Module 8
SEISMIC SLOPE STABILITY
(Lectures 37 to 40)

Lecture 39

Topics

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8.6.6 Newmark Sliding Block Analysis

The pseudostatic method of analysis, like all limit equilibrium methods, provides an index of stability (the factor of safety) but not information on deformations associated with slope failure. Since the serviceability of a slope after an earthquake is controlled by deformations, analyses that predict slope displacement provide a more useful indication of seismic slope stability. Since earthquake induced accelerations vary with time, the pseudostatic factor of safety will vary throughout an earthquake. If the inertial forces acting on a potential failure mass become large enough that the total (static plus dynamic) driving forces exceed the available resisting forces, the factor of safety will drop below 1.0. Newmark (1965) considered the behavior of a slope under such conditions. When the factor of safety is less than 1.0, the potential failure mass is no longer in equilibrium consequently, it will be accelerated by the unbalanced force. The situation is analogous to that of a block resting on an inclined plane (figure 8.8). Newmark used this analogy to develop a method for prediction of the permanent displacement of a slope subjected to any ground motion.
Figure 8.8 Analogy between (a) potential landslide and (b) block resting on inclined plane

Consider the block in stable, static equilibrium on the inclined place of (Figure 8.8.b). Under static conditions, equilibrium of the block (in the direction parallel to the plane) requires that the available static resting force, $R_s$, exceed the static driving force, $D_s$ (Figure 8.9.a). Assuming that the block’s resistance to sliding is purely frictional ($c = 0$)

![Diagram](image1.png)

Figure 8.9 Forces acting on a block resting on an inclined plane: (a) static conditions; (b) dynamic conditions

$$FS = \frac{\text{available resisting force}}{\text{static driving force}} = \frac{R_s}{D_s} = \frac{W \cos \beta \tan \phi}{W \sin \beta} = \frac{\tan \phi}{\tan \beta} \quad (8.4)$$

Where $\phi$ is the angle of friction between the block and the plane. Now consider the effect of inertial forces transmitted to the block by horizontal vibration of the inclined plane with acceleration, $a_h(t) = k_h(t)g$ (the effect of vertical accelerations will be neglected for simplicity). At a particular instant of time, horizontal acceleration of the block will induce a horizontal inertial force, $k_h W$ figure 8b. When the inertial force acts in the downslope direction, resolving forces perpendicular to the inclined plane gives...
Obviously the dynamic factor of safety decreases as $k_h$ increases and there will be (for a statically stable block) some positive value of $k_h$ that will produce a factor of safety of 1.0 (Figure 8.10). This coefficient, termed the *yield coefficient*, $k_y$ corresponds to the *yield acceleration*, $a_y = k_y g$. The yield acceleration is the minimum pseudostatic acceleration required to produce instability of the block. For the block of (Figure 8.9),

\[ k_y = \tan(\phi - \beta) \]  

(8.6)

For sliding in the downslope direction. For sliding in the uphill direction (which can occur when $\beta$ and $\phi$ are small),

\[ k_y = \frac{\tan \phi + \tan \beta}{1 + \tan \phi \tan \beta} \]  

(8.7)
Example 3

Compute the yield acceleration for the slope described in (example 2).

Solution

The yield acceleration can be computed by trial and error, or computed directly for relatively simple slopes. Reviewing example 2, it is apparent that the total moment is equal to

\[ M_t = 4488 \, \text{ft} \cdot \text{ft} + k_h(5685k - \text{ft} / \text{ft}) + 1438k - \text{ft} / \text{ft} + k_h(17825k - \text{ft} / \text{ft}) \]

\[ = 5926 \, \text{ft} / \text{ft} + k_h(23510k - \text{ft} / \text{ft}) \]

The yield coefficient is the value of \( k_h \) that produces a pseudostatic factor of safety of 1. Because the resisting moment is equal to the overturning moment when \( FS = 1 \),

\[ 5926 \, \text{ft} / \text{ft} + k_h(23510k - \text{ft} / \text{ft}) = 10624k - \text{ft} / \text{ft} \]

Or

\[ k_h = \frac{10624k - \text{ft} / \text{ft} - 5926k - \text{ft} / \text{ft}}{23510k - \text{ft} / \text{ft}} = 0.20 \]

Therefore, the yield acceleration is 0.20g.

