ANALYSIS OF SOIL-NAILED EXCAVATIONS STABILITY DURING THE 1989 LOMA PRIETA EARTHQUAKE

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ABSTRACT

The performance of nine different grouted soil-nailed excavations in the San Francisco Bay area during the Loma Prieta earthquake is analyzed on the basis of postearthquake visual inspections, subsequent stability analyses, and dynamic centrifuge model tests. None of the excavations showed any signs of movements or similar distress, even though one of them was located in the vicinity of the earthquake epicenter where there was strong shaking and important seismic-related damage to other structures. The design and construction practices of grouted soil-nailed excavations in California are discussed. It is concluded that a combination of conservative design and construction is the primary reason for excellent seismic stability. It is also confirmed that the method developed and used by Caltrans for calculating the factor of safety is suitable for the stability analysis of the grouted soil-nailed excavations encompassed by the study. This method is based on a bilinear failure surface and the so-called German mode of failure that considers two sliding blocks.

INTRODUCTION

Soil nailing is an in-situ technique of mechanically stabilizing soil masses which has been used in Europe for more than two decades (Stocker and others, 1979; Chapman and Ludwig, 1993; Federal Highway Administration, 1993). In North America, as well as in Japan (Japan Highway Public Corporation, 1987; Ochiai and others, 1992), soil nailing is steadily gaining popularity because it can be used with conventional shoring equipment, it reduces excavation time, it allows construction-related activities to proceed in restricted space, and it can produce significant savings over conventional shoring techniques in the proper ground conditions.

The main feature of soil nailing is that it is an in-situ method where the existing natural soil is reinforced, as opposed to a backfill reinforcement. As shown in figure 1, the inclusions, commonly called nails, are installed during the excavation using a "top-down" construction procedure, unlike reinforced earth walls which are constructed from the bottom up. This allows soil retention in areas where little space is available for the excavation. The soil-nailing concept is to reinforce the soil with passive inclusions, so that the nailed soil mass behaves as a composite unit, similar to a gravity retaining wall supporting a soil backfill (Juran and Elias, 1991; Mitchell and Villet, 1987). In that sense, soil nailing also differs from the conventional tie-back excavation support since the soil nails are not prestressed; that is, their resistance can be mobilized only by the movement of soil mass or the face of the excavation to which the nails are fixed. Figures 2 to 4 show several soil-nailed retaining structures treated in this paper.

At present, there are three major concerns about soil-nailed excavations: (1) the adequacy of the analysis or design methods, (2) the long-term behavior, and (3) the performance during seismic loading. Items (1) and (2) have been addressed by many researchers, including recently by Gassler (1992), Juran and others (1990), Plumelle and others (1990), and Stocker and Riedinger (1990). With respect to soil-nailing performance during earthquakes, no full-scale field observations were available until the Loma Prieta earthquake. During the earthquake, nine soil-nailed structures were subjected to different levels of shaking, including horizontal ground-surface accelerations probably as high as 0.4 g. In spite of such relatively high horizontal accelerations, these structures did not show any visible movements or other signs of distress (Felio and others, 1990; Hudson, 1990). A systematic description of these structures and a discussion of possible reasons why they performed so well are the main purposes of this paper. More details about the corresponding investigation can be found in Tufenkjian and Vucetic (1993).

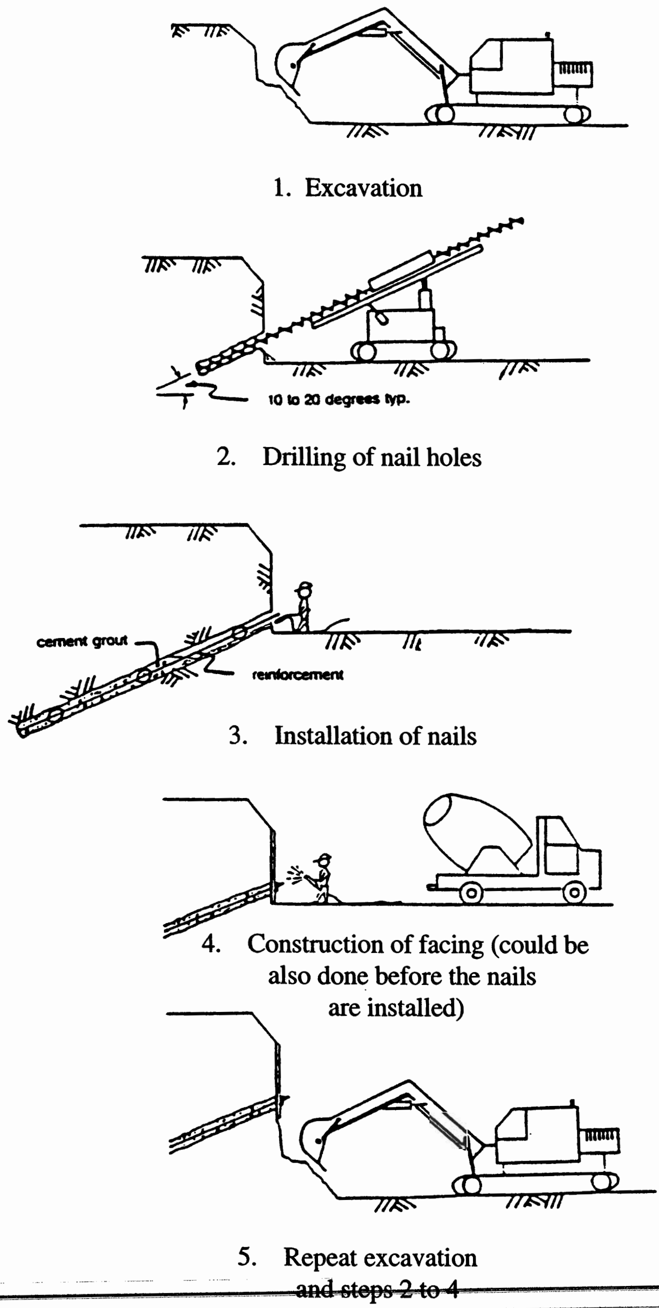


Figure 1.—Steps in the construction of a grouted soil-nailed excavation.

SOIL NAILING PRACTICE IN CALIFORNIA

There are three major steps in the construction of a soil-nailed wall, as illustrated in figure 1. They are (1) excavation, (2) installation of nails, and (3) construction of facing. The excavation generally proceeds in stages ranging from 1.2 to 1.8 m in depth. One of the major requirements for successful soil-nailed systems is that the excavation be capable of self-support for at least a few hours prior to nailing and construction of facing. For the most economical construction, however, the self-support should be able to last 1 to 2 days. As the excavation of each level proceeds, the nails are installed at predetermined locations. These reinforcing elements may be one of several types: driven, grouted, jet-grouted, or even pneumatically propelled into the ground (Myles and Bridle, 1991). However, the vast majority of installations are of the open drilled and grouted type (Chapman and Ludwig, 1993; Federal Highway Administration, 1993).

In California, and North America in general, the most popular type of nails are the grouted nails, such as those shown in figures 2, 3 and 4, since in many locations the soil conditions allow the excavation to stand open long enough. Grouted nails generally consist of Grade 60 mild steel bars (15 to 45 mm in diameter) placed in boreholes of 100 to 250 mm diameter. Plastic centralizers are often used to ensure proper grout cover of the nail. A cement grout is then placed into the boreholes by gravity flow or low pressure. Typical horizontal and vertical spacings range from 1 to 3 m, depending upon the designer's experience and soil conditions. The nails are generally inclined at 10° to 20° from the horizontal.

Either before or after the nails are in place, a facing structure is built. The facing is required to control soil erosion at the excavation face and reduce changes in the moisture content of the soil. The most common type of facing is shotcrete layer, 100 to 250 mm thick, which is usually placed by the shotcrete method and which is reinforced with welded wire mesh. A typical detail of the nail connection to such facing is presented in figure 5. If necessary, a blanket of nonwoven geotexile is placed between the natural soil and the shotcrete to control the drainage. The grouted nail is attached to the facing by bolting the steel bar to a square plate usually 300 to 400 mm wide. For additional reinforcement and strengthening of the facing, horizontal waler bars may be installed to connect the plates. Other methods of attachment are used for driven nails. For permanent walls, the shotcrete facing may not provide for the aesthetic requirements of the project. In such cases, either cast-in-place reinforced concrete facing or prefabricated panels can be used. Figure 6 shows photos of large soil-nailed excavation structures recently completed in California.

THE LOMA PRIETA EARTHQUAKE

The Loma Prieta earthquake (M_s=7.1) was one of the most costly single natural disasters in U.S. history. It caused extensive damage, such as landslides in the epicentral region, liquefaction in various areas of the San Francisco Bay region, structural distress to commercial, industrial, and residential buildings, widespread disruption or total destruction of utility systems, and damage to critical transportation systems. The earthquake has been the subject of a wide range of studies, many of them on geotechnical-related failures, as summarized by Seed and others (1991).

Figure 7 presents an overview of the regional geology and the recorded peak horizontal ground-surface accelerations during the earthquake. The locations of the nine soil-nailed walls considered in this paper are identified on the figure by stars, and the location of the epicenter by a circle. The figure shows that in the epicentral area the measured maximum horizontal ground-surface accelerations, a_{max}, were as high as 0.64 g and the vertical up to 0.60 g. It can be seen that the soil-nailed walls in the northern region (in Richmond, San Francisco, Walnut Creek, and San Ramon) were subjected to seismic forces corresponding to a_{max} of about 0.10 g. In the vicinity of the two walls in Mountain View, an a_{max} of around 0.2 g was measured. In the vicinity of the wall in San Jose, a_{max} was between 0.11 and 0.18 g. The largest a_{max} (0.47 g) recorded near a soil-nailed wall was in Santa Cruz, some 16 km due west of the epicenter.

Most of these locations were visited and inspected 2 days after the earthquake by a team from the University of California, Los Angeles (Felio and others, 1990), and some walls were inspected subsequently by design and construction companies. As stated earlier, no signs of distress or corresponding deformation were found on the walls, indicating excellent performance of such structures during moderate and strong shaking.

