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SEISMIC PERFORMANCE OF HILLSIDE FILLS

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ABSTRACT: Permanent ground deformations in unsaturated, compacted hillside fills under seismic loading conditions are discussed, with emphasis given to fill performance during the 1994 Northridge earthquake. These movements represent a significant yet often unrecognized hazard to developed hillside areas, as relatively modest deformations induced widespread damage totaling hundreds of millions of dollars during the Northridge event. The development of grading standards in the Los Angeles area is reviewed to place the seismic fill deformation problem in context with other issues that have shaped design and construction practices for hillside fills. Field observational data on fill performance during the Northridge earthquake is presented, and typical ground distress patterns are found to include cracking near cut/fill contacts, lateral extension and settlement of fill pads, and bulging of fill slope faces. For most sites, the prevalent mechanism of permanent ground deformation responsible for the fill movements is contractive volumetric strain accumulation within the unsaturated fill soils during strong earthquake shaking (that is, seismic compression).

INTRODUCTION

Developments in seismic design and analysis procedures for earth structures have historically been motivated by concerns about the performance and stability of such critical facilities as earth dams and solid-waste landfills. This is to be expected, given the dire consequences associated with failures of such structures. This paper is concerned with compacted fills in developed hillside areas, a class of earth structures whose seismic performance has historically received relatively little attention, yet which are pervasive throughout urban centers in California and elsewhere. These fills are constructed to create level building pads, with geometric configurations similar to the wedge and canyon fills illustrated in Fig. 1.

In California, the seismic performance of these earth structures is beginning to be recognized as a critical design issue. Such concerns are derived primarily from substantial economic losses to dwellings, pipelines, and other engineered improvements that can be traced to ground deformations in fill induced by the 1994 Northridge earthquake. Such deformations did not typically damage structures to the extent that life safety was threatened. However, the economic losses (mostly borne by insurance carriers) were large as result of homeowner expectations that damaged houses be returned to their pre-earthquake condition. The repair costs associated with such work typically totaled $50,000 to $100,000 per site, but often rose to full replacement value.

The principal objectives of this paper are to call attention to the problem of seismically induced ground deformations in compacted fill soils and to identify the principal ground failure mechanisms responsible for such movements. The paper begins by providing a brief overview of grading standards in the Los Angeles area and a discussion of the degree to which these standards address seismic performance issues. The significance and nature of the problem are documented using field performance data, principally from the 1994 Northridge earthquake. Using existing analytical methodologies, mechanisms are identified that appear to explain the magnitude and pattern of deformations at most sites. This paper is modified from an ASCE geotechnical special publication paper (Stewart et al. 1995) with significant new field data and entirely new analyses.

SOUTHERN CALIFORNIA GRADING STANDARDS

The evolution of U.S. grading standards has historically been closely linked to the state of practice in Los Angeles, where many hillside areas have been developed since World War II. A number of static stability and settlement problems have occurred in these developments, prompting the City of Los Angeles to take a leading role in the drafting and enforcement of grading codes.

Before World War II, hillside development in Los Angeles occurred on a lot-by-lot basis, with most fills limited in size and consisting of poorly compacted soils placed with little or no site preparation. A postwar boom in hillside construction often involved mass grading for large housing tracts, and although the capacity of earth-moving equipment had improved, most fills were still of relatively low quality as no grading codes were in effect. Landslide and erosion damage to these fills during heavy rains in 1951–1952 prompted the City of Los Angeles to adopt the first grading code for a U.S. municipality that year. The purpose of this and subsequent versions of the code has been to provide for life safety and to reduce the potential for major economic (property) losses.

The early code required compaction testing of fill soils; maximum fill slope angles of 1.5H:1V (horizontal to vertical); bottom inspection of areas to receive fill (typically to observe key and bench excavations); and installation of subdrains at the base of fills. The effectiveness of this early code was limited as the code standards were not always complied with, no routine supervision of construction was required, and professional assessments of geologic hazards (such as landslides) were not required unless specifically asked for by inspectors. Changes to the grading codes were made in response to storm damages in 1956 and 1962, and in 1963 the code was amended to require a maximum 2H:1V fill slope angle, geologic and engineering reports addressing slope stability, and routine supervision of construction by engineering geologists and geotechnical engineers (Scullin 1983; J. Cobarubius, personal communication, 1995). In addition, the modified Proctor compaction standard (ASTM D 1557) was adopted in 1964. These codes have proven effective in controlling the storm damage they were designed to mitigate; in a major 1969 storm, damage on 37,000 lots developed before 1963 totaled $6 million, while

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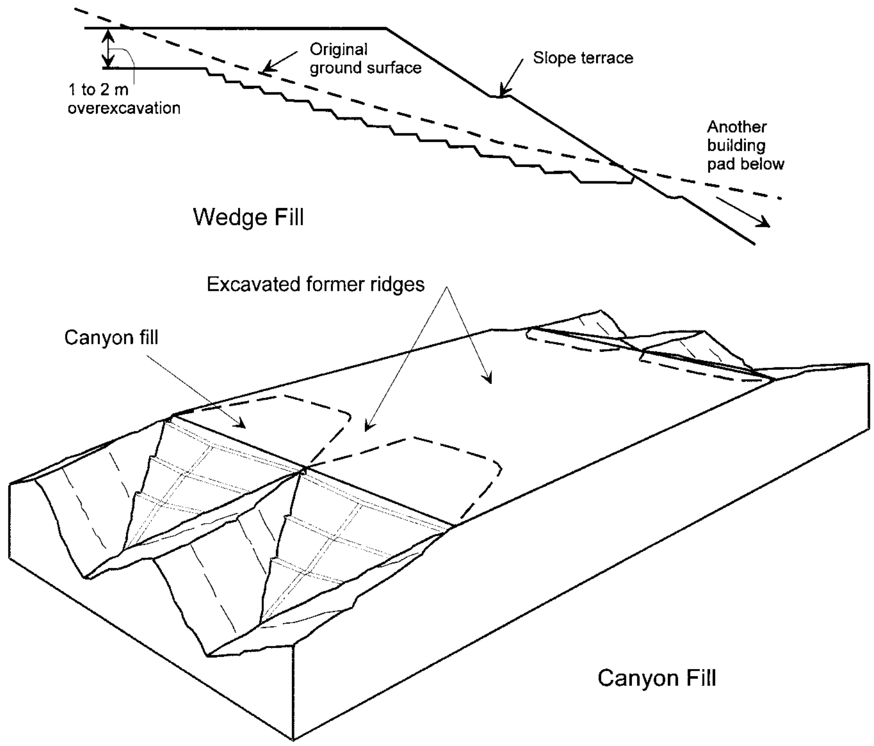

FIG. 1. Geometric Configuration of Wedge and Canyon Fills

damage at 11,000 post-1963 lots totaled only $182,000 (Scullin 1983). Standards of practice have continued to evolve since 1963, largely to address long-term settlements resulting from consolidation or wetting-induced soil collapse. However, in southern California such standards have not as yet been adopted into formal grading codes.

There were essentially no changes to grading provisions by the City of Los Angeles in response to the 1971 San Fernando earthquake, and the city does not currently require seismic stability analyses for hillside fills. Los Angeles County, however, initiated requirements in the mid-1970s that slopes be designed for seismic stability using a pseudostatic analytical approach with a safety factor of 1.1 and a horizontal seismic coefficient of 0.15 (City of Los Angeles 1995). Both the city and county will soon be requiring that seismic landslide hazards be formally assessed in potentially susceptible hillside areas using a displacement-based approach. This change in policy is being made to achieve conformance with the seismic slope stability provisions in the 1990 California Seismic Hazards Mapping Act. The new requirements are setting ''acceptable'' displacement levels for slopes based on serviceability considerations, with the principal intent being to minimize economic losses. This represents a significant extension of the philosophy underlying existing grading codes, which were directed principally toward preserving life safety and preventing major catastrophic property losses during storms.

With this new philosophy, however, comes a need for analysis procedures that can adequately model seismic slope performance at small-to-moderate displacement levels. Most existing pseudostatic and seismic displacement analysis procedures were calibrated to predict ''failure'' conditions under which large slope displacements (a meter or more) could occur. Such techniques may be unreliable for hillside fills where displacements of several centimeters often constitute unacceptable performance. Moreover, as shown in this paper, the source of hillside fill movements can include permanent volumetric strain in addition to permanent shear strains. Accordingly, to analyze fills properly, new analysis procedures will be necessary that are calibrated at small-to-moderate displacement levels and that consider both shear and volumetric strain accumulation.

HISTORIC OBSERVATIONS OF SEISMICALLY INDUCED FILL MOVEMENT

Few studies have focused specifically on the seismic performance of hillside fills or have attempted to document their performance on a broad scale, although the occurrence of ground deformations in fill has been noted following a number of earthquakes. Lawson (1908), in summarizing observations of ground cracking in hillside areas from the 1906 San Francisco earthquake, noted that ''roadways and artificial embankments were particularly susceptible to . . . cracks.'' In summarizing observations from the 1952 Kern County, 1960 Chilean, and 1957 Hebgen Lake earthquakes, Seed (1967) noted ''the effect of earthquakes on banks of well-compacted fill constructed on firm foundations in which no significant increases in pore water pressure develop during the earthquake is characteristically a slumping of the fill varying from a fraction of an inch to several feet.'' In the 1971 San Fernando earthquake, a 12 m thick fill at the Jensen Filtration Plant composed of unsaturated clayey sands and underlain by bedrock underwent 10 to 15 cm settlements that significantly damaged a building constructed on spread footings (Pyke et al. 1975).