When a block on an inclined plane is subjected to a pulse of acceleration that exceeds the yield acceleration, the block will move relative to the plane. To illustrate the procedure by which the resulting permanent displacements can be calculated, consider the case in which an inclined plane is subjected to a single rectangular acceleration pulse of amplitude \( A \) and duration \( \Delta t \). If the yield acceleration \( a_y \) is less than \( A \) (figure 8.11.a), the acceleration of the block relative to the plane during the period from \( t_o \) to \( t_o + \Delta t \) is

\[ a_{rel}(t) = a_b(t) - a_y = A - a_y \quad t_o \leq t \leq t_o + \Delta t \] (8.8.a)
Where \( a_b(t) \) is the acceleration of the inclined plane. The relative movement of the block during this period can be obtained by integrating the relative acceleration twice, that is,

\[
v_{rel}(t) = \int_{t_o}^{t} a_{rel}(t) \, dt = [A - a_y](t - t_o) \quad t_o \leq t \leq t_o + \Delta t \quad (8.8.b)
\]

\[
d_{rel}(t) = \int_{t_o}^{t} v_{rel}(t) \, dt = \frac{1}{2} [A - a_y](t - t_o)^2 \quad t_o \leq t \leq t_o + \Delta t \quad (8.8.c)
\]

At \( t = t_o + \Delta t \), the relative velocity reaches its maximum value. At that time

\[
v_{rel}(t_o + \Delta t) = [A - a_y]\Delta t \quad (8.9.a)
\]

\[
d_{rel}(t_o + \Delta t) = \frac{1}{2} [A - a_y]\Delta t^2 \quad (8.9.b)
\]

After the base acceleration drops to zero (at \( t = t_o + \Delta t \)), the sliding block is decelerated by the friction force acting on its base. The block will continue to slide on the plane, but at a decreasing velocity which eventually reaches zero. The acceleration during this time is given by

\[
a_{rel}(t) = a_b(t) - a_y = 0 - a_y = -a_y \quad t_o + \Delta t \leq t \leq t_1 \quad (8.10.a)
\]

Where \( t_1 \) is the time at which the relative velocity becomes zero (note that the block undergoes negative acceleration or deceleration, during this period). Between \( t_o + \Delta t \) and \( t_1 \) the relative velocity will decrease with time according to

\[
v_{rel}(t) = v_{rel}(t_o + \Delta t) + \int_{t_o+\Delta t}^{t} a_{rel} \, dt = A \Delta t - a_y(t - t_o) \quad t_o + \Delta t \leq t \leq t_1 \quad (8.10.b)
\]

Setting the relative velocity equal to zero at \( t = t_1 \) gives
\[ t_1 = t_o + \frac{A}{a_y} \Delta t \]

Then

\[ d_{rel}(t) = \int_{t_o+\Delta t}^{t} v_{rel}(t) \, dt = A \Delta t (t - t_o) - \frac{A}{2} [t^2 - (t_o + \Delta t)^2] \]

\[ t_o + \Delta t \leq t \leq t_1 \]  \hspace{1cm} (8.10.c)

After time \( t_1 \), the block and inclined plane move together. During the total period of time between \( t = t_o \) and \( t = t_1 \), the relative movements of the block is as shown in (Figure 8.11). Between \( t_o \) and \( t_o + \Delta t \), the relative velocity increases linearly and the relative displacement quadratically. At \( t_o + \Delta t \) the relative velocity has reached its maximum value after which it decreases linearly. The relative displacement continues to increase (but at a decreasing rate) until \( t = t_1 \). Note that the total relative displacement depends strongly on both the amount by which and the length of time during which the yield acceleration is exceeded.

**Figure 8.11** Variation of relative velocity and relative displacement between sliding block and plane due to rectangular pulse that exceeds yield acceleration between \( t = t_o \) and \( t = t_o + \Delta t \)

\[ d_{rel}(t_1) = \frac{1}{2} (A - a_y) \Delta t^2 \frac{A}{a_y} \]  \hspace{1cm} (8.11)
This suggests that the relative displacement caused by a single pulse of strong ground motion should be related to both the amplitude and frequency content of that pulse. An earthquake motion, however, can exceed the yield acceleration a number of times and produce a number of increments of displacement (Figure 8.12). Thus the total displacement will be influenced by strong motion duration as well as amplitude and frequency content. Indeed, application of this approach to a variety of simple waveforms (e.g., Sarma, 1975; Yegian et al., 1991) have been shown that the permanent displacement of a sliding block subjected to rectangular, sinusoidal, and triangular periodic base motions is proportional to the square of the period of the base motion.

