California Housing Damage Related to Expansive Soils

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Abstract: Differential movements of foundations on expansive soils have caused damage to thousands of homes in California. Building damages or just out-of-level floors have resulted in widespread claims and repair expenses. This paper discusses historical and environmental conditions which have led to these problems in California. It describes the current state-of-the-art in foundation design and construction methods used to minimize movements, characterizes out-of-level conditions, and recommends tolerances to assist in evaluating the performance of residential structures. The authors conclude with a summary of considerations and comments on how some of the barriers to upgrading professional engineering practice relative to expansive soils might be overcome.

INTRODUCTION

Expansive soil/rock damage to housing developments has been a problem that has become widely identified in California since the introduction of mass-marketed housing after World War II. Recognition of the problem by designers, developers, and building officials is leading to some mitigation of this problem, although increased development costs coupled with the current (1990s) trend toward "affordability" through construction "economies" may exacerbate it. Recent experience shows that construction of some foundation systems, such as the use of posttensioned slabs-on-ground, originally developed to solve problems, can result in new and costly problems. In any case, the response of the building industry to utilize engineering and construction technology is slow and fragmented.

The writers have provided technical consultation involving problems arising principally from expansive soil damage to a variety of buildings and housing tracts in California. Investigations have varied from brief visual inspections to detailed autopsies involving dismantling of the affected houses. Projects investigated vary from tracts of several hundred moderate cost houses, built between the 1960s and the 1980s in outlying suburbs, to expensive custom homes in the urban areas. The usual case is one where marginally effective foundation designs have led to differential foundation movements ranging from 25–75 or 100 mm (1–3 or 4 in.). Both cause and significance of this level of performance is often disputed.

In the case of a typical single family house that has moved several inches one may hear at one extreme that the damage is insignificant and that the cause is some minor combination of normal wear and tear and normal timber shrinkage, combined with inevitable minor foundation settlement. At the other extreme one may hear (for the same house) that the damage is intolerable and will worsen, that it makes the property "unsalable," or even that the structural safety of the building has been compromised.

Over the past few years there has been a desirable convergence of profes-

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sional engineering opinion on these matters. Accumulated experience now allows for some general guidelines aimed at establishing standards of acceptable performance, as suggested herein.

ENVIRONMENTAL CONDITIONS

Geology of Expansive Claystones

Expansive soils exist in California because of the general geologic activity that is associated with the state's location on the edge of the Pacific Rim. Disturbances of the geologic mantle gave rise to volcanic activity in the central part of California during Tertiary times, some 35,000,000 years ago. Volcanos produced ash that fell to the ground or into water (e.g. tuff) and subsequently weathered to montmorillonite-rich clay. (Much the same process occurred during the recent eruption of Mount St. Helens.) Over time, the result in California has been an accumulation of young sedimentary deposits possessing little secondary cementation or lithification, i.e. so-called compaction claystones, variously interbedded with sandstones and siltstones. Continuing tectonic disturbance of these Tertiary-age beds resulted in their being folded and faulted, forming the hills which range up to a thousand or so feet above sea level along California's coastal regions. The folding process has in some areas produced beds dipping at relatively uniform angles as in the notorious Orinda Formation east of Berkeley or elsewhere mixed in a more chaotic melange as in parts of the Butano Formation on the San Francisco Peninsula.

The more clay-rich members of these formations exist as discrete beds usually several feet thick and seldom visible except where exposed on cliffs or by excavations. When wet, these "rocks" quickly revert to a sticky clay. Swelling occurs when they are exposed to moisture or are unloaded from overburden. They typically give rise to other types of instability, including landsliding, soil creep, or earth flows.

Natural water content of these claystones is typically in the 20–25% range, with plasticity indices (PIs) usually varying between 35 and 50. Volumetric shrinkage proceeds at about 2% for each 1% change in water content, and with exposure to drying, shrinkage proceeds until the shrinkage limit, typically 15%, is reached. This significant shrink potential, present even in rock-like claystones, is itself a major potential source of damage. For example, 15% volumetric shrinkage of only 0.6 m (2 ft) of supporting rock can lead to severe damage to most houses. Engineering attempts to achieve immobility by strengthening foundations may fail because of the large forces and often unpredictable directions involved. Access to moisture produces swell pressures of about 150–500 kPa (3,000–10,000 psf); swell index is 0.03 to 0.08 per log cycle of load.

These troublesome claystones, typically of Tertiary geologic origin, can usually be identified from geologic maps. Other older clayey rocks (mudstones) underline much of the coastal ranges, e.g. the Franciscan Formation, but these older, metamorphosed clays are less plastic and do not give rise to such severe problems as the younger claystones.

Although expansive Tertiary claystones do give rise to serious problems when encountered in hillside land grading (Meehan et al. 1975), the majority of mapped expansive soils are located down slope from expansive claystones. These soils exist as colluvium or, in larger valleys, as alluvial soils selectively deposited in lowland, poorly drained areas of the valley floor. However, the presence of Tertiary claystones in the surrounding hills is a strong indicator of the presence of expansive soils in the valleys below.

Diagnosing Expansive Soils

In the early 1970s, the Federal Housing Administration (FHA) (1973), San Francisco Insuring Office, classified the degree of expansiveness of soils according to their plasticity indices and liquid limits shown in Table 1.

The original FHA chart classified soil groups A, B, C, and D. To this the writers have added a class E, representative of the claystone bedrock found in hillside areas as discussed in the previous section. Swell indices ($C_s/1+e$) for soils D and claystones E are about 0.03 and 0.08, respectively. For cyclic soil moisture changes common in California, houses built on D soils with conventional foundations are likely to suffer 50–75 mm (2–3 in.) differential movements over the years. Conventionally designed shallow foundations usually shift 150–300 mm (6–12 in.) in claystones that fall in the type E classification.