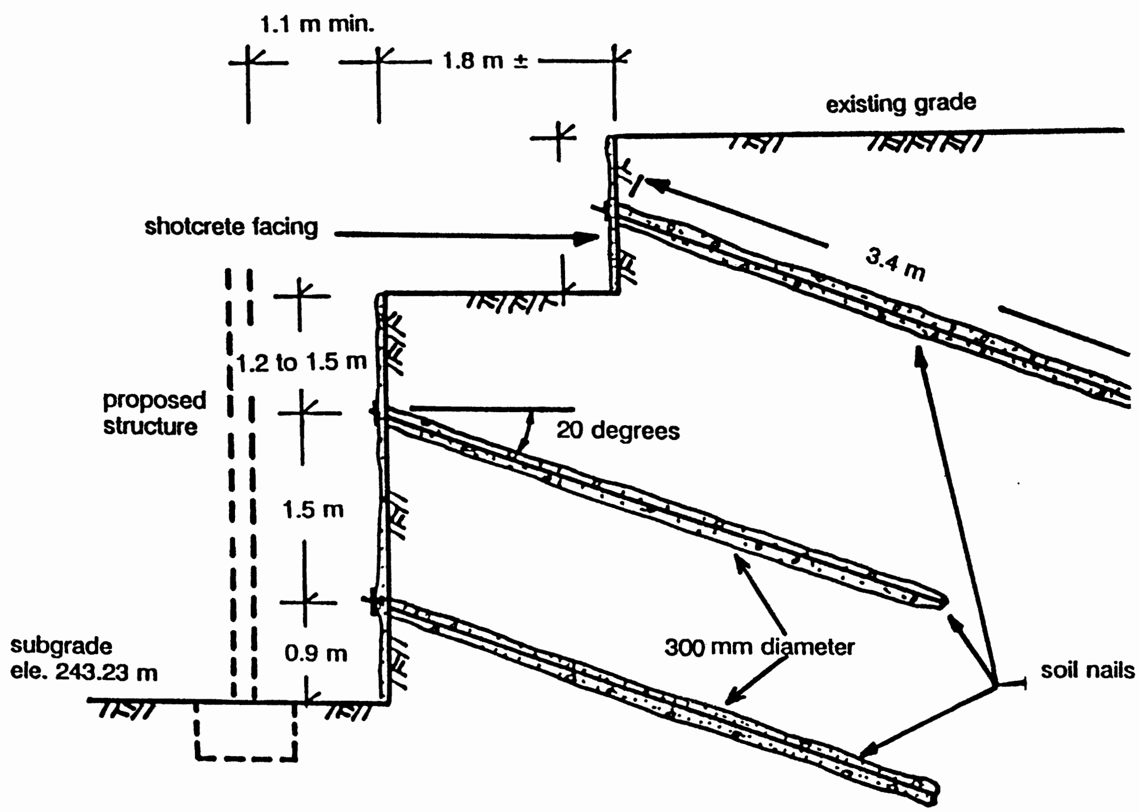


Figure 2.—Cross section of a soil-nailed excavation for a building constructed in Santa Cruz (UCSC wall), Calif. (Felio and others, 1990).

CHARACTERISTICS AND DESCRIPTION OF THE WALLS

The main characteristics of the nine walls inspected after the earthquake are summarized in table 1. In figure 8 the dimensions of the walls are presented in a uniform scale. The variation of the geometry, characteristics of the walls, soil conditions, and estimated ground-surface accelerations are evident. The walls are further characterized in table 2 in terms of the following three dimensionless ratios commonly used as design criteria (Bruce and Jewell, 1987):

$$\text{Length ratio} = \frac{\text{maximum nail length}}{\text{excavation height}} \qquad (1)$$

$$\text{Bond ratio} = \frac{\text{hole diameter} \times \text{nail length}}{\text{horizontal spacing} \times \text{vertical spacing}} \qquad (2)$$

$$\text{Strength ratio} = \frac{(\text{nail diameter})^2}{\text{horizontal spacing} \times \text{vertical spacing}} \qquad (3)$$

Table 3 further compares the three dimensionless ratios computed for the nine San Francisco walls, with the values computed by Bruce and Jewell (1986, 1987) for soil-nailed structures with drilled and grouted nails constructed all over the world. The bond and strength ratios generally fall within the range of other soil-nailed retaining structures. However, the length ratios for the San Francisco walls are generally much higher than those calculated from other sites, suggesting that the San Francisco walls are more conservatively designed. Note from table 2 that the length ratio for the UC Santa Cruz wall is the smallest. This wall is apparently the least conservatively designed of all of the walls, yet it is located in the vicinity of the highest estimated peak horizontal ground-surface acceleration. In spite of these facts, no ob-

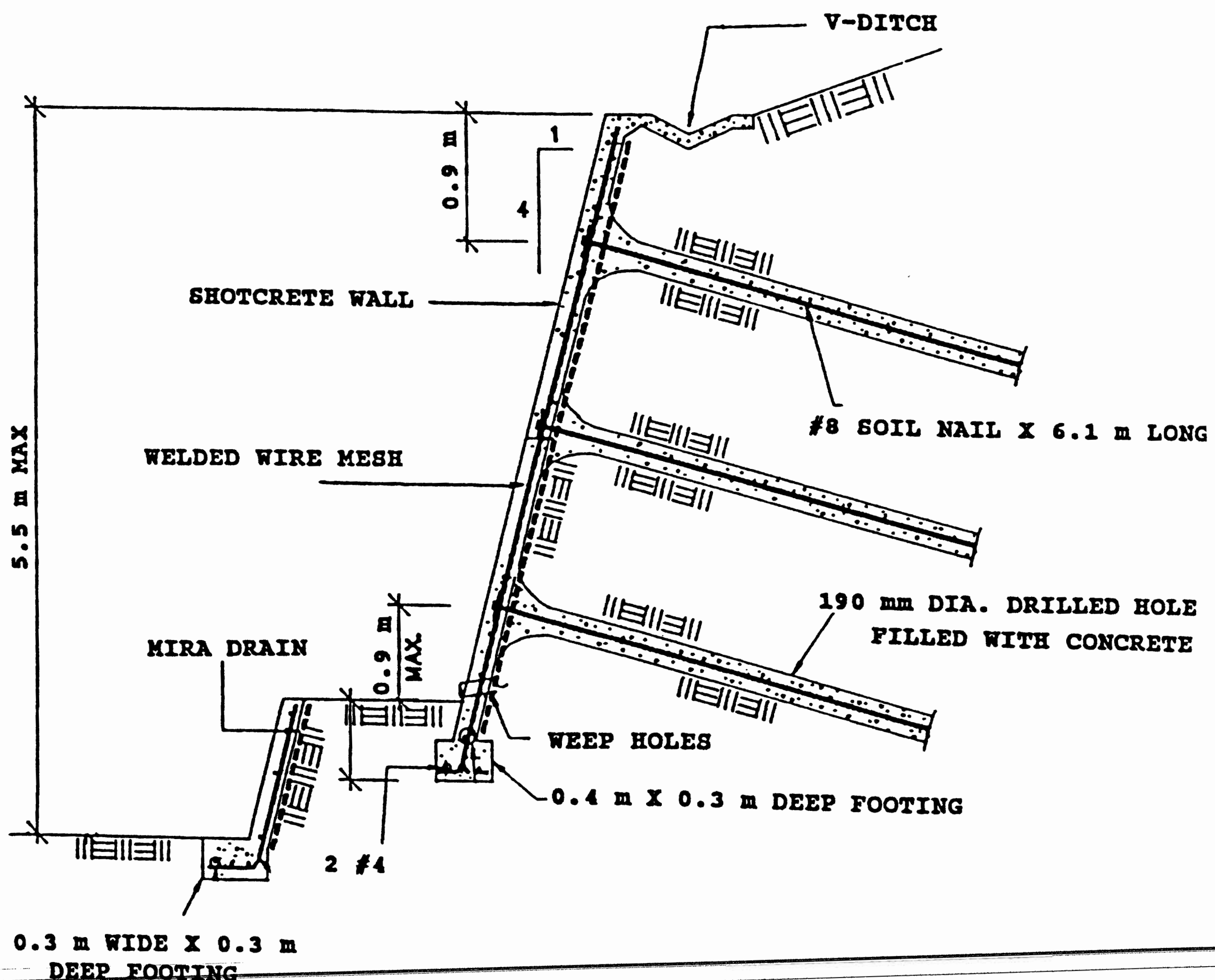


Figure 3.—Cross section of a permanent soil-nailed retaining structure located in San Ramon (NME wall), Calif. (Barar, 1990).

servable damage was noted on the Santa Cruz structure after the earthquake. The walls are described in greater detail below.

MOUNTAIN VIEW, 2350 EL CAMINO REAL (ECR WALL)

Nearly 280 m² of soil-nailing construction was used to provide temporary shoring of an excavation for an office building. The concrete wall for the new structure was to be poured in front of the soil-nailed concrete facing. The subsurface soil consisted of gravelly and clayey sand. The shear-strength parameters used in design were $c = 9.6$ kN/m² and f = 30°, while the soil unit weight was assumed to be 17.3 kN/m³. The soil-nailed wall was completed by May of 1989 and the excavation was still open when the earthquake struck. Postearthquake observations revealed only a few shallow hairline cracks in the concrete facing, typical of flexural cracking if the facing is considered as a vertical slab with the nails acting as reaction points. Note in table 1 that the facing of this wall was relatively thin (100 mm), while the estimated horizontal acceleration was considerable (0.21 to 0.27 g).

MOUNTAIN VIEW, KAISER PERMANENTE PARKING GARAGE (KPG WALL)

Approximately 380 m² of shoring was provided for the construction of a parking garage. Soil nailing was used only on one side of the excavation, while a combination of other shoring techniques were used on the remaining sides. The soil conditions at the site consisted of stiff sandy to clayey