Figure 8.12 Development of permanent slope displacement for actual earthquake ground motion. (After Wilson and Keefer, 1985)

8.6.7 Influence of Yield Acceleration on Slope displacement

Obviously, the sliding block model with predict zero permanent slope displacement if earthquake induced acceleration never exceed the yield acceleration \( \frac{a_y}{a_{\text{max}}} \geq 1.0 \) as illustrated in (Figure 8.13.a). Since the permanent displacement is obtained by double integration of the excess acceleration, the computed displacements for a slope with relatively low yield acceleration (small \( \frac{a_y}{a_{\text{max}}} \)) will be greater than that of a slope with a higher yield acceleration (Figure 8.13.b, 8.13.c). The relationship between slope displacement and \( \frac{a_y}{a_{\text{max}}} \) has been investigated by a number of researchers.
Figure 8.13.a Permanent slope displacements depend on the relationship between the yield acceleration and the maximum acceleration, (a) if the yield acceleration of a slope is greater than the maximum acceleration of a particular ground motion, no displacement will occur. As yield acceleration decreases, as in (b) and (c) slope displacement increases quickly.

Using the rectangular pulse solution developed, Newmark (1965) related single pulse slope displacements to peak base, velocity, $v_{max}$, by

$$d_{rel} = \frac{v_{max}^2}{2a_y} \left(1 - \frac{a_y}{A}\right)$$

(8.12)

[Note that equation 8.12 is equivalent to equation 8.11 with $v_{max} = A \Delta t$]. Analysis of several earthquake motions normalized to peak acceleration of 0.5g and peak velocities of 30 in/sec (76 cm/sec) suggested that the effective number of pulses in an earthquake motion could be approximated by $A/a_y$. Newmark found that a reasonable upper bound to the permanent displacement produced by these earthquake motions was given by

$$d_{max} = \frac{v_{max}}{2a_y} \frac{a_{max}}{a_y}$$

(8.13)

Where $a_y/a_{max} \geq 0.17$. Sarma (1975) and Yegian et al. (1988) derived closed form solutions for the permanent displacement produced by simple periodic (triangular, sinusoidal, and rectangular) input motions (Figure 8.14). Studies of permanent displacement predicted by the sliding block method for actual earthquake motions...
(e.g., Sarma, 1975; Franklin and Chang, 1977; Makdisi and Seed, 1978; Ambraseys and Menu, 1988) show shapes that are similar to those of the sinusoidal and triangular waves at $a_y/a_{max}$ values greater than about 0.5. Ambraseys and Menu (1988) found that the shape at smaller $a_y/a_{max}$ values was influenced by whether or not upslope movements were considered; for the case in which they were not, permanent displacement (in centimeters) caused by actual ground motions were given by for $0.1 \leq a_y/a_{max} < 0.9$, $6.6 \leq M_s \leq 7.3$, and $a_y$ computed using residual soil strength.

Figure 8.14 Variation of normalized permanent displacement with ratio of yield acceleration to maximum acceleration for simple waveforms. The normalized permanent displacement is defined in equation (8.15) (After Yegian et al., 1991)

\[
\log u = 0.90 + \log \left( 1 - \frac{a_y}{a_{max}} \right)^{2.53} \left( \frac{a_y}{a_{max}} \right)^{-1.09} \sigma_{\log u} = 0.30 \tag{8.14}
\]

To allow measures of frequency content and duration to be considered explicitly. Yegian et al. (1991) used the database of Franklin and Chang (1977) to develop the following expression for the median permanent normalized displacement:

\[
\log u^* = \log \left( \frac{u}{a_{max} N_{eq}^2} \right) =
0.22 - 10.12 \frac{a_y}{a_{max}} + 16.38 \left( \frac{a_y}{a_{max}} \right)^2 - 11.48 \left( \frac{a_y}{a_{max}} \right)^3 \sigma_{\log u^*} = 0.45 \tag{8.15}
\]
Where $N_{eq}$ is an equivalent number of cycles and $T$ is the predominant period of the input motion. Considering only this source of uncertainty (i.e., neglecting uncertainty in $a_{max}, a_y, N_{eq},$ and $T$) probabilities of exceeding various displacement can be determined (figure 8.15). Alternative approaches to the probabilities analysis of slope displacements have been presented by Constantinou and Gazetas (1984) and Lin and Whitman (1986).