Although the usual practice is to classify expansive soils by their plasticity index (PI), since 1973 the *Uniform Building Code* (*UBC* 1991) has used a comparable expansion index (EI). Correlation between PI and EI parameters are summarized in Table 2.

In the absence of specific swell tests, FHA publications correlate PI to swelling pressures; PIs of 25% are indicated as inducing an uplift pressure on the order of 20,000 kPa (3,000 psf or 22 psi) or more, depending on colloidal activity (ratio of PI to clay size fraction). This agrees with the writers' local experience with expansive soils. However, type E (classified in Table 1) expansive claystones cause uplift pressures of about 479 kPa (10,000 psf) or more, depending on stress history and confinement.

Current Post-Tensioning Institute (PTI) design recommendations for PT slabs-on-ground are not primarily based on PI; the controlling design pa-

TABLE 1. FHA/HUD Classification of Expansive Soils

Soil group (1)	PI (%) (2)	LL (%) (3)	Classification (4)
A	0–6	0–25	Nonexpansive
B	6–10	25–30	Marginal
C	10–25	30–50	Moderately expansive
D	25+	50+	Highly expansive
E	50+	70+	Expansive claystone

TABLE 2. Plasticity and Expansive Indices Correlations

Soil test (1)	Approximate Ranges				
	(2)	(3)	(4)	(5)	(6)
Plasticity index	5–10%	10–15%	12–25%	20–45%	40%+
Clay content (>2 μm)	5–10%	10–15%	15–25%	25–35%	35–60%
Potential expansion (classification)	Very low	Low	Medium/moderate	High	Very high
Expansion index	0–20	21–50	51–90	91–130	130+
Swell 2.8 kPa (60 psf) in situ	0–3%	3–5%	5–10%	10–15%	15%+
Swell 6.9 kPa (144 psf) in situ	0–2%	2–4%	4–7%	7–12%	12%+
Swell 31 kPa (650 psf) in situ	0%	0–1%	1–4%	4–6%	6%+

rameter is percent of clay fraction. For example, the PTI tables entitled "Differential Swell Occurring at the Perimeter of a Slab for a Center Lift Swelling Condition in Predominantly Montmorillonite Clay Soil" show that at a depth to constant suction = 1.5 meters (5 ft); soil suction constant = 3.2; velocity of moisture flow constant = 0.7; and edge penetration distance = 1.5 meters (5 ft), values of 42 mm (1.66 in.) differential swell for 40% clay and 69 mm (2.71 in.) for 50% clay are given (*Design* 1980). Problems with PT slabs on grade in service include the effects of poor design (which are habitually lack of stiffness and sometimes tendon looping, and seldom required strict inspection specifications), common construction defects (such as misplaced tendons and irregular slab thickness), and long-term performance flaws (including loss of prestress and corrosion).

Characteristics of Expansive Soil Fill

Depending on potential for moisture change—whether increase through gradual soaking or decrease by desiccation—the change in volume of fills made from expansive soils can be significant over several years. For example, a common CL-CH clay with a PI of 30, compacted to 90% ASTM D1557 at 25% water content, will exhibit a swell pressure of about 144 kPa (3,000 psf) and will be subject to long-term swell of about 5% in the upper 3–4.5 m (10–15 ft) and compression of 2% or more will occur below 10 m (30 ft).

Volume changes occurring in the upper few feet beneath light loads will tend to be greatest both on account of lesser confinement and the higher probability of varying moisture environments. For example, with shallow foundations the differential movement between a northern shaded side of a house with irrigation and the opposing dry landscaped southern side of the same house can be shown in theory to be roughly 150 mm (6 in.); the writers have observed such movements to be about 75 mm (3 in.) in this common situation.

Hydrology

Postconstruction ground movements depend on cyclical moisture changes that are affected by local climate. The Bay Area climate is seasonal wet-dry Mediterranean with winter rains of 300–380 mm (12–15 in.) and mild dry summers. Pan evaporation is about 1.5 m (60 in.).

The seasonal "active zone" of surficial wetting and drying (with associated soil shrinkage and swell) is usually taken to be 0.9–1.5 m (3–5 ft) deep, though some recent observations suggest that longer-term weather cycles may lead to deeper changes, on the order of 3 m (10 ft) or more.

Troublesome bedrock is usually found in upland areas with deep permanent water tables. Soil suction tests at one such site with abundant claystone showed a general moisture deficiency with soil suction values for sandy claystone bedrock typically of pF = 3.5–4, or 300–1,000 kPa (6,000–20,000 psf). Compacted fill derived from the same materials exhibited values in the same range. This was in an area that had been developed in the 1970s. Damage was continuing even into the 1990s. A decade of construction disturbance and irrigation had not provided enough moisture for full equilibrium swelling. Some 400 houses in this subdivision, including houses on cut and on fill, had soil damage claims. House level surveys showed vertical misalignments in the range of 40–100 mm (1.5–4 in.).

Local variations in soil moisture are induced by irrigation (often excessive), which creates spotty wet areas in landscaping near and beneath foundations. Local desiccation is common in dry sideyards with extra heat reflected from south-facing walls. Given the variations around the perimeter

of a typical house, swell or shrinkage for the most changeable upper 1 m (3 ft) of moderately plastic FHA type "D" clays typically amounts to 40–75 mm (1.5–3 in.).

DAMAGE

The writers' experience is that expansive soil damage to houses in California begins to appear within the first two or three years after construction, gets worse for several years, and then usually settles into a periodic cyclical pattern. It is a complex response to weather and landscape irrigation, building life, and seasonal and longer term weather cycles, which in California may have periods of five to 15 years.