A systematic survey of distress to single-family dwellings from the San Fernando earthquake (McClure 1973) noted the influence of fills on damage patterns, particularly when residences were constructed over cut/fill contacts. Specifically, this study found that ''ground failure occurred on a higher percentage of sites that were on fill or cut and fill than on those sites which were on cut or natural grade,'' and ''dwellings on cut and fill or fill had more relative damage than dwellings on cut or natural grade.'' In a separate report documenting earthquake effects in residential areas, Slosson (1975) noted that post-1963 fills (that is, fills constructed to post-1963 grading standards) performed markedly better than pre-1963 fills. Incidents of hillside fill movements during the 1989 Loma Prieta earthquake have been reported by several consultants; however, this information has not been compiled, and relatively little published information is available.

OBSERVATIONS FROM 1994 NORTHRIDGE EARTHQUAKE

The locations of about 250 sites where fill movements caused significant damage are shown in Fig. 2. The most concentrated damage occurred on the north flank of the Santa Monica Mountains (for example, Sherman Oaks, Encino, Tarzana, and Woodland Hills), along the north rim of the San Fernando Valley (Porter Ranch and Granada Hills), and in the Santa Clarita Valley area. Other affected areas included the south flank of the Santa Monica Mountains and portions of Simi Valley. Geotechnical conditions and damage statistics for 85 well-documented sites are presented in the Appendix. These data were typically generated by consulting engineers or geologists in response to insurance claims. As such, the data provide a somewhat biased sample by which to assess fill performance (that is, sites for which no claims were made are not included). Moreover, the data from most sites consist of general descriptions of distress and relative movements across improvements (such as houses), but typically lack a sound basis for assessments of absolute movements (that is, there is usually no ''fixed'' reference frame against which to measure total movements). Nonetheless, taken collectively, the data provide a fairly comprehensive picture of the types of ground deformations that occurred in fill and the effect of such deformations on structures.

Sherman Oaks

The hillside areas of Sherman Oaks have a blend of wedge fills dating from before and after the 1963 grading code. Most of these fills are less than 6 to 9 m thick and are underlain by bedrock of the Modelo formation (Tertiary-age bedded sandstones, siltstones, and claystones). Concentrated structural distress and pipe breakage occurred in this area, with approximately 70 to 80 severely damaged (red-tagged) structures and 80 to 90 pipe breaks in the water distribution system (Stewart et al. 1996). Some of this damage has been attributed to landslides, often along bedding planes in the Modelo bedrock (Tan 1994). However, permanent fill deformations in the area were more widespread and appear to have been an important factor in much of the damage.

Overall, 56 documented fill movement sites have been compiled to date (although many more sites are known to have been affected), with ground cracks at most sites having vertical and horizontal offsets of less than 8 cm (although overall structure differential settlements of about 15 to 20 cm occurred in several houses). These movements damaged houses, pools, and patios and occurred in fills of both modern and older (pre-1963) construction, with no clear variations in damage intensity based on fill age. Several deaths were caused in the area by the collapse of stilt-supported homes constructed over a fill slope 18 m in height dating from the 1960s. However, these failures have been attributed to structural deficiencies and are not thought to have resulted directly from fill deformations.

Encino, Tarzana, and Woodland Hills

Postwar development of the north flank of the Santa Monica Mountains west of Interstate (I) 405 progressed gradually to the west through Encino, Tarzana, and Woodland Hills, away from downtown Los Angeles. Most of the fills in Encino date from the 1950s to early 1960s and consist of relatively shallow (<6 m thick) wedges with 1.5–2H:1V slope faces over Modelo bedrock. The fills in Tarzana and Woodland Hills date pri-

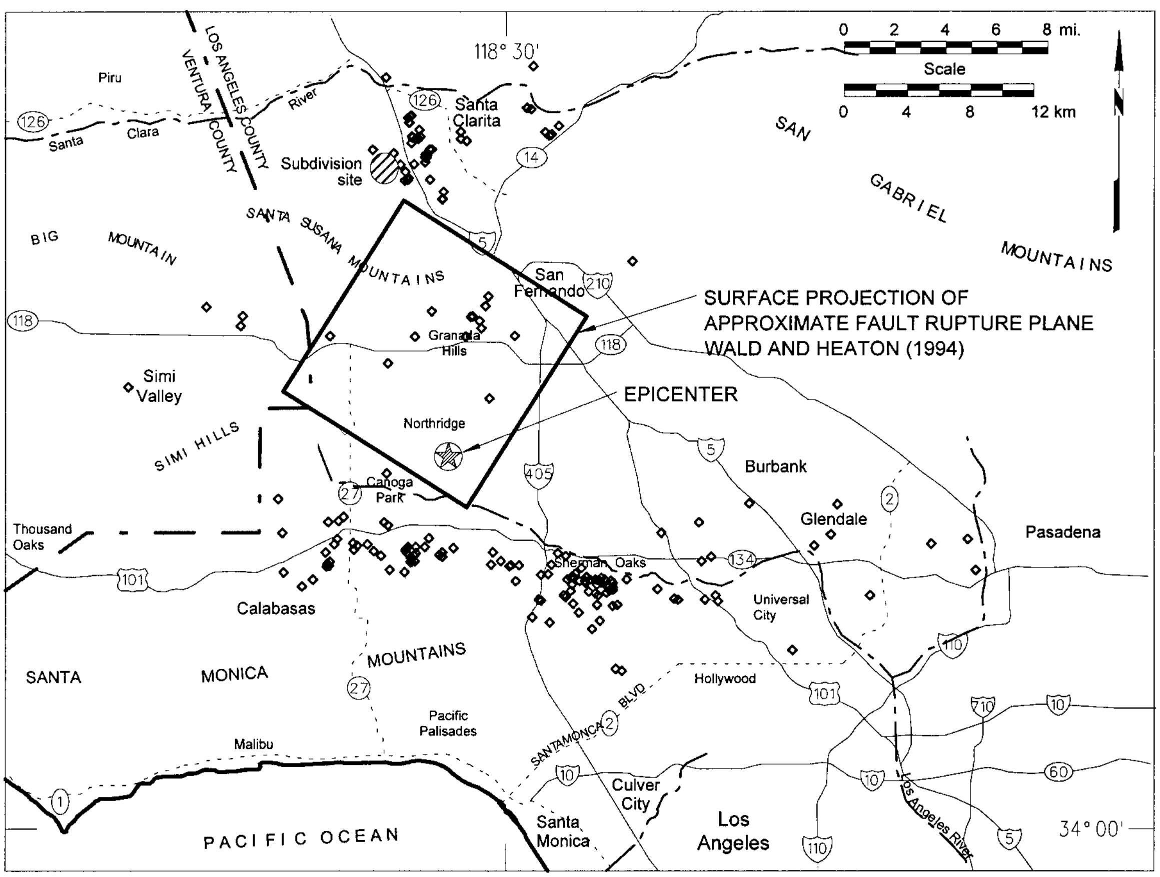

FIG. 2. Locations of Sites Significant Damaged due to Fill Deformations during 1994 Northridge Earthquake (In Total, Several Thousand Sites Were Affected)

marily from the late 1950s to early 1960s and 1970s to 1980s, respectively. These fills share similar geometric characteristics with the Encino fills, although some relatively deep fills (10–15 m) are also present. Damage intensities in these areas, as indexed by red-tagged structures (40) and pipe breaks (about 120 to 130), were less severe than in Sherman Oaks (Stewart et al. 1996). Fifty fill movement sites have been documented to date in this region, many of which occurred in areas of concentrated pipe breakage or structural damage. Surprisingly, damages were more pronounced in the relatively modern Tarzana and Woodland Hills developments than in Encino. Documented fill movements in these areas generally involved ground cracks having about 1 to 8 cm of horizontal or vertical displacement. Ground-floor manometer surveys indicated overall house differential settlements of up to 13 cm.

Porter Ranch and Granada Hills

Development in the Porter Ranch and Granada Hills areas has primarily occurred since the early 1960s. Fills in these areas vary from essentially level fills on broad alluvial planes to deep canyon fills. Structural damage and pipe breakage patterns in portions of this region were unusually severe; however, the most concentrated of these damages resulted from ground failure in natural soils (Stewart et al. 1996). Fewer than 20 incidents of damaging fill deformations have been documented to date in the region. At several sites developed in 1963, fill movements involved 10 to 20 cm horizontal and vertical crack offsets in 5 to 9 m thick wedge fills. Movements in more recent fills were generally smaller, typically involving ground cracks with horizontal and vertical displacements less than 5 cm.