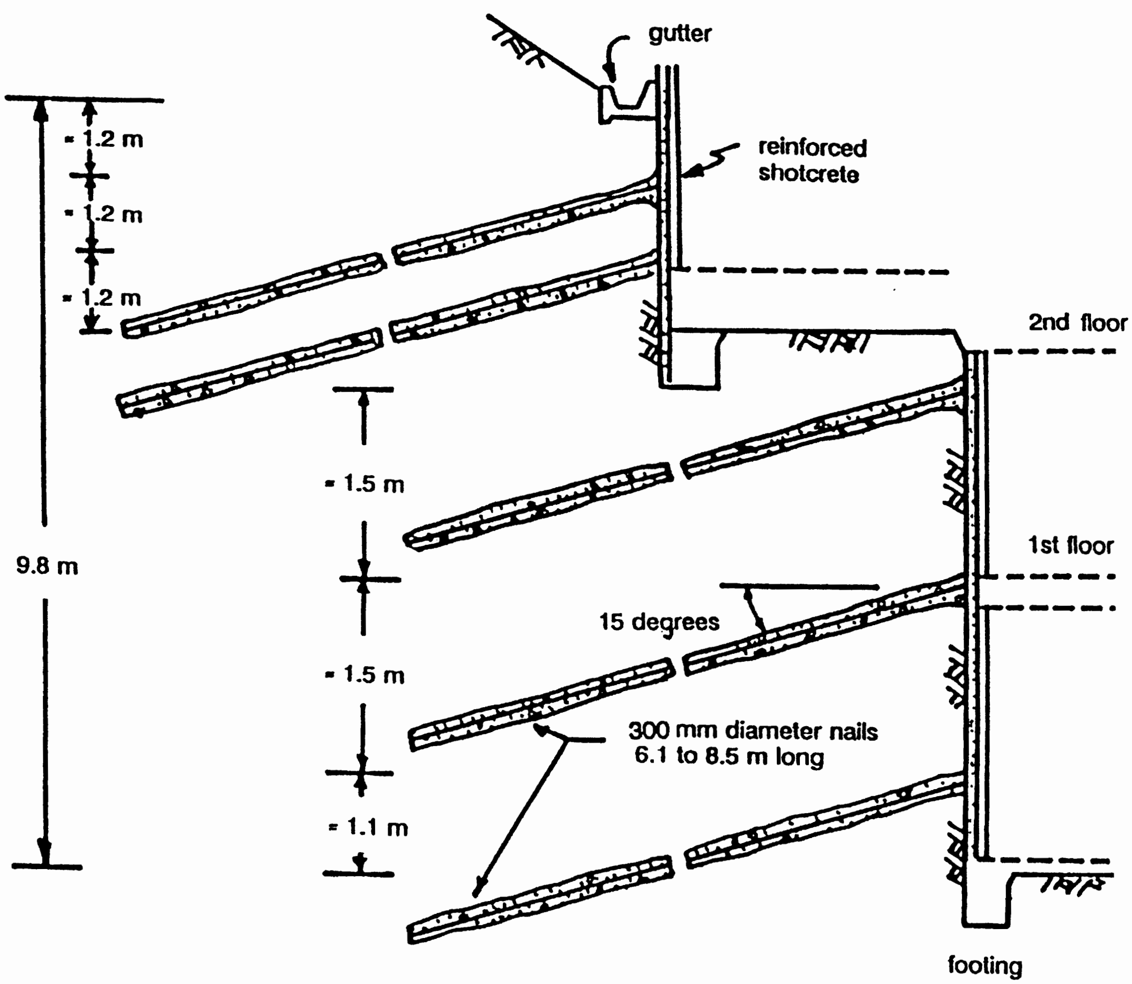
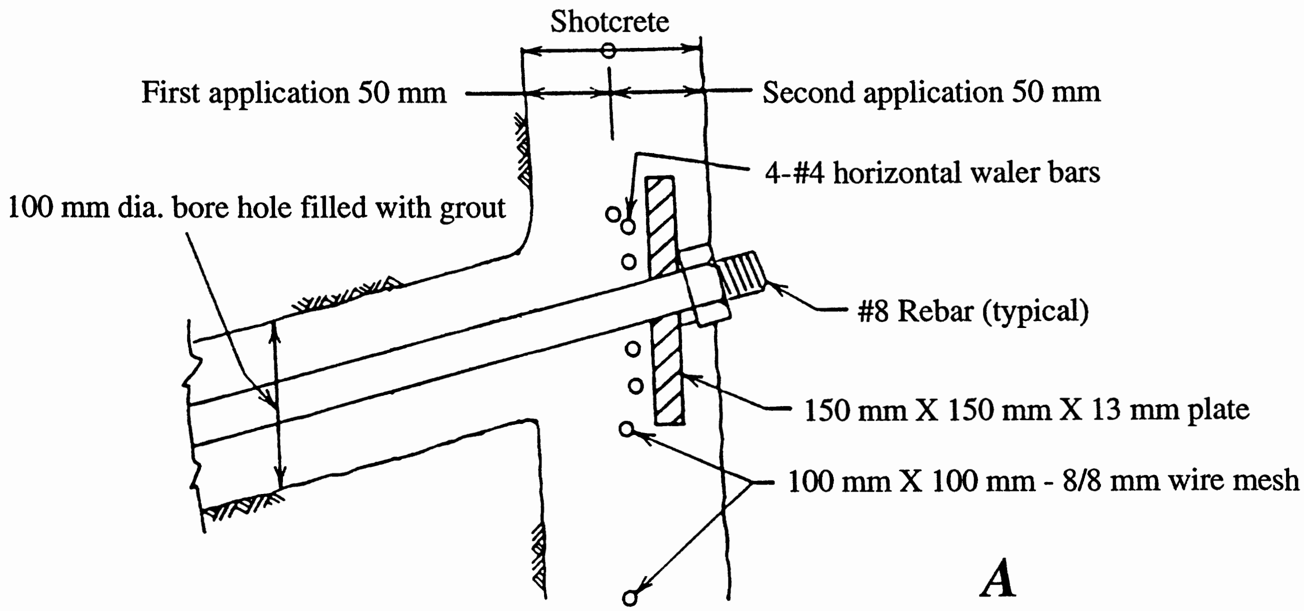


Figure 4.—Cross section of a soil-nailed retaining wall constructed on a steep hill in San Francisco (CVA wall), Calif., to provide adequate space for an apartment complex (Felio and others, 1990).

silt overlying silty to sandy clay. The shear strength parameters used in design were $c = 23.9$ kN/m^2 and f = 14°, while the soil unit weight was assumed to be 18.8 kN/m^3. The construction of the shoring was completed just 8 days before the earthquake. The postearthquake observations revealed no visible distress to the soil-nailed wall, while the opposite side



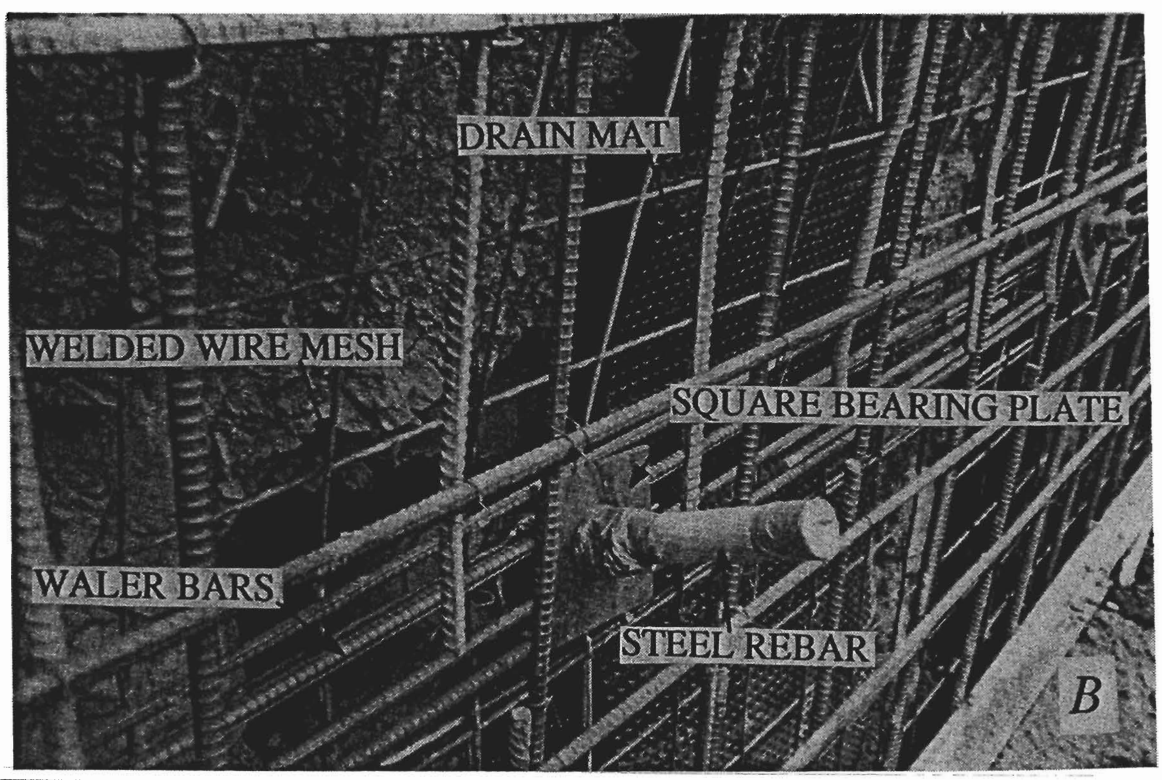
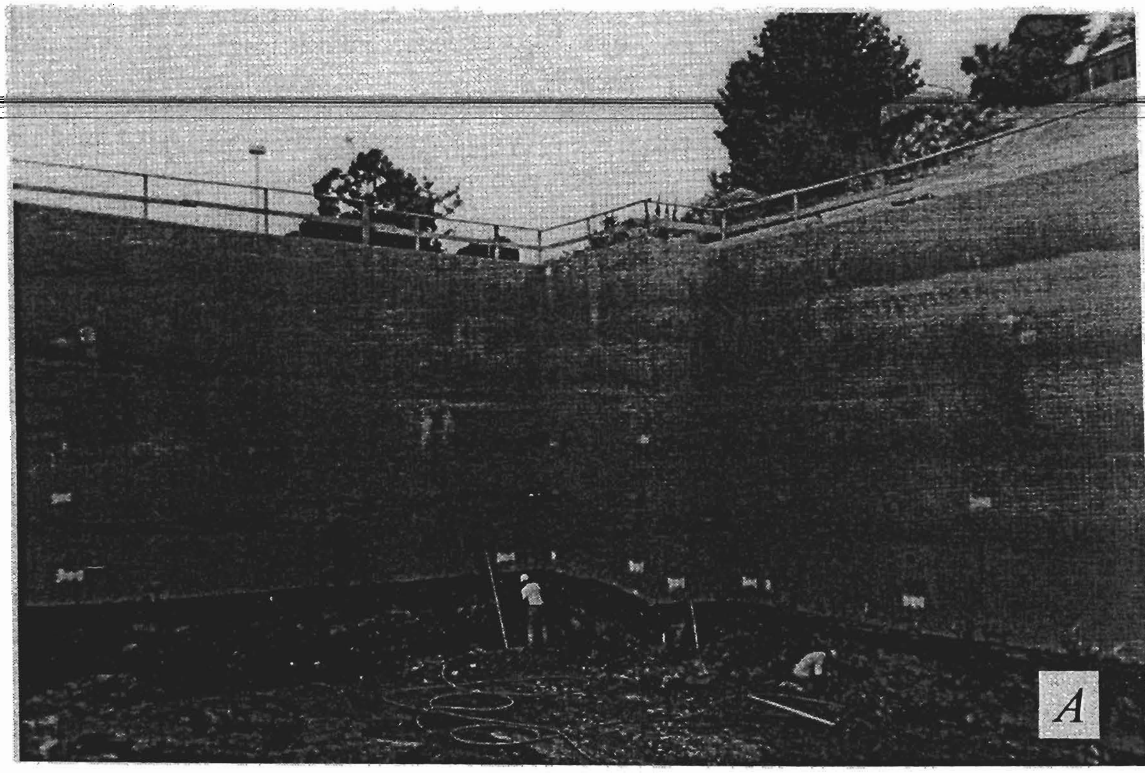


Figure 5.—Connection between grouted nail and facing. *A*, Cross section of a typical connection (from Koerner, 1984). *B*, Strong reinforcement around the nail tip.