Vertical movements corresponding to vertical swelling and shrinkage of the upper few feet of soil or rock have the most pronounced impact on light structures, though differential wetting and drying at the edge of structures or hillside creep may cause horizontal movements as well. Tree roots searching for moisture often cause lifting of unstiffened slabs, resulting in gross differential movements.

Performance of a floor level survey has become a routine diagnostic procedure for damaged houses in California. The survey may be conducted with an optical leveling instrument or infrared (laser) level; the water level manometer-type device is still often used, although with less accuracy. Several readings are made in each room with relevant correction attempts for carpets and other floor coverings, and a contour map of the floor levels is then prepared. Vertical misalignment is often expressed as maximum slope, most usually over a horizontal distance of 6 m (20 ft), though expressions for both longer and shorter horizontal distances may be pertinent.

If a floor level survey is performed some time after construction, out-of-level conditions in some degree will almost always be found. What is important is to distinguish between cause and effect for the following components: (1) Variances in construction (materials and workmanship); (2) deflections of members (flexural stress); and (3) differential movements (yielding of supports).

Observational and other characteristics of various levels of observed floor misalignment are shown in Table 3, with values reduced in most cases to the amount of misalignment per 6 m (20 ft). A conservative but widely used rule of thumb is that the floor levels are acceptable if equal to or less than 25 mm in 6 m (1 in. in 20 ft) out of level. Floor misalignments of 25–50 mm (1–2 in.) over 6 m (20 ft) distances (L/240 to L/120) usually indicate postconstruction movement with associated damage—cracking of walls and ceilings, sticking doors, etc.—of variable acceptability. Movements of more than 50 mm (2 in.) in 6 m (20 ft) are, in the writers' experience, usually associated with moderate to severe damage for typical residential buildings.

These observations conform with the finding of Day (1990) that for houses on slab-on-grade foundations, wall board panel cracking began at angular distortions of L/300 or 25 mm (1 in.) in 7.6 m (25 ft) and structural damage to wood beams and columns at about L/100 or 25 mm (1 in.) in 2.44 m (8 ft). Day's data, derived from a study of 35 San Diego houses, indicate "slight" damage where maximum differential movement is less than 38 mm (1.5 in.); "severe" where greater than 76 mm (3 in.).

How much movement is acceptable? Geotechnical and structural engineers are often asked to comment on the significance of various magnitudes of expansive soil damage and can offer valuable advice and expert opinion on a case-by-case basis, considering such factors as structural safety, ab-

TABLE 3. Wood Frame Housing Differential Movement Standards/Tolerances/Observations

Year (1)	Project (2)	Location (3)	Standard/observed (4)	Reference (5)	δ per 6.1 m (20 ft) (6)	Remarks (7)
1956	General	U.K.	L/1,000	Skempton and McDonald (1956)	12 mm (1/2 in.)	Gypsum plaster cracking
1961	General	U.S.A.	L/500	Sowers	25 mm (1 in.)	Reinforced concrete
1966	General	Australia	>75–100 mm (3–4 in.) per building	Parry	90 mm (3 1/2 in.)	Serious structural damage but safe
1968	General	U.S.A.	L/200	BRAB (1968)	32 mm (1 1/4 in.)	Allowable deflection ratio for slab-on-grade to limit damage to superstructure
1975	Sharon Heights	Menlo Park, Calif.	150 mm (6 in.) + differentials, claystone C_r = 0.12; at PI = 50	Meehan	76 mm (3 in.)	Serious structural damage but livable
1976	General	U.S.A.	L/500	UC Berkeley College of Engrg	25 mm (1 in.)	Brittle cracking begins
1979	Arcadia Vistapark	San Jose, Calif.	32 mm (1 1/4 in.) to 100 mm (4 in.) per house; grade beams broken at piers	Meehan and Karp	64 mm (2 1/2 in.)	Concrete overpours, cracked wallboard
1979	General	U.S.A.	L/500	Johnson	25 mm (1 in.)	Heaving/cracking of slab-on-grade probable
1980	Tennis Villas	Blackhawk, Calif.	38 mm (1 1/2 in.) to 83 mm (3 1/4 in.) per house; cuts/fills, gr bms and 1.8 m (6 ft) perim piers; 546 kPa (11,400 psf); at PI = 41	Meehan and Karp	57 mm (2 1/4 in.)	Moderate damage; isolated interior piers shifted, kitchens/baths low
1981	Silver Springs	Lafayette, Calif.	75 mm (3 in.) to 100 mm (4 in.) per house; hillside cuts/fills, variable fndn systems	Meehan and Karp	64 mm (2 1/2 in.)	Moderate/severe damage exacerbated by lateral movement
1982	General	California	5 mm (3/16 in.) vertical per 1.22 m (4 ft) horizontal	California State Contractors License Board	~25 mm (1 in.)	Acceptable construction tolerance
1982	Plum Island	Foster City, Calif.	64 mm (2 1/2 in.) per house, rc slab cracked at concentrated roof loads	Karp	50 mm (2 in.)	Light/moderate damage
1983	General	U.S.A.	L/500	NAVFAC-DM7	25 mm (1 in.)	Gypsum wallboard cracking
1984	Discovery Bay	Byron, Calif.	25 mm (1 in.) to 75 mm (3 in.) per waterfront house, partial pier/strip footing fdns	Meehan and Karp	57 mm (2 1/4 in.)	Moderate damage exacerbated by lateral movement
1985	General	U.S.A.	L/240 wood construction	AITC	25 mm (1 in.)	Deflected floor beams—no yielding of supports; intended to minimize plaster cracking
1985	Lighthouse Cove	Redwood City, Calif.	25 mm (1 in.) to 65 mm (2 1/2 in.) per unit; water-front 230 mm (9 in.) concrete slab deflected/tilted	Karp	57 mm (2 1/4 in.)	Light damage; 100 mm (4 in.) tilt anticipated by designers
1985	General	Bakersfield, Calif.	25 mm (1 in.) to 75 mm (3 in.) per house	BSK	57 mm 2 1/4 in.)	25 mm/6 m (1 in./20 ft) = Acceptable 50 mm/6 m (2 in./20 ft) = Distress 75 mm/6 m (3 in./20 ft) = Damage
1986	General	California	44 mm (1.75 in.) per house	Cal Vet Insurance	~25 mm (1 in.)	Excess is abnormal settling
1986	Seeno Homes	Pittsburg, Calif.	32 mm (1 1/4 in.) to 100 mm (4 in.) per house; grade beams w/ 1.2 m (4 ft) perimeter piers, interior piers isolated	Meehan and Karp	76 mm (3 in.)	Mod-severe damage, overpours/ eccentric pier loads, poor framing
1987	Warmington Homes	Antioch, Calif.	25 mm (1 in.) to 50 mm (2 in.) per house; tilt/deflection, marginal PT slab design, poor original construction	Karp	50 mm (2 in.)	Light/moderate damage; expansive claystone, transition lots, bad drainage
1988	Bel Marin Keys	Novato, Calif.	38 mm (1 1/2 in.) to 64 mm (2 1/2 in.) per house w/slight lateral; PT slab w/raised floor, some tilt and deflection	Karp	38 mm (1 1/2 in.)	Slight damage; no slab cracks, eventual tilt to 6 in. suggested by local agency
1989	General	U.S.A.	L/240 for concrete construction	ACI Section 9.5.2	25 mm (1 in.)	Supporting nonstructural elements not likely to be damaged by large deflections (no yielding of supports)
1989	Hanna Ranch	Hercules, Calif.	25 mm (1 in.) to 38 mm (1 1/2 in.) per house; PT slab w/column ftgs-concentrated loads; PI = 40, clay content (finer than 2 μm = 40%	Karp	32 mm (1 1/4 in.)	Slight damage to walls; slab shifted/cracked, poor design/ tendons misplaced
1991	General	U.S.A.	L/240	UBC (1991) (Section 2307 and Table 23-D)	25 mm (1 in.)	Member deflection DL + LL limitation (no yielding of supports)