Santa Clarita Area

Development in the Santa Clarita Valley has occurred across a variety of terrain ranging from alluvial plains to deep canyons. Based on data compiled to date, portions of Santa Clarita having significant fill movements included

- West of I-5: This area has primarily been developed since the mid-1980s and has many 6 to 12 m thick, single- and multiple-pad wedge fills with 1.5–2H:1V slope faces. The fills often directly overlie primarily Tertiary-age Saugus bedrock (sandstone and siltstone). Movements in these fills resulted in up to 5 cm crack offsets and caused up to 5 cm total differential settlements in houses. A 0.9 ha school site on a canyon fill in this area is of particular interest because seismic deformations were quantified by pre- and post-earthquake surveys. Fill depths range from 15 to 30 m, and all fill was compacted at water contents greater than the optimum from the modified Proctor standard. The fill materials consist of silty sands (% fines $\approx$ 40–50; PI < 5). The magnitude of surface deformations in areas where the fill was relatively densely compacted (modified Proctor relative compaction > 95%) were significantly smaller than the deformations in adjacent areas with lower levels of compaction (>90%). Using the observed settlement data, Stewart and Smith (1998) estimated average volumetric strains as 0.05–0.1% in 95% RC fill and 0.5% in 90% RC fill.
- Valencia south of McBean Parkway: This area is located on a ridge of uplifted Pleistocene terrace deposits and was developed primarily from the late 1960s to 1970s. Single-pad wedge fills 5 to 9 m thick had extension cracks up to 10 to 30 cm wide, although 3 to 8 cm crack widths were more typical.
- Santa Clara River area: Shallow (<3 m thick) wedge fills in areas near the river overlie liquefiable alluvium. Ground deformations at such lots resulted in house differential settlements of up to 10 cm. Located slightly north of the river is a 12.1 ha postal facility with several canyon fills overlying shallow alluvium and/or rock. Fill depths range from 0 to 24 m, with the fill having been placed with no water content control and a minimum relative compaction (modified Proctor standard) of 90%. Field construction logs suggest that actual compaction levels may have been less than 90%, with as-compacted water content near or below optimum. The fill generally consists of low-plasticity sandy silty clay (% fines $\approx$ 54; PI $\approx$ 13). The maximum observed settlement was about 20 cm in an area with about 21 m of fill ($\sim$1% average volumetric strain).
- Other areas, including (a) portions of Valencia north of McBean Parkway, where movements of 1980s-era wedge fills overlying terrace deposits resulted in up to 7.5 cm of extension across cracks and house differential settlements up to 10 cm; (b) Newhall, where movements of pre-1963 and modern wedge and canyon fills overlying Saugus bedrock resulted in up to 7.5 cm ground-crack displacements and house differential settlements; and (c) an area near Highway 14, where movements of post-1963 canyon fills up to 15 m deep resulted in house differential settlements and ground crack widths generally less than 8 cm.

Example of Effect of Fills on Structures—Santa Clarita Subdivision Case Study

The Santa Clarita Valley area was among the regions most strongly shaken by the Northridge earthquake and experienced significant ground deformations in compacted fill. Recent development in outlying portions of the valley has often occurred in deeply incised canyon/ridge topography, which has required massive grading operations involving deep canyon fills. Engineered improvements constructed across fill and cut areas are often of fairly uniform design and construction. Such sites provide the opportunity to assess the impact of earth fills on the performance of improvements (such as pipelines and houses) by comparing damage statistics for cut-and-fill areas.

One such site is the 8,400 ha subdivision shown in Fig. 3. At the time of the Northridge earthquake, 645 properties in the subdivision had been developed, with the construction having occurred between July 1986 and October 1987. The site is approximately 9 km from the Northridge fault rupture plane (Wald and Heaton 1994), and likely experienced peak ground accelerations on rock on the order of 0.3 to 0.5*g* (Chang et al. 1996). Original topography at the site consisted of numerous canyons and ridges, with a general increase in elevation to the west. Grading operations involved the construction of fills with maximum depths typically on the order of about 15 to 21 m. The fill soils placed at the site are primarily sands and silty sands, with nonplastic fines contents on the order of 15 to 30%. Fill placed at the site was required to have a minimum relative compaction by the modified Proctor standard of 90%. Water content was not controlled during construction, and cut areas were not overexcavated.

We have documented the performance of all major buried pipelines (water, sewer, storm drain, and gas) and most building structures that were in place at the time of the 1994 Northridge earthquake. As shown in Fig. 3, a total of 14 breaks were reported in the water distribution system (15, 20, or 25 cm diameter asbestos-concrete pipes), most of which are described as ‘‘shear failure.’’ All the breaks occurred in fill, generally near cut/fill contacts. The gas and storm-drain lines primarily consist of relatively flexible PVC pipe, and no breaks were reported. A 152 cm diameter reinforced concrete storm drain, constructed in 2.4 m sections, which passes through the subdivision only had minor damage at grout joints that was

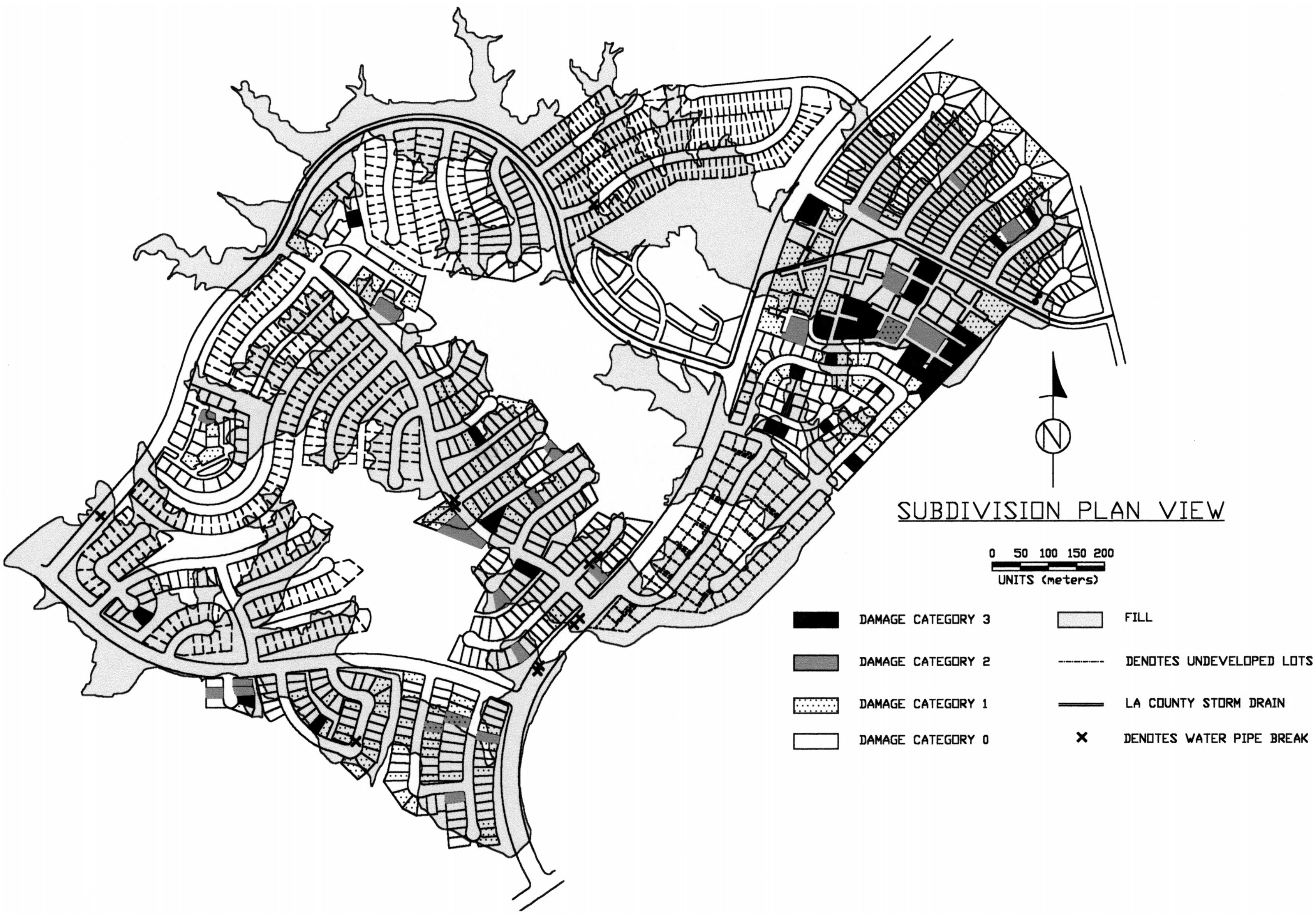

FIG. 3. Plan View of Santa Clarita Subdivision Showing Fill and Cut Zones and Locations of Damaged Water Pipes and Structures

TABLE 1. Damage Statistics for Subdivision as Function of Site Condition. Indicated Are Numbers (and Percentages in Parentheses) of Lots within Each Site Category with Different Damage Levels

Site condition	No damage[a]	Cosmetic damage[b]	Moderate damage[c]	Significant damage[d]	Total
Cut	193 (77%)	49 (20%)	3 (1%)	5 (2%)	250
Cut/fill	159 (66%)	60 (25%)	11 (4%)	12 (5%)	242
Fill	100 (65%)	39 (25%)	8 (5%)	7 (5%)	154
All lots	452 (70%)	148 (23%)	22 (3%)	24 (4%)	646

[a]No damage. No observed distress, or no homeowner request for inspection.
[b]Cosmetic damage. Cracks in walls and ceilings that do not threaten structural integrity.
[c]Moderate damage. Cosmetic damage + damaged roof, chimney, floors, windows, or plumbing, suggesting some ground deformation or intense shaking.
[d]Significant damage. Moderate damage + cracked foundation and displacements observed in soil, suggesting significant ground deformation.

uniformly distributed across the length of the pipe (that is, no concentration of damage in fill areas). Apparently the strength and stiffness of this large-diameter RC section was sufficient to resist damage associated with deformations in fill.

Damage to structures was evaluated based on inspection reports prepared by Los Angeles County staff within one month of the earthquake. Inspections were made upon the request of property owners seeking earthquake relief. Specific damages were documented, such as foundation cracks, wall cracks, and collapsed chimneys, and monetary losses were estimated. Some properties were not inspected, presumably because of little or no earthquake damage. Fig. 3 shows the damage level at each site based on the four categories in Table 1. Also shown in Table 1 is the frequency with which the various damage levels were encountered in cut, fill, and cut/fill transition lots. These data indicate that the likelihood of significant damage (damage category of 2 or 3) on cut/fill or fill lots was more than twice that on cut lots.

The reported damage from this subdivision indicates that the presence of fill significantly affected the likelihood of damage to pipelines and building structures, as all reported pipeline breaks occurred in fill near cut/fill transitions, and the likelihood of significant structural damage in fill or cut/fill areas was more than twice that in cut areas.

