Technical Paper by N. Matasovic, E. Kavazanjian, Jr. and J.P. Giroud

Newmark Seismic Deformation Analysis for Geosynthetic Covers

ABSTRACT: This paper investigates the impact of the following five assumptions on the accuracy of Newmark seismic deformation analysis applied to geosynthetic cover systems: (i) the potential failure mass is noncompliant; (ii) the dynamic response of the potential failure mass is uncoupled from displacement (slip); (iii) permanent displacements accumulate in only one direction; (iv) vertical ground motions do not influence permanent displacement; and (v) the yield acceleration is constant. Information presented in the literature indicates the impact of the assumption of a noncompliant failure mass and the assumption of a seismic response uncoupled from displacement is insignificant for typical geosynthetic cover systems. The results of computer analyses indicate that the effects of two-way sliding and vertical ground motions can, in most practical cases, be neglected. However, the assumption of a constant yield acceleration, when based on residual (or large displacement) shear strength, may result in calculated displacements that are significantly larger than those calculated using a yield acceleration that degrades with accumulated displacement from a peak value to a residual, or large displacement, value. Overall, results of this investigation indicate that conventional Newmark analyses based upon residual shear strength yield conservative results when applied to geosynthetic cover systems.

KEYWORDS: Permanent displacements, Seismic deformation, Newmark method, Composite cover, Residual strength, Two-Way sliding, Vertical acceleration.

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1 INTRODUCTION

1.1 Classical Newmark Seismic Deformation Analysis

The most common approach to seismic deformation analysis is the *rigid block sliding on a plane* approach described by Newmark (1965). In such an analysis, schematically illustrated in Figure 1, it is assumed that a rigid block of weight $W = mg$, where m is the mass of the block and g is the acceleration due to gravity, is resting on a horizontal plane. When the plane is excited by a horizontal acceleration, a seismic inertia force, $[a_h(t)]m$, is induced in the block, where $a_h(t)$ is the horizontal acceleration time history of the block. When the horizontal acceleration is expressed as a fraction of the acceleration due to gravity, it is denoted by $k_h(t)$, where $k_h(t) = [a_h(t)]/g$, and is sometimes referred to as the *seismic coefficient*. Using the seismic coefficient, the seismic inertia force on the block may be written as $[k_h(t)]mg$.

At any time t, the forces at the block/plane interface are the interface normal force, N, and the interface shear force, T. By force equilibrium, the interface shear force is equal to the block inertia force, $T = [k_h(t)]mg$, and the interface normal force is equal to the weight of the block, $N = W = mg$. The magnitude of T is limited by the shear strength of the interface, T_f. Using the Mohr-Coulomb failure criterion, the interface shear strength may be expressed as $T_f = aBL + N \tan\phi$, where a is the interface adhesion, ϕ is the interface friction angle, B is the width of the block/plane interface, and L is the length of the block/plane interface.

The maximum possible block acceleration, k_y, may be found by equating the block inertia force at this acceleration to the interface shear strength: $k_y\ m = (k_y/g)\ mg = aBL + N \tan\phi$. Recognizing that $mg = N$, the maximum possible block acceleration (expressed as a fraction of the acceleration due to gravity) is found as $k_y/g = aBL/N + \tan\phi$. If the horizontal acceleration of the base plane remains less than or equal to k_y, the block and the plane move in unison. When the horizontal acceleration of the base plane exceeds k_y, relative movement (yielding) between the block and the plane occurs. Because it represents the threshold above which relative movement (yielding) between the block and plane occurs, k_y is termed the *yield acceleration*.

Theoretically, when subjected to acceleration levels above k_y, the block shown in Figure 1 can move in both the positive and negative direction parallel to the plane (i.e. $x > 0$ and $x < 0$, with the x-axis parallel to the acceleration), i.e. two-way block sliding may occur. Because earthquake loading is a zero-mean process, if the plane is horizontal and the block is allowed to move in both directions, the resulting permanent (or re-

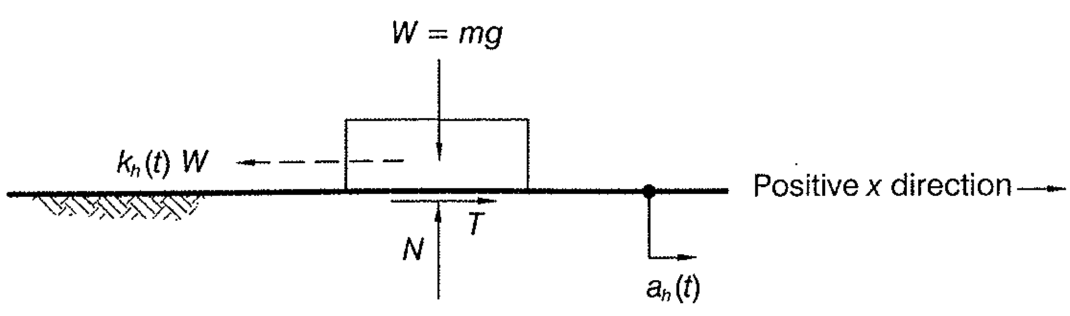

Figure 1. Basic elements of a classical Newmark analysis.

