1998

Expansive rocks in California : an engineering and geologic review

Lauren Jelks Doyel
San Jose State University

EXPANSIVE ROCKS IN CALIFORNIA:

AN ENGINEERING AND GEOLOGIC REVIEW

A Thesis

Presented to

The Faculty of the Department of Civil and Environmental Engineering

San Jose State University

In Partial Fulfillment

of the Requirements for the Degree

Master of Science

by

Lauren Jelks Doyel

December 1998

UMI Number: 1392805

UMI
300 North Zeeb Road
Ann Arbor, MI 48103

APPROVED FOR THE DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

Dr. Kuei-Wu Tsai, Associate Dean, College of Engineering

Dr. Ronald Yeung, Assistant Professor, Department of Civil and Environmental Engineering

Richard L. Meehan, C.E. G.E., Richard Meehan Associates

APPROVED FOR THE UNIVERSITY

William Fish

ABSTRACT

EXPANSIVE ROCKS IN CALIFORNIA
AN ENGINEERING AND GEOLOGIC REVIEW

by Lauren Jelks Doyel

Expansive rocks, also known as swelling or heaving rocks, are present throughout the world and have damaged roads, tunnels, mines, dams, and foundations of commercial, industrial, and residential development. Knowledge of expansive rocks in the engineering profession is localized and not basic knowledge, although case studies have been recorded since 1928. The fields of engineering geology and rock mechanics have the most knowledge about expansive rock, yet little of this knowledge has crossed over into the civil engineering profession. In California expansive rocks are common. To mitigate this problem, a good understanding of geology must be incorporated into the traditional soil investigation used by geotechnical engineers, in addition to improvements in foundation and pavement design. Expansive rock damage is difficult to monitor because damage develops over years. Through case study review, problems in identifying and mitigating expansive rock are studied and improvements suggested.

Acknowledgements

I would like to thank my husband, who has been Herculean in his support of my endeavor; my mother, who never stopped asking when I was going to finish; my parents-in-law for hours of baby-sitting; my two advisers, Dr. Kuei-Wu Tsai for always believing, and Dr. Ronald Yeung for his fresh energy; and my mentor, Richard L. Meehan, to whom I will always be eternally grateful. Thank you also to all the geotechnical engineers and engineering geologists who gave their time and wisdom to me. This is dedicated to my two daughters.

Table of Contents **Page**

List of Tables

List of Figures **Page**

1.0 Introduction

Expansive soils in California and other western states such as Texas and Arizona have been well known to soils engineers and geologists of the American west for forty years (Allen and Johnson 1936; Chen 1975; Holtz and Gibbs 1956; Meehan and Karp 1994; Snethen 1975; Woodward-Clyde 1967). The Highway Research Board published the first discussion of expansive soils in 1936. Most of these soils come from shales and volcanic rocks with high clay content or minerals that preferentially weather into clay. Rock formations containing active clay minerals are unstable and yield expansive clay soils; damage from these expansive soils is well documented over the last several decades. However, these rock formations are also expansive and problematic for suburban residential construction, which has expanded rapidly in California over the last several decades. As will be shown, the history of expansive rock damage in California is related to the housing economy, geography and the evolution of law. Prior to this housing boom, experience with swelling rock was largely restricted to highway, tunnel and dam construction in other parts of the United States (mainly Colorado, Texas, & South Dakota (Chen 1975; Tourtelot 1974), and outside the U.S. in Canada (Peterson 1958) and Europe (Gysel 1977).

In the mid to late 1970s mass housing developed in terrain containing expansive soil in the San Francisco Bay region. The writer has investigated several large housing tract developments in northern California with structural and cosmetic damage in the intercreek basin areas of San Jose, on the hillsides above Stanford University, and the foothills of the Diablo Range in the East Bay in Blackhawk, Pittsburg, and Vallejo (Table

1). These tracts were developed in the mid-1970s to the early 1980s and Meehan and Karp (1994) discuss forensic investigations of these tracts in their paper. In the Los Angeles region, expansive soil damage was recognized somewhat earlier after the postwar housing boom and introduction of mass housing in the 1950s (Meehan and Karp 1994). Expansive soil is often one of many problems common to these large tract developments. Less commonly known is damage from expansive rock, either unrecognized or attributed to fill settlement, expansive fill or poor construction. Further confusion is added by problems with nomenclature due to the nature of expansive rock formation, and differing usage of terms by various professions. Expansive rock is differentiated from expansive soil by location and heaving pattern. Generally expansive soil is found in valleys and intercreek basins, although it may blanket hillsides that have expansive rock formations, and seasonally it shrinks and swells, so there is expansion and contraction. Expansive rock is generally found on low foothills and mountains surrounding valleys that contain expansive soils, and is associated with long-term expansion and heave, there is no cyclical shrinking and swelling within the rock.

Why are expansive rocks and the damage they cause to residential structures in California not widely discussed in the geotechnical literature? Is it possible to predict where expansive rock damage will occur and how much heave will take place? What improvements can the geotechnical engineering profession make in recognition, characterization and mitigation of the heaving rocks? These are some of the issues addressed in the following discussion.

Table 1. Damaged Developments

Completed	Location
1975	Sharon Heights, Menlo Park, CA
1980	Tennis Villas, Blackhawk, CA
early 1980s	Seeno Homes, Pittsburg, CA
1986	Warmington Homes, Antioch, CA
late 1980s	Mariner Point, Vallejo, CA
1989	single family home, Woodside, CA
1989	single family home, Atherton, CA
1989	North Ranch, Thousand Oaks, CA
1990	single family home, Ben Lomond, CA
1991	Crystal Point, Vallejo, CA
1991	single family home, Atherton, CA

Research divided across disciplines may be one reason why expansive rocks are not well known or understood by the geotechnical engineering profession, despite claims to the contrary (Hollingsworth 1990). The literature discussing these mudrocks and their swelling is international, but divided among many disciplines over the years: oil, gas and mineral exploration, rock mechanics, and geology. In few places does the discussion within these groups intersect with each other, except by obscure reference. Rock mechanics, tunneling and geologic publications are not the traditional venues for the discussion of applied geotechnical engineering problems and practice. The discussion and study are still on-going and current, and is marked by revelations such as the recently published study of swelling mechanism of clay minerals (Karaborni et al. 1996), summary and review of swelling rocks (Einstein 1996) and the work of the Colorado Geological Survey (Higgins 1997; Noe 1997).

Additionally, since these rocks are not present in all parts of the country or world, indeed even within all of California itself, the experience and knowledge of expansive rocks tends to be very localized (personal communication, Moran 1997; personal communication, Slosson 1997). The geologic setting of these expansive rocks varies widely, which affects their swelling potential. The terms used to describe expansive rocks are not standardized, nor is there a standard method of characterization that lays out standard properties and observations to be made about expansive rocks (Noe 1997). Failure of engineers to recognize expansive rocks in the investigation stage is due largely to a lack of geologic training. This lack of geologic recognition or regional pattern recognition forestalls any mitigation by planning and design of residential projects; and, mitigation measures implemented are not sure solutions for expansive rock hazard.

Case histories presented and discussed herein, confirm that the phenomenon of expansive rock and subsequent damage to structures is still poorly understood by the practicing geotechnical engineer and the engineering community at large. This can be attributed to localize knowledge, difficulty in understanding swelling phenomenon and measuring and modeling the effects of various factors affecting swelling over time. Several of these case studies also provide a measure of claystone heave over time, and are discussed in the context of new updated geologic maps published by the USGS in 1994 in the San Francisco Bay area. Case study is a method widely used in the engineering (soil mechanics) field advocated by Ralph Peck and Karl Terzaghi. Through case review, and discussions with practicing engineers and geologists, the author attempts to refine and improve the understanding of claystones in California and offers recommendations for assisting geotechnical engineers in recognizing and characterizing potential heaving problems. In large part this involves geotechnical engineers rededicating themselves to understanding geology and a commitment to improving professional standards beyond what is currently practiced. This may be difficult realistically in a market-driven economy where many geotechnical engineers are largely employed by developers in the housing and construction industry, which even mildly described, is extremely volatile. As well, geotechnical engineers are reluctant to try any new empirical approaches to formulation of design in the legal climate of residential housing development where all engineering professionals are regularly sued by unhappy homeowners who typically have paid high prices for a less than perfect product.

1.1 Definition of Terms

Within the literature, expansive soil and expansive rock are discussed together with no distinction between the two. This is common in papers where the focus of the discussion is on the actual swelling of the clay fraction of the soil or rock, or papers based around construction of a particular type of civil engineering works (such as roads). Study of expansive soils predates recognition of expansive rock by a decade or so in the published literature. Within the framework of the discussion presented herein, expansive soil and expansive rock are considered distinct and separate, even though expansive soil is commonly derived from expansive rock. Meehan and Karp (1994) present an excellent summary discussion of expansive soils and their effect on housing.

In discussing expansive rocks, various terms are used, much of the terminology having to do with the mechanics of the rock movement. Terzaghi, in first discussing swelling in the context of the physical properties of very fine soil fractions and consolidation, defines swelling as water content and volume increase in response to load relief. He enumerates three causes of swelling, the contribution of which is dependent on grain size fraction: elastic restitution, intergranular water absorption within a clay layer, and actual swelling of montmorillonite clay particles themselves within the clay fraction of a soil (Terzaghi and Peck 1948). Terzaghi attributes only a small portion of swelling to the latter. In classical soil mechanics then, the mechanism of swelling is defined as changes in volume either due to load relief or changes in water content. This definition of swelling is adopted with this discussion.

Terzaghi distinguishes between squeezing ground and swelling ground on the basis of rate of movement, magnitude of movement and also perceived swelling mechanism (Terzaghi 1950). Squeezing ground occurs in tunnels and moves slowly into the opening or excavation and the movement is largely physical displacement due to stress relief, with very little volume increase of the rock (Lindner 1976). The loss of tunnel volume may be almost imperceptible. Swelling ground on the other hand happens relatively quickly, is due to both stress relief and volume change in rock material due to change in water content (swelling of the clay fraction and minerals) and in fact is very noticeable, closing entire tunnels if not braced rapidly and strongly enough. This is typical in heavily precompressed clays with Plasticity Index (PI) greater than 30% and sedimentary formations containing anhydrite. While this is not the earliest use of these terms (Knopf 1929), Terzaghi's contribution represents an attempt at standardization of the discussion of this phenomenon within the field of soil mechanics and engineering geology. Much of the discussion of this phenomenon is in the context of underground excavations or tunnels and is addressed by the study of Rock Mechanics (Einstein 1996; Lindner 1976).

In Canadian literature and other papers discussing heaving of heavily over-consolidated clay and claystones the terms heave and rebound are used interchangeably (Peterson 1958). In literature describing clays and shales mined for their clay mineralogical value, and used in the manufacture of lightweight concrete, "expansible rock" or "expansible shale" is used. This refers to the tendency of the clay minerals in these rocks to expand under a special heat treatment process and from lightweight aggregate (Burnett 1965). Other terms have been used to describe swelling shales; "black shales" ("At Oahe" 1959; Grattan-Belew and McRostie 1982; Moran 1989 unpublished; Penner, Eden and Gillott 1973; Quigley and Vogan 1970; Vine and Tourtelot 1970), after the swelling rock they have encountered which has a characteristic dark organic color, and "pyritic shales" (Goodman 1989; Hunt 1984), and heaving rock (Noe 1997).

Throughout this paper, the terms swelling and expansive will be used interchangeably to denote movement of rock regardless of the influence of load relief or changes in moisture content. Heaving will be used as Terzaghi used the term, to denote upward displacement of structures on expansive rock (Terzaghi 1950). The term rebound will not be used because it is problematic and has been used several different ways. It is typically used to indicate movement due solely to stress relief (Noe 1997), although the equation for total stress in soil mechanics includes a pore pressure term, which implies changes in moisture content. Peterson in his classic 1958 paper on the Bearpaw shale describes an increase in moisture content as a natural part of rebound. Rebound must also be precisely described, as it can be strictly elastic or include an element or permanent deformation, such as flexural slip along bedding planes (not uncommon in steeply dipping geologic structures).

1.2 Expansive Rock Nomenclature

Expansive rocks are identified by a common response to load relief and increase in moisture content. There are a wide variety of terms used to describe rocks that are typically expansive. Claystones are part of a larger rock group known as mudrocks or shales. The problem of shale description is one that has plagued the civil engineering profession for decades, and is still the subject of much study. Part of the confusion arises out of the purpose for which rocks are classified. Are they classified by geologic origin as opposed to engineering purpose? It is important to note that the classification terms used for these rocks were not developed in a vacuum but within the context of the geologic and engineering investigations of large dam sites worldwide. Mead (1936) coined the terms "cemented shale" and "compaction shale" to distinguish between two types of shale, and signifying inherent differences in their strength characteristics and are referred to as "immature shales" due to their uncemented nature and sometimes soil like properties (Philbrick 1950).

Within the engineering community there are multiple systems for classifying rocks for engineering purposes, each with their strengths and drawbacks (Kirkaldie 1988). One popular engineering classification proposed divides shale into soil-like or compaction shale and rock-like or cemented shale (Finkl 1984). Claystones fall into the soil-like category and are often referred to as compaction claystones. A recent study of mudrocks nationwide attempts to statistically evaluate over 25 different properties of mudrocks and develop relationships which would lead to prediction of swell behavior under various conditions (Sarman and Palmer 1994). While ambitious, this approach to prediction of how claystone will behave is of limited use for the common practicing engineer. Of more practical use would be a characterization similar to one proposed by Underwood (1967) that evaluates the response of shale specimen to changes in total stress and moisture based on laboratory and in-situ tests as well as field observations (Finkl 1984; Underwood 1967). This provides in essence an outline that can be combined with local experience to evaluate claystones. Geology is a key indicator as to whether a claystone or shale is likely to swell in response to environmental changes.

1.3 Expansive Rocks World-wide

Mudrocks are found on the continental margins and in their adjacent seas worldwide. Where tectonism and collision at plate boundaries have elevated the rocks, they have become part of the landscape, such as in the Swiss Alps and in the western U.S. and Canada. In places like the North Sea, these claystones remain hundreds and thousands of meters deep. Fig. 1 shows the location of major tectonic plate borders and also the worldwide location of three age groups of rocks. Fig. 2 shows the standard international geologic time scale. Almost all of the expansive claystones worldwide fall within the age category of Mesozoic-Cenozoic rocks, less than 240 million years old. This is in good agreement with the description of worldwide expansive rock problem. These problems have been primarily recognized in Europe, where there has been a great deal of tunneling through deformed rocks, and in western Canada and the United States,

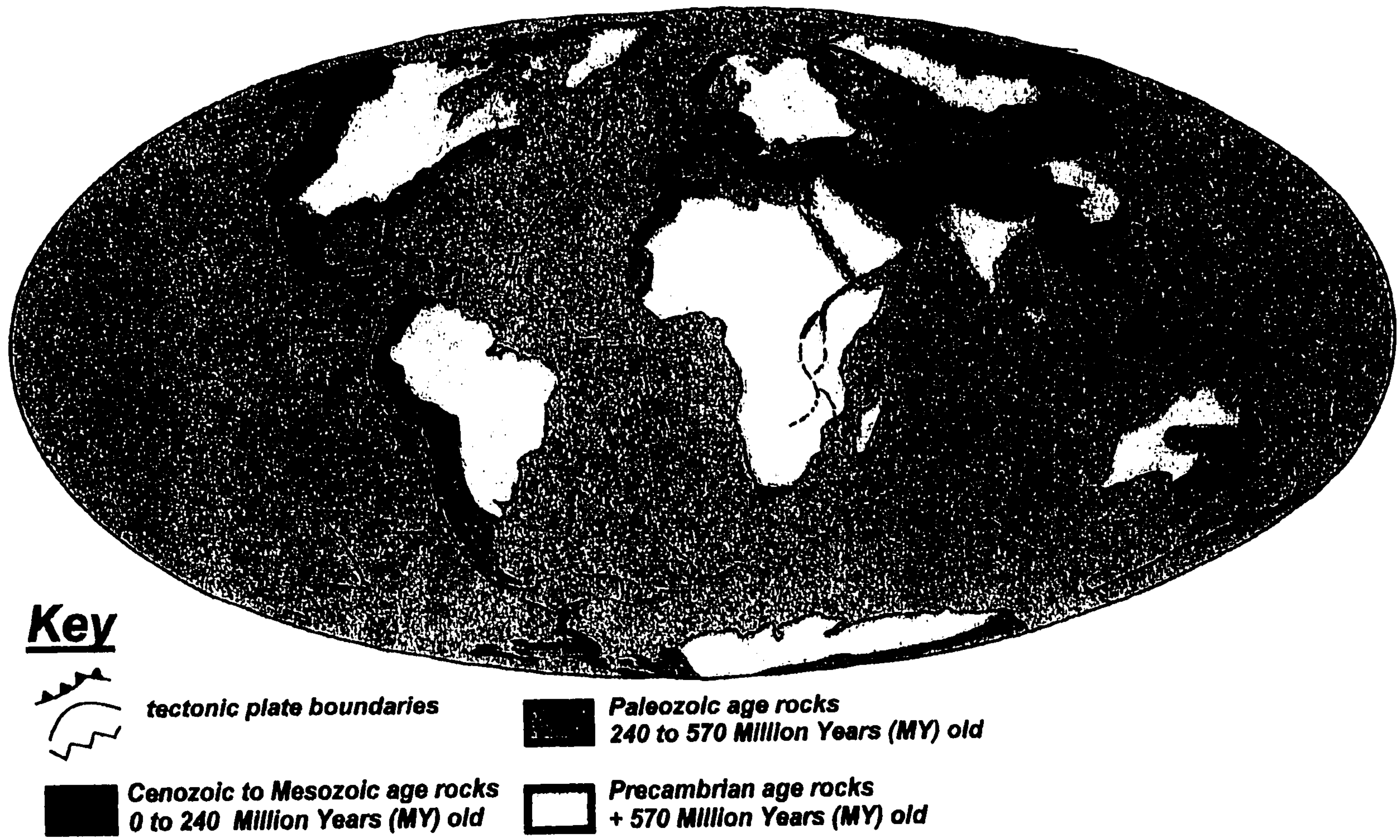


Fig. 1. World-wide distribution of Cenozoic age rocks containing swelling rock minerals.

Phanerozoic Time Scale

Era	Period	Epoch	Millions of Years Ago
Cenozoic	Quaternary	Holocene	0.01
		Pleistocene	2
	Tertiary	Pliocene	5
		Miocene	24
		Oligocene	38
		Eocene	55
		Paleocene	63
Mesozoic	Cretaceous		138
	Jurassic		205
	Triassic		240
Paleozoic	Permian		290
	Pennsylvanian		330
	Mississippian		360
	Devonian		410
	Silurian		435
	Ordovician		500
	Cambrian		570
Precambrian			

(from Harland and Francis, 1971)

Fig. 2. International geologic time scale.

where rebound of rock in general is a problem with excavations in overconsolidated clays and shales

In Africa, the only continent largely free of the younger claystone bearing rocks, some mudrocks occur within South Africa at the southern tip of the continent and have been studied with respect to swelling potential and possible damage to planned highways (Venter 1981). In Europe, problems with tunneling in swelling mudrocks have been encountered commonly in the Swiss Alps, Austria and southern Germany. This is discussed extensively in the rock mechanics literature, and an entire methodology of designing tunnels in swelling rocks has come out this experience (Einstein 1996; Gysel 1977; Kovari, Amstad and Anagnostou 1988). In the North Sea and the Barents Sea off Norway, where these younger mudrocks lie offshore several thousand meters deep beneath the ocean, serious problems are encountered with squeezing rocks during drilling of oil production wells. In England, the mudrocks are the source of much study due to the interest in placing nuclear waste repositories deep in impermeable formations (formations with high clay content). This work is published in a series of technical reports on radioactive waste disposal by the Institute of Geological Sciences of Great Britain (Hobbs et al. 1982).

In South America, the younger rocks that typically contain claystones are located on the western coast of the continent, along the Andes Mountains. However, swelling claystones have been encountered in southeast Brazil in older formations, although this is atypical. In the swelling claystones have affected dam foundations and bridge abutments, typically foundations requiring significant excavation (Da Costa Nunes 1979).

In North America, expansive claystones and shales are found in western Canada and the western United States, as well as along the gulf coast, mainly in Texas. These rocks have damaged residential foundations, tunnels, roadway pavements, embankments and cut/fills and other civil structures, as well as contributed to large-scale slope instability (Lamb and Hanna 1972; Lindner 1976; Peterson 1958; Transportation Research Board 1981).

1.4 Expansive Rocks in California

Worldwide and nationwide, there are many types of rock that swell due to different mechanisms. The three main types of swelling rock are anhydrite (calcium sulfate containing), black shale (rich in organic carbon, iron and sulfur), and claystone. The swelling mechanisms are discussed in section 5: Swelling mechanisms of expansive rocks. The focus of this study is claystone, the most common expansive rock in California; black shale, anhydrite and gypsiferous shale are not included in this discussion. Regardless of the type of expansive rock, all of the problems involve changes in load, exposure and water. Geology is the key indicator as to whether a claystone is likely to swell in response to moisture, stress or other environmental changes. The problems in California are all within steeply dipping bedrock and largely develop in cut/fill construction, distress to structures most often occurring in the cut portion of the development, although there are examples of claystone swelling beneath a fill slope.

The most common type of swelling rock in California is claystone or shale, along with other depositionally related rocks in close proximity, such as tuffs and siltstones. However, these tuffs and siltstones, if not weathered, can behave much like heavily precompressed shales in Canada, that is they are "springy" and tend to rebound (Peterson 1958) even when a there is little load relief. Highly expansive rocks usually contain illite clays or bentonite or bentonite beds. Most often these expansive rocks are claystones of Tertiary age (66.4 Ma to 1.6 Ma) and found in the rounded, weathered coastal ranges of California, although there are some troublesome claystones of late Cretaceous period. There are younger claystones that are just now being recognized as potentially expansive (the Quaternary Santa Clara formation in the San Francisco Bay area, <1.6 Ma) but experience indicates that they are not as expansive as the older claystones.

2.0 Past and Present Experience with Expansive Rocks

Volume changes when environmental changes are inflicted changes in ambient moisture or stress. These expansive mudrocks when disturbed by either drilling, tunneling, or man-made excavation respond by swelling and damaging the structures placed within and on them, for example dam and bridge abutments, tunnel inverts, mine shafts, oil wells, building foundations, and roads.

In the United States, experience with swelling rocks and understanding of clay soils and clay shales has much to do with the history of development of the west in the 20th century, a century of engineering, technology and westward expansion of population and therefore transportation and housing needs. Early on, the chief interest in clay was in particular types of clay minerals and their usefulness to industry. As the west was developed expansive rocks were encountered in the foundations for large dams and in massive road cuts for the highways that were built to provide water, power and transportation to a rapidly expanding population. Later, after development of mass housing in California, the focus was on expansive clay soils that are related to but fundamentally different from expansive rocks. Now with development of building sites previously considered marginal, expansive rocks are commonly encountered.

In California, mineral exploration lead to one of the first published encounters with swelling rock in California by Adolf Knopf in 1929 in a study of the mother lode mining belt for the State Mining Bureau (Knopf 1929). The swelling rock he discovered within a claystone and slate bed in a zone of massive quartz and gold veins in the western foothills of the Sierra Nevada illustrates a good early understanding of the general mechanisms that caused the swelling and predates any discussion in the professional engineering literature. "The practical consequence of the (claystone) gouge is that it causes swelling ground, which makes heavy timbering necessary. In a few weeks, 18 inch timbers will be reduced to matchwood." He attributed the swelling to both material movement into the shaft as well as "...imbibing of water by colloidal material..." (Knopf 1929, p.25). In modern terminology these rocks are referred to as "mylonites" (Goodman 1993). The practical problems of these mylonites are evident in the current geologic evaluation of Pardee Dam in the East San Francisco Bay hills. Here a similar black slate bed contains a "phyllite" (claystone or clay gouge) seam, and discussions and studies are underway to evaluate the significance of this seam to the foundation of the dam (personal communication, Douglas Hamilton 1997). This slate seam was identified by Louderback in 1928 (Louderback 1958) and evaluated as having swelling potential if the foundation was excavated (unloaded) or exposed to water (from the filling reservoir).

Terzaghi and Peck define swelling in the first edition of Soil Mechanics in Engineering Practice (1948) as an increase in water content and volume expansion dependent on the colloidal (clay) fraction present in rock or soil. This is due in part to "elastic restitution" or mechanical rebound, swelling from adsorbed water onto the colloidal fraction, and in small part to the swelling of the clay minerals themselves (i.e. montmorillonite absorbing water within its structure). They note that swelling is not due entirely to the presence of clay. The first appearance of swelling rock in engineering literature was a general description of the weak behavior of clays and clays shales by Casagrande in 1949 (Casagrande 1949). In the early days of soil mechanics, dialogue and interaction among geologists and engineers was common at conferences under the auspices of the Federal Housing Authority, the Transportation Research Board and the National Research Council and the leadership of the foremost authorities in the soil mechanics field - Terzaghi, Peck, Casagrande, Bjerrum and others. Within the geologic profession there was a great enthusiasm for the application of geologic principles to engineering problems, and many papers presented in these types of symposia were gathered and printed. Case histories presented in these symposia trace the recognition and characterization of uncemented shales and the problems they presented for large civil engineering structures. In volumes such as "Applied Sedimentation" (1950), Terzaghi in discussing soft-ground tunneling defines for the first time the terms "squeezing ground" and "swelling ground." Within this same volume, Philbrick presented a paper "Foundation Problems of Sedimentary Rocks" (Philbrick 1950) emphasizing the heterogeneous nature of sedimentary rocks and the effect of geologic history on strength characteristics of rocks, wherein he reports on the swelling of the Fort Union and Bearpaw shale due to load removal.

Shortly afterwards, a case history study of the Bearpaw shale was published by Peterson (1958) as a classic case study of a swelling shale, in this case a heavily over-consolidated shale rebounding due to load removal. The Bearpaw shale, and other shales like it, is heavily over-consolidated due to the glacial ice sheets that were the equivalent of 2,000 feet of overburden. Stress relief due to the removal of these ice sheets is slow, causing gradual deformation of the rock in the form of creep and downhill movement in the Saskatchewan River Valley. When an excavation is made in these clay shales, the mechanics of stress relief are accelerated, and heaving of the foundation occurs. The long-term deformation is a combination of rebound and swelling of clay minerals (increase in moisture content). In the 1950s, there much of the attention was focussed on the understanding of clays and their effect on the composition and strength characteristics of rocks (Skempton 1953).

In the 1960s, swelling rock was noted in the construction and engineering contractors literature and case histories from several decades before were published ("At Oahe" 1959; "Structures" 1960) The majority of the case histories subsequently recorded are in Europe (Gysel 1977) and the Midwest (Lindner 1976; Silver, Clemence and Stephenson 1976; TRB 1981) where rock formations such as the Pierre Shale and the Mancos shale have caused damage to large engineering projects. Application of the concepts derived therefrom to California residential housing tracts began in the mid-1970s (Meehan, Dukes and Shires 1975) when housing damage in northern California was related to in claystone bedrock and the subsequent swelling in response to unloading. In the early to mid-1970s, several forums included swelling shale in the umbrella of topics to be discussed, but none addressed the topic of residential construction on expansive rocks. There is ongoing study of the Pierre shale formation, notable for its swelling problems, and applications to residential construction in Colorado by the Colorado Geological Survey (Higgins 1997; Noe 1997). Meehan makes a passing reference to expansive claystones within a discussion of expansive soils (Meehan and Karp 1994). However, no discussion within the professional literature has occurred within the geotechnical engineering community in California on this topic since 1975 (Meehan, Dukes and, Shires 1975).

3.0 Damage Caused by Expansive Rocks

Damage caused by expansive rocks is varied and depends on many factors that affect the mechanism and rate of swelling, discussed in section 5. In California's prevailing arid environment (rainfall = 10 to 20 inches/year), Tertiary age volcanic and marine rocks from hills in which the claystones are exposed as landslide-prone terrain flanked by clay-rich weathered colluvial soils. The Menlo Park-Woodside foothill area, with its alternating Eocene claystone and sandstone beds, is a typical example. There, temporary equilibrium of intact claystone beds is usually disrupted by excavation of overlying rock, which triggers gradual swell on the order of 5 to 15 % (Meehan, Dukes and Shires 1975). Swelling begins within a short time after excavation but can continue for many years after (tens of years), in part due to the dry environment.

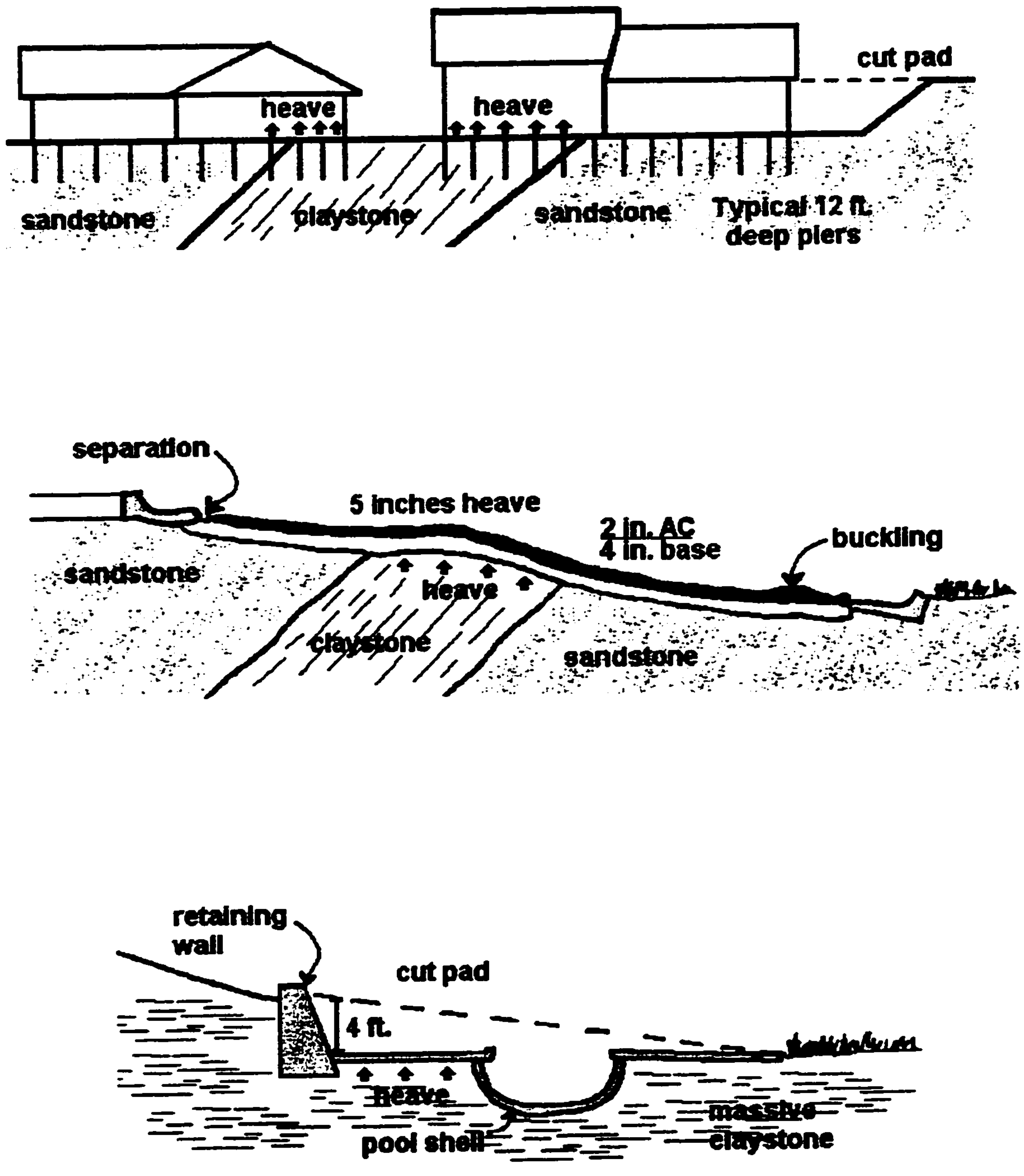


Fig. 3. Typical damage caused by expansive rock.

Fig. 3 illustrates typical damage that occurs to pools, roadways and foundations. The following case histories illustrate experience with expansive rocks in the two most heavily populated areas of California, the greater Los Angeles area and the San Francisco Bay area.

3.1 Case Histories

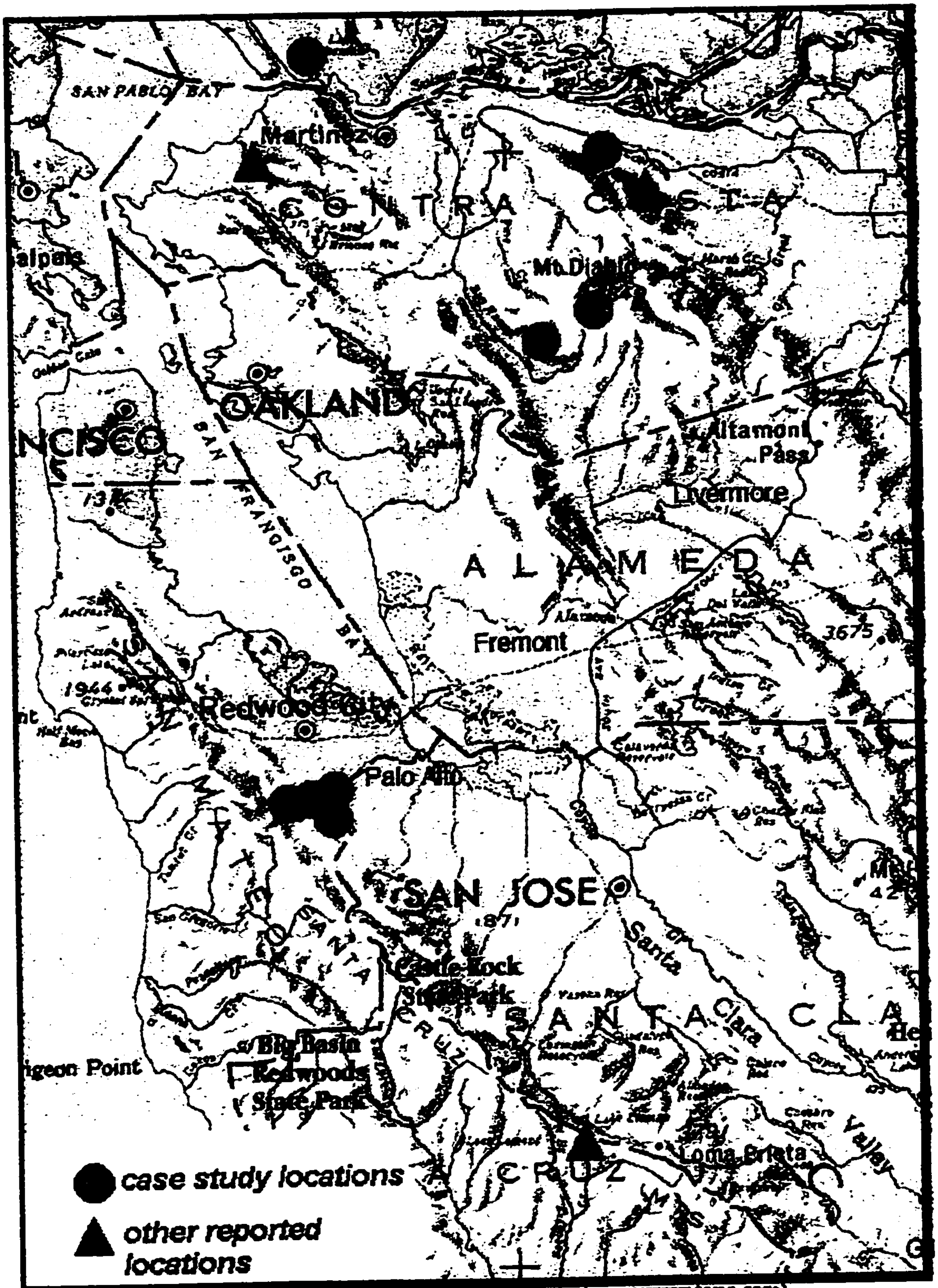
Table 2 lists the case studies discussed herein. These case histories include projects the author has investigated or studied in connection with consulting and/or litigation assignments as well as published and non-published case histories investigated by others. These case studies are organized and discussed by geographic regions. Also included in the discussion are other projects in areas that the author has observed signs of heave due to expansive rock or which have been identified as an area of potentially expansive rock in geotechnical investigation reports. Through these case studies, the author describes the damage, discusses the investigation and diagnosis, and attempts to learn what may have been done differently or what aspects of the expansive claystone problem were overlooked in investigation and design. This method of learning cannot be replicated in textbooks, and is used widely in teaching engineering students. It is especially valuable for expansive claystones, where until very recently there was no general professional on-going discussion of the problem with expansive rocks.

3.1.1 San Francisco Bay Region-Peninsula

Case studies 1-4 are located at the edge of the Coastal Range foothills (see location map, Fig. 4). The geologic mapping shows the area to be alluvial clays, silts, sands, and gravels of the Quaternary Santa Clara formation overlying the Eocene Whiskey Hill

Table 2. Case Studies

Case Study #	Region	Location	Development Type	Author's case	Published case
1	Peninsula, San Francisco Bay area	Sharon Heights, Menlo Park, CA	Subdivision	X	X
2	Peninsula, San Francisco Bay area	Single family home, Atherton, CA	small subdivision (~10 lots)	X	
3	Peninsula, San Francisco Bay area	Single family home, Atherton, CA	Individual lot	X	
4	Peninsula, San Francisco Bay area	2 single family homes, Woodside, CA	single lot	X	
5	East bay, San Francisco Bay area	Seeno Homes, Pittsburg, CA	Subdivision	X	X
6	East bay, San Francisco Bay area	Blackhawk Tennis Villas, Danville, CA	Subdivision	X	
7	East bay, San Francisco Bay area	Sycamore Mills subdivision, Danville, CA	Subdivision	X	
8	East Bay, San Francisco Bay area	Mariner Point and Crystal Point, Vallejo, CA	Subdivision	X	
9a,b	Southern Ventura County, Greater Los Angeles region	North Ranch, Thousand Oaks, CA	Municipal bldg. and subdivision	X	
10a-e	North Los Angeles County, Greater Los Angeles region	Santa Clarita, CA	Subdivision(s), commercial in progress	X	X
11	Los Angeles Basin, Greater Los Angeles region	Porter Ranch and Simi Valley, near Santa Susana mountains	Subdivision		X
12	Los Angeles Basin, Greater Los Angeles region	Chavez Ravine, Los Angeles, CA (Los Angeles Dodgers Baseball Stadium)	Commercial		X



Printed from TOPO! ©1997 Wildflower Productions (www.topo.com)

Fig. 4 Location of case studies 1-8, San Francisco Bay area.

formation (geologic map, Fig. 5), and exposed Whiskey Hill. While the younger Santa Clara formation is not known for containing expansive rocks, the older Whiskey Hill formation is made up of interbedded sandstone, siltstone and claystone beds, notorious for expansive beahvior. These rocks are marine in origin, although they formed during a time when volcanism was common in California during the Tertiary period (38 to 55 million years old). Beds of claystone within the hills above Atherton, Menlo Park and Palo Alto have been troublesome for light residential structures, public structures (e.g. Las Lomitas school, Menlo Park) and pavements. First documented in detail by Meehan, Dukes and Shires (1975), recognition and mitigation of swelling claystone beds continues to be problematic for recent construction.

Case 1: Sharon Heights, Menlo Park, CA

In the foothills of the coast range, about one to two miles from the San Andreas Fault, rapid development occurred above Menlo Park, adjacent to Palo Alto and Stanford University. A new planned development of over 300 single family, single-story ranch style homes was built and completed by 1970. Initially, in the era of increasing awareness of geologic hazards, fault rupture and slope stability were the principle engineering concerns. Homes were built on spread footing foundations and with traditional slab-on-grade garages; some homes have swimming pools. Within the first five years, expansive soils and downhill creep associated with the clay rich shales of the Eocene Butano formation (now known as the Whiskey Hill formation) caused cracking of strip footing foundations, concrete slabs, pavement, and concrete flatwork.

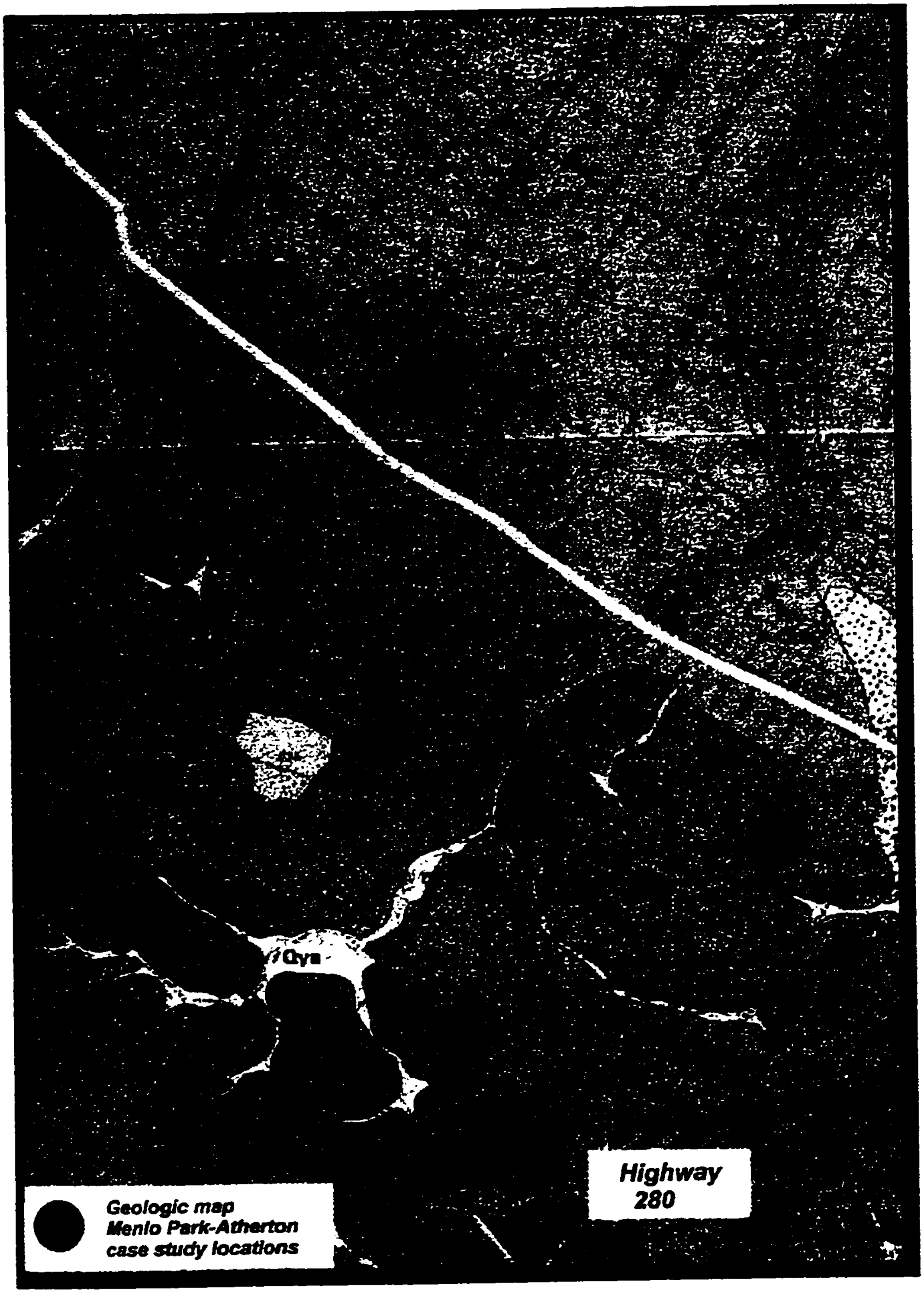


Fig. 5. Geologic map of Menlo Park and Atherton, case studies 1-3. Tw=Tertiary Whiskey Hill formation, QTss=Quaternary Santa Clara formation (from Pampeyan 1993).