**Figure 8.15** Contours of equal probability of exceedance of normalized permanent displacement. (After Yegian et al., 1991)

Recognition of the limitations of peak acceleration as a sole descriptor of strong ground motion has led to the use of other ground motion parameters in slope displacement prediction. Sliding block displacement has been correlated with Arias intensity:

$$\log u = 1.460 \log I_a - 6.642a_y + 1.546 \quad \sigma_{\log u} = 0.409 \quad (8.16)$$

Where $u$ is in cm, $I_a$ is in m/sec, and $a_y$ is in g's (Jibson, 1994) and used to predict area limits of earthquake induced landsliding (Wilson and Keefer, 1985).
Two aspects of seismic slope stability are clearly illustrated by the studied described in the preceding parameters. First, earthquake induced slope displacements are very sensitive to the value of the yield acceleration. Consequently, small differences in yield acceleration can produce large variations in predicted slope displacement. Second, the great variability in distribution of acceleration pulse amplitudes between different ground motions similar amplitudes, frequency contents, and durations can produce significantly different predicted slope displacements. This uncertainty must be recognized in the prediction of earthquake induced slope deformations.

Example 4

Estimate the expected permanent displacement of the slope described in (example 3) if subjected to a ground motion equivalent to the Gilroy No. 1 (rock) earthquake motion. Use the procedures of Newmark (equation 8.13) and Jibson (equation 8.16).

Solution

From (example 1), from module 3, the peak acceleration and velocity of the Gilroy No. 1 (rock) motion are

\[ a_{max} = 0.442 \]

\[ v_{max} = 33.7 \text{ cm/sec} \]

The yield acceleration was computed as 0.20g in example 3. Then using the Newmark procedure (equation 13) an upper bound estimate of the permanent displacement would be

\[ d_{max} = \frac{(33.7 \text{ cm/sec})^2}{2(0.20g)(981 \text{ cm/sec}^2/g)0.200g} = 6.4 \text{ cm} \]

The Arial Intensity of the Gilroy No 1 (rock) motion was computed as \( I_a = 167.7 \text{ cm/sec} \) in (example 6 from module 3). Using the Jibson procedure (equation 16) the average permanent displacement would be given by
\[ \log u = 1.460 \log(1.667) - 6.642(0.20) + 1.546 = 0.545 \]

Or

\[ u = 10^{0.545} = 3.5 \text{ cm} \]

### 8.6.8 Input Method

The accuracy of a sliding analysis depends on the accuracy of the input motion applied to the inclined plane. As originally proposed, the sliding block method assumed the potential failure mass to be rigid, in which case the appropriate input motion would be the ground motion at the level of the failure surface. Actual slopes, however, are compliant—they deform during earthquake shaking. Their dynamic response depends on their geometry and stiffness and on the amplitude and frequency content of the motion of the underlying ground. For slope composed of very stiff soils and/or slopes subjected to low frequency motion (a combination that produces long wavelength), lateral displacements throughout the potential failure mass will be nearly in phase (figure 8.16.a) and the rigid block assumption will be at least approximately satisfied. Lateral displacements in potential failure masses of slope sin softer soils (and/or slopes subjected to higher frequency motion), however, may be out of phase (figure 8.16.b). When this occurs the inertial forces at different points within the potential failure mass may be acting in opposite directions and the resultant inertial force may be significantly smaller than that implied by the rigid block assumption.

![Figure 8.16 Influence of frequency on motions induced in slopes. Long wavelength associated with low frequency motion (a) causes soil above failure surface to move essentially in phase. For higher frequency motion (b) portions of soil above failure surface may be moving in opposite directions](image)

The effects of slope response on the inertial force acting on a potential failure mass can be computed using dynamic stress-deformation analyses (Chopra, 1966). Using a dynamic finite element analysis (figure 8.17.a) the horizontal component of the dynamic stresses acting on a potential failure surface (figure 8.17.b) are integrated over the failure surface to produce the time varying resultant force that acts on the potential failure surface. This resultant force can then be divided by the mass of the
soil above the potential failure surface to produce the average acceleration of the potential failure mass. Although the procedure was developed originally for dams the basic concept can be applied to any type of slope. The average acceleration time history which may be of greater or smaller amplitude than the base acceleration time history (depending on the input motions and the amplification characteristic of the slope), provides the most realistic input motions for a sliding block analysis of the potential failure mass.

