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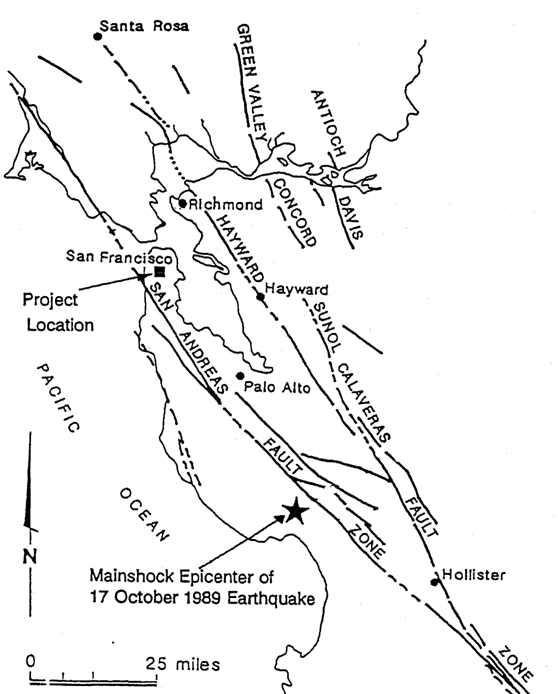
SYNOPSIS

Ground shaking during the Loma Prieta earthquake of 17 October 1989 caused permanent lateral and vertical displacements of a steep hillside in San Francisco, California. These displacements damaged 36 residences along the top of the hillside. Subsequent exploration and analyses indicated the hillside is composed of loose to medium-dense Dune sand that is marginally stable under static conditions, but highly susceptible to movements during ground shaking caused by earthquakes. A retaining system consisting of a combination of drilled soldier piles and permanent tiebacks was designed and constructed to mitigate the potential for future seismically induced slope movement. This paper describes the evaluation of the seismic stability of the slope and the design of the retaining system.

INTRODUCTION

Thousands of structures throughout northern California were damaged during the Loma Prieta earthquake of 17 October 1989 (Richter magnitude 7.1). Among the damaged structures were 36 homes on two city blocks along Eighth Avenue between Moraga and Ortega streets in the Upper Sunset District of San Francisco, California (Figure 1).



SITE VICINITY AND FAULT MAP
FIGURE 1

The affected homes are situated on the eastern side of Eighth Avenue at the top of a 90- to 110-foot-high hillside that slopes down to Seventh Avenue at an inclination of about 1.4:1 (horizontal to vertical) to 1.6:1 (35 to 32 degrees), (Figure 2). Ground shaking during the Loma Prieta earthquake caused permanent lateral and vertical displacements of the hillside. These displacements caused the sidewalks, floor slabs and foundations of most residences along the top of the hillside to crack and/or tilt. Many damaged residences were declared unsafe for occupancy by the City and County of San Francisco, Department of Public Works (DPW).

Treadwell & Rollo, Inc. was retained by the DPW to evaluate the stability of the hillside and to provide recommendations for long-term stabilization of the slope. This paper focuses on analyzing stability and developing design parameters for the selected stabilization system.

SITE HISTORY

The site lies on the leeward side of Sand Hill in a wind-protected environment (Figure 3). Dune sand transported from beach sources by prevailing westerly winds accumulated in this area during the Holocene period to a depth of more than 100 feet. Dune sand is typically clean, well-sorted, subround to subangular, fine-grained sand. The sand grains are predominantly quartz and feldspar.

City records indicate the 1700 and 1800 blocks of Eighth Avenue were graded and paved between 1913 and 1917. Excess Dune