Meehan, Dukes and Shires (1975) first studied the problems within the Sharon Heights subdivision in the early 1970s and published a case study in 1975. At the time, the damage to the subdivision due to moving ground was as follows:

Table 3. Estimated Damages, Case Study 1

Summary of Sharon Heights damage	**1975 $ Value**
22 of 300 homes show foundation distress	$600,000
6,000 of 100,000 linear feet of street and gutter curbing replaced	$100,000
Minor damage, accelerated maintenance lifecycle shortening	$200,000

Movement of and damage to the house foundations and other structures was attributed to the swelling of the claystone member of the Butano formation. Claystone is interbedded with sandstone in beds 5 to 10 feet thick, or alternately is massive and mixed chaotically with sandstone. The claystone has faint bedding, and is pervasively fractured throughout. Expansive clay soil three to five feet deep (locally referred to as adobe soil) covers both sandstone and claystone and also causes heaving of structures. A master set of joints cuts the rock at one to two foot intervals, with a subset of joints one to two inches apart. Properties of the claystone are shown on Table 4.

Meehan observed two houses built in 1972 with pier and grade beam foundations drilled 6 to 12 feet (Fig. 2a). The houses were each partially founded on a simple bed of claystone and a cut pad. The portion of the house on the claystone bed and at the interface experienced distress, including racking of doors and windows, cracking of foundation and stucco.

Table 4. Claystone Properties, Case Study 1

Property	Value	Comments
Atterberg Limits	LL = 70, PI = 50, SL = 15%	
Grain size	% fines =95%+ clay = 50%	
Total unit weight	135 pcf	
Moisture content	15-30%	Fluctuates seasonally close to surface
Clay type	100% montmorillonite	
Fine swell	100%+	
Swelling index	Cs = 0.10-0.12	0.10 for intact specimen, 0.12 – disturbed
Swell pressures	8,000-18,000 psf 18,000-20,000 psf	Natural water content Air dried

(after Meehan, Dukes and Shires 1975)

Based on observations in the field and lab, Meehan theorizes that even under minor load relief, significant swelling can occur at natural moisture content, on the order of one inch of heave for every foot of cut. If claystone has been dried out pre-development, and then to conditions become moist due to presence of a pool or irrigated lawn, even greater heave might occur. General observations of duration of significant swell suggested a five to seven year post construction period during which swelling of claystone is most significant.

An interview with the Menlo Park building department revealed a multi-prong approach to mitigating and preventing damage by expansive rocks, using construction methods and planning and building policy measures. Various mitigation measures documented by Meehan, Dukes and Shires (1975) include mixing lime with crushed claystone for use on building pads, injecting a limestone paste in drilled holes, or anchoring foundations on three to eight feet deep nominally (one #4 bar) reinforced piers and grade beam. None of these attempts at stabilizing a shallow foundation in expansive claystone were successful. Eventually deep, heavily reinforced foundations, 20 to 30 feet deep piers and grade beams came into use as a means of anchoring the foundation either below the zone of seasonal moisture change or through claystone into sandstone bedrock. Piers are belled to resist the heave exerted by claystone and clay soil on the upper portions of the foundation. One other method used was a deep lime stabilization pad; onsite material mixed with lime up to 10 feet deep, with conventional footing foundations. The city of Menlo Park Building Department reports this has worked moderately well, except around the edges of the lime stabilized zone, indicating that more distance was needed between structures on the lime stabilized pad and surrounding swelling rock.

Pavement damage mitigation measures include a five foot zone of lime stabilization along streets and also used of full paving. The pavement section for the street was modified from a typical several inches of AC over five to eight inches of base materials to a full eight inch thick AC section directly on grade. This prevents migration of water through the gravel base under the streets, which facilitates swelling and damage. Pavement observations indicate that this has worked moderately well.

However, recent observations of concrete piers installed as monuments in the streets of Sharon Heights, and ongoing foundation repairs suggest that while swelling may have diminished it has not ceased. One house has had its foundations replaced twice in 30 years, first with temporary pipe pile replacement and then permanent heavily reinforced concrete piers. Meehan, Dukes and Shires (1975) conclude that it is nearly impossible to prevent damage to pools, and that maintenance of expected substandard performance is cheaper than radical experimental preventative measures.

Damages to houses ran about five percent of market value in 1975, $3,000 to $5,000 for foundation repair, with house values at $60,000 to $100,000 (1975 dollars). However, today foundation replacement is typically $100,000 to $150,000 for houses valued at $800,000 to more than $1,000,000 or about 10 to 15% of house value. Even after the knowledge and experience gained by the City of Menlo Park and some local engineers, houses built in the same hills just north in neighboring towns also suffered damage from expansive rocks. The following case studies 2 through 4 are also located in the Menlo Park area in the Whiskey Hill formation.

Case 2: Orchard Hills Subdivision, Atherton, CA

Orchard Hills subdivision is located at the edge of the Coastal Range foothills (see location map, Fig. 4). The geologic mapping shows the area to be alluvial clays, silts, sands, and gravels overlying the Whiskey Hill formation (geologic map, Fig. 5). In the Orchard Hills subdivision, one of the lots was developed using a traditional cut and fill operation, with a 10 foot cut at the back, and a five foot fill on the front of the lot. The lot and subdivision were developed during the period 1987 to 1989. Preliminary subdivision and the individual lot geotechnical reports identified claystone in the borings, although they referred to the geologic map which showed the surficial geology as alluvial sediments. The soils report described stiff expansive "adobe" soils to a depth of three feet, stiff sandy clay to four feet underlain by severely weathered claystone and/or conglomerate bedrock to a depth of ten feet or more. Testing of the soil yielded plasticity indices ranging from 20 to 49% (moderately to highly expansive soils). Foundation recommendations were drilled pier and grade beam, with 12-inch diameter piers drilled 11 feet minimum into finished grade, and minimum eight feet into native material. Because the bedrock dips steeply across the lot, piers were founded in all claystone, part claystone or no claystone depending on the plan location of the pier. In 1990, the owners began to notice cracks in the walls and sticking doors in one corner of the house, which according to a floor level survey in 1992, had heaved 3.5 inches relative to the rest of the house (see Fig. 6).

According to Meehan, Dukes and Shires (1975), for every foot of cut, there should be a corresponding one inch of heave in claystones in this area. From Meehan's hypotheses, the expectation is that in claystone, the area with greater cut would have the most foundation movement. However, the area with greatest movement, about 3.5 inches heave over three years, is in the area with the least amount of cut, one to two feet.

The explanation for this seemingly non-conforming behavior is in local geologic quirks. Subsequent investigations by other geotechnical engineers and geologists revealed, upon examination of the bore holes and comparison with depth of piers, that the area with greatest cut had piers mostly founded in the sandstone inter-layered with siltstone/claystone. The area with greatest heave (and least cut) is claystone throughout the entire depth of piers. Another significant contributing factor is the type of claystone found across the lot. During the investigation of the damage home, careful descriptions were made of the rock across the site, indicating a waxy, fine claystone in the vicinity of heave, and silty claystone elsewhere. Testing of the claystone using the free swell test

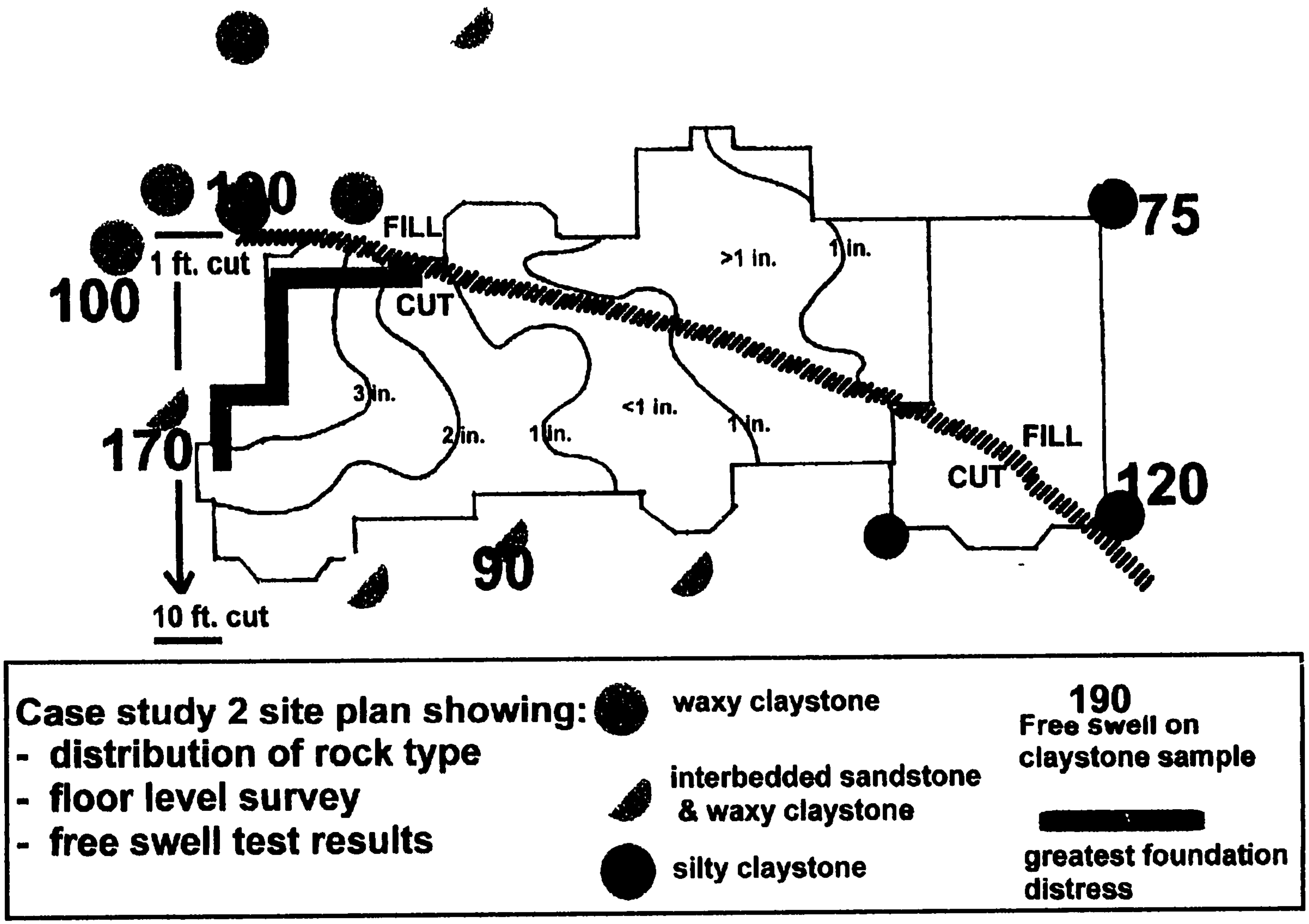


Fig. 6. Case study 2 site plan..

also indicated that the claystone in that portion of the lot had the highest values of free swell tests (100 to 190%) compared to the silty claystone (70 to 100%). The difference in physical composition of the claystone as indicated by description (waxy) and testing (high free swell values) explains the seemingly anomalous behavior of the claystone, where the greatest amount of heave occurred in the area of least overburden relief.

Foundation recommendations for different custom homes in this subdivision are pier and grade beam. However, depending on the engineer's experience with expansive rock, different recommendations were made. The house in case study 2 had eight foot deep piers lightly to moderately reinforced, whereas the house built next to it at the same time, had 15 foot deep piers with heavy reinforcing. Older houses in this area all have conventional spread footings. These different recommendations illustrate both a changing awareness over time of the need for more heavily reinforced foundations, and also a difference in level of awareness depending on the engineer's level of sophistication and experience. This case illustrates the geotechnical engineer's lack of geologic understanding and demonstrated the dangerous misuse of existing geologic information by those not trained in geology. The Orchard Hills case study is also a perfect example of one geotechnical engineer performing a preliminary geotechnical report in support of a tentative subdivision map application (planning stages) and a subsequent geotechnical engineer performing lot specific geotechnical investigation. The lot engineer was sued by the homeowner's insurance company and used, as a defense, reliance on the previous engineer's representation of rock formation found on-site. The subdivision report investigation included a review of a local geologic map, which reported one type of rock on the site, the Santa Clara formation, a rock formation not known for swelling. The report also contained several borings scattered across the site and noted that sandstone, conglomerate and claystone were found on-site. A later geotechnical report by a different engineer relied on the previous general subdivision report and the geologic map to report Santa Clara formation on the site, and also noted claystone found on-site. A geologist trained to recognize the geometry of rock formations and rock contacts would have recognized that the Santa Clara formation is a very thin veneer over the more expansive Whiskey Hill formation, the source of the expansive rock problems in neighboring Menlo Park.

Case 3: Fletcher Way, Atherton, CA

Just to the west of the Alameda de Las Pulgas and the Orchard Hills subdivision (see Fig. 3), in Atherton and Redwood City, the hills contain abundant claystone beds, which are now familiar to the building officials of these cities. In Redwood City, all residential foundations west of the Alameda must be pier and grade beam with minimum 15-foot pier depths (personal communication, Redwood City Building Dept. 1994). While foundation requirements such as these clearly have resulted from the recognition of expansive claystones, the behavior of these claystones is still not well understood as illustrated by this case.

The owners of an existing house decide to remodel a guesthouse and install a pool in the side yard on a slight hill. The hill was notched from one to four feet to make a flat pad for the pool and decking (as in Fig. 2c). The soils investigation for the pool was carried out in 1991, and recognized highly expansive soil and claystone. A subdrain was installed under geotechnical supervision to keep water from entering the pool foundation area and soaking the claystone. The pool was constructed in the spring of 1992 and by the following winter, heave at one end of the pool had occurred, one inch over one year in an area with a minimal one to three foot cut. When the claystone was tested, it yielded PI's of 29 to 44% and a modified free swell of less than 10 (which corresponds to a PI of 15 to 28%).

The soil engineer recognized the potential for claystone heave, but the mechanism assumed for swelling was misunderstood. The engineer assumed that the claystone would swell only if water were introduced into the claystone by man-made means. Ignored were other factors that contribute to swelling of claystones such as exposure of fresh rock, reduced overburden stress, and infiltration of moisture. The erroneous assumption was that as long as "extra" water from the pool or drainage did not reach the rock under the above conditions, infiltrating rainfall would not change the claystones' equilibrium.

In situations like this, it may be that avoidance of the troublesome claystone bed is the best solution. If there is no time pressure, an alternative might be to cut the slope and let it swell for a year or so, but the deeper excavation for the pool itself is problematic. However, it has been reported that claystones in this area continue to swell and move after many years. Meehan reports (personal communication, 1995) that after replacement of foundations with 30 foot heavily reinforced concrete piers, some houses in Sharon Heights described in his 1975 paper are still experiencing some movement, although within tolerance limits. The implication is that had this house not had heavily reinforced foundations installed the movements after 20 years would be enough to cause severe cosmetic and perhaps structural problems.

Case 4: Audifferd Lane, Woodside, CA

Woodside, California is a semi-rural town set in the foothills of the Coast Range behind Atherton and Menlo Park. Horse trailers, ranchettes, and custom homes dominate the oak covered rolling hills. Two custom homes were recently completed construction in 1990 on side by side lots. One home was mildly distressed on one corner (lot A), the second home (lot B) was severely distressed through the central portion of the house and was investigated by several engineers during litigation (Fig. 7). In 1994, the author was asked to investigate the home in connection with allegations that installation of a pool in 1992 had caused the whole problem by directing water under the house. Observations at the site and floor level surveys conducted by other consultants showed the central portion of a U-shaped ranch style house on lot B bowed upward approximately five inches. Interior sheet rock cracking and door and windows were racked out of level, and exterior stucco and some concrete flatwork were also cracked beyond what would be considered normal shrinkage cracking. While the driveway pavement showed no severe signs of distress, the court at the end of the street had a slight bulge near the center and damaged pavement typical of expansive claystones, AC paving cracked and distorted upwards (Fig. 8). The house on lot A suffered less distress, approximately one to two inches of heave across one corner of the house, apparently aligned with the presence of claystone and portion of the lot that was cut.

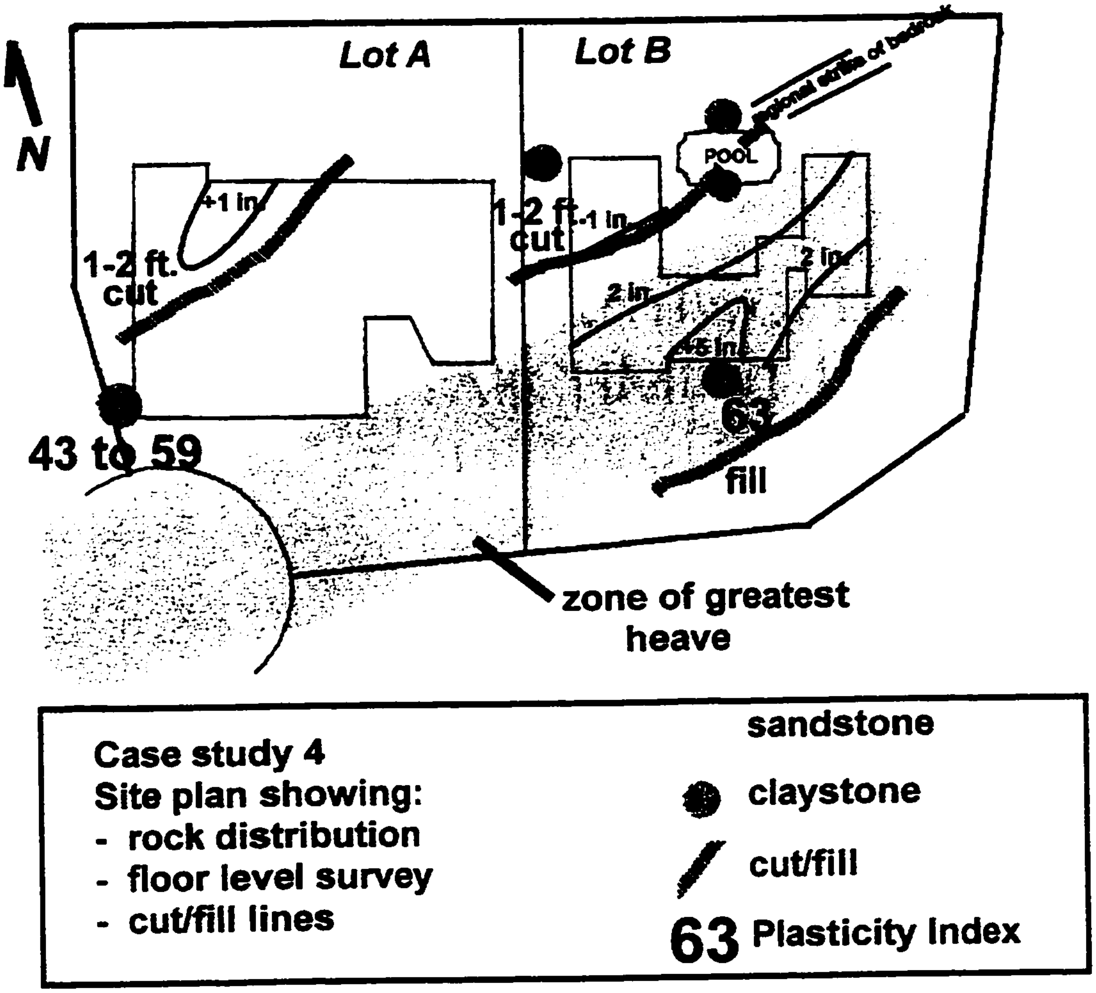


Fig. 7. Site plan case study 4. Water ponded beneath the floor in the area of the +5 in. floor level contour on lot B, causing the claystone to swell. On lot A, the greatest heave occurred in an area of cut.

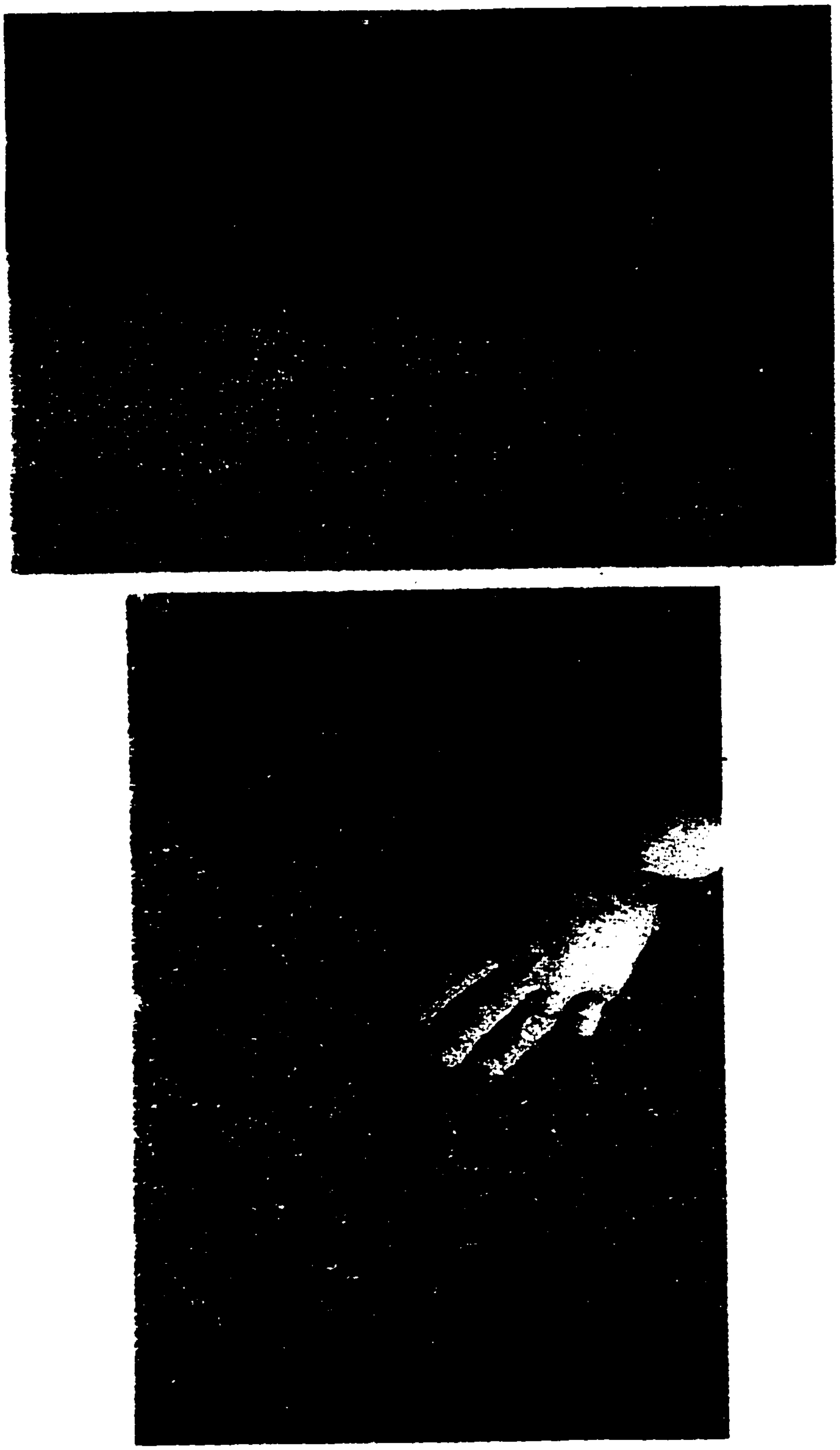


Fig. 8. Damage to the cul de sac pavement on Audifferd Lane. Entire area heaved upward, with cracks in pavement averaging one inch in width.

In 1987, the parcel was subdivided into two lots for development of these custom homes. A typical two-boring basic geotechnical investigation was conducted for the parcels, although both an engineering geologist and soils engineers were involved in the investigation. Eight feet of critically expansive soil was encountered in Boring B-1 (LL of 92%, PI of 67%), overlying claystone bedrock. No tests were performed on the claystone. Interbedded sandstone, siltstone, and claystone was found in Boring B-2, typical of the tertiary Whiskey Hill formation which contains expansive rock. No geologic mapping was performed, although a USGS map was consulted, indicating Santa Clara formation on the site. Santa Clara formation is a Quaternary alluvial formation consisting of interbedded sandstone, conglomerate, and claystone, and is not known for being particularly expansive.

In 1989, two custom homes were constructed on the two parcels, and in the summer of 1992, a pool was installed in the open portion of the U-shaped house on parcel B. The main foundation concerns of the geotechnical engineer were soil creep and expansive soils. Pier and grade beam foundations were used for both houses, parcel A on 12-inch diameter piers eight feet deep; and parcel B, closer to the hillside and partially on colluvial soils, on 12-inch diameter piers 11 to 16 feet deep. Both lots were slightly cut one to two feet on the northwest corner of the building footprint (see Fig. 7). However, when the pool was constructed, the soil engineer noted expansive claystone bedrock in the pool excavation. The soils engineer recognized the potential for heaving of the pool if the claystone became saturated and recommended an 18-inch deep subdrain system around the perimeter of the pool deck area to prevent water infiltrating the deck slab and into the pool subgrade.

Shortly after the houses were occupied, the house on parcel B began to show signs of cracking in 1991. The kitchen and front entrance area on parcel B showed signs of racking and distortion (Fig. 9), and the hardwood floor in the house began to pull apart. Experts for parties involved in litigation concluded that the pool had leaked, or the pool contractor had left an opening in the formation through which water could enter and become trapped beneath the foundation, causing soils to swell, or fill to compress. In fact, a claystone bed was located beneath the center portion of the house, flanked by sandstone on either side. Several possible pathways for water to saturate the claystone existed, through the more permeable sandstone, through fractures and joints in the claystone (not uncommon) or through the colluvial soils that filled a subsurface topographic depression near the center portion of the house on lot B. The piers that originally were supposed to be eight feet deep were extended in depth through the fill and colluvial soils and founded in the top of claystone bedrock. They were deep enough to be affected by the swelling of the claystone, but not embedded far enough to resist uplift. Damage to the house on parcel A can be attributed to the presence of claystone, the tendency of the flat back yard at the base of a hill to collect water and a small one to two foot cut, the greatest cut corresponding to the area of greatest heave. In the litigation over damage to the house on parcel B, the engineers and geologists disagreed as to the mechanism of damage. Some thought perhaps a portion of the fill was settling, others held the opinion that the swelling was shallow, due to expansive soils. Other experts

Fig. 9. Lot B kitchen area exterior cracking. Wide arrows show cracks of greater than 0.25 inches, thin arrows indicate sense of shear.

expressed the opinion that water was migrating vertically down into the claystone and causing swelling, and thereby damaging the house foundations. The presence of water under the foundation was largely attributed to a hole in the foundation wall made by the pool contractor when piping was installed. However, the back and side yards that collected drainage from uphill, were extremely flat, and were landscaped with a heavily watered lawn and other plant materials not typical of semi-arid northern California. The homeowner on lot B was demanding foundation replacement, repair to the interior and exterior damage of his home, as well as moving and storage costs. This totaled $675,000, approximately the cost of the lot ($312,500) and construction costs for the house ($369,000).

3.1.2 San Francisco Bay Region-East Bay

The East Bay region of the San Francisco Bay area has seen the most explosive growth over the last two decades, both in affordable tract homes such as those constructed along the I-80 corridor in Hercules, Antioch and Pittsburg, and custom exclusive homes such as Blackhawk Ranch in Danville. This area along the foothills and in valleys surrounding Mt. Diablo and along the upper reaches of the bay into the delta is known for landslides (hillsides) and expansive soils (valley floors).

Case 5: Seeno Homes, Pittsburg, CA

During the mid-1970s through the early 1980s, one builder in the foothills behind Antioch and Pittsburg (Fig. 4) developed a group of eight subdivisions. These houses were three and four bedroom tract houses, typically with five set floor plans. Grading of the subdivisions was typical cut/fill designed to level the ridges and fill the swales and valleys, with cuts up to 50 feet and fills also up to 50 feet deep. Foundations were a combination of shallow pier and grade beam (typical pier depth of five feet) or conventional footings and slab-on-grade. In the early 1980s, after several of the subdivision phases were finished, complaints began about cracking slabs, sticking doors and racked frames, typical of fill settlement and/or expansive soils. In some cases, the developer attempted to repair the damage for the homeowner. In other cases, the developer bought back the house, repaired it and either rented or resold it.

Several houses further up in the hills, and developed on cut lots, showed signs of differential movement. Because they were on cut lots, this at first did fit the typical pattern of soil damage elsewhere in the subdivision. Fig. 10 shows the location of the houses. Lot A was investigated independently by the author before involvement or awareness of the larger subdivision problems. The center and garage portion of the house had heaved by several inches above the eastern portion of the house and there were signs of distress in the interior center in the form of cracked and separated sheet rock and out-of-level floors (Fig. 11).

While the investigation was in progress on lot A, a larger subdivision investigation began which focussed on five test "cases" or houses. Lot B was one of the chosen test houses, and located immediately southwest of lot A. The western portion of this house was also elevated with respected to the rest of the house. Beneath the house, the posts supporting the floor joist were all tilted top towards the east, and the western portion of sub grade beneath the house was also several inches higher than the eastern portion of the sub grade (Fig. 12). Fig. 13a is a photograph taken from the street in front of lot B,

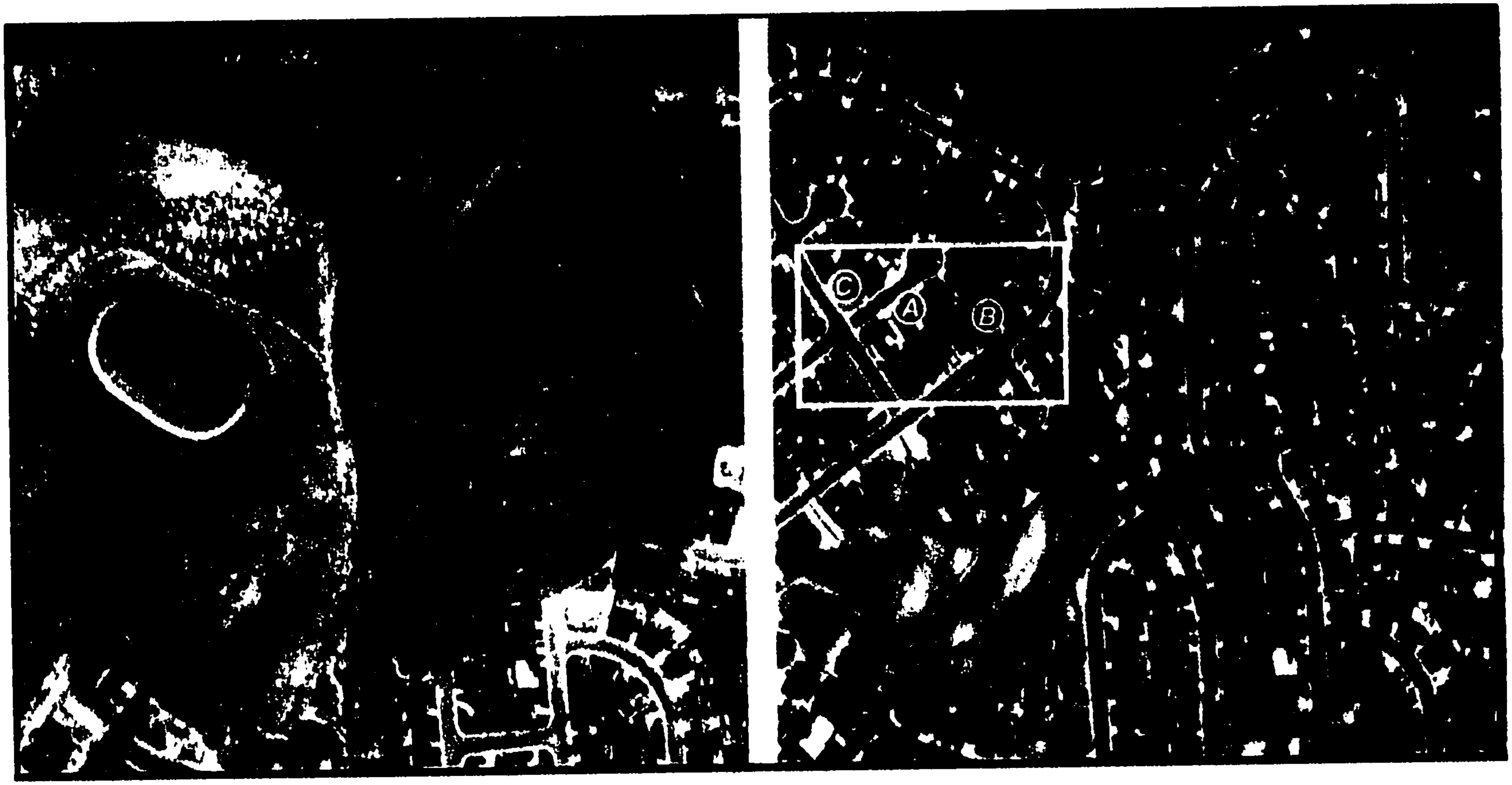


Fig. 10. Aerial photographs, case study 5. The photo on the left was taken before development and shows the old reservoir. The photo on the right shows the site after development, and the locations of lots A, B, and C.

Fig. 11. Lot A, initial house investigated by the author. The center portion of the house is up relative to the right side, as can be seen from the front of the garage, and shown by the arrows.

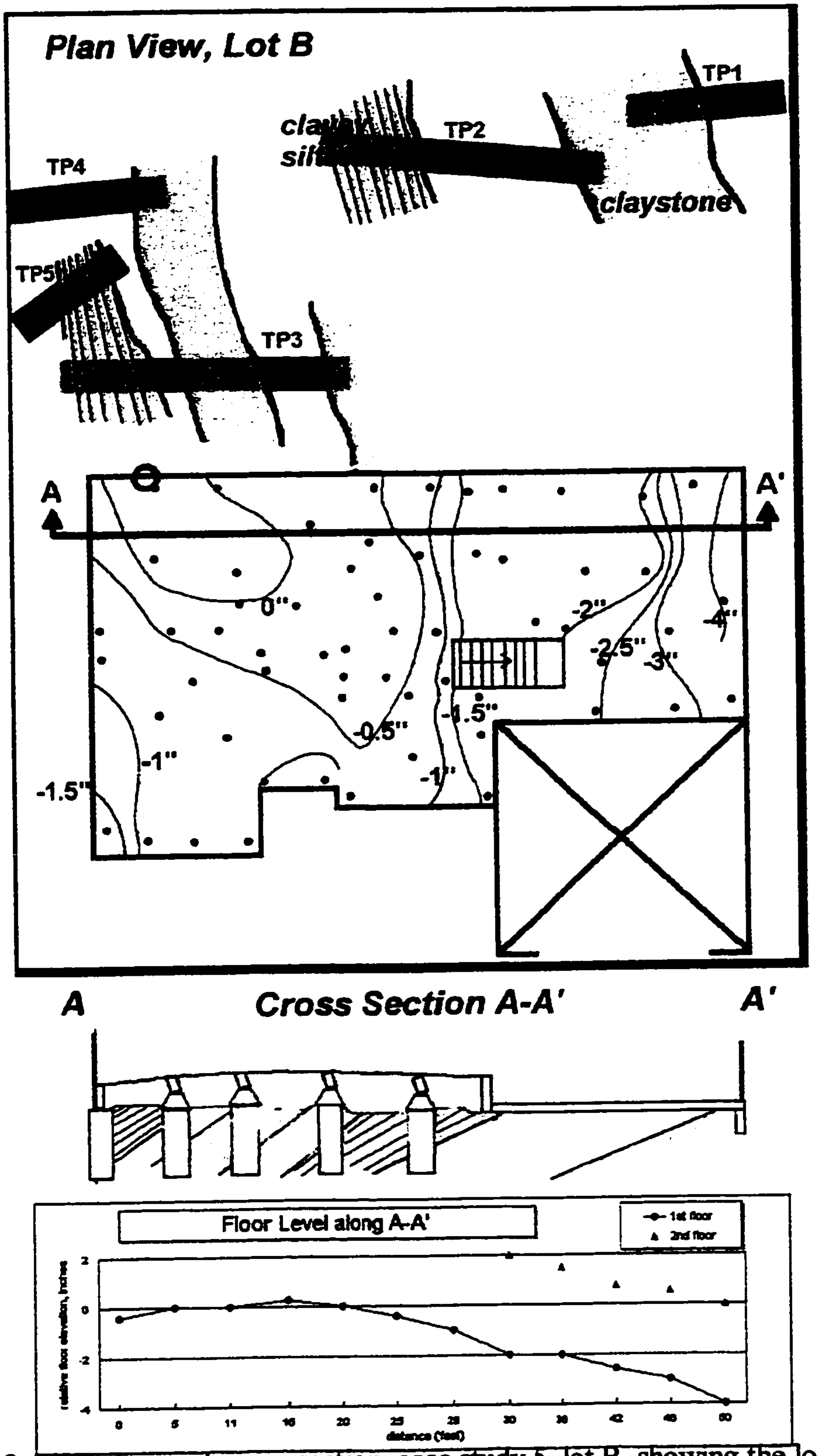


Fig. 12. Plan view and cross-section, case study 5, lot B, showing the location of the expansive claystone beds relative to the house and to the floor level contours.

Fig. 13. Lot B house and street damage, and relationship to location of claystone bed. Water is ponded in front of lot B's driveway due to the heaved gutter and pavement.

and shows the misaligned roof (the gutter drains the wrong way) and the location of the claystone bed responsible for the heave. Fig. 13b shows the disturbed street drainage (ponded water in gutter) in front of the house.

Driving the streets of the neighborhood revealed humps in the road, approximately five to seven inches wide that lined up with disturbed curb and gutter as well as misaligned roofs. Trenching by other consultants during investigation revealed interbedded siltstone, claystone, and sandstone of the Lawlor/Tehama formation. The movement beneath lot B was apparently along bedding plane direction and behaved as a thrust fault. An old military reservoir had occupied the site, with 50 feet of combined material removal between the reservoir and native material. The site stood empty for a year before construction began. Post construction movement was on the order of two to three feet, two years after the lot was cut. Fig. 14 shows the relationship of the old military reservoir; lots A, B, and C; and the mapped claystone beds.

Discussions with other homeowners in the area revealed ongoing damage to the southwestern corner of the house on lot C, a fact none of the consultants involved in the investigation knew, but which might have been guessed at, given the traces of the heaving claystone beds in the streets. Lot C is still experiencing heave that is distressing the house, over 20 years after construction.

Other homes in Seeno Homes subdivisions in the hills contain expansive claystone damage, like the Lean residence, developed on a cut lot amidst others stair-stepped down the hillside. A recent visit by the author to inspect the subdivision in July 1997 revealed other areas where claystone beds crossed the streets, although some of the evidence is

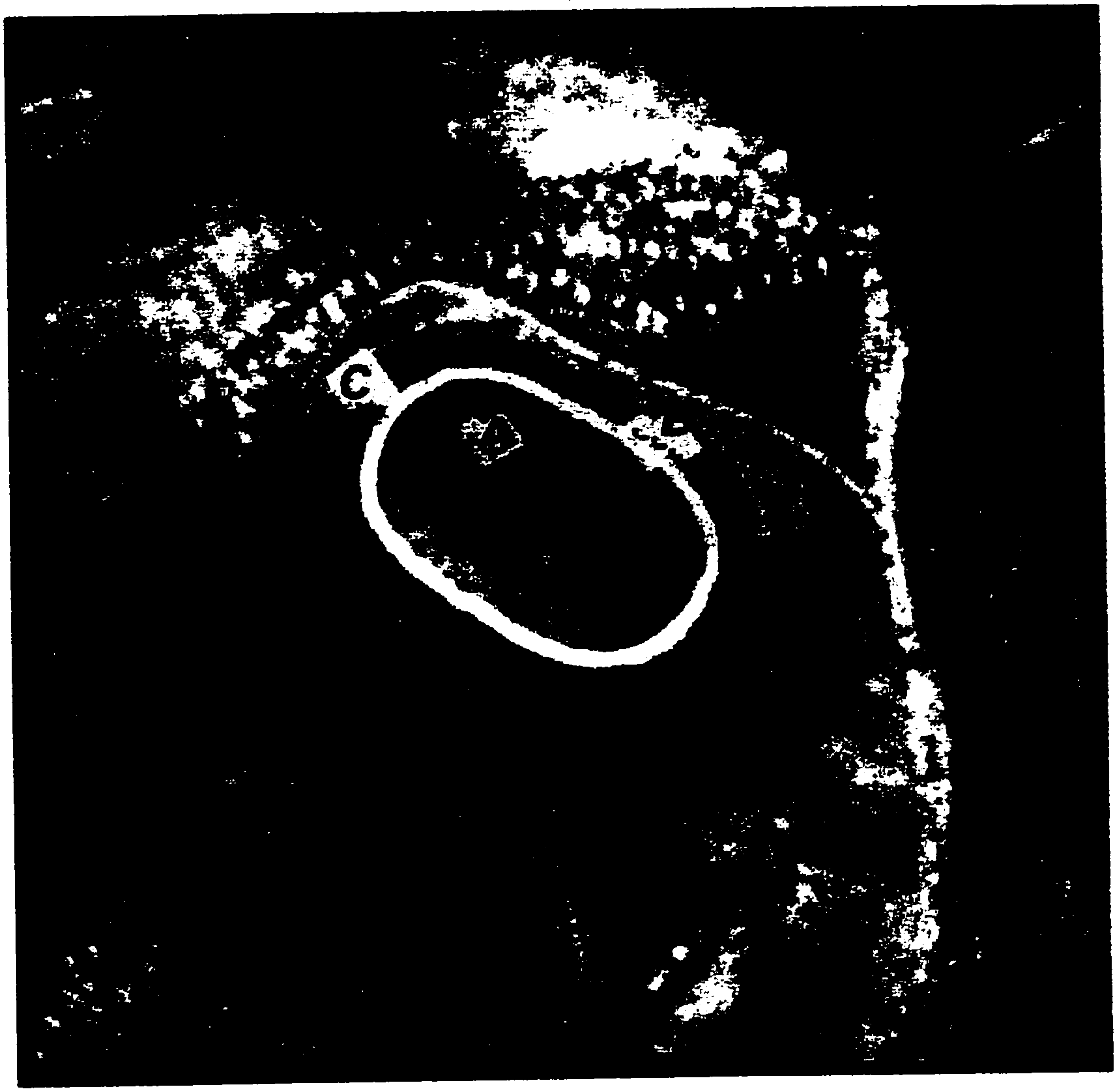


Fig. 14. Relationship of lots A, B, C, claystone beds (shown by red crosses) and old military reservoir (shown by white ellipse).

quite subtle. If a claystone bed is present, often at the crest of a hillside street, a slight hump is perceptible. Inspection of the curbing and sidewalk on either side of the street may provide additional evidence. Sometimes the hump in the road is a relic of grading or fill settlement, so it is important to note whether the area you are inspecting is cut, fill or a transition area. If this cannot be discerned from the surrounding terrain, aerial photographs are the most reliable guides, particularly if there are pre-grading and post-grading photos available.