normality with respect to local experience, prognosis for continued deterioration, and so on.

There are foundation design and performance guidelines that have been adopted for residential developments on reclaimed land (fill over bay mud) in the Bay Area that note acceptable planar out-of-level (nonstructural tilt) performance. In San Mateo County a local ordinance was enacted allowing tilt of 100 mm (4 in.) in 15.2 m (50 ft) or $L/150$. At one large marine-front development in Marin County local building officials requested the project's geotechnical engineers to provide a remedial scheme (before permitting) for leveling any slabs that tilted during occupancy more than 150 mm (6 in.) across the building, at another smaller project the same officials required a scheme to be incorporated into the structure to allow for periodic leveling of the building above a semirigid spread footing foundation. As for structural bending (rigidity) criteria, maximum deflection under sustained (dead) loading of 25 mm (1 in.) in 18 m (60 ft) or about $L/720$ is all that is allowed by a city ordinance in San Mateo County, which may be compared to maximum (1991 UBC §2307/Table 23-D) computed allowable deflection of $L/240$ or 25 mm (1 in.) in 6 m (20 ft) for structural members, specified for dead + live loading. Deflection is defined as being "measured by the maximum vertical deviation from a straight line of Length L connecting any two points on the upper surface of a footing or supported slab."

Sometimes homeowners have been told that their out-of-level floor slab (on grade) conditions are "deflections," which are limited by building codes. Often the "deflection" proves to be tilt, not deflection, or a combination of tilt and deflection. Tilt is an out-of-level condition that does not cause stress in the floor system, and is not addressed in the Uniform Building Code. Deflection is a function of the load and is exponentially a function of the span, and is inversely related to the modulus of elasticity and static moment of inertia of the section. Deflection is proportional to structural stress, but tilt, and out of levelness due to original construction, are not related to stress.

Creep strain in concrete slabs, most of which takes place during the first year after construction and tapers off radically thereafter, and ACI Code "allowable" computed deflection for floor construction attached to nonstructural elements likely to be damaged by large deflections, is $L/480$ or 25.4 mm (1 in.) in 12.2 m (40 ft). The magnitude of deflections in reinforced concrete structures depends on so many influences that no precise calculation is possible; deflections can only be estimated to $\pm 25\%$. Posttensioned (PT) slabs-on-ground present even more constraints on accurate calculation of deflections; variances in construction and postconstruction measurement inaccuracies drastically compound the interpretation of surveys. For PT slabs, the absence of cracking, not the measured levelness of the surface, may often be the best indicator of satisfactory performance.

A common condition that causes alarm is hairline cracking. Civil engineers are aware that all reinforced concrete members crack, generally starting at loads well below service level, and also even prior to loading due to restrained shrinkage. Flexural cracking due to loading is not only inevitable, but actually necessary for the reinforcement to act effectively. Visible cracks, usually from shrinkage and temperature stresses, of width not exceeding about 0.25 mm (0.010 in.), are not significant. A "significant" crack may be one that has a width in excess of about 0.40 mm (0.015 in.). However, a measurement of crack width alone is not enough; the importance of cracks

this size or even larger must be evaluated by qualified engineers considering their location, orientation, and other factors.

