borderline cases requiring use of dual symbols



Plasticity chart
for laboratory classification of fine grained soils

From Wagner 1957.

[a] *Boundary classifications*: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example *GW-GC*, well-graded gravel-sand mixture with clay binder.

[b] All sieve sizes on this chart are U.S. standard.

Field Identification Procedure for Fine Grained Soils or Fractions

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to shaking):
After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.
Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.
Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (Crushing characteristics):
After removing particles larger than No. 40 sieve size, mould a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.
High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (Consistency near plastic limit):
After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally losing its plasticity, and crumbles when the plastic limit is reached.
After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.
The toughness of the thread near the plastic limit and of the stiff lump when it finally crumbles, the more pronounced is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.
Highly organic clays have a very weak and spongy feel at the plastic limit.

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FIGURE A.61–UNIFIED SOIL CLASSIFICATION
(FROM LAMBE AND WHITMAN, 1969)

APPENDIX B

```
C*************************************************************************
C
C      USBM DEFINED FILE STRUCTURE AND VARIABLE DEFINITIONS
C
C*************************************************************************
C
C      THIS IS AN UNFORMATTED FIXED RECORD LENGTH FILE
C
C      128 WORDS PER RECORD DIVIDED AS FOLLOWS:
C
C      8 WORDS RESERVED FOR TEMPORARY INTEGER SORT KEYS
C      90 WORDS RESERVED FOR 90 INTEGER VARIABLES IN MAIN DATA BASE
C      30 WORDS RESERVED FOR 15 REAL VARIABLES IN MAIN DATA BASE
C
C      *** 8 WDS SORT KEYS *** 90 WDS INT.VAR. *** 30 WDS FOR 15 REAL VAR. ***
C
C*************************************************************************
C
C      LIST OF REAL-VARIABLES IN USE OF THE 15 AVAILABLE
C
C*************************************************************************
C
C
C     R(1)  CASE   - CASE NUMBER EXPECTED RANGE 10. TO 16000. LAST DIGIT
C                    (1=MSR,2=MSR WITH E PREFIX, 3=MSIF, 4=DUPLICATE MSR)
C                    IS FOR IDENTIFICATION OF DUPLICATES OTHERWISE RANGE
C                    WOULD BE 1-1600   F7.0
C     R(2)  REPL   - '81 REPLACEMENT VALUE   6 DIGITS   F8.0
C     R(3)  ESTMSR - '81 MSR ESTIMATE   6 DIGITS  F8.0
C     R(3)  SETMSR - '81 SETTLEMENT (MSR)   6 DIGITS   F8.0
C     R(4)  CONEST - '81 CONTRACTORS ESTIMATE   6 DIGITS   F8.0
C     R(5)  ESTSIF - '81 MSIF ESTIMATE (CONTRACTOR)  6 DIGITS  F8.0
C     R(7)  SETSIF - '81 SETTLEMENT (MSIF)   6 DIGITS  F8.0
C     R(8)  ROUGH  - '81 TOTAL REPAIR COST (BASEMENT, EXTERIOR AND INTERIOR
C                    DAMAGE REPAIR ESTIMATE FOR THE HOUSE PROPER)
C                    6 DIGITS   F8.0
C     R(9)  MISTAT - MINE STATUS 0=ACTIVE,1=ABANDONED 1 DIGIT   F3.0
C     R(10) BASREP - '81 BASEMENT REPAIR COST   6 DIGITS  F8.0
C     R(11) PFREP  - PERCENT BEARING OF BASEMENT BEARING WALLS REQUIRING
C                    REPLACEMENT   3 DIGITS   F5.0
C     R(12) PFRTG  - PERCENT OF PERIMETER BASEMENT WALLS REQUIRING
C                    REPOINTING AND SEALING  3 DIGITS  F5.0
C     R(13) PSREP  - PERCENT OF SLAB REQUIRING REPLACEMENT   3 DIGITS   F5.0
C     R(14) VTILT  - VISIBLE GENERAL TILT OF STRUCTURE (.GE.0.001 SPAN)
C                    (1=YES,0=NO)  F3.0
C     R(15) VDSTO  - VISIBLE STRUCTURAL DISTORTION OF SUPERSTRUCTURE
C                    (1=YES,0=NO)  F3.0
C
C-------------------------------------------------------------------------
C
C
C-------------------------------------------------------------------------
C
```

```
C*****************************************************************************
C
C       LIST OF THE INTEGER VARIABLES IN USE OF THE 90 AVAILABLE
C
C*****************************************************************************
C
C
C     M(1)  IQ      - QUADRANGLE (RANGE 1 TO 75)    I3
C     M(2)  ITFND   - TYPE OF FOUNDATION (0=UNKNOWN,1=BASEMENT,
C                     2=BI-LEVEL BASEMENT,3=AT GRADE ON FOOTINGS)     I2
C     M(3)  IEMB    - FOOTING EMBEDMENT IN TENTHS OF FEET(00 TO 80) I3
C                     NOTE SPECIAL CODE 01 MEANS EMBEDMENT IS UNKNOWN
C                     DATA IS ONLY FOR FOUNDATIONS OTHER THAN TYPE 3
C     M(4)  IMFND   - MATERIAL OF BASEMENT BEARING WALL (0=NONE,1=BLOCK,
C                     2=CONC.+BLOCK,3=STONE,4=STONE+OTHER,5=UNKNOWN)   I2
C     M(5)  NSTORY  - NUMBER OF STORIES IN TENTHS(0=0,10=1,15=ONE AND ONE-HALF,
C                     20=2,25=TWO AND ONE-HALF,30=3)  I3
C     M(6)  MPDIML  - MAXIMUM PLAN LENGTH OF SUPERSTRUCTURE (EXCLUDES DETACHED
C                     GARAGES AND PORCHES UNLESS ENCLOSED) IN FEET (0 TO 999)  I4
C     M(7)  MPDIMW  - SAME BUT WIDTH
C     M(8)  MFDIML  - MAXIMUM BASEMENT PLAN  LENGTH IN  FEET
C                    (0 TO 999) (APPLIES TO TYPE 1 AND 2 FOUNDATIONS) I4
C     M(9)  MFDIMW  - SAME BUT WIDTH
C
C     NOTE: M(10) THROUGH M(14) VALUES ARE DEFINED IN THE "1981 RESIDENTIAL
C           COST MANUAL" BY COERT ENGLESMAN, PUBLISHED BY VAN NOSTRAND
C           REINHOLD COMPANY
C
C     M(10) ISHAP   - SHAPE FACTOR (00 TO 70, 0=RECT,20=ELL,35=SEMIIRREG,
C                     AND 70=IRREGULAR)  I3
C     M(11) LIV     - LIVING AREA IN SQUARE FEET     I5
C     M(12) LIVF    - LIVING AREA FACTOR ( +2 TO -6 REPRESENTING +.02 TO -.06,
C                     FOR REPLACEMENT VALUE CALCULATIONS)
C                     1 DIGIT INTEGER (WITH SIGN IF NEGATIVE)    I3
C     M(13) ICMF    - CONSTRUCTION METHOD FACTOR    (2 DIGITS AND SIGN,
C                     FOR REPLACEMENT VALUE CALCULATIONS)    I4
C     M(14) ICQF    - CONSTRUCTION QUALITY FACTOR (900 TO 2500,
C                     FOR REPLACEMENT VALUE CALCULATIONS)  I5
C     M(15) IFACE   - FACING TYPE (11=BRICK, 12=STONE, 13=BLOCK, 21=WOOD,
C                     22=ALUMINUM SIDING, 23=INSULBRICK OR ASPHALT SHINGLES,
C                     34=COMBINATION RIGID AND FLEXIBLE)  I3
C                     NOTE:   10 SERIES IS RIGID, 20 SERIES IS FLEXIBLE
C     M(16) IGARAG  - GARAGE TYPE (0=UNKNOWN,1=INTEGRAL,2=ATTACHED,
C                     3=DETACHED,4=UNKNOWN)     I2
C
C    DAMAGED COMPONENT VARIABLES  - ALL ARE 0=NO DAMAGE, 1=DAMAGE   I2
C
C     M(17) IDGEN     - GENERAL UNSPECIFIED
C     M(18) IDBAS(1)  - BASEMENT UNSPECIFIED
C     M(19) IDBAS(2)  - BASEMENT WALL
C     M(20) IDBAS(3)  - BASEMENT DOOR
C     M(21) IDBAS(4)  - BASEMENT WINDOW
C     M(22) IDBAS(5)  - BASEMENT FLOOR
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C      M(23) IDEXT(1) - EXTERIOR UNSPECIFIED
C      M(24) IDEXT(2) - EXTERIOR WALL
C      M(25) IDEXT(3) - EXTERIOR DOOR
C      M(26) IDEXT(4) - EXTERIOR WINDOW
C      M(27) IDEXT(5) - EXTERIOR PAVEMENT
C      M(28) IDINT(1) - INTERIOR UNSPECIFIED
C      M(29) IDINT(2) - INTERIOR WALL
C      M(30) IDINT(3) - INTERIOR DOOR
C      M(31) IDINT(4) -  BLANK
C      M(32) IDINT(5) - INTERIOR CEILING
C      M(33) IDINT(6) - INTERIOR FLOOR
C      M(34) IDROOF   - ROOF
C      M(35) IDCHIM   - CHIMNEY AND FIREPLACE
C      M(36) IDGAR(1) - GARAGE UNSPECIFIED
C      M(37) IDGAR(2) - GARAGE WALL
C      M(38) IDGAR(3) - GARAGE DOOR
C      M(39) IDGAR(4) - GARAGE WINDOW
C      M(40) IDGAR(5) - GARAGE FLOOR
C      M(41) IDUTIL   - UTILITIES
C
C      BASEMENT CRACK DIMENSION PARAMETERS
C
C      FOR EACH OF TWO LOCATIONS (SLAB AND BEARING WALLS)
C      THE FOLLOWING 6 DIMENSIONS ARE IDENTIFIED
C      A) SURFACE AREA IN SQUARE FEET   I5
C      B) SUM OF CRACK LENGTHS IN FEET  I4
C      C) MEAN CRACK LENGTH IN FEET   I4
C      D) MEAN CRACK WIDTH IN HUNDREDTHS OF AN INCH   I4
C      E) SUM OF CRACK WIDTHS, WHERE THE WIDTHS ARE PARALLEL TO THE
C         LENGTHS OF WALLS OR SLABS.  FOR WALLS, THE VALUES ARE
C         AVERAGED FOR PARALLEL WALLS OF THE PLAN DIMENSION IN
C         WHICH THE GREATEST EXTENSION OCCURRED IN HUNDREDTHS
C         OF AN INCH.    I4
C      F) SAME AS E) ABOVE BUT FOR CRACKS ROTATED 90 DEGREES
C         WITHIN THE SLAB OR WALL PLANES.  I4
C      NOTE: ZERO WILL BE USED FOR CRACK DATA IF THE AMOUNT OF CRACKING
C            IS ZERO AND GENERALLY IF THE PATTERN IS UNKNOWN.