Case 6: Blackhawk Tennis Villas, Danville, CA

Blackhawk Ranch was developed as luxury bedroom community in the foothills of the eastern coastal ranges in the San Ramon Valley (Fig. 4). The rocks that form the rolling hills of the Blackhawk Ranch area consist of the Tertiary Orinda and Neroly formations, steeply dipping interbedded sequences of sandstone, siltstone and conglomerate that cut across smaller ridges which extend into the valley at right angles (Fig. 15). Blackhawk Tennis Villas, a 30-home spa and tennis club development, were constructed straddling a valley and adjacent ridge on the west side. Site reconnaissance investigations were performed in 1973 and 1975 in conjunction with other adjacent development investigations, and the original soil investigation for the development was conducted in 1977. Foundation recommendations in 1978 by a structural engineer, called for seven foot deep piers on four foot centers with light reinforcing, four #4 bars down the center of the piers. The basic development scheme was typical cut/fill on a large scale, using the material cut from the top of one ridge to fill in the valley. Grading took place in 1978 and homes were constructed in 1982-1983 in a semicircle, fifteen of them on cut lots and the remaining on deep fill (Fig. 16). During 1982, 1983 and 1986, the slopes behind some of the lots had debris flows occur during the heavy rainstorms (Fig. 17).

In 1985, progressive distress was noted in the houses. Some houses on cut lots were beginning to experience heave, and houses on cut/fill lots were experiencing either heave or fill settlement, along with a host of other typical construction problems. This case history focuses on the houses on cut lots where the fill thickness is less than pier depth. These are the houses that apparently experiencing heave, as opposed to possible settlement of the deep fill, and made up mostly of fat clay (CH/CL) material. Table 5 summarizes information about the cut lots, estimated cut, rock type, engineering characteristics of the rock and differential movement observed in the houses.

The soil investigation for the subdivision recognized the presence of swelling soils on-site and recommended post-tensioned slabs on fill lots and pier and grade beam foundations on other lots. Pier and grade beam recommendations were for 10-inch diameter piers not less that five feet deep with a maximum allowable skin friction of 500 psf and greater than three feet of spacing on-center. Interior isolated piers were only to be used on lots where the PI<15%; and where the PI>25%, deeper, higher capacity piers were advised. However, no Atterberg Limits tests were performed prior to construction, and only a few Atterberg Limits tests and swell tests were performed on fill during grading of the tennis courts section of the development. These tests indicated the fill material used at the surface of the tennis courts had low PI's. There were no tests

Fig. 15. Typical interbedded sedimentary rocks, Orinda formation, Blackhawk Ranch, Danville, CA.

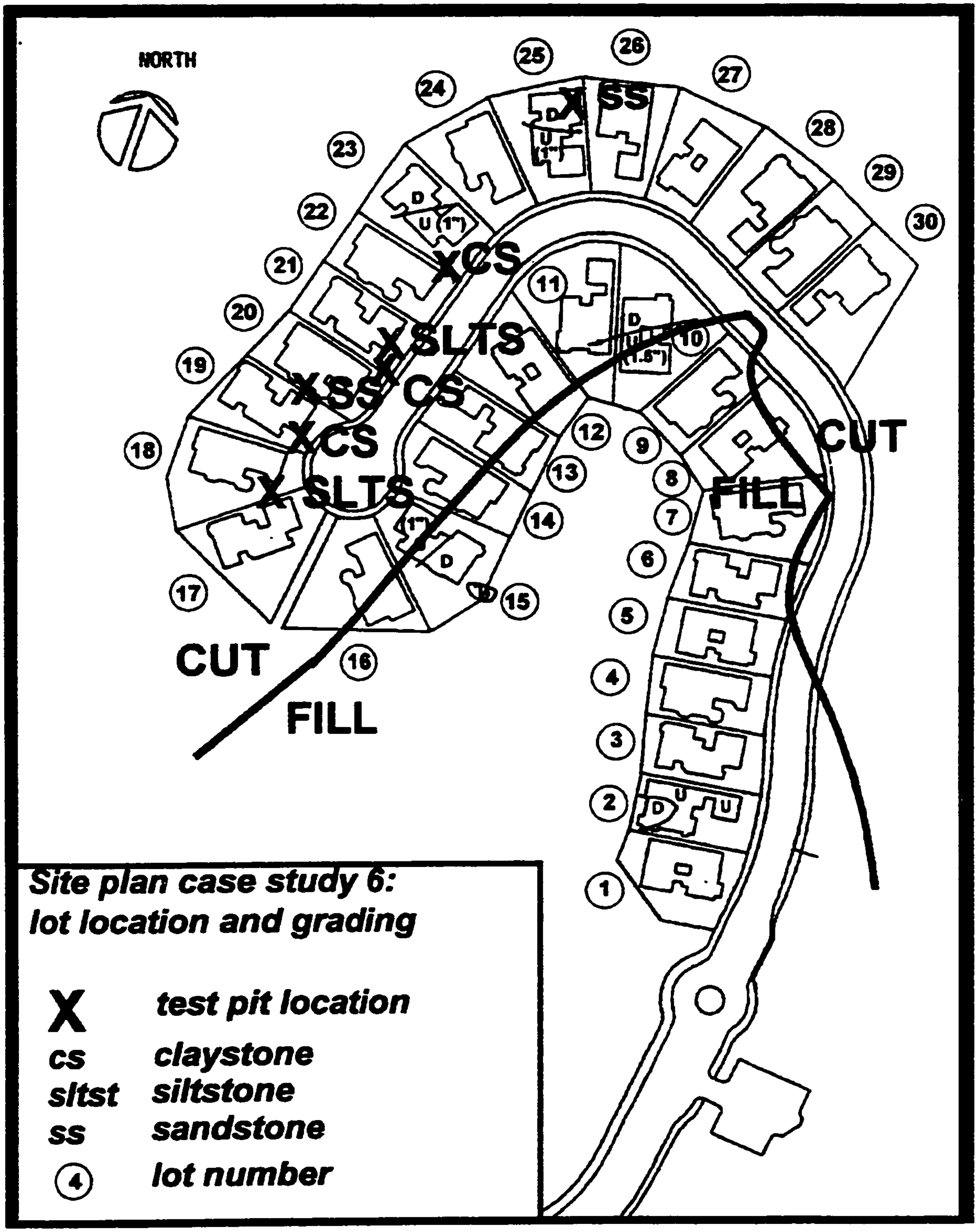


Fig. 16. Site plan, case study 6.

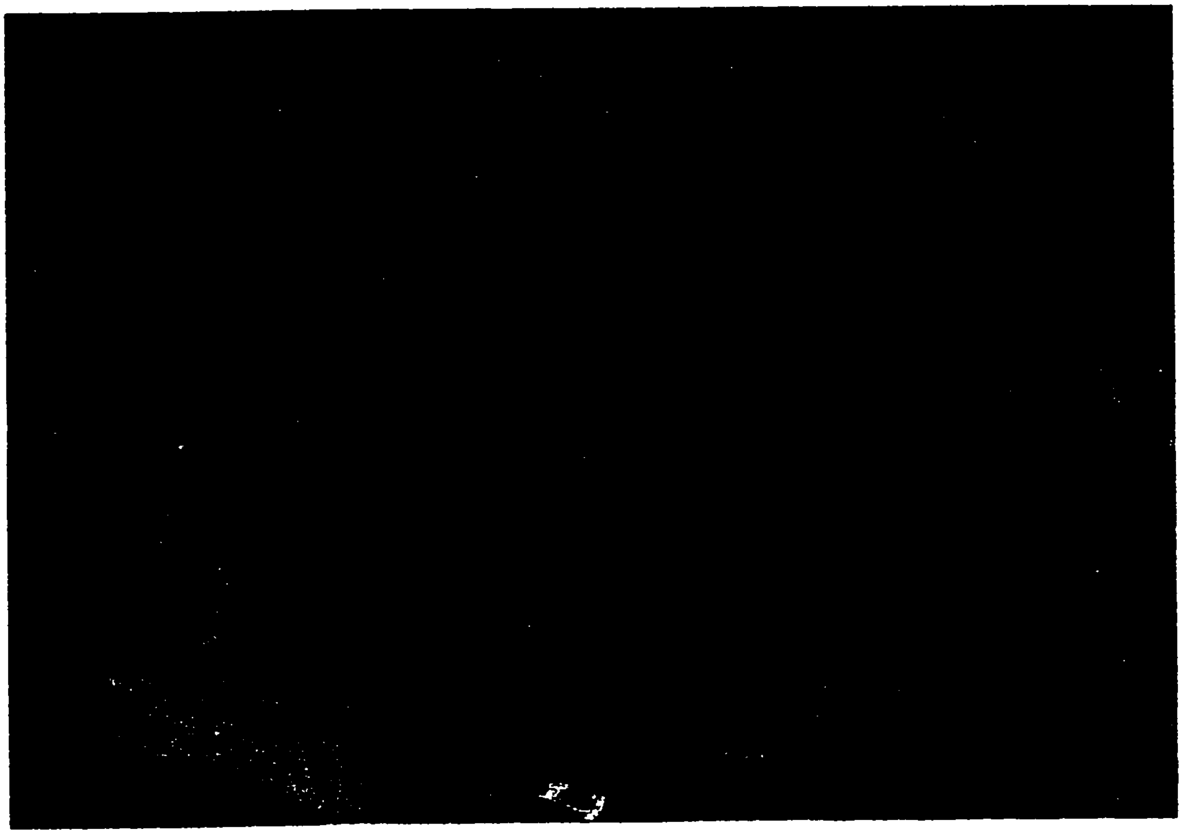


Fig. 17. Oblique aerial view, looking northeast, case study 6.

Table 5. Summary of Cut and Heave, Case Study 6

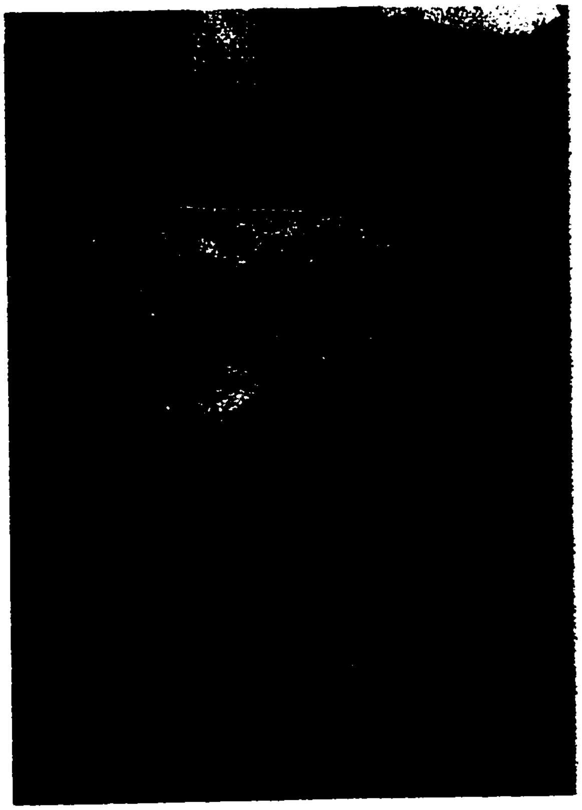
Lot #	Maximum Cut (feet) Field est.	Pre-grading Topographic feature	Rock type noted on lot, preconstruction	Soil in boring, Post-construction investigation	PI	LL	Differential movement (1987)
11	33	Swale	Gray green clay shale	NA	NA	NA	NA
12	41	Swale	Gray green clay shale	NA			1.0
13	41	Nose of ridge	Sandstone/coarse siltstone	Waxy siltstone and hard gray v.waxy claystone	15-23	38-58	2.0
17	55	Slope	NA		NA	NA	NA
18	54	Knob	NA	Siltstone	15-26	37-44	NA
19	50	Knob/swale transition	NA	Clayey siltstone/silty claystone	16-23	38-49	2.1 inches
20	53	Knob/swale transition	NA	Sandstone	NA	NA	2.0 inches
21	54	Swale	NA		NA	NA	NA
22	51	1/2 knob		Siltstone/claystones	21	42	1.9 inches
23	46	1/2 knob	NA	Lean claystone (slickensided)	20-24	44-51	1.5 inches
24	44	Swale	Dk gray grn claystone with thin sandstone/siltstone interbeds	Silty lean claystone	15	38	NA
25	56	Swale	NA	NA	NA	NA	NA
26	56	Knob/swale transition	Sandstone interbedded w/siltstone	Clayey sandstone/silty claystone	17	39	NA
27	42	Knob	NA	NA	NA	NA	NA

documented in the soil investigation report or the soil engineer's communications with the developer that supported the soil engineer's own foundation recommendations.

Post-construction investigation soil tests were conducted on the surface soil of the fill lots, Atterberg limits and swell tests, indicating that at standard pressures of 500 to 1,000 psf, swell was on the order of 3 to 10 %, and PI's of 24 to 34%. The fill material was a fat clay (CH) and had begun to creep and settle; the house with the greatest differential settlement was on a lot with the deepest fill in the development. However, fill settlement had nothing to do with houses on cut lots, which showed signs of distress as well.

One of the houses (cut lot) was completely torn apart during destructive testing, and foundation piers excavated. Pictures of the piers as constructed compared to the piers as designed illustrates graphically the difference between the what is imagined in the engineering office and what is actually constructed in the field (Fig. 18). These pictures show offset piers, eccentric loading of the footing and interior piers and mushrooming of pier tops; all of these are very common in housing construction and often contribute to worsen differential movement in areas with expansive rock and soil. Often in pier and grade beam foundations, the piers are designed for minimum depth; when constructed sloppily, there is greater surface area for swelling soil and rock to push up against vertically, and therefore greater pier movement. It is possible to predict the location of claystone beds by examining aerial photographs of the area, which show characteristic swales often containing debris flows where the weaker claystone is almost melting away like ice cream on the hillsides. Mapping of debris flow scarps show the preferential presence of claystone beds (Fig. 19), and examination of the logs of test pits and soil borings compared to topographic expression on Fig. 20 confirm this. Few pre-construction and post-construction tests were performed on the soils themselves, and with the exception of one or two houses, it is inferred that swelling rocks caused the damage. It may be a combination of stress relief (due to load removal) and swelling, due to the fact that most houses are not severely out of level (Table 5), and the fact that beneath homes experiencing heave, both claystone and siltstone have been found, albeit a very waxy siltstone. The strongest evidence is that the Orinda formation is notorious for its claystone beds, the cut lots have had over 35 to 50 feet of rock removed from them, which is an overburden relief of 3,500 to 5,800 psf. Swell tests suggest that this could lead to 5 to 15% expansion.

Recent observations of the subdivision show distressed pavement in cut areas in the form of general cracking in the center of the street and AC paving pulled away from the PCC curbing (Fig. 21). A slight hump across the street on lot 9 lines up with a slightly misaligned garage entryway (Fig. 22) where the left side of the garage entry is up relative to the outer portion of the garage. Examination of the grading plan (Fig. 16) shows this is a transition lot, from cut on the left side of Fig. 22 to fill on the right side. In addition the perpendicular orientation of bedrock strike to this cracking and offset lead the author to conclude that this displacement is due to the transition from cut to fill in this area, and not expansive rock.



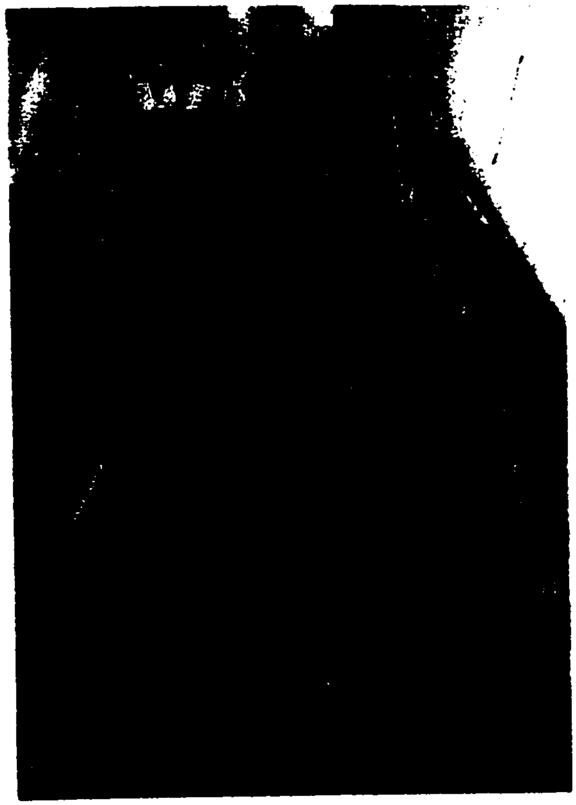


Fig. 18. Pier conditions, lot 20.

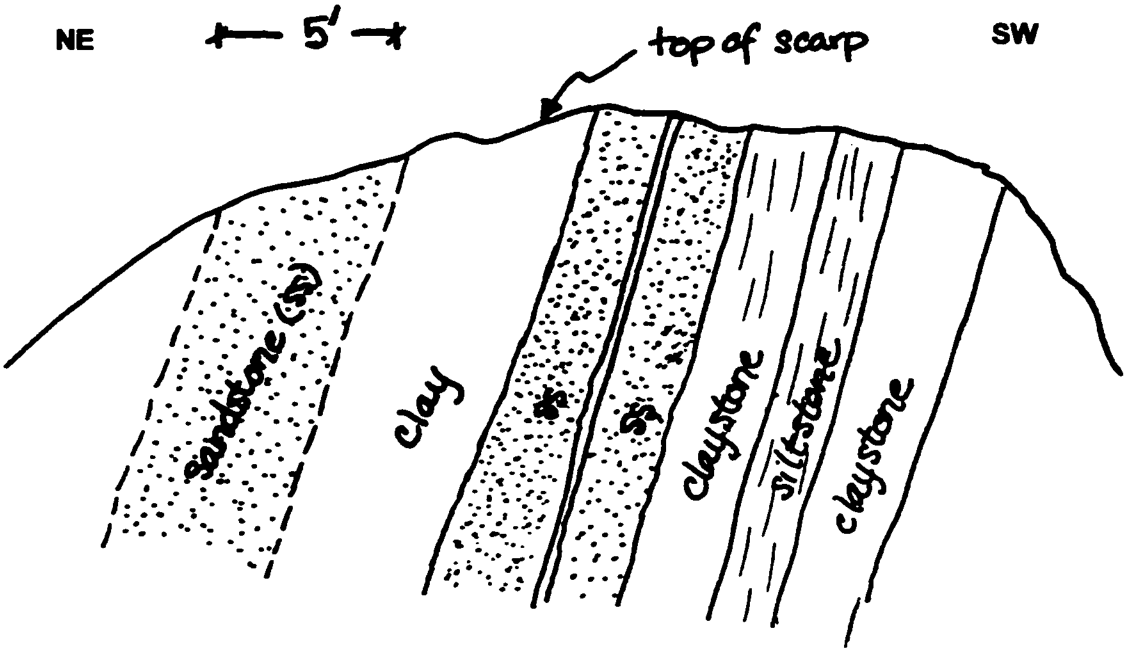


Fig. 19. Log of debris flow scarp showing interbedding of different rock types.

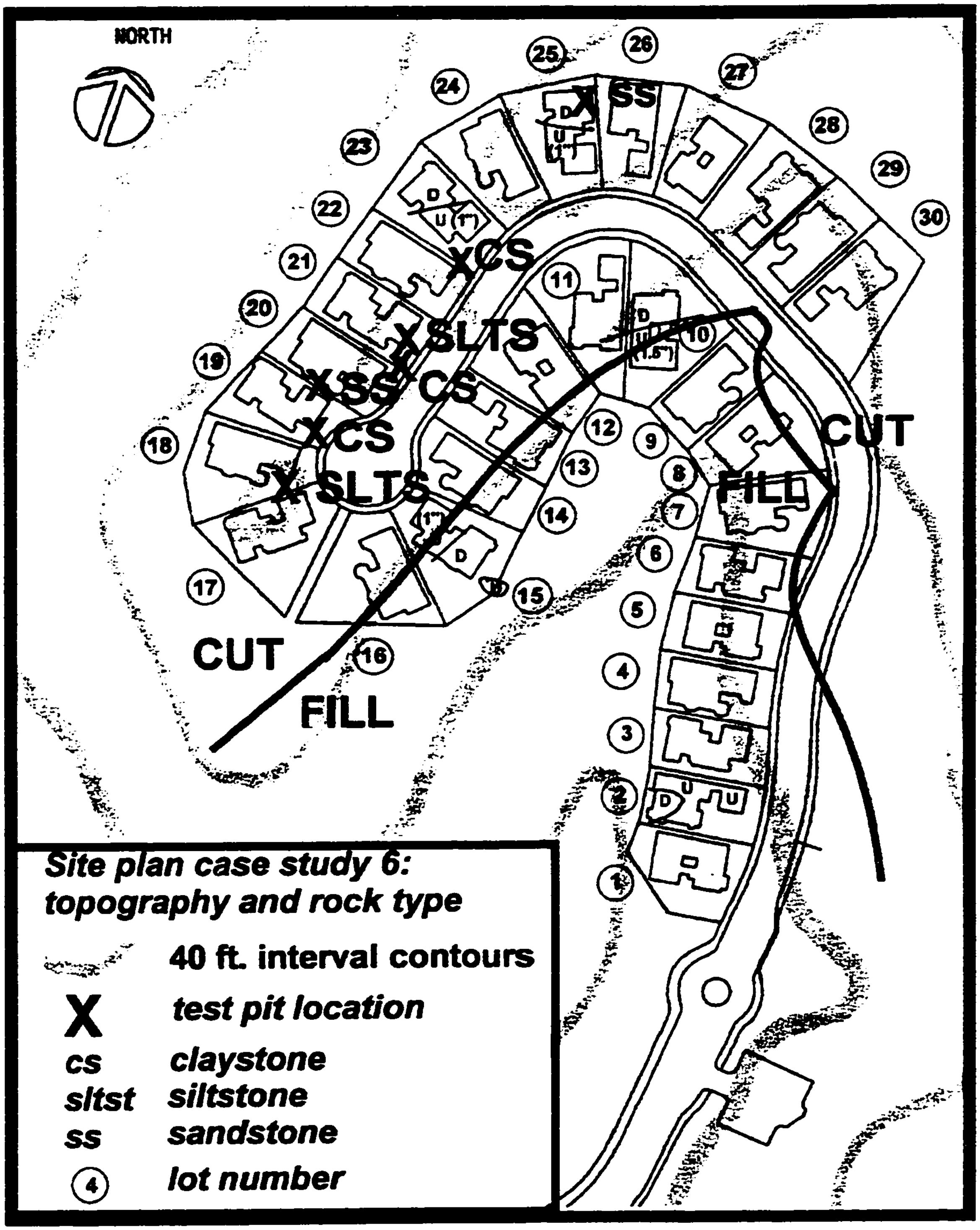


Fig. 20. Case study 6 site plan, showing pre-grading topography and relationship to rock type.

Fig. 21. Pavement damage due to expansive rock in front of lot 20.

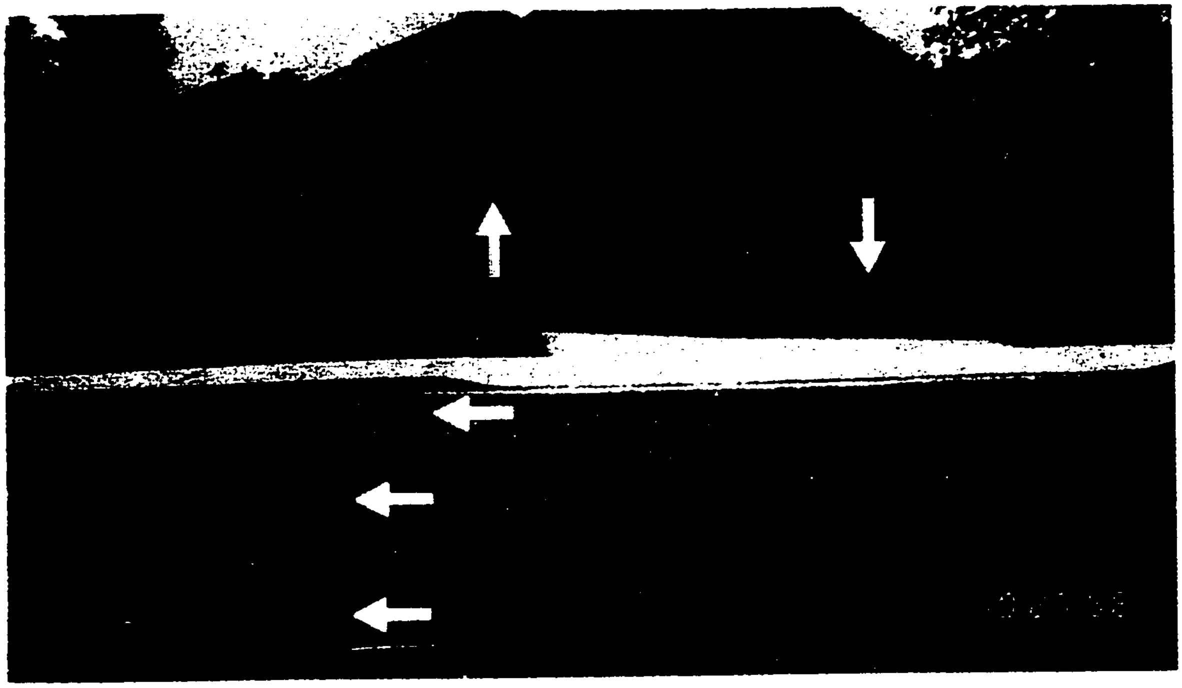


Fig. 22. Cracking along cut/fill transition due to fill creep.

Case 7: Sycamore Mill, Danville, CA

The Sycamore Mill subdivision in Danville California is located at the base of a rounded ridge that is made of folded and inclined beds of sandstone, siltstone and claystone of the Orinda formation (see Fig. 4). In 1976, a soil investigation was performed for a subdivision of single family homes. Some of the lots in the hills were created by cuts of up to 50 feet. The claystone encountered was an olive gray sandy claystone with gypsum and caliche (calcium carbonate) evident in the cracks and joints of the claystone. Test pits excavated into unweathered claystone revealed highly slickensided joints. The gypsum and jointed claystone are evidence of swelling potential either through stress relief (rebound), swelling of clay minerals; or gypsum crystal growth.

The engineering geologist projected the claystone across the subdivision and concluded that at least seven lots would be affected, and that the claystone should be over-excavated by four feet and a blanket of non-expansive fill placed. The intent of this was to keep the claystone bed out of contact with the foundation and ostensibly below any seasonal changes in moisture. However, the engineering geologist did not take into account long-term stress relief, that is the unloading of this claystone bed of 20 to 50 feet of overburden as well as exposure of fresh claystone to conditions that accelerate weathering and formation of gypsum crystals in the joints.

Currently, pavement in some areas of this subdivision is experiencing significant distress, as is the PCC flatwork (Fig. 23). While there are some slight humps in the road that appear to be aligned with the general direction of bedding, there are no obvious

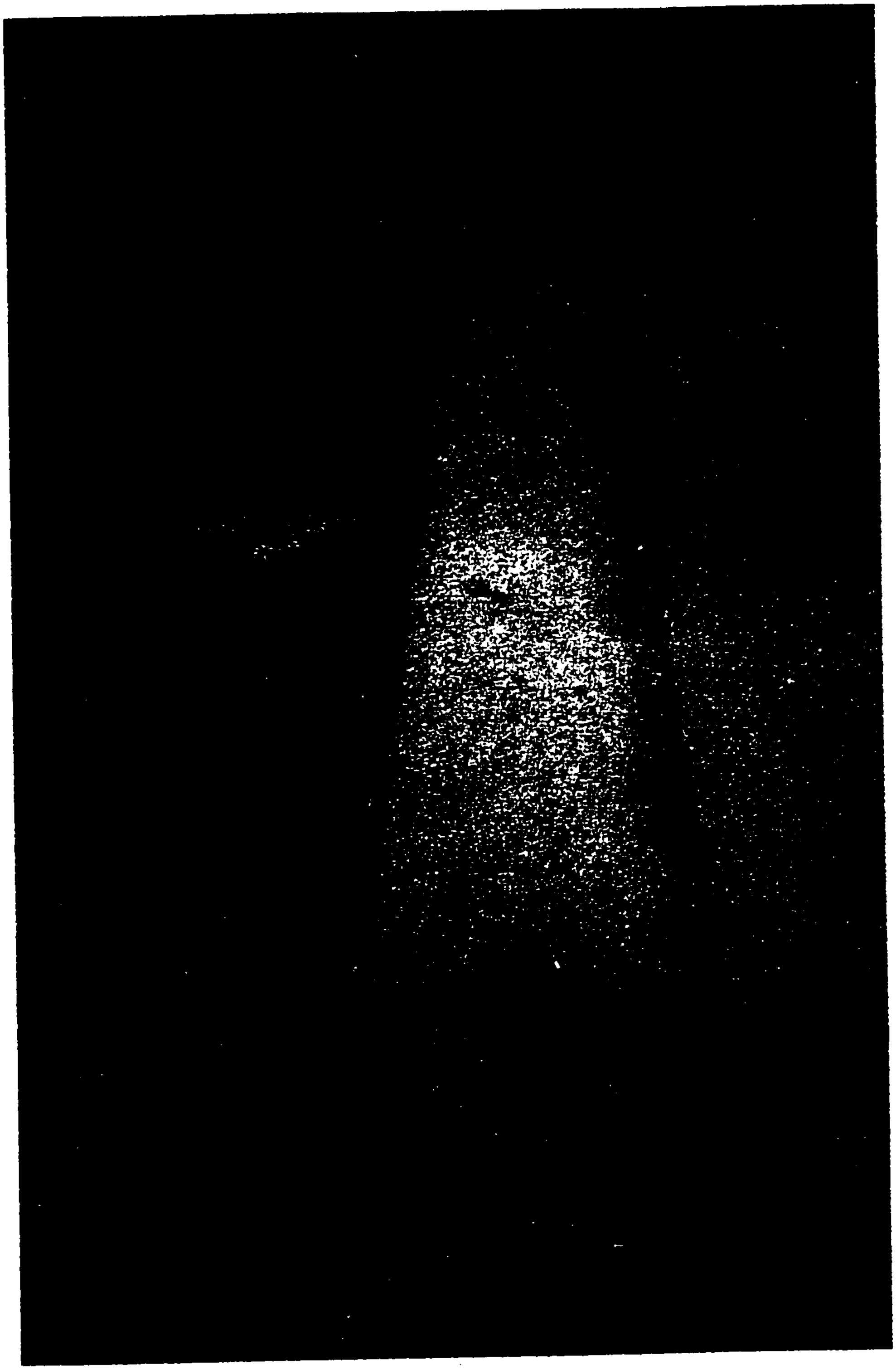


Fig. 23. Damaged pavement, case study 7 area.

Fig. 24. Claystone beds are expressed as bumps in street and sidewalk pavement.

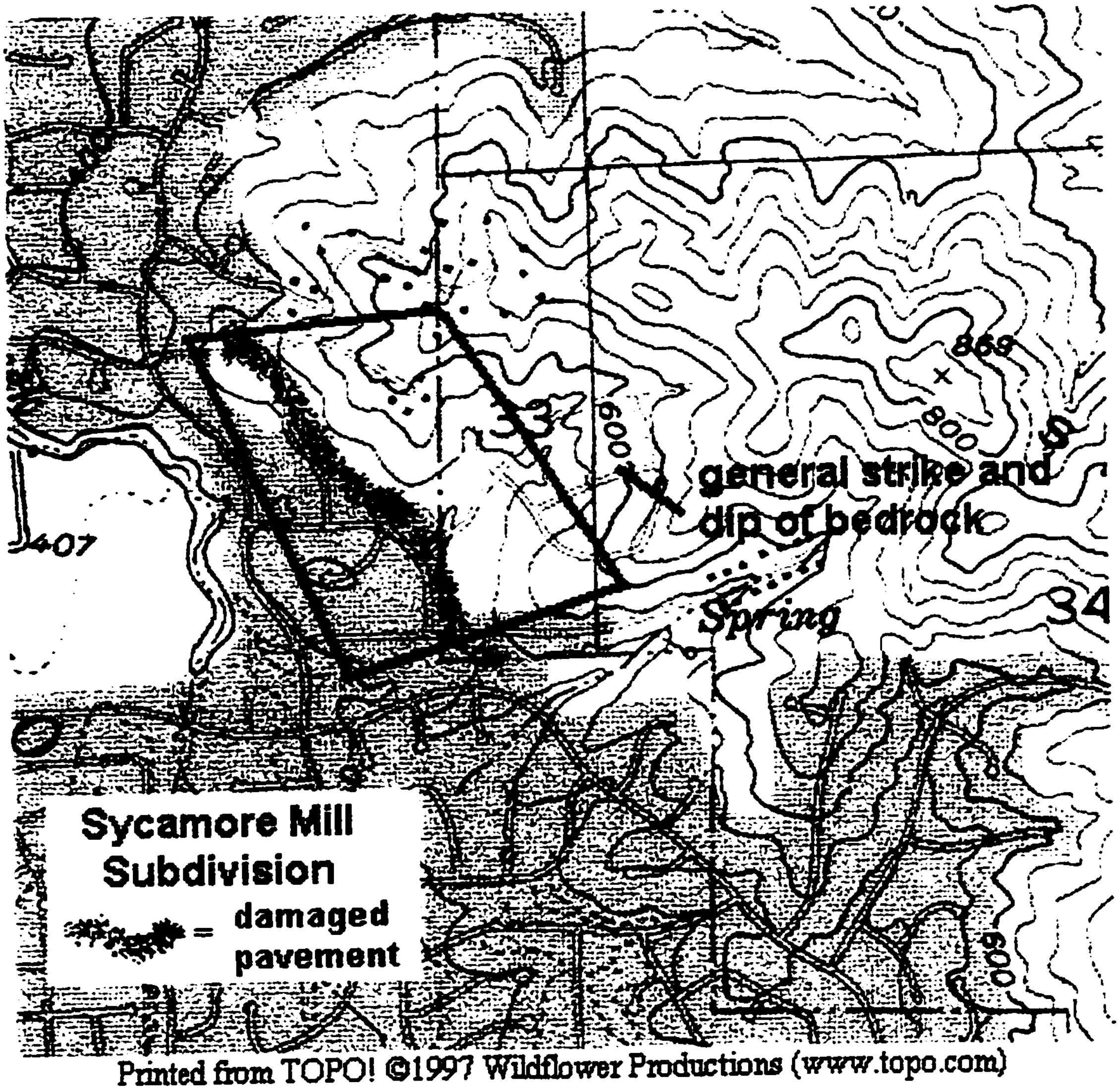


Fig. 25. Case study 7 site plan showing area of general pavement distress. Note that the general trend of pavement distress is almost parallel to the strike of bedding, indicating that one or possibly several closely spaced claystone beds are causing the damage.

signs of distress in the houses adjacent to these humps. However, in an adjacent neighborhood, expansive beds are found expressed as humps in the street (Fig. 24). (This may be due to maintenance or repairs that have been effected.) Fig. 25 shows the general areas of greatest pavement distress, notably on the lots that were flagged by a geotechnical engineer in the mid-1970s.

Case 8: Mariner Point, Vallejo, CA

Northwest across the Carquinez Straits from Pittsburg are the bluffs of Vallejo, where development has boomed along the Highway I-80 corridor towards Sacramento. In the early 1980s, two subdivisions were constructed using cut and fill lots along a ridge top on the bluffs. In 1989, the author was called to investigate damaged homes and streets in the subdivision, including settlement due to deep fills, expansive soils and expansive rock. One house in particular had suffered differential heave, up to six inches across the structure, such that the house looked noticeably crooked from the street. The road in front the house had at least two large 3 to 4 inch high and 7 to 10 feet wide humps resembling speed bumps, one hump lined up with the distressed portion of the house. These humps are shown in Fig. 26. The second hump could be traced under the garage of a nearby house, which also showed signs of displacement and heave (Fig. 27).

Examination of the subdivision development records, and observations in the field, showed that this portion of the development had been notched into the hillside, with cuts up to approximately 15 to 20 feet. The bedrock in the area is late Cretaceous Panoche shale (also known as Knoxville shale), which contains interbedded siltstone, sandstone and claystone as well as discrete beds of tuff, a result of intermittent volcanic activity

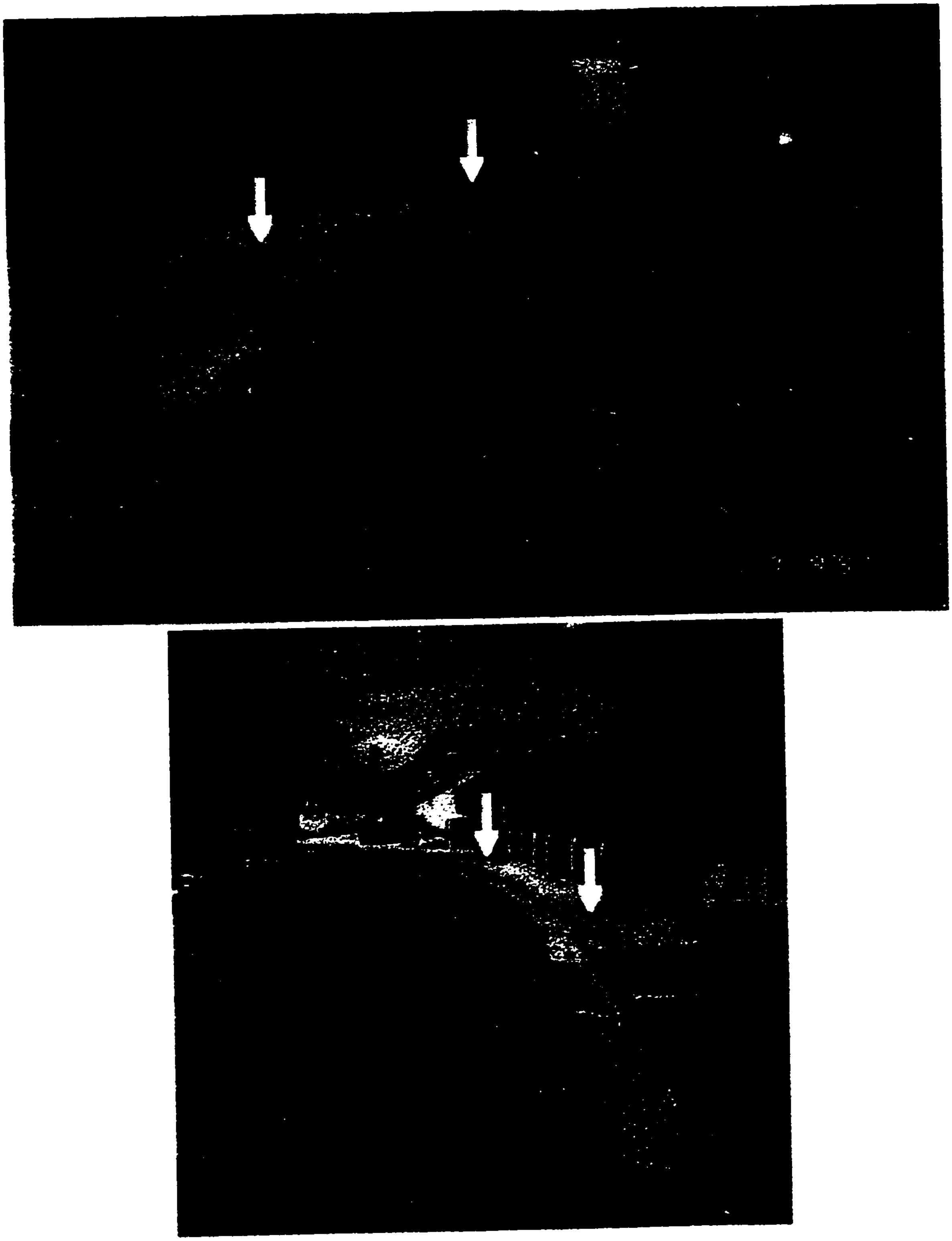


Fig. 26. Case study 8, claystone beds expressed as bumps in streets and sidewalk.