![Figure 8.17 Evaluation of average acceleration for slope in embankment. Finite element analysis predicts variation of shear and normal stresses on potential failure plane with time.](image)

**Figure 8.17** Evaluation of average acceleration for slope in embankment. Finite element analysis predicts variation of shear and normal stresses on potential failure plane with time.

### 8.6.9 Other Factors Influencing Slope Displacement

The standard sliding block analysis is based on the assumption of rigid perfectly plastic stress-strain behavior on a planar failure surface. Conditions for actual slopes may vary from these assumptions in a number of ways.

The shear strength of some soils is rate dependent. Since earthquake induced shear stresses are applied at different rates, the shear strength (and hence the yield acceleration) can vary with time throughout an earthquake (e.g., Hunger and Moregenstern, 1984; Lemos et al., 1985). Consideration of rate dependent strength in a sliding block analysis is complicated by differences between strain rates in the field and in the laboratory tests used to measure the strength. Lemos and Coelho (1991) and Tika-Vassilikos et al. (1993) suggested procedures for incorporating rate dependent field strength into numerical sliding block analyses.

In the field, soil rarely behaves as perfectly plastic materials. Instead, they usually exhibit strain hardening or strain softening stress strain behavior (figure 8.18) after yielding. The yield acceleration of slopes comprised of strain hardening or strain softening soils will vary with slope displacements. Consequently, the permanent displacement of a slope in strain hardening materials will be smaller than predicted by a conventional sliding block analysis; the reverse will be true for strain softening soils. Modification of sliding block analyses for consideration of displacement dependent strength is fairly straightforward.
Many slopes fail by mechanism that differs from the planar failure mechanism assumed in sliding block analyses. Neglecting the effects of rate and displacement dependent strength, the stability of a block on a plane will be the same both before and after a pulse of displacement because the geometry of the block relative to the plane is unchanged. Movement of a slope on a nonplanar failure surface, however, tends to flatten the slope, thereby reducing the driving forces. As a result, the yield acceleration should increase due to changes in the geometry of the unstable soil. For most slopes, however, this effect does not become significant until large displacement have occurred.

**8.6.10 Makdisi-Seed Analysis**

Makdisi and Seed (1978) used average acceleration computed by the procedure of Chopra (1966) and sliding block analyses to compute earthquake induced permanent deformations of earth dams and embankments. By making simplifying assumptions about the results of dynamic finite element and shear beam analyses of such structures, a simplified procedure for prediction of permanent displacements was developed.

In the simplified procedure, the yield acceleration for a particular potential failure surface is computed using the dynamic yield strength [80% of the undrained strength of the soil. The dynamic response of the dam/embankment is accounted for by an acceleration ratio that varies with the depth of the potential failure surface relative to the height of the dam/embankment (figure 8.19).
Figure 8.19 Variation of average maximum acceleration with depth of potential failure surface for dams and embankments. (After Makdisi and Seed (1978))

By subjecting several real and hypothetical dams to several actual and synthetic ground motions scaled to represent different earthquake magnitudes. Makdisi and Seed computed the variation of permanent displacement with $a_y/a_{\text{max}}$ and magnitude. Scatter in the predicted displacements was reduced by normalizing the displacement with respect to the peak base acceleration and the fundamental period of the dam/embankment (note that the normalized displacement has units of seconds). Prediction of permanent displacements by the Makdisi-Seed procedure is accomplished with the charts shown in (figure 8.20).

Figure 8.20 Variation of normalized permanent displacement with yield acceleration for earthquakes of different magnitudes: (a) summary for several earthquake and dams/embankments; (b) average values. (After Makdisi and Seed (1978)).