C
C      M(42) ICRKS(1) - SLAB CRACK DIMENSION A
C        43        2                        B
C        44        3                        C
C        45        4                        D
C        46        5                        E
C        47        6                        F
C      M(48) ICRKW(1) - WALL CRACK DIMENSION A
C        49        2                        B
C        50        3                        C
C        51        4                        D
C        52        5                        E
C        53        6                        F
C      M(54)  -  '81 INTERIOR REPAIR COST   I5
C        55   -  SEVERITY INDEX PREDICTED
C        56   -  CONCRETE SLAB PERCENT OF PLAN AREA  I4
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C        57   -  SLAB CRACK ORIENTATION (0=NONE, 1=RANDOM, 2=ALIGNED,
C                3=GRID, 4=UNKNOWN)  I4
C        58   -  WALL CRACK ORIENTATION (0=NONE, 1=VERTICAL, 2=LATERAL
C                3=DIAGONAL, 4=L&D, 5=V&D, 6=L&V, 7=ALL, 8=UNKNOWN)  I4
C        59   -  FLOOR SLAB HEAVED OR SAGGED (1=YES, 0=NO)  I4
C        60   -  BEARING WALL BUCKLING (1=YES, 0=NO)  I5
C        61   -  BEARING WALL PORTION OF TOTAL BASEMENT REPLACEMENT
C                COST LESS EXCAVATION 'A'  I4
C        62   -  BASEMENT PERIMETER IN FEET  I4
C        63   -  BASEMENT BEARING WALL LENGTH IN FEET  I4
C        64   -  '81 EXTERIOR REPAIR COST    I4
C        65   -  SITE VISITED TO CHECK STRUCTURE CHARACTERIZATION
C                (0=NO,1=YES)  I4
C
C
C     OTHER VARIABLES
C
C     M(66) ISEVER   -  SEVERITY INDEX (0 TO 500 REPRESENTING
C                       0.00 TO 5.00) MAY INCLUDE ROUGH QUALITATIVE
C                       ESTIMATES WHICH WILL BE IDENTIFIED BY A ZERO
C                       BASEMENT REPAIR COST.   I4
C     M(67) IDVIS    - VISUAL DAMAGE DESCRIPTION (PHOTO,SKETCH) 0=NO,1=YES I2
C     M(68) IDVER    - VERBAL DAMAGE DESCRIPTION 0=NO,1=YES   I2
C     M(69) ITOVR    - TOTAL OVERBURDEN IN FT    I4
C     M(70) IROVR    - ROCK OVERBURDEN IN FT  I4
C     M(71) IHARD    - HARD ROCK OVERBURDEN IN FT  I4
C     M(72) ISOIL    - SOIL TYPE (1=RESIDUAL/COLLUVIAL,2=ALLUVIAL,
C                      3=TERRACE)   I2
C     M(73) IMBL     - MINING BELOW TYPE (0=NONE, 1=RETREAT-COMPLETE
C                      EXTRACTION,2=GENERAL ACTIVE MINING,3=RETREAT-
C                      60% TO 90% MINED,4=RETREAT-PILLARS SPLIT/SLICED
C                      <60% MINED,5=BARRIER PILLAR,6=NO RETREAT FULL
C                      PILLARS REMAIN,7=LONGWALL,8=UNKNOWN)   I2
C     M(74) IMAD     - MINING ADJACENT TYPE (SAME TYPES AS ABOVE)  I2
C     M(75) IANG     - ANGLE OF STRUCTURE RELATIVE TO ADJACENT MINING TYPE,
C                      IF APPLICABLE, IN 10THS OF DEGREES    I4
C     M(76) IDDAT(1) - MONTH OF DAMAGE    I3
C     M(77) IDDAT(2) - DAY OF DAMAGE    I2
C     M(78) IDDAT(3) - YEAR OF DAMAGE    I2
C     M(79) IMDAT(1) - MONTH OF MINING   I3
C     M(80) IMDAT(2) - DAY OF MINING  I2
C     M(81) IMDAT(3) - YEAR OF MINING  I2
C     M(82) IBOR     - BORING LOGS AVAILABLE (0=NO,1=YES)  I2
C     M(83) ISEAM    - COAL SEAM (1=PITTSBURGH,2=UPPER FREEPORT,3=LOWER
C                      FREEPORT,4=LOWER KITTANNING,5=LOWER KITTANNING(CLAY),
C                      6=UPPER KITTANNING)    I2
C     M(84) IAGE     - STRUCTURE AGE IN YEARS     I3
C     M(85) ITHSM    - MINED HEIGHT IN 10THS OF FEET    I3
C     M(86) IMIMD    - MINE MAP AVAILABILITY, 0=YES, 1=NO,  I2
C     M(87) IGRAD    - EMBEDMENT SLOPE CONFIGURATION (LEVEL=0, LENGTH TO LENGTH
C                      (TYPICALLY FRONT TO BACK)=1, WIDTH TO WIDTH=2,
C                      DIAGONAL=3, UNKNOWN=4)  I2
C     M(88) IGRD     - GENERAL TOPOGRAPHY (0=UNKNOWN,1=FLAT,2=VALLEY BOTTOM,
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                    3=SLOPE TOE,5=TERRACE,6=SLOPE CREST,7=RIDGE,
                    8=PLATEAU,11=SLOPE GENTLE 5-10%,12=SLOPE MODERATE
                    10-20%, 13=SLOPE SEVERE >20%)  I3
M(89) ISMDP     - ABANDONED COAL SEAM (1=YES,0=NO)      I3
M(90) IDSDP     - ABANDONED MINE HEIGHT IN TENTHS OF FEET  I3
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APPENDIX C

DESCRIPTION OF STRUCTURE

FILE & SOURCE	QUAD	STORIES	FACING	SHAPE	QUALITY	BASEMENT AGE	LIVING AREA	FOUNDATION GRADE	FOUNDATION TYPE	BASEMENT TYPE	BASEMENT EXTERIOR	BASEMENT LENGTH	BASEMENT WIDTH	SUPER LENGTH	SUPER WIDTH
1821.	AMITY	10	21	0	1299	63	1337	4	1	1	60	33	27	33	27
2541.	BARNESBO	20	21	20	1682	40	1663	0	1	3	50	28	28	28	28
4411.	BARNESBO	20	22	10	1682	61	1950	3	1	1	50	30	24	54	28
4831.	BARNESBO	10	23	0	1815	48	2249	3	1	3	50	40	28	54	28
2741.	BRADDOCK	10	11	10	1475	19	2132	1	2	1	5	39	30	39	30
3541.	BRADDOCK	10	11	10	1475	20	2441	1	2	1	15	44	27	44	27
4301.	BRADDOCK	10	11	20	1630	21	2085	1	1	1	40	42	32	46	42
4331.	BRADDOCK	20	11	0	1520	21	2516	2	1	1	50	33	25	45	25
1621.	BRAD R P	15	11	0	1625	19	2003	1	1	1	45	35	25	35	25
1631.	BRAD R P	10	11	0	1630	20	1731	1	1	1	40	38	25	38	25
1641.	BRAD R P	15	11	0	1475	20	1907	1	2	1	35	35	26	35	26
1661.	BRAD R P	10	11	20	1630	20	2331	1	1	1	35	38	38	38	38
1691.	BRAD R P	20	11	0	1520	22	2527	2	1	1	65	33	25	45	25
1711.	BRAD R P	10	11	0	1630	19	2102	1	1	1	40	42	26	42	26
1731.	BRAD R P	15	11	0	1475	20	1907	1	2	1	35	35	26	35	26
1751.	BRAD R P	20	11	0	1520	16	2320	1	1	1	35	32	25	32	25
1901.	BRAD R P	10	11	0	1630	0	1574	1	1	1	35	35	25	35	25
1911.	BRAD R P	10	11	0	1630	16	1741	1	1	1	45	38	25	38	25
2201.	BRAD R P	10	34	0	1590	16	1776	1	1	1	30	40	25	40	25
2261.	BRAD R P	10	11	0	1630	17	1831	1	1	1	35	38	25	38	25
2271.	BRAD R P	10	11	0	1630	16	1807	1	1	1	30	40	25	40	25
2471.	BRAD R P	10	34	0	1438	21	1761	1	2	1	30	35	27	35	27
5481.	BRAD R P	10	11	0	1630	19	2316	1	1	1	20	40	25	40	25
171.	BRIDGEVI	10	13	0	1630	26	2402	1	1	1	40	46	31	46	31
301.	BRIDGEVI	10	12	70	1868	33	2175	1	1	1	60	51	28	51	28
351.	BRIDGEVI	15	21	20	1798	34	2545	1	1	1	40	54	30	54	30
3751.	BRIDGEVI	20	11	20	1728	34	2385	1	1	1	60	46	25	46	25
4021.	BRIDGEVI	10	34	20	2350	16	5444	2	1	1	35	58	46	82	58
1521.	CARM CEY	10	21	0	1815	43	1568	3	1	1	50	38	29	38	29
4471.	BRIDGEVI	10	11	20	1868	34	2190	1	1	1	30	48	28	48	28
3661.	BRID N S	10	34	35	1590	17	3471	4	1	1	65	60	54	60	54
3671.	BRID N S	10	34	0	1590	17	2510	1	1	1	30	50	30	50	30
3681.	BRID N S	10	11	35	1630	17	2742	2	3	0	0	0	0	80	30
3691.	BRID N S	20	34	20	1485	18	2250	1	1	1	55	42	25	42	25
3701.	BRID N S	10	34	20	1438	18	4423	1	2	1	50	50	46	50	46
4731.	BRID N S	20	34	20	1485	19	3033	2	1	1	70	38	30	62	30
611.	CALIFORN	10	11	20	1630	26	1349	4	1	1	70	38	25	38	25
131.	CALI W C	10	11	70	1630	20	2218	2	3	0	0	0	0	52	40
181.	CALI W C	15	23	20	1798	49	1860	0	1	5	35	26	24	26	24
271.	CALI W C	10	13	20	1630	22	1623	2	1	1	60	30	26	56	34
291.	CALI W C	10	22	20	1550	21	2483	1	1	1	50	48	42	48	42
311.	CALI W C	10	22	70	1815	37	1372	3	1	1	65	40	36	40	36
371.	CARMICHA	15	21	0	1682	50	1109	3	1	1	50	25	22	25	22
951.	CARMICHA	10	21	0	1815	34	1118	3	1	1	50	30	24	30	24
961.	CARMICHA	10	23	20	1815	53	1505	4	1	5	1	48	26	48	26

DESCRIPTION OF STRUCTURE

FILE & SOURCE	QUAD	STORIES	FACING	SHAPE	QUALITY	BASEMENT AGE	BASEMENT LIVABLE AREA	BASEMENT GRADE	BASEMENT FDTN TYPE	BASEMENT MATERIAL	BASEMENT EXPOSED LENGTH	BASEMENT LENGTH	SUPER LENGTH	SUPER WIDTH
1261.	CARMICHA	15	23	20	1550	23	1727	3	1	1	65	34	26	34	26
4751.	CARMICHA	15	21	20	1798	105	1394	3	1	4	50	30	30	30	30
411.	CARM CEY	15	23	0	1798	41	1324	3	1	1	45	28	26	28	26
421.	CARM CEY	10	21	0	1815	27	1065	0	1	1	50	32	22	32	22
691.	CARM CEY	10	12	35	1630	0	1809	3	1	1	45	38	26	38	26
701.	CARM CEY	10	21	0	1550	0	1852	1	1	1	40	46	25	46	25
711.	CARM CEY	10	21	0	1550	0	1336	3	1	1	50	34	25	34	25
721.	CARM CEY	10	21	20	1550	0	1351	3	1	1	55	28	23	28	23
731.	CARM CEY	15	21	20	1550	0	2067	2	1	1	50	36	26	48	26
781.	CARM CEY	10	21	10	1550	0	1280	3	1	1	45	32	26	32	26
3941.	CARM CUM	10	21	0	1550	30	1483	4	1	1	50	38	29	38	29
3951.	CARM CUM	15	23	20	1550	0	1846	3	1	1	50	39	24	39	24
4001.	CARM CUM	10	11	20	1630	0	1729	2	1	1	55	40	36	52	36
4141.	CARM CUM	15	21	0	1682	32	2511	3	1	1	60	30	28	30	28
5161.	CARM CUM	15	21	20	1682	42	1499	3	1	1	50	34	26	34	26
5621.	CARM CUM	10	11	0	1329	37	1964	0	1	1	45	50	25	50	25
5871.	CARM CUM	10	11	0	1329	36	1964	0	1	1	45	50	25	50	25
5881.	CARM CUM	10	11	0	1329	36	1964	0	1	1	45	50	25	50	25
6131.	CARM CUM	10	11	0	1329	36	1964	0	1	1	45	50	25	50	25
461.	CARM FAI	10	21	0	1550	25	1840	4	1	1	60	40	30	40	30
581.	CARM FAI	10	11	0	1550	0	1737	1	1	1	35	38	30	38	30
641.	CARM FAI	10	34	70	1842	28	1413	2	3	0	0	0	0	68	63
741.	CARM FAI	15	11	70	1625	0	2321	2	1	1	50	34	31	65	31
801.	CARM FAI	10	11	20	1630	0	2121	2	1	1	45	38	38	60	38
941.	CARM FAI	15	21	0	1798	52	1370	0	1	1	50	33	24	33	24
1561.	CARM FAI	10	21	0	1815	41	1693	3	1	1	45	40	26	40	26
221.	COLV B C	20	22	35	1682	74	1809	3	1	1	50	30	16	43	30
4431.	COLV B C	20	21	0	1682	107	2641	3	1	3	45	30	26	30	26
2061.	COLV B C	20	21	20	1682	41	1803	3	1	1	55	40	24	40	24
911.	CURTISVI	10	11	20	1630	20	2701	2	1	1	50	43	41	53	41
3731.	CURTISVI	20	11	20	1728	107	1733	4	1	3	30	34	18	34	18
391.	DONO W N	0	0	0	900	24	1170	4	1	1	1	45	26	45	26
841.	DONO W N	20	11	20	1728	162	4272	3	1	3	50	45	23	45	45
5531.	DONO W N	10	22	0	1815	51	1685	0	1	1	35	28	25	40	28
6411.	DONO W N	10	21	0	1815	53	1491	0	1	1	30	40	24	40	24
1281.	EBENSBUR	10	21	0	1815	0	1056	4	1	5	1	32	22	32	22
1311.	EBENSBUR	10	21	35	1550	24	1822	4	1	1	1	36	36	36	36
1961.	EBENSBUR	20	12	20	1800	72	2925	4	1	3	1	36	36	36	36