ENGINEERING DESIGN CONSIDERATIONS

Site Investigation

Diagnosis of expansive soil conditions should begin with a review of local geological conditions. Specific indicators of expansive soil may be either: (1) Clay-rich Tertiary sediments sometimes identifiable from geological maps or from characteristic land forms (hills with softened or scarred topography arising from soil creep and landsliding); or (2) derivative recent clayey alluvium, including upland creep or flow deposits or lowland alluvium, particularly quiet-water, fine-grained adobes soils deposits in flats, interfluvial basins, or bay margin areas.

A summary of the geological characteristics of representative problem areas in the San Francisco Bay Area is shown in Table 4. Each of these areas has given rise to problems involving hundreds of houses built mostly since 1970 and often with 3–6 ft deep pier foundations which were originally intended to mitigate the effects of expansive soils. Where the potential for or presence of expansive clay minerals is noted from preliminary review of geotechnical data, the investigator should reconnoiter the developed areas of the neighborhood for local evidence of soil movement. This may appear as damage to existing infrastructure or damage to older buildings. Out-of-line, rolling pavements and curb lines are generally good telltales. Exposed expansive soils in undeveloped areas will show shrinkage cracks in summer, presence of sticky clay in winter. Linear shrinkage of dry surficial soils can be easily determined by direct measurement of crack frequency and width. Summer crack width is a good direct measure of future movement potential for shallow foundations. These preliminary investigations will provide a rational basis for specific site investigations including soil borings and laboratory tests. As always, these more detailed investigations are most effective when they are used to refine preliminary concepts based on geological and engineering reconnaissance work. (As is often the case in geotechnical problems, the most effective diagnostic technique is based on a combination of multidisciplinary skills drawing on geology, soil mechanics, and local building design experience. The need for and difficulty of combining reasoning from various domains has been a problem in geotechnical practice for years, as discussed by Terzaghi and his followers.)

There is an extensive body of research literature on the subject of laboratory testing procedures for identifying and predicting behavior of expansive souls. The writers' experience has been that the presence of problem soils is clearly indicated by simple Atterberg Limit testing of disturbed samples. Where more detailed or confirmatory data are needed swell tests (ASTM D4546-85) provide a good simulation of expansive soil behavior under various conditions. ASTM D4546-85 is fundamental and comprehensive and in our opinion should be the definitive Uniform Building Code (UBC) standards test for expansive soil behavior.

Foundation Design

Building foundations in expansive soil must be engineered. The need for proper construction of buildings on expansive soil was identified at least 35 years ago (Holtz and Gibbs 1956) and was mandated by the state of California (State 1965), and is required by the UBC.

TABLE 4. Examples of Expansive Claystones and Soils in San Francisco Bay Area

Geologic formation (1)	Location (2)	Expansive deposit (3)	Epoch (4)	Years (5)	Bedding (6)	Engineering values (7)
Butano	Menlo Park coast range foothills	claystone beds	Oligocene	30,000,000	chaotic sandstone/ clay stone mix or massive well bedded	PI = 50; % finer than 200 = 95
Recent alluvium	Vistapark Tracts, South San Jose (and other subdivisions)	black surficial soils, interfluvial clayey "pond" deposits ("adobe")	Pliocene	10,000,000	deposited along poorly drained area flanking Canoas Creek	PI = 35–40; % finer than 200 = 80–98 >0.002 mm = 50% at PI = 39
Orinda	East Bay Diablo Range foothills	claystone beds interbedded with sandstone and conglomerate	Lower Pliocene	9,000,000	claystones interbedded with sandstone and siltstones	PI = 35–50; % finer than 200 = 95 >0.002 mm = 40% at PI = 39
Lower Tehama Lawlor tuff	Antioch and Pittsburg foothills (Los Medanos Hills)	claystone beds and associated expansive soil; white and grayish pumice	Upper Pliocene	3,000,000	claystones interbedded with sandstone and siltstones (tuffaceous sandstone and volcanics) 300 ft outcrop 8 mi long	PI = 35–40; % finer than 200 = 75–85
Panoche/Knoxville	Vallejo (bluffs)	5–10 ft thick tuff bed; white, chalky, granular, fractured	Paleocene	60,000,000	single bed of tuff in Panoche/Knoxville (tuffaceous claystones)	PI = 47–55; general, PI tuff = 63; % finer than 200 = 92

> Soil Classification-Expansive Soil: Foundations for structures resting on soils with an expansion index greater than 20, as determined by U.B.C. Standard No. 29-2, shall require special design consideration [*Uniform* (1991), section 2904(b)].

Additionally, HUD guidelines [Federal, section G-6 (1973)] have been an important influence in California, calling for

> Special Structural Designs: . . . The seal of a registered structural or civil engineer will be interpreted by the FHA to mean that the designing engineer has familiarized himself with any unusual soil problems that might exist at the site and has submitted a design that will prevent future development of structural defects in the foundations and superstructure attributable to differential movement of the supporting soils.

Since 1965 (in northern California), generic grade beam and pier foundation designs have been developed to mitigate the effects of cyclical near surface shrinking and swelling. In areas of expansive soil the aim has been to attain stable anchorage below the seasonal shrink-swell zone, which has often been taken, in moderately expansive soil, as beginning at a depth of about 0.75–1 m (30–36 in.) and extending to depths of about 1.1–1.5 m (42–60 in.). Pier depths have generally ranged from about 1.2–1.8 m (4–6 ft) in tract developments. Experience over the severe wet-drought cycle of the last decade has shown that these depths are insufficient. In recent years there has been a commendable trend to use deeper and well reinforced concrete piers, typically 3.0–3.7 m (10–12 ft), in highly expansive soils. In the case of expansive claystone beds, custom houses are often supported on deep—more than 6.1 m (20 ft)—heavily reinforced concrete piers. Even so, the large eccentric and differential forces that may follow disturbance of the claystone's previous environment may move or fail these piers.