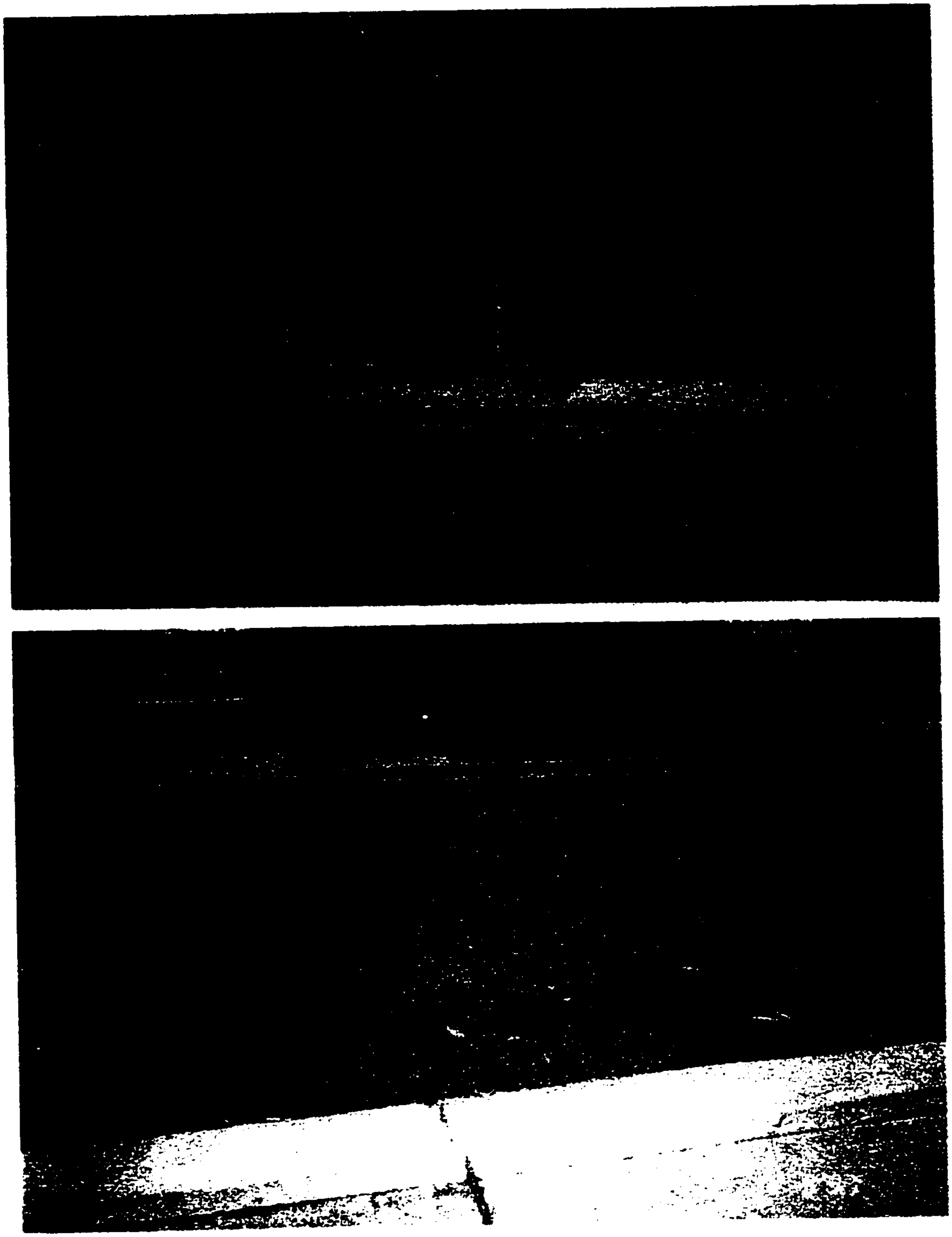


Fig. 27. Case study 8, damage to houses and pavement. Broad hump in upper photo extends underneath garage and slab.

during the late Cretaceous period. The humps in the road observed onsite correspond with two silty, clayey beds of tuff. Heaving of the these tuff beds began shortly after development and is continuing today, more than a decade later, as evidenced by repeated patching and continued presence of humps in the street. The badly distorted house has since been re-leveled but no repair of the street has been performed. The swelling is likely due to a combination of load relief, alteration of tuff to clay and swelling of the clay minerals.

3.1.3 San Francisco Bay Region-Other Projects

There are many residential projects constructed on expansive rocks in the Bay area, particularly in the East Bay along the northern portion of the Diablo Range, for example in the Hanna Ranch development in Hercules and new subdivisions in Antioch (Fig. 3, locations A and B). The majority of these subdivisions are constructed using large cuts and fills to allow for standardization of lot and house construction, a style of land development conducive to creating foundation problems on swelling rocks. The geotechnical concerns are focused on cut/fill transition lots and expansive soils. The typical geotechnical report contains a section titled "Expansive Soils/Rocks" acknowledging that there is rock that may swell, but beyond cursory acknowledgement, there is no further mention or discussion.

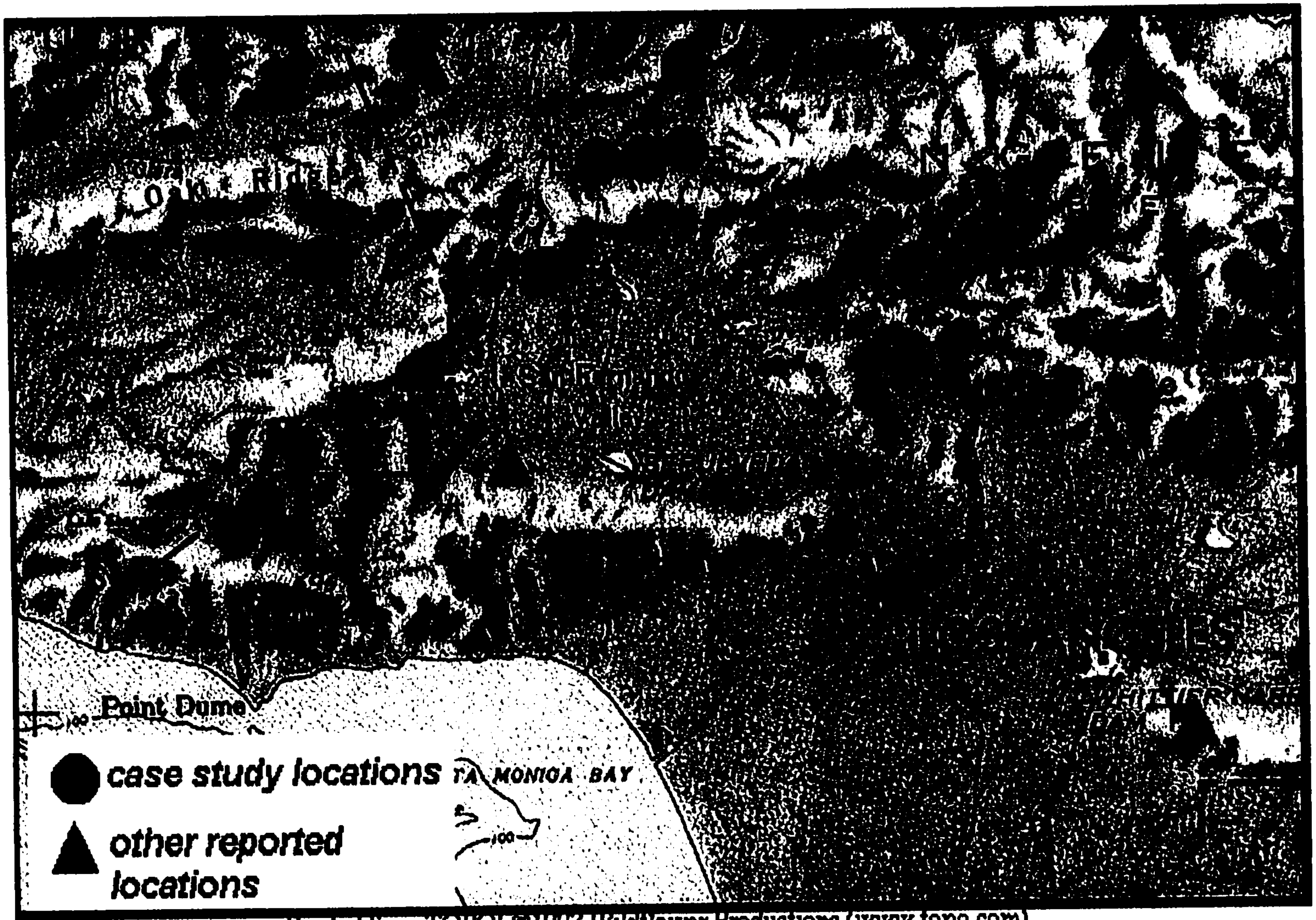
Observations of these subdivisions by the author reveal typical patterns of occurrence of damage. Newer subdivisions do not show many signs of damage due to presence of claystone. Older adjacent subdivisions have low relief broad humps in the streets and disturbed curb and pavement edges, indicative of long term creep. This may be due to seasonal wetting and drying, and likely aggravated by watering of the lawns that front the sidewalk and street.

Most residential projects examined display typical patterns of expansive rock damage, largely due to claystone. There are some projects that are slightly unusual either in location or damage which at first are not readily recognized as expansive rock areas. One such single family home on Skyland Ridge (Fig. 3, location C) in the Santa Cruz Mountains was reconstructed on a new cut building pad after the original house was destroyed in the Loma Prieta earthquake. An 8 to 10 foot cut into soil and rock was made to accommodate the new house. Bedrock under the building site was described as a clayey siltstone and shale. A newly poured basement slab began exhibiting polygonal cracking less than a year after construction. Heave was centered on the eastern portion of the basement, the side closest uphill, therefore with the largest cut. A four to six inch layer of expansive silty clay underlay the slab. The dispute over the cracked slab focused on the concrete mix and installation, and subgrade preparation. The geotechnical engineer noted the presence of expansive soil, and required that no expansive soil be used within the upper two feet of any fill.

Atterberg Limits and other laboratory tests indicate the four to six inch soil layer had a PI of 41% and swell potential of 5 to 15%, not enough to cause the type of cracking observed. Even if 15% swelling had occurred, this would indicate only one inch of movement at most. Cores through the basement slab revealed the clayey siltstone beneath had a PI of 43%, and was more expansive than the four to six inch soil layer "left" under the slab. Observations indicate that the expansive bedrock on site is contributing to damage. The four to six inch "soil layer" under the slab may be no more than a weathering of the bedrock after it was cut. Fresh claystones and siltstones typically display a rapid weathering when exposed to the elements, and the Santa Cruz Mountains are relative wet compared to the rest of the San Francisco Bay area. None of the parties involved recognized the presence of expansive rock until a geotechnical engineer experienced in swelling rock investigated the problem.

3.1.4 Greater Los Angeles Region-Southern Ventura County

In southern California, the coast range topography is similar to that of the San Francisco Bay area, as well as the climate. In San Jose the annual average rainfall is about 7 to 10 inches per year, in Los Angeles the average annual rainfall is about 15 inches per year. However, the core of the coast ranges in southern California is considerably younger. The rocks that make up the distinct coast ranges in and around the Los Angeles Basin are younger, mostly Miocene age and younger, than most of the rocks in the northern California coast ranges. These rocks were formed during a time when southern California had a series of basins filling rapidly with sediment and active volcanism at the same time. It is common to find beds of pure bentonite clay within the shale beds, whereas this is not true in northern California. When looked at regionally, the locations of the few case studies presented here, although ostensibly in different mountain ranges, actually are part of the same features and line up along northwest trends. Fig. 28 shows the locations of the case studies within this region.



Printed from TOPO! ©1997 Wildflower Productions (www.topo.com)

Fig. 28. Greater Los Angeles case study locations.

Case 9: North Ranch, Thousand Oaks, CA

The community of Thousand Oaks is about an hour north of Los Angeles on Highway 101, geographically a basin and long narrow stream valley in the Santa Monica Mountains within the Transverse Ranges. This area has experienced tremendous growth in the 1980s as the population of Los Angeles burst out of the basin confines northward. Housing was built in the out-lying areas for those seeking to escape the pressure and smog of Los Angeles yet live close enough to commute into the San Fernando Valley. Housing has filled the Triunfo valley floor and the newest developments are on the ridges and in the steep canyons on either side of Highway 101. Large cuts and fills are required to develop in steep canyon terrain.

Geologically the area is a mix of Miocene marine and non-marine sandstone, siltstone, shale and conglomerate, with basaltic intrusions and flows (Jennings 1991). This is typical of the changing sea level and basin deposition in the Miocene in the Los Angeles basin, punctuated by volcanic activity. Several of the rock formations in the Thousand Oaks area contain claystones and shales that cause unstable slopes and expansive soils, such as the Modelo formation and the Topanga formation, although the Modelo formation is chiefly fine-grained shales and siltstones while the Topanga formation consists mainly of sandstone. The City Hall complex of Thousand Oaks (location 9b on Fig. 28) was built on the south side of Highway 101, with a basement, on cut/fill pads in the Modelo formation, (Fig. 29). It has since been abandoned because of building foundation damage due to expansive rock, with swelling possibly aggravated by the basement excavation. Maximum cut slopes are about 15 feet high and although no

Fig. 29. Case study 9b, the abandoned Thousand Oaks City Hall. Note the cut slope on right two stories high; the building also had a lower level below ground.

data on swelling is available, the paving in front of city hall is clearly distressed and cracks indicative of swelling can be seen in the exterior walls.

One example of expansive rock damage is typical of the development in Los Angeles and southern California and some of the problems that occur during the process of housing development. North Ranch (location 9a, Fig. 28), an exclusive development in the hills on the north side of Thousand Oaks, created cut/fill pads in Miocene Modelo formation. During the project, the developer began to run short of funds and chose to finish the project partly as finished houses and partly as ready-to-build lots. One of the lots, lot 114, was purchased by a real estate investor in 1989 who constructed a $1.7 million house which was finished in the summer of 1992. Shortly after construction, in the fall of 1992 the pool shell cracked, as evidenced by loss of water, and the house developed cracking in exterior window frames and uneven floors typical of expansive bedrock. A chronology of these events is shown on Fig. 30.

The Modelo formation in the area of the North Ranch development is made up of sections of steeply dipping fissile alternating thin beds of diatomaceous siltstone and silty claystone and sections of thin to thick-bedded sandstone. The siltstone and claystone section contains scattered, thin (several inches thick) bentonite beds (from volcanic ash). Long narrow canyons dissect these beds, exposing a many-layered alternating sandwich of rock in the canyon walls if the soil were stripped away. Removing a portion of ridge top created lot 114, which revealed the presence of bentonite beds (Fig. 31).

The City of Thousand Oaks required a soils and geologic report on the development prepared by a civil engineer. The Environmental Impact Report for the

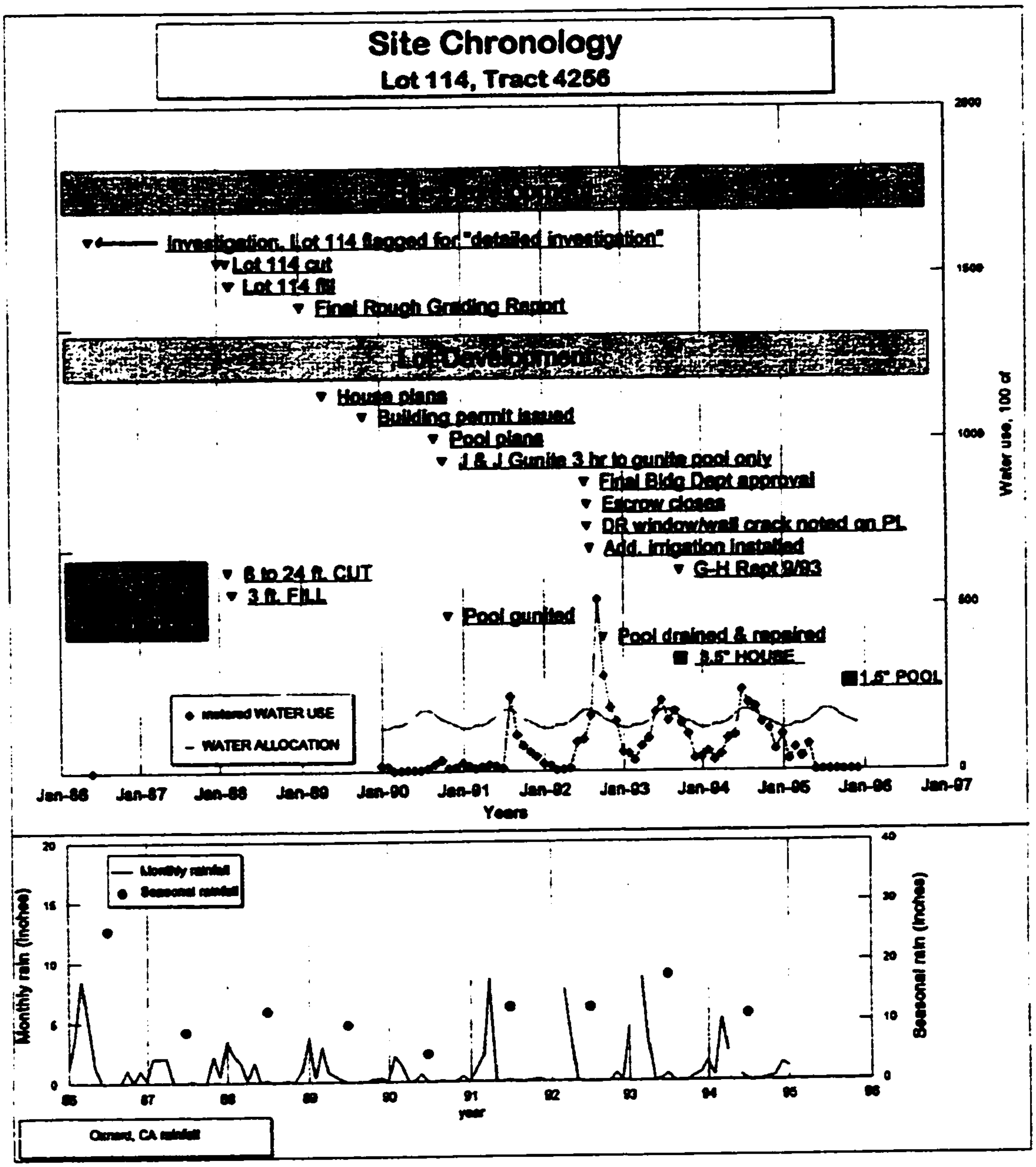


Fig. 30. Site chronology of case study 9a.

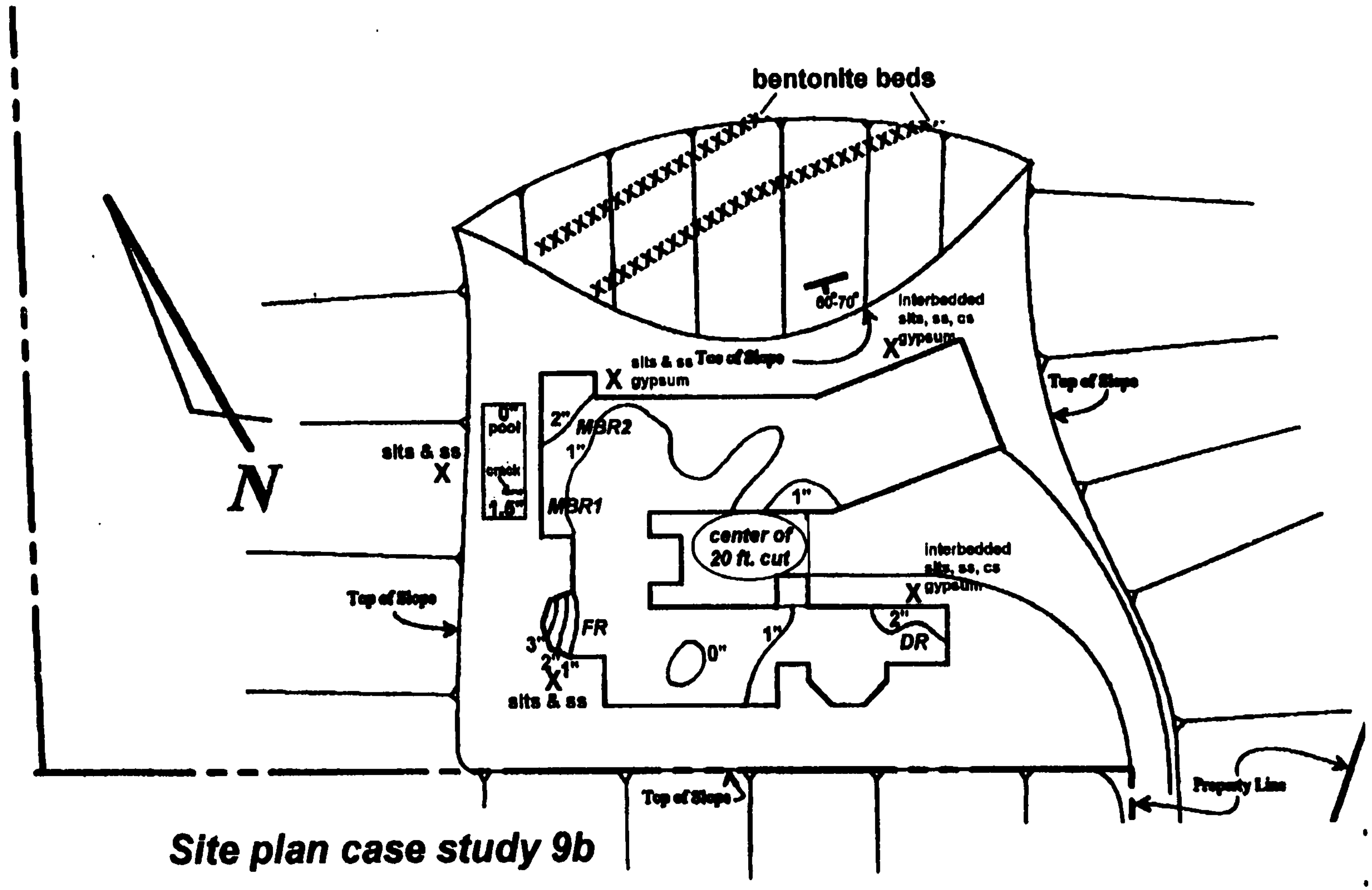


Fig. 31. Site plan case study 9a.

development was mainly concerned with view-shed, oak tree preservation and drainage issues rather than geologic hazards. The initial soils report for the project in 1986 had flagged problem lots within the project for further individual lot investigation prior to construction, and lot 114 was one of a dozen or so that were flagged due to the presence of bentonite beds within siltstones and shales. Rough grading occurred in March 1988, with 8 to 24 feet of cut removed exposing dense bedrock. The lot was over-excavated by three feet, and compacted fill placed to lot grade within a month or so. The purpose in over excavating, according to the project soils engineer, was to make it easier to excavate the foundation footing and install landscaping. A provision requiring five feet of material removal and recompaction for lots that encountered expansive bentonite beds was included in the soil report. Lot 114 was mapped geologically with several other lots on the same ridge and a geologic map prepared. Bentonite beds were noted on the cut slope behind the lot, but none noted on the pad itself.

Grading was approved in 1989 and a building permit issued that same year. Expansion tests conducted on the soils of the development (and included in the final rough grading report) indicated that they were moderately to highly expansive (up to 18%), with the soils on lot 114 being classified as having the potential to swell up to 9.7%. Foundation recommendations were based on this classification of soil. The geotechnical engineer in the final rough grading report made it explicitly clear that final lot grading approval, expansion testing and lot-by-lot foundation recommendations would be required for natural and transition lots (cut/fill). The engineer also made it clear that it was not possible to fully mitigate the effects of expansive soils. Explicit recommendations were made by the engineer in the final rough grading report regarding necessity of positive lot drainage, minimum landscape watering, warnings to owners regarding changing the drainage and educating property owners about expansive soils conditions. Despite all the discussion of expansive soils, no explicit mention was ever made of expansive bedrock in the soil investigation.

From 1990 to 1992, a large U-shaped several thousand square foot single family home was constructed on the lot using conventional spread footings 24 inches below grade (Fig. 31). A pool was also constructed on the lot adjacent to the house, six feet deep, which places the bottom of the pool three feet into bedrock below the fill pad placed on the lot. The house was sold in July 1992 and the real estate disclosure statement noted cracking of the exterior wall near the dining room, possibly related to leaking. This indicates that probably even as the house was being constructed, heaving of the bedrock was occurring. In September 1993, the house distress was first investigated on behalf of the homeowner. After approximately four years, the largest floor level difference was 3.5 inches in the dining room. The pool shell cracked 1.5 years after it was built and was out of level 0.75 inches after 2.5 years and 1.5 inches after five years. Fig. 32 illustrates the changes in floor slope over time in various parts of the house.

Test pits on the lot reveal diatomaceous siltstone and claystone, shown on Fig. 31. Testing of the claystone indicated a moderate expansion potential, and test pits showed fractured bedrock with small gypsum crystal deposits within the fractures. The rock on the central portion of the lot was less weathered than near the edges. Some of the claystone was characterized as highly bentonitic. The geotechnical engineer concluded

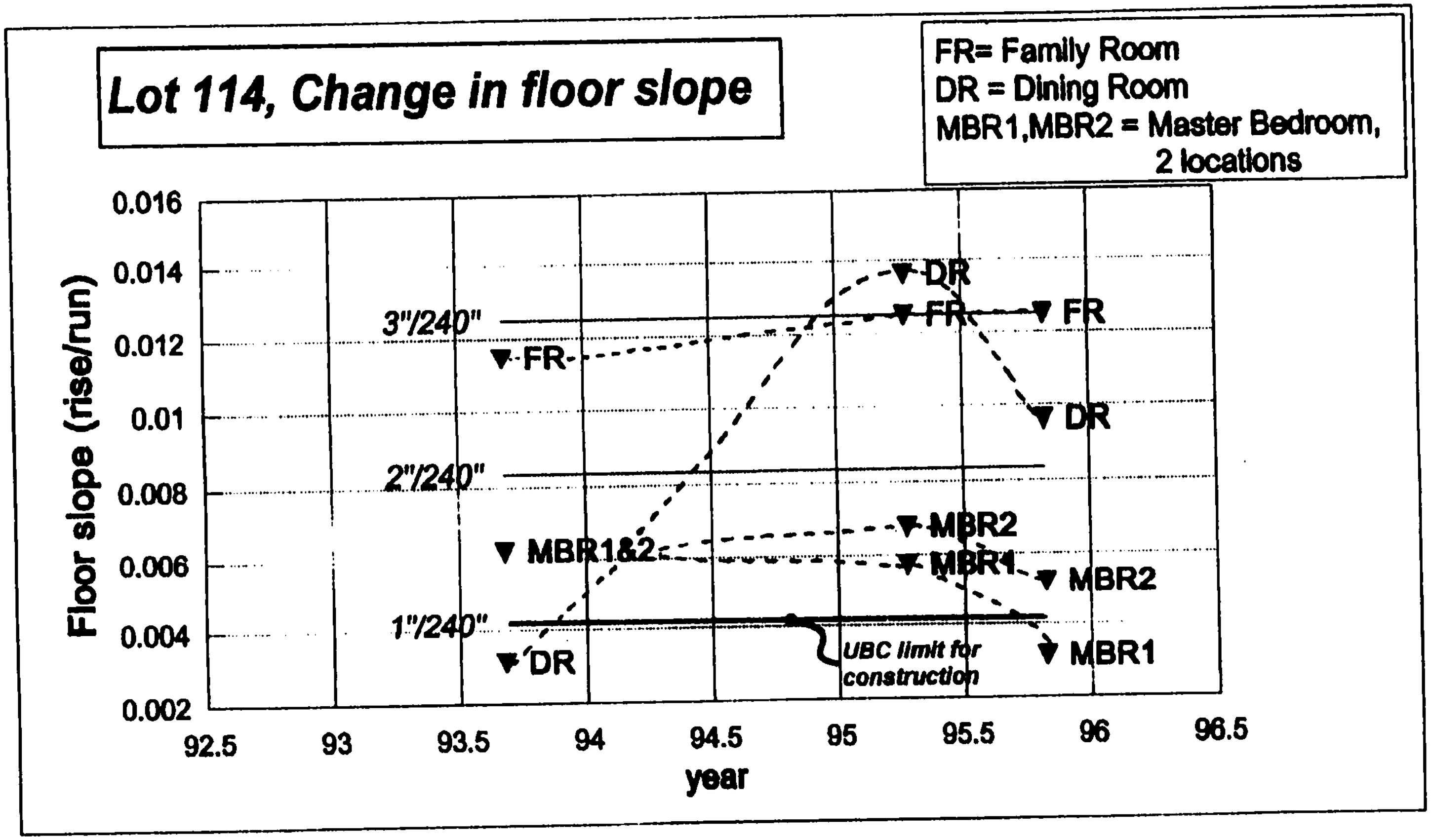


Fig. 32. Floor level measurements, case study 9a.

that damage to the house was due to expansive bedrock, specifically, but could not completely explain the heave in certain portions of the house, nor the difference in timing of the floor movements. The author believes that the family room heaved first due to water from the leaking pool saturating the rock on the west side of the house. The dining room area swelled later because it took longer for the moisture condition to change beneath the three foot fill blanket; there was no direct source of water to the rock. The south side of the house did not experience much movement, in part due to its southern exposure, which tends to keep the soil drier, but also because drainage was better in this area due to the adjacent downhill slope.

In this case, the geotechnical engineer for the original developer flagged problems on individual lots, and apparently had a program for further lot investigation. However, when the lots were sold, even though they had been mapped, they still had restrictions on the lots or requirements for mitigating possible problems beyond those in the general report for the entire development. A minimum fill pad was placed on the lot; a conventional foundation was used and constructed in such a manner as to aggravate the damage. A repair estimate for the house prepared in conjunction with litigation totaled $1.05 million, with foundation replacement alone making up almost 25% of the cost. Even disallowing 50% for litigation inflation, the total repair to the house is well over several hundred thousand dollars in order to restore the house to showcase status.

3.1.5 Greater Los Angeles Region-Northern Los Angeles County

Northern Los Angeles County has the greatest number of documented locations of expansive rock damage to residential housing. In part this is due to the mountainous nature of this region, where large cut and fill lots are common, a development style that is conducive to expansive rock damage. Many of these case studies have been published in geological publications not regularly read or reviewed by geotechnical engineers, although engineers author some of the case studies.

Case Study 10: Santa Clarita Valley, CA

The Santa Clarita Valley is situated just north of the San Fernando Valley along Highway 114 (Antelope Valley Freeway) in southern California. Within the valley and the adjacent hillsides, many residential developments have been damaged by ground instability (Hollingsworth 1990; personal communication, James Slosson 1997). This ground instability is readily observable on the approach to Newhall from the Ventura area, where the landscape has a messy disturbed appearance. At one point before entering the Santa Clarita Valley itself, it is clear that the entire hillside on the south side of the freeway has failed.

The valley itself was once a sedimentary basin amidst volcanic and granitic mountains (Saul 1990) during the Tertiary period. During this time a stream valley and delta discharged into a lake, until the advancing seas during Miocene and Pliocene times created a nearshore depositional environment. Subsequently the shoreline retreated from the basin, once again leaving a stream valley. The rock column represents a mix of sandstone, conglomerate, siltstone and claystone with occasional tuff beds, all derived from the surrounding terrain. The stratigraphy represents streambed, deltaic shoreline and quiet lacustrine and marine sedimentary facies typical of depositional basins of the time.

The formations that contain troublesome claystones, as noted by Saul (1990) and Hollingsworth (1990) are the Mint Canyon and Saugus formations. The Mint Canyon formation represents a wide variety of sedimentary environments deposited in the middle Miocene when a large lake occupied the center of the depositional basin, known as the Soledad Basin (Saul 1990). Alluvial and deltaic sediments consist of foreset beds (the sediments deposited at the edge of the delta) and topset beds where sediments were deposited at the bottom of the lake at the toe of the delta. The deltaic sediments contain claystones and occasional tuff beds interbedded with thin siltstone and sandstone beds, all part of the bottom lakebed facies (or foreset-bottomset facies). The occasional tuff beds are from ash falls during the volcanically active Miocene.

The Saugus formation is younger than the Mint Canyon formation and straddles the Tertiary-Quaternary boundary. The sea level rise during the late Miocene retreated and a resumption of streambed and lake bed sedimentation occurred. The Saugus formation contains clay rich sediments. Other geologic formations contain claystones and clayey siltstone beds as lenses or discontinuous deposits, but the Mint Canyon and Saugus formations have been better documented.

These claystones have damaged homes in several neighborhoods, starting in the 1960s with the Princess Park neighborhood (location 10a, Fig. 28) and continuing up through the late 1970s in the city of Santa Clarita (location 10b, Fig. 28) and the Forrest Park neighborhood (location 10c, Fig. 28). A reconnaissance of the neighborhoods, both older construction and newer construction, revealed that the majority of the area is hillside development, with cuts on ridge tops from 8 to 10 feet up to 80 feet. In older

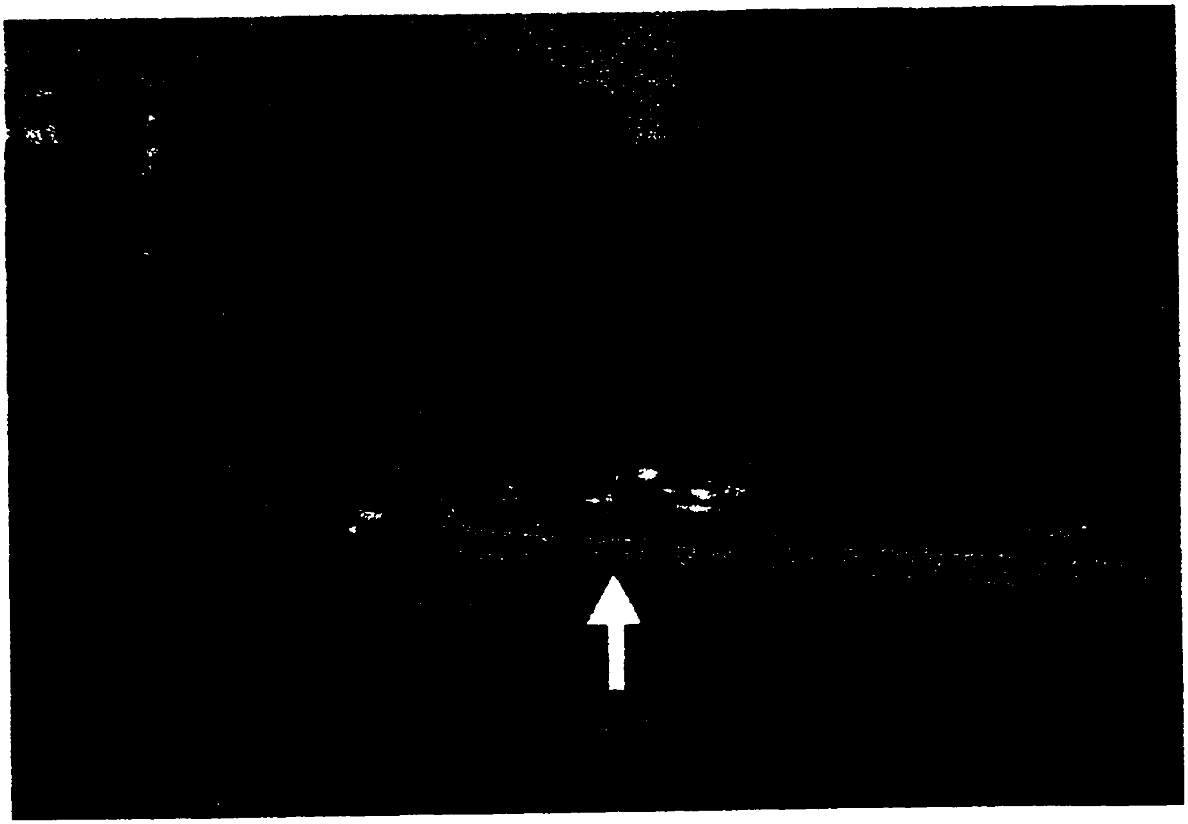


Fig. 33. Street humps in the Princess Park neighborhood, case study 10a.

developments, damage to houses could be seen from the outside, as well as in humps in the roads and disturbed concrete flatwork (Fig. 33).

One case study shows an early awareness of expansive rock, but a misunderstanding of the mechanism of damage (location 10b). In the late 1970s, early 1980s the geotechnical consultant for the project recommended a four foot thick non-expansive fill cap for building sites crossed by claystones beds within the Saugus formation. Geologic maps for the subdivision were prepared which showed the location of claystone beds. However, rather than use stiff foundations, a conventional slab-on-grade foundation was used, since it was apparently assumed that either no water would penetrate the four foot cap, or that heaving problems were taken care of by the non-expansive fill. The house had differential movement of three inches across the contact between an expansive sandy claystone bed and non-expansive sandstone and conglomerate. Perched water within the four feet of fill had saturated the claystone. Swell tests using a surcharge of 570 psf, combined load from the fill cap and foundation system, showed that up to six percent swell would occur in the claystone under this load.

Location 10c is in Forrest Park just north of Santa Clarita and is similar to the case study 9b. A newly constructed pool excavated into clayey siltstone bedrock of the Mint Canyon formation experienced heave within four to five months after completion. The shallow end of the pool cracked, however the deep end of the pool suffered little damage, similar to the pool in case study 9b. Expansion Index tests and swell tests under varying loads showed that the shallow end of the pool was too lightly loaded (300 psf) to resist swelling of over 8% while the load equivalent to the deep end of the pool (700 psf) swelled less than half that much (3.3%). Based on the dry densities of approximately 100 pcf, the 8 to 10 foot cut initially made on the lot removed about 800 to 1,000 psf overburden load. Typical residential pools are about three feet deep in the shallow end and less than eight feet deep in the deep end. Based on average dry density of 100 pcf, the presence of the pool would not have replaced the load removed by the pool excavation. It is unlikely the cut lot had a chance to equilibrate with the new ambient moisture environment before construction, a process which based on published studies takes more than 10 years. Even if this had been the case, the presence of the pool excavation likely introduced water into the subsurface, and swelling likely could not have been avoided.

Current development continues in the same post W.W.II style of large cuts and fills, as seen in the Sunset Hills development (Fig. 34), close to the Princess Park neighborhood. Fig. 35 shows the geology of the area and the close proximity of new developments to older developments damaged by expansive rocks. Smaller developments are being developed at the fringe of large rural lots, on the hillsides too steep for agriculture or riding rings and stables, where the majority of houses sit on at least one or more acres and most have horses. Typical suburban subdivisions are being added in the foothills at the end of rural roads in the hillside areas. This is typical of the Santa Clarita Valley and Simi Valley, as well as the Napa and Sonoma Valleys in northern California, once rural and now increasingly suburban areas outside heavily populated regions like Los Angeles and San Francisco.

Beachgrove Court (10d) is typical of the housing development, on cut/fill lots that straddle ridges and valleys in the steep terrain in and around the Santa Clarita Valley. It is adjacent to the Princess Park neighborhood, on the south side of the valley along Highway 14. In 1994, after the Northridge earthquake, many claims were made against homeowners' insurance policies for damage due to groundshaking, such as cracked slabs, cracked sheet rock etc. However, some of these claims were not legitimate, damage existed before the earthquake, and the pattern of damage did not resemble the typical earthquake damage. A single story, typical 1970s style tract house similar in floor plan to those built in Pittsburg and Antioch, was investigated for earthquake damage. The house was built on a cut pad, with the back of the house built on the largest cut (Fig. 36). The foundation is concrete slab-on-grade.

When inspected in 1994, a floor level survey showed the back southeast portion of the house up five to six inches relative to the front of the house. The interior north south walls tilted 0.5 to 0.75 inches to the west, and the east west walls tilted one inch to the north. The inspection report notes that many of the cracks existed prior to the earthquake, and that the concrete slab-on-grade was cracked throughout the house. It is also noted that the garage slab and much of the exterior concrete flatwork is cracked.

The engineer concluded that the "fill" on site had massively consolidated as a result of the earthquake. However, this does not fit with the observations. No street failure was observed, despite the street being on the deepest part of this "massive fill consolidation" that supposedly cracked the slabs and flatwork. Also the engineer noted that most of the cracking existed before the earthquake. An alternate, and more likely, explanation is that

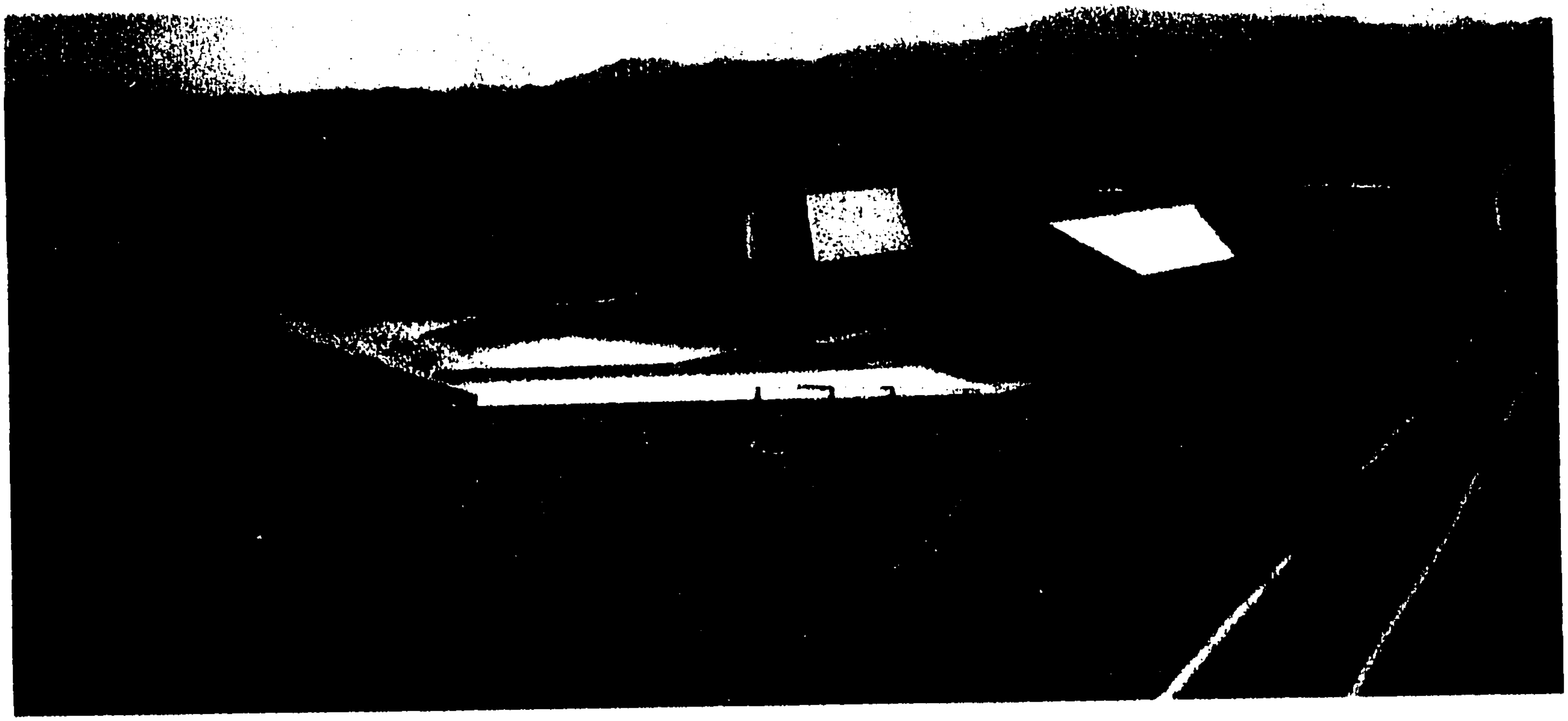


Fig. 34. Typical mass graded lots for new residential development, Santa Clarita, CA.