Example 5

Assume that a failure surface that extends over the upper two thirds of the earth dam has a yield acceleration of 0.24g. Estimate the permanent displacement that would occur if the base of the dam was subjected to the Gilroy No. 1 (rock) motion.
The Gilroy No. 1 motion was recorded in the 1989 Loma Prieta earthquake which had a magnitude of 7.1. The peak acceleration was 0.442g. From example 6 from chapter 7, the fundamental period of the dam is

\[ T_o = \frac{2\pi}{19.2 \text{ rad/sec}} = 0.33 \text{ sec} \]

Using figure 19b with \( \frac{a_y}{a_{\text{max}}} = 0.24g/0.442g = 0.54 \) and \( M = 7.1 \), the average normalized displacement is about 0.04. Therefore,

\[ u = 0.04 \ a_{\text{max}} \ T_o = 0.04(0.442g)(32.2 \text{ ft/sec}^2/g)(0.33 \text{ sec}) = 0.19\text{ft} = 2.3 \text{ in} \]

The Makdisi-Seed simplified procedure is widely used for estimation of permanent displacement in dams and embankments. Because the procedure is based on the dynamic response characteristics of dams and embankments, its results must be interpreted with caution when applied to other types of slopes.

### 8.6.11 Stress-Deformation Analysis

Just as stress deformation analyses of static slope stability are usually performed using static finite-element analyses, stress-deformation analyses of seismic slope stability are usually performed using dynamic finite-element analyses. In such analyses the seismically induced permanent strains in each element of the finite element mesh are integrated to obtain the permanent deformation of the slope. Permanent strains within individual elements can be estimated in different ways. The strain potential and stiffness reduction approaches estimate permanent strains using laboratory test results to determine the “stiffness” of soils subjected to earthquake loading. Nonlinear analysis approaches use the nonlinear inelastic stress-strain behavior of the soil to compute the development of permanent strains throughout an earthquake.

### 8.6.12 Strain Potential Approach

In their landmark investigation of the slides that occurred in the Upper and Lower San Fernando dams during the 1971 San Fernando earthquake, Seed et al. (1973) developed a procedure for estimating earthquake induced slope deformation from the results of linear or equivalent linear analyses. In this procedure the cyclic shear stresses are computed in each element of a dynamic finite element analysis. Using the results of cyclic laboratory tests, the computed cyclic shear stresses are used to predict the strain potential, expressed as a shear strain, for each element. Deformation area then estimated as the product of the average strain potential along a vertical section through the slope and the height of that section. The method implicitly assumes that the strains that develop in the field will be the same as those
that develop in a similarly loaded laboratory test specimen and that the maximum shear stress acts in the horizontal direction in all elements. Consequently, the strain potential approach estimates only horizontal displacements. Analyses based on the strain potential approach are clearly very approximate, and their results should always be interpreted with that fact in mind.

8.6.13 Stiffness Reduction Approach

Another method for estimation of permanent slope displacement was developed by Lee (1974) and Serff et al. (1976). In this approach, computed strain potentials are used to reduce the stiffness of the soil as illustrated in figure 8.21. Earthquake induced slope displacement are then taken as the difference between the nodal point displacement from two static finite element analyses one using the initial shear moduli and the other using the reduced shear moduli. The technique can be used with linear or nonlinear models. Unlike the strain potential approach, the stiffness reduction approach can estimate vertical as well as horizontal movements. It is very approximate procedure; however, ad is subject to many of the limitations of the strain potential approach. Work energy principles can be used to provide a more fundamental procedure for stiffness reduction (Byrne, 1991; Byrne et al., 1992).

![Figure 8.20 Procedure used to reduce stiffness from initial value, $G_i$, to final value, $G_f$, in stiffness reduction approach to estimation of permanent slope deformation (After Serff et al., 1976)](image)

8.6.14 Nonlinear Analysis Approach

Permanent slope deformations can also be computed by finite element analyses that employ nonlinear inelastic soil models. The basic procedures of nonlinear finite element analysis of each structure were introduced. The seismic performance of slopes has been analyzed with two and three dimensional finite element analyses using both cyclic stress strain models (e.g., Finn et al., 1986) and advanced constitutive models (e.g., Prevost, 1981; Mizuno and Chen, 1982; Kaai, 1985; Daddazio et al., 1987). The most common application of these techniques to date has been the analysis of earth dams. Examples of such analyses can be found in Prevost et
al. (1985), Griffiths and Prevost (1988) Finn (1990), Elgamal et al. (1990), and Succarich et al. (1991). The accuracy of nonlinear finite element analyses depends primarily on the accuracy of the stress strain or constitutive models on which they are based.