3621.	EBENSBUR	10	21	20	1550	19	2165	2	1	1	1	46	28	77	39
3631.	EBENSBUR	20	11	0	1728	29	2421	3	1	1	1	34	25	34	25
2731.	EBENSBUR	20	21	0	1682	41	1717	4	1	2	1	26	24	26	24
71.	EBENSB N	10	11	0	1630	15	2600	2	1	1	1	40	40	40	40
671.	EBENSB N	20	11	0	2150	0	7181	1	1	1	1	90	30	90	30
1101.	EBENSB S	10	11	0	1630	25	2606	1	1	1	1	65	25	65	25
1001.	EBENSB S	10	21	0	1798	29	2017	3	1	1	1	38	26	38	26

NOT REPRODUCIBLE

DESCRIPTION OF STRUCTURE

FILE & SOURCE	QUAD	STORIES	FACING	SHAPE	QUALITY	BASEMENT AREA	LIVING AREA	GARAGE	FOUNDATION	BASEMENT	BASEMENT EMBED	BASEMENT LENGTH	BASEMENT WIDTH	SUPER LENGTH	SUPER WIDTH
341.	GARARD F	15	21	0	1798	29	2252	4	1	1	1	31	30	31	30
231.	ELLSWORT	20	12	0	1728	119	4404	4	1	3	1	38	34	38	34
11.	GARARD F	15	23	0	1275	41	1535	4	1	1	25	30	24	30	24
601.	JOHNSTOW	15	21	20	1798	37	2959	1	1	4	35	48	32	48	32
2341.	HACKETT	15	21	70	1798	34	5064	3	1	1	60	99	32	99	32
751.	HACKETT	15	11	20	1848	138	3114	3	1	3	70	45	39	45	39
851.	HACKETT	10	23	20	1275	50	1288	0	1	4	40	30	30	30	30
821.	HACK BEN	20	22	20	1682	102	2259	3	1	4	40	38	25	38	25
831.	HACK BEN	10	21	20	1182	47	1015	0	1	1	35	27	27	27	27
1421.	HACK KAM	15	11	20	1625	20	2524	2	1	1	35	50	32	35	32
1431.	HACK KAM	15	23	20	1798	47	2523	3	1	1	70	38	32	38	32
1441.	HACK KAM	10	11	20	1868	30	2665	2	1	1	70	53	35	80	35
1451.	HACK KAM	10	22	20	1550	16	3036	1	1	1	35	65	34	65	34
3921.	HACK KAM	10	23	0	1815	42	1649	1	1	1	20	53	22	53	22
4941.	HACK KAM	10	11	0	1630	23	1585	0	1	1	60	35	28	35	28
1221.	HACK KAM	10	11	20	1630	25	2328	2	1	1	55	40	40	65	40
811.	NANTY GL	10	21	0	1815	30	1902	4	1	1	1	42	28	42	28
771.	NANTY GL	10	11	0	1630	24	3042	1	1	1	65	76	26	76	26
121.	NANTY GL	20	22	20	1678	55	2237	4	1	3	1	29	26	29	26
2351.	NANT G V	10	21	35	1798	33	2127	2	1	1	55	34	30	54	30
2561.	NANT G V	10	21	70	1650	0	2399	1	2	5	55	46	41	67	46
3741.	NANT G V	10	21	0	1798	25	2397	0	1	1	50	38	30	38	30
3781.	NANT G V	10	21	10	1815	31	1927	2	1	1	60	39	27	55	27
3791.	NANT G V	10	21	10	1550	23	1934	2	1	1	45	41	24	65	24
3801.	NANT G V	10	11	20	1630	0	1885	2	3	0	0	0	0	64	39
3831.	NANT G V	10	11	20	1630	22	2248	3	1	1	60	45	37	65	37
3891.	NANT G V	10	11	35	1630	16	1889	3	1	1	55	44	26	44	26
3571.	NANT G V	15	21	35	1400	26	2309	1	2	1	40	43	32	56	32
141.	N KENS E	10	13	0	1868	41	1268	3	1	1	50	28	28	28	28
151.	N KENS E	10	13	0	1815	39	1238	3	1	1	35	30	26	30	26
631.	N KE W R	10	21	0	1550	16	1747	3	1	1	1	31	28	31	28
3013.	SMITHTON	10	11	0	2350	0	4001	1	1	1	60	73	29	73	29
331.	VALENCIA	10	11	20	1630	24	2151	3	1	1	50	40	36	40	36
501.	VALENCIA	25	21	0	1682	79	3803	3	1	3	50	38	32	38	32
871.	WASH E R	10	34	0	1590	24	1832	1	1	1	35	48	24	48	24
881.	WASH E R	10	11	20	1630	25	1260	3	1	1	50	32	27	32	27
921.	WASH E R	20	22	0	1400	22	2451	1	3	0	0	0	0	42	25
1211.	WASH E R	10	11	35	1630	21	1900	2	1	1	70	38	28	50	28
241.	WAYNESBU	10	21	0	1815	54	1239	3	1	5	1	33	22	33	22
321.	WAYNESBU	20	21	0	1682	81	3276	3	1	3	50	36	32	36	32
1471.	WAYNESBU	20	21	20	1682	87	2374	3	1	3	35	25	25	40	25
1601.	WAYNESBU	10	21	20	1815	52	1595	2	1	1	50	35	35	49	35
3991.	CARM CUM	20	23	35	1682	58	1728	4	1	3	1	32	30	32	30
2431.	BRIDGEVI	15	11	20	1798	31	3300	3	1	1	1	40	30	75	30

DESCRIPTION OF DAMAGE

FILE & SOURCE	SEVERITY	DAMAGE DESCRIPTION: VISUAL	VERB	BASEMENT: GENERAL	BASEMENT: UNSPECIFIED	BASEMENT: SEEPAGE	BASEMENT: CRACK	BASEMENT: DOOR	BASEMENT: WINDOW	BASEMENT: FLOOR	EXTERIOR: UNSPECIFIED	EXTERIOR: SEEPAGE	EXTERIOR: CRACK	EXTERIOR: DOOR	EXTERIOR: WINDOW	EXTERIOR: FLOOR	INTERIOR: UNSPECIFIED	INTERIOR: SEEPAGE	INTERIOR: CRACK	INTERIOR: DOOR	INTERIOR: WINDOW	INTERIOR: FLOOR	GARAGE: CHIMNEY	GARAGE: UNSPECIFIED	GARAGE: SEEPAGE	GARAGE: CRACK	GARAGE: DOOR	GARAGE: WINDOW	GARAGE: FLOOR	GARAGE: UTILITY
1821.	0	1	1	0	0	1	0	0	1	0	0	1	1	0	1	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
2541.	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4411.	366	1	1	0	0	1	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4831.	0	1	1	0	0	1	0	0	1	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
2741.	0	1	0	0	0	1	0	0	1	0	1	0	0	0	0	1	1	0	1	0	0	0	1	1	0	1	0	0	0	0
3541.	0	1	0	0	0	1	1	0	1	0	1	1	0	1	0	1	1	1	0	1	1	0	0	0	0	1	1	0	0	0
4301.	124	1	1	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	0
4331.	0	1	1	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0
1621.	168	1	1	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	0
1631.	0	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0
1641.	0	1	1	0	0	1	1	1	1	0	1	1	1	1	0	1	1	1	1	0	0	0	1	1	1	1	0	0	0	0
1661.	0	1	1	0	1	1	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1	0	0
1691.	0	1	0	0	0	1	1	1	0	0	1	0	1	0	0	0	1	0	0	0	1	0	1	1	0	1	0	0	0	0
1711.	0	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
1731.	0	1	1	0	0	0	0	0	1	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
1751.	0	1	1	0	0	1	0	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	1	0	0	0	1	0	0
1901.	0	1	0	0	0	1	0	1	1	0	1	0	1	0	1	0	0	0	0	0	0	0	0	1	0	0	1	0	0	0
1911.	0	1	1	0	0	1	0	1	0	0	1	0	1	0	1	0	0	0	0	0	0	0	0	1	0	0	1	1	0	0
2201.	0	0	1	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2261.	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2271.	0	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2471.	0	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	0	0	0
5481.	0	1	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	0	0	0
171.	172	1	1	0	0	1	0	0	1	0	1	0	0	0	0	1	0	1	0	0	0	0	1	0	0	1	0	0	0	0
301.	179	1	1	0	0	1	0	0	1	0	1	0	0	1	0	1	0	1	0	0	0	0	1	0	0	1	0	0	0	0
351.	0	0	1	0	0	1	1	1	0	0	0	1	0	0	0	1	1	1	1	0	0	0	1	1	0	0	0	0	0	0
3751.	0	0	1	0	0	1	0	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4021.	0	1	1	0	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1521.	236	1	0	0	0	1	0	0	0	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
4471.	53	1	1	0	0	1	0	1	0	0	0	0	1	0	0	1	0	1	0	0	1	0	1	0	0	0	0	0	0	0
3661.	500	0	1	0	0	1	1	0	1	0	1	1	0	1	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0
3671.	500	0	1	0	0	1	1	0	1	0	1	1	1	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	1	0
3681.	0	0	1	0	0	0	0	0	0	0	1	1	0	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
3691.	500	0	1	0	0	1	1	0	1	0	1	1	0	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
3701.	500	0	1	0	0	1	1	0	1	0	1	1	0	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
4731.	500	1	0	0	0	1	1	0	1	0	1	1	0	1	1	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0
611.	435	1	0	0	0	1	1	0	1	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
131.	0	1	1	0	0	0	0	0	0	0	1	0	0	1	0	1	0	1	1	0	0	0	0	0	0	0	0	0	0	0
181.	0	1	1	0	0	0	1	1	0	0	0	0	0	0	1	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0
271.	50	1	1	0	0	1	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
291.	198	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	0	0	0
311.	0	1	1	0	0	1	0	0	0	0	0	0	0	1	0	0	0	1	0	0	1	0	0	0	0	0	1	0	0	0
371.	0	0	1	0	0	1	0	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
951.	272	1	1	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

DESCRIPTION OF DAMAGE

FILE & SOURCE	SEVERITY	DAMAGE DESCRIPTION: VISUAL	DAMAGE DESCRIPTION: VERBAL	BASEMENT: GENERAL	BASEMENT: UNSPECIFIED	BASEMENT: SWELLING	BASEMENT: DOOR	BASEMENT: WINDOW	BASEMENT: FLOOR	EXTERIOR: UNSPECIFIED	EXTERIOR: SWELLING	EXTERIOR: DOOR	EXTERIOR: WINDOW	EXTERIOR: FLOOR	INTERIOR: UNSPECIFIED	INTERIOR: SWELLING	INTERIOR: DOOR	INTERIOR: CEILING	INTERIOR: FLOOR	INTERIOR: ROOF	GARAGE: CHIMNEY	GARAGE: UNSPECIFIED	GARAGE: SWELLING	GARAGE: DOOR	GARAGE: WINDOW	GARAGE: FLOOR	GARAGE: UTILITY
961.	241	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1261.	164	1	1	0	0	1	1	0	1	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0
4751.	259	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
411.	100	1	1	0	0	1	0	0	1	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
421.	225	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	1	0	0	0	1	0	0	0	0	0	0
691.	0	1	1	0	0	1	0	0	1	0	1	0	0	1	0	1	1	1	0	0	0	0	0	0	0	0	0
701.	0	1	1	0	0	1	0	0	0	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	1	0
711.	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
721.	153	1	1	0	0	1	0	0	0	0	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0
731.	0	1	0	0	0	1	0	0	0	0	0	0	0	1	0	1	0	1	0	0	0	0	0	0	0	0	0
781.	100	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
3941.	282	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3951.	0	1	1	0	0	1	0	0	1	0	0	0	0	1	0	1	0	1	0	0	0	0	0	0	0	0	1
4001.	0	1	1	0	0	1	0	0	1	0	0	0	0	1	0	1	0	1	0	0	0	0	1	0	0	0	0
4141.	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0	0	0	0	0
5161.	0	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5621.	255	1	1	0	0	1	1	0	1	0	1	1	1	1	0	1	0	1	0	0	0	0	0	0	0	0	0
5871.	67	1	0	0	0	1	0	0	1	0	0	1	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0
5881.	33	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
6131.	273	1	1	0	0	1	1	0	1	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0
461.	0	1	0	0	0	1	0	0	1	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0
581.	394	1	1	0	0	1	0	0	1	0	1	0	0	1	1	0	0	0	0	1	0	0	1	0	0	1	0
641.	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	0	1	0	0	1	0
741.	196	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
801.	0	1	0	0	0	1	0	0	0	0	1	0	1	0	0	1	0	1	0	0	0	0	0	0	0	0	0
941.	0	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
1561.	0	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