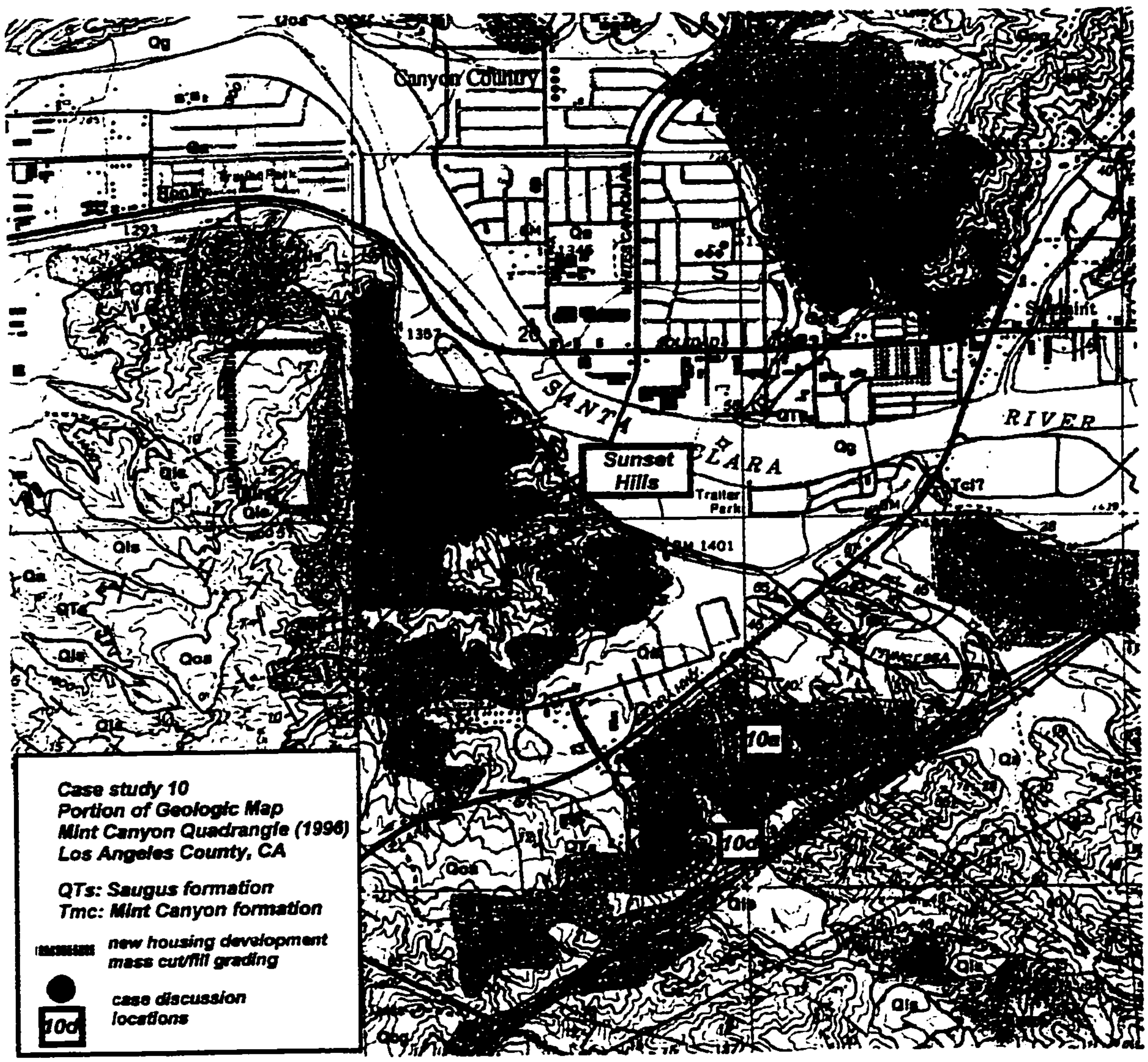


Fig. 35. Geologic map of a portion of the Santa Clarita Valley (modified from Dibblee 1996).

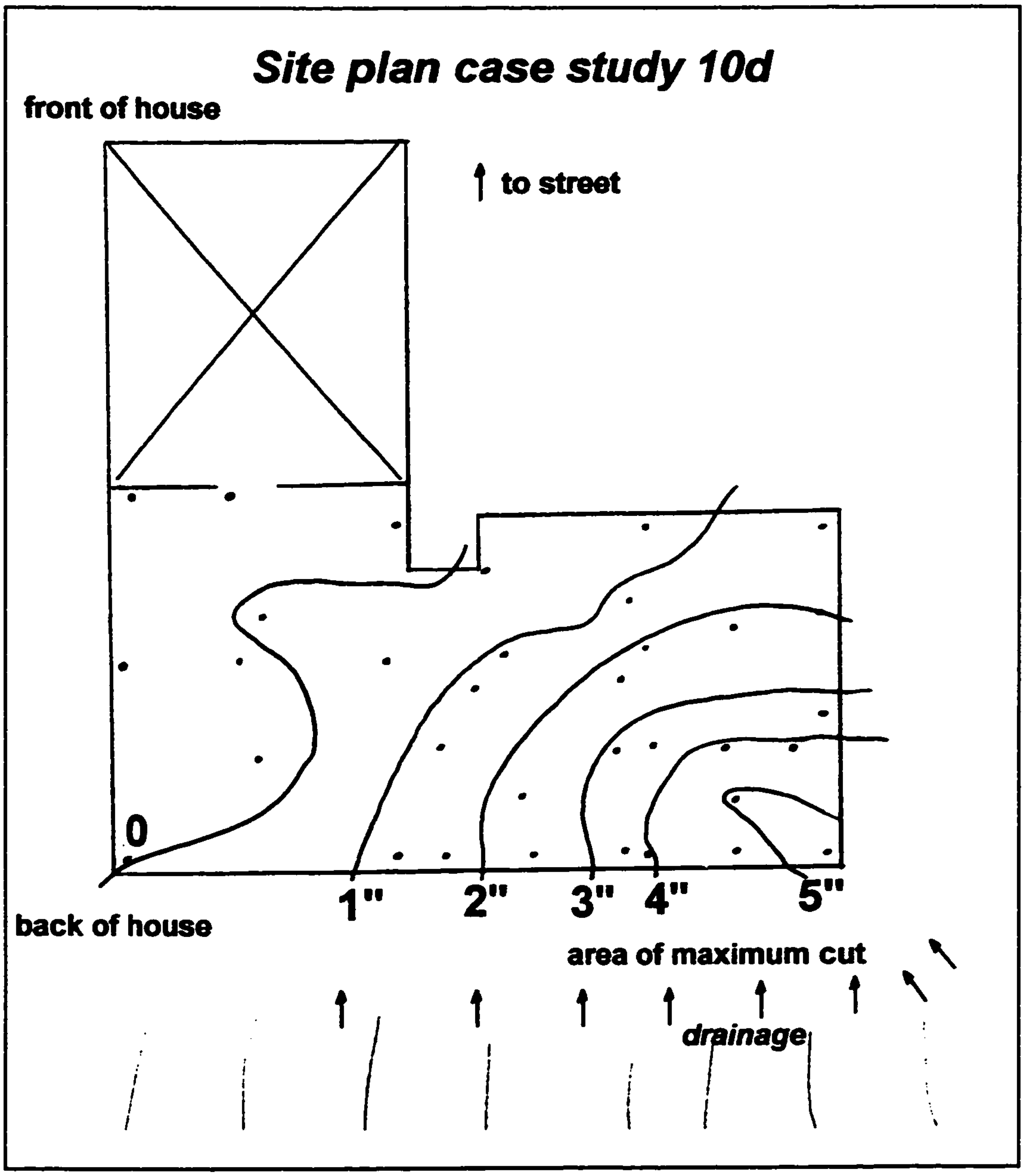


Fig. 36. Site plan, case study 10d, showing location of heave and general grading and drainage features.

the building site, which is located in the hills above Newhall and Simi Valley, is underlain by expansive rock of the Mint Canyon formation. The back southeast portion of the house is on cut, and collects surface run off from the slope behind. Because it sits on the area of greatest load removal and collects moisture, which infiltrates into the ground surface, the house has heaved 5.5 inches (see Fig. 36).

This large heave accounts for the northward and westward tilt of the walls. Observations of other houses with five to six inches of heave confirm that displacement of that magnitude gave a crooked or "funhouse" appearance to the building. The style of home indicates that it has been present since at least the late 1970s, early 1980s. However, the inspecting engineer never considered alternate possibilities, examined the surrounding neighborhood, or tested any of the rock or soil to observe, confirm or disconfirm his theory of damage. This is a prime example of the lack of recognition of expansive rocks.

Case study 11: Santa Susana mountains, Simi Valley and Porter Ranch, CA

Two other case studies were published by Hollingsworth (1990) related to the same type of rocks found in the Santa Clarita Valley. These two case studies are located on the southern flanks of the Santa Susana mountains, in Simi Valley (location 11a, Fig. 28) and the Porter Ranch neighborhood of the San Fernando Valley (location 11b, Fig. 28). Bedrock beneath both sites is Saugus formation. Both subdivisions were graded in the mid 1970s, with four feet of cut on the building at location 11a and up to 80 feet of cut on the building pad on location 11b. Foundation recommendations were for a 30-inch non-expansive fill cap and allowed for use of conventional slab-on-grade foundation.

Location 11a is a one story stucco house with slab-on-grade foundation and was constructed in the mid 1970s with a pool on the lot. The subdivision was graded in 1973 and 1974. At the time, up to four feet of material was removed from the building pad. The foundation recommendations called for a 30-inch non-expansive cap of fill and use of conventional foundation. Up to three inches of heave occurred, centered on the five to six foot wide claystone bed. The UBC Expansion Index (EI) for the claystone was 142, and other testing showed that six to seven percent swell would occur under a surcharge of 300 psf, equivalent to the foundation and fill load.

At location 11b, where there was greater overburden relief, differential heaving across the structure was on the order of four inches. During investigation of the house distress, trenches showed a sheared and faulted claystone bed crossed underneath the house, directly underneath the bearing footing that is heaved upward the greatest amount. The UBC Expansion index of the claystone bed was between 130 to 165, and swell tests indicated that between 7 to 9.5% heave would occur under the foundation and fill loads. This case history illustrates a phenomenon similar to case study 5, where swelling has caused slippage along bedding planes, and gives the appearance of a fault. This is important to note because presence of recent slippage in the subsurface on or near faults is a geologic hazard that has specific regulatory ramifications for housing development in California. Recently the author received a call from an engineering geologist working on a commercial warehouse development in the Santa Clarita Valley. He observed displacement in a claystone bed on a cut pad, and had noted this displacement in his report, calling it a fault. However, the geotechnical engineer for the project asked that the term fault be removed, lest it raise concern among the building and planning department and possibly prevent development on a portion of the project. Others have used the term fault to describe slippage along bedding planes or preexisting zones of weakness (Noe 1997). There is no standard for describing this phenomenon, and is an issue that has yet to be resolved in the engineering, geology and residential development regulatory system.

3.1.6 Greater Los Angeles Region-Los Angeles Basin

Case Study 12: Chavez Ravine, Los Angeles Dodger Stadium, Los Angeles

The Los Angeles Dodger's stadium is constructed on a low rise of hills, the remnants of the Santa Ana Range just northwest of downtown Los Angeles in the Elysian Fields section of the city. The reason these low hills have persisted long after the Los Angeles River and other streams have eroded the rest of the range is due to the resistant sandstone that makes up the majority of the Elysian Hills. When Dodger stadium was completed in 1962, the unique soils and foundation engineering aspects of the project were written up in 1965 (Smoots and Melickian 1965). The stadium was constructed in the center of a ring of terraced parking lots on cut and fill. In a discussion of post-construction project performance, heaving was noted in the eastern roadways and parking lots where "expansive shale" daylights. (When looking at maps of the project it appears it is actually the southern parking lots and roadways.) The heave is such that six to eight inches of material has been shaved off the parking lot from time to time, indicating that the heave is extensive (more than eight inches) and ongoing. The cause of the heave is attributed to the presence of an unknown water source "...required to expand the shale." Apparently the engineers viewed the expansive shale as similar to expansive soil, and that the shale will not swell if water is not allowed to come into contact with the shale. This does not account for other factors causing the swelling, deformational rebound (elastic and non-elastic) and weathering and adjustment of the claystone to new ambient moisture content.

The location noted by Smoots and Melickian was a parking lot east of the stadium, however, it is more likely the parking lot was located south of the stadium and that the authors were in error. A geologic map overlay on the current topographic map reveals the location of the shale mentioned by Smoots and Melickian, and therefore the heaving parking lot (see Fig. 37). Examination of topographic maps before the project and after the project show that the stadium was constructed by leveling a peak of elevation 726 to 600 feet for construction of the stadium itself, and cutting and filling surrounding local north south trending ridges. The parking lot on shale was cut 25 to 125 feet down to elevation 500 from greater than elevation 600, thereby exposing fresh and unweathered shale that had not been exposed to current climate conditions. An overburden load of approximately 2,400 to 15,000 psf was removed from the parking lot areas, with no significant structural load to replace it. As demonstrated by swell tests, even a load change of as little as 500 psf can make a significant difference in how much swell occurs. Currently, the parking lot is still experiencing maintenance problems.

When looking at locations of reported expansive rock problems, the location of Dodger stadium fits well within the area of expected problems. It is in the same arc of mountains that includes the Santa Monica Mountains to the northwest and the Santa Ana mountains to the southeast, both of which have Miocene expansive rocks. While no life

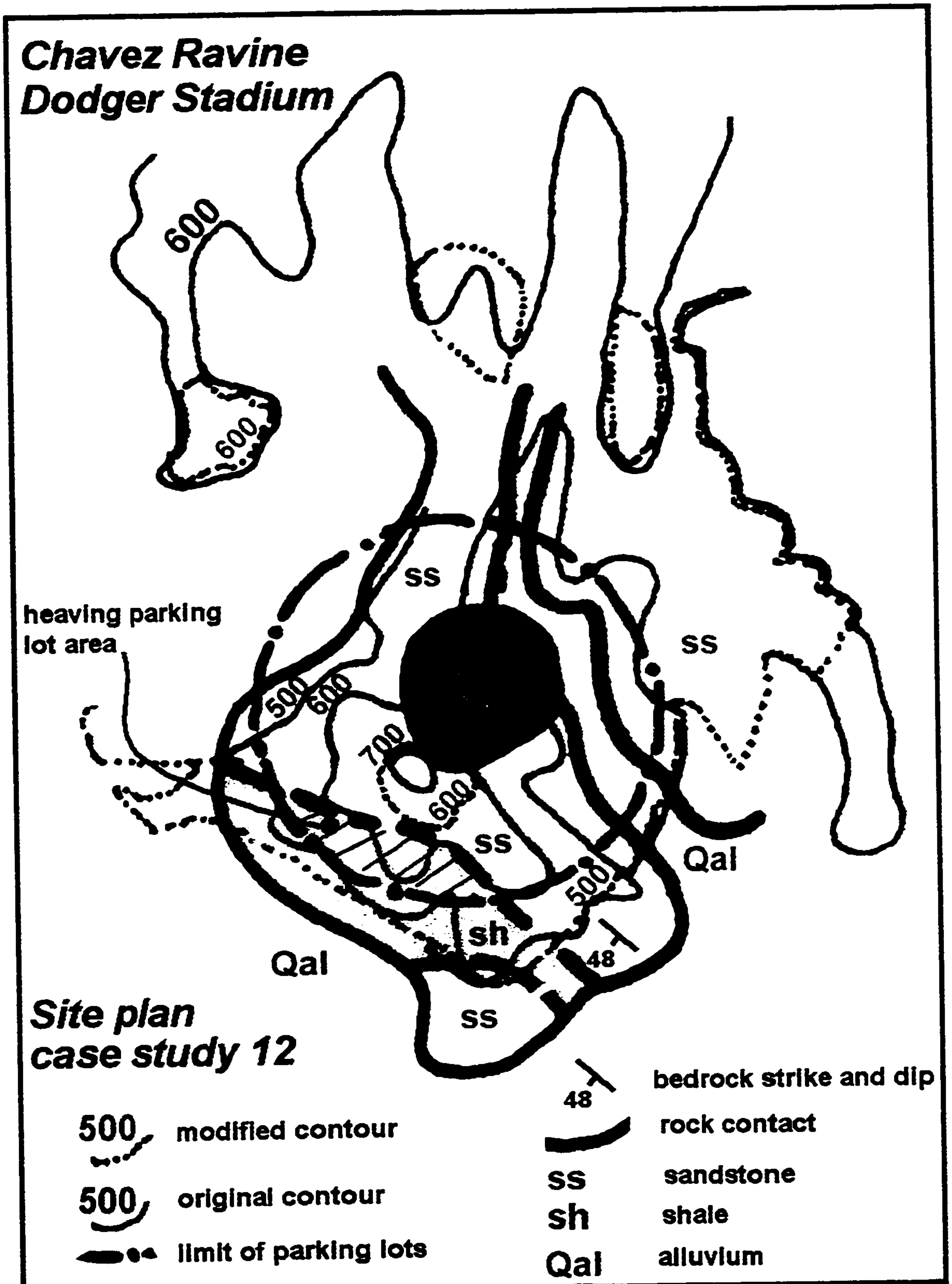


Fig. 37. Site plan case study 12. Hachured area shows where parking lot heave occurred.

threatening or serious structural damage resulted from heaving of the shale in this location, it illustrates the point that it is important to be aware of formations containing expansive rock members. Locations of other problems and an understanding of the swelling mechanisms can assist the engineer in recognition and mitigation of these problems.

3.2 Case Study Discussion

3.2.1 Patterns of Expansive Rock Damage

Common themes in the case studies are sequence of lot development, changes in total stress and moisture content of the expansive rock, and patterns of damage to pavements and structures that are typical of expansive rocks. Generally, the sequence of lot development and subsequent construction damage occurs over a several year period, and importantly over several wet and dry seasons, when freshly cut claystone is exposed to the atmosphere. Within the first or two year, houses constructed on cut lots containing swelling rocks begin to exhibit cracks in interior drywall and exterior stucco in the corner of doors and windows, and hairline cracks begin to form in the concrete grade beams, usually close to or immediately over the piers. If there is a readily available source of water nearby that was not present before construction, such as heavily irrigated landscaping, a leaking pool, or a climate change from drought conditions to normal rainfall conditions (not uncommon in California), then this may be accelerated or more dramatic. Floor level surveys often show the interior of the house has high and low spots; with the transition area between them showing the most cracking in doorways, interior openings and ceilings. In some cases, this occurs while the house is still being built (such as the Thousand Oaks case) and can occur even after substantial foundation repair or upgrade (case study 1). Over time, the cracks get larger and a sense of shear or uplift is observable. In the grade beam, typically these cracks will be open more at the top than at the bottom, indicating that there is uplift occurring.

Usually swelling of claystone manifests itself in several ways, and often along discrete beds of claystone or claystone containing zones. Swelling will occur over and along the claystone bed, as commonly observed in case studies 1-4 on the San Francisco peninsula, case studies 5, 8 and 10. Swelling may also occur along a zone within a claystone bed or along a contact with other rock types, where there are pre-existing weaknesses in the rock due to slickensides and shear zones created by either flexural slip during folding or other tectonic events. These pre-existing weak zones allow relative displacement to occur along bedding planes or between rock types, such as occurred in case study 8 on lot B. Where beds of expansive rock are massive, swelling may occur across an entire area and express itself through general deep cracking in the ground surface, similar to cracks in expansive soil. Uplift pressures may approach 20,000 psf, far in excess of foundation bearing pressures or street design pressures. Housing damage, in response to large uplift pressures, follows certain patterns with acceptable tolerances depending on structure type; examples may be found in Table 6 taken from Meehan and Karp (1994) which includes houses that have been damaged by heave of expansive claystones. The geometry of damage to the house often indicates clearly the location and orientation of the swelling culprit. If a claystone bed passes under one corner of a house, the house will begin to take on a tilted appearance, usually noticeable in the roofline,

Table 6. Wood Frame Housing Differential Movement Standards/Tolerances/Observations

YEAR	PROJECT	LOCATION	STANDARD/OBSERVATION	REFERENCE*	DIFF /20'	REMARKS
1968	General	USA	L/200	BRABB	1¼"	Allowable Deflection Ratio for slab-on-grade to limit damage to superstructure
1975	Sharon Heights	Menlo Park, CA	6"+ differentials, claystone Ca~0.12 @ PI~50	RLM	3"	Serious structural damage but livable
1976	General	USA	L/500	UC Berkeley Coll of Engrg	1"	Brittle cracking
1980	Tennis Villas	Blackhawk, CA	1.5" to 3.3"/house; cuts/fills, GB & 6' perimeter piers. 11,400 PSF @ PI~41	RLM/LBK*	2¼"	Moderate damage; isolated interior piers shifted, kitchens/baths low
1981	Silver Springs	Lafayette, CA	3" to 4"/house; hillside cuts/ fills, variable fdn systems	RLM/LBK	2½"	Moderate/severe damage exacerbated by lateral
1982	General	California	3/16" per 4'	California State Contractors License Board	~1"	Acceptable construction tolerance
1986	Seeno Homes	Pittsburg, CA	1¼" to 4"/house; GB w/4' piers perimeter/isolated interior	RLM/LBK	3"	Moderate/severe damage, overpours/eccentric pier loads, poor framing
1987	Warmington Homes	Antioch, CA	1" to 2"/house; marginal PT slab design, deflection/tilt/ original construction	LBK	2"	Light/moderate damage; expansive claystone, transition lots, poor slab construction
1989	Hanna Ranch	Hercules, CA	1" to 1½"/house; PT slab w/col footings-concentrated loads PI~40, % finer than 2 microns ~ 40	LBK	1¼"	Slight damage to walls; slab shifted/cracked, Poor design/tendons misplaced

(from Meehan and Karp 1994)

the gutter line or decorative beams outlining the garage opening (Fig. 13a). This should not be confused with sagging that typically occurs across the garage door span (in the author's experience) in two car garages of tract homes built recently, which is usually due to a greater span than the beams are capable of handling. Typically a floor level survey will show the portion of foundation on expansive claystone is noticeably higher relative to the rest of the house (case study 4 and 5). In some cases this may only be slight, a matter of an inch or so and not noticeable unless inside the house. In extreme cases, it may completely wrack the house, such as individual beds of tuff in Mariner Point in Vallejo where one house was wracked five to six inches out of level. The distortion was clearly observable from the street in the out-of-kilter appearance of the house. Within the case studies, measured heave varied from 1.5 inches to greater than 8 inches (see Table 7).

Other typical residential structures susceptible to heaving claystone damage are residential streets and swimming pools (ubiquitous in California), with concrete flatwork decks being especially susceptible to damage. Occasionally a pool shell will crack, which leads to introduction of moisture into the subsurface, causing further swelling and cracking as a vicious cycle of cracking, leaking and further cracking develops (case studies 3, 9b and 10c). Pavements are subject to the combined efforts of swell, creep, and bearing failure as moisture content of soil increases. Damage tends to be cyclical and long term over claystones beds, and is marked by constant patching of road, curb displacements and a permanent humped appearance (Meehan, Dukes and Shires 1975). Various case studies and papers have estimated that swelling of expansive rocks occurs over an 8 to 12 year period before diminishing (Hollingsworth 1990; Meehan and Karp 1994). However, it is important to note that in the Sharon Heights case, one of the longest observed developments, heave and creep is still occurring and causing damage to streets and houses in some areas (personal communication, Don Johnson 1998).

Attempts have been made to introduce a systematic observation of cracking in residential structures as a diagnostic tool, and assign a geotechnical classification to these patterns, as described by Audile (1996). This is of interest to the novice who has never

Table 7. Case studies differential movement

Case Study #	Formation	Location	Cut	Heave
1	Whiskey Hill (Butano)	Sharon Heights, Menlo Park, CA	4 to 8 ft	1 in./ft cut
4	Whiskey Hill (Butano)	Orchard Hills Atherton, Ca	2 to 8 ft	3.5 in.
3	Whiskey Hill (Butano)	Atherton, CA	4 to 8 ft	2 in.
5	Panoche (Knoxville)	Seeno Homes Pittsburg, CA	20 ft unload	4 in.
6	Orinda	Blackhawk Ranch, Danville, CA	5 to 60 ft	1 to 2+ in.
8	Panoche (Knoxville)	Mariner Point Vallejo, CA	5 to 9 ft	5 in.
Northern California, location C	unnamed shale	Ben Lomond, CA	minimal grading	~ 1 in.
9	Modelo	North Ranch Thousand Oaks, CA	10 to 24 ft	3.5 in.
10	Mint Canyon	San Gabriel Mts., Forrest Park, CA	8 to 10 ft	>1 in.
11	Saugus	Simi Valley, CA	0 to 4 ft	3 in.
11	Saugus	Porter Ranch, Los Angeles, CA	25 to 90 ft	4 in.
12	Puente	Dodger stadium, Chavez Ravine, Los Angeles, CA	0 to 80 ft	6 to 8 in.
	Topanga	west hills above Los Angeles, CA	60 to 80 ft	3 in.

observed residential cracking patterns before, but overdoes the classification of cracks. Too much emphasis is placed on the detailed description of cracking rather than the use of this information in conjunction with geologic data, laboratory data and other observations. The important thing to note about cracking is: whether it exists; where it is in the house; the sense of movement (shear) from the cracking pattern; and the type and loading of the foundation. These observations should then be correlated to information regarding the geology, lot grading, drainage, maintenance, construction practice and quality. Expansive soil typically causes lots of cracking in doorways, windows and other openings as well as sticking of doors all over a house, whereas cracking or heave from discrete beds of expansive rock will be localized or a pattern of shear consistent with bedding width, strike and dip will develop if there are several claystone beds involved. This can be complicated by the swelling mechanism, for example, or patterns of moisture availability.

3.2.2 Location of Expansive Rocks in California

Expansive claystones in California are located throughout the state, but only become problematic when development encounters them. This development may be in the form of highways or mass housing. The case studies discussed herein and other case histories not included are for the most part located in the younger mountain ranges of California, the Coast ranges, the Diablo Ranges of northern California and the Santa Monica range, the Transverse ranges of southern California as well as the coastal area of San Diego (Day 1994; Hollingsworth 1990; Meehan, Dukes and Shires 1975; Meehan and Karp 1994; Smoots and Melickian 1965). Fig. 5 and Fig. 28 show the locations of case studies investigated by the author and information gathered from others. These locations are not all inclusive of all problems reported or known in California, although the author is currently investigating this topic. A summary of expansive claystone engineering properties from the case histories discussed is presented in Table 8.

Typically, the location of expansive claystone damage is new housing development in the hills and low mountains surrounding an already heavily developed area. It is commonly accepted among those associated with the housing industry that the best and easiest to develop sites in suburban California have been used up many years ago. Cutting the tops and knobs of ridges and filling in the ravines, gullies and small stream swales in between the knobs makes a suitable site for housing development. As much as 25 to 50 feet of cut is made in some area to create flat lots, either on the ridge top or stair stepped down the hillside (see Fig. 34). The fill is then used to level out the site by filling in the swales and ravines. The Blackhawk Tennis Villas development is a perfect example of this type of development, which is commonly found in the case studies discussed. What do the locations in California have in common aside from being hillside development? Does this mean that every hillside in California potentially contains expansive claystones?

It is clear from the type and age of rock formations in the case studies that there is a geologic pattern. The majority of the expansive rocks are claystones formed in a marine or quiet water environment, and are typically interbedded, sometimes very thinly, with non-expansive beds in the same formation. These rocks are typically upper Cretaceous to

Table 8. Damaged Project Expansive Rock Properties

case	Project	LL	PI	Free Swell, %	Swell pressure, psf	Swell Index, Cs	Expansion Index	Comments
1	Sharon Heights, Menlo Park		50	100%	4,000 to 9,000	.10 to .12		
2	Orchard Hills, Atherton		48	100 to 190%	> 3,000		167	
3	Fletcher Way, Atherton	60-72	34 to 44					
4	Audifferd Lane, Woodside	92	49-59					6 years heave already
5	Seeno Homes						>200	
6	Blackhawk Ranch		34			.05 to .09		
9	North Ranch Thousand Oaks, CA					.08 to .16	66 to 200	
10	Santa Clarita					.023 to .066	132 to 165	surcharge loads 300 to 1500 psf
10	Forrest Park, Los Angeles					.051 to .088	107 to 120	surcharge loads 300 to 1500 psf
11a	Porter Ranch					.07 to .094	120 to 165	surcharge loads 300 to 1500 psf
11b	Simi Valley					.07 to .065	142	surcharge loads 300 to 1500 psf
	Troublesome Claystones		40	>100%	> 3,000 psf	Cs > .10	EI>130	

Eocene in age in northern California and generally Miocene in age in southern California, which were volcanically periods. A closer examination of the geologic origin of these rocks reveals a common geologic pattern that can be used to identify areas containing expansive rock formations.

4.0 Geology of Expansive Rocks in California

Claystones, shales and other types of expansive rocks are generally fine-grained and contain an abundant amount of clay. The usual environment for deposition of such fine grained rocks is in a basin, usually quiet water settings, where the source rock contains abundant clay minerals or minerals that weather into clay. In California, the hillside sites that have expansive rock problems are made up of geologic formations that are usually Tertiary in age (2 to 65 million years old), Eocene to Miocene, marine in origin, and related to widespread volcanic activity that produced clay rich sediments in California at that time. There are some formations from the late Cretaceous period, such as Panoche (Knoxville) shale that have damaged residential developments like Mariner's Point development in Vallejo, but these sediments were also deposited in shallow marine environment at the edge of the great central basin California more than 65 million years ago. Clearly there is a relationship between expansive rocks, mostly claystones in California, and the age of rock formations, their location with respect to continental margins and tectonic plate boundaries and the depositional origins of the rocks. Looking at the National Geographic's Atlas map of distribution of Cenozoic rocks worldwide (Fig. 1), it is possible to predict in which country expansive soils and rock problems will be found. A comparison to the geologic and engineering literature's record of case studies bears this out (Gysel 1977; Lindner 1976; Madsen 1979; Peterson 1958; Taylor and Smith 1986; Venter 1981).

It is the clay minerals that give the mudrocks their swelling characteristic. Expansive rocks, typically claystones in California, are usually among the youngest rocks exposed at the surface of the earth for several reasons: they are found at the continental margins where new material is accumulating and volcanic activity is common, they have not been altered yet by burial diagenesis and thereby lose the expandable clay minerals they contain, nor has the claystone been metamorphosed into well-cemented shale.

4.1 Basic Clay Mineralogy

Clay minerals are sheet minerals (or sheet silicates) made up of layers of four sided and eight sided structures of Hydroxyl (OH), oxygen, silicon, aluminum and magnesium. Many of the sheet silicates are essentially uncharged structures, like the micas, and are largely responsible for the storage of water in the soil from season to season. Some clay minerals contain water and cations (positively charged atoms) as part of their basic structure and the balance of charge between the clay mineral structure and water/cations has a large effect on the swelling behavior of claystones made up of these minerals. Because of the nature of their construction, these particular clay minerals are highly susceptible to breakdown and swelling.

There are three types of clay minerals that are most susceptible to swelling (in order of most to least swell), montmorillonite (smectite), illite and kaolinite. Their structure is illustrated in Fig. 38. Montmorillonite, the most common clay mineral found in expansive rocks causing damage, is made of the pyrophyllite structure with sheets of

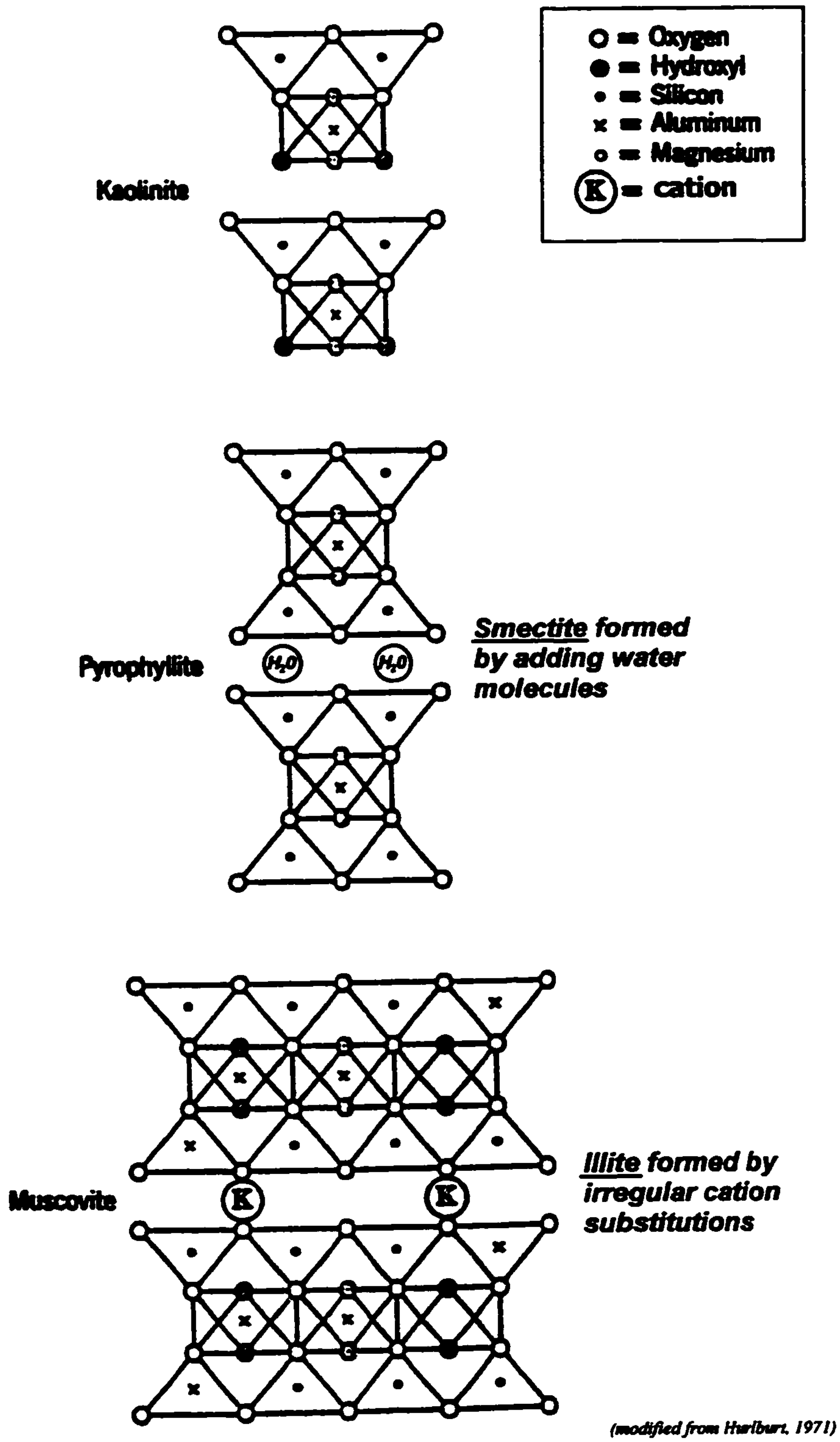


Fig. 38. Clay mineralogical structure (from Hurlbut 1971).

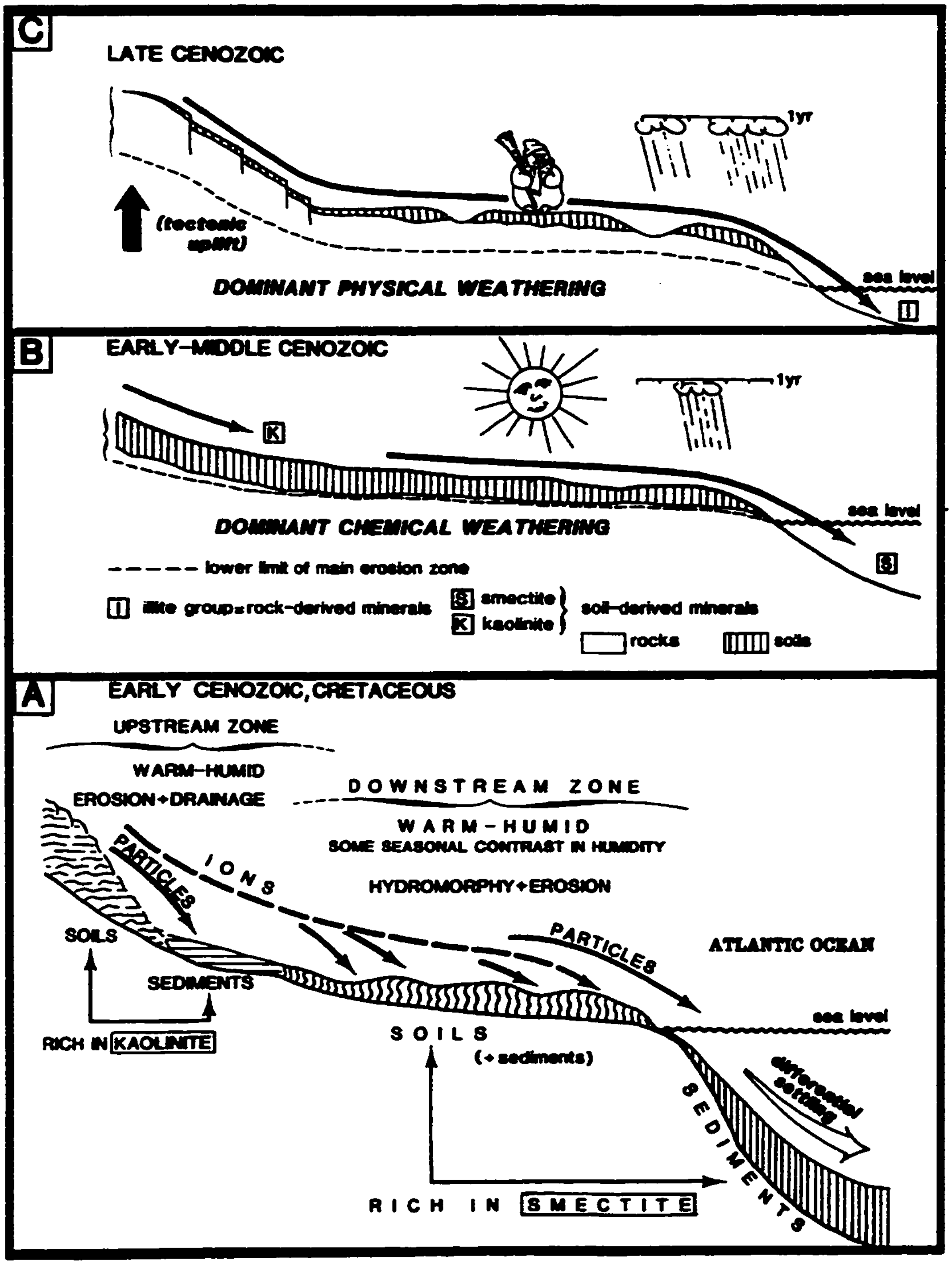


Fig. 17.8. Tentative interpretation of clay sources in late Mesozoic-early Paleogene sediments (A) and late Paleogene-late Cenozoic sediments (B, C) of the Atlantic range. (After Chamley 1979)

Fig. 39. Chamley's interpretation of clay sources, and their relationship to climate and weathering.

molecular water inserted between the layers. Illite is a mineral structurally like muscovite (a mica mineral), but with irregular cation substitutions between the clay sheets, which causes it to behave more like montmorillonite than a benign micaceous mineral. One of the basic structural differences between the clay minerals are the lack of a bottom sheet on kaolinite, which plays an important role in limiting its ability to swell in response to moisture changes (Hurlbut 1971).

These three clay minerals are present in varying portions in siliceous mudstones, depending on the source of the sediment being deposited and also depending on the rate of erosion and deposition of clay sediment and the amount of weathering the clay mineral is subjected to before being deposited in water or buried. Montmorillonite is preferentially formed from volcanic rocks, particularly when volcanic ash and glass are deposited in marine basins, where there is an abundance of cations to react with the minerals.

Tectonic activity and climate largely control these factors influencing the formation of swelling clay minerals, with the distant past and present climate of western North America favorable for the formation of montmorillonite (Chamley 1989; Tourtelot 1974). Fig. 39 illustrates how weathering affects clay mineral composition of sediment (from Chamley 1989). The more weathered a soil is, the greater percentage of kaolinite it contains. The less weathered a soil is, the more smectite (and therefore montmorillonite) it contains. Therefore, clay rich rocks and soils formed in a somewhat cooler and less tropical climate with less intensive weathering tend to contain clay minerals with greater swell potential than heavily weathered soils formed in warmer climates. This model fits well with the observation than the Tertiary period was one of cooling down of the earth, particularly during the early Tertiary in the Pacific Northwest. Deposition of clays in sedimentary basins are often used as climate markers to indicate whether the area was experiencing a cooling or a warming trend during the time of deposition (Chamley 1989).

4.2 General Stratigraphy

Taylor and Smith (1986) catalogued the minerals that make up mudrocks worldwide and compared North America with the United Kingdom. Clay minerals typically make up 50 to 70% of the mudrocks, Table 9 from Taylor and Smith gives an indication of the swelling potential of the clay minerals. The age of mudrocks is correlated with their dominant clay mineral make up in Fig. 40 (Taylor and Smith 1986). This chart is revealing when compared to the age of the expansive mudrocks which have been described as troublesome in the literature; there is a good age correlation with rocks having the highest percentages of expandable clay minerals, montmorillonite (a subgroup of the smectite clay group) and illite, and lowest percentages of kaolinite. For example, in the western United States in the author's experience, rocks which have caused heaving damage are all within the upper Cretaceous to Tertiary in age (2 to 140 million years old). In central Europe and the Swiss Alps, rocks as old as upper Jurassic have posed problems for tunneling. This corresponds well with description of the clay mineralogy of rocks in the United Kingdom. This chart also demonstrates the loss of expandable clay minerals that typifies older rocks and comes with burial diagenesis.

Table 9. Clay Mineral and Swelling Potential

Clay mineral	Average free swell (%)	Range (%)
Na-smectite	1,500	1,400-1,600
Ca-smectite	102	65-145
Illite	89	60-120
Kaolinite	28	5-60

(from Taylor and Smith 1986)

4.3 Depositional Environments

California's expansive rocks are related to the general geologic activity that goes with the state's location on the edge of the Pacific Rim. Crustal disturbances gave rise to volcanic activity in the central part of the state during Tertiary times, that is some 35 million years ago. Volcanoes produce ash, which falls to the ground or into water and is deposited as beds of ashy sediment (e.g. tuff) which subsequently weathers to montmorillonite-rich clay. (Much the same process occurred during the recent eruption of Mount St. Helen.) Claystones and shales are the result of deposition of clay particles in low energy settings (environments). Clays accumulate in lakes and oceans, in quiet bays, deltas or deep-water basins. Tourtelot (1974) makes an excellent discussion of the geologic origin of swelling clays. As a tectonically active area located at the edge of an ocean, the clays that were deposited in these basins frequently were not subjected to much weathering during transport, and consequently, a large percentage of the clay remained smectite, the mineral most susceptible to swelling.