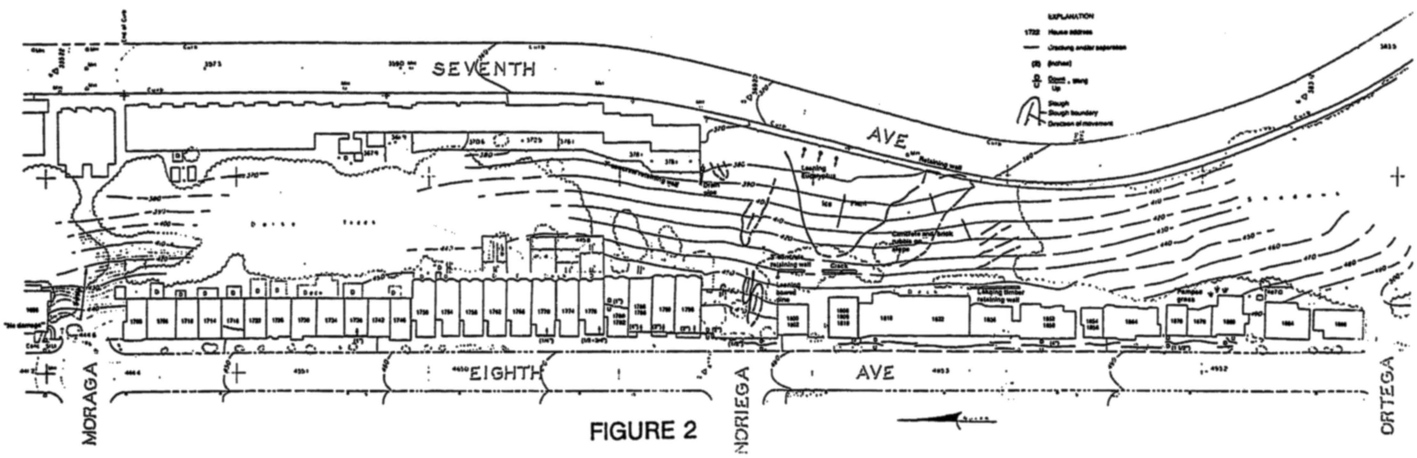


FIGURE 2

TOPOGRAPHIC MAP OF SAN FRANCISCO

PREPARED BY THE SAN FRANCISCO DEPARTMENT OF CITY PLANNING · 1967

FIGURE 3

sand from the grading was placed on the slope that existed on the eastern side of Eighth Avenue. The residences on the 1700 block were constructed in the 1920s and 1930s. The homes were generally constructed on level lots with the top of the slope located immediately behind the residences. The residences on the 1800 block were constructed over a long period of time with one residence reportedly constructed prior to 1906. Most of these structures were built on level pads cut into the hillside or built on the slope itself. Many homes built prior to code enforcement share common foundations. The foundations supporting the structures on both blocks consist of shallow, continuous and individual concrete footings.

Since their construction, several residences had experienced movements, apparently due to slow downslope movement (creep) of the soil on the steep hillside behind the homes. The foundations of some homes were underpinned to mitigate these movements. In the 1957 Daly City earthquake (Richter magnitude 5.3), homes on the 1700 block reportedly moved up to six inches.

EARTHQUAKE-INDUCED DAMAGE

The 17 October 1989 earthquake occurred when a segment of the San Andreas fault northeast of Santa Cruz ruptured over a length of approximately 28 miles. The earthquake was assigned a surface wave magnitude of 7.1 by the U.S. Geological Survey. The fault rupture was bilateral (in two directions), resulting in only about 8 to 10 seconds of strong ground shaking. Its duration was about one half the duration normally associated with an event of this magnitude. The epicenter of the earthquake was approximately 60 miles southeast of the Eighth Avenue site. Accelerometers in San Francisco typically recorded peak ground accelerations of between 0.1 and 0.2 times gravity (0.1g - 0.2g) during this earthquake. An accelerometer near the Eighth Avenue site recorded a peak bedrock acceleration of 0.1g.

Earthquake-induced damage to the residences on Eighth Avenue was manifested differently on the 1700 and 1800 blocks. On the 1700 block, much of the damage occurred from about the midpoint to rear of the homes. The lateral and vertical soil movement caused the rear foundation to rotate and settle and the concrete slab-on-grade floor and footings along the sides of the residence to crack and tilt. Water-level surveys indicated differential settlements of up to 14 inches across an individual structure.

At the northern end of the 1800 block, the ground movement typically involved entire structures, as indicated by the sidewalk cracking and settling in front of the residences. Based on observation of the sidewalk damage, it is estimated that the structures shifted laterally as much as six inches and settled up to four inches. Interior cracking and water-level survey indicated differential movement had occurred within the structures. The distress was generally minimal at the southern end of the 1800 block.

A surface geologic reconnaissance of the hillside was performed shortly after the earthquake (Figure 2); however, much of the slope was obscured by heavy vegetation. Observed features included several subtle breaks and troughs in the slope, indicating shallow slumping. There were also tilted walls, utility poles, and trees on the slope. Three large eucalyptus at the base of the slope were tilted back toward the slope. Several old concrete retaining walls at the base of the slope had shifted laterally and cracked, with sand apparently overtopping some of these walls during the earthquake.

SUBSURFACE CONDITIONS

Two subsurface exploration programs were performed at the site. The first program was performed by Harlan Tait Associates (HTA) for the Eighth Avenue homeowners. The program consisted of drilling six borings in the sidewalk in front of the residences with a truck-mounted, rotary-wash drill rig and seven borings at the rear of the residences with portable drilling equipment. The borings on the sidewalk were drilled up to a maximum depth of 119 feet, while the maximum depth of the borings at the rear of the residences was limited to 40 feet because of the type of drilling equipment used.

At the front of the residences, the HTA borings generally encountered 5 to 10 feet of loose to medium-dense sand fill with small amounts of rubble. The fill is underlain by medium-dense, natural Dune sand. The sand becomes denser with depth and is generally very dense at depths of 40 to 50 feet. The Dune sand extends to the maximum depth explored (119 feet). Groundwater was measured at a depth of 65 feet in a boring drilled in front of the residences.

The borings drilled behind the residences encountered very loose to loose sand fill to depths ranging from 5 to 15 feet. Below the fill, loose to medium-dense Dune sand extended to the maximum depth explored. Groundwater was not encountered in the borings drilled at the rear of the residences.

Treadwell & Rollo performed a second exploration program. The focus of this program was to estimate the thickness of loose to medium-dense material behind the residences. Use of truck-mounted equipment was not possible because of limited access. Consequently, it was decided to perform a seismic refraction survey using small explosives to produce seismic energy. The survey was performed by NORCAL Geophysical Consultants and consisted of three seismic refraction lines at the locations shown on Figure 4.