221.	0	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4431.	166	1	1	0	0	1	0	0	0	0	0	1	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0
2061.	0	0	1	0	0	1	0	0	1	0	0	0	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0
911.	45	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
3731.	0	1	0	0	0	1	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
391.	300	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
841.	0	1	1	0	0	1	0	0	1	0	0	0	0	0	1	0	0	0	0	0	0	0	0	1	0	0	0
5531.	0	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	0	0
6411.	0	1	1	0	0	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0
1281.	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1311.	0	1	1	0	0	1	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
1961.	0	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3621.	0	1	1	0	0	1	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
3631.	195	1	0	0	0	1	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0	0	0	0	0	0
2731.	0	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
71.	0	1	1	0	0	1	0	0	1	0	1	1	1	0	0	1	1	1	0	0	0	0	0	0	0	0	0
671.	0	1	1	0	0	1	0	0	1	0	0	1	0	0	0	1	0	1	1	0	0	0	0	0	0	0	0

DESCRIPTION OF DAMAGE

FILE & SOURCE	SEVERITY	DAMAGE DESCRIPTION: VISUAL	DAMAGE DESCRIPTION: VERBAL	GENERAL: UNSPECIFIED	BASEMENT: UNSPECIFIED	BASEMENT: SEEPAGE	BASEMENT: WALL	BASEMENT: DOOR	BASEMENT: WINDOW	BASEMENT: FLOOR	EXTERIOR: UNSPECIFIED	EXTERIOR: SEEPAGE	EXTERIOR: WALL	EXTERIOR: DOOR	EXTERIOR: WINDOW	EXTERIOR: FLOOR	INTERIOR: UNSPECIFIED	INTERIOR: SEEPAGE	INTERIOR: WALL	INTERIOR: CEILING	INTERIOR: FLOOR	INTERIOR: DOOR	GARAGE: CHIMNEY	GARAGE: UNSPECIFIED	GARAGE: SEEPAGE	GARAGE: WALL	GARAGE: DOOR	GARAGE: WINDOW	GARAGE: FLOOR	GARAGE: UTILITY
1101.	126	1	0	0	0	1	0	0	1	0	1	1	1	1	0	1	0	1	0	0	0	0	0	1	0	0	1	0
1001.	100	1	0	0	0	1	0	0	1	0	0	0	0	1	0	1	0	1	0	0	0	0	0	0	0	0	1	0
341.	0	1	0	0	0	1	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
231.	0	1	1	0	0	1	1	0	1	0	0	1	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	1
11.	259	1	0	0	0	1	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
601.	393	1	1	0	0	1	0	0	1	0	0	0	0	1	0	0	0	1	0	0	0	0	1	0	0	1	0	
2341.	0	1	1	0	0	1	1	1	1	0	0	1	1	0	1	1	1	1	0	0	0	0	0	0	0	0	0	0
751.	0	1	1	0	0	0	0	0	1	0	1	1	0	0	0	1	1	1	0	0	0	0	0	0	0	0	0	0
851.	0	1	0	0	0	1	0	0	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
821.	0	1	0	0	0	1	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
831.	146	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1421.	0	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1431.	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1441.	395	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
1451.	0	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
3921.	100	0	1	0	0	1	0	0	1	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0
4941.	188	1	1	0	0	1	0	0	1	0	1	0	0	1	0	1	0	1	0	0	1	0	0	0	0	0	0	0
1221.	100	1	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	
811.	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
771.	0	1	1	0	0	0	1	0	0	1	0	0	0	0	0	0	1	0	1	0	0	0	0	1	0	0	0	0
121.	0	0	1	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2351.	0	1	1	0	0	1	0	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0
2561.	29	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3741.	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3781.	0	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3791.	0	1	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3801.	0	1	1	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
3831.	0	1	1	0	0	1	0	0	1	0	1	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	0	
3891.	0	0	1	0	0	0	0	1	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
3571.	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
141.	0	1	1	0	0	1	0	0	1	0	1	0	0	0	0	0	1	1	0	0	1	0	0	0	0	0	0	0
151.	404	1	1	0	0	1	0	0	1	0	1	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
631.	0	0	1	0	0	1	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1	0	0	0	0	0	
3013.	0	0	1	1	0	1	0	1	1	1	0	0	1	0	1	0	1	0	1	0	0	0	0	0	1	1		
331.	0	0	1	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0		
501.	352	1	0	0	0	1	0	0	0	0	1	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0		
871.	137	1	1	0	0	1	0	0	0	0	1	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0		
881.	268	1	1	0	0	1	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
921.	0	1	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
1211.	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
241.	156	1	1	0	0	1	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0		
321.	276	1	1	0	0	1	1	0	1	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0		
1471.	217	1	1	0	0	1	0	0	0	0	0	0	0	0	1	1	1	0	0	0	0	0	0	0	0	0		
1601.	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		

DESCRIPTION OF DAMAGE

FILE & SOURCE	SEVERITY	DAMAGE DESCRIPTION: VISUAL	DAMAGE DESCRIPTION: VERBAL	BASEMENT: GENERAL	BASEMENT: UNSPECIFIED	BASEMENT: SEWAGE	BASEMENT: DAMPNESS	BASEMENT: WINDOW	BASEMENT: FLOOR	EXTERIOR: UNSPECIFIED	EXTERIOR: SEWAGE	EXTERIOR: DAMPNESS	EXTERIOR: WINDOW	EXTERIOR: FLOOR	INTERIOR: UNSPECIFIED	INTERIOR: SEWAGE	INTERIOR: DAMPNESS	INTERIOR: CEILING	INTERIOR: FLOOR	INTERIOR: ROOF	GARAGE: CHIMNEY	GARAGE: UNSPECIFIED	GARAGE: SEWAGE	GARAGE: DAMPNESS	GARAGE: WINDOW	GARAGE: FLOOR	OTHER
3991.	52	1	1	0	0	1	0	0	1	0	0	0	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0
2431.	0	0	1	0	0	0	0	0	1	0	0	0	0	1	1	0	0	0	0	0	0	1	0	0	0	0	0

DAMAGE LEVEL VARIABLES & DAMAGE FORM VARIABLES

FILE & SOURCE	SEVERITY	REPORTING %	REPSLAB %	REPSLAB	IT	DISTURB	SEVPRED	SLABS %	SLABCRK	WALLCRK	SLABHEV	WALLBUK	A	PERIMFT	BKNGFT
1821.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
2541.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
4411.	366	34.	68.	63.	0.	1.	387	100	2	5	0	0	68	128	158
4831.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
2741.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
3541.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
4301.	124	60.	11.	0.	0.	0.	102	100	0	4	0	0	55	148	174
4331.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1621.	168	100.	0.	43.	0.	0.	208	100	1	5	0	0	60	120	164
1631.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1641.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1661.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1691.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1711.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1731.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1751.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1901.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1911.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
2201.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
2261.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
2271.	0	0.	0.	0.	0.	0.	0	0	0	0	0	1	0	0	0
2471.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
5481.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
171.	172	85.	12.	25.	0.	0.	192	100	3	6	0	0	53	154	200
301.	179	61.	26.	11.	0.	0.	121	100	1	7	0	1	58	134	203
351.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
3751.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
4021.	0	0.	0.	0.	0.	0.	0	0	0	0	0	1	0	0	0
1521.	236	100.	0.	75.	0.	0.	263	100	3	8	0	0	55	132	157
4471.	53	53.	0.	0.	0.	0.	94	100	0	3	0	0	61	152	223
3661.	500	0.	100.	100.	1.	0.	0	100	4	8	0	0	46	228	228
3671.	500	0.	100.	100.	1.	1.	468	100	4	8	0	0	50	160	187
3681.	0	0.	0.	0.	1.	0.	0	0	0	0	0	0	0	0	0
3691.	500	0.	100.	100.	1.	0.	0	100	4	8	0	0	55	134	134
3701.	500	0.	100.	100.	1.	0.	532	100	4	8	0	0	48	194	194
4731.	500	0.	0.	0.	1.	0.	0	100	4	8	0	0	48	184	184
611.	435	29.	71.	100.	0.	0.	319	100	2	5	0	0	56	106	106
131.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
181.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
271.	50	50.	0.	0.	0.	0.	182	100	0	8	0	0	55	136	162
291.	198	100.	0.	50.	0.	0.	223	100	1	8	0	0	51	180	199
311.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
371.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
951.	272	22.	78.	0.	0.	0.	256	100	1	5	0	0	55	108	108

DAMAGE LEVEL VARIABLES & DAMAGE FORM VARIABLES

FILE & SOURCE	SEVERITY	REPORT	REPAIR	REPAIR		DIST	SEV	SLAB	SLAB	SLAB	WALL	WALL		PER	BRICK
		INT	LAB	SLAB	IT	TOT	PRED	LAB	BLK	HEAV	BLK	BLK	A	FIT	FIT
		%	%	%				%							
961.	241	65.	25.	50.	0.	0.	229	100	1	1	0	0	59	132	183
1261.	164	78.	16.	0.	0.	0.	305	100	0	6	0	0	62	120	174
4751.	259	100.	0.	100.	0.	0.	153	100	1	5	1	0	54	120	152
411.	100	100.	0.	0.	0.	0.	147	100	1	1	0	0	55	108	108
421.	225	41.	54.	0.	0.	0.	119	100	3	4	0	0	58	108	118
691.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
701.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
711.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
721.	153	77.	23.	0.	0.	0.	149	100	0	2	0	0	58	102	102
731.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
781.	100	100.	0.	0.	0.	0.	215	100	0	4	0	0	54	114	114
3941.	282	100.	0.	100.	0.	0.	110	100	3	0	0	0	55	142	142
3951.	0	0.	0.	0.	0.	0.	0	0	0	0	0	1	0	0	0
4001.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
4141.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
5161.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
5621.	255	17.	73.	0.	0.	0.	107	100	0	7	0	0	53	150	172
5871.	67	67.	0.	0.	0.	1.	107	100	1	5	0	0	53	150	172
5881.	33	33.	0.	0.	0.	0.	337	100	0	1	0	0	53	150	172
6131.	273	100.	0.	100.	0.	1.	116	100	3	3	1	0	57	75	100