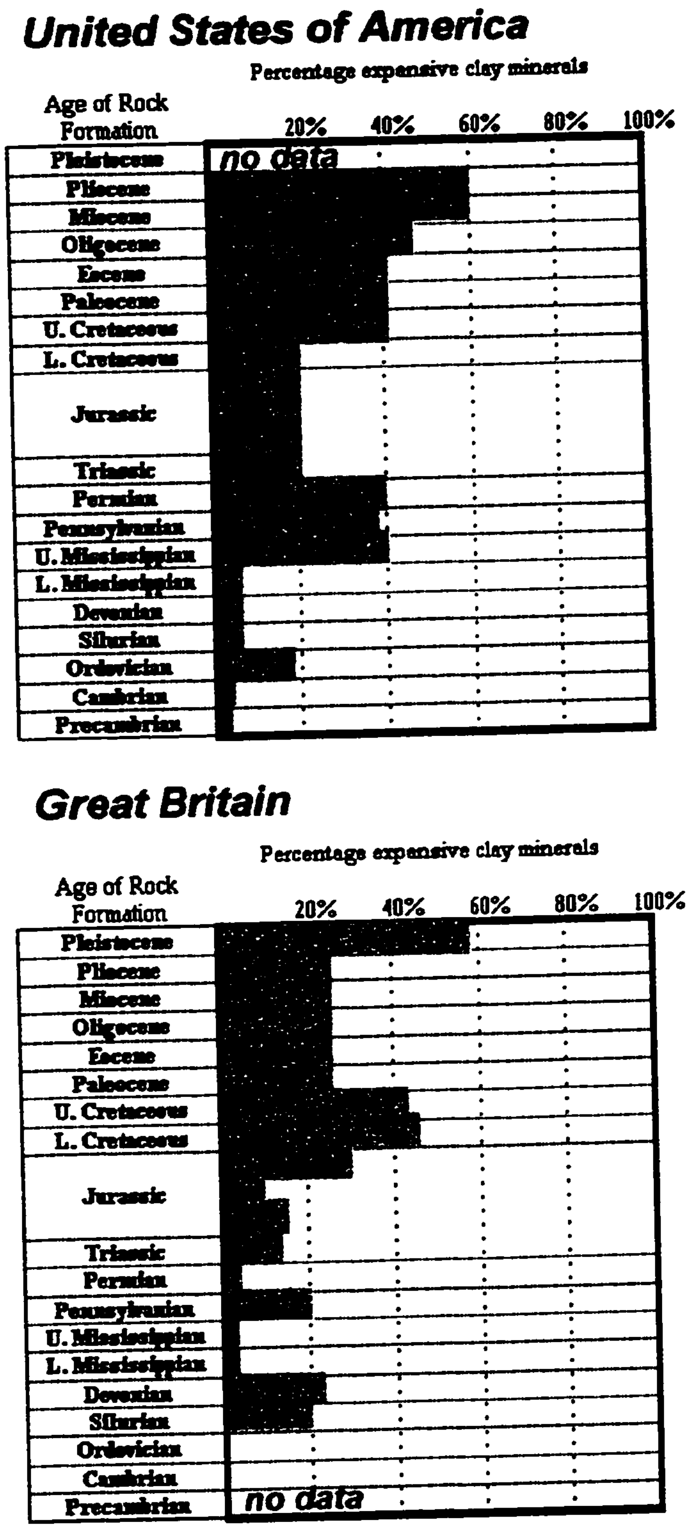


Fig. 40. Percentage of swelling clay correlated with age of rocks (after Taylor and Smith 1986).

Accumulation of clay particles in quiet water basins occurred during different time periods in northern and southern California. In northern California the last major accumulation of fine sediments in interior basins occurred during the Eocene and Pliocene period (CDMG 1951, Sedlock 1996). Examples are the Whiskey Hill formation and the Kreyenhagen formation. In southern California, it is the Miocene period during which the Pacific plate boundary formed pull apart basins in which clay sediment accumulated, as represented by the Miocene Saugus, Puente and Modelo formations.

As the clay sediments accumulate in these basins, they consolidate. Consolidation is the process of volume reduction by expulsion of water, and hence decreases in porosity. As clay consolidates, the clay minerals and particles mechanically rearrange to tighter packing. Claystones formed by deep burial in this manner are compaction claystones. Once consolidation is complete, further volume reduction occurs by the processes of diagenesis. Diagenesis is the chemical and physical breakdown and rearrangement of clay minerals, including crystallization of elements. If a claystone is subjected to tectonic pressure and heating, it will become hard, cemented, and fissile, turning into shale. However in California, where the landscape is young and processes occur fairly rapidly, the clay rich deposits are often not subjected to long burial, or metamorphic processes. The cumulative result is an accumulation of young sedimentary deposits possessing little secondary cementation or lithification and rich in smectite clay, i.e. so-called compaction claystones variously interbedded with sandstone and siltstones. In California, both claystones and tuffs can be related in the depositional sequence and are frequently found together the Panoche formation and the Kreyenhagen formation in the Diablo Range of the east bay.

Continuing tectonic disturbance of these beds has resulted in their being folded and faulted, lifted in some cases to hills ranging up to a thousand or so feet above sea level. The folding process has produced gently to steeply dipping beds, typical of the East Bay Coast Range, while other tectonic processes such as faulting have produced a mixed chaotic melange, typical in the Whiskey Hill formation on the San Francisco Peninsula. Typically the more clay-rich members of these formations quickly revert to highly plastic clay with (PI's of 25 to 50%) when exposed to wetting or drying, unloading or disturbance by excavation (Meehan and Karp 1994). These same rocks are frequently known for other types of instability, including landslides, soil creep or flows. Older clays present in the basement rocks in the San Francisco Bay region are less plastic and do not give rise to severe problems.

How a claystone forms determines its strength characteristics, whether it is strongly or weakly cemented and what kind of physico-chemical bonds there are between its particles. It also determines whether a claystone is over-consolidated or normally consolidated. The amount of consolidation is key to understanding how a claystone might swell when cut, either vertically in response to removal of overburden or horizontally into trench cuts. Thick sheets of glacial ice have over consolidated the claystones and shales in Canada and the northern United States. These clays and shales tend to swell horizontally into cuts, trenches and river valleys as they are down cut. The horizontal stress in these clay and shales is many times greater than the vertical stress

(Peterson 1958). In areas with great tectonic deformation, such as the Swiss Alps or the Rocky Mountains, where rocks have been folded and uplifted thousands of feet, clays and shales deform in all directions, sometimes preferentially if there is a controlling factor such as joints or mono-oriented crystals

In California, active mountain-building forces and crustal deformation along faults is likely to provide additional stress on weak, clayey beds. Failure is more likely to occur along bedding planes with claystone beds or at claystone bed contacts. This phenomenon was apparent in case study 5 at the parcel A in Pittsburg where a 50 foot high reservoir embankment was removed for development. In test pits logged by geotechnical engineers in case study 6, shear zones were noted along contacts that also demarcated the boundary of swelling areas.

4.4 Geologic Relationships

Because claystones and other expansive rocks in California generally share similar physical and chemical characteristics, it is reasonable to assume that their depositional environments are similar. A review of published case studies in the United States and descriptions of the expansive bedrock indicate that almost all of the rock formations are of upper Cretaceous or Tertiary age. This is true in the author's experience as well.

Rocks age from upper Cretaceous to Tertiary span a large time frame, 95 million years. While the gross generalization of expansive bedrock associated with this broad time frame is somewhat useful, is it possible to make a more refined correlation within that time frame in California? A comparison of geologic formations in California known to contain expansive claystones and age correlation in northern and southern California reveal similarities in geologic source rocks and depositional environment, despite very different geographic regions. Tertiary and Cretaceous geologic correlation of the Pacific Coast and Midwest from Popenoe et al. (1960) and Wood (1941) reveal correlation between units in California and the Midwest rock known for expansive bedrock. Within the upper Cretaceous, the Pierre shale in the Midwest, the Eagle Ford shale in Texas, the Holly shale (in the Santa Ana Mountains, southern California), and the Panoche formation (Diablo Range, northern California) are similar in age. The Panoche and Holly shale straddle the time period between the Eagle Ford shale and the Pierre shale in the geologic columns. The Pierre shale is well known for its expansive properties and has been studied in detail ("At Oahe" 1959; Higgins 1997; "Structures" 1960; Noe 1997). While general age correlation is useful, it is only a beginning point for determining whether a particular rock formation is likely to contain an expansive rock layer and how those rock layers are likely to respond to changes in load and moisture content.

To find expansive rocks in California, particularly when there is no previous local experience as a guide, it is important to understand the geology of California and the origins of the rocks, their history. The geologic origins of rocks contain the key as to whether they are likely to be expansive or not. California contains relatively young terrain, largely Franciscan and younger in age (140 Ma), while the largest portion of rocks falls with the Cretaceous and Tertiary ages (140 to 2 Ma). The basement, or bottom of the bedrock, consists of oceanic crustal rocks that were brought to the North American continent (California) by the eastward movement of the Pacific Plate. In the eastern portion of the state, the Sierra Nevada formed a volcanic upland that eroded rapidly and deposited large amounts of sandstone, shale and limestone in deep marine and inland basins. The San Francisco Bay region was then a volcanic arc, accumulating rocks from far off to the west, along a subduction zone. The rocks melted as they descended into the earth's subsurface and were then reconstituted as continental or submarine volcanic rocks.

This subduction zone eventually became the San Andreas Fault, which divides California, north to south, into two different geologic environments. There is a great deal of debate in the geologic community on exactly where the rocks on the west side of the San Andreas fault came from and exactly when they arrived in California (Sedlock 1996). However, it is generally accepted that the plate margin in the San Francisco Bay region transformed from a subduction zone to a transform boundary (the San Andreas, Hayward and Calaveras fault systems) between 7 and 11 Ma (Sedlock 1996). Much of the current landscape has formed since the Pliocene (6.5 Ma). In southern California, this transition occurred at 25 Ma, when the east Pacific rise collided with the continental edge (Brown 1993). Northern California developed a strike slip fault along this boundary, while more complex forces caused the plate boundary to buckle in southern California, forming the Transverse Ranges, pull apart basins inland and along the coast, and eventually creating a new ocean rift basin in Baja around 5 Ma.

During the Cretaceous periods, large amounts of sediments were deposited in thick sequences into what is now the San Joaquin Valley (CDMG 1951). These sediments consist of rhythmically interbedded sandstone, siltstone and claystone deposits. During late Cretaceous through Eocene time, sea level was high and dinosaurs still roamed the earth. A climate change, during which the earth cooled and sea level dropped as glaciers formed, marked the boundary between the Oligocene and the Eocene and the beginning of a dramatically cooler earth. This combined with the change in the Pacific continental margin can be used to examine the age distribution of the expansive claystones in northern and southern California and construct unifying geologic factors that might help predict the likely presence of highly expansive claystones.

In northern California, along the volcanic arc before the Pacific margin became the San Andreas fault, volcanic activity provided abundant clay for deposition into marine sedimentary basins, with virtually little to no weathering, during a period of high sea level in Cretaceous through Eocene period. After the Oligocene transition to a cooler time period, terrestrial deposition dominated the regional sedimentary rocks. With the reemergence of volcanic activity during the Pliocene, and as the plate margin was pushed inward under the continental rocks, the majority of the clayshale deposited was terrestrial, not marine, in origin.

In southern California, there is a similar pattern of Eocene marine shale present along the coastline near San Diego. However, as sea level began to drop, a lack of volcanic source rocks and a highland source area lead to a distinct lack of terrestrial clay shale deposits in this area. Further north in Los Angeles, clay shales are dominantly marine, however they are post-Eocene. If sea level dropped after the Eocene, how can this be explained? The Los Angeles basin was undergoing extension at this time, because the east pacific spreading plate margin had been overridden near Baja, California, at about 25 Ma (post-Eocene). A series of pull apart basins were formed which allowed deposition of thick marine sedimentary sequences. Volcanic activity increased as these basins were forming, providing a rich source of illite clay for deposition in claystone beds. However, when examining these geologic relationships, it cannot be assumed that everywhere there are sedimentary rocks of Eocene age in northern California and Miocene age in southern California, that they contain expansive rock members. These geologic relationships are useful in locating potentially expansive rocks, and understanding their stratigraphy and clay mineralogy. Understanding why and how these rocks swell is also critical to any attempt to describe their behavior.

5.0 Swelling Mechanisms

5.1 What Is Swelling?

Terzaghi defined swelling of rock as the change in volume or water content due to load relief. Swelling is controlled by the same principles that control consolidation, and typically occurs very slowly (Terzaghi 1950). This has also been referred to as rebound (Peterson 1958) or deformational rebound. Many studies have been made of the mechanics of swelling of clays, and periodic reviews of swelling rock have been published in rock mechanics (Einstein 1996; Lindner 1976). Other studies are the result of the study of rock mechanics as part of the search for suitable nuclear waste repositories, such as the studies completed in the early 1980s by the Institute of Geological Sciences in Britain.

The studies of the mudrocks being considered for nuclear storage at Harwell have yielded valuable understanding of the swelling mechanisms of clay particles in claystones, including correlation of engineering indicators of swell potential (Hobbs et al.

1982). The forces contributing to swelling are grouped into three types: repulsion forces, contact forces, and clamping forces. Repulsion forces correspond to attraction of water to clay minerals and the subsequent expansion of the clay mineral structure to accommodate the water. Repulsion forces consist of hydration forces (adsorption of water onto the surface of the clay minerals) and osmotic forces (charges that hold water tightly between clay particles and causing them to move apart).

Contact forces are related to the elastic rebound of clay particles. In over consolidated clays, this may be the most important factor in swelling, and is related to relief of overburden pressures (Hobbs et al. 1982; Meehan, Dukes and Shires 1975). It is also related to development of shear zones between layers of bedrock that can develop when large stress relief occurs quickly. Clamping forces are related to contact forces and are the equivalent of confining pressures that overburden places on rock, or that may be replaced by distribution of foundation loads in structures.

Mudrocks with high clay mineral content are most typical of expansive rocks, but even shales or siltstones with trace amounts of clay can create a fair amount of expansive force. Also, differing predictions of swell based on different index tests may indicate that more than one swelling mechanism is at work. Changes in moisture content, load, and chemistry occur in combination and contribute varyingly to swelling of claystones depending on the mineral make up of the rock. Research into the swelling of clay minerals is still on-going, and recent studies indicate that swelling mechanisms still may not be fully understood. This may explain the somewhat erratic and inhomogeneous nature of swelling and the inability to predict the behavior of swelling claystones.

5.2 Basic Mechanisms of Swelling

When expansive rocks swell, it is in response to a disturbance of equilibrium. Even in a tectonically active area such as California, it can be assumed that enough geologic time has passed for rock to have come to a state of equilibrium the climate and geologic processes. The traditional soil mechanics approach to consolidation or swelling generally deals only one direction of stress, vertical stress (z direction). While rock mechanics uses a more complicated model and generally deals with all three directions of stress, x, y, and z, the assumption of a state of equilibrium makes it easier to deal with only changes in the total stress of the rock.

The soil mechanics definition of total stress (in the vertical direction) is the summation of effective stress and pore pressure. In looking at the problem of expansive rock, this translates to load and water. In California and elsewhere, expansive claystones are perennially short of water. Claystone undisturbed typically have reached a moisture balance, with the claystone at depth having a lower moisture of 8 to 10% below that of the weathered rock at the surface, as shown by the case studies presented. When load is removed, that is the total stress of the rock is lowered, pore pressure becomes negative and the rock tends to absorb water and swell as a result. Effective stress is lowered, and elastic decompression occurs. While these are the basic mechanisms of swelling, it is complicated by differences in mineralogy and chemistry of expansive rocks, the inter-relationship between total stress, pore pressure and available water, time factors (occurrence and alteration of rock).

Unless discussing a unique and clearly identifiable mechanism of swelling in expansive rocks, such as growth of gypsum crystals or jarosite and other sulfites in pyritic shales (Moran 1989), or an immediate phenomenon of swell (Higgins 1997), it is difficult to distinguish between swelling due to hydration of clay minerals, plastic deformation due to long term changes in clay mineralogy and ambient moisture content, and elastic rebound due to load relief. Recent studies of swelling pressure in rock mechanics (Mesri, Pakbaz and Cepeda-Diaz 1993) support Terzaghi's initial suggestions that elastic rebound plays a small part in long term swelling of rock, and suggest that clay content and availability of water are the most important factors. Rebound is a more important phenomenon in crystalline rocks (Harper et al. 1979; Nichols and Savage 1976), although it does occur in sedimentary rocks with large magnitude of overburden removal (up to 90 feet) (Arnold and Hanegan 1974).

5.2.1 Changes in Moisture Content

Increase in moisture content is typical of swelling claystones. Moisture is introduced by long term changes in the climate (increase rainfall), grading and exposure of fresh unweathered rock to the atmosphere changes in the ambient moisture available), or by construction (e.g. drainage facilities, tunneling). Clay minerals, especially montmorillonite, have water as part of their structure, and due to the plate-like nature of their structure, they are sensitive to changes in moisture. This tends to be truer of fine-grained claystones. Water molecules within the clay mineral structure are either absorbed into the interstices of the clay mineral or form water molecule layers between the sheets of clays. While moisture in the upper few feet of soil changes seasonally, longer-term changes in moisture content occur over drought cycles (years) and over long term climate changes (decades and centuries) and occur to much greater depths. Recent studies of montmorillonite and modeling of the swelling process have shown that swelling between clay sheets does not occur linearly throughout the progressive formation of water molecule layers (Karaborni et al. 1996). Rather, their modeling shows that water molecules preferably form one, three or five layers of water; or, water molecules are completely absorbed with the clay mineral interstices. These preferred layer configurations correspond with minimum swelling pressures measured throughout the observations of swelling. Therefore adding one more molecule of water than can be accommodated by one layer, forces the clay sheets apart to a three layer configuration (perhaps by creating the need for more water by an energy imbalance). One conclusion that might be drawn from this is that relatively small changes in moisture content could produce large changes in swelling pressures. This also represents a fundamental change in swelling models from a linear model to a non-linear, episodic model.

Changes in the chemistry of claystone rocks can occur with the addition of water, by creation of a cation imbalance, by leaching of salts or by inducing growth of crystals within the rock fabric (Lindner 1976). This is largely a problem with fine-grained claystones containing smectite and illite. In the Swiss Alps, the Tertiary and Jurassic mudrocks often contain layers of anhydrite, or calcium sulfite, which is completely lacking in water and typically, formed as an evaporite. It was discovered that water seeping into tunnels had passed throughout evaporite beds and leached out the calcium sulfate. When the water found its way into the tunnel or fractures near the tunnels, gypsum crystals would form, causing additional swelling pressures on the rock (Gysel 1977). Gypsum crystal formation has also been observed in claystones in the Bay area.

5.2.2 Changes in Total Stress

Changes in total stress of rock are caused by tectonism (uplift or burial), erosion, and excavation (removal of confining pressures). When this occurs, claystones tend to rebound or swell in response to this pressure relief. This is mechanism is mainly seen in coarse-grained, kaolinitic claystones. Peterson (1958) explains this as part of a two step rebound process: (1) short term immediate elastic rebound and (2) long term "time rebound," or deformational rebound, that takes place over many years. Elastic relief is self-explanatory and is illustrated by Fig. 41 from Peterson. A spillway constructed in the Bearpaw shale, an expansive shale in western Canada, experience heave in the bottom slab, as shown on the upper portion of the figure. The maximum heave of four inches coincided with the deepest excavation of overburden and shale, approximately 45 feet, or about 1 inch or heave for every 10 feet of cut.

Time rebound is more complicated and involves long term softening of the rock, an increase in natural moisture content and accompanying change in volume. Peterson illustrates this in Fig. 42, which summarizes the hardness and moisture content of the Bearpaw shale from the surface to some depth. The shale has a profile that typically is soft rock near the surface with natural water contents of 30 to 35% and increases in hardness with depth, with a corresponding decreasing natural water content to about 20%. This is demonstrative of the effects of "time rebound" stress relief. During the retreat of the last glacial stage in Canada, it is estimated that approximately 2,000 feet of sediment

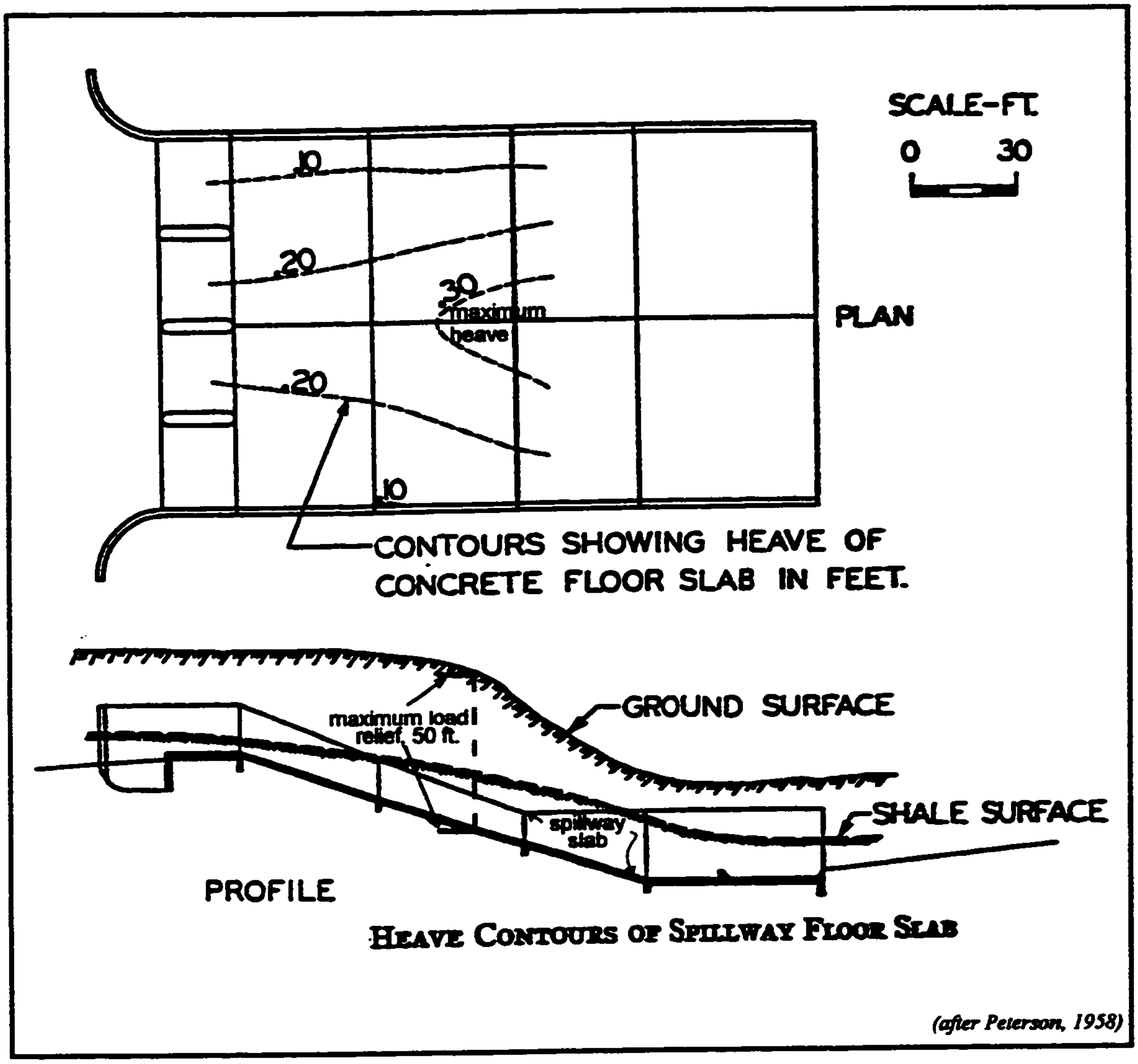


Fig. 41. Swelling due to load relief in the Bearpaw shale in Canada (modified from Peterson 1958).

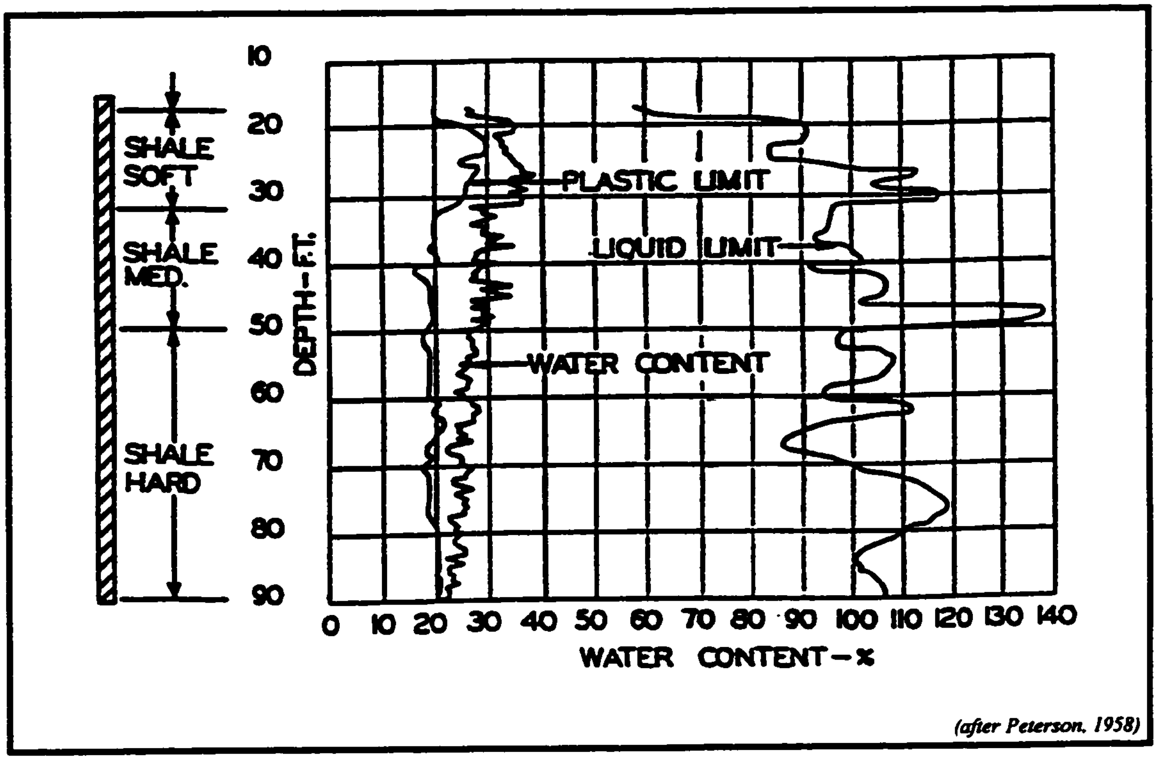


Fig. 42. Changes in moisture content and rock durability with depth, Bearpaw shale, Canada (from Peterson 1958).

was removed by erosion from this area along with glacial ice. Peterson estimates a removal of overburden stress equal to 100 to 150 tsf (200 to 300 ksf), or the equivalent of an excavation one-third of a mile deep in material averaging 125 pcf unit weight.

Contributing to the "time rebound" is the exposure of fresh rock to weathering, and the formation of new clay minerals. Also the immediate elastic relief creates fractures in the rock, which allows additional infiltration of water into the rock. Sometimes the swelling manifests itself as failure and movement along joints and/or bedding planes within the rock. This has been observed in the Bearpaw shale and in California claystones, where slickensides along bedding planes, indicating movement, have been observed in exploratory trenches (Hollingsworth 1990; personal communication, Jo Crosby 1996; personal communication, Allen Seward 1997).

5.2.3 Changes in Chemistry

Cations are present within the crystal structure of clay minerals, mainly concentrated at the outer layer of the sheets, where they are attracted to the negative polar ends of the water molecules (Lindner 1976; Taylor and Smith 1986). These cations are exchangeable, meaning they can be replaced or removed. The principle cations involved are calcium, magnesium, potassium and sodium, in descending order of their natural abundance and ability to be exchanged. The chemical balance can be changed by introduction of water that has no cations (dissolved solids), which induces a concentration gradient and causes the cations to move out from the sheets of clay into the water. As this occurs, the clay sheets become more negatively charged and the forces of repulsion increase, causing swelling. This phenomenon has been documented in numerous studies (Higgins 1997; Lindner 1976; Noe 1997). Ironically, the lower the cation concentration in the clay mineral to begin with, the higher the repulsion forces that can develop. This is because if there are an excess number of cations that can be lost without creating a negative charge between sheets, then there is no corresponding increase in repulsion forces due to loss of cations (Taylor and Smith 1986).

The same principle applies to the leaching of salts from the claystones. If solute free water is repeatedly introduced into saline rich claystones, aside from inducing swelling due to moisture changes, swelling due to loss of cations will occur. This has been found to be true in even in claystones not susceptible to moisture changes, typically coarse-grained kaolinitic claystones (Taylor and Smith 1986).

5.3 Factors Affecting Swelling

Many different factors affect swelling of claystones. Because this is true, when investigating a house of other structure damage by swelling, it is important to have a thorough understanding of site geology, site physiography, the development of the site and the changes that have occurred on-site as a result of development.

5.3.1 Mineralogy

Smectite clays make up the largest percentage of swelling clays in the United States, both in soil and in rock (Olive et al. 1989). In California, the predominant type of clay mineral found in swelling claystones is montmorillonite, part of the smectite group of clay minerals (Chamley 1989). Chamley characterizes the Tertiary period as dominated by smectite clays, and a greater variety of clay minerals during the Quaternary (illite, smectite, kaolinite, and chlorite, all different stages of weathering). This is due to the fact that volcanic rocks weather preferentially into smectite clays, regardless of climate. The weathering of volcanic rocks, the temperate climate and the constantly changing ever-rising western continental edge all combined to provide a stream of smectite rich clays into the Miocene basins in southern California and the shallow volcanic arc areas in northern California. This is particularly evident in the thin beds of montmorillonite that are found within the Modelo formation and that underlie many of the large landslides on the Palos Verde Peninsula.

5.3.2 Grain size

Typically claystones are made of 60 to 65% fines (passing #200 sieve), the majority of fines being clays size minerals. The bulk of the remaining minerals consist of 20 to 35% quartz. The more fine-grained the sediments that make up the claystone, the waxier it will appear and the more likely to contain smectite clay minerals (the most expansive). This waxy looking claystone is often more expansive than other adjacent coarser-grained claystone beds, as observed in case studies 1 and 2.

5.3.3 Rock Fabric

Because clay minerals are plate-like structures, and swelling forces develop between these plates, the orientation of these minerals in the rock (or rock fabric) can have a dampening or magnifying effect on swelling of claystones. If the clay and soil particles are flocculated, oriented randomly within the rock, then there is less surface area between clay sheets to develop swelling pressures. However, if the clay has a dispersed structure, a preferred orientation, instead of a flocculated structure, this concentrates the swelling force in one direction (personal communication, Jo Crosby 1997). This is also indicated by a waxy appearance of the claystone, as discussed in the previous section 5.3.2.

5.3.4 Stress History

The stress history of a site, as discussed above, has much to do with the swelling of claystones. Particularly relevant to geotechnical engineering is the disturbance of a site due to cut/fill activities or foundation excavations. Even if no immediate elastic rebound manifests itself, change in the total stress of a material can induce an increase in pore pressure as the material begins to equilibrate to the new state of stress. Construction methods have been developed for dams to combat the effect of load removal, including immediate pouring of foundations to replace overburden load (Da Costa Nunes 1979), and covering exposed expansive rock with shotcrete to prevent water absorption ("At Oahe" 1959). Removal of old structures can produce the same effect as the removal of glacial ice, as the author observed on lot B in case study 5. The claystone bed beneath the old reservoir embankment swelled along a preferred bedding/shear plane more than one year after load removal, as shown by foundation displacement. The physical environment in which the claystone is exposed can have a large effect on the swelling of expansive rocks. Exposure of unweathered claystone to the elements, and changing the moisture climate of the claystone will produce the greatest change in swelling regardless of the swelling mechanism. This can be due to irrigation, grading, and construction of structures over claystone (building or drainage), or natural processes (such as landslides, gullying, and change in natural water content).

5.3.5 Drainage

The availability of water is a consideration in almost all foundation design and construction now, especially in the western U.S. where there have been many problems with expansive soils. Some of the sources of water that have been found to cause problems in the past have been irrigation, pool drains, landscaping and even drainage systems designed to carry water away from the site. The focus has been on sources of surface water or shallow water, which are observable and controllable. While it is laudable to minimize the availability of water to sites with swelling clays and rocks, rarely is consideration given to the natural availability of water, particularly in California, where the seasons have radically different moisture regimes and droughts are not uncommon.

The more important consideration, mainly for rocks that swell in response to moisture changes, is to maintain the moisture at a certain level, minimizing the effects of environmental changes. This is not always possible, for sometimes the available water is from an unusual source. The fractures in the otherwise generally impermeable claystone act as a conduit for water from surrounding more permeable rocks, and can cause swelling. This is typical in the Bay area where claystones are part of interbedded sequences of sandstone and siltstone. This illustrates that it is not always possible to control swelling of claystones, even when the mechanism of swelling itself is understood.

All of these factors must be considered in the design of the project and considering how best to handle the presence of expansive rock onsite. It is the job of the geotechnical engineer to consider the above factors in the grading and foundation recommendations.

However, expansive rocks are not often recognized by the geotechnical profession, the swelling mechanism and factors affecting it are not understood, or the capacity for swell is under-predicted, as is evident in the case studies. Techniques have evolved over time to deal with expansive soil and rock, however, there is no common knowledge regarding expansive rocks in the geotechnical engineering profession.

6.0 Geotechnical Engineering Practice and Recognition of Expansive Rocks

How are expansive claystone hazards addressed in the current practice of soils engineering? The usual method for addressing such hazards, not to life, but to property and investment, is through a geotechnical engineering investigation and report prepared for the development project. To understand how the geotechnical engineering report is used in property development, it is necessary to discuss briefly the land development process.

6.1 Typical Investigations and their role in land development

Land development typically proceeds in six stages. First, the developer must acquire the property; second, the developer must get approval from the planning commission for the development proposed. This second stage requires an Environmental Impact Statement (EIS) by the local planning department describing the effect of the development on existing community services, such as utilities, schools and roads, and identifying any new services required. The third stage requires a tentative subdivision map approval. Once the developer has the permit for the land use and an approved subdivision map, then the process of land development begins in earnest. The fourth stage is application for a final subdivision map, which contains more details about infrastructure and building construction. The fifth stage is application for a building and grading permit (if grading is going to occur), and the sixth stage is the actual construction of the project.

Geotechnical and geologic reports are required at certain stages of the project, requiring the involvement of a geotechnical engineer and/or a geologist or engineering geologist. Which professional is required to be involved depends on the type of project and on the city ordinances. The typical stages of development requiring geotechnical input are in stage two (EIS), stage three (tentative subdivision map) and stage five (grading or building permit application). Often times if a developer runs out of money and cannot complete construction of homes, they will sell the lots for construction of homes by individual homeowners or other developers. If development proceeds on an individual lot basis, typically a geotechnical report will be required for the lot.

The first place geologic and geotechnical input is in stage two or 3, during the planning phases of the development. A portion of the EIS addresses geologic and seismic safety issues, identifying the hazards and the possible mitigation measures. This analysis is usually made based on the planning department's own knowledge and consultation with staff or hired consultants, as well as a feasibility or preliminary geotechnical or geologic analysis of the site by the developer's engineer or geologist.

A feasibility study includes a general physical description, and sometimes includes the result of preliminary field investigations. Geologic hazards and potential geotechnical difficulties are identified on the site, and a generalized analysis is made of the planned construction. General recommendations may be made for development layout, grading, building foundations, and drainage. It is at this point that the identification of swelling rocks on-site should be made, because their presence can fundamentally affect the design, layout, and cost of the project. The involvement of the geotechnical engineer is crucial in outlining limitations and hazards on a project that has already been conceived and planned in a particular form by a developer. The market-driven developer, constrained by a fixed budget or profit margin, may be unwilling or unable to incorporate changes and recommendations, which may result in higher total cost, or unit cost. Interested parties have used geologic hazard issues in the planning process, such as the presence of faults, to oppose development. Geotechnical engineers may be pressured to downplay the extent of geologic hazards and are sometimes pressured to do so in their reports by developers (author's personal experience). Subsequent geotechnical reports may build or rely on the analysis of prior reports, although advised by manuals of professional practice not to do so (Hunt 1984), and so misunderstandings or miscalculations of geotechnical and geologic hazards occur.

After the fifth stage, an application for grading and/or building permit is typically where geotechnical investigations are required. These investigations usually consist of a brief review of readily available existing information, a few borings and sampling and testing of materials on-site. Selection of the geotechnical engineer is based on a proposal process, in which previous experience and economical budget are the key factors.

6.1.1 Physical Investigation

Geotechnical investigations use a variety of techniques to explore a development site. Numerous manuals have been written describing the techniques and detailing case histories illustrating the successes and pitfalls of site investigation (Das 1989; Hunt 1984). These investigation techniques can be described as office study and field investigation. Office study includes studying existing local area soils reports, United States Geological Survey maps and reports, and aerial photo analysis. Field investigation techniques include mapping, soils borings, test pits or trenches, and geophysical (seismic lines). These techniques are common to all soils investigations, depending on the project being contemplated.

For California residential construction, the most commonly used techniques in geotechnical investigations are review of existing geologic maps and aerial photos and soil borings, occasionally with a few test pits or trenches (when examining a site for seismic activity). The number and spacing of borings depends on existing information (e.g. previous soil investigations), the soil investigation budget, physical features of the site, and the spacing of construction. Selected samples are sent to the laboratory for determination of index properties relevant to grading and foundation design.

Laboratory testing is the most common method of testing the properties of potentially expansive rocks. There are no common in-situ methods used for testing the swelling capacity of rock typically used in residential or commercial development, although there are techniques in heavy surface or underground construction such as the Menard pressuremeter (Silver, Clemence and Stephenson 1976). However, these tests are not designed for surface applications and do not address surface issues, such as long term foundation softening.

6.1.2 Laboratory Testing

The purpose of laboratory testing is to develop engineering parameters for the materials on-site. The most important principle to understand, and often difficult to grasp, is that the laboratory test should reproduce what has occurred and what will occur to the soil/rock in the field. There are several parameters that must be determined first before selecting the type of testing to use as well as interpretation of test results. These parameters have to do with fundamental issues common to all geotechnical problems, disturbed v. undisturbed sampling, remolded v. in-situ samples, and loading conditions.

First, load history and future expected loads, geologically current loading (meaning cut/fill), and future loading from fill and structures. Natural moisture content of the rock at the surface and at depth are also critical, along with an understanding of existing drainage and how planned development will change those patterns. Style and rate of rock weathering must also be considered. If a soil profile is expansive, then the engineer must be suspicious of the rock beneath the soil, even though it appears to be hard competent shale or siltstone. Engineers have constructed foundations on what appears to be hard rock only to come back several years later and found a foundation on soft weathered claystone (personal communication, James Slosson 1997).

A number of different tests designed to indicate swelling behavior of rocks have been developed over time, with the Atterberg limits and free swell the most commonly used nationwide for claystones. Table 16 outlines testing methodologies and values indicating troublesome claystones and can be compared to Table 8, which gives values for some of the claystones encountered by the writer. Inclusion of swell pressure data in geologic maps (Pampeyan 1993) is an indication of awareness in California of the usefulness of this type data. Seldom is one type of test diagnostic. For example, tests on the siltstone beneath the Skyland Ridge residence in the Santa Cruz mountains (Fig. 3, location C) indicated that the siltstone had a low PI. Similarly, some of the UBC expansion pressure tests on the Whiskey Hill formation do not correlate with the swell pressure given. A free swell or other test that used an intact sample of the rock may have been more instructive on the character of the siltstone. In the Orchard Hills case, the waxy claystone tested yielded a PI of 47% and free swell of 90%. A detailed description of the textural difference among claystones onsite as well two different tests for swelling during the soil exploration would have indicated that there were going to be problems on a portion of the site.

Some soil tests are more sensitive than others, and are better able to predict swelling, such as the Expansion Index (ASTM 1994a). All are useful, although some are more obscure that others. Some engineers have their own testing procedures for expansive soils and rocks, based on their experience and application of the test results such as shown in Fig. 43. Following is a summary of the most common tests performed to determine swelling characteristics of soil and rock. Understanding the procedure is the most critical part of soil testing for expansive soils, because only then can the results be properly interpreted.

SAMPLE DESCRIPTION	TEST SURCHARGE PSF	MOISTURE CONDITION	MOISTURE CONTENT %	DRY DENSITY PCF	% EXPANSION
Boring 2	144	Natural	10.5	113	0
Depth 1½ ft.		After Saturation	27.7	104	+9
Gray brown		Air Dry	11.2	109	+4
silty clay (CH)		Oven Dry	0	114	-1
Boring 9	144	Natural	19.2	96	0
Depth 4 ft.		After Saturation	30.5	88	+9
Gray green		Air Dry	17.9	95	+1
claystone		Oven Dry	0	112	-14
	144	Natural	14.6	106	0
Depth 1 ft.		After Saturation	27.1	95	+12
Dark brown		Air Dry	16.0	100	+6
silty clay (CH)		Oven Dry	0	106	0
Boring 14.	144	Natural	20.5	103	0
Depth 4 ft.		After Saturation	26.4	96	+7
Brown silty		Air Dry	14.6	104	-2
clay (CH)		Oven Dry	0	122	-16

DESCRIPTION OF EXPANSION-CONTRACTION TEST PROCEDURE

An undisturbed sample of soil, at its natural moisture content, confined in the 1-inch-high, 2.375-inch-ID cylinder in which it was obtained in the field, is immersed in water while under a surcharge pressure. Measurements of expansion or contraction are taken until movement ceases. The surcharge is removed and the sample air dried, then oven dried. By measuring the dimensions of the sample under these various conditions, it is possible to determine the soil volume under the following conditions: 1) at field moisture content, 2) when completely saturated under the given surcharge, 3) when air dry, and 4) when oven dry. The dry density is computed from the dry weight of the specimen and its volume under the various moisture conditions. The percent expansion, relative to the natural field volume of the sample, is directly related to the various volumes and inversely related to the various dry densities of the sample.

EXPANSION-CONTRACTION TEST DATA

Prepared by MAR Date 9/30

Job Number 1642-A Client Blackhawk

Location Danville, California

Fig. 43. Cooper Clark and Associates used an expansion-contraction test, described above, to gauge the potential volume change of soils (Cooper Clark and Associates 1975).