Summary of Fill Movement Characteristics

The characteristics of the fill movements in the areas discussed above were similar. With the exception of the Porter Ranch and Granada Hills area, fills constructed prior to and following the 1963 grading provisions appeared to undergo similar types and magnitudes of deformations. Although modern fills generally were compacted to higher relative compaction levels than pre-1963 fills (and thus would be expected to have less deformation), they are also significantly deeper (for example, see slope heights in the Appendix). Thus, the similarity of the deformation magnitudes is not surprising. An important finding from the school and post office sites in Santa Clarita was that fill deformations appear to be sensitive to relative compaction and as-compacted water content. Characteristic fill deformation features are illustrated in Fig. 4 and discussed below.

- Cracks near cut/fill contacts: The most commonly observed location of ground cracks in building pads was at cut/fill contacts, or above the nearest bench to cut/fill contacts. Cut/fill cracks typically had less than 8 cm of lateral extension and 3 cm of localized differential settlement of the fill relative to the cut. Damage to structures crossing these features was often significant (Fig. 5). Where investigated with trenching or downhole logging (for example, Fig. 6), these cracks were found to become thinner with depth and could only be traced to depths of 1 to 2 m. Hence, the cracks did not appear to be surface expressions of shear failures along fill/native soil-rock contacts. In addition to this cracking in building pad areas, cracks occurred at contacts between fill slope faces and side canyon walls.
- Lateral extension in fill pad: Evidence of lateral extension of fill pads was commonly observed in the form of tensile cracking parallel to the top of the slope and the opening of relatively large (>3 cm) separations at cold joints between concrete slabs and footings (Fig. 7) or between pools and pool decks. These features typically involved about 3 to 10 cm horizontal or vertical offsets, but significantly wider cracks (<30 cm) occurred at some sites. The setback of tensile cracking from the top of slope tended to increase will fill depth, and most houses constructed with Uniform Building Code-level setbacks (one-third slope height) were not damaged by this cracking.
- Settlement: Fill-pad settlements increased with fill depth,

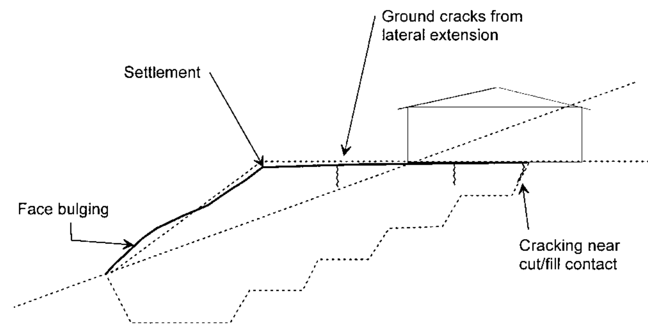

FIG. 4. Schematic Showing Typical Damage to Fill Slope

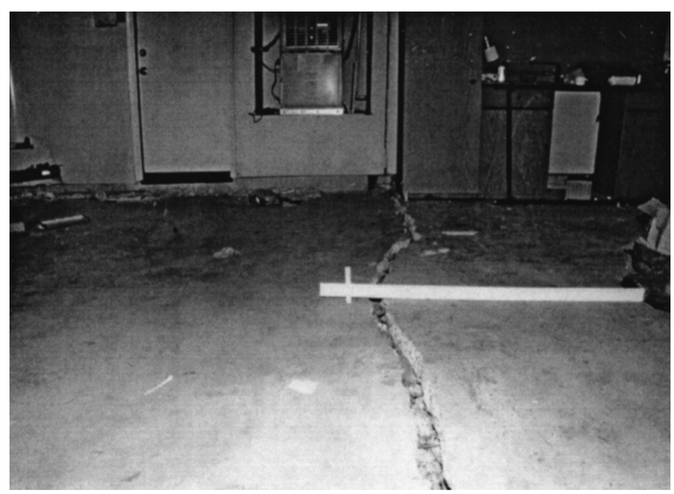

FIG. 5. Cracked Floor Slab above Cut/Fill Contact—Displacements Are 1.9 cm (V) and 5 cm (H)

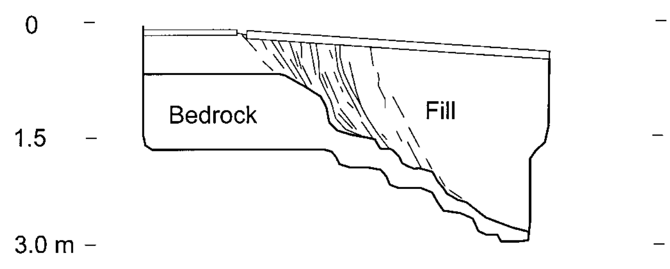

FIG. 6. Trench Log Showing Cracking Pattern near Cut/Fill Contact—Cracking Extends to Depth of about 2 m (Adapted from Seward, Confidential Geologic Report, 1994)

resulting in differential settlements across the surface of fills. These settlements were often measured within houses by means of manometer floor level surveys. A typical criterion allows for 2.5 cm floor-level differentials within 6 m (0.4% floor slope), although Los Angeles County requires engineers to design for 1.25 cm settlement in 9 m (0.14% slope) (Pearson 1995). Using data from the sites in the Appendix, maximum floor slopes for fills underlain by stable rock or terrace deposits ranged from 0.3 to 2.4%, with an average of 1.1%, and data for fills underlain at least in part by soil (primarily alluvium) had a range of 0 to 2.2%, with the same average of 1.1%. Average maximum floor slopes for pre- and post-1963 construction were 1.3 and 1.0%, respectively, although numerous examples of large (>2%) floor slopes were found in fills from both eras. Overall, hillside fill settlements resulted in floor slopes that significantly exceeded normal tolerances for houses, although some of these movements were likely derived in part form pre-earthquake consolidation or hydrocompression.

FIG. 7. Evidence of Extensional Ground Deformation at Back of House (Top of Slope Is to Left)

FIG. 8. Uplifted Down Drain Indicating Compression of Fill Slope Face

- Face bulging/shortening: Detailed slope face inspections were performed at a number of sites, and at a limited number of these, fill-slope face bulging was evident from movements of concrete surface drains running cross-slope (terrace drains) and down-slope (downdrains). Terrace drains had cracks oriented perpendicular to the slope contours that widened in the down-slope direction, providing evidence for face bulging of the center of the fill. Uplifted downdrains were observed in some large fills at approximately one-third the slope height (Fig. 8), indicating shortening of the lower slope face.

FACTORS CONTRIBUTING TO SEISMICALLY INDUCED FILL DEFORMATIONS

A number of ground-failure mechanisms can induce the types of fill deformations described previously. Rogers (1992) investigated seismically induced deformations of highway embankments and identified the deformation mechanisms of (1) dynamically induced settlement (from volumetric compression or slope instability in fill) and (2) seismic activation of landslide complexes in underlying materials. Some fill movements from the Northridge event can be traced to permanent deformations of soils or rock underlying the fill, often due to landsliding, liquefaction, or densification of these materials. In most cases, however, fill movements appear to have resulted from deformations within the fill mass itself. At several sites investigated with trenching or downhole logging, no evidence of significant movements on distinct sliding surfaces within the fill was found. Investigated below is the possibility that deformations within fill resulted from accumulation of volumetric or distributed shear deformations during strong shaking.

The following sections present generalized analyses of typical fill geometries that underwent permanent ground deformations during the Northridge earthquake. The principal objective is to evaluate if accumulation of permanent volumetric and shear deformations can account for typical observed fill movements, using appropriate ground-motion levels and typical soil properties. Also evaluated is the degree to which soil/topographic amplification may have influenced ground motions in these fills.

Fill Geometries and Material Properties

Two fill geometries are considered. Fill A, shown in Fig. 9(a), is a hillside wedge fill, commonly found in many areas affected by the Northridge earthquake, including the Santa Monica Mountains. A 2H:1V slope angle is used here with a slope height of 12 m, which is typical of many southern California wedge fills. Fill B [Figure 9(b)] is a 2D section up the axis of an actual canyon fill in the Santa Clarita area affected by the Northridge earthquake. This fill has an average slope angle of 2.3H:1V and a slope height of 16 m. This geometry of Fill B is representative of many canyon fills in the Santa Clarita region.

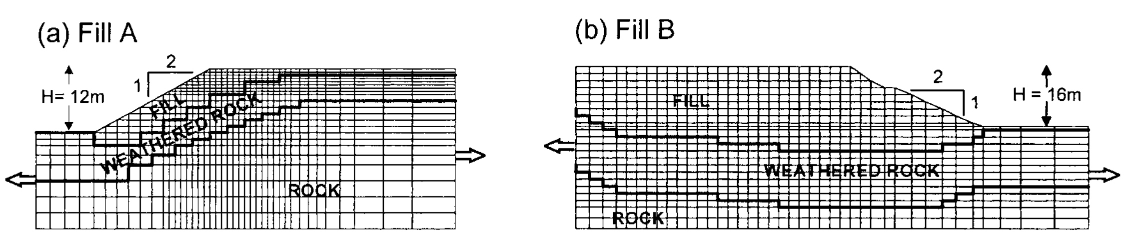

FIG. 9. Fill Slope Configurations and Finite-Element Meshes Used in Model Studies. Diagrams Indicate Horizontal Positions of Graphs in Fig. 10

Hillside fills are often underlain by relatively firm bedrock or native soils. For these analyses the material underlying the fill is assumed to be sedimentary bedrock with a surface shear wave velocity of V_s = 600 m/s, increasing to 900 m/s at a depth in rock of 10 m. This is a typical velocity profile based on data recently compiled by the ROSRINE program (Resolution of Site Response Issues from the Northridge Earthquake) for ''rock'' sites in Tertiary-age sedimentary bedrock deposits in southern California.