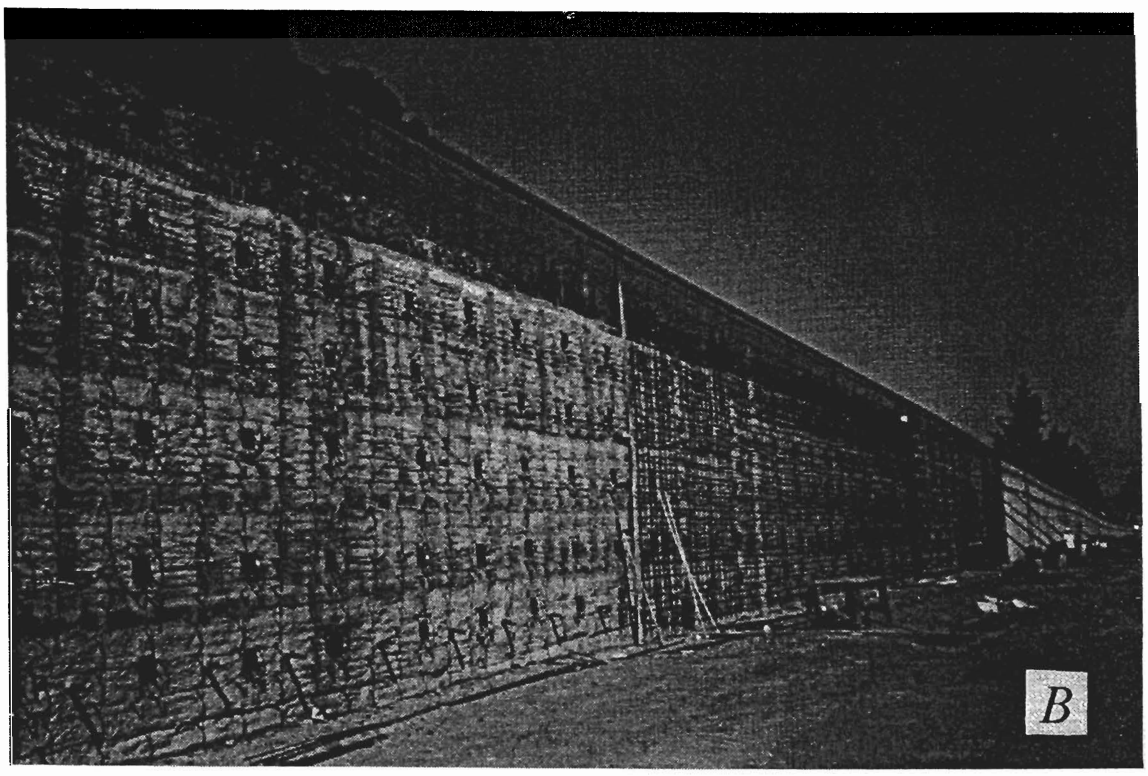


Figure 6.—Soil-nailed structures recently completed in California. *A*, Soil-nailed excavation for an underground structure of a building. *B*, A highway retaining soil-nailed wall showing different stages of the construction of permanent facing.

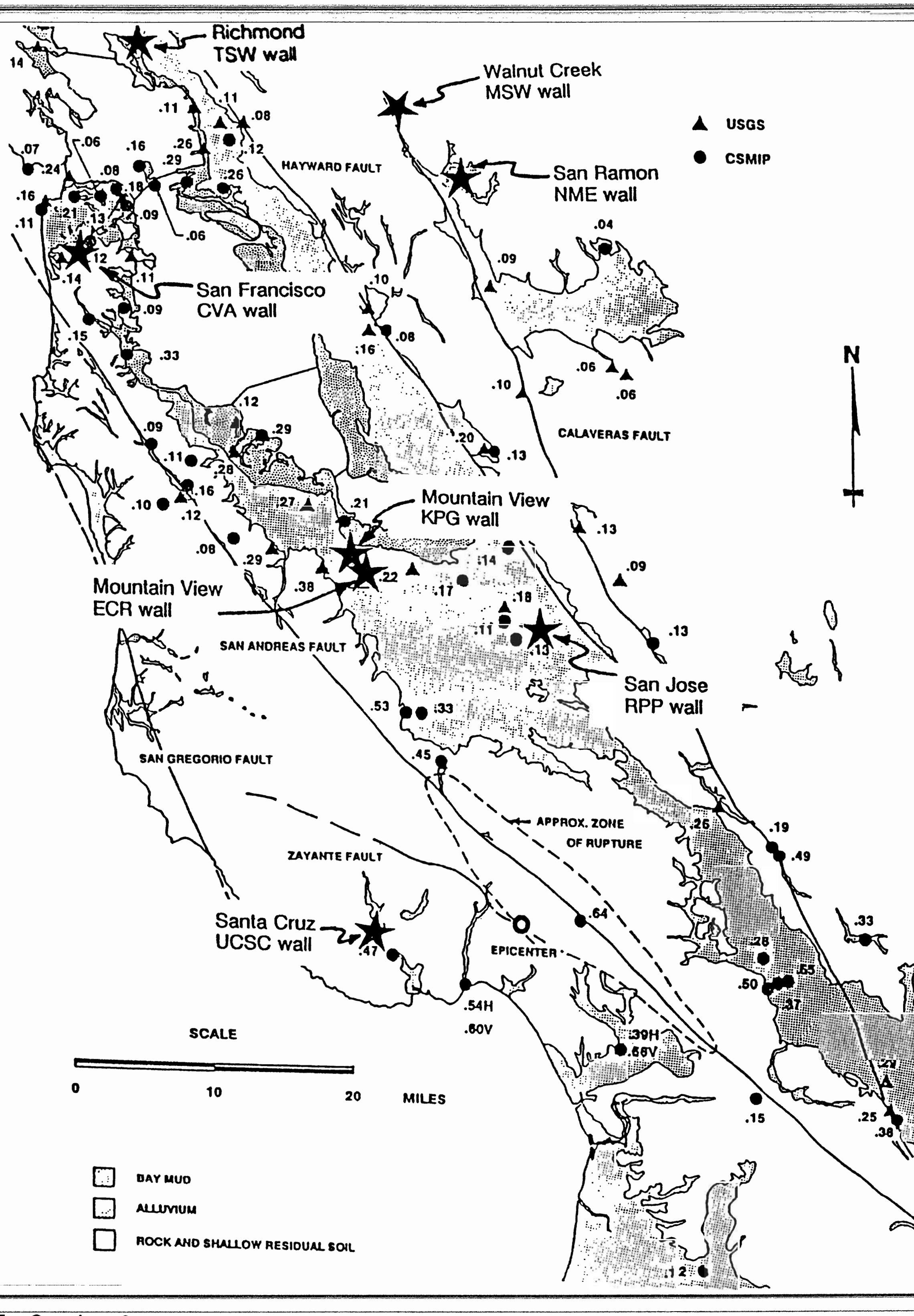


Figure 7.—Overview of regional geology and recorded peak horizontal ground-surface accelerations during the Loma Prieta earthquake (from Seed and others, 1991).

of the excavation, which used cantilevered soldier beams with a concrete facing between the beams, revealed some vertical hairline cracks in the facing.

SANTA CRUZ, UNIVERSITY OF CALIFORNIA AT SANTA CRUZ (UCSC WALL)

Approximately 350 m² of shoring was required to construct a new science library on the UCSC campus. The soil conditions at the site consisted of sandy silt to sandy clayey silt extending from the ground surface to a depth of approximately 6.4 m. The soil has an average dry unit weight of 13.8 kN/m³ and a moisture content ranging from 26.2 percent in the clayey silt near the surface to 13.9 percent in the sandy silt at 6.4 m. Shear strength properties used in design were c = 23.9 kN/m² and f = 25°. The cross section at the highest location of the wall is shown in figure 2. Construction of the wall was completed on September 28, 1989, less than 3 weeks before the earthquake. Since three sides of the excavation were soil nailed, at least one side may have been subjected to the full strength of the earthquake, which produced in the vicinity of the excavation peak horizontal ground-surface accelerations of about 0.47 g. It should be noted that this wall was located closest to the epicenter and presumably was subjected to the strongest shaking, while at the same time it had the smallest length ratio and thinnest facing among the nine walls examined (see tables 1 and 2). Prior to the earthquake, some wall and column spread footings had been poured (see fig. 2). A postearthquake inspection revealed significant cracking in the concrete of the footings. This cracking was not attributed to shrinkage since foundations constructed after October 17 showed fewer cracks. As opposed to that, the inspection of the soil-nailed wall after the earthquake revealed no cracking. A week after the earthquake, nine nails were tested to 150 percent of their design pull-out load. The tests showed no loss in the carrying capacity of the nails due to the seismic activity.

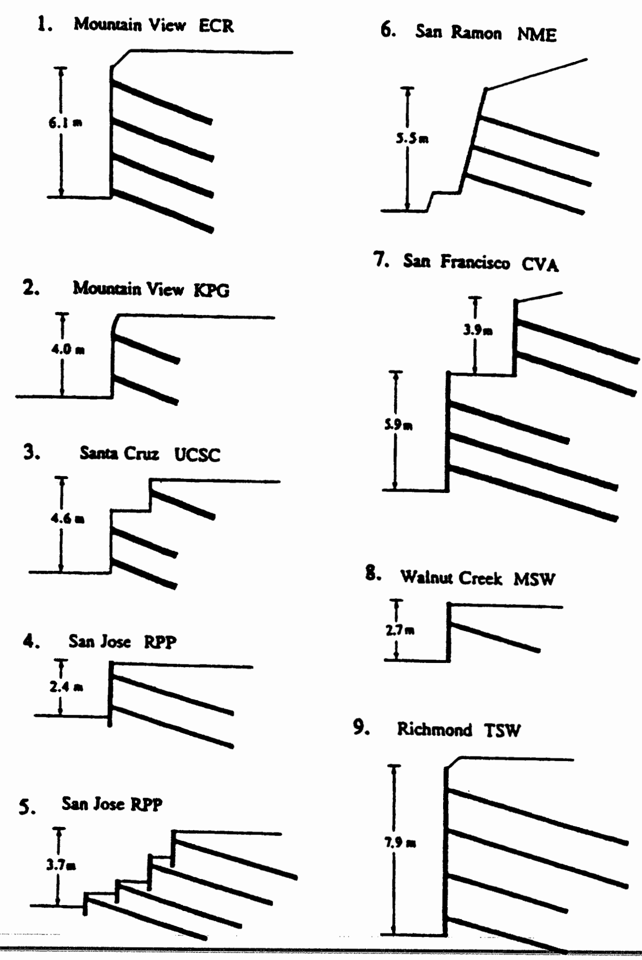


Figure 8.—Dimensions of the investigated soil-nailed walls.