6.2 Common Methods of Testing for Swelling Soils

6.2.1 Atterberg Limits (ASTM D4318)

Volume change is typically predicted by using this standard test method. Atterberg Limit characterizes the state of soil depending on water content (Lambe and Whitman 1969). Casagrande (1947) demonstrated a key relationship between the soils, their Atterberg limits, and the soils geologic origin. The larger the PI, the greater its plasticity (more water holding capacity) which is indicative of the presence of clay minerals, which can adsorb large amounts of water and in doing so, swell. Casagrande showed that the larger a PI for soil the greater its plasticity, and for soils of similar geologic origin, when plotting the PI against the LL, these soils plot as straight-line trend. It therefore follows that soils and compaction claystones (which are little more than well-indurated soils) with similar PI's would have similar engineering characteristics. Atterberg limits have been recommended for use in characterizing expansive soils and have been used by geotechnical engineers as a parameter indicating special foundation treatment (Chen 1975; Lindner 1976; Meehan and Karp 1994).

However, there are several problems with the use of Atterberg limits as parameters to characterize swelling. It is not a measure of the magnitude of swell likely to occur, and the classification language is vague in its description of swelling hazard associated

Table 10. Comparison of Swell Potential Based on Atterberg Limits.

PI	LL	FHA classification	Chen (1975) (based on LL only)
0-6	0-25	Non-expansive	Low
6-10	25-30	Marginal	Low
10-25	30-50	Moderately expansive	Medium to High
25+	50+	Highly expansive	High
50+	70+	Expansive claystone	>60 Very High

with ranges of LL and PL's. Table 10 is a comparison of several different classifications of swelling potential based on Atterberg limits, the FHA classification of expansive soils, and several others (Chen 1975; Meehan and Karp 1994).

Atterberg limits may not be useful for characterizing swelling rocks because it depends largely on the clay mineral present. The clay content of expansive rocks and the soil generated by them can change over time due to weathering rock. Also, as discussed earlier in section 5, if the swelling mechanism is not related to the water content of the soil, but more related to stress relief (or a combination of the two), then the capacity for a rock to swell may be mischaracterized. Also the presence of a clay mineral known as halloysite can contribute to misleading Atterberg Limits (ASTM 1994a).

However, it is useful as a simple index for comparison of soils or rock, and as a general indicator of types of clay minerals present. Table 11 (Das 1989) shows the relationship between Atterberg Limits and different clay minerals. Plastic limit is a useful and easy test to give an idea of how much water clay will absorb, and therefore how over-consolidated clay will react upon exposure to moisture. There are instances where clay shale can absorb more water than its plastic limit as reported by Peterson (1958). Typical Atterberg limits for claystones and clay soils in the San Francisco Bay area are represented in Table 12 (Meehan and Karp 1994).

Table 11. Atterberg Limits and Clay Minerals.

Clay mineral	LL	PL	PI (theoretical)
Montmorillonite	100-900	50-100	50-850
Nontranite	37-72	19-27	18-53
Illite	60-120	35-60	25-85
Kaolinite	30-110	25-40	5-75
Hydrated halloysite	50-70	47-60	3-23
Dehydrated halloysite	35-55	30-45	5-25
Attapulgite	160-230	100-120	60-130
Chlorite	44-47	36-40	8-12

(from Das 1989)

6.2.2 Soil Suction (ASTM D5298-92)

Soil suction is method pioneered by the University of Texas as a way to measure the swell potential of soils. It measures a soil's affinity for water absorption and retention, and can provide indications of the likelihood of swelling, as well as information about other soil parameters. It combines into one test consideration of natural water content and in-situ stress and is useful in examining expansive rocks where it is difficult to disentangle the contributions of each to swelling.

Table 12. Typical Claystone Values for San Francisco Bay Region

Geologic formation	Location	Expansive deposit	Typical PI
Whiskey Hill (former Butano)	Menlo Park, CA coast range foothills	Claystone beds	50
Recent alluvium	Vista Park tracts, San Jose CA	Black surficial soils ("adobe")	35-40
Orinda	East Bay, Diablo Range foothills	Claystone beds	35-50
Lawlor/Tehama tuff	Antioch & Pittsburg foothills (Los Medanos Hills)	Claystone beds and associated expansive soil	35-40
Panoche/Knoxville	Vallejo (bluffs over looking the Carquinez Strait)	5 to 10 ft thick tuff bed mixed with claystone	47-55

(from Meehan and Karp 1994)

One of the main difficulties with this method of testing is developing a practical understanding of what the test is measuring. Particularly in California where soil suction is not as popular as it is in Texas, there are few engineers that use it, preferring instead the Expansion Index (developed at UC Berkeley). It may be a highly accurate and reliable test, but it is difficult to grasp intuitively, unlike Atterberg limits or the free swell test. However, as the use of post-tensioned slabs becomes more common, this test may become more widely used by geotechnical engineers in California.

6.2.3 UBC Expansion Index (ASTM D4829-88)

The Expansion Index is derived from a Federal Highway Association method used for pavement design. The test is meant to provide an index to the expansion of compacted soils when wetted. This is an index method, and therefore comparable to other indices such as the plasticity index (PI) and the swell index (Cs). In northern

California, this has been the most commonly used test in conjunction with Atterberg limits for predicting magnitude of swell.

Table 13. ASTM Expansion Index Classification

Expansion Index, EI	Potential Expansion
0-20	Very low
21-50	Low
51-90	Medium
91-130	high
>130	Very high

Table 13 (ASTM 1994a) shows the classification of expansive soils based on this index. All conditions are kept constant so as to allow data comparisons between organizations. However, this also means that loading, moisture conditions, in place structure and soil water chemistry are not accounted for. When evaluating results of this test, and what it means to the design and construction of a project, these other four factors must be considered and could be controlling parameters.

6.2.4 Free Swell

Free swell measures the absorption of water by the clay minerals, and the amount of free swell is directly related to the type of clay mineral present. Tourtelot (1974) makes a good comparison of free swell and clay mineral type; the data is from pure clay samples not mixed clay samples. Free swell measures the total volume change of disaggregated claystone or soil, theoretically so that water is available to all the clay particles within the soil. It is a measure of total swelling due to the presence of clay, and therefore is not useful in determining the role of stress relief in swelling.

Krynine and Judd (1957) first adapted the free swell test as a method of determining the swelling capacity of a soil. Free swell has been used to determine swelling capacity of claystones as well (Meehan, Dukes and Shires 1975). It should not be confused with ASTM D4546-90 which also measures "free swell", but under a seating load in a consolidometer.

6.2.5 Swelling Index

The Swell Index (Cs) is determined using the consolidation test, and is typically one-fifth to one-tenth of the Compression index (Cc). It is much smaller in magnitude than Cc and may not be measured closely enough if done only incidentally determine other soil parameters. Typical swell indices and PI's for standard clays and for highly expansive clays and claystones in California are presented in Table 14.

Table 14. Swell Indices and Atterberg Limits

Clay or claystone formation	LL	PI	Cs	Source
Boston blue clay	41	21	.07	Das 1989
Chicago clay	60	40	.07	Das 1989
New Orleans clay	80	55	.05	Das 1989
Montana clay	60	32	.05	Das 1989
Very highly expansive claystone in San Francisco Bay area	70	40	.08	Meehan & Karp 1994
Modelo formation		50	.10 to .12	case study 9
Orinda formation (claystone)		34	.05 to.09	case study 6

Table 15. Comparison of Atterberg Limits, EI and Swell Percent

SOIL TEST	APPROXIMATE RANGES			
Plasticity Index & Clay Content (>#200 sieve)	5-15%	10-25%	20-45%	35%+
	5-15%	10-25%	20-30%	30-45%
Expansive Classification & Weighted Expansion Index	Low	Moderate	High	Very High
	0-20	20-60	60-100	100+
Swell 60 lb. in-situ	0-4%	3-9%	8-12%	12%+
Swell 144 lb. in-situ	0-2%	2-6%	6-10%	10%+
Swell 650 lb. in-situ	0-1%	1-3%	3-5%	5%+

This table illustrates the fact that not all clays with high PI's are necessarily highly expansive. Often environmental conditions control whether the clay or claystone will swell. Swell index is another index property that is comparable between soils and organizations because it is an index property. In southern California, the swell index is the most widely used soils test for expansive soils, commonly used to help determine the thickness of non-expansive fill mats meant to mitigate the swelling problem.

6.2.6 Swell Pressure

Swell pressure is a common test used in rocks mechanics to predict the swelling of clay shales and is now commonly used test in soil investigations for new development in the East Bay. The swell pressure of a rock is the vertical confining pressure in a consolidometer at which 0% swell occurs. This can be very high, up to 18,000 psf for the expansive claystone in case study 1. It is a good gauge of how "hot" or expansive the rock is, and gives a concrete parameter that can be used in project design. In rock mechanics, studies have been conducted of the relationship between swelling pressure and other rock indices such as mineralogy (Madsen 1979), and void ratio (Mesri, Pakbaz and Cepeda-Diaz 1993) and environmental factors such as the moisture activity index (Huang, Aughenbaugh and Rockaway 1986) in an attempt to predict swell pressure based on those indices.

The typical use of this test in forensic investigation is to estimated existing loads to examine how much swell will occur at a specific load, and then compare the observed sheave to that predicted by the test. There may be correlation problems in that predicted swell is taken from a sample that has already undergone weathering, moisture content and possible clay mineral changes, whereas the observed heave is typically the result of swelling of fresh rock.

6.2.7 Other Methods

There are other methods of examining the shrink swell capacity of expansive soils and claystones that have been used by geotechnical engineers and are included in the ASTM Soil and Rock manual, such as ASTM D4546-90, one dimensional swell or settlement potential of cohesive soil. In part this is because the procedure may be outmoded or too complicated. There are more methods invented and used exclusively by engineers who have extensive field experience or good working knowledge of laboratory derived soil properties and soil behavior. Another test, the expansion-contraction test (Cooper Clark and Associates 1975) used in the Blackhawk Ranch development, is an example of an engineer devising a simple and straightforward examination of the soils swelling potential. It develops data about the soil behavior in extreme conditions, where the soil is saturated as well as baked dry. Meehan and Karp (1994) recommend measurement of the summer crack width of adobe soil as a good measure of potential future movement of shallow soils. However, tests developed for soil may not translate well to rock. These tests may under-predict swelling by ignoring the contribution of total stress relief to swell.

While laboratory testing is important, in order for the testing to be useful, it must be considered within the context of the geologic and environmental setting, as well as the construction project planning and design. The data for swell prediction may be available, such as Atherton pool case, but the environmental details were ignored (such as exposing unweathered bedrock), and heaving occurred in measurable and damaging amounts.

6.3 Interpretation of Lab Tests and Characterization of Swelling Hazard

Table 16 summarizes the most common methods used by geotechnical engineers in California for testing of expansive claystones and the typical values obtained from these tests for problematic claystones in California. The Atterberg Limit is the simplest and least expensive way to characterize the expansive nature of a soil or claystone. It is also the most commonly used, and therefore likely represents the largest body of existing expansive rock data. Expansion Index or swell index most commonly used to determine the magnitude of predicted swell. Soil suction provides the most description of the soil's total stress, and therefore possibly accounts for load relief as well as water availability in the environment, but traditionally has not been widely used in California and therefore little comparative data is available. As the use of post-tensioned slabs becomes more widespread, this may change.

Once in the lab, the source of the soil sample and the purpose of its testing becomes detached and unrelated. While the ASTM describes testing methodology, oftentimes adjustments or modifications to the testing procedure are desirable to more closely mimic field conditions. After results are in hand, there is a tendency to forget the circumstances under which the sample was obtained and treated in the lab, or the reason for choosing to conduct the test a certain way. These are all critical considerations, particularly since it is common to make detailed calculations or site-wide extrapolations based on the lab results.

Table 16. Laboratory Testing Procedures/Results Indicative of Expansive Rocks

Test	Procedure/Source	Typical Expansive rock Results
Atterberg Limits, Plasticity Index	ASTM	PI=40+, LL>60
Free Swell	Meehan, Dukes and Shires (1975) adapted from Krynine and Judd (1957)	Typically 100% or more
Swell index, Cs	ASTM 4546-90	Disturbed, Cs=.12 undisturbed, Cs=.10
Swell pressure	ASTM 4545-90	>3,000 psf critical
Expansion Index, EI	ASTM D4829-88	EI>130, typically critical

7.0 Integrating Geology and Engineering

7.1 Legacy of Soil Engineering

Geology emphasizes the understanding of the source material or rocks and the depositional environment as well as force that shaped the rock's position in time and space. Engineering focuses on characterizing the current properties of rock that affect constructed facilities. Laboratory testing of soil and rock samples emphasizes the need to integrate the two, in order to understand the material being tested, why it is being tested, and whether the test results are reasonable. Oftentimes, geologic characteristics do not translate well into engineering. Dr. Karl Terzaghi, considered the father of modern soil mechanics, left a difficult legacy to fulfill. He had a unique understanding of geology and was able to combine that with theoretical soil mechanics in a way that many have since tried to reproduce, but have been unable. Both Peck and Terzaghi considered observation and geologic understanding of a building site the most important information an engineer could develop (Terzaghi and Peck 1948). Unfortunately, what Peck and Terzaghi feared most about the future of soil mechanics has come true, soils engineering is very specialized, driven by the nature of engineering study in universities, changes in the economy and the construction business, and the geotechnical professional's involvement in construction. When reading the case studies of Terzaghi and Peck and their roles in construction projects, and studying the development of their recommendations for approaching a project, clearly their experience was either with large corporate or municipal clients, and project inspection was a daily occurrence by the chief engineer. These were the kind of projects that could support in depth study, they were large complicated and for clients who had money and could not afford to have the project fail. Geotechnical engineering today is not practiced in this manner and has become de-professionalized, with bid for services required, and the advent of marketing.

7.2 Role for Geology in Engineering

Several introductions to geotechnical engineering books give the impression that all geotechnical engineers have a sound background in geology (Attewell and Farmer 1976; Hunt 1986; Spangler and Handy 1982). While this may be true for those geotechnical engineers who have made the effort to train themselves or obtained degrees in both geology and civil engineering, this is NOT true for the profession as a whole.

A good description of the difference between geology and engineering is summed in the preface to Attewell's text *Principles of Engineering Geology* (Attewell and Farmer 1976, p.xi-xii) :

> They (geologists) prefer to approach a problem intuitively, indirectly, and in general qualitative terms, often preferring the problem to the results. Complexities are emphasized and simplifications only hesitatingly accepted and in the case of engineers "(they) are trained to be analytical, to depend on theory, and rely on numerical data, on abstractions of natural conditions...often carried to excess with the tendency to unduly simplify in order to be able to minimally analyze a problem because, due to training and environment, (they) are dominated by their orientation towards results."

Engineering geology is an attempt to bridge this gap. The Association of Engineering Geologists defines it as "the discipline of applying geologic data, techniques, and principles...so that geologic factors affecting the planning, design, construction, operation, and maintenance of engineering structures...are adequately recognized, interpreted, and presented for use in engineering and related practice." (Hoose 1993, p.iii).

Expansive soils and bedrock are considered a geologic hazard. Their existence, and the effects on man-made structures, is directly linked to the geologic past. However, as often practiced in residential developments, cities and counties only require the signature of a registered geotechnical engineer on soil investigations. If there is a special zone marked by the city or county, such as an Alquist Priolo Zone (which identifies active fault traces) an engineering geologist may be required to perform an investigation to ascertain whether intended construction will intersect a fault trace. Otherwise, little to no geologic input is required. This is frequently the flaw that leads to problems with housing developments, a lack of geologic input and little to no understanding by the engineer of the geologic material characteristic or spatial variability.

Previously uncertainty in foundation materials has been handled by conservative design and material property estimates using a combination of the observational approach with prudent physical investigation and judgement, an approach successfully used and championed by Terzaghi and Peck (1948). Construction projects no longer typically include site exploration with lead-time designed to allow a thoughtful approach to the project. On public projects, lawsuits by contractors against developers or public agencies related to contract language and "changed soil conditions" (as it relates to excavation, shoring, etc.) has also increased cost risks associated with public projects. With private projects, the prevailing philosophy is still less money spent on development and construction and more money spent on project appearance and marketing. In order to deal with the reality of reduced investigation budgets, and as computer software and technology become more sophisticated, a statistical approach to dealing with uncertainty has become popular. An example of this approach is a research on relating 100 statistical relationships between various rock parameters in an attempt to predict swelling in mudrocks worldwide (Sarman, Shakoor, and Palmer 1994). However, it is the author's opinion that the observational method, when combined with thoughtful exploration to test hypotheses and gather data on soil parameters identified within a geologic context still works well, when engineers use a multidisciplinary approach.

7.3 Geology Controls Soil and Rock Characteristics

What can geology with its "qualitative" approach bring to geotechnical engineering? Simply put, it can provide a background context within which to evaluate soil exploration and testing results to check for consistency and a sense of the larger engineering and geologic setting. The AEG Professional Handbook provides a checklist (Hoose 1993) of specific descriptions of bedrock, structural features, surficial (unconsolidated) deposits, surface and subsurface hydrogeologic conditions.

More generally, the geologist can help the soils engineer better predict the behavior of soils on-site by determining the type and degree of variability of soils and rock on-site, their depositional original, as well as reconstruct the geologic and loading history of the site (especially critical for clays and clay shales). The geologist provides important observations of the granular and fabric characteristics of fine grained rocks and correlate this with data on mechanical behavior of the soils. Karl Terzaghi wrote the classic paper on the relationship between the geologic origin of a material and its engineering behavior in the 1950s. Terzaghi examines mineralogical, depositional and post-depositional geologic history of sediments and correlates this information with their engineering properties. Key properties discussed that are relevant to expansive rock include the physical constituents of the rock (mineralogy and cation composition), the plasticity index, clay activity, pattern of stratification, grain size and a variety of chemical and moisture content changes that occur after deposition (Terzaghi 1956). Some of the important geologic points to remember when dealing with expansive rocks are as follow:

1). Fundamentally, familiarity with the pattern of stratification of expansive rock affects the ability to apply the knowledge gathered from test pits bore holes and laboratory testing across a site.

2). Many of the expansive rock formations are overconsolidated claystones or shales, and have developed joints and fissures in response to load removal and tectonic uplift. These joints and fissures tend to short circuit the normal path of moisture into the subsurface and can allow water to penetrate deeply into expansive rock. This means that the zone of swell may be much deeper than is typically considered in foundation design.

3). Overconsolidated shales and compaction claystones tend to swell long after laboratory tests predict they will, and greater heave may occur than is predicted by laboratory testing. Terzaghi attributed this to unquantified "secondary time effects."

8.0 Recognition of Expansive Rock Hazard in California

Expansive rock does not appear to be a newly recognized phenomenon, and has been studied and discussed in various professional literature venues, as discussed earlier. Why are geotechnical engineers not universally aware of this problem? Why is the knowledge of expansive rocks limited to a few engineers? Examining the universe that engineers operate in may shed some light on this.

8.1 Economic, Legal and Professional Framework

As with any geologic hazard or engineering safety or use issue, there are economic, legal and professional issues that affect how the problem is viewed and mitigated. How it is described, who it becomes a problem for (what parties are involved), what types of financial institutions are involved, and what law is made through civil litigation all changes the professional engineer's approach to a geotechnical problem.

8.1.1 Economics

The economic factors that have affected and continue to affect the problem of expansive rocks (and expansive soils) are the general economy of a region, and therefore the housing market and the engineering job market, as well as the property insurance market. Much has been written about the damage in dollars due to expansive soils nationwide and, in particular, in California. Damage refers to minor cracking or what could be referred to as cosmetic damage, to irreparable damage to buildings, roads and infrastructure.

In the early 1970s the ASCE Research Council on the Behavior of Expansive Earth Materials estimated damage due to expansive soil nationwide at $2.255 billion per year.

The FHWA estimated that this in fact might be low by a factor of two. A decade later this revised estimate was revised to $7 billion nationwide, and $10.5 million per year in California (Krohn and Slosson 1980). In 1973, the estimates were considered shocking, and the difference in estimates between decades is likely attributable to the lack of recognition or underreporting.

Damage due to expansive rocks in California has not been quantified, as often the problem is localized, misunderstood (damage attributed to another cause) or misidentified (thought to be due to expansive soil). That damage is not limited merely to heave, as discussed in this study, but also slope creep (a lateral support problem) and landslides.

The estimate for damage to housing nationwide is $300 million, and walks, driveways and parking areas $110 million for a total of $410 million per year damage to single family residential areas. This does not include condominiums, which in California make up the lion share of litigation over housing damage, including foundation distress due to expansive soils. This is almost 20% of the total problem, damage to dwelling structures and appurtenances.

Several of the case histories provide anecdotal description of damage in dollar amounts. In case study 2, the house originally was purchased for over a million dollars. When the insurance litigation was over, the insurance company paid to have the house razed, and a new house, with a new foundation, constructed for close to $2.5 million (personal communication, Rex Upp 1997). The demand for damages in case study 4 was $675,000, the total original cost for both the lot and construction of a new custom home.

This is well above the five percent of cost damage estimated in case study 1, Sharon Heights (Meehan, Dukes and Shires 1975).

In the case study 3, the damage was to the pool (loss of use, increased maintenance) and to the pool house. Pools on average cost about $40,000 to $50,000 to install, in this particular case, more because of a complex drainage blanket required by the geotechnical engineer. If the pool house continues to shift, underpinning may be required, which is likely to cost another $40,000 to $75,000. Losses estimated for the case histories are summarized in Table 17. Tables 18 and 19 describe typical costs associated with housing projects due to damage by expansive claystones.

Table 17. Estimated Loss in Case Studies

Case study	Project	Location	Estimated Loss per house	Year
1	Sharon Heights	Menlo Park, CA	$10,000	1975
6	Tennis Villas	Blackhawk, CA	$250,000	1989
5	Seeno Homes	Pittsburg, CA	$125,000	1988
4	Single family home	Woodside, CA	$675,000	1994
2	Single family home	Atherton, CA	$2,500,000	1993
9b	North Ranch	Thousand Oaks, CA	$1,200,000	1994

Table 18. Accelerated Maintenance Cost Items

Item	Caused by:
Increased maintenance costs	Frequent patching, caulking and painting of building exterior is necessary as claystone swells and causes new cracks. Leveling of floors and adjustment of doors.
Decreased life cycle of exterior facilities	Roads must be resurfaced and or pavement replaced approximately every 5 to 7 years.
Structure replacement	Option 1.) Complete foundation replacement. Option 2.) Interior floor supports and post replaced with adjustable foundation components.

Table 19. Typical Maintenance Costs

Item	Unit	Unit Cost
Concrete Flatwork	Sf	$10
Asphaltic concrete: remove and replace	Sf	$6-8
Patch, caulk and paint (every 3-5 years)	Per home	$1,500-2,000
Patching of roof	Lump sum	$500-1,500
Seasonal adjustments to floor level, shim house	Lump sum	$5,000-6,000
Underpinning an average 1,500-2,000 sf house	Lump sum	$100,000-150,000
Complete foundation replacement	Lump sum	$150,000-200,000

Costs based on estimates and actual repairs taken from author's case experience and histories.

8.1.2 Legal Trends in Residential Development and Engineering

In the post W.W.II boom of the 1950s, an explosion in population in California and the corresponding economic boom lead to the introduction of mass housing. Most of the easy building sites in the flat alluvial plains were occupied and with the advent of large construction machinery, housing developers could shape the land to fit the desires of the public, and the standard building practice. Construction moved out into the surrounding hillsides, for space, initially cheap land and good views. Engineering as practiced then was by small weak fragmented firms with no professional education, research support or interdisciplinary connections to assist in dealing with the increasingly complicated sites occupied by housing developments. These small firms were dominated by the market-driven building industry, which was interested in putting up as many houses as fast as possible (Meehan and Karp 1994).

California has gone through at least two of these cycles since the 1950s, with downcycles in the late 1970s, early 1980s, followed by a boom from 1986 to 1988, a crash in 1989 lasting until 1994-1995 and the latest boom ongoing (Perkins 1998). In the author's personal experience, the booms are characterized by short extreme swings in housing prices, which have seen a 10 to 20% escalation in the short space of several months. The market for single family houses leads the trend, with condominiums and multiple unit dwellings lagging behind by about six months to a year.

As the housing market overheats in the urban and suburban areas, developers invest in more rural areas one or two hours drive away, certain those fleeing ridiculous prices in the suburban areas, will pay prices above the average in rural areas because they are more "reasonable". One only has to look at the latest issue of the San Francisco Chronicle Sunday Real Estate section at the map of new home development, and compare prices to see this phenomenon. Busts are equally as volatile, and are peppered by stories of investors (or speculators) caught in the downturn who must declare bankruptcy or sell properties at well below the purchase price.

The result of the construction industry cycles has been a lack of rational design method, which still holds true today (Meehan and Karp 1994). The building industry still moves in boom bust cycles, throwing up houses as fast as possible on the upswing and cutting corners in their haste by not allowing for proper site geotechnical and geologic evaluation, or varied approaches to building foundations and lot development. In tight housing markets, the above holds true as well, and costs are ratcheted down by squeezing as much as possible out engineers for as little as possible.

One instance in particular, related to the author by a fellow graduate student in the early 1990s, illustrates what happens to small geotechnical engineering firms during a downcycle in the construction industry. The firm the fellow student worked for was performing soils investigations for small 20 to 25 house subdivisions in the Santa Clara Valley. The budget for the soil report was $4,000 to $5,000, including boring(s), sampling, lab testing, and a report containing foundation recommendations. Each of these houses, even in a down housing market, could sell for $200, 000 to $250,000. This totals four to five million dollars worth of liability should the homeowners find something wrong and decide to sue the developer and subsequently everyone associated with the project, including the engineer.

As housing prices have gone up in California, where the average price for a home is twice the national average of $120,000, people have come to expect more for their money and have little tolerance for any problems with an expensive house, particularly new houses. New construction also carries with it an implied warranty that protects the homeowner from defective construction, and carries through with the sale of a house to the first owner (Acret 1995).

Issues of insurance have also driven homeowners to seek compensation and/or mitigation from developers. During the particularly wet years of the late 1970s and early 1980s, many hillsides failed, damaging houses in California and bringing a flurry of construction claims against property insurance companies (Meehan and Jelks 1986). From this came the earth movement exclusion clause in insurance policies, which excludes damage from all types of earth movement, including expansive soils and rock.

Damage due to expansive rocks and soils has caused Homeowners Associations to sue housing developers, and all the contractors and professionals involved in the construction of the development. The most common subcontractors sued over soil problems are the soil engineers, grader, landscaper and the foundation construction contractor. Before a lawsuit is filed, there is rarely any thorough investigation into the actual cause of damage; the shotgun approach is the most common approach to litigation over construction defects. All parties involved are sued, with the largest targets being the contractors or professionals who have insurance policies which are viewed as sources of funding for corrective repair.

8.1.2.1 Statue of Limitations and Professional Standards

In the civil justice system there are limited time periods on filing lawsuits and rules governing the number of years a lawsuit may be active before it must settle, go to trial or be dismissed. Numerous books on construction law have been published for engineers that are meant to guide them through the causes of legal action, legal standards for judging their work, and statues of limitations. Table 20 briefly summarizes the key time limitations on various causes of action, summarized from Construction Law (Acret 1995). The important point for engineers is to understand the legal standard applied to them, that reasonable due care was used in the performance of engineering duties that met the standard of care of the profession at the time. Currently attorneys must file a certificate of merit stating that they have consulted an engineer licensed in the same profession and that engineer has criticized the project engineer's performance as below the standard of care.

Table 20. Legal Statues in Construction Law

Time Limit	Significance
1 year	Lawsuit must be filed with in one year of discovering construction defect or damage
2 years	Time limit for breach of oral contract
3 years	Time limit for damage to real property
5 years	Time limit for trial once suit has been filed against contractor or subcontractor, must meet legal standard of negligence.
10 years	Time limit to file suit against developer, must meet legal standard of fraud.

Ralph B. Peck observed in his 1973 address The Direction of Our Profession that it was true that "...our practice falls short of our knowledge." He continued, "I am persuaded that many more failures of foundations or earth structures occur because a potential problem has been overlooked rather than because the problem has been recognized but incorrectly or imprecisely solved." (Peck 1984, p.26). He attributed the discrepancy between knowledge and practice to too much specialization and a lack of appreciation for the "interrelation of various branches of engineering."

8.1.2.2 Building Regulation and Code

In most California cities, there are criteria established and gateways set up within the building departments that trigger requirements for soils investigations and reports. Examples of such criteria and gateways that trigger this requirement are: lot slope exceeding a limit, application for a grading permit, construction in an area known to have foundation problems. In Redwood City, the building department requires a soil investigation if a pier and grade beam foundation are to be used. In Atherton, as a result of the building department's experience with the Orchard Hills subdivision (case study 2), they now require soils reports west of a well-known street, Alameda de Las Pulgas, in an area where most of the problems have occurred.

The existence of a soil report does not necessitate nor guarantee that expansive claystones will recognized as a geologic and geotechnical hazard. All of the developments the author has studied that have damage due to claystone heave have soils reports that recognized the presence of claystones or geologic formations containing claystones. Case study 3 illustrates that even though a soils report recognizes expansive claystones on-site, characterizes the claystone using one of the recommended tests, and suggests remedial measures to prevent or counteract swelling, heaving may still occur. In this case, it is apparent that the soil engineer did not understand the cause of the swelling.

Swelling of rock and resultant foundation movement does not become problematic in the building code until the levelness of the floors exceed a limit ratio taken as the slope of the floor, and height difference over distance. For wood frame construction this is L/240 (Building Research Advisory Board 1968). The construction limit on floor slope is 1 inch over 20 feet. In practice, and the author's experience, out-of-level floors become noticeable to homeowners and cracking begins to threaten the water integrity of a structure at 1.5 to 2 inches over 20 feet.

Given the current economic, legal and professional climate, it behooves every engineer to approach residential development with care. This includes being aware of hazards that threaten property values as well as safety. Geotechnical Engineers must develop the ability to recognize expansive rocks and consider them in project design.

8.2 Field Recognition of Expansive Rocks

The first description of field recognition and testing of expansive rock for inclusion in residential design considerations was by Meehan in 1975 during residential damage investigation and repair. In 1979, a paper was published (O'Rourke, Ridley and McConnell 1979) describing a one year study of ground cracking and movement along the sandstone claystone contact as a part of a seismic study for the Stanford Linear Accelerator. In the mid 1980s, there were workshops held in other parts of the country having to do with the study of known expansive clay and shale in those areas. Until recently, there have been no significant papers that provide guidelines for predevelopment recognition and testing techniques aimed at identifying and characterizing the hazard from expansive rock.

Recently the swelling rock committee of the International Society for Rock Mechanics (ISRM) recently published their recommendations for field identification of expansive rock (ISRM 1994). These field techniques borrow from those used by soils engineers to identify expansive soils, noting cracks in the soil, examining street and concrete flatwork. The author and other engineers have successfully used many of these field techniques in forensic investigations to determine the cause of damage and identify expansive rock. This is logical since the reason the damage has occurred is because expansive rock was not identified as being present on-site in the geotechnical investigation.

Table 21. ISRM Recommended Field Identification Techniques

Field Identification	Simple Field Tests	Confirmation tests
Examination of ground surface for shrinkage cracks	Smear Test	Swell
Examination of cuts/trench walls for shrinkage	Taste Test	Swell pressure
Examination of rock cores	Water Reaction Test	

(from ISRM 1994)

Field identification of phenomenon due to expansive rocks occurs in two steps: observation followed by a simple test to confirm the observations. Confirmation of the field identification can be accomplished by laboratory testing of a sample if samples can be obtained at this phase or by consulting a geologic map to see if the rock formation is known for expansive rock problems.

8.2.1 Field Observation Techniques

The goal of field observations in expansive rock is to observe performance of structures in areas of cut, not fill. This is the most difficult aspect of field observation, determining whether the area observed has been cut or filled. In some areas this is obvious, in other areas where the entire terrain has been obscured, it is not very obvious.

1.) Obtain good pre-development topographic maps or stereo aerial photographs of the area of interest. This will help determine area of likely cut and fill.

2.) Observe adjacent neighborhoods. Drive the surrounding neighborhood in the area of interest. The following lists typical signs of heave due to expansive rock heave:

a.) Humps in the road (Fig. 26). These humps represent discrete beds of expansive rock. Where these humps cross the road, corresponding displacement of the concrete curb on either side of the road. Often these humps can be traced across blocks to other streets, and frequently, damage to houses and garages are observed where they are located on top these beds.

b.) Ponded water in the gutter (Fig. 13). Frequently, flow to the storm drains is disrupted by these humps, and the ponded water additionally defines the limit of these beds when their vertical expression is subtle.

c.) Misaligned roof eaves, gutters and garage doors (Fig. 10). Often frames that present geometric outlines are good reference points for observation of misalignments, such as roof gutters and garage openings and decorative trim. However, garage doors themselves are not very useful, as often times they sag in the middle due to poor installation of excessive spanning with a heavy load.

d.) Shrinkage cracks in soil (Fig. 44). Expansive soils on hillsides and ridgetops usually have formed in place, and therefore it is likely that the rock beneath the soil is expansive. Expansive soils on a ridgetop near case study 9a are shown in Fig. 44.

3.) Take measurements or estimate displacements. Observed humps and associated curb and other structure displacements should be measured or estimated, along with notes on likely depth of cut or fill. This can help later when estimates of structural displacements are made based on laboratory testing. Frequency and width of humps in

Fig. 44. Soil cracks indicative of expansive rock. This picture was taken on a hillside near case study 9a, and the soil was formed in place.

the road should be noted. This information can be used to characterize the rock formation and lot by lot development. The same is true for cracking in soil immediately above rock. Measurement of the width of cracks across a fixed distance can give an estimate of volume change likely to occur seasonally.

4.) Gather anecdotal information. Another field observation technique not often employed by engineers is gathering anecdotal information from homeowners in the immediate or adjacent neighborhood. This is the method the author used to discover damage on lot C in case study 5. While inspecting lot A, the author noticed a hump in the road and a corresponding hump in the sidewalk across the street trailing off in a direction that would place the claystone bed beneath the corner of the house on lot C. A woman sitting in her car in front of the house looked puzzled by the author's bent over inspection of the road, and so the author explained what she was doing and why. As understanding came across the woman's face, she remarked "Oh, is that why she (the house owner across the street) has so much damage in her living room and kitchen." The location of these rooms as described by the woman confirmed the author's suspicion of the presence of an approximately five foot wide claystone bed causing damage in three different blocks of the neighborhood. Despite a lawsuit against the developer, the homeowner on lot C had never known about the litigation and the problems with ground beneath her neighbor's houses.

5.) Consult building department officials. In areas where development is rapidly occurring on a large scale, and there are no neighborhoods or neighbors to speak with, local city or building inspectors can substitute as a rich source of local information about what types of foundations are being used by which developer and why. Because they are observing the construction in many places, the inspectors have the opportunity to observe any problem developers are having as they occur.

6.) Consult a geologist or engineering geologist. At a bare minimum, it is worth at least one day of site reconnaissance work spent in the field with a registered engineering geologist or geologist, preferably experienced either with expansive rock or the geology of the development project area. The best way to assure continued quality and care in a project if expansive rocks are identified onsite is to have a geologist as part of the development team of consultants.

8.3 Geologic Map Shows Expansive Rock Formations

After observing many housing developments damaged by expansive claystones, reviewing existing geologic and special purpose maps, and discussing expansive rocks with geologists, engineering geologists and geotechnical engineers, it is clear that a map depicting known and suspected expansive rock formations might be a useful tool for professions involved in hillside development. Those formations known to contain expansive rocks which have damaged residential development and other structures in California can be found either of the regional geologic sheets published by the California Division of Mines and Geology (CDMG) or California's Geologic map (Jennings 1991). These formations are clustered around the central and southern coastal regions of California because that is where the development is, and therefore the damage. However, these maps should only be used as a starting point for investigating the expansive potential of the rocks within the formations on specific development sites. One engineer remarked that all sedimentary rocks ought to be suspect with regard to expansive potential but a closer study of the geologic column, stratigraphy and formations known to contain expansive rock reveal that some generalizations can be made. In the San Francisco Bay region, expansive rock formations in the coastal range seemingly causing the most problems are Eocene claystone beds. Further inland, expansive rock formations are found in upper Cretaceous, Eocene, and possibly some Miocene formations. Fig. 45 is a portion of the geologic map of California (Jennings 1991), which covers the same area as Fig. 4, and shows the locations of rocks in these age groups. In the greater Los Angeles region, it appears that, based on the case studies, most of the expansive rocks formations near the coast are Miocene age. Fig. 46 is a portion of the California geologic map covering the same area as Fig. 14.

Many sedimentary formations contain claystones or siltstones so fine that they are mistaken for claystones. The key is to understand which claystones are likely to pose a problem for development. For example, in case studies 2 and 4, the claystone in the Santa Clara formation generally does not cause heaving problems, though it may contribute to expansive soils on the Santa Clara Valley floor and adjacent lowlands. However, the claystone of the Whiskey Hill formation, in many places shallowly capped by the Santa Clara formation has claystone beds that are highly expansive. It is critical to be able to distinguish between these two rock formations, as they occur together in many places.

There are several existing specialty maps related to expansive rocks in the U.S. However, the majority of them address swelling soils, are only useful in depicting general

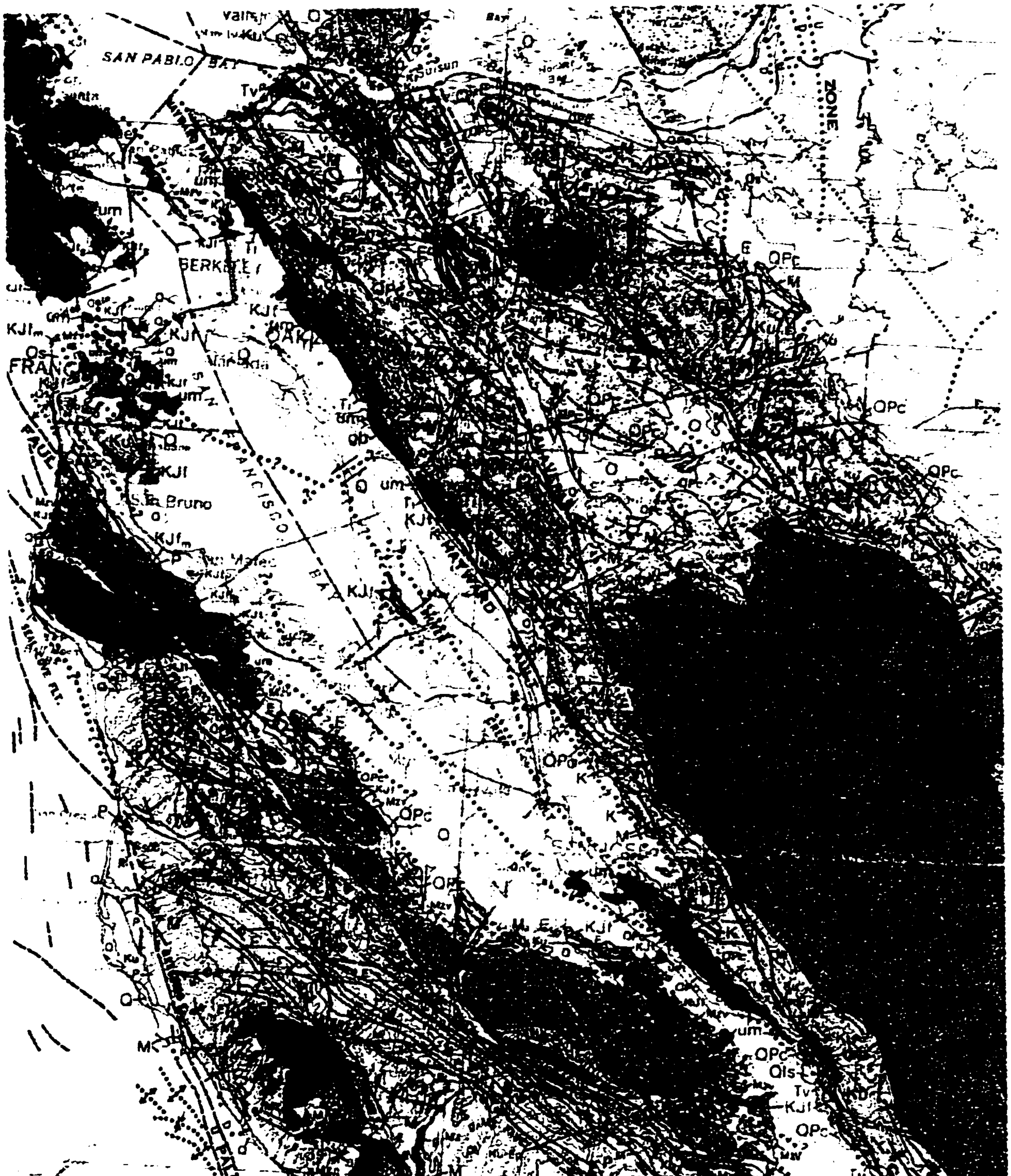


Fig. 45. Geologic map of California, San Francisco region (from Jennings 1991). The geologic formations typically containing expansive rock formations are those marked E (Eocene) and Ku (upper Cretaceous), colored yellow-green and pale green, respectively. The Whiskey Hill formation and Panoche shale are included in these designations.