Interior isolated piers in the middle of bedrooms or living rooms may have dead loads of 45.5 kg (100 lb) or less, far short of what is needed to counteract uplift forces on the piers due to soil expansion. Ideally, piers should only be subject to adhesion forces on the sides of the upper shaft surface. However concrete overpours ("mushrooms") are typical. It is not unusual to find overpours measuring an average of 610 mm (24 in.) in diameter or more. For a 610 mm (24 in.) accidental pier cap on a 305 mm (12 in.) diameter pier, additional contact area will be about 0.22 m^2 (340 sq in.), which will allow uplifting forces of about 33,360 N (7.5 kips) to be applied to the piers, or about 75 times the total dead load on a lightly loaded interior pier. Anchorage, the intended function of a drilled pier, is not achieved, and the resulting movements are one of the most common causes of damage.

The writers' usual recommendations for highly expansive soils is a reinforced concrete drilled cast-in-place straight shaft (pier) and grade beam/foundation wall system. For custom houses (long spans, unusual loading patterns) the system for each building should be designed by an architect or engineer experienced in foundation design. The shafts should derive their support from skin friction between the soil walls and the drilled shaft. Grade beams must be designed to span between all piers and must contain adequate bending reinforcement to provide resistance to factored negative and positive movements; they must contain web reinforcement (ties) to position the horizontal bars in the grade beam as well as to resist shear stresses or at least minimize cracking, especially near the shafts. Because of the potential for the building to be influenced by heave or subsidence due to volumetric change in the soil, piers should generally extend to some minimum depth determined by the project's geotechnical engineer, to achieve anchorage in stable materials, with the depth being confirmed by the engineer during drilling. For moderately expansive alluvial clays pier depths are typically 3 m (10 ft); for expansive bedrocks, especially in cut areas where rebound is likely, greater depths are needed.

Requiring the engineer to be present during drilling not only provides independent verification that the piers are drilled to sufficient depth to maintain stability under any potential moisture condition, but also affords the engineer the opportunity to observe the cuttings to corroborate the site conditions estimated by scattered, small diameter exploratory borings. Ordinarily the geotechnical engineer will provide the designer with design parameters, such as friction values; however, slopes, lateral pressures and available resistance, and anchorage requirements for stability will almost always control over vertical building loadings.

The bottom of pier excavations should be reasonably free of loose cuttings and soil fall-in prior to installing reinforcing steel and placing concrete. It is essential that the contractor be made aware of the subsurface conditions outlined in the soil report and that, before beginning work, construction equipment be engaged that is appropriately sized to perform the expected tasks. Any accumulated water in pier holes should be removed prior to placing reinforcing steel and concrete, or the concrete should be tremied to the bottom of the hole. Care should be taken during concrete placement to avoid "mushrooming" at the top of the pier (below the grade beam) because increased uplift pressure or uneven set will result due to the overpours. The drilling and cleaning of pier holes should be performed under the observation of the geotechnical engineer to confirm that they are excavated (and constructed) in accordance with the recommendations noted in the report and any supplements; a permanent record should be made of conditions noted during drilling.

Spacing of the piers will be determined by the foundation designer; however, piers should not be closer together than 3 pier diameters. The writers recommend that piers be installed inboard of the building corners (instead of at the building corners) maintaining the minimum diagonal across-the-corner distance of 3 pier diameters (the grade beams will span across the piers and join in double cantilever); this method has proven to be effective in eliminating cracking due to unbalanced moments, shears, and problems inherent to constructing piers at corners.

For highly expansive soil locations the writers usually recommend "high capacity" (fewer piers, longer spans) perimeter foundation systems. Where interior piers are required they should be interconnected and constructed as recommended for the perimeter except that grade beam height and position may have to be adjusted for the floor system utilized. The 1991 UBC [section 2908(b)] requires interconnection to resist seismic forces. Ideally, floor systems should span between perimeter members to eliminate crawl space and ventilation interference. Shaft spacing should be on the order of 2.1–2.7 m (7–9 ft), or more.

Practical diameter-to-depth ratios of shafts should generally not exceed 1:10, to allow cleaning of the excavation and positioning of reinforcement. The writers recommend a minimum of four vertical reinforcing bars, all

bent into the grade beam and tied along with and to the beam's upper longitudinal reinforcement; pier bars should bend laterally into the grade beam about 40 bar diameters outward from their respective pier faces. Closed #4 ties, square with 130° end bends, or spirals, should be used to confine and position vertical bars. Soil faces in each shaft should be thoroughly moistened prior to concreting, and all concrete should be mechanically vibrated during placement. Cold joints must be carefully cleaned and approved by the responsible engineer before placing adjacent concrete.

All piers must be tied together with grade beams or tie beams that extend both ways to ensure group action and to minimize differential movements. Grade beams should be sufficiently wide to accommodate the pier bars. Grade beams should be deep enough to provide adequate underfloor clearance and must be designed to span between the piers in accordance with structural requirements, subject to approval by the geotechnical engineer; but grade beams in any case should be reinforced with not less than four continuous longitudinal bars to provide continuity and an ample moment capacity margin, regardless of structural calculations. Web reinforcement of closed #4 (with 130° end bends) bars should be used near the piers to minimize diagonal tension cracking (and to position principal steel) even if not required by concrete shear resistance calculations.