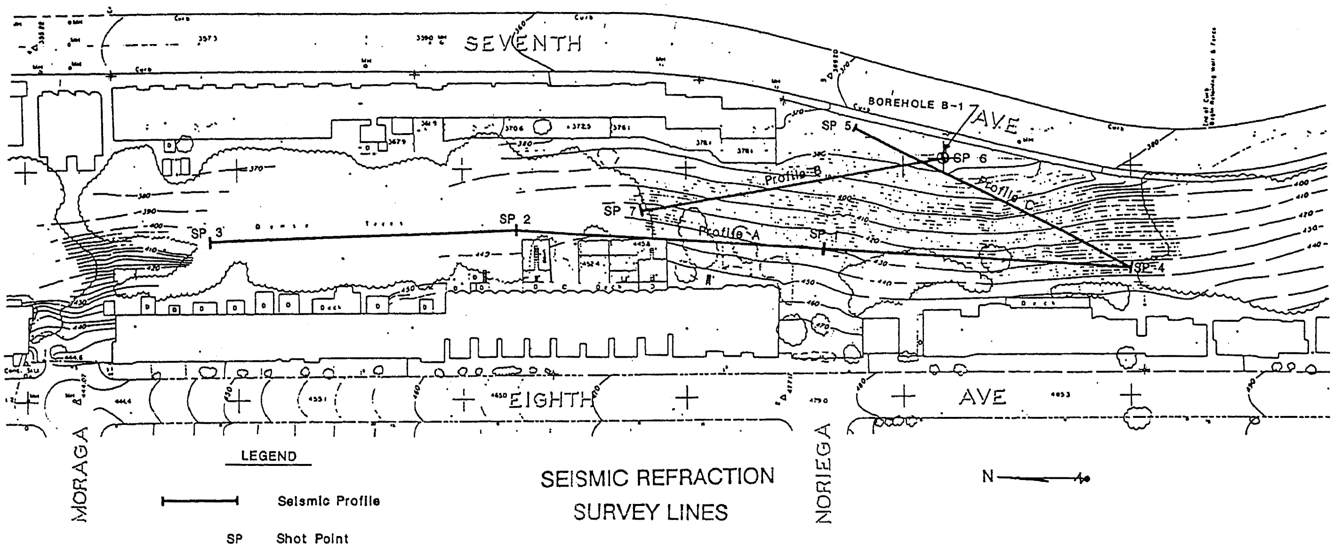


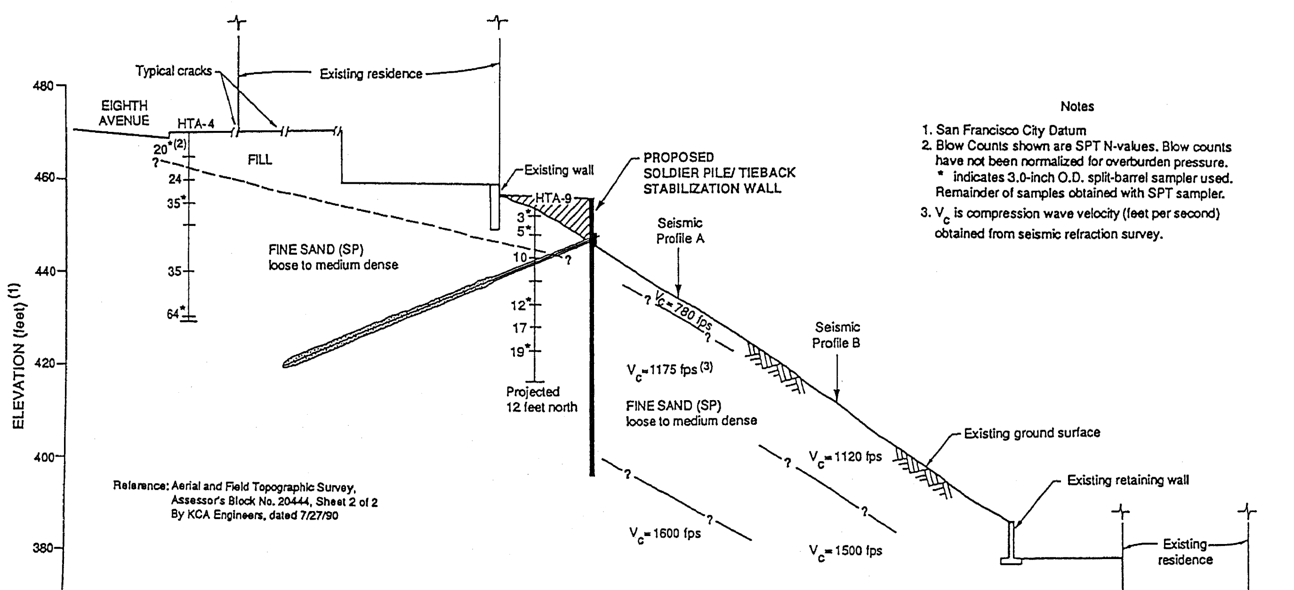
FIGURE 4

A test boring was drilled at the base of the slope to correlate soil type and density with compression wave velocities. This boring encountered bedrock of the Franciscan formation at a depth of 57 feet below the ground surface.