SAN JOSE, RIVERPARK PROJECT (2 RPP WALLS)

These two retaining walls were designed and built as permanent structures along the Guadalupe River in San Jose, approximately 40 km north of the epicenter. The subsurface soil consists of silty and sandy clays to a depth of about 4.5 to 6 m. According to the geotechnical report, these clays have an intermediate to high plasticity with an approximate average dry unit weight and moisture content of 14.1 kN/m³ and 22 perecent, respectively, and an undrained shear strength ranging from 72 to 240 kN/m², as interpolated from static cone penetration tests. The clays are underlain by a 3 m zone of dense, clayey, silty, gravelly sand with an average dry unit weight and moisture content of about 17.3 kN/m³ and 15 percent, respectively. The shear strength parameters used in design were c = 23.9 kN/m² and f = 0°, while the total unit weight was assumed to be 19.6 kN/m³. Since these are permanent walls, the concrete surface was finished off with architectural concrete and in some places clad with granite. The postearthquake observations revealed no signs of distress.

SAN RAMON, NATIONAL MEDICAL ENTERPRISES COMMUNITY HOSPITAL (NME WALL)

This soil-nailed retaining structure forms a part of a permanent retaining wall used for the roads and landscape that surround the medical center. The wall cross section is shown in figure 3. According to the geotechnical report, the soil conditions consist mainly of engineered fill up to a maximum depth of 24 m, generated from cut-and-fill operations performed previously. Therefore, the soil-nailed retaining structure was built in fill material. The fill consists of sandy and silty clay of moderate to high plasticity, with an average dry unit weight and moisture content of approximately 17.1 kN/m³ and 18 percent, respectively. The shear strength prop-

Table 1.—*Summary of soil-nailed walls investigated*

Project No.	Location (see fig. 7)	Height of wall (m)	Nail details: Spacing (m)	Length (m)	Diameter: Grout (mm)	Rebar (mm)	Inclination (degrees)	Facing thickness (mm)	General soil Type	Soil unit weight (KN/m³)	Shear strength design parameters: c (KN/m²)	ϕ (degrees)	Estimated horizontal ground surface acceleration (g)
1	Mountain View ECR	6.1	1.7V 1.5H	5.2	300	32	20	100	clayey sand	17.3	9.6	30	0.21 to 0.27
2	Mountain View KPG	4.0	2.0V 1.5H	3.4	300	25	20	100	sandy to clayey silt	18.8	23.9	14	0.21 to 0.27
3	Santa Cruz UCSC	4.6 2-levels	1.5V 1.8H	3.4	300	32	20	75	sandy to clayey silt	18.1	23.9	25	0.47
4	San Jose RPP1	3.7 4-levels	1.8V 1.8H	6.1	180	25	15	200	alluvial clay, silt and sand	19.6	23.9	0	0.10 to 0.15
5	San Jose RPP2	2.4	1.8V 1.8H	6.1	180	25	15	200	alluvial clay, silt and sand	19.6	23.9	0	0.10 to 0.15
6	San Ramon NME	5.5	1.8V 1.8H	6.1	190	25	15	150	engineered fill	18.8	47.9	0	0.05 to 0.10
7	San Francisco CVA	9.8 2-levels	1.8V 1.5H	6.1 to 8.5	300	25	15	200	fill over silty clay and highly weathered siltstone	19.6	9.6	35	0.10 to 0.15
8	Walnut Creek MSW	2.7	(one row) 1.8H	4.6	190	25	15	200	fill over medium stiff to stiff clay	18.8	14.4	28	0.01 to 0.10
9	Richmond TSW	7.9	1.8V 1.8H	6.1 to 9.1	190	25	15	200	alluvium deposits	18.8	19.1	28	0.05 to 0.15

erties used in design were $c = 47.9$ kN/m² and f = 0°. Since this is a permanent structure, the facing of the soil-nailed wall was finished with a colored architectural concrete finish. The postearthquake walk-through revealed that the surface of the concrete remained smooth and free of cracks.

SAN FRANCISCO, CRESTA VISTA APARTMENTS (CVA WALL)

This wall demonstrates the unique concept of using soil nailing on a permanent basis to retain the slope and cut on a

Table 2.—*Dimensionless ratios for San Francisco area soil-nailed walls*

Project No.	Location	Length ratio $\left(\frac{\text{max. nail length}}{\text{excav. height}}\right)$	Bond ratio $\left(\frac{\text{hole dia. x nail length}}{\text{H. spacing x V. spacing}}\right)$	Strength ratio (10^{-3}) $\left(\frac{(\text{nail dia.})^2}{\text{H. spacing x V. spacing}}\right)$
1	Mountain View ECR	0.85	0.61	0.40
2	Mountain View KPG	0.85	0.34	0.21
3	Santa Cruz UCSC	0.74	0.38	0.38
4	San Jose RPP	1.6	0.34	0.19
5	San Jose RPP	2.5	0.34	0.19
6	San Ramon NME	1.1	0.36	0.19
7	San Francisco CVA	1.5 to 1.7	0.68 to 0.94	0.28
8	Walnut Creek MSW	1.7	not applicable	not applicable
9	Richmond TSW	1.2	0.36 to 0.53	0.19

Table 3.—*Dimensionless ratios for soil-nailed walls*

	Drilled and grouted in granular soils (Bruce and Jewell, 1986, 1987)	Drilled and grouted in morraine and marl (Bruce and Jewell, 1986, 1987)	San Francisco walls
Length ratio	0.5 to 0.8	0.5 to 1.0	0.7 to 0.25
Bond ratio	0.3 to 0.8	0.15 to 0.20	0.34 to 0.94
Strength ratio (10^{-3})	0.4 to 0.8	0.1 to 0.25	0.19 to 0.40

steep hill to make room for the development of a housing project. The wall cross section is shown in figure 4.The 9.8-m-high soil-nailed structure was constructed at the toe of a 45.7-m-high slope to allow for the construction of apartment units. The wall is about 90 m long and consists of two levels. Due to the permanent nature of the structure, a 200 mm reinforced concrete facing and a small footing at the base were used. The soil conditions at the site can be described as colluvium and residual soil deposits.The design parameters used were cohesion c = 9.6 kN/m^2 and an angle of internal friction of f = 35°. The inspection that took place 3 days after the earthquake showed no signs of distress to the wall and no indications of lateral movements or tension cracks in the hill behind the wall.

WALNUT CREEK, MINI STORAGE FACILITY (MSW WALL)

The project consisted of a three-story building with two levels above grade and one level of basement below. The soil-nailed wall was integrated into the final basement wall. The soil at the site consists mainly of fill material up to 3 m depth, including a nonuniform mixture of gravel, sand, and clay. The underlying soil consists of stiff silty clay. The average dry unit weight and moisture content of the fill is 16.5 kN/m^3 and 20 percent, respectively. The shear strength parameters assumed in design were c = 14.4 kN/m^2 and f = 28°, while the total unit weight of the soil was assumed to be 18.8 kN/m^3. The postearthquake observations revealed no signs of distress on the surface of the wall or at grade behind the wall.

RICHMOND, TEMPORARY SHORING WALL (TSW WALL)

Soil nailing was used here to construct a temporary shoring wall which has the tallest single-level vertical face of any of the walls examined in this paper. A permanent retaining wall was eventually built in front of the soil-nailed wall. Unfortunately, soil stratigraphy data is not available for this site. However, the shear strength parameters used in design were c = 19.1 kN/m^2 and f = 28°, while the unit weight of the soil was assumed to be 18.8 kN/m^3. A walk-through of the site following the earthquake did not reveal any signs of distress attributable to seismic activity.

METHODS OF ANALYSIS

Most of the current design methods for soil-nailed retaining structures under static loads are derived from classical slope-stability analyses, which incorporate a limit equilibrium approach. Accordingly, they evaluate global factors of safety along assumed failure surfaces such as those shown in figure 9. They are usually referred to as the German method (Stocker and others, 1979; Gassler and Gudehus, 1981; Lambe and Jayaratne, 1987), Davis method (Shen and others, 1981; Bang and others, 1992), French method (Schlosser and others, 1983), and Caltrans method (computer program SNAIL: Caltrans, 1993). The differences in the methods result from the definition of the factor of safety, assumed failure surface shape, and the assumed contribution of the soil nails to the stability. In that respect, the methods are contradictory, and because of the lack of full-scale observations of actual failure mechanisms, different points of view about their applicability have emerged.

The German method (fig. 9*A*) assumes a bilinear failure surface passing through the toe of the excavation. The failing soil mass is broken into two parts. The first part contains most of the nailed soil mass, while the second part forms the active earth pressure wedge behind it—behind the "soil-nailed gravity wall." The analysis considers the tensile and pull-out resistance of the nails crossing the failure surface and, of course, the forces of interaction between the nailed mass and active wedge behind it. The assumed failure surfaces are consistent with the concept of soil nailing, that is, the nailed soil mass behaves like a reinforced block.

The Davis method incorporates a parabolic failure surface that also passes through the toe, as shown in figure 9*B*. The sliding surface either passes entirely through the nails or intersects the ground surface somewhere beyond the reinforced zone. In the analysis, the tensile and pull-out resistance of the nails crossing the failure surface are considered the governing stabilizing forces. Because of its successful track record and easy implementation, it has been a popular design method in the United States. This has been the case in spite of the fact that the assumption of a parabolic slip surface (which does not change slope when crossing from the nonreinforced to the reinforced zone) has not been adequately verified by laboratory or field tests.

The French method follows procedures similar to the Davis method, but assumes a circular failure surface passing entirely through the nails, as shown in figure 9*C*. But, unlike the previous two methods, this method considers the shear and bending of the nails, which adds to the complexity of the analysis.

The Caltrans method also assumes a bilinear failure surface, just like the German method. However, unlike in the German method, the bilinear failure surface may pass entirely through the nails (see fig. 9*D*).

More recently, a kinematical limit analysis approach has been proposed for the design of soil-nailed retaining structures (Juran and others, 1990). It differs from the other analysis procedures in that it suggests a method for estimating nail forces. In this way, it may provide a check on local stability at each level of nail reinforcement. The method assumes that the failure surface is defined by a log-spiral passing partially through the nails and that the failure occurs by rotation of a quasi-rigid body along this surface.

All of the San Francisco walls examined in this study were designed using a modified version of the Davis method (Barar, 1990; Felio and others, 1990). Seismic forces were accounted for by using an equivalent static horizontal force $H = W \times k_h$, applied at the center of gravity of the potentially unstable soil nailed mass, where W is the weight of the moving soil mass and k_h is the horizontal seismic coefficient.

It should be mentioned at this point that the Davis method, as well as the German and Caltrans methods, has a certain degree of the inherent conservatism in that the potential stabilizing effects of the shear and bending resistances of the nails are ignored. New studies (Jewell and Pedley, 1992; Federal Highway Administration, 1993) show, however, that the effects of bending stiffness are small. Also, the contribution of the steel reinforced facing to the strength of the system is unaccounted for. The lack of full understanding of the role of facing in the global and local stability apparently led to the difference by a factor of 3 (75 mm vs. 200 mm) in the thicknesses of the facing among the nine walls under consideration. Some designers and construction companies feel comfortable with thinner facing, while some prefer more conservative thicker facing. Figure 5, for example, illustrates a rather heavily reinforced facing with a sturdy nail contact. The role of the facing in soil reinforcing stability is just beginning to be studied as a separate issue (Tatsuoka, 1992), and it should definitely be given more attention in the future.