Shear-strain dependent modulus reduction and material damping in the bedrock are modeled using curves for sand at depths >100 m by EPRI (1993). Separate models for fill material properties are considered for Fills A and B, due to the different grading standards employed during development of the Santa Monica Mountain and Santa Clarita locations. The relatively modern fills in Santa Clarita (Fill B) are modeled as moderately to well-compacted sandy materials with relative densities of $D_r \approx 45$ to 60% and $V_s \approx 200$ to 300 m/s. The older fills in the Santa Monica Mountains (Fill A) are modeled as moderately compacted sandy materials with a relative density of $D_r \approx 45\%$ and $V_s \approx 150$ to 200 m/s. These models correspond roughly to relative compaction levels by the modified Proctor standard of ~85 to 95% (Santa Clarita) and ~85 to 90% (Santa Monica Mountains) (Lee and Singh 1971). These compaction levels are reasonably representative of the soil properties documented in the Appendix. Shear modulus reduction and material damping in the sandy fill are modeled using curves for sand proposed by Seed et al. (1984). These curves are consistent with recent test results for southern California soils by Stokoe et al. (1999) and Lanzo et al. (1997).

Ground Motions Considered

Different sets of ground motions are used here to simulate conditions for Fill A, which is representative of conditions in the Santa Monica Mountains south of the fault rupture plane, and Fill B, which is more representative of conditions in Santa Clarita, north of the fault rupture plane. Different ground motions are needed for these two areas as a result of different rupture directivity effects (for example, Somerville et al. 1997), which are forward (large-amplitude pulselike motion) in Santa Clarita, and backward (lower amplitude, but longer duration) in the Santa Monica Mountains. The motions selected for the two areas are listed in Table 2 along with key engineering parameters describing the selected motions. It should be noted that the Newhall motions (Santa Clarita area) were recorded on soil and have been deconvolved with the program SHAKE91 (Idriss and Sun 1991) to evaluate appropriate rock motions. All other stations were located on sedimentary rock site conditions or shallow soil overlying rock. The motions for Fills A and B have been scaled to have common peak accelerations of 0.3 and 0.4*g*, respectively. These are typical ground motion levels in the respective areas based on local recordings (Chang et al. 1996).

Results

Ground Motion Amplification

The analyses were performed using the finite-element program QUAD496 (Hudson and Idriss 1996), which employs a time domain solution of the equations of motion and equivalent linear dynamic soil modeling. The finite-element models employed in the studies are shown in Fig. 9. The models were extended 300 m in both horizontal directions from the top of slope to minimize the effects of lateral boundary conditions on the results.

The results of these analyses support the notion of ground-motion amplification in fill. Fig. 10 shows the ground-motion amplification along the surface of the fill relative to cut (bedrock) areas at spectral periods of $T = 0$ (peak acceleration); $T = 0.3$ s; and $T = 1.0$ s. Amplification in these figures is defined as the spectral acceleration on the fill surface normalized by the median spectral acceleration of the input motions at the same period. Amplification in Fill A (which has relatively shallow depths of fill and a resonant period of $T_s < 0.2$ s) is most pronounced at low spectral periods and is negligible at $T = 1.0$ s. The response of Fill B is more nonlinear than that of Fill A, which is due in part to the higher peak acceleration used for scaling the Fill B motions (that is, 0.4*g*, as compared to 0.3*g* for Fill A). The nonlinearity reduces amplification levels at small spectral periods (for example, $T = 0$ and 0.3 s), and shifts the strain-softened resonant period in the deepest part of the fill to $T \approx 0.7$ to 0.8 s. Accordingly, the most

TABLE 2. Earthquake Motions Used in Finite-Element Analyses

Station	Source[a]	r (km)[b]	MHA[c] (g)	PGV[d] (cm/s)	$T_m(s)$[e]	N_{cyc}[f]
(a) Santa Clarita Site						
Newhall FS, deconvolved	CSMIP	7.1	0.57	84.2	0.49	10.4
Los Angeles Reservoir, abutment	LADWP	7.1	0.49	56.1	0.85	3.3
Pacoima, Kagel Canyon	CSMIP	8.2	0.36	45.6	0.67	6.2
Castaic Dam, downstream	DWR	19.3	0.21	27.4	0.68	11.6
Lake Piru/Santa Felecia Dam	LADWP	21.4	0.23	21.9	0.57	6.1
Castaic, Old Ridge Route	CSMIP	22.6	0.51	48.3	0.62	5.3
(b) Santa Monica Mountains Site						
Encino Dam, abutment	LADWP	16.8	0.20	18.7	0.43	10.2
Beverly Hills, Mullholland Dr. (1)	USC	19.6	0.46	60.8	0.69	10.5
Beverly Hills, Mullholland Dr. (2)	USC	20.8	0.52	35.1	0.33	6.7
Los Angeles, Wonderland	USC	22.7	0.14	10.1	0.40	8.3
Low Franklin No. 2 Dam, base	LADWP	23	0.15	10.1	0.38	8.9
Los Angeles, Chalon Rd.	USC	23.7	0.20	21.2	0.57	10.6
Los Angeles, N. Faring St.	USC	23.9	0.26	21.7	0.49	7.4

Note: Santa Clarita sites have forward directivity, Santa Monica Mountains backward directivity.
[a]CSMIP = California Strong Motion Instrumentation Program; LADWP = City of Los Angeles, Department of Water and Power (C. Davis, personal communication, 2000); DWR = California Department of Water Resources; USC = University of Southern California.
[b]Closest distance to fault.
[c]Maximum horizontal acceleration, geometric mean of two components.
[d]Peak ground velocity, geometric mean of two horizontal components.
[e]Mean period, geometric mean of two horizontal components (Rathje et al. 1998).
[f]Number of equivalent cycles at 0.65 × MHA (Liu et al. 2001).

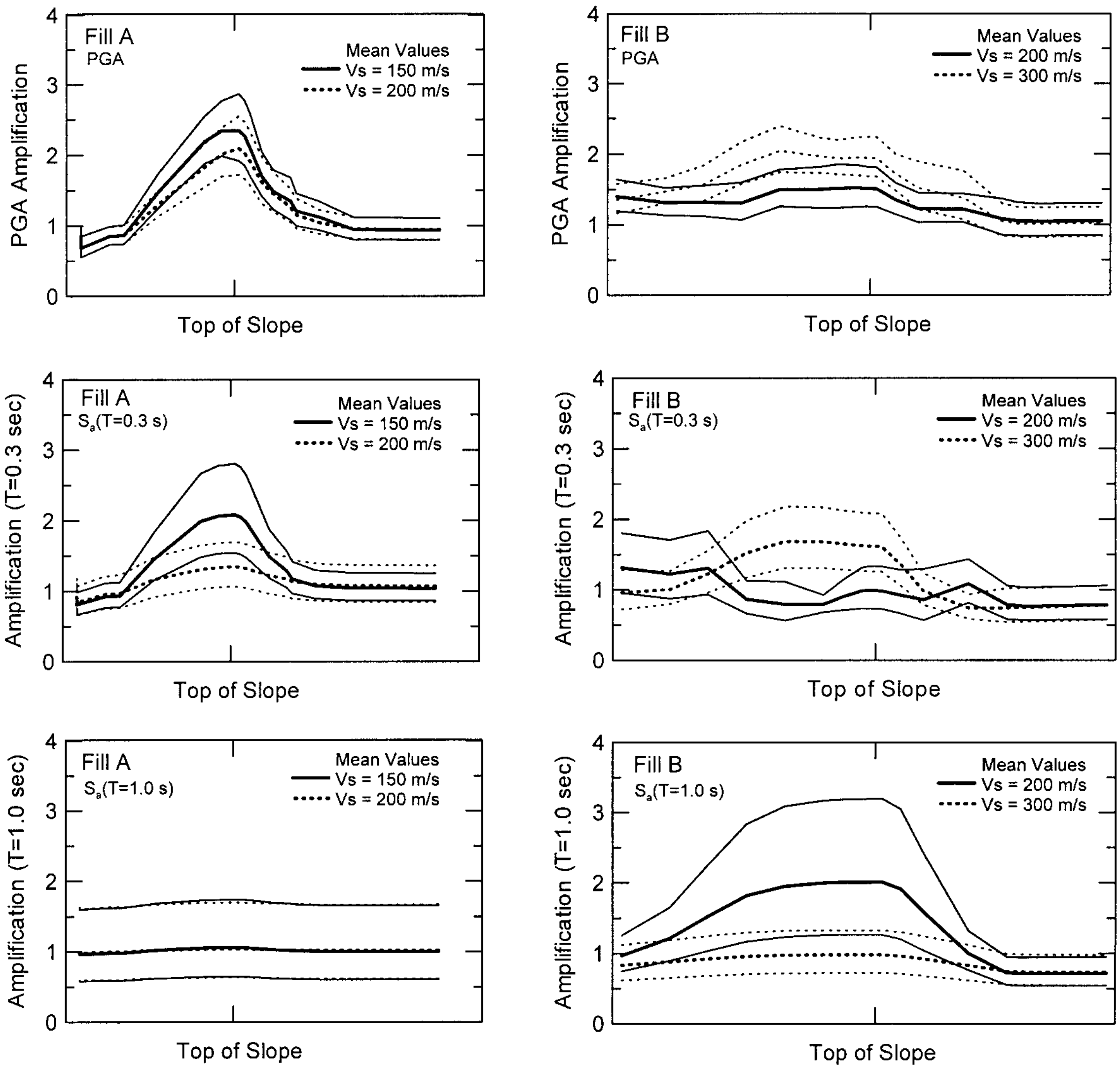

FIG. 10. Variation of Normalized Spectral Accelerations across Fill Surface—Shown Are Mean (μ) and $\mu \pm$ One Standard Deviation (σ) for Respective Shear Wave Velocities (V_s)

pronounced amplification for Fill B in Fig. 10 is at T = 1.0 s, near the strain-softened resonant period of the fill. For both Fills A and B, amplification levels near resonance are approximately 2 to 3, amplification at periods significantly greater than the resonant period is nearly unity, and PGA amplification, while highly variable, ranges from about 1.5 to 3.