The seismic refraction survey indicated the compression wave velocities in the sand at midslope varied from 780 feet per second (fps) at shallow depths to 1780 fps at depth. On the basis of this survey, it was concluded that the loose to medium-dense Dune sand extends to a depth of at least 75 feet. The relative density of the sand is estimated to range from 30 to 60 percent based on analysis of the geophysical data. Typical subsurface profiles (perpendicular to the slope) for the 1700 and 1800 blocks are shown on Figures 5 and 6, respectively.

Laboratory testing on the Dune sand consisted of grain-size analyses and direct shear and triaxial shear strength tests on remolded samples. As discussed previously, the Dune sand is a uniform, fine-grained sand. Grain-size analyses indicate the mean particle size (D_{50}) is typically between 0.2 and 0.25 mm, the uniformity coefficient (C_u) averages 1.7, and the fines content (particles passing the No. 200 sieve) is generally less than five percent.

Direct and triaxial shear strength tests on remolded samples indicate the effective friction angle varies from approximately 31 degrees for a relative density of 20 percent to 40 degrees for a relative density of 80 percent.



TYPICAL SUBSURFACE PROFILE - 1700 BLOCK

FIGURE 5

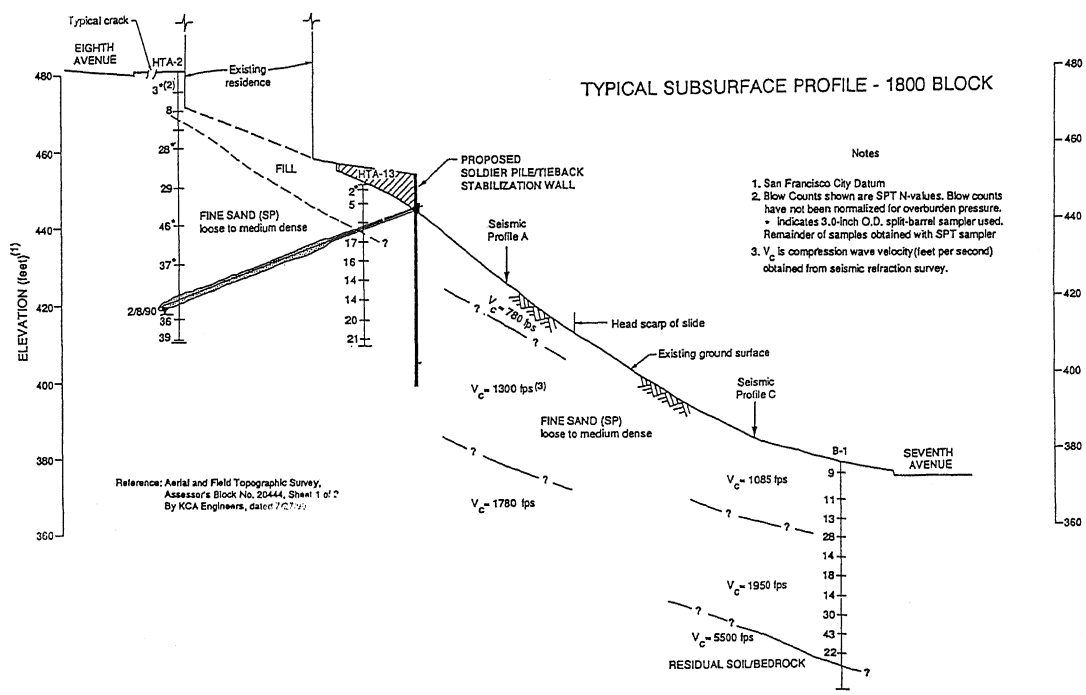


FIGURE 6

SLOPE STABILITY ANALYSES

Methodology

For slopes composed of cohesionless sand, the critical failure mechanism under static conditions is usually surface ravelling or shallow sliding. This failure mechanism can be analyzed using the simple infinite slope analysis. In this analysis, the slip surface is assumed to be a plane parallel to the ground surface and the end effects are neglected. For static conditions and in the absence of groundwater, the factor of safety (F.S.) against sliding is calculated using the formula:

$$F.S. = \frac{\tan\phi}{\tan\theta}$$

where ϕ is the friction angle of the sand and θ is the inclination of the slope in degrees.

For slopes composed of soil that neither builds up large pore pressures during earthquake shaking nor undergoes significant strength loss, the seismic stability is generally evaluated using the pseudostatic method. For this method, the effects of an earthquake on a potential slide mass are represented by an equivalent static horizontal force determined as the product of the seismic coefficient k, which is some fraction of gravity, and the weight of the potential slide mass. This method assumes the sliding mass behaves as a rigid body. In practice, it is commonly assumed that the seismic coefficient is a fraction (generally 1/3 to 2/3) of the estimated peak ground acceleration at the site. Amplification of the ground acceleration by the slope is generally not considered. In his 1979 Rankine lecture, H. Bolton Seed recommended using the following design criteria for embankment slopes:

> Design criteria: For embankments constructed of soils which do not build up large pore pressures due to earthquake shaking nor show more than 15% strength loss (usually cohesive soil such as clay, silty clay, sandy clay or very dense cohesionless soil), based on acceptable deformations due to earthquake shaking and crest acceleration less than 0.75g.

