CHAPTER 10

Seismic Slope Stability

Earthquakes expose slopes to dynamic loads that can reduce the soil shear strength and cause instability. During the past 30 years, significant advances have been made in the understanding of earthquake ground motions, nonlinear stress–strain properties of soils, strength losses due to earthquake loading, and dynamic response analyses for earth slopes. This progress has resulted in development of sophisticated procedures for analyzing stability of slopes subjected to earthquakes. At the same time, advances have been made in the use of simpler procedures for screening analyses, to determine if more complex analyses are needed.

ANALYSIS PROCEDURES

Detailed, Comprehensive Analyses

Comprehensive analysis procedures are generally used for any large embankment or any slope or embankment where the consequences of failure are high or significant soil strength losses occur. Although the specific details and steps of these procedures may vary, the general approach that is used is as follows (Seed, 1979; Marcuson et al., 1990):

1. Determine the cross section of the slope and underlying foundation that is to be analyzed.
2. Determine, with the aid of geologists and seismologists working as a team, the anticipated acceleration–time history for the ground beneath the dam. This should account for attenuation of motion away from the causative fault and amplification of motion as waves propagate upward through foundation soils overlying the bedrock.
3. Determine the static and dynamic stress–strain properties of the natural soils and fill materials within and beneath the slope.
4. Estimate the initial static stresses in the slope or embankment prior to the earthquake. This may involve the use of static finite element analyses in which the sequence of construction is simulated, or simpler methods.
5. Perform a dynamic finite element analysis to compute the stresses and strains induced in the embankment by the earthquake acceleration–time history.
6. Estimate the reductions in shear strength and increases in pore water pressure that will result from the earthquake. The most sophisticated dynamic analyses may include computations of reductions in strength as an integral part of the dynamic analysis in step 5.
7. Compute the stability of the slope using conventional limit equilibrium procedures with the reduced shear strengths determined in step 6. This may require analyses using both undrained and drained shear strengths to determine which strengths are most critical.
8. If the analyses indicate that the slope will be stable after the earthquake, compute the permanent displacements. If strength losses due to cyclic loading are small, a *Newmark-type sliding block analysis* may be used for this purpose (Newmark, 1965). However, if strength losses are significant, other methods should be used. For example, Seed (1979) showed that pseudostatic analysis procedures did not adequately reveal the problems with large displacement for the Upper Van Norman Dam, and he used the concept of a *strain potential* to evaluate the displacements. Conceptually, a complete nonlinear finite element analysis should be able to calculate any permanent displacements in a slope or dam; however, such analyses are very complex, involve considerable

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uncertainties, and are seldom performed in practice.

The details involved in evaluation of dynamic soil properties and performing the type of dynamic response analyses outlined above are beyond the scope of this book. However, simpler procedures, to determine if detailed analyses are needed, are described in the following sections.

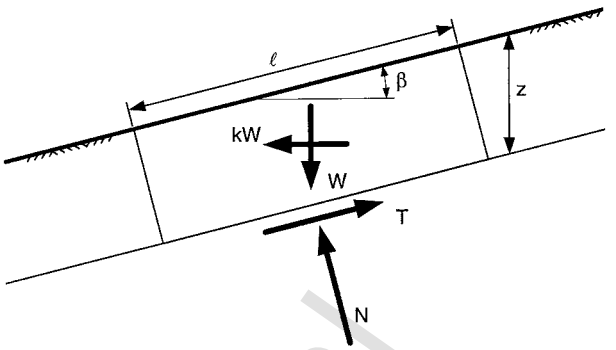
Pseudostatic Analyses

One of the earliest procedures of analysis for seismic stability is the *pseudostatic procedure,* in which the earthquake loading is represented by a static force, equal to the soil weight multiplied by a seismic coefficient, *k*. The pseudostatic force is used in a conventional limit equilibrium slope stability analysis. The seismic coefficient may be thought of loosely as an acceleration (expressed as a fraction of the acceleration, *g*, due to gravity) that is produced by the earthquake. However, the pseudostatic force is treated as a static force and acts in only one direction, whereas the earthquake accelerations act for only a short time and change direction, tending at certain instances in time to stabilize rather than destabilize the soil.

The term *pseudostatic* is a misnomer, because the approach is actually a static approach that is more correctly termed *pseudodynamic;* however, the term *pseudostatic* has been used for many years and is common in the geotechnical literature. The vertical components of the earthquake accelerations are usually neglected in the pseudostatic method, and the seismic coefficient usually represents a horizontal force.

Application of a seismic coefficient and pseudostatic force in limit equilibrium slope stability analyses is relatively straightforward from the perspective of the mechanics: The pseudostatic force is assumed to be a known force and is included in the various equilibrium equations. This is illustrated in Figure 10.1 for an infinite slope with the shear strength expressed in terms of total stresses. Similar equations can be derived for effective stresses and for other limit equilibrium procedures, including any of the procedures of slices discussed in Chapter 6.

An issue that arises in pseudostatic analyses is the location of the pseudostatic force. Terzaghi (1950) suggested that the pseudostatic force should act through the center of gravity of each slice or the entire sliding soil mass. This would be true only if the accelerations were constant over the entire soil mass, which they probably are not. Seed (1979) showed that the location assumed for the seismic force can have a small but noticeable effect on the computed factor of safety: For the Sheffield Dam, changing the location of the pseudostatic force from the centers of gravity to the bottoms of the slices reduced the factor of safety from 1.32 to 1.21 for a seismic coefficient of 0.1.



Resolving forces perpendicular to slip plane:

$$N = W\cos\beta - kW\sin\beta \quad (1)$$

Resolving force parallel to slip plane:

$$T = W\sin\beta + kW\cos\beta \quad (2)$$

Weight of sliding block:

$$W = \gamma \ell z\cos\beta \quad (3)$$

Substituting (3) into (1) and (2):

$$N = \gamma \ell z\cos^2\beta - k\gamma \ell z\cos\beta\sin\beta \quad (4)$$

$$T = \gamma \ell z\cos\beta\sin\beta + k\gamma \ell z\cos^2\beta \quad (5)$$

For the stresses on the slip plane:

$$\sigma = \frac{N}{\ell} = \gamma z\cos^2\beta - k\gamma z\cos\beta\sin\beta \quad (6)$$

$$\tau = \gamma z\cos\beta\sin\beta + k\gamma z\cos^2\beta \quad (7)$$

Finally, for the factor of safety (total stresses):

$$F = \frac{s}{\tau} = \frac{c + \sigma\tan\phi}{\tau} = \frac{c + (\gamma z\cos^2\beta - k\gamma z\cos\beta\sin\beta)\tan\phi}{\gamma z\cos\beta\sin\beta + k\gamma z\cos^2\beta} \quad (9)$$

Figure 10.1 Derivation of the equation for the factor of safety of an infinite slope with a seismic force (kW)—total stress analyses.