The factors of safety for the CVA and TSW soil-nailed walls obtained by the Davis method, modified to account for earthquake forces by the pseudostatic technique, are presented in table 4. The location of the assumed failure surfaces that yield minimum factors of safety for the TSW wall are shown in figure 10. In general, the factors of safety are relatively low, especially for the range of estimated peak horizontal ground accelerations during the Loma Prieta earthquake. According to such low factors of safety, and given the fact that some soil-nailed structures were probably subjected to much larger horizontal forces, some visible damage should have occurred during the earthquake. This should have been expected in particular for the UCSC Wall in Santa Cruz, which had the smallest length ratio and facing thickness, and yet is likely to have undergone horizontal seismic forces as large as 0.4 *g*. The lack of visible damage on any of the walls, except very thin cracks on the ECR wall, suggests that either the design, analysis, or construction, or most likely their combination, may have been more conservative than necessary. The lack of damage also indicates that the assumed failure surface and mechanism of failure of the Davis method may not be fully appropriate for the nine walls treated here. In the following section, the components of the analysis, design, and construction that appear to be on the conservative side, and therefore could be responsible for such excellent seismic performance, are discussed.

POSSIBLE REASONS FOR THE OBSERVED BEHAVIOR

Since soil nailing is a relatively new soil-stabilization technique, with very little practical experience of full-scale static failures and practically no experience of seismic failures, the design and construction are usually quite conservative. The preliminary design of a soil-nailed retaining structure proceeds much like that of retaining walls, by trial and error. Based mainly on the expected excavation height and the soil strength properties, tentative characteristics of nails and facing (length, diameter, horizontal, and vertical spacings of nails, and the thickness and reinforcement of facing, etc.) can be assumed and some sort of stability analysis performed. The assumed values and characteristics depend primarily on the designer's experience with other satisfactorily constructed soil-nailed walls, which may lead to an overly conservative design, and to a lesser extent on charts and dimensionless parameters derived by others, such as those by Bruce and Jewell (1986, 1987) and Guilloux and Schlosser (1982). Table 3 shows, for example, that the length ratios and bond ratios for the nine walls considered here are on the conservative side in comparison with the values suggested by Bruce and Jewell (1986, 1987).

The main components of the conservative design and construction for seismic loads include (1) conservative and most probably unrealistic assumption of the failure mechanism, (2) no consideration of the contribution of the facing in the stability analysis, and (3) conservative construction due to the lack of field experience and understanding of the various aspects of soil-nailed excavation seismic response. The first two components are discussed below.

FAILURE-MECHANISM ASSUMPTION

Due to a lack of full-scale observations of failures and corresponding failure mechanisms under both static and seismic loads, there is currently no consensus among designers on which failure mode is the most realistic among the four basic modes presented in figure 9. To cast more light on possible modes of failures under dynamic loads, two series of dynamic centrifuge tests were conducted, one in 1991 (Tufenkjian and others, 1991; Tufenkjian and Vucetic, 1992; Vucetic and others, 1993) and the other in 1996 (Vucetic and others, 1996). Figures 11 and 12 show the main features of the models tested and results obtained in 1991.

The centrifuge tests were performed at the Rensselaer Polytechnic Institute (RPI) Geotechnical Centrifuge Research Center on a 3-m radius Accutronic 665-1 centrifuge (Elgamal and others, 1991). The scale factor was 50 in all of the tests. Accordingly, to simulate prototype geostatic stresses, the models had to undergo a centrifugal acceleration of 50 *g*. For dynamic testing, a servo-hydraulic earthquake simulation shaker mounted on the centrifuge platform was used. Four models were tested in 1991.

They represented 7.6-m-high soil-nailed excavations with grouted nails, corresponding roughly to an excavation height of a two- to three-story underground garage. The effects of two important characteristics of soil-nailed structures were tested: the length of nails (expressed in terms of the length ratio), and the axial and flexural rigidities of the nails.

Three length ratios were tested, 0.33, 0.67, and 1.0, which could be characterized as the ratios corresponding to short, medium, and long nails (see table 3). These ratios cover approximately five out of the nine walls listed in table 2. The four other walls have very large length ratios between 1.5 and 2.5. Two axial and flexural rigidities of the nails were used, one that can be considered regular and the other than can be considered small. By varying the axial and flexural rigidities of the soil nails, their effect on the failure surface geometry and stability could be assessed. As shown in figures 11*A* and 11*C*, three displacement transducers (LVDT's) were used to record the lateral movements of the facing and the vertical soil settlement behind the facing. During dynamic loading, four accelerometers were utilized to measure the accelerations of the model box and in various locations within the model box. The soil used in the experiments was fine sand. The sand was partially saturated to generate an apparent cohesion, necessary for a rough simulation of in-situ cohesion and cementation. Other details of the 1991 testing are described by Vucetic and others (1993).

Figures 11*B* and 12 show a typical failure mechanism obtained in the tests under horizontal dynamic loads. In all four tests the failure surface never started at the ground surface above the nails. Instead, it started at the ground surface behind the ends of the nails. Figure 11*B* reveals that the failure mechanism involves three soil "zones" and two soil "blocks," with two failure surfaces, one of which consists of two parts. The primary failure surface extends from behind the nails at the ground surface down to the end of the second row of nails, at which point it changes curvature and continues down to the bottom of the excavation through the toe. The secondary failure surface develops within the sliding soil mass and divides zones 1 and 3. Such deformation patterns after the tests point to the following failure mechanism. The soil above the second row of nails in zone 1 moves horizontally under

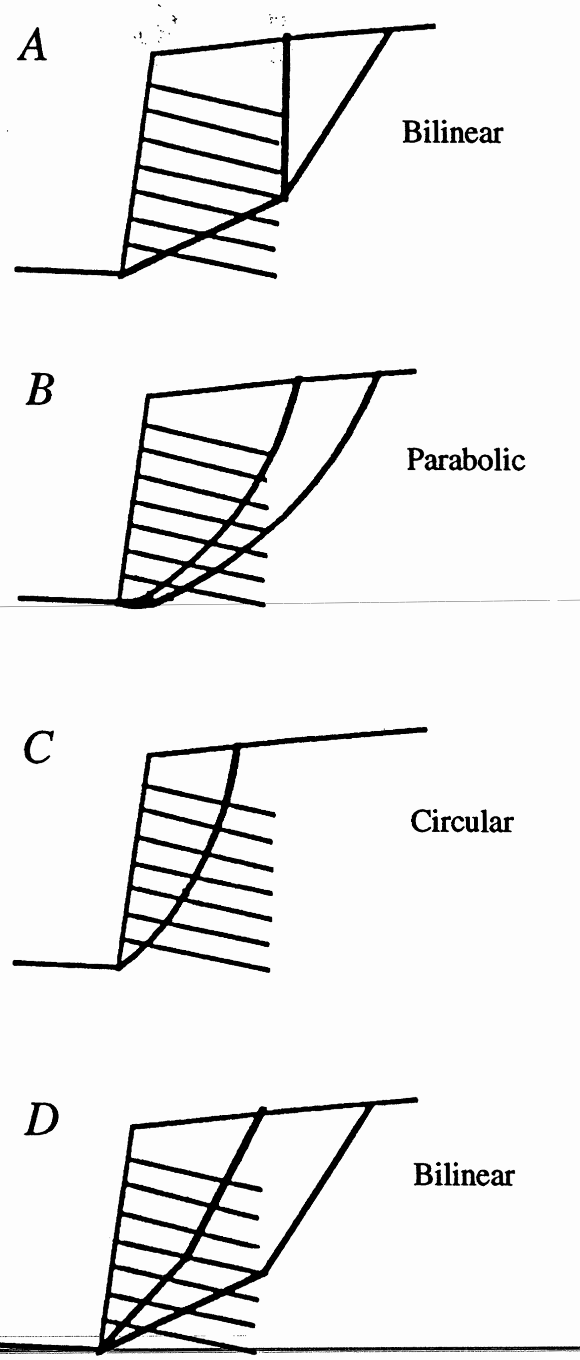


Figure 9.—Assumed failure surfaces used in analyses. *A*, German Method. *B*, Davis Method. *C*, French Method. *D*, Caltrans Method.

Table 4.—*Calculated factors of safety using the Davis method (see also Hudson, 1990)*

Horizontal acceleration coefficient, k_h	Cresta Vista apartments CVA	Temporary shoring wall TSW
0	1.31	1.19
0.1	1.14	1.06
0.2	1.00	0.94

Indicates the factors of safety corresponding to the range of estimated horizontal peak ground-surface accelerations near the site.

large inertial forces as a relatively rigid block held together by the nails. Consequently, the soil in zone 2 is pushed outward by the horizontal friction along the interface between the upper zone 1 and the lower zone 2. Accordingly, the failure surface passes through the bottom row of nails. In such a mechanism, the bottom nails obviously act as anchors between the back soil and the facing, while the top nails hold the soil together in the upper part of the excavation. As zones 1 and 2 move horizontally outward during seismic shaking, the lateral stresses in zone 3 are greatly reduced. Consequently, zone 3 represents a typical failure wedge behind a retaining wall, the retaining wall being zone 1. This mechanism and kinematics of the soil movement resemble the geometry of German method for static stability evaluation, shown in figure 9*A* (Gassler and Gudehus, 1981), while they contradict the assumption that rotation of one monolith occurs along a continuous circular or parabolic failure surface.

To examine the factors governing the failure corresponding to the above mechanism, the forces and factors of safety for the TSW Wall in Richmond (see fig. 10 and table 4) are reevaluated. Figure 13 shows the assumed failure surfaces and governing forces, while figure 14 shows the corresponding polygons of forces. To account for the effects of dynamic horizontal forces the pseudostatic method of analysis is used again, where the dynamic action is represented by the static horizontal force $H = W \times k_h$. Two definitions of the factor of safety, *FS*, based on the German

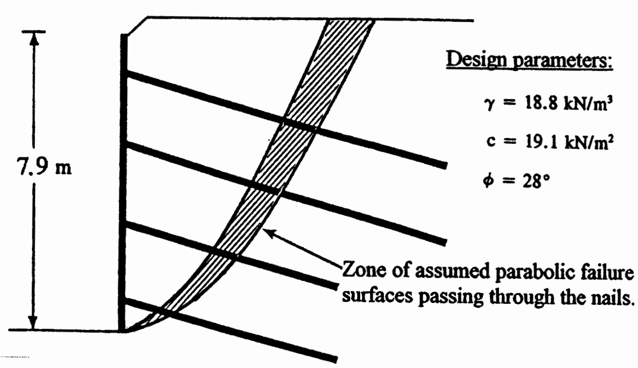


Figure 10.—Assumed failure surfaces passing through the nails for the factor of safety evaluation of the TSW wall in Richmond using Davis method and its modifications.

type of failure mechanism are considered below. First is the definition for static stability proposed by Stocker, Korber, Gassler and Gudehus (1979), which is adapted here for the dynamic stability by adding to the poligon of forces a horizontal force $H = W \times k_h$. The second is the definition proposed and used by Caltrans (1993). Accordingly, the two methods for the calculation of *FS* are called here the SKGG method and the Caltrans method.