Uplift distress often arises when grade beams are in direct contact with the ground. Forming the bottom of the grade beam at an angle ("knife-edge"), with the outside leg vertical and leaving a void inboard avoids direct contact and provides a barrier to wildlife, which would otherwise take up residence in the crawl space; recent usage of this older method coupled with other newer procedures has proven to be effective. Less expensive methods include concreting the grade beams over some degradable material (such as several inches of hay covered with building felt set into the bottom of the grade beam form), placing a split half of a sonotube (crown up) under the grade beam form, or the use of cardboard spacers such as DesLauriers Econ-o-Void or Savway Carton Forms. Some designers have specified the use of Styrofoam strips under grade beams, apparently believing that it is compressible or biodegradable [it is neither; Styrofoam resists more than 120 kPa (2,500 psf) pressure and if used during construction must be dissolved with solvent after the concrete achieves strength]. The writers' experience is that the use of voids is difficult to control and often not long lasting, and formed knife-edges along are not usually successful due to construction problems and later infilling with soil and water. The writers usually recommend that the grade beams and piers, with particular attention being paid to their connection, be designed to resist uplift forces due to expansive soil. In any event, it is essential that all grade beams be formed and poured neatly, avoiding concrete spillage and overpours. Ground pre-soaking will mitigate heave pressures during service, but in some claystones water penetration may be very slow.

The grade beams should be fully formed (not poured "neat") and the bottom of the grade beams should be set at least 150 mm (6 in.) below any point of the adjacent final grade. Where grade beams are more than 0.91 m (36 in.) deep and/or exposed above grade for any reason they should contain vertical reinforcement in accordance with ACI 318 section 14.3.2 and other requirements for grade beam walls. A minimum lap, for construction of this type of foundation system, of 40 bar diameters must be provided at all splices, intersections, and corners regardless of calculations indicating shorter bond lengths may be adequate.

Slabs-on-Grade (Floating within Pier and Grade Beam Foundation System)

One of the advantages of a pier and grade beam system is that a slab may float between the grade beams without the problem of unpredictable edge support being provided at spread footings. However, slabs-on-ground should ordinarily not be constructed in living areas where expansive soils are present; if such slabs are required to satisfy architectural needs it will be necessary to replace the native expansive soil with cohesionless material to a depth sufficient to ensure moisture equilibrium. The slab must be structurally reinforced, using enough steel (ACI 318 section 10.3.3) to provide a full 75% of the balanced steel ratio.

Foolproof methods to assure proper steel placement, and continuous construction supervision must be specified by the architect or engineer. If radiant heating systems are proposed, full floating slabs cannot be used and any lack of proper design and/or construction will probably be ruinous. Subgrades must be stable, and slabs must be thick enough to allow not only for concrete clearance [minimum 25 mm (1 in.)] between mid-depth reinforcement and the coils, but also sufficient concrete cover [minimum 32 mm (1.25 in.)] over the coils to provide for proper finishing and curing. To demonstrate the additional expense in providing proper design for the use of radiant heating in slabs on expansive soil, a slab containing standard radiant heating coils [17.5 mm outside diameter (OD) (11/16 in.)] and designed for full-factored positive and negative moment capacity [f'_c = 41.44 MPa (3,000 psi); fy = 414 MPa (60 ksi)] will theoretically have to be at least 165 mm (6.5 in.) thick reinforced with #5 bars spaced at about 146 mm (5.75 in.) on-center, both ways.

For garage areas, to avoid or minimize cracking, a slab-on-grade should be constructed over 150–200 mm (6–8 in.) of river run gravel base placed on a carefully prepared subgrade; this cushion provides a capillary break, precludes direct contact between the soil and the concrete, and helps to distribute loads. Geotextile may be rolled over the subgrade before the base is placed. After the base, a suitable membrane should be placed (if moisture migration is to be prevented) followed by a 25–38 mm (1–1.5 in.) lift of moist sand to aid in concrete curing. Care must be taken not to shift or mix the sand during concreting. The slab should float, be at least 127 mm (5 in.) thick, and be reinforced with at least #4 bars spaced at 12 in. on center both ways (welded wire mesh is ineffective and impossible to hold in place during construction). Slabs-on-ground are flexural members and steel reinforcing is required [UBC (1991) sections 2610(f) and 2607(m)]. Bars should be positively supported on small concrete blocks (dobees) or chairs so that the steel uniformly ends up in the middle of the slab, not in the sand. One of the most common construction defects in residential construction is allowing the reinforcing steel to drop and stay at the bottom of the slab.

Site Grading

Fills constructed of expansive soils shrink during dry weather and swell during wet, resulting in alternate shrinking and heaving (especially at slab edges). In addition to inducing differential heaving and settlement of supporting soil (depending on differentials in loading), this also generates stresses in the foundation system that in turn result in movements and distress to the structures supported by them. The shrink/swell behavior of the soils will also cause excessive cracking and deformation in pavements and concrete

flatwork that have been placed on subgrades that have not been suitably prepared.

Adverse soil conditions may be aggravated and compounded by concentrations of water near the building from roof and site drainage and ponding of water in crawl spaces. Surface drainage must avoid ponding of water at foundations. However, in the writers' experience, surface and subsurface drainage provisions do not always solve or correct the expansive soil problem. Capillarity is one of the most significant factors and is almost impossible to control; moisture rise in fine grained soils may be 7.6 m (25 ft) or more (Burmister 1951). Lawn irrigation and shading and exposing of ground surfaces by owner's lot development (or neglect) will usually lead to differential wetting or drying of the ground sufficiently to fully activate soil expansion or shrinkage. Expecting owners to take special preventive action is not a practical strategy.

Foundation Construction

As indicated, the spillage of concrete during construction of foundations for light-framed structures often results in damage to the buildings. The fact that such workmanship is common or that supposedly "unreasonable" efforts are needed to avoid it is not a good excuse. Concrete spillage is a code violation

> Design of Formwork: Forms shall result in a final structure which conforms to the shape, lines, and dimensions of the members as required by the plans and specifications, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape [*Uniform* (1991), section 2606(*a*)].