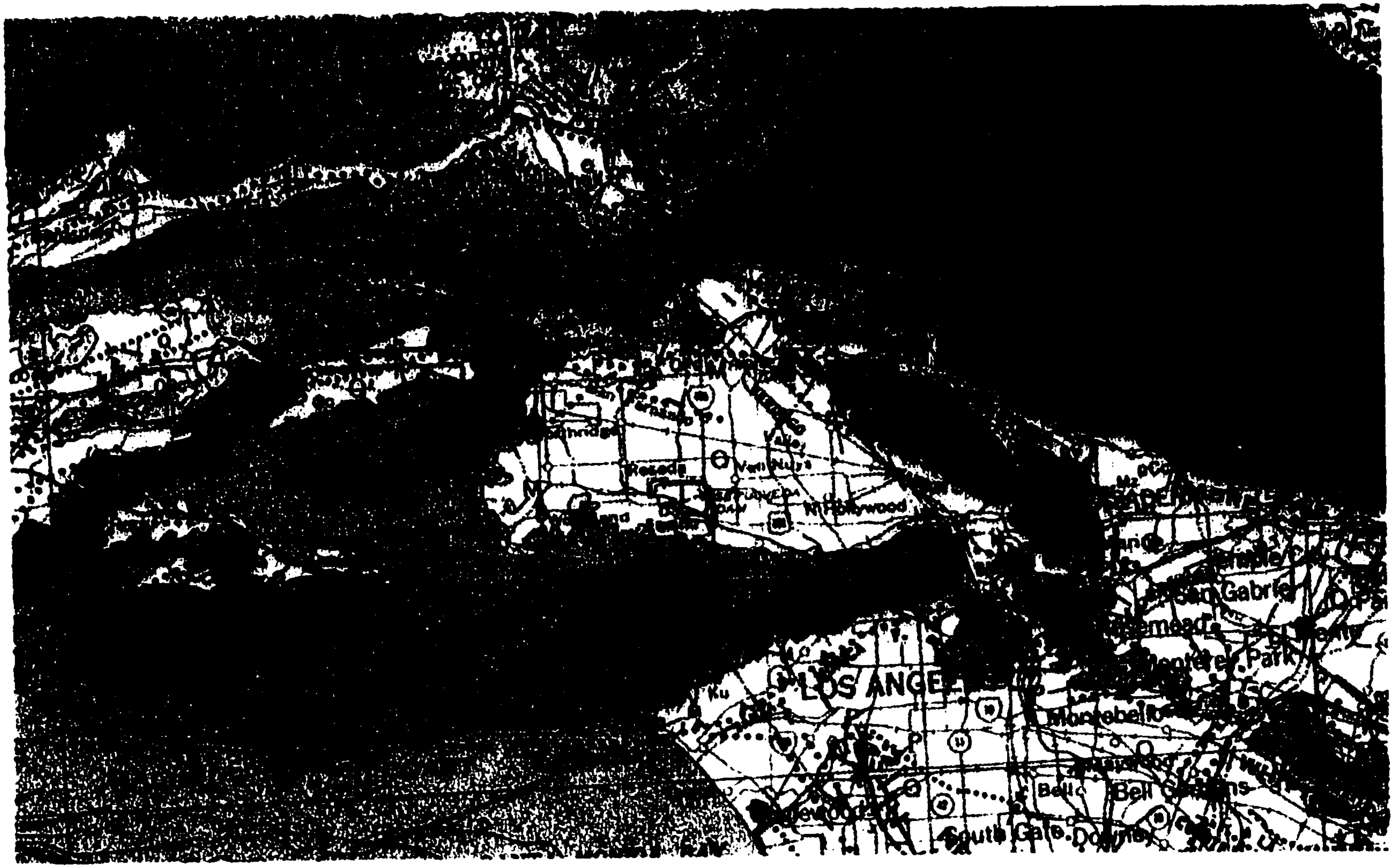


Fig. 46. Geologic map of California, Los Angeles region (from Jennings 1991). The geologic formations typically containing expansive rock formations are those colored pale orange and marked M (Miocene). The Modelo formation and Mint Canyon formation are included in this designation.

areas of swelling soil, but make no recommendations for field observation or testing to confirm the presence or absence of swelling soils. It is impossible to use the maps to determine with any reasonable degree of accuracy whether in fact swelling soil is present on a specific site. A good example of this type of map is the map of swelling soils in the U.S. by Olive et al. (1989).

Although general regional geologic maps can give a good indication of whether an area contains expansive claystones, reliance on these maps is not recommended. As previously discussed (section 8.2), in two of the case studies' geologic maps showed the bedrock to be a younger, non-expansive rock, made up of unconsolidated alluvial sediments (see Fig. 5). However this non-expansive formation, the Santa Clara formation, is a thin veneer over the Whiskey Hill formation, locally infamous for its claystone beds which form humps in roads and defy 30 foot pier foundations' attempts to resist movement. In fact, in case study 2 during litigation over damage to the house, the engineer of record claimed to have relied upon the subdivision geotechnical report, which although describing claystone present on the site, pointed out that geologic maps identified alluvial sediments on the site. A good understanding of the characteristic structure of the geologic formations in the area, the tendency of alluvial gravel and sand to cap underlying massive bedrock in some areas, would have enlightened the engineers and emphasized the need to pay close attention to rock type in the bore holes.

8.4 Characteristics of Expansive Rock Formations

The expansive rock formations encountered in the case studies can be readily identified on the large 1:250,000 geologic sheets for California published by the

California Division of Mines and Geology. Portions of this map are presented here for the use of other professionals as a starting point for checking for the presence of expansive rocks, along with the field techniques discussed in section 8.2.

8.4.1 Checklist of Critical Indices of Claystone Properties

Recognition and identification of expansive claystone conditions can be best accomplished through a combination of geological reasoning and local experience. Typical indicators are either clay-rich Tertiary sediments identifiable from geological maps or characteristic land forms (hills with softened or scarred surfaces arising from soil creep and landsliding) A site location in the foothills or adjacent areas where past experience indicates such problems also provides a good indicator of the presence of expansive rocks. Experienced geologists should be able to locally recognize these soil or rock types at once on the basis of their geomorphology, characteristic gross appearance -- shrinkage cracks in summer, presence of sticky clay in winter -- or signs of instability, including street or curb cracks and misalignments in nearby developed areas built over similar geology.

A summary of the geological profile of a few of the better known problem areas in the San Francisco Bay area is shown on Table 13, taken from Meehan and Karp (1994). Each of these areas has given rise to problems involving several hundred houses built mostly since 1970 and usually with "special" foundations which were intended to mitigate the effects of expansive soils, but which neglected the effect of expansive claystone beds. Two case studies discussed below fall within the Whiskey Hill formation and illustrate the problem geotechnical engineers have with identification and mitigation of claystones.

9.0 Mitigation of Expansive Rock Problems

Two approaches can be used to mitigate the effect of expansive rocks: a design approach and a planning approach. The design approach has been typically used in California, either at the lot development stage or the foundation design stage of development. The planning approach is once advocated first by Ian McHarg in his classic Design with Nature (McHarg 1969), in which development and land use are planned around the natural limitations of the land.

9.1 The Design Approach and Evolution of Foundations

In California, design approaches have evolved over time. Foundation design has evolved from engineers' experience with expansive soils (Fig. 47). Pier and grade beam foundations came into use in many places in the early to 1970s to early 1980s as a way to anchor foundations in the soil zone below the shrink-swell zone. This zone, susceptible to seasonal moisture change, was at one time thought to be several feet deep. Meehan, Dukes and Shires (1975) used this approach in a modified manner by attempting to socket the foundations of a house on the expansive claystone (Whiskey Hill formation) into rock 30 feet below grade using heavily reinforced piers (wire cage and #4 rebar) attached to a grade beam. While this approach limited differential movement to less than two inches (personal communication, Meehan 1997) it did not arrest foundation movement in response to swelling rock, although it reduced an estimated four to five inches of movement to two inches. This is in massively bedded or chaotic claystone

Evolution of Foundations

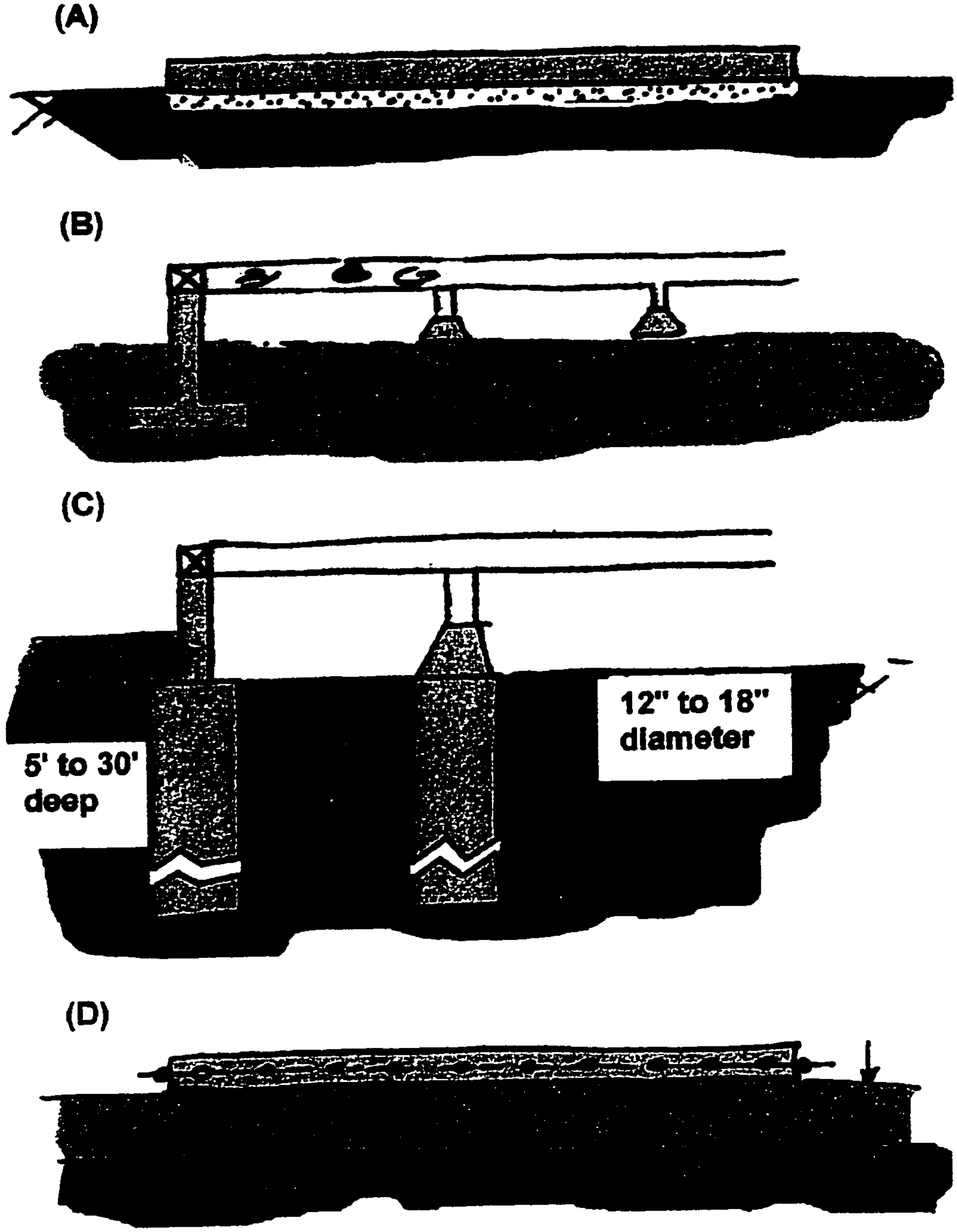


Fig. 47. Evolution of residential foundations: (A) slab-on-grade with a capillary break, common before the 1960s; (B) spread footings with isolated interior footings, common in the 1960s and currently; (C) pier and grade beam, meant to anchor the foundation below an unstable zone and used commonly since the 1970s; (D) post-tensioned slab, designed to act as a unit in an unstable zone and the current foundation of choice in new developments.

melange, as opposed to distinct claystone beds interbedded within a sequence of siltstone and sandstone, such as is found within the Orinda or Panoche formation.

Within relatively thinly bedded sedimentary sequences, which include expansive claystone beds, engineers experienced in expansive rock have used a modified pier and grade beam foundation. Rather than using uniform pier depth, pier depths are modified so that the bottom of the pier is socketed below a minimum depth into non-expansive rock, such as sandstone or siltstone, an approach used for the repair of the house in Orchard Hills case study.

A more recent choice in foundation design is to use post-tensioned slabs so that the foundation will move as a unit and eliminate differential movement within the structure. This foundation is an outgrowth of experience with expansive soils in Texas and is widely used in areas where there are problems with expansive soils and or differential foundation movement due to other causes. The main problem with post-tensioned slabs is variety of house footprints. Post-tensioned slabs work best when the basic shape of the house footprint is square. Problems emerge when a building is long in one dimension relatively speaking, or there are projections from a square footprint. The foundation may dish if not stiff enough or uneven stresses may cause irregular projections in the post-tensioned slab to crack and behave differently than the rest of the slab.

Use of post-tensioned slabs as a panacea to all foundation problems is becoming popular in California, where many new local building codes require the use of post-tensioned slabs, such as the city of Antioch, CA. Developers are using these slabs as a better alternative to cheaper traditional strip footings, but they are not as expensive and are easier to construct than pier and grade beam foundations (Fig. 48).

9.2 The Lot Development Approach

Ecological planning advocates from the 1960s and early 1970s endorsed an approach to development that relied upon a thorough understanding of landforms, land use patterns, natural resources and geologic hazards, the entire "natural system." Karl Terzaghi approached soil engineering and foundation design in a similar manner, advocating thorough understanding of the geology of a site as the best start to engineering design.

The most common method of mitigating expansive rock has been to over-excavate the expansive material on a lot by lot basis and place several feet of non-expansive fill as a building pad over the expansive material. The reasoning behind this approach is to isolate the expansive materials from seasonal moisture changes (the same reasoning behind early use of pier and grade beam foundations) and to provide a uniform non-expansive base for the foundation to rest upon (Berlogar, Long and Associates 1976). However, this ignores one important aspect of swelling rock altogether, the response to load relief. Cuts of 80 to 120 feet are common in hillside grading. Often fresh rock is exposed, with unaltered clay minerals, and moisture content usually less than weathered rock. In Thousand Oaks, case study 9b, three feet of non-expansive fill was placed on a lot with 26 feet of cut, or a load removal of approximately 2,600 psf. The house built on the lot had strip footings, with an average 500 psf load. The percent swell of the material was estimated by the soil engineer at 0.08 to 0.12 or 8 to 12% swell. Given a load deficit

Fig. 48. Construction of post-tensioned slab foundations in a new development in San Ramon, CA.

of 2,100 psf, this could have potentially resulted in 1.6 to 2.4 feet of swell ultimately. A leaking pool and poor site drainage accelerated swell in some parts of the house. In the case of Audifferd Lane, over five inches of differential heave occurred in a short time, despite the fill wedge that capped the claystone on-site. Discussions by the author with engineers working in expansive soils and rock shows that engineers are either using deep heavily reinforced foundations, up to 30 feet deep, or they are using thick non-expansive blanket fills, up to ten feet thick. Strict drainage control measures are recommended and often included in real estate sale documents.

9.3 Recommendations for a Revised Approach to Geotechnical Investigations

What is needed in geotechnical engineering is a return to the roots of soil mechanics as practiced by those who are mainly responsible for its development, notably Terzaghi and Peck. This entails keeping in mind the fundamental rules for practicing good engineering as well as sound geotechnical engineering. Peck listed these as a good understanding of theoretical soil mechanics, lots of experience (on the job and for young engineers, case studies), and an understanding of the importance of geology. While it may not be possible nor realistic in this day and age for young engineers to have a strong grounding in geology, that is no excuse for not taking into account geological factors, or overlooking some important geological aspect of a site.

Following are the writer's recommendations for investigative steps to determine the presence of expansive claystone bedrock, and how to determine its likely behavior in response to grading and moisture changes:

1.) Check the regional USGS map (usually 7.5 or 15 minute quadrangles). Are Tertiary volcanic, upper Cretaceous, Eocene marine, or Miocene formations present? If yes, this is a warning sign that expansive rocks may be present.

2.) Talk to local building officials. Do certain areas tend to have problems with expansive soils? Expansive soils are derived from expansive claystone beds. Are particular areas prone to structural foundation problems? Are there particular areas with "special" building requirements?

3.) Drive the streets and examine them for humps in the road, offset or displaced curbs, severe pavement cracking, and patched areas not related to utility lines. These are indicators of heaving claystone beds.

4.) Consult a local geologist familiar with the area and the rock formations, preferably a geologist with a lot of experience in the region. If drilling bore holes, have the geologist present as well as the soils engineer to examine samples and drill cuttings.

5.) Perform an investigation designed to characterize the rock and determine its behavior. This is where the greatest problem lies with "cook book" geotechnical investigations, indiscriminate sampling and testing. Focus on two things:

a.) Site mapping using geologic maps and test pits and/or trenches. Test pits provide better exposure, intact rock specimens, and a look at the rock in-situ. Borings, if used, should be located according to the geology of the site.

b.) Proper Testing. No matter which of the tests are used to indicate swelling properties, Atterberg Limits, free swells, swell pressure etc., it should be a test the engineer is familiar with and has some experience in interpreting. Testing the right material at the right depth, based on identification using mapping, test pits/trenches and borings, is critical. Loading conditions must be clearly defined and represent future conditions in order for tests to be valid and useful predictors of foundation behavior.

6.) Understand the stress history. The stress history of a rock is important in order to understand the mechanisms by which rocks might swell. It is an over consolidated claystone that might swell when cut several feet? Is the rock fresh or weathered? What type of grading is planned for the site? These factors are important.

10.0 Case Histories Revisited

Based on the cases studies presented, the most common reason for foundation failures in expansive claystones is, the author believes, a fundamental lack of geologic training and experience. This is not new to the geotechnical engineering profession. Both Ralph B. Peck and Karl Terzaghi were fearful that as the training of engineers course of study became crowded with what Peck called the "scientific method", that a good solid founding in geology would fall by the wayside (Peck 1984).

In California, housing construction is where the majority of problems have arisen from expansive claystones. Developers' budgets for geotechnical investigations are slim, and during grading and soils work on-site, typically it is a technician performing density tests on fill that is on-site and responsible for the grading work. Developers expect that there will be problems, and several phenomena have occurred in response to this: the collapsible corporation, designed to hide and protect corporate assets, and more recently the comprehensive development insurance policy, designed to prevent fighting between the developer and the sub-contractors on a project in the event of construction litigation.

There appears to be a fundamental lack of understanding of the mechanisms by which foundation failures occur in expansive rocks; this is coupled with unanticipated environmental effects on expansive rock. Other issues which underpin this lack of knowledge (either through lack of experience or lack of education) are professional issues such as classification of shales, tests to determine characteristics of shales and claystones, confusion due to differences in terminology among the many professions interested in swelling rock, interdisciplinary exchange of information and interdisciplinary cooperation. Investigations into failures due to expansive claystones often fail to identify swelling rock as the failure mechanism. Some of the fundamental problems with these investigations can be generally categorized into two areas, recognition of swelling rock and characterization of swelling rock.

Often, swelling rock has not been recognized as potential hazard in the geotechnical report, although the expansive soil such rock produces is fairly well recognized. It was not common in the 1980s and is not common today in geotechnical investigations to perform geologic mapping of potential development sites at a level of detail that would allow for identification and delineation of areas containing potential swelling rock. Commonly, landslide features and areas of potential landsliding are mapped, along with general rock type and strike and dip of bedding, but nothing more detailed.

In the 1990s, in projects where the potential swelling of rock has been recognized, the characterization of swelling rock itself and its response to changes in the environment are not complete or well understood. Of four reports reviewed in a newly developing area in Antioch, CA, one of the consultants recognized the potential for swelling rock in the Kreyenhagen formation, which contains a thick claystone bed sequence. Commonly, expansive rock information is grouped with expansive soil information, and sometimes not discussed at all, despite the section head in the report that reads "Expansive soil and rock." The common problem areas are: lack of description of claystone (often incomplete), testing of claystone (meaning and proper use/interpretation), and a grasp of the long term nature of swelling mechanisms.

Dissemination of professional information is critical for keeping current on new problems and solutions civil engineering, and occurs through meetings of professional organizations, conferences and workshops and professional publications. While in the civil engineering profession the American Society of Civil Engineers (ASCE) most often serves as a clearinghouse, many of the advances in geotechnical engineering are published in the literature, but there is broad range of specialty engineering publications within the ASCE itself, as well as other professions. Unless there is cross-disciplinary training for engineers, a problem well studied in one field may be not be well studied in another.

Advances in the field are not always effectively communicated back to the engineering students. Several engineers interviewed have made the comment that the geotechnical engineering profession seems to go through learning cycles on engineering problems, with younger engineers rediscovering or relearning what older engineers already have known. Research on problems encountered by county building departments often found that the staff engineers and geologists, unless they knew of expansive soil and rock problems by personal experience, had little knowledge.

Geotechnical engineering has not had a forum for publication until recently. The previous Journal of Geotechnical Engineering had been co-opted by those trying for a broader appeal, afraid that geotechnical engineering was too narrow a specialty. In researching expansive shales and claystones literature, the author used many different terms, databases and searches to ensure no aspects of the problem were missed. Terminology was different for different parts of the country and the world, and in different disciplines. Currently, the ASCE has formed a special Geo-Institute in attempt to gather a broad range of engineering problems in one forum for discussion and information sharing. The ASCE has also recently started publishing Practice Periodicals, meant to record state of the art practices in various fields of civil engineering. These two new forums may prove useful as means of greater information sharing on difficult interdisciplinary topics such as expansive rock.

Very often in California, litigation accompanies the discovery of a new or pervasive geotechnical problem. This hampers research and sharing of information among professionals studying such for several reasons: different professions approach the problem in a different way, legal issues of professional liability, consultant advocacy of a client's position on the cause of failure, differences in training and experience, and personal biases.

11.0 Summary and Conclusions

Expansive claystones are found worldwide and affect a wide variety of civil and private structures. While there may be difference in age, clay minerals and stress histories of these rocks, they can be generally characterized by development of high swelling stresses shortly after excavation and continuing long term development and expression of swelling pressures. The swelling pressures tend to be inhomogeneous and very difficult to predict by the use of laboratory testing data. An inherent problem with the results of laboratory testing is that the nature of sampling these sensitive rocks disturbs them and they swell, dry out and possibly chemically alter before they are tested, or they are altered by the lab tests themselves. The best predictors of swelling are experience with the rock formation in question, or data gathered using at least two of the laboratory methods discussed.

Regardless of how good the prediction of swell may be, it is difficult to mitigate the swelling. This is due to the nature of swelling mechanisms and their interaction. Swelling may be controlled to a tolerable level, but never fully mitigated. This is demonstrated by the long term experience in Sharon Heights in Menlo Park, California where many of the houses on expansive claystones have had their foundations replaced with thirty feet deep heavily reinforced concrete piers. While swelling and subsequent deformation of the houses are much less than previously recorded, the rock is still swelling and measurably deforming the foundations over the twenty years since some of the foundations have been replaced.

In California, expansive claystone beds within various Tertiary rock formations are problematic for foundation and structure designers. In order to successfully tackle a design or development problem in an expansive claystone area, both a geologist and geotechnical engineer should be consulted and work together. Many geotechnical engineers do not recognize the problem of heaving claystones even with seemingly small cuts (less than three feet). A solid understanding of the type and location of claystone on-site, its physical make-up and swell indicators (whether PI, free swell or other test) is crucial for a successful project. There are three critical factors that every engineer should keep in mind:

1.) A good geologic understanding, not just map reading, is the basis for a sound geotechnical investigation.

2.) Proper rock description, identification, testing will refine experience and judgement.

3.) When constructing on or near claystones a fundamentally conservative approach is desirable, particularly when considering changes in soil moisture (i.e. controlling water is unrealistic).

The methodology and investigation priorities outlined above for identifying the likely presence of claystones, and testing engineering indicators of swelling, is applicable not just to California but other parts of the U.S. and the world. In the search for the best way to identify and understand expansive claystones, it is imperative that the practicing geotechnical engineer be able to grasp and utilize whatever methods are described. Used correctly, some basic geologic information, keen field observations and a limited but appropriate lab testing can yield much useful information regarding claystone response to environmental changes.

12.0 References

Acret, J. (1995). *California construction law manual.* Shepard's/McGraw-Hill, Inc., Colorado Springs, CO, 794p.

Allen, H., and A. W. Johnson. (1936). "The results of tests to determine the expansive properties of soils." *Proceedings of the Highway Research Board*, vol. 16.

American Society of Civil Engineers (ASCE) and Association of Engineering Geologists (AEG). *Seminar on Geotechnics of Clay Shale*, March 15, 1984.

Arnold, A. B., and G. L. Hanegan. (1974). "Rebound and foundation settlement at Castaic dam." *Bulletin of the Association of Engineering Geologists*, vol. 11 no. 1, AEG, 15-27.

ASTM. (1994a). *Annual book of ASTM standards, vol. 4.08 soil and rock (I).* American Society for Testing Materials, Philadelphia, PA, 975p.

ASTM. (1994b). *Annual book of ASTM standards, vol. 4.09 soil and rock (II).* American Society for Testing Materials, Philadelphia, PA, 515p.

"At Oahe: contractor licks shale by building backwards." (1959). *Engineering News Record*, vol. 163, July 23, 42-43.

Attewell, P. B., and I. W. Farmer. (1976). *Principles of engineering geology.* Chapman and Hall, London, 1045p.

Audile, H. S. (1996). "Geotechnical nomenclature and classification system for crack patterns in buildings." *Environmental Engineering and Geoscience*, vol. 2 no. 2, 225-248.

Berlogar, Long and Associates. (1976). *Geotechnical Investigation, Sycamore Mill, subdivision 3868*. Unpublished report, 41p.

Brown, K. (1993). *Cycles of Rock and Water at the Pacific Edge*. HarperCollins Publishers, New York, NY, 309p.

Building Research Advisory Board. (1968). "Criteria for selection and design of residential slabs-on-ground." *Publication No. 1571 National Academy of Sciences Report No. 33 to Federal Housing Administration, NTIS No. PB-261 551*, Federal Housing Administration, Washington, D. C.

Burnett, J. L. (1965). *Expansible shale resources of the San Francisco Bay area.* California Division of Mines and Geology (CDMG) Special Report 87, CDMG, Sacramento, CA, 32p.

CDMG. (1951). *Geologic guidebook of the San Francisco Bay counties, Bulletin 154.* CDMG, San Francisco, 392p.

Casagrande, A. (1948). "Classification and identification of soils." *American Society of Civil Engineers Transactions*, vol. 113, ASCE, 901-991.

Casagrande, A. (1949). *Notes on swelling characteristics of clay shales*. Harvard University, Cambridge, MA, 16p.

Chamley, H. (1989). *Clay sedimentology*. Springer-Verlag, New York, 623p.

Chen, F. H. (1975). *Foundations on expansive soils*. Elsevier Press, Amsterdam, 463p.

Cooper Clark and Associates. (1975). *Report, soil and geologic investigation, phase I Blackhawk Ranch development*. Unpublished report, 30p.

Da Costa Nunes, A. J. (1979). "Foundations on expansive rocks in Brazil." *Proceedings of the Fourth Congress of the International Society for Rock Mechanics*, September 2-8, 1979, vol. 1, 99-105.

Das, B. M. (1989). *Principles of geotechnical engineering.* PWS Publishing Company, Boston, n. pag.

Day, R. W. (1994). "Weathering of expansive sedimentary rock due to cycles of wetting and drying." *Bulletin of the Association of Engineering Geologists*, AEG, vol. 31 no. 3, 387-390.

Dibblee, Jr., T. W. (1989). *Geologic map of the Los Angeles quadrangle, Los Angeles county, California.* Dibblee Geological Foundation Map DF-22, Dibblee Geological Foundation, Santa Barbara, CA, scale 1:24000.

Dibblee, Jr., T. W. (1996). *Geologic map of the Mint Canyon quadrangle, Los Angeles county, California.* Dibblee Geological Foundation Map DF-57, Dibblee Geological Foundation, Santa Barbara, CA, scale 1:24000.

Einstein, H. H. (1996). "Tunneling in difficult ground - swelling behavior and identification of swelling rocks." *Journal of Rock Mechanics and Rock Engineering*, vol. 29 no. 3, 113-124.

Finkl, C. W., Jr., ed. (1984). *The encyclopedia of applied geology.* Van Nostrand Reinhold Co., New York, 564p.

Goodman, R. E. (1989). *Introduction to rock mechanics.* 2nd ed. John Wiley & Sons, New York, 562p.

Goodman, R. E. (1993). *Engineering geology: rock engineering in construction.* John Wiley & Sons, New York, 432p.

Grattan-Belew, P. E., and G. C. McRostie. (1982). "Evaluation of heave prevention methods for floors founded on shale in the Ottawa region." *Canadian Geotechnical Journal*, vol. 19, 108-111.

Gysel, M. (1977). "A contribution to the design of a tunnel lining in swelling rock." *Rock Mechanics*, vol. 10, 55-71.

Harper, T., G. Appel, M. W. Pendleton, J. S. Szymanski, and R. K. Taylor. (1979). "Swelling strain development in sedimentary rock in northern New York." *International Journal of Rock Mechanics, Mineralogic Science and Geomechanical Abstracts*, vol. 16, Pergamon Press, London, 271-292.

Higgins, J. D. (1997). "Expansive behavior of shale rock masses: a case study." *Characterization of Weak and Weathered Rocks Masses, AEG Special Publication #9*, P. Santi and A. Shakoor, eds., Association of Engineering Geologists, Sudbury, MA, 107-122.

Hobbs, P. R. N., H. Yeow, S. T. Horseman, and P. D. Jackson. (1982). "Swelling properties of the mudrocks at Harwell." *Radioactive Waste Disposal Series.* Engineering Geologic Unit, Institute of Geological Sciences, Great Britain.

Hollingsworth, R. A. (1990). "Expansive bedrock of the Mint Canyon and Saugus formations." *Geology and Engineering of the Western Soledad Basin, Los Angeles County, California*, southern California section, Association of Engineering Geologists, Los Angeles, 145-153.

Holtz, W. G., and H. T. Gibbs. (1956). "Engineering properties of expansive soils." *American Society of Civil Engineers Transactions*, vol. 121, ASCE, 641-680.

Hoose, S. N., ed. (1993). *Professional practice handbook.* AEG, Sudbury, MA.

Huang, S. L., N. B. Aughenbaugh, and J. D. Rockaway. (1986). "Characterization of swelling potential of shale strata." *Proceedings of the 27th Symposium on Rock Mechanics: Key to Energy Production*, June 23-25, 69-76.

Hunt, R. E. (1984). *Geotechnical engineering investigation manual.* Mcgraw-Hill, New York, 983p.

Hunt, R. E. (1986). *Geotechnical engineering techniques and practice.* Mcgraw-Hill, New York, 729p.

Hurlbut, C. S. (1971). *Dana's manual of mineralogy.* John Wiley & Sons, New York, 579p.

International Society for Rock Mechanics (ISRM). (1994). "Suggested methods for rapid field identification of swelling and slaking rocks." *International of Journal Rock Mechanics, Mineral Science and Geomechanics*, vol. 31 no. 5 Elsevier Science Ltd., Great Britain, 547-550.

Jennings, C. W. (1991). *Geologic map of California, 4th printing.* California Division of Mines and Geology, scale 1:750,000.

Karaborni, S., B. Smit, W. Heidug,, J. Urai, and E. van Ort. (1996). "The swelling of clays: molecular simulations of the hydration of montmorillonite." *Science*, vol. 271, 23 February, 1102-1104.

Kirkaldie, L., ed. (1988). "Rock classification system for engineering purposes, ASTM STP 984." *Symposium on Rock Classification Systems for Engineering Purposes*, ASTM, Cincinnati , OH, 167pp.

Knopf, A. (1929). *The mother lode system of California.* United States Geological Survey Professional Paper PP157, U.S. Govt. Printing Office, 88p.

Kovari, K., C. Amstad, and G. Anagnostou. (1988). "Design/construction methods - tunneling in swelling rocks." *Key Questions in Rock Mechanics*, Cundall et al., eds., Balkema, Rotterdam, 17-32.

Krohn, J.P., and J. E. Slosson. (1980). "Assessment of expansive soils in the United States." *Proceedings of the 4th International Conference on Expansive Soils*, Denver, CO, vol. 1, 596-608.

Krynine, D. P., and W. R. Judd. (1957). *Principles of Engineering Geology and Geotechnics*. McGraw-Hill, New York, 730p.

Lamb, D. P., and S. J. Hanna, eds. (1972). *Proceedings of the workshop Expansive Clays and Shales in Highway Design and Construction, vol. 1 & 2.* Federal Highway Administration, Denver, CO.

Lambe, T. W., and R. V. Whitman. (1969). *Soil mechanics.* John Wiley & Sons, New York, 553p.

Lindner, E. (1976). "Swelling rock: a review." *Proceedings of Conference on Rock Engineering for Foundations and Slopes*, ASCE, New York, 140-181.

Louderback, G. D. (January 16, 1928). Geology of the Pardee Dam site, for East Bay Municipal Utilities District. Unpublished rept., 6p.

Louderback, G. D. (1950). "Faults and engineering geology." *Engineering Geology (Berkey volume)*, Geological Society of America, 125p.

Madsen, F. T. (1979). "Determination of the swelling pressures of claystones and marlstones using mineralogical data." *Proceedings of the Fourth Congress of the International Society for Rock Mechanics, September 2-8, 1979*, vol. 1, 237-243.

McHarg, I. L. (1969). *Design with nature.* The Natural History Press, Garden City, NY, 197p.

Mead, W. J. (1936). "Engineering geology of damsites." *Transactions 2nd International Conference on Large Dams*, International Committee on Large Dams, Paris, 1781-1792.

Meehan, R. L., and L. Jelks. (1986). "The battered exclusion: who pays how much for landslides?" *For the Defense*, May 1986, 19-26.

Meehan, R. L., M. T. Dukes, and P. O. Shires. (1975). "A case history of expansive claystone." *Journal of Geotechnical Engineering Division*, ASCE, vol. 101 no. 9, 933-948.

Mesri, G., M. C. Pakbaz, and A. F. Cepeda-Diaz. (1994). "Meaning, measurement and field application of swelling pressure of clay shales." Geotechnique, vol. 44 no. 1, 129-145.

Moran, D. E. (1989). *Bedrock heave and black shale bibliography.* Douglas E. Moran, Inc., unpublished, 10p.

Nichols, T. C., and W. Z. Savage. (1976). "Rock strain recovery - factor in foundation design." *Rock Engineering for Foundations and Slopes*, ASCE, vol. 1, New York, 34-53.

Noe, D. (1997). "Heaving bedrock Hazards, Mitigation, and Land-Use Policy: Front Range Piedmont, Colorado." *Environmental Geosciences*, vol. 4 no. 2, 48-57.

Olive, W. W., A. F. Chleborad, C. W. Frahme, J. Schlocker, R. R. Schneider, and R. L. Schuster. (1989). *Swelling clay map of the coterminous United States, Miscellaneous Investigation Map series I-1940.* U. S. Geological Survey, Washington, D.C., scale 1:750,000.

O'Rourke, J. E., A. P. Ridley, and J. R. McConnell. (1979). "Instrumentation and field monitoring of ground cracking." *Journal of Tectonophysics*, vol. 52, Elsevier Scientific Ltd., Amsterdam, 505-518.

Pampeyan, E. H. (1993). "Geologic Map of the Palo Alto and part of the Redwood Point 7.5" Quadrangle, San Mateo and Santa Clara Counties, California." *U.S. Geological Survey Miscellaneous Investigation Series Map I-2371*, U.S. Govt. Printing Office, scale 1:24,000.

Peck, R. B. (1984). *Judgment in geotechnical engineering: the professional legacy of Ralph B. Peck.* Dunnicliff, J. and D. U. Deere, eds., John Wiley and Sons, New York, 332p.

Penner, E., W. J. Eden, and J. E. Gillott. (1973). "Floor heave due to biochemical weathering of shale." *Proceedings of Eighth International Conference on Soil Mechanics and Foundation Engineering*, vol. 2 no. 2, session 4, 151-158.

Perkins, B. (1998). "It's official...the boom is on." *San Jose Mercury News*. Real Estate section, November 15, 1998.

Peterson, R. (1958). "Rebound in the Bearpaw shale, western Canada." *Bulletin of the Geological Society of America*, GSA, vol. 69, 1113-1124.

Philbrick, S. S. (1950). "Foundation problems in sedimentary rocks." *Applied Sedimentation*, P.D. Trask, ed., Wiley & Sons, New York, 147-167.

Popenoe, W. P., R. W. Imlay, and M. A. Murphy. (1960). "Correlation of the Cretaceous formations of the Pacific coast (United States and northern Mexico)." Bulletin of the Geological Society of America, GSA, vol. 71, 1491-1540.

Quigley, R. M., and R. W. Vogan. (1970). "Black Shale Heaving at Ottawa, Canada." *Canadian Geotechnical Journal*, vol. 7, 106-112.

Sarman, R., A. Shakoor, and D. F. Palmer. (1994). "A multiple regression approach to predict swelling in mudrocks." *Bulletin of the Association of Engineering Geologists*, AEG, vol. 31 no. 1, 107-121.

Saul, R. B. (1990). "The Mint Canyon formation and associated rocks in the Soledad basin." *Geology and Engineering of the Western Soledad Basin, Los Angeles County, California*, southern California section, Association of Engineering Geologists, Los Angeles, 26-34.

Sedlock, R. L. (1996). "Tectonic framework, origin, and evolution of the San Francisco Bay region." *Recent Geologic Studies of the San Francisco Bay Region*, Sangines, E. M., D. W. Anderson and A. V. Buising, eds., Pacific Section of the Society of Economic Paleontologists and Mineralogists, Sante Fe Springs, CO, 1-18.

Silver, V. A., S. P. Clemence, and R. W. Stephenson. (1976). "Predicting deformations in the Fort Union formation." *Proceedings of Conference on Rock Engineering for Foundations and Slopes*, ASCE, New York, NY, 13-31.

Skempton, A. W. (1953). "The colloidal activity of clays." *Third International Conference of Soil Mechanics and Foundation Engineering,* vol. 1, Switzerland, 57-61.

Smoots, V. A., and G. E. Melickian. (1966). "Los Angeles Dodgers stadium: effect of geological factors on design and construction." *Engineering Geology of Southern California*, Association of Engineering Geologists, Los Angeles section, Glendale, CA, 364-369.

Snethen, D. R., F. C. Townsend, L. D. Johnson, D. M. Patrick, and P. J. Vedros. (1975). *A review of engineering experiences with expansive soil in subgrade.* FHWA Report RD75-48, FHWA, Washington, D. C., 137p.

Spangler, M. G., and R. L. Handy. (1982). *Soil engineering.* Harper & Row Publishers, New York, 819p.

"Structures Don't Settle, but Watch Out for Heave." (1960). *Engineering News Record*, vol. 164, February 4, 46-48.

Taylor, R. K., and T. J. Smith. (1986). "The engineering geology of clay minerals: swelling, shrinking and mudrock breakdown." *Clay Minerals,* Clay Mineral Society of America, vol. 21 no. 3, 235-260.

Terzaghi, K., and R. B. Peck. (1948). *Soil mechanics in engineering practice.* John Wiley and Sons, New York, 566p.

Terzaghi, K. (1950). "Geologic aspects of soft ground tunneling." *Applied Sedimentation*, P.D. Trask, ed., Wiley & Sons, New York, 1193-1209.

Terzaghi, K. (1956). "Influence of geological factors on the engineering properties of sediments." *American Society of Civil Engineers Transactions*, vol. 121, ASCE, 557-618.

Tourtelot, H. A. (1974). "Geologic origin and distribution of swelling clays." *Bulletin of the Association of Engineering Geologists*, AEG vol. 11 no. 4, 259-275.

Transportation Research Board (TRB). (1981). "Shales and swelling soils." *Transportation Research Record, vol. 790*, Washington, D.C., 94p.

Underwood, L. B. (1967). "Classification and identification of shales." *Journal of Soil Mechanics and Foundation Engineering*, ASCE, Vol. 93 no. SM6, 97-116.

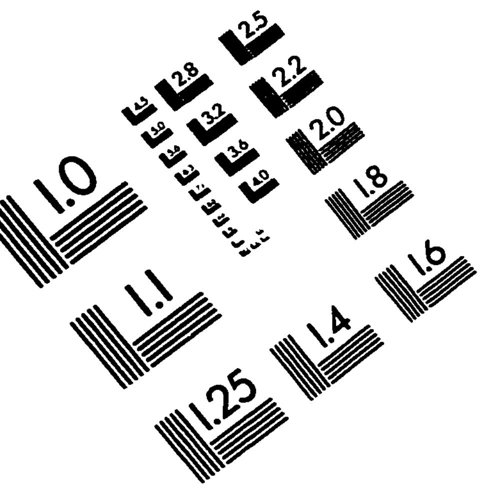
Venter, J. P. (1981). "Free-swell properties of some South African mudrocks." *Proceedings of the International Symposium on Weak Rock*, Tokyo, 21-24 September 1981, 243-252.

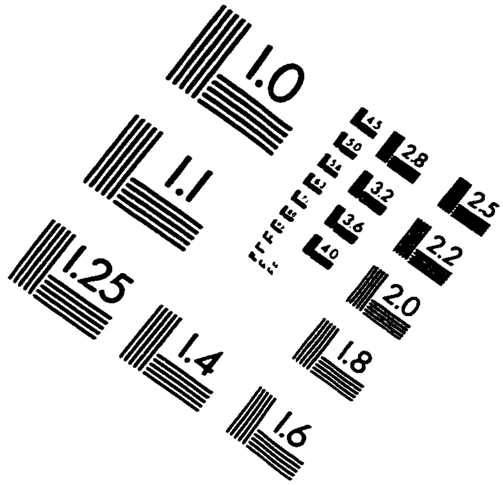
Vine, J. D., and E. B. Tourtelot. (1970). "Geochemical investigations of some black shales and associated rocks." *U.S. Geological Survey Bulletin 1314-A*, United States Geological Survey, Denver, CO. 42p.

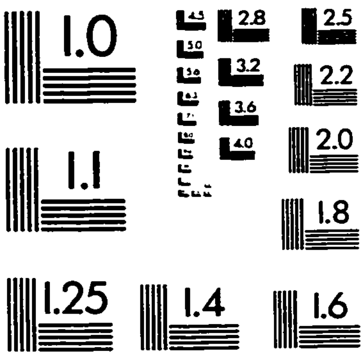
Wood, H. E. (1941). "Nomenclature and correlation of the North American continental Tertiary." *Bulletin of the Geological Society of America*, vol. 52, GSA, Washington, D.C., 1-48.

Woodward-Clyde and Associates. (1967). *A review paper on expansive clay soils.* Portland Cement Association, Los Angeles, CA, 134p.

IMAGE EVALUATION TEST TARGET (QA-3)

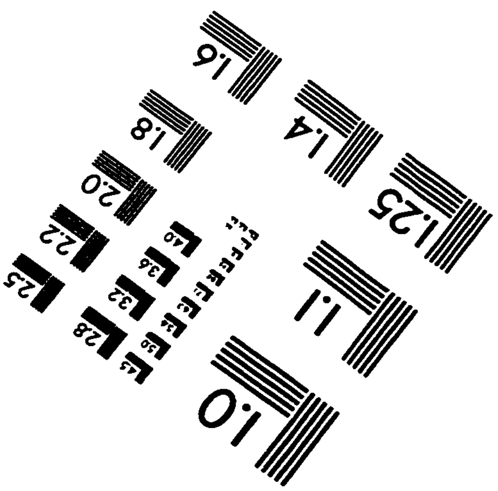


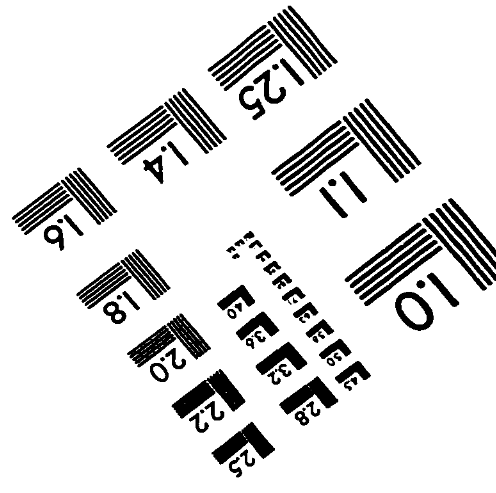




150mm

6"





APPLIED IMAGE . Inc
1653 East Main Street
Rochester, NY 14609 USA
Phone: 716/482-0300
Fax: 716/288-5989