According to the SKGG method, the factor of safety is calculated as

$$FS = \frac{Z_a}{Z_e}, \tag{4}$$

where Z_a = cumulative axial pull-out force of the nails beyond the failure surface and Z_e = mobilized cumulative axial force of the nails beyond the failure surface. Therefore, the entire factor of safety is based on the pullout of the nails. For the TSW wall, *FS* for different seismic coefficients, k_h, was calculated. The results of this calculation are presented in terms of the *FS* vs. k_h relationship in figure 15. On the same figure the equivalent relationship between *FS* and k_h obtained by the Caltrans method is presented as well.

The bilinear failure surface assumed in the Caltrans method is similar to the failure surfaces assumed in the SKGG method and thus to the deformation patterns and failure surfaces observed in the centrifuge tests. In fact, as indicated in figure 13, the forces and their positions relative to the free bodies for the TSW wall are the same for the SKGG and Caltrans methods. However, the methods differ fundamentally in their definitions of the factor of safety. The Caltrans method applies a unique factor of safety to the soil cohesion, *c*, soil friction angle, ϕ, and the cumulative nail pullout force, Z_a :

$c' = c/FS$ = mobilized cohesion,
$\phi' = \tan^{-1}[(\tan\phi)/FS]$ = mobilized friction angle, and
$Z_e = Z_a/FS$ = mobilized pullout force.

The method then utilizes these "mobilized" parameters in the force equilibrium equations to solve for the interwedge forces, *F* (see figs. 13 and 14). Since these forces must be equal in magnitude and opposite in direction, assumed value of *FS* is systematically varied until this condition is fulfilled, which then yields the corresponding *FS* used in design. The Caltrans method has been coded into a computer programming language and can be run on personal computers. The computer program is called SNAIL (Caltrans, 1993). By inputting the geometry of the slope and details of the soil strength and nail properties, the program can systematically vary the location of the bilinear failure surfaces, until the one producing the lowest factor of safety is found. The program also has an option to calculate the *FS* for a specified surface, as well as for considering seismic forces by the pseudostatic technique.

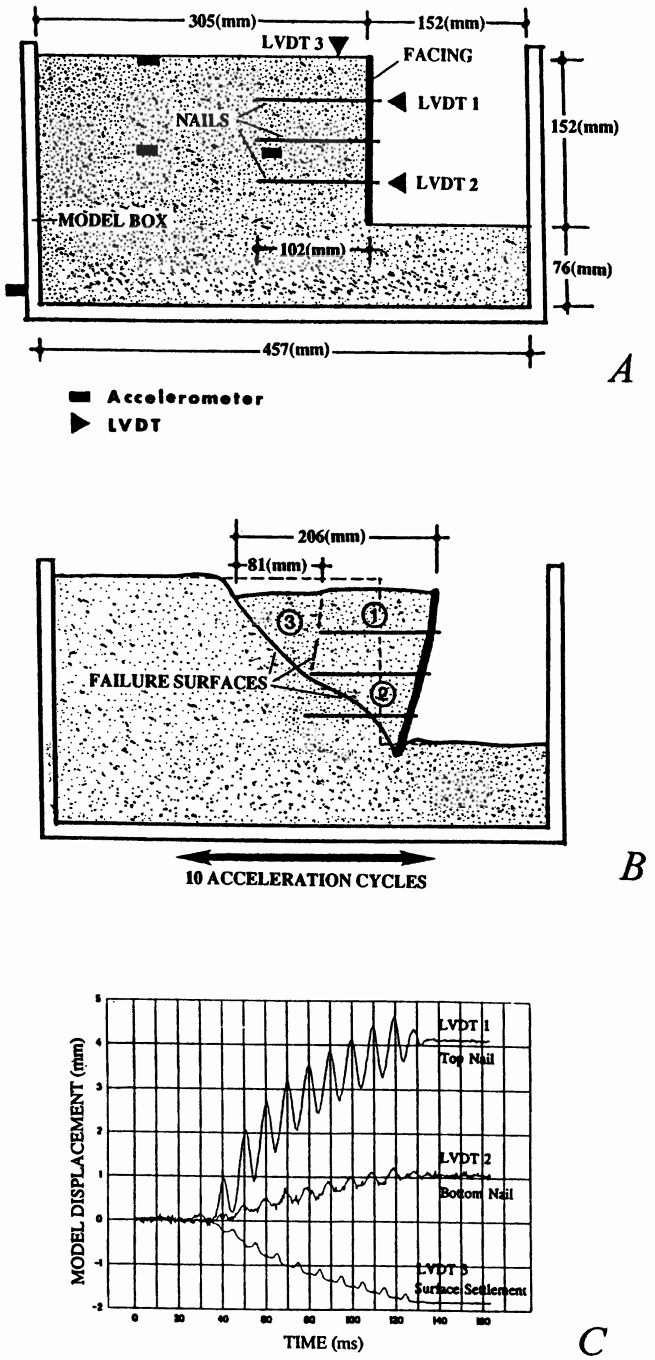


Figure 11.—Features of the typical centrifuge model test with length ratio of 0.67 (Tufenkjian and others, 1991; Vucetic and others, 1993). *A*, Longitudinal cross section of the soil-nailed excavation centrifuge model box. *B*, Failure mechanism obtained in the centrifuge due to strong horizontal shaking. *C*, Typical records of soil mass movements during shaking with 10 cycles of 0.27 *g* cyclic acceleration amplitude.

Several interesting conclusions can be derived from figure 15. First is that for $FS = 1$ (the conditions of the failure of the wall), $k_h \approx 0.37$ is obtained by both methods. Second, this k_h value is much larger than $k_h \approx 0.1$ to 0.20 corresponding to $FS = 1$ calculated according to the Davis method (see table 4). Third, $k_h \approx 0.37$ is in relatively good agreement with the amplitude of the cyclic acceleration of 0.45 g that was required in the centrifuge testing for the failure of the soil-nailed excavation model of similar length ratio (see Vucetic and others, 1993). And fourth, the *FS* versus k_h relationship for the SKGG method has a singularity point, while the same relationship for the Caltrans method does not.

Based on these observations it can be concluded that the failure mechanism according to the German method seems to be more appropriate than that of the Davis method. However, figure 15 also shows that using the SKGG method to calculate the factor of safety may not be suitable for the calculation of stability involving the horizontal forces ($W \times k_h$), because it is too sensitive to the variation of k_h. By varying k_h from 0.3 to 0.4, *FS* varies from -1 to $-\infty$ and then from $+\infty$ to 0.7—that is, as noted above, the function $FS = f(k_h)$ has a singularity point. The reason for such sensitivity of *FS* with respect to k_h can be easily understood from the polygon of forces in figure 14*B*. For example, if the force $H_1 = W_1 \times k_h$ is increased by only 15 percent, the Z_e force will double—that is, change by 100 percent. Consequently, the $FS = Z_a/Z_e$ will change dramatically too. Such sensitivity of *FS* comes from the fact that *FS* is defined on the basis of forces which are of secondary importance for the stability of the structure. In other words, force Z_e is relatively small compared to the other forces in the polygon. More dominant forces are apparently the reaction force Q_1 and cohesion force C_1 mobilized along the failure surface.

In the configuration of forces such as shown in the polygon in figure 14*B*, corresponding to the German method type of failure mechanism, the role of the nails is predominantly to interact with the soil and form the nailed block. Such a large soil block is evidently seismically very stable, and its stability is governed by the large forces of friction and cohesion at the interfaces with the surrounding soil, not by the small forces such as Z_e. This, of course, would change if Z_e is relatively large, that is, corresponding to very long nails installed deep beyond the failure surface. In such a case the kinematics of the failure would be different too. Instead of predominantly sliding along the failure surfaces, the facing and thus the soil mass would be forced to rotate around the bottom row of nails which are anchored beyond the failure surface.

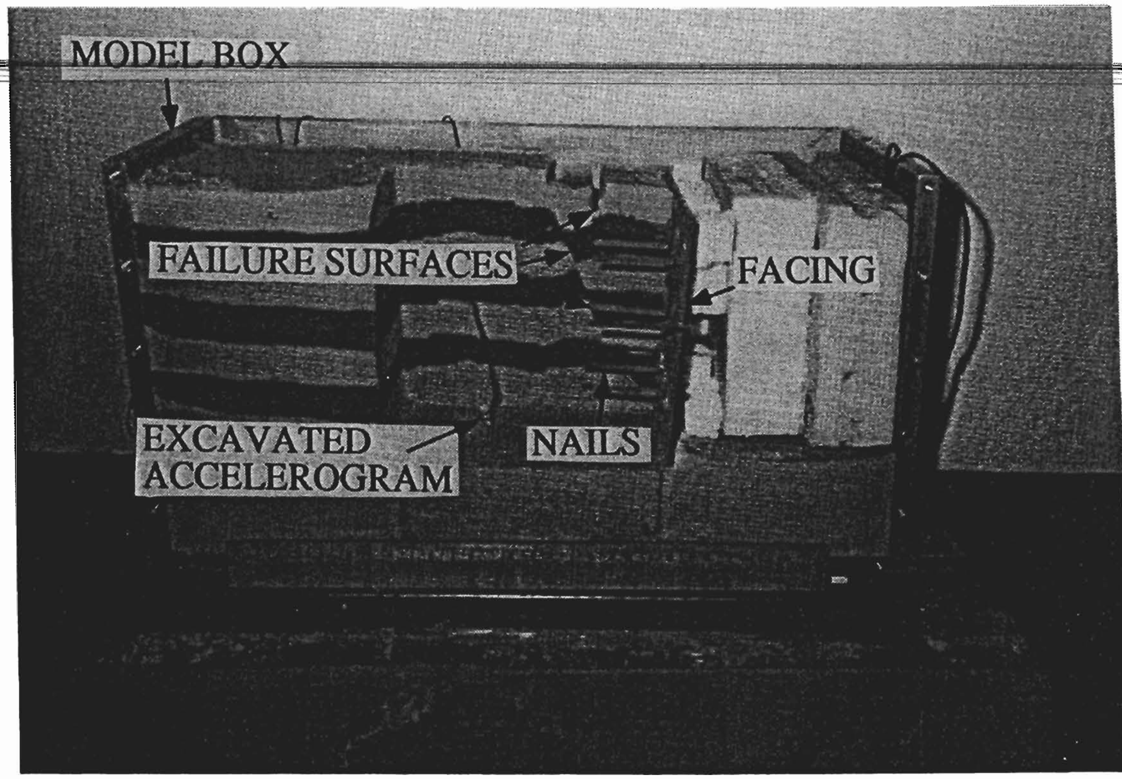


Figure 12.—Excavated model of a test with the length ratio of 0.33 (short nails) shaken by 10 cycles of the uniform acceleration amplitude of 0.10 g.