Dynamic analyses of the response of many dams to earthquakes (Makdisi and Seed, 1978) indicate that peak accelerations increase (i.e., they are amplified) from the bottom to the top of a dam. Thus, the location of the resultant seismic force would be expected to be *above* the center of gravity of the slice. In the case of circular slip surfaces, this would reduce the moment about the center of the circle due to the seismic forces, in comparison to applying the force at the center of gravity of the slice, and the factor of safety would be expected to *increase*. This reasoning is consistent with the results of Seed (1979) for the Sheffield Dam, which showed that the factor of safety *decreased* when the seismic force was located *below* the center of gravity

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of the slice. Assuming that the pseudostatic force acts through the center of gravity of the slice is probably slightly conservative for most dams. Thus, it appears that Terzaghi's suggestion is reasonable. For most pseudostatic analyses the pseudostatic force is assumed to act through the center of gravity of each slice. If a force equilibrium (only) procedure is used, the location of the pseudostatic force has no effect on the factor of safety computed.

For many years, seismic coefficients were estimated based on empirical guidelines and codes. Typical values for seismic coefficients used ranged from about 0.05 to about 0.25 (Seed, 1979; Hynes-Griffin and Franklin, 1984; Kavazanjian et al., 1997). However, with the development of more sophisticated analyses, particularly displacement analyses such as the sliding block analyses described in the next section, correlations can be made between the seismic coefficient, the expected earthquake accelerations, and the probable displacements. Most seismic coefficients used today are based on experience and results from deformation analyses.

Sliding Block Analyses

Newmark (1965) first suggested a relatively simple deformation analysis based on a rigid sliding block. In this approach the displacement of a mass of soil above a slip surface is modeled as a rigid block of soil sliding on a plane surface (Figure 10.2). When the acceleration of the block exceeds a yield acceleration, a_y, the block begins to slip along the plane. Any acceleration that exceeds the yield acceleration causes the block to slip and imparts a velocity to the block relative to the velocity of the underlying mass. The block continues to move after the acceleration falls below the yield acceleration. Movement continues until the velocity of the block relative to the underlying mass goes to zero, as shown in Figure 10.3. The block will slip again if the acceleration again exceeds the yield acceleration. This stick-slip pattern of motion continues until the accelerations fall below the yield acceleration and the relative velocity drops to zero for the last time. To compute displacements, the accelerations in excess of the yield acceleration are integrated once to compute the velocities and a second time to compute the displacements (Figure 10.3). Given an acceleration–time history and yield acceleration, the integration can be performed numerically. Movements in the upslope direction are neglected (i.e., all displacements are assumed to be "one-way"). Details are beyond the scope of this chapter but can be found in the literature (e.g., Jibson, 1993; Kramer, 1996; Kramer and Smith, 1997).

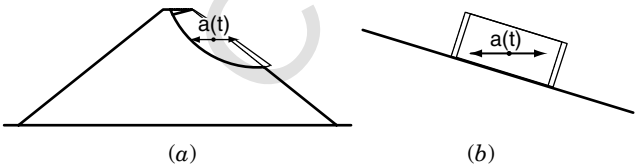


Figure 10.2 (*a*) Actual slope; (*b*) sliding block representation used to compute permanent soil displacements in a slope subjected to earthquake shaking.

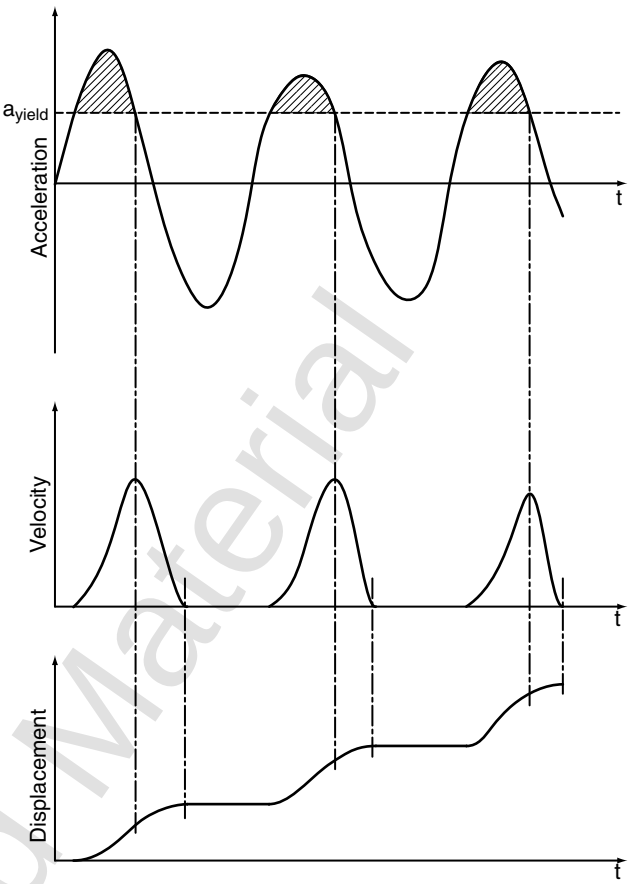


Figure 10.3 Double integration of acceleration–time history to compute permanent displacements.

Limit equilibrium slope stability analyses are used to compute the values of yield acceleration, a_y, used in sliding block analyses. The yield acceleration is usually expressed as a seismic yield coefficient, $k_y = a_y/g$. The seismic yield coefficient is the seismic coefficient that produces a factor of safety of unity in a pseudostatic slope stability analysis.

The value of k_y is determined using conventional slope stability analysis procedures. However, rather than searching for the slip surface that gives the minimum static factor of safety, searches are conducted to

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find the slip surface that gives the minimum value of k_y. The slip surface giving the minimum value of k_y is usually different from the one giving the minimum factor of safety for static conditions.

Recapitulation

- Comprehensive analysis procedures exist and are used to evaluate seismic stability for large embankments and slopes where the consequences of failure are high or significant soil strength losses are anticipated.
- Pseudostatic analysis procedures approximate the earthquake loads as a static force and are less accurate than other procedures, but useful as a screening tool.
- The pseudostatic seismic force is usually assumed to act through the center of gravity of the soil mass, and this assumption seems reasonable.
- Newmark-type sliding block analyses provide a simple way of estimating permanent displacements in a slope caused by an earthquake.
- Improvements have been made in the criteria for selecting seismic coefficients for pseudostatic analyses through studies of embankment displacements using sliding block analyses and case studies.