Earthquake magnitude	Design criteria
6-1/2	FS = 1.15 for seismic coefficient = 0.1
8-1/4	FS = 1.15 for seismic coefficient = 0.15

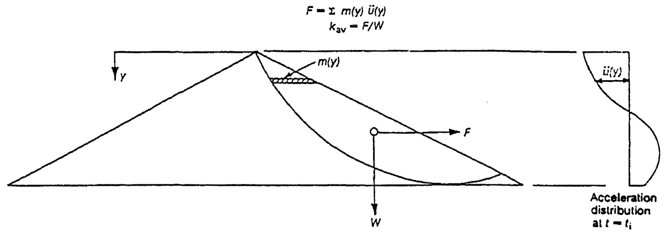
For granular, free-draining material with a plane sliding surface, the critical value of the seismic coefficient, k_c, which will reduce a given factor of safety for a stable static condition (FS_o) to a factor of safety of 1.0 with an earthquake loading, can be determined by the formulas:

$$k_c = (FS_o - 1)\sin\theta \quad \text{(for } k_c \text{ parallel to slope)}$$

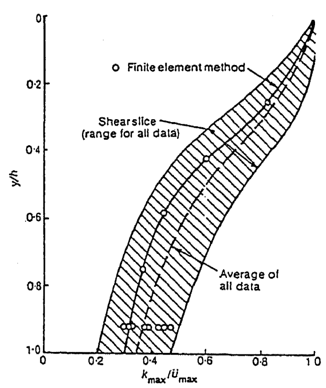
$$k_c = (FS_o - 1)\tan\theta \quad \text{(for } k_c \text{ horizontal)}$$

Therefore, for a 2:1 (26.6 degrees) slope with a minimum factor of safety of 1.5 against sliding, the critical seismic coefficient is about 0.22 to 0.25, depending on the orientation of the seismic force.

For embankment slopes, simplified methods have been developed for computing displacements from different levels of earthquake shaking (Makdisi and Seed, 1978; Hynes-Griffin and Franklin, 1984). For these methods, which are based on Newmark's displacement-type analyses, the critical (or yield) acceleration is defined as the acceleration that will reduce the factor of safety against sliding of a potential slide mass to unity. The estimated acceleration imposed on the potential slide mass from a particular earthquake is then estimated from graphs that take into account the variation of acceleration over the height of the embankment (Figure 7).



Determination of effective acceleration for potential slide mass

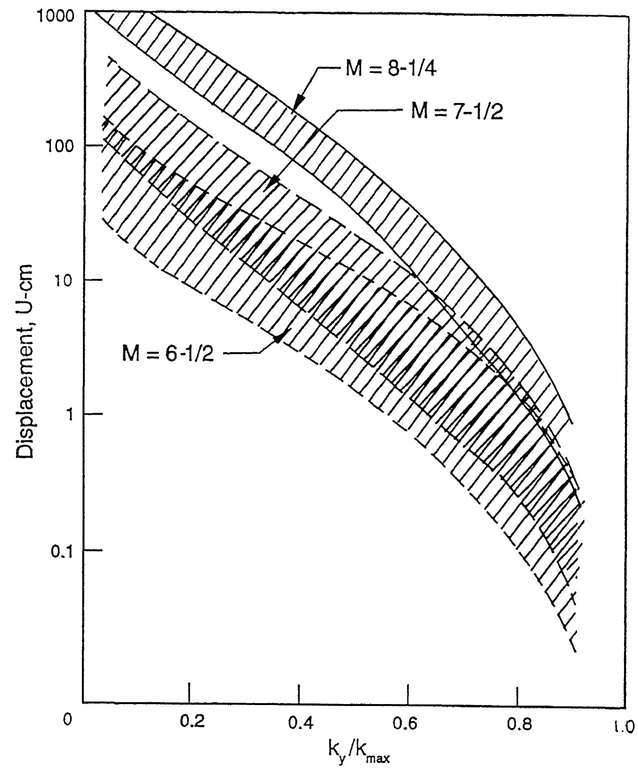


Variation of effective peak acceleration, k_{max}, with depth of base of potential slide mass (after Makdisi and Seed)

FIGURE 7

Deformations are then estimated based on the ratio of the critical acceleration to the estimated acceleration of the slide mass for a particular earthquake (Figure 8). On the basis of their analyses, Hines-Griffith and Franklin concluded that permanent displacements for deep-seated sliding surfaces should be limited to less than 1 meter (39 inches) if the ratio of critical acceleration to peak bedrock acceleration is at least 0.5.

The primary problem with using the above methods to estimate slope deformation is that they were developed for earth dams. It is not clear whether the variation of peak acceleration over the height of a dam would be similar to the variation over the height



VARIATION OF PERMANENT DISPLACEMENT WITH YIELD ACCELERATION (AFTER MAKDISI AND SEED)

FIGURE 8

of a slope which is part of a larger hill or mountain. Further, to use these methods, it is necessary to estimate the peak acceleration at the top of the slope. It is known that slopes tend to amplify ground motions; however, little has been published regarding methods to estimate this amplification. In the example given at the end of the Makdisi-Seed paper, finite element analysis is used for estimating the maximum acceleration at the crest of the dam. For the Hynes-Griffin and Franklin method, curves were developed which indicate that the amplification of the peak bedrock acceleration varies from about 2.5 at the base of the dam to about 3.5 at the crest.