Ground-motion amplification in fill, while significant, does not by itself explain the observed permanent deformations of hillside fills. Although some spatial incoherence between ground motions in cut-and-fill portions of building pads occurs as a result of these amplification effects, computed relative displacements associated with this incoherence are very small relative to the observed deformations. Hence, the importance of ground-motion amplification in fill is primarily associated with its contribution toward the permanent soil deformation mechanisms discussed below.

Volumetric Deformation (Seismic Compression)

Most hillside fills are unsaturated, and hence the potential for significant seismically induced pore-pressure generation and/or liquefaction is small. However, contractive volumetric strains (that is, seismic compression) can accumulate in these unsaturated soils during strong shaking. Previous studies have found that volumetric strains in clean sands from cyclic loading depend on relative density, shear-strain amplitude, and the number of loading cycles, but are relatively insensitive to static vertical stresses and the frequency of loading (Silver and Seed 1971; Youd 1972). Pyke et al. (1975) found that both directions of horizontal earthquake shaking contribute in proportion to their relative intensities to overall settlements and that vertical shaking can contribute as well. Other studies have documented the potential for soils with significant fines to accumulate volumetric stain under cyclic loading (Chu and Vucetic 1992; Whang 2001), with the deformation being sensitive to the plasticity of the fines and the water content during compaction. At the same relative compaction, the potential for seismic compression tends to decrease with increasing soil plasticity or for soils compacted wet of the line of optimums. For these analyses the soils are assumed to be sandy, and existing analytical formulations for seismic compression in sands are employed (Tokimatsu and Seed 1987).

Figs. 11(a–b) are plots of peak dynamic shear strain versus depth computed from the finite-element analyses. The strain levels are higher in the fills than in the underlying rock, with the magnitude of strains in the fill being sensitive to the fill/weathered rock impedance contrast and fill geometry. The largest strains (<0.2 to 0.4% for Fill A, 0.4 to 1.2% for Fill B) occur near the fill/weathered rock interface, and strains decrease toward zero near the ground surface. However, throughout the fill profile, shear-strain amplitudes consistently exceed typical threshold strains for volumetric deformation in silty sands (~0.01 to 0.05%; Vucetic 1994), suggesting that nearly

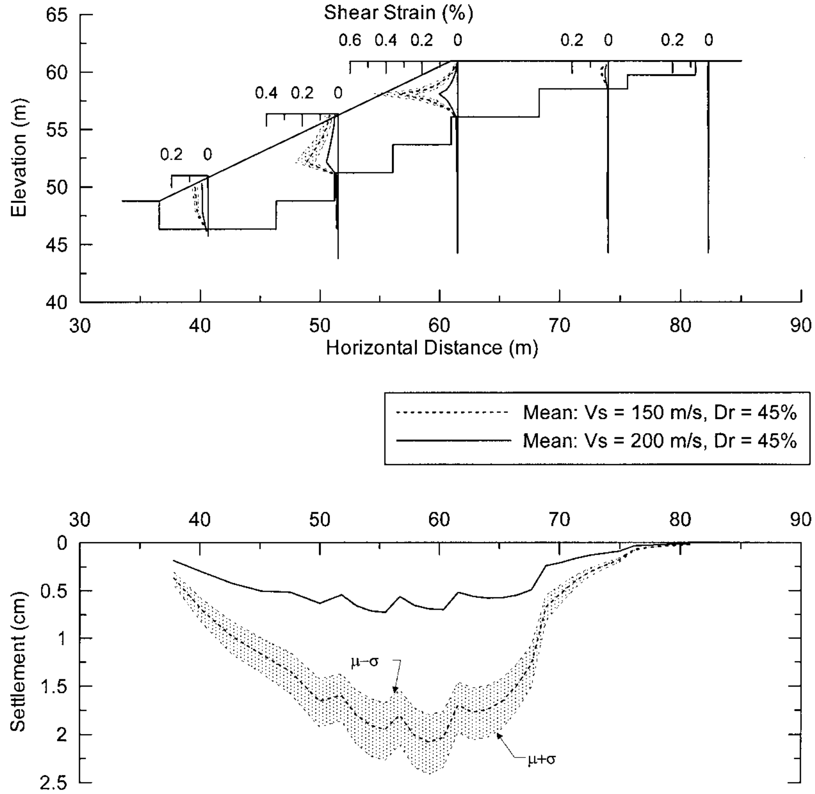

FIG. 11(a). Profiles of Peak Horizontal Shear Strain with Depth in Fill A and Variation of Computed Settlement across Fill Surface Resulting form Seismic Compression (Using Tokimatsu and Seed 1987 Formulation for Clean Sand)

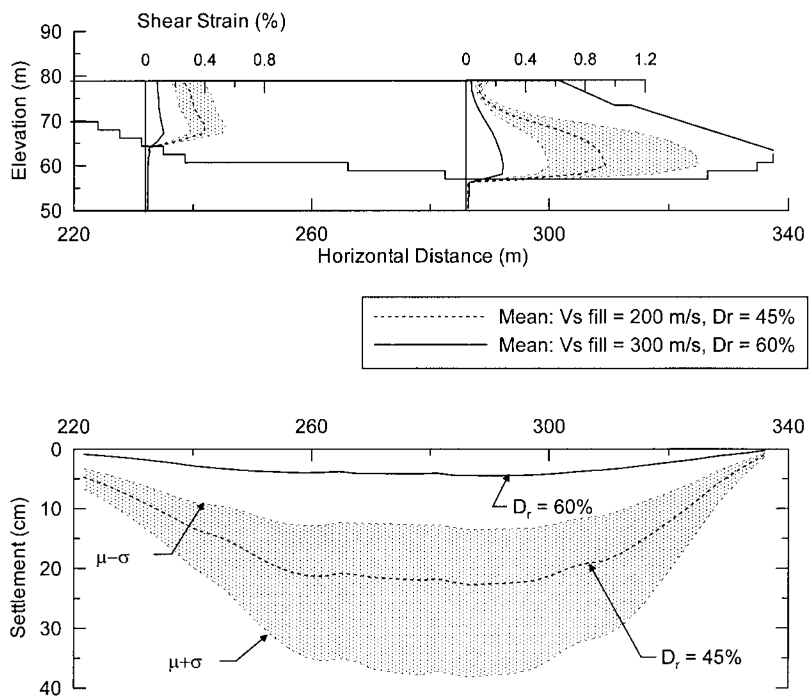

FIG. 11(b). Profiles of Peak Horizontal Shear Strain with Depth in Fill B and Variation of Computed Settlement across Fill Surface Resulting from Seismic Compression (Using Tokimatsu and Seed 1987 Formulation for Clean Sand)

the full depth of the fills likely experienced volumetric strains. Volumetric strains computed from these shear strains by the Tokimatsu and Seed (1987) procedure were evaluated and integrated to provide the settlement profiles in Figs. 11(a–b). As expected, settlements increase with depth of fill, with substantial variations observed near cut/fill boundaries (for example, near the 70 m mark in Fill A). Other key observations from Figs. 11(a–b) are

1. The estimated settlements, which range from 0.5 to 2.0 cm for Fill A and 5 to 20 cm for Fill B (mean values), are generally consistent with field observations, indicat-

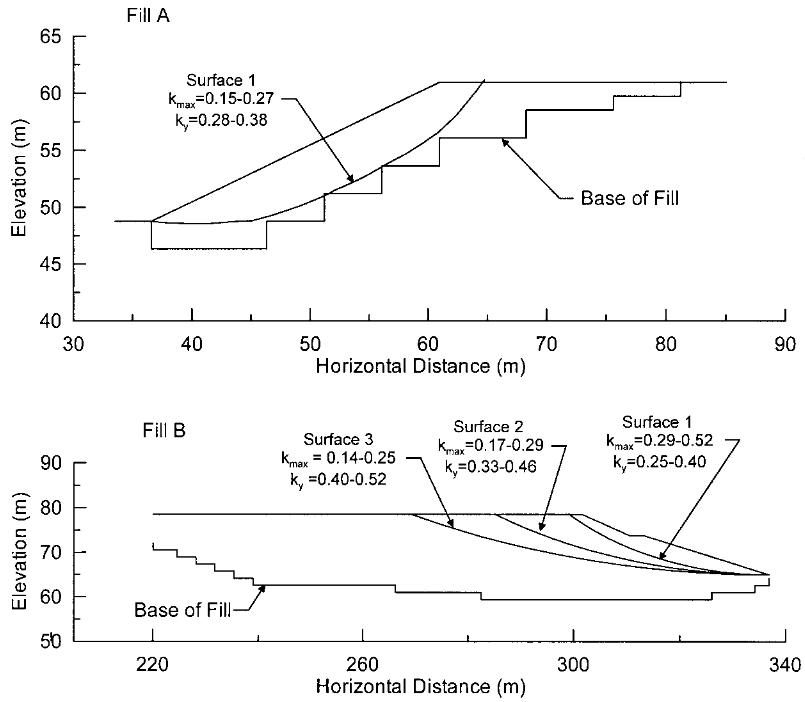

FIG. 12. Slip Surfaces Considered for Newmark Displacement Analysis

ing that seismic compression may be able to explain many observed occurrences of ground deformation in compacted fills.