461.	0	0.	0.	0.	0.	0.	275	100	1	0	0	0	57	148	196
581.	394	22.	72.	75.	0.	0.	275	100	1	8	0	0	53	134	144
641.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
741.	196	100.	0.	50.	0.	0.	234	100	3	3	0	0	52	122	122
801.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
941.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1561.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
221.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
4431.	166	72.	28.	0.	0.	0.	182	100	0	1	0	1	59	108	108
2061.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
911.	45	45.	0.	0.	0.	0.	48	100	0	4	0	0	51	168	189
3731.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
391.	300	50.	50.	50.	0.	0.	258	100	2	5	0	0	50	140	140
841.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
5531.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
6411.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1281.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1311.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1961.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3621.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3631.	195	100.	0.	50.	0.	0.	174	100	0	5	1	0	54	128	128
2731.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
71.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
671.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0

DAMAGE LEVEL VARIABLES & DAMAGE FORM VARIABLES

FILE & SOURCE	SEVERITY	REPOINT %	REPAL %	REPSLAB %	TILT	DISTORT	SEVPRED	SLAB %	SLABCRK	WALLCRK	SLABHEV	WALLBUK	A	PERIMETR	BRKNGTH
1101.	126	100.	0.	14.	0.	0.	114	100	1	0	1	0	53	160	224
1001.	100	100.	0.	0.	0.	0.	102	100	0	1	0	0	53	132	132
341.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
231.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
11.	259	28.	72.	0.	0.	0.	222	100	1	1	0	0	55	108	108
601.	393	33.	51.	100.	0.	0.	403	100	2	5	0	0	54	160	210
2341.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
751.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
851.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
821.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
831.	146	79.	21.	0.	0.	0.	113	100	0	4	0	1	55	108	108
1421.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1431.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1441.	395	52.	48.	100.	0.	0.	374	100	3	8	0	0	50	148	148
1451.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3921.	100	100.	0.	0.	0.	0.	147	100	0	5	0	0	51	150	150
4941.	188	100.	0.	50.	0.	0.	187	100	1	5	1	0	56	124	149
1221.	100	100.	0.	0.	0.	0.	26	100	1	5	0	0	54	160	192
811.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
771.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
121.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
2351.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
2561.	29	29.	0.	0.	0.	0.	20	38	0	5	0	0	50	197	197
3741.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3781.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3791.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3801.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3831.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3891.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3571.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
141.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
151.	404	23.	77.	75.	0.	1.	371	100	2	1	0	0	54	112	112
631.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
3013.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0
331.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
501.	352	23.	77.	50.	0.	0.	357	100	3	1	0	0	48	140	140
871.	137	76.	15.	0.	0.	0.	178	100	1	5	0	0	62	144	226
881.	268	66.	34.	50.	0.	0.	250	100	2	5	0	0	52	128	128
921.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
1211.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0	0	0
241.	156	75.	25.	0.	0.	0.	201	100	3	5	0	0	56	110	110
321.	276	32.	68.	0.	0.	1.	259	100	1	7	0	0	65	76	76
1471.	217	50.	38.	0.	0.	0.	216	50	0	4	1	1	63	100	132
1601.	0	0.	0.	0.	0.	1.	0	0	0	0	0	0	0	0	0

DAMAGE LEVEL VARIABLES & DAMAGE FORM VARIABLES

FILE & SOURCE	SEVERITY %	REPOWAL %	REPSLAB %	REPSLTT %	DISTLT %	DISVPRED %	SLAB %	SLABCRK %	WALLCRK %	SLABVED %	WALBUK %	A	PERIMFT	BKNGFT
3991.	52.	52.	0.	0.	0.	0.	62	70	0	8	0	0.	60	124	136
2431.	0	0.	0.	0.	0.	0.	0	0	0	0	0	0	0.	0	0

BASEMENT CRACK DIMENSIONS

FILE & SOURCE	SEVERITY	AREA SLAB FT2	CRACK LENGTHS FT SUM	CRACK LENGTHS FT MEAN	CRACK WIDTHS .01-IN MEAN	CRACK WIDTHS .01-IN PAR W	CRACK WIDTHS .01-IN PAR L	AREA BEARING WALL FT2	CRACK LENGTHS FT SUM	CRACK LENGTHS FT MEAN	CRACK WIDTHS .01-IN MEAN	CRACK WIDTHS .01-IN PAR H	CRACK WIDTHS .01-IN PAR L
1821.	0	0	0	0	0	0	0	0	0	0	0	0	0
2541.	0	0	0	0	0	0	0	0	0	0	0	0	0
4411.	366	608	30	10	13	17	0	1106	14	7	25	25	0
4831.	0	0	0	0	0	0	0	0	0	0	0	0	0
2741.	0	0	0	0	0	0	0	0	0	0	0	0	0
3541.	0	0	0	0	0	0	0	0	0	0	0	0	0
4301.	124	1141	0	0	0	0	0	1218	46	8	9	0	5
4331.	0	0	0	0	0	0	0	0	0	0	0	0	0
1621.	168	875	64	8	13	16	14	1148	10	5	2	1	0
1631.	0	0	0	0	0	0	0	0	0	0	0	0	0
1641.	0	0	0	0	0	0	0	0	0	0	0	0	0
1661.	0	0	0	0	0	0	0	0	0	0	0	0	0
1691.	0	0	0	0	0	0	0	0	0	0	0	0	0
1711.	0	0	0	0	0	0	0	0	0	0	0	0	0
1731.	0	0	0	0	0	0	0	0	0	0	0	0	0
1751.	0	0	0	0	0	0	0	0	0	0	0	0	0
1901.	0	0	0	0	0	0	0	0	0	0	0	0	0
1911.	0	0	0	0	0	0	0	0	0	0	0	0	0
2201.	0	0	0	0	0	0	0	0	0	0	0	0	0
2261.	0	0	0	0	0	0	0	0	0	0	0	0	0
2271.	0	0	0	0	0	0	0	0	0	0	0	0	0
2471.	0	0	0	0	0	0	0	0	0	0	0	0	0
5481.	0	0	0	0	0	0	0	0	0	0	0	0	0
171.	172	1426	67	34	16	16	0	1400	90	5	7	16	3
301.	179	1191	60	8	2	3	0	1421	156	6	8	33	6
351.	0	0	0	0	0	0	0	0	0	0	0	0	0
3751.	0	0	0	0	0	0	0	0	0	0	0	0	0
4021.	0	0	0	0	0	0	0	0	0	0	0	0	0
1521.	236	1064	66	11	13	28	7	1099	0	0	0	0	0
4471.	53	1188	0	0	0	0	0	1561	20	3	3	2	0
3661.	500	2213	0	0	0	0	0	1596	0	0	0	0	0
3671.	500	1500	0	0	0	0	0	1309	0	0	0	0	0
3681.	0	0	0	0	0	0	0	0	0	0	0	0	0
3691.	500	900	0	0	0	0	0	938	0	0	0	0	0
3701.	500	1968	0	0	0	0	0	1358	0	0	0	0	0
4731.	500	1620	0	0	0	0	0	1288	0	0	0	0	0
611.	435	682	40	40	38	0	38	742	40	6	12	32	0
131.	0	0	0	0	0	0	0	0	0	0	0	0	0
181.	0	0	0	0	0	0	0	0	0	0	0	0	0
271.	50	1092	0	0	0	0	0	1134	35	7	2	3	0
291.	198	1552	156	14	15	26	37	1393	32	11	7	9	0
311.	0	0	0	0	0	0	0	0	0	0	0	0	0
371.	0	0	0	0	0	0	0	0	0	0	0	0	0

BASEMENT CRACK DIMENSIONS

FILE & SOURCE	SEVERITY	AREA FT2	SLAB CRACK LENGTHS FT SUM	SLAB CRACK LENGTHS FT MEAN	SLAB CRACK WIDTHS .01-IN MEAN	SLAB CRACK WIDTHS .01-IN PARL	BEARING WALL AREA FT2	BEARING WALL CRACK LENGTHS FT SUM	BEARING WALL CRACK LENGTHS FT MEAN	BEARING WALL CRACK WIDTHS .01-IN MEAN	BEARING WALL CRACK WIDTHS .01-IN PARL
951.	272	720	65	8	6	11	5	756	35	7	21	52	0
961.	241	1040	53	6	23	17	21	1281	20	7	33	38	0
1261.	164	884	0	0	0	0	0	1218	31	16	7	2	4
4751.	259	816	0	0	0	0	0	1064	9	3	2	3	0
411.	100	728	27	5	3	2	1	756	16	3	2	2	0
421.	225	704	35	18	13	13	5	826	56	9	8	2	7
691.	0	0	0	0	0	0	0	0	0	0	0	0	0
701.	0	0	0	0	0	0	0	0	0	0	0	0	0
711.	0	0	0	0	0	0	0	0	0	0	0	0	0
721.	153	610	0	0	0	0	0	714	44	11	2	0	2
731.	0	0	0	0	0	0	0	0	0	0	0	0	0
781.	100	779	0	0	0	0	0	798	50	4	2	2	1
3941.	282	964	65	11	10	25	10	994	0	0	0	0	0
3951.	0	0	0	0	0	0	0	0	0	0	0	0	0
4001.	0	0	0	0	0	0	0	0	0	0	0	0	0
4141.	0	0	0	0	0	0	0	0	0	0	0	0	0
5161.	0	0	0	0	0	0	0	0	0	0	0	0	0
5621.	255	1250	0	0	0	0	0	1204	58	5	10	31	0
5871.	67	1250	58	5	6	11	3	1204	32	6	2	4	0
5881.	33	1250	0	0	0	0	0	1050	12	6	8	6	0
6131.	273	625	147	11	5	8	10	700	16	2	4	2	0
461.	0	1200	74	11	3	6	1	1372	0	0	0	0	0
581.	394	1050	94	9	18	39	21	1008	0	0	0	0	0
641.	0	0	0	0	0	0	0	0	0	0	0	0	0
741.	196	918	100	11	12	7	13	854	11	4	2	1	0
801.	0	0	0	0	0	0	0	0	0	0	0	0	0
941.	0	0	0	0	0	0	0	0	0	0	0	0	0
1561.	0	0	0	0	0	0	0	0	0	0	0	0	0
221.	0	0	0	0	0	0	0	0	0	0	0	0	0
4431.	166	612	0	0	0	0	0	756	14	4	7	9	0
2061.	0	0	0	0	0	0	0	0	0	0	0	0	0
911.	45	1471	0	0	0	0	0	1323	51	9	2	1	1
3731.	0	0	0	0	0	0	0	0	0	0	0	0	0
391.	300	1125	26	26	25	25	0	980	58	6	13	28	0
841.	0	0	0	0	0	0	0	0	0	0	0	0	0
5531.	0	0	0	0	0	0	0	0	0	0	0	0	0
6411.	0	0	0	0	0	0	0	0	0	0	0	0	0
1281.	0	0	0	0	0	0	0	0	0	0	0	0	0
1311.	0	0	0	0	0	0	0	0	0	0	0	0	0
1961.	0	0	0	0	0	0	0	0	0	0	0	0	0
3621.	0	0	0	0	0	0	0	0	0	0	0	0	0
3631.	195	885	0	0	0	0	0	696	23	6	2	2	0
2731.	0	0	0	0	0	0	0	0	0	0	0	0	0

BASEMENT CRACK DIMENSIONS

FILE & SOURCE	SEVERITY	AREA FT2	SLAB CRACK LENGTHS FT SUM	SLAB CRACK LENGTHS FT MEAN	SLAB CRACK WIDTHS .01-IN MEAN	SLAB CRACK WIDTHS .01-IN PAR W	SLAB CRACK WIDTHS .01-IN PAR L	BEARING WALL AREA FT2	BEARING WALL CRACK LENGTHS FT SUM	BEARING WALL CRACK LENGTHS FT MEAN	BEARING WALL CRACK WIDTHS .01-IN MEAN	BEARING WALL CRACK WIDTHS .01-IN PAR H	BEARING WALL CRACK WIDTHS .01-IN PAR L
71.	0	0	0	0	0	0	0	0	0	0	0	0	0
671.	0	0	0	0	0	0	0	0	0	0	0	0	0
1101.	126	1625	35	9	3	2	1	1568	0	0	0	0	0
1001.	100	955	0	0	0	0	0	924	28	4	6	6	0
341.	0	0	0	0	0	0	0	0	0	0	0	0	0
231.	0	0	0	0	0	0	0	0	0	0	0	0	0
11.	259	720	34	17	8	11	0	756	28	7	11	13	0
601.	393	1436	64	32	28	56	0	1470	64	5	13	41	0
2341.	0	0	0	0	0	0	0	0	0	0	0	0	0
751.	0	0	0	0	0	0	0	0	0	0	0	0	0
851.	0	0	0	0	0	0	0	0	0	0	0	0	0
821.	0	0	0	0	0	0	0	0	0	0	0	0	0
831.	146	713	0	0	0	0	0	756	14	5	8	1	9
1421.	0	0	0	0	0	0	0	0	0	0	0	0	0
1431.	0	0	0	0	0	0	0	0	0	0	0	0	0
1441.	395	1189	33	11	25	19	14	1036	70	7	8	18	0
1451.	0	0	0	0	0	0	0	0	0	0	0	0	0
3921.	100	1166	0	0	0	0	0	1050	38	4	3	6	0
4941.	188	952	108	5	7	14	14	1043	47	4	3	5	0
1221.	100	1336	60	20	9	5	0	1344	49	7	5	11	0
811.	0	0	0	0	0	0	0	0	0	0	0	0	0
771.	0	0	0	0	0	0	0	0	0	0	0	0	0
121.	0	0	0	0	0	0	0	0	0	0	0	0	0