Because the homes are present at the top of the Eighth Avenue slope, it was necessary to estimate deformations that might occur during an earthquake after the slope was stabilized. Therefore, despite the possible shortcomings of the available simplified methods for analyzing slope movement, the Makdisi-Seed method was used, with some judgements applied regarding peak ground accelerations.

Back Analysis of Failure

It was not possible to determine the maximum depth of sliding by examining soil samples obtained from test borings because of the cohesionless nature of Dune sand. Further, readings from several slope inclinometers installed and monitored

behind the residences after the earthquake indicated no ongoing movement of the slope. Therefore, the depth of the sliding surface(s) was not available for the back analysis of the slope failure.
Based on the results of the direct shear tests and taking into account the plane strain conditions of the slope, it is estimated the effective friction angle of the Dune sand comprising the slope varies from about 32 degrees in the loose surficial fill to about 36 degrees in the native sand. Considering that slope inclination (where vegetation was scarce or absent) varied from about 32 to 35 degrees and that creep of the slope was occurring prior to the earthquake, it is obvious the static factor of safety against sliding was near 1.0. Therefore, the slope was susceptible to sliding during even slight ground shaking.

On the basis of the cracks and damage observed at the top of the slope, it appeared the slope failed in a series of wedges, with the deepest wedge corresponding to a factor of safety of 1.0 for a given pseudostatic coefficient.

The stability of a series of wedges was analyzed using the computer program TSLOPE (distributed by TAGA Engineering), which uses Spencer's Method to model noncircular slip surfaces. This analysis indicated the depth (measured from the top of the slope) to the critical sliding surface for a peak ground acceleration of 0.1g is about 15 to 20 feet. For a wedge 5 to 10 feet deep, the critical acceleration (for a factor of safety of 1.0) is about 0.05g. Using the Seed-Makdisi graph, the estimated deformation of a 5- to 10-foot-deep wedge is on the order of 5 to 10 centimeters (2 to 4 inches) for a peak ground acceleration of 0.1g and a magnitude 7 earthquake (a peak bedrock acceleration of 0.1g was measured in the site vicinity). This computed deformation is somewhat lower than the maximum horizontal displacements observed (roughly 6 inches) and considerably lower than the maximum vertical displacements observed (roughly 14 inches). Therefore, it is believed that the peak ground acceleration was greater than 0.1 g due to amplification of the bedrock motions by the loose to medium-dense Dune sand and/or amplification of the ground motion by the slope.

Predicted Slope Performance during Future Seismic Events

Since the 17 October 1989 earthquake, the U.S. Geological Survey (USGS) has indicated that there is a relatively high probability (approximately 65 percent) of an earthquake with a Richter magnitude of 7 or greater occurring in the Bay Area in the next 30 years. The probability of a magnitude 7 or greater earthquake occurring on the northern extension of the San Andreas fault, which is about 5 miles southwest of the Eighth Avenue site, is approximately 20 percent. The maximum credible earthquake for the San Francisco area is an event similar to the great San Francisco earthquake of 18 April 1906, which had an estimated magnitude of 8.3. The USGS estimates the probability of magnitude 8 earthquake (recurrence interval of about 300 years) to be less than 10 percent over the next 30 years.

Considering that a major earthquake is probable in the San Francisco Bay Area in the next 30 years and allowing for some amplification of ground motions by the slope, a peak ground acceleration (PGA) of 0.45g was selected for design. This PGA corresponds roughly to a magnitude 7-1/2 earthquake on the northern extension of the San Andreas fault.

We performed a simplified deformation analysis for a series of wedges using a PGA of 0.45g. Our analysis indicated that a wedge at a depth of about 10 feet would move about 100 centimeters (39 inches). This movement would clearly cause severe damage and perhaps collapse of the residences.

SLOPE STABILIZATION

To help reduce the homeowners' repair costs, the Federal Emergency Management Agency agreed to provide funding for constructing a system that would stabilize the hillside below the homes. Only a few stabilization schemes were considered to be technically feasible because of the difficult access to the top of the slope. Four alternatives were considered: 1) a grouted earth buttress, using either jet or chemical grouting techniques, 2) slope reinforcement with large grade beams and soil anchors, 3) a series of soil-nailed walls with nails installed by jet grouting techniques, and 4) a soldier pile/tieback wall.

Because of the required width and depth of a grouted buttress, it was apparent that this solution would not be economical. Cost analyses also indicated that alternatives 2 and 3 were not economical, primarily because the grade beams and soil-nailed walls would have to cover most of the slope to be effective. It was therefore concluded that a wall consisting of a combination of drilled soldier piles and tiebacks would be the most appropriate stabilization system. It was decided to construct the wall about 20 feet behind the residences to limit the height of the above-grade portion of the wall but still give the homeowners some backyard area and the contractor some working room. The backfill between the wall and the residences was an important aspect of the design because it provides lateral confinement for the foundations of the homes. In choosing this system, it was recognized that the slope below the wall would still be unstable during earthquake loading.