2. Substantial benefit is gained by increasing the compaction level of sandy soils, as indicated by a comparison of the results for different D_r soils in Fig. 11(b).

Finally, it should be noted that the deformations associated with seismic compression occur primarily in the direction of principal stress, which has a lateral component within the fill section. Hence, the effects of seismic compression are expected to consist of surface settlement and minor lateral extension, which is consistent with field observations of fill pad movements.

Permanent Shear Deformations

Permanent shear deformations within a fill slope generally occur in the direction of a driving static shear stress (that is, downslope). As noted previously, field observations indicated that shear deformations were generally not localized across a distinct shear failure surface (that is, landsliding). In the absence of landsliding, shear deformations can occur across a distributed zone where seismically induced shear stresses exceed an effective soil yield stress. Engineers typically evaluate the susceptibility of a slope to deformations across either distinct or distributed yield zones using a Newmark analysis of a sliding rigid block (Newmark 1965; Franklin and Chang 1977) or a simplified procedure for estimating Newmark-type displacements that captures the dynamic response of a flexible slide mass (for example, Makdisi and Seed 1978; Bray et al. 1998). Because both calculate sliding of a block along a distinct sliding surface, they do not model the actual deformation process and only provide an index of performance.

The potential for distributed shear deformations in Fills A or B are assessed here using a sliding block analysis, which requires time histories of horizontal equivalent acceleration (HEA) for a potential sliding mass (that is, material above an assumed slip surface). The HEA time histories were calculated using QUAD496 for the recordings listed in Table 2 and the slip surfaces depicted in Fig. 12. The ranges of maximum horizontal equivalent acceleration (MHEA/g = k_{max}) from these analyses are indicated in Fig. 12. Also shown on Fig. 12 are ranges of yield coefficients (k_y) for these surfaces using material strengths of c = 5 kPa and ϕ = 32 to 38 degrees. The single surface shown for Fill A and Surface 1 for Fill B are the critical surfaces for static and seismic stability using these strength parameters. Seismically induced shear displacements across the indicated range of k_y were calculated using these HEA time histories and the Franklin and Chang (1977) procedure. The calculated displacements were <0.01 cm for Fill A and for Surfaces 2 to 3 in Fill B, and were <1 cm for Surface 1 in Fill B. These small displacements suggest that permanent shear deformations were unlikely to have contributed significantly to observed deformation patterns across the developed portions of fills having geometries similar to those investigated here. In the absence of such shear deformations, observed fill pad movements are most likely explained by seismic compression in fill.

CONCLUSIONS

Earthquake-induced deformations of hillside fills represent a problem with significant economic ramifications to developed hillside areas in seismically active regions. During the 1994 Northridge earthquake, fill deformations damaged thousands of residences, resulting in economic losses totaling hundreds of millions of dollars. Characteristic fill deformations from the Northridge event consisted of cracking at cut/fill contacts and lateral extension and settlement in fill. Localized horizontal and vertical crack offsets from these deformations were typically less than 8 cm. These types of movements, coupled with possible ground-motion amplification in fill, were found to cause significantly greater damage in fill areas than in cut areas in a large subdivision in Santa Clarita. Data from sites with known amounts of fill movement indicate such deformations are sensitive to the relative compaction of the fill and can occur in clean sands as well as fills with significant fines content.

The pseudostatic seismic landslide analysis procedures used to design some of these slopes were ''correct'' to the extent that significant landsliding along well-defined slip surfaces was generally not observed. However, these analyses, as well as more sophisticated Newmark-type sliding block analyses, cannot predict the ground deformations and distress patterns observed at the majority of sites. Rather, the primary cause of the observed ground deformations appears to have been seismic compression in fill, which has largely been ignored in the design of residential fills. The potential for seismic compression can be evaluated using currently available procedures in a three-step approach: (1) calculate dynamic shear strains in fill (using either ground response analyses such as QUAD496 or SHAKE91 or simplified procedures such as those in Tokimatsu and Seed 1987); (2) evaluate volumetric strains knowing shear-strain amplitude and equivalent number of uniform strain cycles [using published results for clean sands (Silver and Seed 1971) or soils with fines (Whang 2001), or material-specific laboratory testing]; and (3) integrate volumetric strains to estimate surface displacements. Additional calibrations of this simplified approach are warranted.

While standards of practice in California are evolving to more rationally consider seismic slope instability from shear failures (largely as a result of the seismic slope stability provisions in the 1990 California Seismic Hazards Mapping Act), significant additional progress is needed in the development and implementation of robust procedures for evaluating seismic compression. It is hoped that the documentation of actual ground deformations in hillside areas provided here will increase the awareness of this important seismic hazard among practitioners and regulators and aid in the development of effective design and mitigation measures.

APPENDIX. DATA FOR SELECTED FILL DEFORMATION SITES (1994 NORTHRIDGE EARTHQUAKE)

Fill Properties: date of const.	slope angle (H:V)	slope height (m)	setback (m)	fill density (g/cm^3)	Rel. Comp.[B]	N_1[C]	Approximate Site Stratigraphy[D]	Comments[E]	Diff. Stlmnt.[F] Max. Overall: stlmnt (cm)	Max. Overall: distance (m)	Max. Slope: stlmnt (cm)	Max. Slope: distance (m)
SAN FERNANDO VALLEY[A]												
Sherman Oaks												
1962				1.4,1.6			0-4.9m fill, CL/ML, Modelo br.	damaged house	13	15	5	2.1
1960s	1.5:1	18					1-18m fill, SM; Modelo br.	collapsed stilt homes, 2 deaths				
1960s		15					1.5-9m+ fill	damaged fndn from settlement				
1961	1.5:1	5,18	1.5	1.6	84		2.1-4.0m fill, SM; Modelo br.	2.5cm lat. cr. in fndn; slab, fill cr				
N/A	flat			1.6		14,6[M]	0.6-1.0m fill; alluvium (no gw. to 4.5m)		5	4.6		
1980	1.5:1	30					fill over rock	cr. in house at c/f				
N/A							6-12m fill over Qls which moved					
1977	1.5:1						up to 1.5m fill		8	15	2.5	3
N/A	1.5:1	1-2 m wall						wall mov't; bedrock landslide	19	14	(2.1%)	
1975	1:1	>7.5				25-40	0-4.3m fill, ML (PI=28); Modelo br. (no gw)		7	NA		
1968	2:1	3-6		1.8	71-86	34-60	4.5-9m fill, silty and cl.; 1.5-4.9m alluv., cl. silt, sand; Modelo br.	increasing settlement with fill depth	16	15		
1974	1.5-2:1	23	15	1.9	84-89		2-8m fill, SM; Monterey ss and shale	minor str. distress; 1cm pav't/gnd. cr., 2cm pav't sep.	15	18	6	4.6
Encino												
1963	1.5:1	4.5	9	1.5	74		1.5-3m fill, clay; Modelo br. (no gw)	2.5-5cm fill settlement nr. house, no lat.				
Tarzana												
N/A	3:1		0	1.7		8	0.3-1.8m fill, SM (PI=22); 0-1m CL; Modelo br. (no gw)		5	21	4	3
1960	1.5:1	4.5,18		1.8	80-90		9-14m fill, MH (PI=14-20); Modelo br (no gw)	1-2cm pav't cr.				
N/A	1.5:1	0-7.5		1.9	83-92	24-36	9-15m fill, ML/CL; Modelo br. (no gw)		9	18		
1964	1.5:1		12				0-3m fill, SM; Modelo br. (no gw)	mov't of 1.5m ret wall mov't; <2cm cr. near wall				
N/A	1.5:1	7.5					0-4.3m fill, CL; 0.3-0.6m cl.; Modelo br. (no gw)	3.5cm d.s.; slab separations				
1959	1.3:1	7.5	6	1.9	85-96		1-4.3m fill, CL/ML; alluv/colluv, SC (gw 7m)	lat, settlement cr. top of slope				
1959		5.2					1-1.5m fill, SC/CL; >6m alluv., cl.; Modelo br. (gw 2.5-6m)	3cm d.s. in pool	8	14		
1959			3	1.9	96		1.2-3.3m fill, SC (%fines=20); alluv./colluv, ML/CL (gw 6.4m)	lat, settlement cr. top of slope				
Woodland Hills												
1962				1.6	75,92	11,19	2.3-3.3m fill, SC.; colluv. to 4.5m, SW; Modelo br. (gw 4.3m)	1cm ftng-slab sep.; 0.5cm tilting backyard pool				
1957	1-1.5:1	3-3.6		1.7	77-87		>3m fill, clay (no gw)	up to 15cm H pav't cr., 7.5cm vertical				
1982	2:1	11	7	1.4			1-3m fill, clay; Modelo br.	cr. in garage slab				
N/A	flat	1.5		1.7		14-28	1.2-2.1m fill (PI=37); alluvium		9	7.6	5	3
1960	1.5:1	3.6-4.5	3	1.8	83-91		>3m fill	2cm cr. in slab	13	27	8	9.1
N/A	2-3:1		0	1.8		14	0-1m fill, SC/CL; 0.6m SC/CL; Modelo br (no gw)	cr. walls	8	14	3.8	4.6
N/A	4.5 m wall		0			11	0-4.5m fill, SM; 3' ML; Modelo br. (no gw)	5cm d.s. in house	NA			
1964	2:1	1.8	6	1.8	75-83		1.8m fill, cl. silt; res. soil, sandy silty cl.		4	3.7		
1962	1.5:1	3-6	3	1.7		8,14	0.6-4.9m fill, SC/ML; Modelo siltstone br. (no gw)	0.5cm cr.	6	14	3.8	3
N/A				1.4		3	1.3-2.7m fill, ML; Modelo siltstone br. (no gw)	0.5cm cr.	8	6.1		
1957	1.5:1			1.5-1.8		24	0-6m fill; rock		8	4.6		
Calabasas												
<1975	1.5:1	7.5+					4.5-6m fill; br.	cr. through top of slope				
Canoga Park/Northridge												
N/A	flat					3-8	2m fill (%fines=20-40); loose alluv (gw 3.9-5.4m)	surface settlement, attributed to liq.	13	6.1		
N/A		1.2-2.4				30-40	0-1.5m fill, CL (PI=23); CL and SC (no gw)		6	6.1	2.5	3
Northern Area												
N/A							fill; Chatsworth br (ss & shale)	S. Sus. Pass Rd. soil slippage near top of slope				
1963	2:1	3		1.9	84-90		2.4-4.5m fill, SM/SC; alluv. SC/CL. (gw 0.6-4.2m)	5cm pav't sep.; 20cm d.s. cr. in ftng.				
1960	flat			1.9			0-1.5m fill SM; alluv., SM/SC to 20-21m (gw 4.8-13m)	0.5-2cm lat . grnd. cr., no vert.				
1963	1.5:1	26	9	2			0.3-8.1m fill SM/SC; 1m' residual SM; Saugus br. (no gw)	c/f distress, 10-15cm cr (H&V) in house, 13cm V in garage	10	4.6		
1987	2:1		18	2.1		35	up to 20m fill (PI=19); Saugus br. (no gw)	3cm tilting of pool; deck cr.				
1981	2:1	21	3	2.1			0.6-7.2m fill, SM (%fines=40); Saugus br. (no gw)	5cm d.s. in fill, distress @ c/f line				
1988	2:1	21-24	7.5				>15m fill	c/f distress; incr. ext. w/ fill depth, small settlement	1.5	9.1		
CENTRAL LOS ANGELES[A]												
1954	1.5:1	24	1.5	1.8			4.5m fill, SC; Topanga br. (no gw)	distressed res.; cr. 3m from t/s; 5cm settlement nr. slope	3.5	15	3	7.6
<1969		2.5-3		1.9		13-24	<2.7m fill, CL (PI=35); SC/CL.; clystn br. (no gw)	no grnd cr.; str. damage	5	2.4		