PSEUDOSTATIC SCREENING ANALYSES

Pseudostatic analyses provide a useful way of screening for potential seismic stability problems, especially when the soils involved are not expected to lose a significant amount of their strength due to the earthquake. Makdisi and Seed (1977) found that for clayey soils, dry or partially saturated cohesionless soils, or very dense saturated cohesionless soils, 80% of the static undrained strength represents an approximate threshold between large and small strains induced by cyclic loading. Substantial permanent strains may be produced when these nonliquefiable soils are subjected to cyclic loads near their full undrained strengths. Essentially elastic behavior was observed when these same soils were subjected to large numbers of cycles (>100) at 80% of their undrained strengths. Accordingly, Makdisi and Seed (1977) recommended the use of 80% of the static undrained strength as the dynamic yield strength for nonliquefiable soils.

Use of pseudostatic analyses as screening analyses can be a simple process. A suitable seismic coefficient is determined based on an appropriate criterion and the factor of safety is computed. The computed factor of safety provides an indication of the possible magnitude of seismically induced displacements. Criteria for selection of seismic coefficients and for determining what are acceptable factors of safety have been developed by Makdisi and Seed (1978), Hynes-Griffin and Franklin (1984), Bray et al. (1998), and Kavazanjian et al. (1997) by comparing the results of pseudostatic analyses with field experience and the results of deformation analyses.

Several methods for using pseudostatic analyses to determine the need for more detailed studies are summarized in Table 10.1. Each involves these components:

1. *A reference peak acceleration,* a_{ref}. The reference accelerations used are either the peak acceleration in bedrock beneath the slope, or the peak soil acceleration at the top of the slope. Peak bedrock acceleration is easier to use, because determining peak acceleration at the top of the slope requires a dynamic response analysis.
2. *Acceleration multiplier.* The seismic coefficient used in the pseudostatic analysis is equal to a_{ref}/g multiplied by an acceleration multiplier, a/a_{ref} [$k = (a_{ref}/g)(a/a_{ref})$]. Values of acceleration multiplier ranging from 0.17 to 0.75 have been recommended, as shown in Table 10.1.
3. *Shear strength reduction factor.* Most authorities recommend using reduced shear strengths in pseudostatic analyses. As shown in Table 10.1, the strength most often recommended is 80% of the static shear strength, following the findings of Makdisi and Seed (1977). For landfills with geosynthetic liners, Bray et al. (1998) recommended using residual strengths because the geosynthetic interface strength is reached at small deformations that usually are exceeded during landfill construction.
4. *Minimum factor of safety.* All of the screening criteria summarized in Table 10.1 stipulate a minimum acceptable factor of safety. The values are either 1.0 or 1.15.
5. *Tolerable permanent deformation.* Each set of criteria summarized in Table 10.1 carries with it the notion that a certain amount of earthquake-induced deformation is tolerable. The magnitudes of deformation judged to be tolerable vary from 0.15 m in the case of landfill base liners to 1.0 m for dams.

Each of the suggested methods outlined in Table 10.1 is complete within itself, and should be viewed in this way: If a pseudostatic analysis using the specified reference acceleration shown in column (2), the acceleration multiplier shown in column (3), and the strength reduction factor shown in column (4) results

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Table 10.1 Suggested Methods for Performing Pseudostatic Screening Analyses

(1) Reference	(2) Reference acceleration, a_{ref}	(3) Acceleration multiplier, a/a_{ref}	(4) Strength reduction factor	(5) Minimum factor of safety	(6) Tolerable displacement
Makdisi and Seed (1978)	0.2 g ($M \approx 6\frac{1}{2}$)	0.5	0.8	1.15	Approx. 1 m
Makdisi and Seed (1978)	0.75 g ($M \approx 8\frac{1}{4}$)	0.2	0.8	1.15	Approx. 1 m
Hynes-Griffin and Franklin (1984)	PHA_{rock}	0.5	0.8	1.0	1 m
Bray et al. (1998)	PHA_{rock}	0.75	Recommend using conservative (e.g., residual) strengths	1.0	0.30 m for landfill covers; 0.15 m for landfill base sliding
Kavazanjian et al. (1997)	PHA_{soil}	0.17 if response analysis is performed	0.8[a]	1.0	1 m
Kavazanjian et al. (1997)	PHA_{soil}	0.5 if response analysis is not performed	0.8[a]	1.0	1 m

[a]For fully saturated or sensitive clays.

in a factor of safety greater than or equal to the value shown in column (5), this indicates that the permanent displacements induced by the earthquake will not be larger than those shown in column (6).

Although the methods summarized in Table 10.1 differ with regard to details, they employ the same procedure for screening conditions that would lead to development of large permanent seismically induced displacements. It will be noted that the methods that are more stringent with respect to seismic coefficient use tighter criteria for displacements that are considered tolerable.

The criteria by Hynes-Griffin and Franklin (1984) were developed for earth dams and reflect results of extensive analyses not reflected in Makdisi and Seed's (1978) criteria. The criteria by Bray et al. (1998) are applicable to landfills, and like Hynes-Griffin and Franklin's criteria, reflect the results of deformation analyses. The criteria developed by Hynes-Griffin and Franklin and by Bray et al. are based on peak horizontal bedrock acceleration, PHA_{rock}, and do not require site response analyses. However, the criteria are more conservative than those proposed by Kavazanjian et al. (1997), which do require response analysis. The additional effort involved in performing a response analysis can be weighed against the increased conservatism required if a site response analysis is not performed. Simplified procedures for estimating site response have been discussed by Kavazanjian et al. (1997).

Recapitulation

- Several simple screening criteria have been developed for evaluating seismic stability using pseudostatic analysis procedures.
- The screening criteria differ in the reference seismic acceleration, acceleration multiplier, strength reduction factor, acceptable factor of safety, and tolerable displacement criterion used.

DETERMINING PEAK ACCELERATIONS

Peak accelerations can be determined for highly seismic areas, where extensive data from previous earthquakes has been accumulated, using empirical attenuation relations. For sites where less information is available, peak accelerations can be established using

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the U.S. Geological Survey Geohazards Internet Web site (*http://eqhazmaps.usgs.gov/*). The Web site provides peak rock acceleration (PGA_{rock}) based on latitude and longitude or zip code. Example results are shown in Table 10.2.

SHEAR STRENGTH FOR PSEUDOSTATIC ANALYSES

The shear strength appropriate for use in a pseudostatic analysis depends on whether the analysis is being performed for short-term (end-of-construction) conditions or for a slope that has been in existence for many years. Pseudostatic analyses may need to be performed for both short- and long-term conditions depending on the particular slope.

Because seismic loading is of short duration, it is reasonable to assume that except for some coarse gravels and cobbles, the soil will not drain appreciably during the period of earthquake shaking. Thus, undrained shear strengths are used for most pseudostatic analyses (with the exception noted later of soils that tend to dilate when sheared and may lose strength after the earthquake as they drain).