Additionally, and compounding the problem, posts on piers that support floor girders are often not centered on the piers and/or are not plumb. These construction defects create eccentric loading conditions (uplift in lightly loaded interior areas) that are responsible for erratic, stressful movements in the house. Here again strict adherence to standard specifications will eliminate the problem. The American Concrete Institute (Hurd 1989), sets forth concrete formwork construction criteria shown in Table 5.

Construction Tolerances

Results of postconstruction floor level surveys of slab-on-grade construction are often alarming to homeowners. It is well known in the industry that residential slabs are usually finished with their edges struck level with the perimeter screeds, but with the slabs' central area slightly lower [typically about 19 mm (0.75 in.) over the width of the slab] due to drainage through

TABLE 5. Recommended Tolerances for Building Footings

(1)	(2)
Variances in plan dimensions	−12.7 mm (1/2 in.), +51 mm (2 in.)[a]
Misplacement or eccentricity	2% of the footing width in the direction of misplacement, but not more than 51 mm (2 in.)[a]
Reduction in thickness	−5% of specified thickness

[a]Applies to concrete only, not to reinforcing bars or dowels.

the membrane pierced by stakes used to support temporary interior screeds, aggregate segregation due to use of a jitterbug (which pushes coarse aggregate down to provide fine aggregate and paste, which shrinks and evaporates more, at the surface so it can be troweled into a smooth finish), and the wearing down by finishing (darby, wood float, and steel troweling).

Slab-on-grade level tolerance criteria should be considered in connection with the three general phases of the construction. Each will of course have an effect on the other. Grading of the slab's subgrade should have a tolerance limit of +6.4 mm (+0.25 in.) high construction to −12.7 mm (−0.25 in.) low construction. Thickness of the concrete—for slabs less than 305 mm (12 in.) thick— should not be more than +9.5 mm (+0.37 in.) thicker to not less than −6.4 mm (−0.25 in.) thinner than the specified thickness. Slab surface tolerance, subject to the variations inherent in the finishing process, are indicated in ACI Standard 117-90 as being plus or minus 3.2 mm (±0.37 in.) in 3 m (10 ft).

Unless these variances in original construction, which are nonstress inducing, are considered in any evaluation of performance, they may lead to a false belief that the level differentials measured during occupancy mean in-service differential movements.

Subfloor Construction

Homeowner complaints initially thought to be related to foundation movements may actually be caused by subfloor deflections and movements that are not due to a yielding of supports. Often joists or other repetitive members are sized to economic and design limits using values for expected loading and lumber quality. Frequently a downgrade in wood size, stress grade, or other quality parameter including an increase in moisture content, coupled with extra loading, occurs that creates overspans, springy floors, slopes, and deflections.

Floors that are flush using glulams with joists hung with hangers nailed to the glulams will distort local floor area slopes to more than $L/100$. The joists are green and will shrink as they dry, but the already dry glulams, relatively stable dimensionally, will not shrink. The lowered elevation at the top of each joist will eventually cause significant unevenness in the wood subfloor. The shape of the glulams will eventually become quite pronounced (outlined) in the finished floor.

A frequently overlooked reason for postconstruction differential measurements of floor elevations on hillside houses is cumulative shrinkage of sawn wood members comprising the higher downhill sidewalls. The equilibrium moisture content (EMC) for Douglas fir lumber (typically used in California construction) assumes in-service ranges from 7% to 14%; however, at the time of construction the moisture content will likely be more than twice that amount. In-service ("air-dry") seasoning, the loss of bound water (moisture in the cellulose cell walls) from the fiber saturation point (FSP) (about 30% average), which occurs after the loss of free water within the cells by drying, will result in very significant tangential shrinkage (depth reduction in beams, joists, and rafters). Lesser, but still significant radial shrinkage (height reduction in sills, plates, and decking) and relatively small longitudinal shrinkage (length reduction in studs and cripples) will also occur. For a steep hillside home built using sawn beams and headers to frame view windows and sliding doors to decks on several levels, cumulative shrinkage of the relatively high, below main level downhill sidewall framing has been calculated to be in excess of 50 mm (2 in.), compared to almost

nothing for the uphill below main level framing (tangential shrinkage, worst case). Floor slopes due to differential sidewall shrinkage have been misdiagnosed as differential foundation movements. Skilled designers and builders lessen potentials for such shrinkage effects by using glulams and carpentry techniques.

SUMMARY

Experience over the last 30 years, planning and repairing light-framed buildings, emphasizes the need to consider the following during design and construction:

1. Comprehensive prebuilding design geotechnic investigation of the site
2. Architectural and engineering building design compatible with site features
3. Foundation design appropriate to achieve stable interaction of soils and structure
4. Construction performed with architect's observation and engineer's inspection

CONCLUSION

Design of shallow residential or other light-framed building foundations on expansive soils is an art which often presents more difficulties than design of foundations for heavy loads; in the latter case traditional soil mechanics theory and textbooks provide good guidance. In the case of expansive soils, the ground itself imposes loads (or deformations) on the structure, which may control. Traditional design criteria, such as bearing capacity, are not relevant. These simple facts are recognized by only a few in the building business.

The civil engineering research establishment has not been responsive to the expansive soil problem, in spite of claims over the years that damage costs arising from expansive soils dwarf the damage costs arising from more newsworthy and more heavily researched events such as earthquakes. The expansive soil problem is multidisciplinary and, from the viewpoint of engineering science, technically, relatively trivial in California. Misguided attempts to standardize and economize, superficial soils investigations, and lack of interaction between knowledgeable professionals and builders before and during construction are the usual precursors to expansive foundation problems; the importance of proper implementation of design and engineering inspections and other verification during construction cannot be overemphasized. Improvements in preventive design practices are less a matter of better advanced theory than of information dissemination, development of coherent quality standards, and coordination among practicing professionals and the construction industry.

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