Selection of Critical Slip Surface

In designing the soldier pile/tieback retaining system, it was necessary to estimate the maximum depth of the critical slip surface for the design PGA. To estimate this depth, slope stability analyses were performed on a series of wedges using the pseudostatic method to simulate earthquake loading. The seismic coefficient for each wedge was a fraction of the PGA and varied from 0.95PGA (.43) at a depth of 10 feet to 0.6PGA (0.27) at a depth of 50 feet. This fraction was determined using the variation of peak ground acceleration over the height of an embankment given in the Makdisi-Seed paper. The base of the potential slide mass was conservatively taken as the point at which the wedge slip surface intercepted the proposed wall (the Makdisi-Seed method assumes a circular slip surface).

The slope stability analysis indicated that the critical slip surface could be as deep as 40 to 50 feet for the design PGA. Designing the retaining structure for this depth of sliding would have required several rows of tiebacks and extensive hillside excavation. It was therefore decided to look at the potential displacements along the slip surfaces using the Makdisi-Seed charts. These charts indicated that movement on a slip surface at a depth of 40 feet would be less than 3 centimeters (1.2 inches), movement at a depth of 30 feet would be about 10 centimeters (4 inches), and movement at a depth of 20 feet would be about 30 centimeters (12 inches). These deformation estimates ignore the reinforcing effect of the soldier piles on the slip plane.

Because of the relatively large deformation at a depth of 20 feet, it was decided that this would be the location of the "critical" slip surface for designing the structural members, including tiebacks. To limit displacements on deeper slip surfaces, the soldier piles were extended about 5 to 10 feet below the deepest slip surface for the design PGA. This resulted in soldier piles that extended 45 to 55 feet below the ground surface. Potential displacements along slip surfaces deeper than 20 feet are computed to be less than a few inches for the design PGA.

Design Wall Pressures

The pressures used for designing the retaining structure are shown on Figure 9. The pressure for static conditions was computed using the following formula given in the Navy Design Manual 7.02 (1986):

$$p = 0.5k_o\gamma H$$

where p = uniform wall pressure in psf
k_o = 0.5 for loose sand
γ = moist unit weight of soil
H = retaining wall height

The lateral force increase on tied-back walls during an earthquake is typically computed using the Mononobe-Okabe equation. This equation assumes active conditions exist behind such walls. The pressure distribution is generally assumed to be uniform and the equivalent uniform ground acceleration is taken to be two thirds of the peak ground acceleration. For an equivalent ground acceleration of 0.3g (two-thirds of 0.45g), the computed dynamic pressure increment would be about 15H. Studies have shown that for rigid walls, the seismic-induced pressure may be 2 to 3 times that estimated using the Mononobe-Okabe equation.

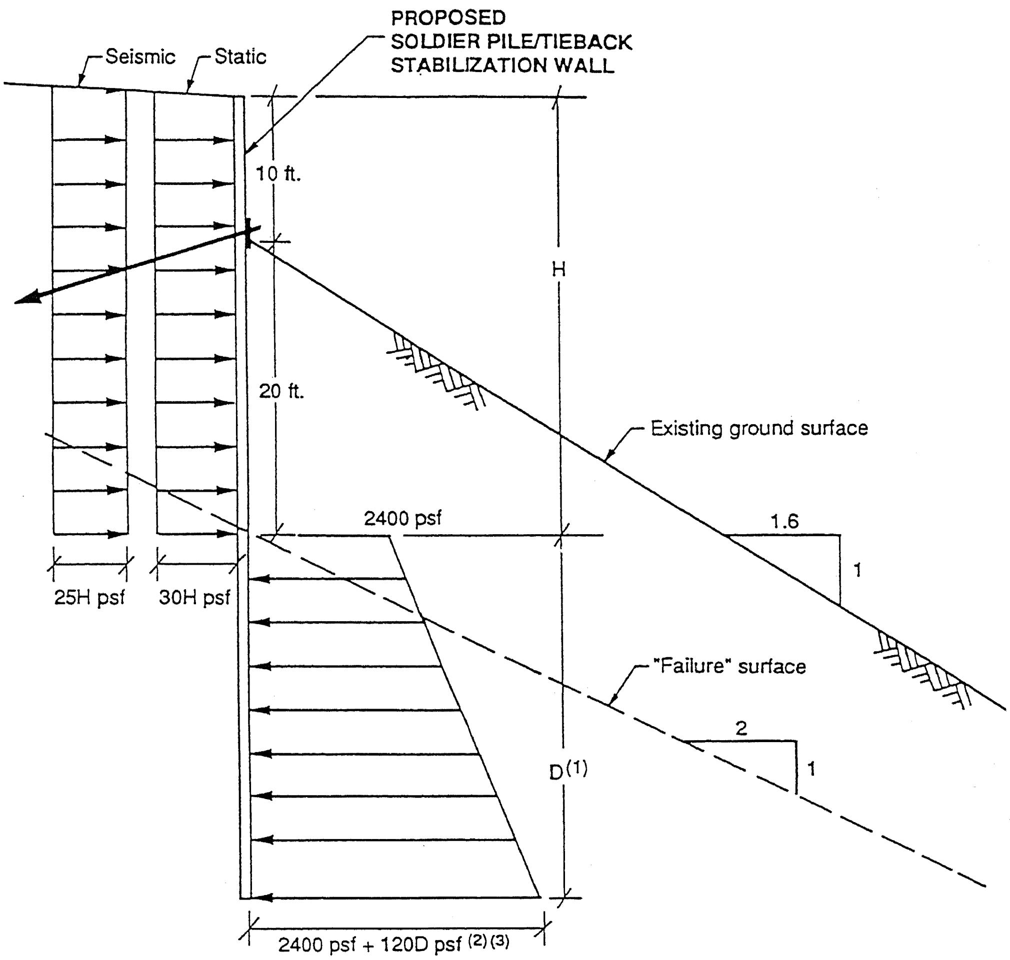
Another important consideration in establishing the design seismic pressure for a permanent wall is the residual pressure that remains on the wall following a seismic event. This pressure can be a substantial portion of the pressure developed during the earthquake. The design seismic pressure was increased from 15H to 25H to account for the possibility that the subject retaining structure could experience several seismic events over its life.