Earthquakes Immediately after Construction

Pseudostatic analyses for short-term stability are only appropriate for new slopes. Undrained shear strength can be evaluated using conventional unconsolidated–undrained testing procedures and samples identical to the ones that would be tested to determine the shear strength for static conditions. The analyses are performed using shear strengths expressed in terms of total stresses.

Earthquakes After the Slope Has Reached Consolidated Equilibrium

All slopes that will be subjected to earthquakes should be evaluated for long-term stability using values of undrained shear strength that reflect the eventual long-term conditions, including consolidation or swell after the slope is constructed. The manner in which the undrained shear strength is determined for this condition depends on whether we are dealing with an existing slope or a slope that is yet to be built.

Existing slopes. If a slope has reached consolidated equilibrium, the shear strength can be determined by taking representative samples of the soil and performing tests using unconsolidated–undrained testing procedures. The stability analysis is then performed much like a short-term stability analysis, using shear strength parameters expressed in terms of total stresses.

New slopes. For new slopes it is necessary to simulate the effects of future consolidation and swell in the laboratory using consolidated–undrained testing procedures (Seed, 1966). The testing and analysis procedures are almost identical to those described in Chapter 9 for rapid drawdown: The soil is first consolidated to a state of effective stress and then sheared with no drainage. The difference between rapid drawdown and seismic loading in a pseudostatic analysis is that the undrained loading for rapid drawdown is due to lowering the water level adjacent to the slope, while the undrained loading for an earthquake is caused by seismic forces.

Once the appropriate shear strength envelopes are determined from the results of consolidated–undrained triaxial shear tests, the slope stability computations are performed using a two-stage analysis procedure nearly identical to the procedures described in Chapter 9 for rapid drawdown. A first-stage analysis is performed for conditions prior to the earthquake (no seismic coefficient) to compute the consolidation stresses, σ'_{fc} and τ_{fc}. These stresses are then used to estimate the undrained shear strength for seismic loading using the same procedures as for rapid drawdown. The undrained shear strength is then used in the second-stage computations (with seismic coefficient) to compute the pseudostatic factor of safety for the slope. For rapid drawdown a third stage of computations is performed to account for the likelihood of drainage during drawdown, but drainage *during* an earthquake is much less likely, and thus a third stage of computations is usually

Table 10.2 Peak Rock Acceleration Results

Zip code	PGA_{rock} with 10% probability of excedence in 50 years (500-year return period)	PGA_{rock} with 5% probability of excedence in 50 years (1000-year return period)	PGA_{rock} with 2% probability of excedence in 50 years (2500-year return period)
24060	0.054 g	0.097 g	0.194 g
78712	0.012 g	0.021 g	0.039 g

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not necessary. However, drainage *after* the earthquake could adversely affect stability and should be considered as discussed in the final section of this chapter.

Simplified procedure (*R* envelope and single-stage analysis). Although the two-stage analysis procedure described above is the proper way to perform a pseudostatic analysis for a slope, analyses are sometimes performed using a simple single-stage procedure. In the single-stage procedure the *R* shear strength envelope is used. The *R envelope,* as it is called in U.S. Army Corps of Engineers' terminology, is obtained by plotting results of consolidated–undrained triaxial tests as shown on the Mohr–Coulomb diagram in Figure 10.4. In this diagram circles are plotted as follows: The minor principal stress is the effective stress, σ'_{3c}, *at consolidation;* the diameter of the circle is the principal stress difference *at failure,* $\sigma_{1f} - \sigma_{3f}$. Because the two stresses used to plot the circles (σ'_{3c} and $\sigma_{1f} - \sigma_{3f}$) exist at different times during a test, the circles are not actually Mohr circles—a Mohr's circle represents the state of stress at a point *at an instant in time* (e.g., at consolidation or at failure). Similarly the *R* envelope drawn on such a diagram is not actually a Mohr–Coulomb failure envelope. However, the envelope that is drawn on such a diagram is often quite similar to the corresponding τ_{ff} vs. σ'_{fc} envelope that is plotted from consolidated–undrained tests described in Chapter 9. To illustrate the similarities between the *R* and τ_{ff} vs. σ'_{fc} envelopes, the two envelopes are plotted in Figure 10.5 for four different soils. The four soils and properties are summarized in Table 10.3. It can be seen that the *R* envelope in each case plots below the τ_{ff} vs. σ'_{fc} envelope. There may be other cases where the *R* envelope lies above the τ_{ff} vs. σ'_{fc} envelope.

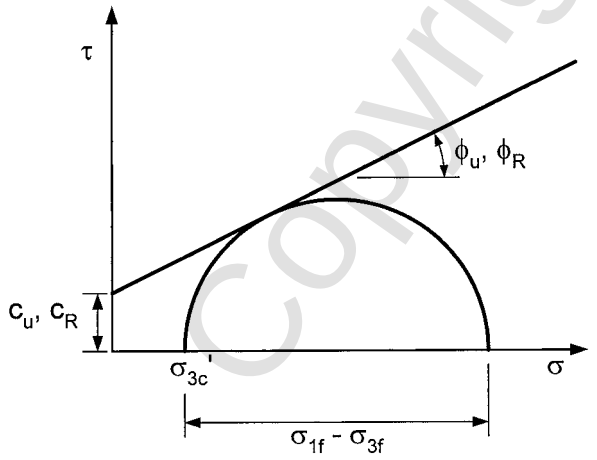


Figure 10.4 *R* (total stress) undrained shear strength envelope plotted from consolidated–undrained triaxial compression tests.

In the simplified pseudostatic procedure a single set of computations is performed for each trial slip surface using the appropriate seismic coefficient and the intercept and slope (c_R and ϕ_R) of the *R* envelope as Mohr–Coulomb shear strength parameters. In this simplified approach it is important to use the proper pore water pressures: The *R* envelope in this case is being used as an envelope approximating the relationship between the undrained shear strength ($\approx\tau_{ff}$) and effective consolidation pressure ($\approx\sigma'_{fc}$). Thus, pore water pressures equal to those during consolidation (e.g., for steady-state seepage) should be used in the computations.[1]

To illustrate the differences between the simple single-stage procedure using the *R* envelope and the more rigorous two-stage procedure such as the one described earlier, a pseudostatic slope stability analysis was performed for the slope shown in Figure 10.6. A seismic coefficient of 0.15 was used and computations were performed for the slope with both dry (zero pore water pressure) and fully submerged conditions prior to earthquake loading. Computations were performed using each of the four different sets of soil strength properties shown in Figure 10.4 and Table 10.3. Results of the computations are summarized in Table 10.4 and plotted in Figure 10.7. In all cases the factor of safety computed by the simpler procedure is less than the factor of safety computed by the more rigorous two-stage procedure. The factor of safety from the single-stage procedure varies between about 80 and 90% of the value from the two-stage analysis.