APPENDIX. CONTINUED

1960		9		2		18-28	<6.4m fill, SM; conglomerate br. (no gw)	0.5cm cr.	11	>15	3.8	3
SANTA CLARITA[A]												
Valencia, South of McBean												
<1971	1.5:1	10.5	6	2.1	90-95		1.5-4.3m fill, CL; 1.8-2.7m native SM/ML; dense sand (no gw)	damage at c/f; settlement, tilting in fill				
N/A							1.5-9m "potentially compressible mat"	"differential mov't"	10	9.1		
1968	1.5:1	9	7	1.7-2.0	75-89		0.6-4.5m fill, SM; alluv., SW gr., cobbles (no gw)	d.s. lat. mov't in fill; 5-8cm ftg./slab cr.				
1968	2:1	13.5	6				0-8m fill; 1.8-2.7m alluv.; Saugus br. (no gw)	bldg damaged, 25cm d.s. cr. in yard	4	9.1		
1971	1.5:1	6	8	1.9	80		1.5-6.4m fill, SW, cobbles; 1.2m topsoil, SC; Terrace dep. (no gw)	10-30cm wide cr. in fill slope nr. house, 0.5-2.5cm cr. in slab				
1968				1.8-1.9	85		0-1.2m fill, sandy w/ gr.; sand w/ gr. and cobbles (no gw to 1.8m)	broken ftng.; 13cm cr., 2.5cm pav't sep., 2cm pav't cr.				
1969				1.9	85	<5	4.3-6m fill, SM (PI=8); Terrace dep. (no gw)	2.5cm slab cr., leaning walls				
1968		10.5					1.8-6.7m fill, SM; 1.7m alluv., SM/SC (no gw)	house destroyed, no damage details				
1986	2:1	7.5	18	2.1	93-100	30	5.8m fill, SC; 1,8 alluv., SC; Terrace dep., sand w/ gr. (gw 4.5m)	no c/f line @ res; cr. ftng; 2.5cm patio-house slab sep.	9	15		
1987				1.4, 1.9			fill over Terrace deposits (no gw)	2.5-10 cm cr., mostly in fill				
Valencia, North of McBean												
1988				2.1	82-100		0.6-13m fill, sand, gr.; Terrace dep, SC/GC (gw 3m)	lat. mov't in fill, slab cr.	6	15	5	6
1987			12	1.8			1.8-4.3m fill, SM, gr; Terrace dep., GW (no gw)	cr. floor slab; damage @ c/f; d.s. in fill				
1987	2:1	9-11	6	2.1	88-93		9-18m fill, sand w/ gr. (no gw to 3m)	1cm lat. cr offset	10	13	7	8
1987				2.1-2.3	93-97		0-3.4m fill, SM; coarse sand and gr. (no gw)	2.5-6cm slab, ftng. cr @ c/f & in fill	8	13	5	4.6
1977	flat			1.8-2.0			1.2m fill, SM; native SC, (no gw to 3m)	<6cm cr. @ c/f line				
1987				2.1	96	20,34	up to 10m fill, SC; Saugus br. (no gw)		6	11		
1989	2:1	9.6	3	2.1	90	>35	2.1-6.7m fill, SM; Terrace dep. (no gw)	lat. mov't, 5-10cm cr. in fill, 7.5cm cr. house, cr. masonry, etc.	18	20	14	11
N/A	2:1		15	1.9-2.1		<5	1.2-6.4m fill, sands/cobbles; Terrace dep. (no gw)	cr. in pool lining				
N/A	2:1	2.4-3		2.1	93		1.8-3m fill, SM; (no gw)	2.5-10cm cr. @ c/f line	4	15	2	3
1978	1.5:1	2.1		2			2.1-3m fill, SM; alluv., SC and CL (no gw)	pav't sep. and cr.	9	12		
Santa Clara River Area												
1979	flat			2.1	92-100	50	2.4m fill, SC (%fines=20-30); loose alluv. (gw 3m)	liquefaction, lat. mov't, 9cm d.s.				
1984	flat			1.9-2.1	73-82	15	7.5m fill, SM/SC; 3-4.5m alluv., sand, Saugus br. (no gw)	inc. settlement w/ fill depth	8	12	3	2.4
1984			9	1.8-1.9			0.6-2.1m fill, sandy; 1.5-5 alluv, CL; Saugus br. (no gw)	fill mov't H, V	5.5	6.1		
1986	2:1	3.6					0.3-1.8m fill; Saugus br. (no gw)	fill mov't H, V; <1cm cr. at c/f	9	11	6	5.5
Newhall												
1984	1.5:1	15	9	1.9	82		0.9-3.4m fill, SC; Saugus br. (no gw)	slab cr., leaning ret. wall	6	9.1	3.8	2.4
N/A							0.6-0.9m fill, SM; Saugus br. (no gw)	tilted ret. wall, slabs				
1983-1986				2.1	87	16	0-10.7m fill, SM; Saugus br. (no gw)		1.2	12	2.5	6.1
1959	1.5:1	5.4	6	2	83-86	30-50	7.6-11.3m fill, SM/SC; 4.0-4.9m alluv., sand; Saugus br. (no gw)	1cm pav't cr., 2.5cm pav't sep.	7	9.1		
Canyon Country												
N/A								<13cm floor slab, grnd settlement	3.5	7.6	3	2.4
Near H14												
1965	nat. rel. 14 m			1.9	83-92	>40	2.1-16m fill; Mint Cny br. (gw base of fill)		6	6.7		
1965	nat. rel. 20 m			2.0-2.1	90	18-35	0.9-8m fill, silt; alluv, colluv, SM; (gw 8m)		0			
West of I5												
1985	2:1	>12					2.7-5.1m fill, sandy; Saugus br. (no gw)	distress @ c/f; 4.5cm lat. mov't				
1985	2:1	>12					3-7.5m fill, SM; siltstone/sandstone br.	1cm V and H cr; damage @ c/f				
N/A	1.5:1	3-4.5					0.9-7.5m fill; Saugus ss/sandstone	fill, slope mov't, damaged bldgs				
N/A							14-30m fill; Saugus ss/siltstone					
1986	1.5-2:1	7.5	3	2.1	80-85	40	0-11m fill, SM/ML; Saugus sandstone br. (no gw)	1cm cr. in slab, ftng.	4	9.8		
SIMI VALLEY[A]												
	flat			1.9			0.6-1.8m fill, SM/ML; alluv. to 15m, SC/CL.; Terrace dep, SM (no gw)	2cm cr., chimney damage	8	12	5	5.5
	flat			2	90	28-48[M]	4.6-4.9m fill, SC/CL; native CL (no gw)	residence destroyed, no details				
1989	2:1			2.1	90	25-30	9.8-13.1m fill, SM/SC (%fines=30, PI=15); br (gw 8m)		5	12		
1965			4.5	1.8	70		4.3m fill, CL/SC; 2.4m alluv., CL/SC; rock (gw 2.7m)	red-tagged bldg.	23	14		

[A] Detailed site locations withheld to preserve client confidentiality

[B] Compaction relative to Modified Proctor maximum dry density (ASTM D1557), measured following 1994 earthquake

[C] Standard Penetration Test blow count, unless denoted by [M] which indicates 2.5-inch outslide diameter Modified California sampler

[D] Soil classified by Unified Soil Classification System. Also cl = clayey, gr = gravel, gw = groundwater, alluv = alluvium, br = bedrock

[E] d.s. = differential settlement, H = horizontal, V = vertical, c/f = cut/fill contact, cr. = crack, t/s = top of slope

[F] Established by manometer surveys

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