As opposed to the SKGG method of stability evaluation, the Caltrans method does incorporate all significant forces in the definition of FS and consequently yields a more meaningful relationship between FS and k_h. This, along with the above discussion, leads to the following conclusions: (1) the German-method type of failure mechanism seems to be appropriate, (2) the Caltrans method for calculating FS seems to yield appropriate and meaningful results, and (3) the Davis method used in the design of the San Francisco Bay area soil-nailed walls seems to be overly conservative, apparently because it employs an unlikely failure mechanism.

To confirm the above conclusions, the factors of safety of the eight soil-nailed walls subjected to the Loma Prieta earthquake were calculated by the Caltrans method. The four-level RPP wall located in San Jose could not be accurately reproduced using the SNAIL program. The wall geometries were scaled from figure 8 and the soil design parameters were taken from table 1. The calculated factors of safety are shown in table 5, where those corresponding to the range of estimated horizontal ground-surface accelerations near each soil-nailed wall are indicated by double-headed arrows. Note that the factors of safety are generally much greater than unity for the range of estimated ground-surface accelerations, even for the UCSC wall which was subjected to horizontal accelerations in the range of 0.4 to 0.5 g. Recall from Table 4, for example, that the Davis method predicted failure ($FS = 1.0$) for the TSW wall for the acceleration coefficient between

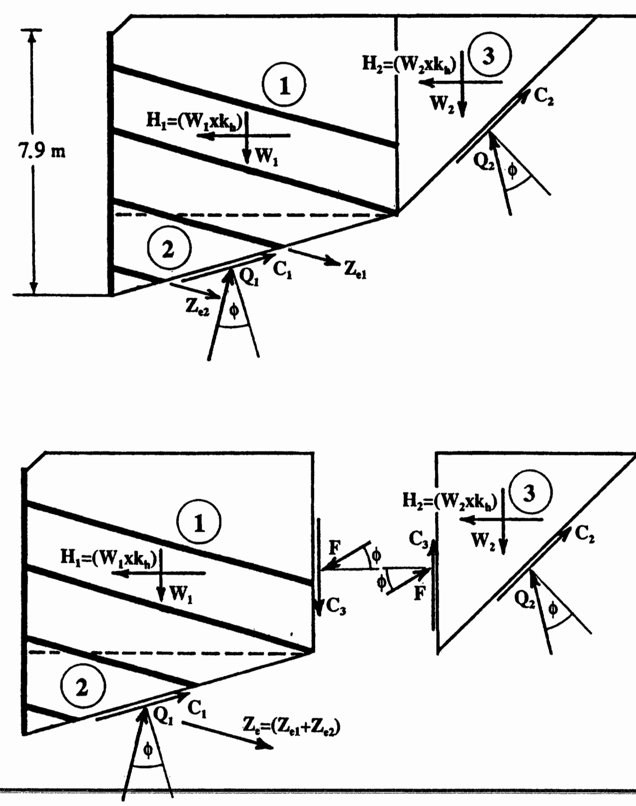


Figure 13.—Failure surfaces and forces for the TSW wall employed in both SKGG and CALTRANS methods for the calculation of the factor of safety according to the German type of failure.

0.10 and 0.20, while the Caltrans method required an acceleration coefficient between 0.4 and 0.5. Such large values of *FS* obtained by Caltrans method are in agreement with the excellent performance of the walls during the earthquake.

ROLE OF FACING

None of the methods discussed above account explicitly for the contribution of the facing in the evaluation of the global factor of safety, although they do incorporate the evaluation of punching shear around the nail connection. In other words, the factor of safety is calculated without considering axial and flexural rigidities of the facing. In that respect, there is no consensus on what the contribution of the facing to the global stability of soil nailed structure really is. However, it is obvious that stronger facing and stronger contact between the facing and the nails will make the nailed soil mass more coherent. The failure mechanism of such a coherent soil mass is likely to be of the German type—that is, behaving as a large seismically stable block. In addition, the inability of the nails (which are firmly fixed to the facing) to move freely decreases the likelihood of local failures, especially in the zones most critically stressed during construction and seismic loading. As suggested earlier, the lack of full understanding of the role of facing in the global and local stability apparently led to the difference by a factor of 3 (75 mm vs. 200 mm) in the thicknesses of the facing between the nine walls considered here.

CONCLUSIONS

Postearthquake inspections of nine soil-nailed walls following the Loma Prieta earthquake indicated superior performance and no signs of distress, even though one of the walls was subjected to horizontal accelerations probably as high as 0.4 *g*. It was shown that the excellent performance may be attributed to a conservative design, generally conservative stability analysis which is mainly the result of an

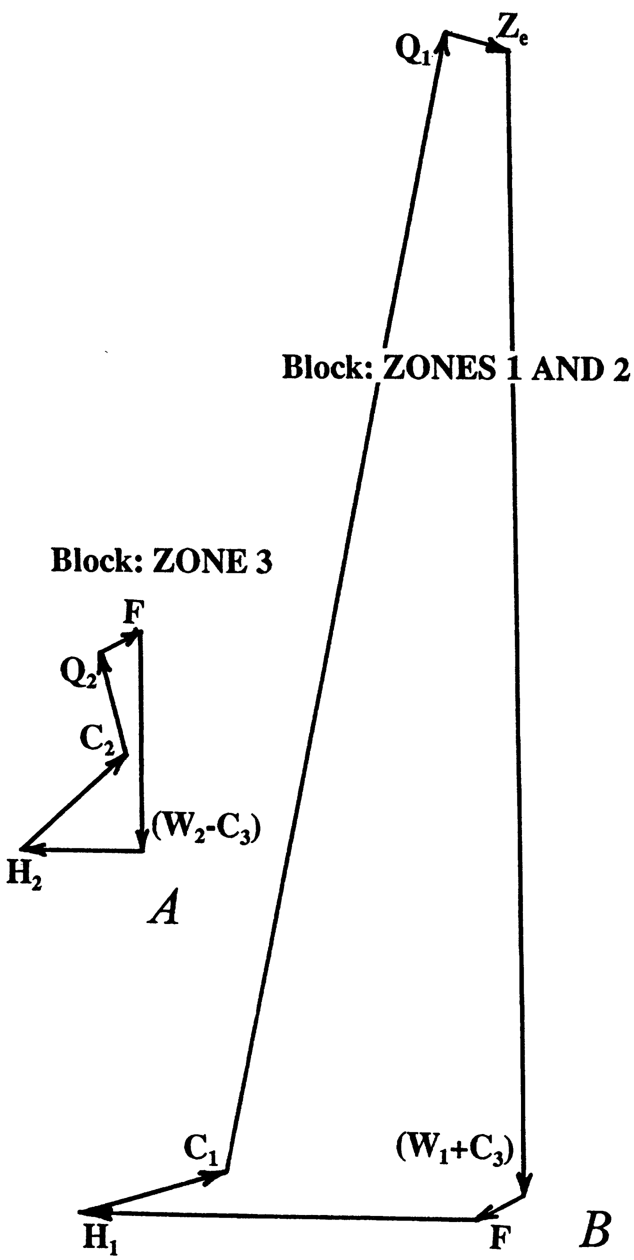


Figure 14.—Polygons of forces identified in figure 13. *A*, Polygon for block comprising zone 3. *B*, Polygon for block comprising zones 1 and 2.

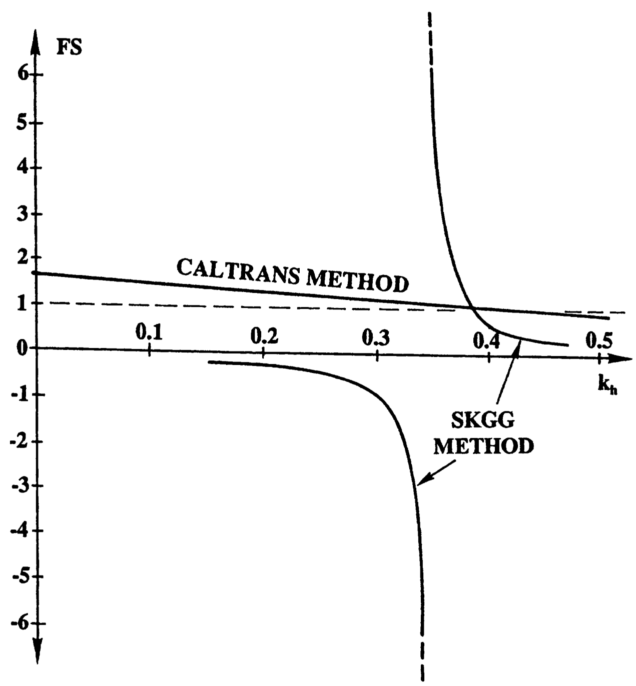


Figure 15.—Variation of the factors of safety, *FS*, with the seismic coefficient, k_h, for the TSW wall.

Table 5.—Calculated factors of safety using the Caltrans method for San Francisco area soil-nailed walls

Horizontal acceleration coefficient, k_h	Soil-nailed wall							
	ECR	KPG	UCSC	RPP (single-level)	NME	CVA	MSW	TSW
0.0	2.28	2.53	3.30	5.15	1.88	1.59	3.07	1.64
0.1	2.00	2.26	2.75	4.44	1.52	1.35	2.68	1.51
0.2	1.74	1.96	2.32	2.80	1.27	1.17	2.35	1.39
0.3	1.51	1.71	2.01	1.99	1.10	1.02	1.97	1.19
0.4	1.31	1.43	1.76	1.54	0.97	0.89	1.64	1.03
0.5	1.16	1.18	1.53	1.26	0.86	0.78	1.39	0.91

Indicates the factors of safety corresponding to the range of estimated horizontal peak ground-surface accelerations near the site.

unlikely mechanism and geometry of failure, and conservative construction.

Because seismic failures of soil-nailed excavations have not occurred in the past and are therefore absent from the literature, dynamic centrifuge testing was performed to provide evidence of the most probable failure mechanism. The centrifuge testing revealed that the most likely failure mechanism is the German type of failure mechanism. Furthermore, a simple analysis of the dynamic centrifuge test results and field observations showed that the Caltrans method for calculating the factor of safety, which also incorporates the German type of failure mechanism, yields very consistent and logical results. Accordingly, the Caltrans method implemented by the computer program SNAIL seems to be an appropriate method for calculating the static and dynamic stability of grouted soil-nailed excavations of the type discussed in this paper.

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