Recapitulation

- Unconsolidated–undrained tests performed on undisturbed or laboratory compacted specimens can be used to determine the shear strength for pseudostatic analyses of *existing slopes* or *new slopes at the end of construction.*
- To determine the shear strength for *new slopes after the slope has reached consolidated equilibrium,* shear strengths are determined using consolidated–undrained testing procedures and analyses are performed as two-stage analyses using procedures similar to those used for rapid drawdown.

[1] Although the *R* envelope is called a *total stress envelope* in many texts (Terzaghi and Peck, 1967; Peck et al., 1974; Wu, 1976; Sowers, 1979; Dunn et al., 1980; Holtz and Kovacs, 1981; Lee et al., 1983; McCarty, 1993; Liu and Evett, 2001; Abramson et al., 2002; Das, 2002), when the envelope is used in pseudostatic slope stability computations, effective stresses and appropriate pore water pressures must be used.

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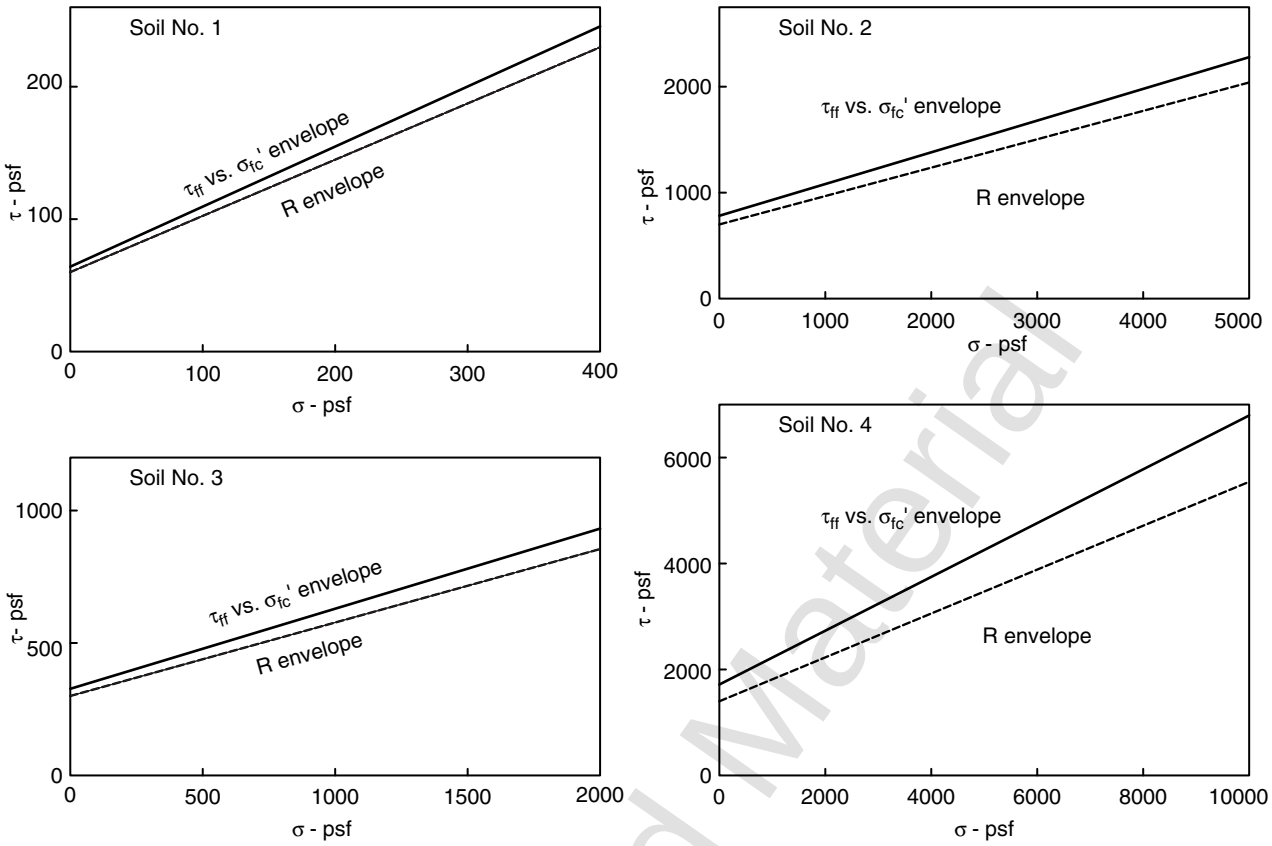


Figure 10.5 Comparison of τ_{ff}-σ'_{fc} shear strength envelope with R (total stress) envelope for undrained shear.

- A simplified single-stage analysis procedure using the R strength envelope appears in many cases to produce conservative estimates for the pseudostatic factor of safety after the slope has reached consolidated equilibrium.
- Reductions in strength of up to 20% caused by cyclic loading during an earthquake are probably offset by the effects of a higher loading rate during an earthquake compared to normal loading rates in static tests.

Effects of Rapid Load Application

Pseudostatic analysis procedures are appropriate only for cases involving soils that do not lose significant strength during an earthquake. Several of the screening guidelines summarized in Table 10.1 allow for moderate strength losses by using a nominal strength reduction factor. However, even if no such factor is used, strength losses of no more than 15 to 20% can probably be safely ignored in selecting shear strength for pseudostatic analyses because of strain rate effects. Most soils that are subjected to undrained loading at the rates imposed by earthquakes will exhibit strengths that are 20 to 50% higher than the shear strength measured in conventional static loading tests where the time to failure is several minutes or longer. Soils typically show an increase in undrained shear strength of 5 to 25% per tenfold increase in strain rate (decrease in time to failure). Considering earthquake loading with a period of 1 s, the time to increase the load from zero to the peak would be approximately 0.25 s. If a static test is performed with a time to failure of 10 minutes (600 s), and the shear strength of the soil increases 10% per tenfold decrease in the time to failure, the effect of strain rate on the strength during an earthquake would be expected to be about 34% higher:

$$10\% \times \log_{10} \frac{600}{0.25} = 34\%$$

Such increases in strength due to load rate effects will offset a 15 to 20% reduction in strength due to cyclic loading. Thus, there is some basis for not reducing the shear strength used in pseudostatic analyses, provided, of course, that the analysis is only being

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Table 10.3 Summary of Soil Properties Used in Comparison of R and τ_{ff} vs. σ'_{fc} Strength Envelopes

Soil no.	Description and reference	Index properties	c' (psf)	ϕ' (deg)	c_R (psf)	ϕ_R (deg)	d^a (psf)	ψ^b (deg)
1	Sandy clay (CL) material from Pilarcitos Dam; envelope for low (0–10 psi) confining pressures. (Wong et al., 1983)	Percent minus No. 200: 60–70 Liquid limit: 45 Plasticity index: 23	0	45	60	23	64	24.4
2	Brown sandy clay from dam site in Rio Blanco, Colorado (Wong et al., 1983)	Percent minus No. 200: 25 Liquid limit: 34 Plasticity index: 12	200	31	700	15	782	16.7
3	Same as soil 1 except envelope fit to 0–100 psi range in confining pressure (Wong et al., 1983)	Percent minus No. 200: 60–70 Liquid limit: 45 Plasticity index: 23	0	34	300	15.5	327	16.8
4	Hirfanli Dam fill material (Lowe and Karafiath, 1960)	Percent minus No. 200: 82 Liquid limit: 32.4 Plastic limit: 19.4	0	35	1400	22.5	1716	26.9

[a] Intercept of τ_{ff} vs. σ'_{fc} envelope—can be calculated knowing c', ϕ', c_R, and ϕ_R.
[b] Slope of τ_{ff} vs. σ'_{fc} envelope—can be calculated knowing c', ϕ', c_R, and ϕ_R.