2351.	0	0	0	0	0	0	0	0	0	0	0	0	0
2561.	29	1634	0	0	0	0	0	1379	20	5	9	5	0
3741.	0	0	0	0	0	0	0	0	0	0	0	0	0
3781.	0	0	0	0	0	0	0	0	0	0	0	0	0
3791.	0	0	0	0	0	0	0	0	0	0	0	0	0
3801.	0	0	0	0	0	0	0	0	0	0	0	0	0
3831.	0	0	0	0	0	0	0	0	0	0	0	0	0
3891.	0	0	0	0	0	0	0	0	0	0	0	0	0
3571.	0	0	0	0	0	0	0	0	0	0	0	0	0
141.	0	0	0	0	0	0	0	0	0	0	0	0	0
151.	404	780	28	28	31	31	0	784	21	7	27	28	0
631.	0	0	0	0	0	0	0	0	0	0	0	0	0
3013.	0	0	0	0	0	0	0	0	0	0	0	0	0
331.	0	0	0	0	0	0	0	0	0	0	0	0	0
501.	352	1216	121	12	9	15	16	980	10	3	25	11	0
871.	137	1152	20	10	4	0	1	1582	20	5	14	10	0
881.	268	975	27	27	38	38	0	896	21	7	14	20	0
921.	0	0	0	0	0	0	0	0	0	0	0	0	0
1211.	0	0	0	0	0	0	0	0	0	0	0	0	0
241.	156	694	70	5	3	5	3	770	18	4	15	13	0

BASEMENT CRACK DIMENSIONS

FILE & SOURCE	SEVERITY	AREA SLAB	CRACK LENGTHS FT SUM	CRACK LENGTHS FT MEAN	CRACK WIDTHS .01-IN MEAN	CRACK WIDTHS .01-IN PARW	CRACK WIDTHS .01-IN PARL	AREA BEARING WALL	CRACK LENGTHS FT SUM	CRACK LENGTHS FT MEAN	CRACK WIDTHS .01-IN MEAN	CRACK WIDTHS .01-IN PARW	CRACK WIDTHS .01-IN PARL
321.	276	336	0	0	0	0	0	532	80	4	34	164	2
1471.	217	625	0	0	0	0	0	924	33	7	42	18	23
1601.	0	0	0	0	0	0	0	0	0	0	0	0	0
3991.	52	736	0	0	0	0	0	952	0	0	0	0	0
2431.	0	0	0	0	0	0	0	0	0	0	0	0	0

OVERBURDEN AND MINING DATA

FILE & SOURCE	TOTAL FT	ROCK FT	HDR ROCK FT	SOIL TYPE	COAL SEAM	MINING TYPE	MINING HEIGHT	BADJ	ADJ MINING	DAMAGE MINING	DATES MINING	MINE INACT	EMBCUMENT	GENTOP	ABANDS	ASSIMILATE
1821.	663	657	247	1	1	57	6	1	79	174	0 0	1	0	2	0	0 0
2541.	360	0	0	1	3	43	7	7	0	174	0 0	1	0	12	0	0 0
4411.	460	0	0	1	3	43	6	1	50	1076	0 0	0	0	11	0	0 0
4831.	470	0	0	1	3	43	1	1	0	877	377	0	0	11	0	0 0
2741.	703	0	0	1	2	60	4	4	0	1274	0 0	0	0	8	1	70 0
3541.	688	683	0	1	2	60	4	4	0	175	0 0	0	0	8	1	70 0
4301.	608	0	0	1	2	70	4	4	0	676	0 0	0	1	12	0	0 0
4331.	608	0	0	1	2	70	4	8	0	676	0 0	0	1	12	0	0 0
1621.	555	0	0	1	2	70	4	1	225	173	0 0	0	3	6	0	0 0
1631.	525	0	0	1	2	70	5	1	124	173	0 0	0	2	12	0	0 0
1641.	555	0	0	1	2	70	4	1	170	173	0 0	0	3	6	0	0 0
1661.	602	0	0	1	2	70	4	1	454	173	0 0	0	2	6	0	0 0
1691.	580	0	0	1	2	70	4	1	408	273	0 0	0	2	12	0	0 0
1711.	580	0	0	1	2	70	4	1	366	273	0 0	0	3	6	0	0 0
1731.	575	0	0	1	2	70	4	1	472	273	0 0	0	3	6	0	0 0
1751.	550	0	0	1	2	70	4	1	203	273	0 0	0	3	13	0	0 0
1901.	525	0	0	1	2	70	4	1	160	373	0 0	0	3	12	0	0 0
1911.	550	0	0	1	2	70	4	4	0	373	0 0	0	3	12	0	0 0
2201.	550	0	0	1	2	70	4	4	0	973	0 0	0	1	2	0	0 0
2261.	567	0	0	1	2	70	4	4	0	1073	0 0	0	1	13	0	0 0
2271.	523	0	0	1	2	70	4	4	0	1073	0 0	0	1	2	0	0 0
2471.	620	0	0	1	2	70	4	4	0	274	0 0	0	0	12	0	0 0
5481.	545	0	0	1	2	70	4	1	255	677	0 0	0	2	12	0	0 0
171.	455	0	0	1	1	70	2	8	0	766	0 0	1	0	8	0	0 0
301.	334	0	0	1	1	70	6	1	182	568	0 0	0	1	3	0	0 0
351.	340	0	0	1	1	70	6	1	224	968	0 0	0	1	3	0	0 0
3751.	430	0	0	1	1	70	1	1	0	675	0 0	0	0	6	0	0 0
4021.	300	0	0	1	1	70	4	1	218	376	1175	0	1	13	0	0 0
1521.	414	354	90	3	1	80	4	1	301	972	0 0	1	0	1	0	0 0
4471.	315	0	0	1	1	70	6	1	247	1276	0 0	0	3	12	0	0 0
3661.	425	0	260	1	1	70	3	1	496	575	475	1	0	12	0	0 0
3671.	412	0	260	1	1	70	3	1	496	575	475	1	0	12	0	0 0
3681.	400	0	255	1	1	70	3	1	496	575	475	1	0	12	0	0 0
3691.	369	0	250	1	1	70	3	1	536	575	475	1	0	12	0	0 0
3701.	435	0	260	1	1	70	3	1	536	575	475	1	0	7	0	0 0
4731.	407	0	260	1	1	70	3	1	536	177	475	1	0	12	0	0 0
611.	354	0	0	1	1	67	5	1	10	1269	0 0	0	4	2	0	0 0
131.	372	358	166	1	1	67	4	1	193	1166	966	1	0	8	0	0 0
181.	343	0	0	1	1	67	4	1	185	666	666	0	0	6	0	0 0
271.	317	0	0	1	1	67	4	1	262	568	0 0	0	0	11	0	0 0
291.	324	0	0	1	1	67	4	1	203	568	0 0	0	1	11	0	0 0
311.	335	0	0	1	1	67	4	1	296	568	0 0	0	0	11	0	0 0

OVERBURDEN AND MINING DATA

FILE & SOURCE	TOTAL FT	ROCK FT	HARD ROCK FT	SOIL TYPE	COAL SEAM	MINING TYPE	BED LOW	BED ACC	ADJACENT GOB	DAMAGE MINING MONTH	DAMAGE MINING YEAR	MINING HRS	MINING LUGS	MINERAL MINED	GENERAL TOPOGRAPHY	ABANDONED MINES	ABANDONED MINE STATUS
371.	478	447	258	3	1	70	4	1	177	168	466	1	0	1	1	0	0 0
951.	380	0	0	3	1	70	4	1	168	771	171	0	0	3	12	0	0 0
961.	338	0	0	3	1	70	4	1	210	771	0 0	0	0	4	0	0	0 0
1261.	372	365	0	3	1	70	4	1	339	472	771	0	0	0	1	0	0 0
4751.	294	289	0	3	1	70	5	1	380	577	0 0	0	0	1	2	0	0 0
411.	428	341	173	3	1	80	3	1	201	1268	0 0	1	0	0	1	0	0 0
421.	410	359	67	3	1	80	3	1	189	169	0 0	0	0	0	1	0	9 0
691.	426	0	0	3	1	80	3	1	294	370	0 0	0	0	0	1	0	0 0
701.	426	0	0	3	1	80	3	1	314	370	0 0	0	0	0	1	0	0 0
711.	420	0	0	3	1	80	2	8	0	370	0 0	0	0	0	1	0	0 0
721.	416	0	0	3	1	80	3	1	200	370	0 0	0	0	0	1	0	0 0
731.	420	0	0	3	1	80	3	1	232	370	0 0	0	0	0	1	0	0 0
781.	415	0	0	3	1	80	3	1	268	570	0 0	0	0	0	1	0	0 0
3941.	359	0	0	3	1	70	2	8	0	276	0 0	0	1	0	1	0	0 0
3951.	359	0	0	3	1	70	3	8	0	276	0 0	0	0	0	1	0	0 0
4001.	384	0	0	3	1	70	4	8	0	276	0 0	0	0	0	1	0	0 0
4141.	380	0	0	3	1	70	4	4	0	476	975	0	0	0	1	0	0 0
5161.	380	0	0	3	1	70	3	1	479	878	0 0	0	0	0	1	0	0 0
5621.	370	0	0	3	1	70	3	3	0	879	0 0	0	0	0	1	0	0 0
5871.	405	0	0	3	1	70	3	3	0	1079	0 0	0	0	0	1	0	0 0
5881.	400	0	0	3	1	70	3	3	0	1079	0 0	0	0	0	1	0	0 0
6131.	387	0	0	3	1	70	5	3	15	280	0 0	0	0	0	1	0	0 0
461.	430	343	173	3	1	80	3	1	276	169	069	1	0	0	1	0	0 0
581.	428	0	0	3	1	80	4	1	250	1269	1268	0	0	0	1	0	0 0
641.	420	0	0	3	1	80	2	8	0	170	0 0	0	0	0	1	0	0 0
741.	422	0	0	3	1	80	4	1	326	370	370	0	0	0	1	0	0 0
801.	433	0	0	3	1	80	4	1	149	770	1268	0	0	0	1	0	0 0
941.	426	0	0	3	1	80	4	1	170	771	0 0	0	0	0	1	0	0 0
1561.	417	0	0	3	1	80	4	1	197	772	0 0	0	0	0	1	0	0 0
221.	267	0	0	1	4	33	6	1	186	867	0 0	0	0	0	12	0	0 0
4431.	293	0	0	1	3	33	6	7	97	1076	875	0	0	1	12	0	0 0
2061.	453	0	0	1	4	33	1	8	0	773	0 0	0	1	1	11	0	0 0
911.	209	0	0	1	2	70	6	1	423	371	0 0	0	0	1	11	0	0 0
3731.	300	0	0	1	2	70	6	1	201	675	0 0	0	0	0	12	0	0 0
391.	590	0	0	1	1	71	4	1	170	1068	0 0	0	0	4	13	0	0 0
841.	490	0	0	1	1	71	6	8	0	970	0 0	0	0	3	7	0	0 0
5531.	160	0	0	1	1	65	6	1	181	779	0 0	0	0	3	11	0	0 0
6411.	150	0	0	2	1	65	6	1	0	780	0 0	0	0	3	11	0	0 0
1281.	700	0	0	1	4	38	7	7	0	672	0 0	0	0	4	12	0	0 0
1311.	795	0	0	1	4	38	7	7	0	672	0 0	0	0	4	11	0	0 0
1961.	435	0	0	1	4	38	7	7	0	573	0 0	0	0	4	12	0	0 0
3621.	560	0	0	1	4	38	6	7	187	475	1274	0	0	4	11	0	0 0

OVERBURDEN AND MINING DATA

FILE & SOURCE	TOTAL FT	ROCK FT	HORIZON FT	SOIL TYPE	COAL SEAM	MINING METHOD	BED LOW	BED ACC	AD JAN	DAMAGE MINING MO NTH	DAMAGE MINING YEAR	MINING TAG	MINING HAS	EMBANK MENT	GEOLOGY MAP	ABANDONED MINES	ABANDONED STATUS
3631.	580	0	0	1	4	38	6	1	172	475	0	0	0	4	11	0	0 0
2731.	535	0	0	1	4	38	7	7	0	1174	0	0	0	4	12	0	0 0
71.	700	0	0	1	4	45	4	1	167	966	466	0	0	4	6	0	0 0
671.	570	0	0	1	4	45	1	1	0	170	0	0	0	4	11	0	0 0
1101.	570	0	0	1	4	38	7	7	0	272	0	0	0	4	1	0	0 0
1001.	553	0	0	1	4	38	7	7	0	1071	0	0	0	4	1	0	0 0
341.	490	0	0	1	1	80	3	1	202	868	768	0	0	4	13	0	0 0
231.	587	567	214	1	1	60	1	1	0	1067	0	0	1	0	4	13	0 0
11.	430	398	186	3	1	80	6	1	358	766	766	1	0	4	11	0	0 0
601.	420	0	0	1	4	57	1	1	0	1269	1169	0	0	1	12	0	0 0
2341.	450	0	0	1	1	65	3	8	0	1173	573	0	0	0	6	0	0 0
751.	340	0	0	1	1	65	1	1	0	470	470	0	0	0	2	0	0 0
851.	425	0	0	1	1	65	3	8	0	1170	0	0	0	1	13	0	0 0
821.	308	0	0	1	1	65	4	1	226	870	0	0	0	1	3	0	0 0
831.	335	0	0	1	1	65	4	1	191	970	667	0	0	1	13	0	0 0
1421.	425	0	0	1	1	65	4	1	250	872	0	0	0	1	8	0	0 0
1431.	425	0	0	1	1	65	4	1	250	872	0	0	0	0	8	0	0 0
1441.	423	0	0	1	1	65	4	1	250	872	0	0	0	0	8	0	0 0
1451.	403	0	0	1	1	65	4	1	250	872	0	0	0	1	8	0	0 0
3921.	450	0	0	1	1	65	2	8	0	1075	0	0	0	1	2	6	0 0
4941.	358	0	0	1	1	65	4	1	227	1077	674	0	0	0	12	0	0 0
1221.	425	0	0	1	1	65	3	1	218	572	572	0	0	3	8	0	0 0
811.	550	0	0	1	4	30	1	1	0	770	0	0	0	0	4	11	0 0
771.	550	0	0	1	4	30	1	1	0	470	0	0	0	0	0	1	0 0
121.	228	0	0	1	3	31	6	1	171	1066	866	0	0	4	11	0	0 0
2351.	665	0	0	1	4	30	1	1	0	1173	0	0	0	0	3	11	0 0
2561.	695	0	0	1	4	30	1	1	0	574	0	0	0	0	3	11	0 0
3741.	660	0	0	1	4	30	1	1	0	675	0	0	0	0	1	11	0 0
3781.	660	0	0	1	4	30	1	1	0	875	0	0	0	0	0	11	0 0
3791.	670	0	0	1	4	30	1	1	0	875	0	0	0	0	0	11	0 0
3801.	660	0	0	1	4	30	1	1	0	875	0	0	0	0	2	11	0 0
3831.	660	0	0	1	4	30	1	1	0	875	0	0	0	0	0	11	0 0
3891.	655	0	0	1	4	30	1	1	0	1075	0	0	0	0	0	11	0 0
3571.	660	0	0	1	4	30	1	1	0	175	0	0	0	0	2	11	0 0
141.	260	220	0	1	2	70	4	3	210	1266	0	0	0	0	1	13	0 0
151.	260	220	0	1	2	70	4	3	68	1266	0	0	0	0	2	13	0 0
631.	284	0	0	3	2	72	5	1	233	1269	068	0	0	4	12	0	0 0
3013.	303	0	0	1	1	60	6	1	140	174	0	0	0	0	1	12	0 0
331.	410	0	0	3	2	75	4	1	70	668	0	0	0	0	1	8	0 0
501.	365	0	0	3	2	75	4	1	137	469	469	0	0	0	8	0	0 0
871.	593	0	0	1	1	50	5	4	0	1170	0	0	0	0	1	13	0 0
881.	602	0	0	1	1	50	5	4	10	1170	0	0	0	0	1	6	0 0

OVERBURDEN AND MINING DATA

FILE & SOURCE	TOTAL	ROCK	HARD ROCK	SOIL TYPE	COAL SEAM	MINING METHOD	MINING DEPTH	AREA MINED	DAMAGE DATES	MINING DATES	MINING TYPE	EMERGENCY	GENERAL PURPOSE	ABANDONED MINE	ASSISTANCE
921.	605	0	0	1	1	50	4	4	0	371	0 0 0	0	13	0	0 0
1211.	608	0	0	1	1	50	1	1	0	572	0 0 0	0	5	0	0 0
241.	707	519	139	1	1	70	6	1	0	1067	1067 1	0	4	2	0 0 0
321.	695	687	194	1	1	70	5	6	49	768	0 0 1	0	1	2	0 0 0
1471.	755	0	0	1	1	70	1	1	423	872	0 0 0	0	1	12	0 0 0
1601.	509	0	0	3	1	70	7	7	0	173	0 0 0	0	0	1	0 0 0
3991.	390	0	0	3	1	65	4	8	0	276	276 0	0	4	1	0 0 0
2431.	200	0	0	1	1	70	1	1	0	174	0 0 0	1	4	7	0 0 0

COSTS IN 1981 DOLLAR VALUES

FILE& SOURCE	BASE EST	TOTAL EST	REPLACE VAL	MSR EST	MSR SETTLE	CONT EST	MSIP EST	MSIP SETTLE
1821.	0.	0.	35100.	4040.	9090.	0.	0.	0.
2541.	0.	0.	50500.	1000.	1900.	0.	0.	0.
4411.	7100.	7100.	58600.	1600.	5100.	0.	0.	0.
4831.	0.	0.	72600.	19000.	33000.	0.	0.	0.
2741.	0.	0.	66000.	2900.	4400.	0.	0.	0.
3541.	0.	0.	74100.	5300.	10500.	17300.	0.	0.
4301.	2600.	2900.	55300.	1600.	1900.	0.	0.	0.
4331.	0.	0.	59200.	4800.	4000.	0.	0.	0.