The above forces imposed on the wall are resisted by tiebacks, as well as by passive pressure acting on the faces of the soldier piles. Computing the available passive pressure has to take into account several factors, including sloping ground on the downhill side of the wall, spacing between the soldier piles, and the effect of ground shaking on the passive failure wedge. To account for potential downslope movement of the hillside below the wall, passive resistance was ignored for the upper 20 feet of soil, although the contribution of the weight of this soil was included. The passive pressure was also reduced to take into account: 1) overlapping passive failure wedges because of close spacing (four feet) of the soldier piles, 2) sloping ground below the wall, which significantly reduces the size of the passive failure wedge, and 3) the thrust of the passive wedge away from the wall during ground shaking. The resulting equivalent fluid weight for computing passive pressure is 120 pounds per cubic foot (pcf), which is about 25 percent of the computed pressure for level ground conditions.

Tieback and Soldier Pile Design

Structural design of the retaining system was performed by SOH & Associates of San Francisco. The retaining system consists of steel soldier piles spaced at four feet, center to center. Each soldier pile is installed in a two-foot-diameter hole and backfilled with concrete. To prevent ravelling sand between soldier piles, drilled piers filled with lean concrete are located between each soldier pile. These piers extend ten feet below the adjacent hillside grade. The upper 14 feet of each soldier pile is coated with a bituminous protective coating to prevent corrosion. The lower portion of the soldier piles is coated with standard rust-inhibitive primer.

One row of tiebacks is located about ten feet below the top of the wall to resist lateral earth forces. Tieback design criteria included extending the tiebacks behind the deepest potential slip surface. This resulted in a tieback free length of 45 feet. Further, because of the close spacing of the soldier piles, the tieback angle was varied from 20 to 25 degrees at



Notes: 1. Minimum embedment, D, varies from 25 to 35 feet.
2. Passive pressure does not include a factor of safety.
3. Passive pressure may be assumed to act over twice the soldier pile width.

LATERAL EARTH PRESSURES FOR WALL DESIGN

FIGURE 9

every other soldier pile to limit the potential for overlapping tieback failure zones. The design tieback loads are 88 kips for static conditions and 161 kips for static plus with double corrosion protection.

CONSTRUCTION

Malcolm Drilling Company of South San Francisco constructed the 1,300-foot-long wall at a cost of approximately $4,800,000. Construction took about eight months and was completed in December 1991. Construction access to the hillside was provided by installing a sidehill fill with a temporary soldier pile bulkhead at the base of the fill. Shafts for the soldier piles were drilled in the loose to medium Dune sand with the aid of drilling fluid (Supergel). Concrete was placed in the shafts using a tremie pipe. The holes for the 80-foot-long tiebacks were drilled using a Klemm rig and continuous, smooth casing. Tieback bond lengths were established by installing and testing two preproduction tiebacks. All the production tiebacks were tested with no failures. After the wall was completed, the sidehill fill was removed. The hillside below the wall was covered with erosion control fabric and planted with soil-fixing vegetation.

SUMMARY AND CONCLUSIONS

Ground shaking during the Loma Prieta earthquake of 17 October 1989 caused permanent lateral and vertical displacements of a steep hillside composed of loose to medium-dense Dune sand. These displacements damaged 36 residences constructed along the top of the hillside. Peak ground accelerations at the site are believed to be have been less than 0.2g. To mitigate the potential for future earthquake-induced movement adjacent to the homes, a soldier pile and tieback wall was designed and constructed near the top of the slope behind the residences. Considering that there exists a high probability of a large earthquake in the San Francisco area in the next 30 years, a peak ground acceleration of 0.45g was used for design.

The seismic stability of the slope was analyzed using pseudostatic methods. The critical or yield acceleration for a particular slide mass was determined using limit equilibrium analyses. The effective peak acceleration of the potential slide mass during the design earthquake was then estimated from a graph that takes into account the variation of acceleration over the height of an embankment. Slope deformation was then estimated based on the ratio of the yield acceleration to the acceleration of the slide mass.

Uncertainty exists as to whether the graph, which was developed for earth dams, is directly applicable to slopes which are part of a larger hill or mountain. Dynamic finite element analysis would likely give some insight into this problem; however, the expense of such an analysis precludes using it on most projects. Accordingly, developing simplified methods to estimate the variation of accelerations over the height of slopes would be very useful to the practicing engineer.

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