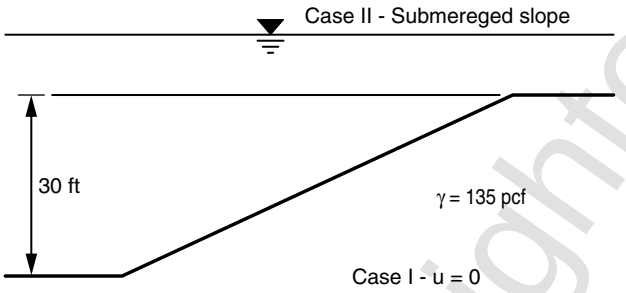


Figure 10.6 Slope used to compare simple, single-stage and rigorous, two-stage pseudostatic analyses.

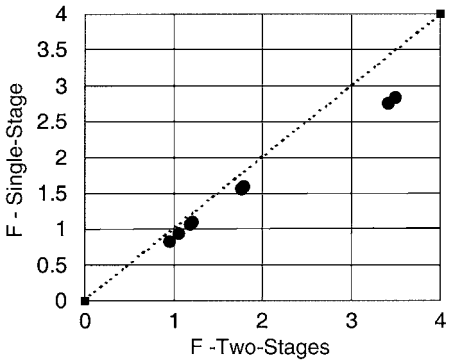


Figure 10.7 Comparison of factors of safety by simplified single-stage pseudostatic and more rigorous two-stage pseudostatic analyses.

Table 10.4 Summary of Pseudostatic Safety Factors Computed Using Simple Single-Stage and Rigorous Two-Stage Procedures

	Case I: dry slope		Case II: submerged slope	
Soil	Single-stage analysis	Two-stage analysis	Single-stage analysis	Two-stage analysis
1	0.95	1.06	0.83	0.95
2	1.56	1.77	1.59	1.79
3	1.07	1.19	1.10	1.21
4	2.76	3.42	2.83	3.49

used for cases where significant (more than 15 to 20%) strength losses are not anticipated.

POSTEARTHQUAKE STABILITY ANALYSES

Following an earthquake, the stability of a slope may be diminished because cyclic loading has reduced the shear strength of the soil. The reductions in shear strength are generally treated differently depending on whether or not liquefaction occurs. Stability follow-

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ing an earthquake can be evaluated using a three-step process.

Step 1. Estimate if Liquefaction Will Occur

The first step in evaluating strength losses is to determine if the soil will liquefy. The procedures for doing this are semiempirical, based on results of field tests and case histories, mostly for horizontal ground. According to Youd et al. (2001) four different field tests are suitable for measuring soil resistance to liquefaction: (1) cone penetration tests, (2) Standard Penetration tests, (3) shear-wave velocity measurements, and (4) for gravelly sites, the Becker penetration test. Various correlations have been developed that relate the resistance or stiffness characteristics of the soil measured in these tests to the cyclic shear stresses required to cause liquefaction. The cyclic shear stresses required to cause liquefaction are generally expressed as a normalized ratio of cyclic shear stress to effective vertical consolidation pressure, $\tau_{cyclic}/\sigma'_{vo}$, known as the *cyclic resistance ratio* (CRR). Based on one or more of the field tests described above, an estimate is made of the cyclic resistance ratio using appropriate correlations. The cyclic resistance is then compared with the seismically induced *seismic stress ratio* (CSR) to determine if liquefaction will occur.

Step 2. Estimate Reduced Undrained Shear Strengths

If the soil is expected to liquefy, reduced values of the undrained residual shear strengths are estimated.[2] Seed and Harder (1990) suggested the relationship shown in Figure 10.8 between the undrained residual shear strength and the corrected standard penetration resistance, $(N_1)_{60}$. Poulos, Castro, and their co-workers have proposed an alternative approach based on the concept of *steady-state shear strength* (Poulos et al., 1985). The steady-state strength is estimated based on the void ratio in the field and a relationship between steady-state strength and void ratio determined in the laboratory. A third approach to determining the undrained residual shear strength has been proposed by Stark and Mesri (1992) and Olson and Stark (2002). They have proposed determining an equivalent c/p ratio representing the ratio of the undrained residual shear strength to the effective preearthquake consolidation pressure. Stark and his co-workers have developed empirical correlations that relate values of c/p to Standard

[2] *Undrained residual shear strength,* should be distinguished from *residual shear strength,* used to describe the long-term "drained" shear strength of soils that have previously experienced large static shear strains.

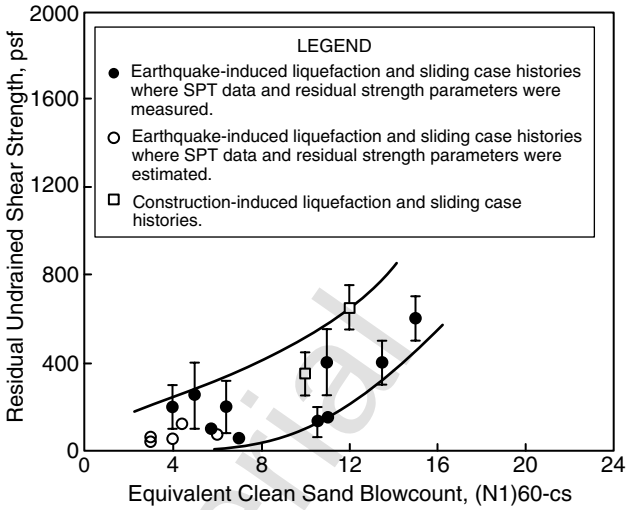


Figure 10.8 Relationship between corrected "clean sand" blowcount (N1)60-cs and undrained residual strength from case studies. (After Seed and Harder, 1990.)

Penetration resistance (Figure 10.9) and cone penetration resistance (Figure 10.10).