1621.	3500.	3500.	54800.	1000.	11500.	0.	0.	0.
1631.	0.	0.	44900.	600.	7000.	0.	0.	0.
1641.	0.	0.	52200.	4000.	10500.	8200.	0.	0.
1661.	0.	0.	61800.	600.	7000.	0.	0.	0.
1691.	0.	0.	59500.	2000.	10000.	0.	0.	0.
1711.	0.	0.	54500.	1000.	2100.	0.	0.	0.
1731.	0.	0.	52200.	400.	3400.	0.	0.	0.
1751.	0.	0.	55800.	1000.	17500.	0.	0.	0.
1901.	0.	0.	41700.	4000.	8300.	0.	0.	0.
1911.	0.	0.	45200.	1000.	19100.	18000.	0.	0.
2201.	0.	0.	44900.	3000.	4000.	0.	0.	0.
2261.	0.	0.	47500.	2000.	1100.	0.	0.	0.
2271.	0.	0.	46900.	3000.	16000.	0.	0.	0.
2471.	0.	0.	47000.	3800.	3800.	0.	0.	0.
5481.	0.	0.	46600.	7200.	72500.	0.	0.	0.
171.	4600.	9100.	68300.	8500.	8900.	0.	0.	0.
301.	4200.	12400.	77200.	9500.	11500.	0.	0.	0.
351.	0.	0.	86100.	11100.	16200.	0.	0.	0.
3751.	0.	0.	74300.	4400.	3700.	0.	0.	0.
4021.	0.	0.	234200.	8000.	5000.	0.	0.	0.
1521.	5700.	0.	51600.	400.	2600.	0.	0.	0.
4471.	1300.	2400.	74200.	3200.	5800.	0.	0.	0.
3661.	20200.	0.	99600.	14000.	55700.	0.	0.	0.
3671.	15200.	0.	69600.	8800.	54400.	0.	0.	0.
3681.	0.	0.	80600.	1800.	72500.	0.	0.	0.
3691.	10200.	0.	60300.	1800.	62400.	0.	0.	0.
3701.	17200.	0.	130800.	8800.	72500.	0.	0.	0.
4731.	15600.	0.	79700.	7200.	72500.	0.	0.	0.
611.	7100.	7300.	39700.	8700.	7700.	0.	0.	0.
131.	0.	0.	69900.	52800.	67600.	0.	0.	0.
181.	0.	0.	63300.	6800.	4900.	0.	0.	0.
271.	1300.	1300.	48000.	600.	600.	0.	0.	0.
291.	6100.	6100.	68500.	1900.	1600.	0.	0.	0.
311.	0.	0.	48100.	3200.	2700.	0.	0.	0.
371.	0.	0.	35800.	2500.	2700.	7000.	0.	0.
951.	4300.	4300.	36800.	3600.	3100.	0.	0.	0.
961.	5700.	5700.	50500.	7300.	4800.	0.	0.	0.
1261.	2700.	4100.	50400.	2200.	12000.	0.	0.	0.
4751.	5700.	0.	48100.	5100.	29000.	0.	0.	0.
411.	1700.	1700.	48500.	9500.	6300.	0.	0.	0.
421.	4000.	4100.	35000.	2900.	1500.	5200.	0.	0.
691.	0.	0.	54200.	6900.	13500.	0.	0.	0.
701.	0.	0.	51100.	2700.	1900.	0.	0.	0.
711.	0.	0.	37500.	1400.	3900.	0.	0.	0.
721.	2200.	0.	38700.	1400.	2200.	0.	0.	0.

COSTS IN 1981 DOLLAR VALUES

FILE& SOURCE	BASE EST	TOTAL EST	REPLACE VAL	MSR EST	MSR SETTLE	CONT EST	MSIF EST	MSIF SETTLE
731.	0.	0.	61500.	2700.	1200.	0.	0.	0.
781.	1700.	2300.	36300.	1900.	1500.	0.	0.	0.
3941.	6600.	6600.	41700.	4800.	10400.	0.	0.	0.
3951.	0.	0.	54900.	4800.	5300.	0.	0.	0.
4001.	0.	0.	50600.	4800.	8200.	0.	0.	0.
4141.	0.	0.	74700.	1600.	1300.	0.	0.	0.
5161.	0.	0.	52700.	2600.	4800.	0.	0.	0.
5621.	5700.	7800.	46300.	500.	6100.	0.	0.	0.
5871.	1700.	4800.	46300.	6000.	12900.	0.	0.	0.
5881.	1000.	2400.	46300.	600.	6700.	0.	0.	0.
6131.	4500.	5900.	46300.	3300.	7200.	0.	0.	0.
461.	200.	200.	50700.	5200.	8500.	0.	0.	0.
581.	8700.	0.	47900.	8700.	20600.	0.	0.	0.
641.	0.	0.	50300.	3800.	7700.	0.	0.	0.
741.	4000.	4000.	77100.	1400.	1700.	0.	0.	0.
801.	0.	0.	62700.	2400.	4600.	0.	0.	0.
941.	0.	0.	47200.	2400.	5900.	0.	0.	0.
1561.	0.	0.	54600.	2200.	5800.	0.	0.	0.
221.	0.	0.	55600.	5100.	1100.	0.	0.	0.
4431.	2500.	0.	77100.	9600.	9600.	0.	0.	0.
2061.	0.	0.	54700.	10100.	5000.	0.	0.	0.
911.	1300.	1300.	78300.	1200.	1600.	0.	0.	0.
3731.	0.	0.	53000.	200.	11200.	0.	0.	0.
391.	6700.	0.	19100.	15900.	18300.	0.	0.	0.
841.	0.	0.	129400.	1400.	10100.	17600.	0.	0.
5531.	0.	0.	54400.	9600.	6000.	0.	0.	0.
6411.	0.	0.	49000.	3300.	2200.	0.	0.	0.
1281.	0.	0.	34700.	6600.	8900.	0.	0.	0.
1311.	0.	0.	51700.	1800.	2000.	0.	0.	0.
1961.	0.	0.	96700.	10100.	6500.	0.	0.	0.
3621.	0.	0.	60800.	1800.	3700.	0.	0.	0.
3631.	3900.	5000.	72600.	1800.	3700.	0.	0.	0.
2731.	0.	0.	51100.	1900.	4400.	0.	0.	0.
71.	0.	0.	74000.	33400.	34100.	0.	0.	0.
671.	0.	0.	275800.	13700.	32900.	0.	0.	0.
1101.	3400.	3400.	74100.	11000.	16000.	0.	0.	0.
1001.	1800.	2400.	68300.	4800.	3200.	0.	0.	0.
341.	0.	0.	26200.	3800.	4400.	0.	0.	0.
231.	0.	0.	130800.	10200.	24000.	0.	0.	0.
11.	4000.	4000.	37500.	12300.	15400.	0.	0.	0.
601.	12200.	12400.	98200.	7300.	26900.	0.	0.	0.
2341.	0.	0.	184500.	20000.	24800.	15500.	0.	0.
751.	0.	0.	102400.	5500.	10500.	10700.	0.	0.
851.	0.	0.	30300.	2700.	3400.	3600.	0.	0.
821.	0.	0.	68500.	2200.	2900.	0.	0.	0.
831.	2300.	2300.	22200.	2700.	2400.	0.	0.	0.
1421.	0.	0.	77200.	4400.	3800.	0.	0.	0.
1431.	0.	0.	85400.	1100.	400.	0.	0.	0.
1441.	9800.	0.	88600.	4400.	113800.	0.	0.	0.
1451.	0.	0.	83700.	2200.	5300.	0.	0.	0.
3921.	2000.	2700.	53200.	7000.	5800.	0.	0.	0.
4941.	4200.	5600.	46900.	2900.	13900.	0.	0.	0.
1221.	2300.	2300.	68800.	6600.	9600.	0.	0.	0.

COSTS IN 1981 DOLLAR VALUES

FILE& SOURCE	BASE EST	TOTAL EST	REPLACE VAL	MSR EST	MSR SETTLE	CONT EST	MSIF EST	MSIF SETTLE
811.	0.	0.	61400.	1100.	2400.	0.	0.	0.
771.	0.	0.	86500.	2700.	7000.	0.	0.	0.
121.	0.	0.	67700.	7000.	10200.	0.	0.	0.
2351.	0.	0.	74400.	7100.	2100.	0.	0.	0.
2561.	1100.	1100.	87800.	1000.	600.	0.	0.	0.
3741.	0.	0.	81100.	900.	6000.	0.	0.	0.
3781.	0.	0.	62800.	1800.	3300.	0.	0.	0.
3791.	0.	0.	53800.	1800.	3500.	0.	0.	0.
3801.	0.	0.	55700.	1800.	700.	0.	0.	0.
3831.	0.	0.	66400.	1800.	3200.	0.	0.	0.
3891.	0.	0.	56600.	3500.	3200.	0.	0.	0.
3571.	0.	0.	69400.	1800.	2600.	0.	0.	0.
141.	0.	0.	43000.	12300.	25600.	0.	0.	0.
151.	7000.	0.	40700.	16900.	31200.	0.	0.	0.
631.	0.	0.	48200.	2900.	2200.	0.	0.	0.
3013.	0.	0.	168900.	0.	0.	0.	60100.	41300.
331.	0.	0.	63600.	15900.	22800.	0.	0.	0.
501.	8200.	0.	111000.	5800.	9600.	0.	0.	0.
871.	3000.	4000.	51800.	13700.	15300.	0.	0.	0.
881.	5500.	5500.	38000.	16400.	16400.	0.	0.	0.
921.	0.	0.	69900.	3200.	3300.	0.	0.	0.
1211.	0.	0.	57000.	4400.	3800.	0.	0.	0.
241.	2700.	0.	40700.	6100.	6400.	0.	0.	0.
321.	4400.	6700.	95700.	6400.	8200.	0.	0.	0.
1471.	3000.	6300.	72100.	1100.	4400.	0.	0.	0.
1601.	0.	0.	53500.	6100.	29000.	0.	0.	0.
3991.	1200.	2600.	53100.	1200.	6000.	0.	0.	0.
2431.	0.	0.	111800.	1000.	6700.	0.	0.	0.

APPENDIX D

Multiple Linear Regression

Functions of Basic Parameters*

I Damage and Associated Repair Functions

Crack Area, Bearing Walls - mean width times sum of lengths

Crack Area, Slabs - mean width times sum of lengths

Crack Dimensions and Relevant Areas (Slab and Bearing Walls) - Areas in square feet; lengths in feet; and crack widths in hundredths of an inch for cases where all of the following are known:

1. slab area
2. sum of slab crack lengths
3. mean slab crack length
4. mean slab crack width
5. average apparent slab length extension
6. average apparent slab width extension
7. bearing wall area
8. sum of bearing wall crack lengths
9. mean bearing wall crack length
10. mean bearing wall crack width
11. average apparent bearing wall length extension for the group of parallel walls in plan exhibiting the greatest of such an extension
12. average apparent bearing wall height extension for the group of parallel walls in plan exhibiting the greatest such extension

$$\text{Average apparent length extension} = \frac{1}{W}\sum_{i=1}^{n}\left[\vec{l}_i \cdot \overset{\Delta}{W}\left(\vec{w}_i \cdot \overset{\Delta}{L}\right)\right]$$

i - crack
l - crack length
w - crack width
W - slab width
L - slab length

*Note: Functions in parentheses denote those that were available, but not utilized in the regression analyses.

Approximated average apparent slab length extension $= \frac{1}{W} \sum_{i=1}^{j} (w_j \, l_j)$

if $\frac{\vec{l}_j \times \overset{\Delta}{W}}{l_j \, W} \leq \sin 30°$,

or, $= \frac{1}{2W} \sum_{i=j+1}^{\kappa} (w_\kappa \, l_\kappa)$,

if $\sin 30° < \frac{\vec{l}_\kappa \times \overset{\Delta}{W}}{l_\kappa W} \leq \sin 60°$ and

or, $= 0$,

if $\sin 60° < \frac{\vec{l}_\ell \times \overset{\Delta}{W}}{l_\ell \, W}$

Note: That similar approximations presented above were employed to facilitate data collection for the 5th, 6th, 11th and 12th variables listed.

Crack Patterns, Bearing Walls - 8 dummy variables:

0 - none
1 - verticals only
2 - laterals only
3 - diagonals only
4 - verticals and laterals
5 - verticals and diagonals
6 - laterals and diagonals
7 - all
8 - unknown

Crack Patterns, Slabs - Four dummy variables identifying:

0 - none
1 - random
2 - aligned
3 - grid
4 - unknown

General Tilt - One dummy variable identifying cases of general tilt of homes $\geq$ 0.1 percent of span

Replacement Percentage, Bearing Walls

Replacement Percentage, Slab

Repointing Percentage, Perimeter Basement Bearing Walls - Percent of basement perimeter requiring repointing.