Although a soil may not liquefy during an earthquake, it is possible that pore water pressures will increase in the soil and the shear strength may be reduced. Marcuson et al. (1990) suggest that in this case the pore water pressures due to the earthquake can be related to the factor of safety against liquefaction, defined as the cyclic shear stress divided by the cyclic shear stress required to cause liquefaction (based on estimates of the cyclic resistance ratio described earlier). Marcuson et al. present the curves shown in Figure 10.11 for estimating the residual excess pore water pressures. However, care must be exercised in using such curves and defining pore water pressures that will be used in an effective stress representation of shear strength. It is possible that the shear strength corresponding to an effective stress analysis will actually be greater than the original undrained shear strength of the soil because the pore water pressures that are estimated as residual values may not be as large as the pore water pressures when the soil is sheared to failure with no drainage. Accordingly, it is recommended that if pore water pressures are estimated and used in an effective stress analysis, a check be made to ensure that the shear strength does not exceed the undrained shear strength before the earthquake.

As an alternative to the effective stress approach suggested by Marcuson for soils that lose some strength but do not liquefy during an earthquake, reduced values of undrained shear strength can be used.

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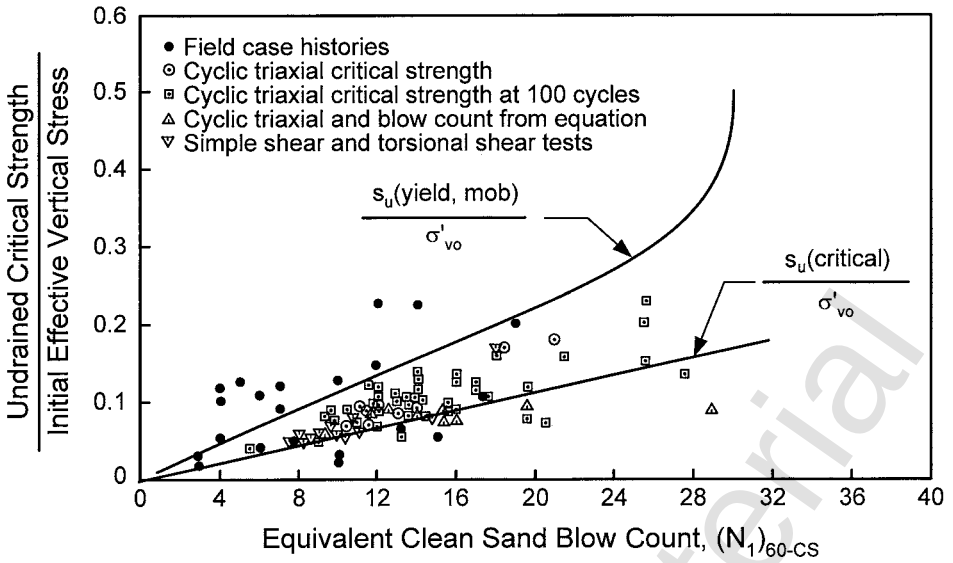


Figure 10.9 Relationship between undrained critical strength ratio and equivalent clean sand blow count. (From Stark and Mesri, 1992.)

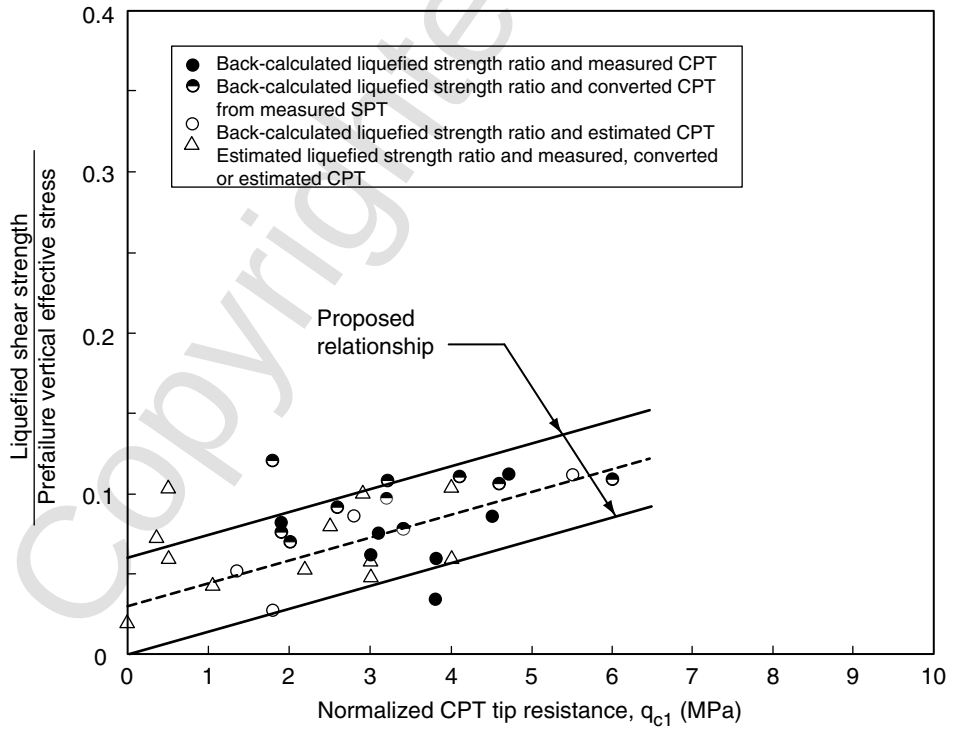


Figure 10.10 Liquefied strength ratio relationship based on normalized CPT tip resistance. (After Olson and Stark, 2002.)

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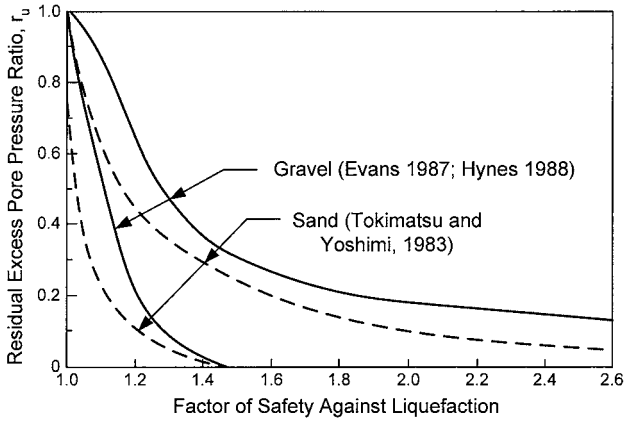


Figure 10.11 Typical residual excess pore water pressure ratios as a function of the factor of safety against liquefaction for sand and gravel. (After Marcuson et al., 1990.)

Reduced undrained shear strengths can be estimated by performing laboratory tests in which specimens are consolidated to stresses comparable to those expected in the field before the earthquake, subjected to loads simulating the earthquake, and finally, sheared to failure in a static load test.

Step 3. Compute Slope Stability

Once the postearthquake shear strengths have been determined, a conventional static slope stability analysis is performed. For some soils and slope geometries, the undrained shear strength after seismic loading may represent the minimum shear strength, and the shear strength will gradually increase with time after the earthquake. For these soils and slopes, the slope stability computations can be performed using undrained shear strengths that reflect the effects of cyclic loading as discussed in the two preceding sections. However, for other soils, especially those that dilate when sheared, the shear strength may decrease with time after the earthquake as the soil drains and water migrates from zones of high pore water pressure to zones of lower pressure. This was illustrated by Seed (1979) in Figures 10.12 and 10.13 for the Lower San Fernando Dam. The factor of safety computed using undrained strengths immediately after the earthquake (Figure 10.12) was 1.4, while the factor of safety accounting for partial drainage and redistribution of pore water pressure (Figure 10.13) was only 0.8.

In cases where some combination of undrained and drained (or partially drained) shear strengths control the stability, it seems appropriate to perform stability analyses that use the lower of the drained and undrained shear strengths, as is done for rapid drawdown. Procedures similar to the multistage analysis procedures described in Chapter 9 can be used for this purpose. Specifically, the procedures suggested for the analysis of stability following an earthquake involve the following two analysis stages for each trial slip surface:

- *Stage 1.* Stability computations are performed using undrained shear strengths that reflect the effects of cyclic loading for low-permeability ma-

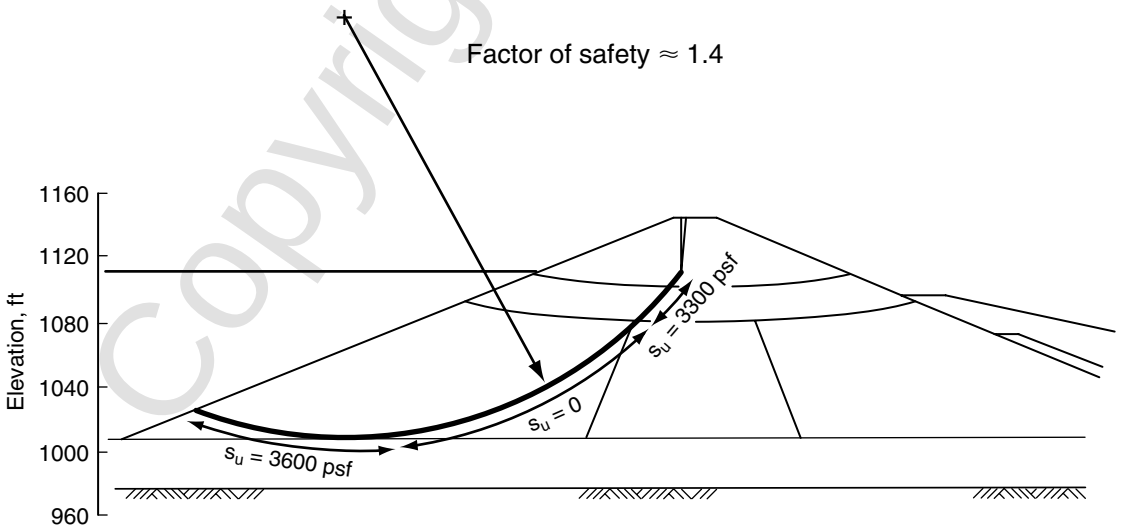


Figure 10.12 Stability of lower San Fernando Dam immediately after earthquake. (After Seed, 1979.)

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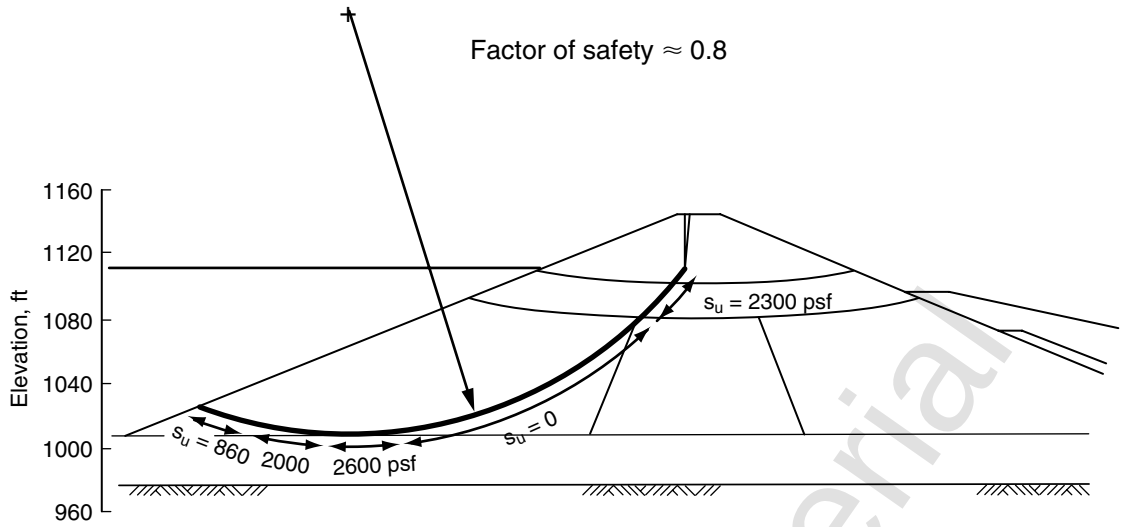


Figure 10.13 Stability of lower San Fernando Dam after partial drainage and redistribution of pore water pressures following the earthquake. (After Seed, 1979.)

terials[3]; effective stresses and drained shear strengths are used for high-permeability soils.

- *Stage 2.* Based on the total normal stress that is computed from the first-stage stability analysis and the pore water pressures that will exist after complete drainage (full excess pore water pressure dissipation), the fully drained shear strength is estimated. This is done slice by slice along the slip surface in all low-permeability soils. If the drained shear strength is less than the undrained shear strength, the drained shear strength is assigned to the slice; otherwise, the undrained shear strength is assumed to be applicable. Stability computations are then repeated. The computations will involve a mix along the slip surface of total stresses (where undrained strengths are used) and effective stresses (where drained strengths are used). The factor of safety computed from the second-stage analysis is the factor of safety after the earthquake.

[3] If an effective stress approach, such as the one suggested by Marcuson et al. (1990), where excess pore water pressures are used to represent the postearthquake strengths, effective stress shear strength parameters and excess pore water pressures are used in lieu of undrained shear strengths for the first-stage analysis.

Recapitulation

- Static slope stability analyses should be performed using reduced undrained shear strengths that reflect the earthquake load effects to determine that the slope is sufficiently stable after the earthquake.
- For soils that may lose strength as they drain after the earthquake, a two-stage analysis can be performed using the lower of the drained and undrained shear strength along each slop surface.

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