SI - Severity index multiplied by 100 (and $(SI)^2$, $(SI)^3$, $(SI)^{-1}$, $e^{(SI/100.)}$, SI predicted and ln (SI))

Superstructure Distortion - One dummy variable identifying such cases

II Damage Cost Functions

Basement Repair Cost - 1981 dollar value estimated by writers

Exterior Repair Cost - 1981 dollar value estimated by writers

Geometric Mean of MSR Estimate and Settlement (and its natural logarithm)

Interior Repair Cost - 1981 dollar value estimated by writers

MSR Estimate - Damage repair cost estimated by MSR personnel in 1981 dollars (and ln (MSR Estimate))

MSR Settlement - Settlement paid in 1981 dollars (and ln (MSR Estimate))

Total Repair Cost - 1981 dollar value estimated by writers, including basement (with integral garage), interior and exterior damage repair costs. Pavement, roof, utility and attached garage damage repair costs were neglected for simplicity. (and ln (Total Repair Cost))

III Geologic Functions

O_{RH} - Overburden thickness, hard rock, which is the sum of thicknesses, in feet, of rock units that are predominately limestone or sandstone

O_R - Overburden thickness, rock, in feet

O - Overburden thickness, total, in feet

Soil Type - One dummy variable differentiating residual/ colluvial from alluvial/terrace soil deposits

IV Ground Strain Functions

M/O - The maximum horizontal ground strain and subsidence profile slope function

M/O^2 - The maximum subsidence profile curvature function

ME/O - Product of extraction percentage and mined height, divided by overburden thickness for each of the adjacent and below mining types

V Mining Functions

Abandoned Mine Height - Height in 10ths of feet

Abandoned Seam - A dummy variable indicating presence of an abandoned seam above that of the actively mined seam precipitated damage

Coal Seam - Three dummy variables identifying the four coal seams

1 - Pittsburgh
2 - Upper Freeport
3 - Lower Freeport
4 - Lower Kittanning

D - Angle to Adjacent Mining from the vertical, in degrees, for the shortest path between the perimeter of the structure and the area of the adjacent mining type

Delay from Mining to Damage - in months

E - Extraction Percentages, distinct percentages for both the below and adjacent mining types:

1.	full retreat	85%
2.	retreat, split/notch	70%
3.	retreat, split	50%
4.	barrier pillar	0%
5.	advance	35%
6.	longwall	95%
7.	unknown	-

M - Mined height, in tenths of feet.

Mining Type - Fourteen dummy variables identifying the mining type employed below and adjacent to the site:

1 - full retreat
2 - 60 to 90% retreat - pillars split and notched
3 - < 60% retreat - pillars split
4 - barrier pillar
5 - advance - full pillars
6 - longwall
7 - unknown

OtanD - Distance to adjacent mining, the total overburden thickness multiplied by the tangent of the angle to the adjacent mining type

VI Site Functions

Embedment - Mean value, in tenths of feet, by 1/2-foot intervals, of the depth of basement walls below ground surface.

Embedment Configuration - Four dummy variables identifying:
0 - level
1 - sloping across width
2 - sloping across length
3 - sloping diagonally
4 - unknown

Topography - Ten dummy variables to identifying the various forms of topography encountered:

0 - unknown
1 - flat
2 - valley bottom
3 - slope toe
5 - slope terrace
6 - slope crest
7 - ridge or hilltop
8 - plateau
11 - gentle slope, ≥ 5, $< 10\%$
12 - moderate slope, ≥ 10, $< 20\%$
13 - steep slope, $\geq 20\%$

All features were determined with respect to the site radius - assumed for simplicity to be about 200 feet.

VII Structure Functions

Age - In the year damage occurred, in years.

'A' - Ratio of cost of basement bearing wall replacement to combined cost of basement slab and bearing wall.

Basement Perimeter - Sum of lengths of all basement perimeter walls.

Bearing Wall Length - Sum of lengths of all basement bearing walls

Deflection Ratio Terms - Determined for basement walls along width and length of structure, associated with bending and shearing - as presented by Burland and Wroth (1974):

$w\ L^3/[RI(E)]$

$w\ L/[TT(E/BB)8']$

$w\ W^3/[IR(E)]$

$w\ W/[TT(E/BB)8']$

where:

E - modulus of elasticity

G - modulus of rigidity

BB - E/G, a function of physical construction and Poisson's ratio

TT - Mean thickness of basement wall, reduced for hollow block.

NOTE: Table B-1 presents values for TT, E, BB and RI computed for typical homes

Deflection Ratio Divided by Maximum Bending or Diagonal Strains - Various wall dimension functions presented by Burland and Wroth (1974):

1. L/H
2. H/L
3. $(L/H)^2$
4. W/H
5. H/W
6. $(W/H)^2$

Facing Type - Two dummy variables identifying rigid, flexible and combination exterior facings, where, combination facings have at least 25 percent of each type

Foundation Type - One dummy variable differentiating split-level from single level basements. Slab-on-grade foundations were excluded from the damage severity analysis

H - Height of structure's exterior walls, assuming 8-feet per floor and including the basement:

$H = (S+1) \times 8$ feet

(and H^2, H^3, H^{-1} and H^{-3})

LA - Living area (Engelsman, 1981), including:

1. 50% of unfinished basements, assumed unfinished when unknown
2. 65% of attached or integral garages (which are defined to support a fully enclosed portion of the superstructure)
3. 50% of roofed porches, neglected if unknown
4. 50% of unfinished attics having a headroom $\geq$ 5-feet or 100% of finished attics where the headroom is $\geq$ 5-feet. Homes built prior to 1955 were assumed to have a roof pitch of 40%. Pre-'55, 1-story homes and those built since were assumed to have a roof pitch of 25% unless designated to have an additional 1/2 story - obviously a finished attic for which the roof pitch was also assumed to be 25%.

L - Basement length

(and L^2, L^3, L^{-1}, L^{-2} and L^{-3})

LW - Rectangular area encompassing the basement

(and $(LW)^2$, $(LW)^{-1}$ and $(LW)^{-2}$)

QF - Quality factor (Engelsman, 1981) ranging from economy through average to high quality, and variable with the types of home as presented in the following table. Adjustments were required such that older homes (built prior to 1955) were typically shifted one-third to the next higher level due to basic differences in construction. Further, for homes with rigid exterior facings a shift of one-tenth to the next higher level was assumed (1/20-for combination facings)

Number of Stories or Type of Home	Quality Level: Economy	Average	High
1	1.00	1.55	2.35
1.5	1.00	1.55	2.30
2	1.00	1.45	2.15
split-level	1.00	1.40	2.15
duplex	1.00	1.15	1.60

eg.:

For an average quality home built prior to 1955 and having brick facing:

$QF = (2/3\ QF'avg + 1/3\ QF'high)\ 9/10 + 1/10\ QF'high$

RI - Cross-sectional moment of inertia of basement bearing walls neglecting reductions for windows and doors. Two types were assumed:

1. A 7'-4" block (8" x 8" x 16") wall on an 8" x 16" wide concrete footer where the blocks are 60-percent hollow

2. A 2' x 8' stone wall

Replacement Value - 1981 dollar value estimated by writers. The replacement value, R, is computed according to (Engelsman), 1981:

$R = LA \times C \times (0.94)\ (QF)\ (1.0 + LAF + CMF + SF + SH)$

C - Cost per square foot dependent on type of home.

1 story	$18.20
1-1/2 story	$19.25
2 story	$18.10
2-1/2 story	$19.14
Split-level	$21.25
Duplex	$20.30

LAF- Living area factor which reduces cost as the living area rises:

$\leq 1000\ ft^2$:	+0.02
$> 1000, \leq 1600\ ft^2$:	+0.00
$> 1600, \leq 2400\ ft^2$:	-0.02
$> 2400, \leq 4000\ ft^2$:	-0.04
$> 4000\ ft^2$:	-0.05

CMF- Construction method factor, which reduces cost for homes built in developments, etc:

contemporary:	+0.06	
custom :	+0.10	(assumed for high quality homes)
development :	-0.05	

SH - Story height factor, this height was assumed to be 8-feet throughout as it was rarely known. The factor is assumed to equal 0.00.

S - Number of stories rounded down to an integer, excluding the basement

SF - Shape factor of the superstructure (Engelsman, 1981) that reflects increased home replacement value as the home becomes increasingly irregular:

rectangle	= +0.00
semi-L	= +0.01
L	= +0.02

W - Basement width
(and W^2, W^3, W^{-1}, W^{-2} and W^{-3})

w - Lineal load transmitted by the basement walls or footings to the subgrade. w is computed:

$$w = AA + Trid + \theta LW \left(\frac{S + Attic + Gamma}{3L + W}\right)$$

The superstructure's areal load portion of the lineal subgrade load function was derived as follows: Basements were assumed to be rectangular, few are not, and their bearing walls were assumed to carry floor, roof or attic loadings (including interior superstructure partition and bearing walls) proportionate to their span. Interior basement bearing walls spanning the basement width were assumed to exist for each integer multiple, greater than one, of the ratio of basement length to width. Thus a bearing wall of span equal to basement width would carry W/(3L+W) of the areal load. Assuming a single floor load of θLW then this same bearing wall would carry:

$$\theta LW \frac{W}{(3L+W)} = \frac{\theta LW^2}{3L+W}$$

Thus, the lineal load on bearing walls per floor is:

$$\frac{\theta LW^2}{W(3L+W)} = \frac{\theta LW}{3L+W}$$

This may be multiplied by the integer number of stories plus the attic roof load ratios, relative to normal living area loads, to determine w, excluding the basement and exterior walls.

AA - Lineal load on the basement due to the exterior walls which varies with exterior facing type and number of stories

Attic - Proportion of attic floor to normal living area floor load, which varies with roof pitch and attic use. If finished, the number of stories is increased by 1/2, as for a Cape Cod style home, and the roof would be assumed to have a pitch of 40 degrees (Engelsman, 1981):

semi-irregular = +0.035
irregular = +0.07

Gamma - Proportion of roof load to normal living area floor load

θ - Typical floor load per unit area, both live and dead, includes interior superstructure walls

Trid - Lineal weight of the basement walls, which varies with the two typical wall components (stone or block)

NOTE: Table B-1 presents values of these parameters typical of various basic home types.

Table D-1

TYPICAL HOMES
FOUNDATION LOAD AND SECTION PARAMETERS

Number of Stories	Exterior Facing Type	Basement Bearing Wall Components	Pre/Post 1955 Year of Construction	Roof Pitch (degrees)	'AA' Super Structures' Exterior Wall Weight (lb/ft)	'TRID' Basement Bearing Wall Weight (lb/ft)	'THETA' Living Area Floor Load (lb/ft^2)	'ATTIC' Attic: Living Area Floor Load Ratio	'GAMMA' Roof: Living Area Floor Load Ratio	'E' Basement Bearing Wall Modulus of Elasticity ($10^8 lb/ft^2$)	'RI' Basement Bearing Wall Moment of Inertia (ft^4)	'BB' Elasticity: Rigidity Modular Ratio	'TT' Basement Bearing Wall Thickness (ft)
0.0	None	Block	-	25	0	500	25	.25	.6	2.3	42.8	2.5	0.48
1.0	Rigid	Block	-	25	288	500	25	.25	.6	2.3	42.8	2.5	0.48
1.5	Rigid	Block	-	40	288	500	25	.75	.8	2.3	42.8	2.5	0.48
2.0	Rigid	Block	Post-	25	576	500	25	.25	.6	2.3	42.8	2.5	0.48
2.0	Rigid	Block	Pre-	40	576	500	25	.50	.8	2.3	42.8	2.5	0.48
1.0	Flexible	Block		25	96	500	25	.25	.6	2.3	42.8	2.5	0.48
1.5	Flexible	Block		40	96	500	25	.75	.8	2.3	42.8	2.5	0.48
2.0	Flexible	Block	Post-	25	192	500	25	.25	.6	2.3	42.8	2.5	0.48
2.0	Flexible	Block	Pre-	40	192	500	25	.50	.8	2.3	42.8	2.5	0.48
2.5	Flexible	Block	-	40	192	500	25	.75	.8	2.3	42.8	2.5	0.48
1.0	Rigid	Stone	-	25	288	2400	25	.25	.6	2.3	85.3	2.5	2.00
1.5	Rigid	Stone	-	40	288	2400	25	.75	.8	2.3	85.3	2.5	2.00
2.0	Rigid	Stone	Post-	25	576	2400	25	.25	.6	2.3	85.3	2.5	2.00
2.0	Rigid	Stone	Pre-	40	576	2400	25	.50	.8	2.3	85.3	2.5	2.00
1.0	Flexible	Stone	-	25	96	2400	25	.25	.6	2.3	85.3	2.5	2.00
1.5	Flexible	Stone	-	40	96	2400	25	.75	.8	2.3	85.3	2.5	2.00
2.0	Flexible	Stone	Post-	25	192	2400	25	.25	.6	2.3	85.3	2.5	2.00
2.0	Flexible	Stone	Pre-	40	192	2400	25	.50	.8	2.3	85.3	2.5	2.00
2.5	Flexible	Stone	-	40	192	2400	25	.75	.8	2.3	85.3	2.5	2.00
1.0	Comb.	Block	-	25	192	500	25	.25	.6	2.3	42.8	2.5	0.48
1.5	Comb.	Block	-	40	192	500	25	.75	.8	2.3	42.8	2.5	0.48
2.0	Comb.	Block	Post-	25	384	500	25	.25	.6	2.3	42.8	2.5	0.48
2.0	Comb.	Block	Pre-	40	384	500	25	.50	.8	2.3	42.8	2.5	0.48