sidual) displacement at the end of the earthquake shaking is generally close to zero. Therefore, to apply the model shown in Figure 1 to the seismic deformation analysis of a slope or an embankment, the classical Newmark analysis assumes the block can move in only one direction, i.e. downslope. The *classical Newmark analysis* is then carried out following the integration procedure schematically depicted in Figure 2 to calculate the downslope movement. The direction of the horizontal inertia force which causes downslope block movement is referred as the *out-of-slope* direction and is indicated in Figure 1 as the positive x direction.

As illustrated in Figure 2, the classical Newmark procedure may be implemented by numerical integration of the acceleration and velocity time histories of the block. First, the velocity time history of the relative movement between the block and the plane is calculated by integration of the acceleration time history of the plane modified by the yield acceleration of the block, with relative block movement (sliding) beginning each time the yield acceleration is exceeded in the out-of-slope direction and continuing until zero velocity is calculated for the sliding block. The cumulative relative displacement of the sliding block is then calculated by integrating the relative velocity time his-

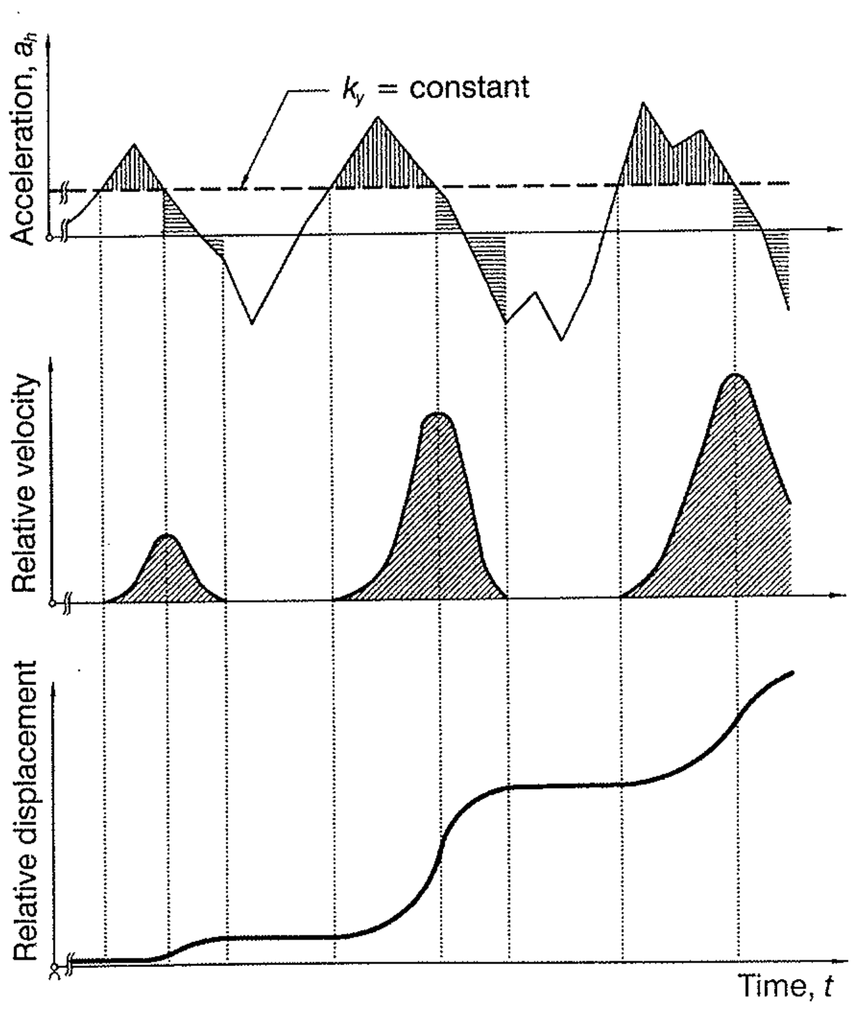

Figure 2. Classical Newmark analysis integration scheme.

tory, as shown in Figure 2. Since relative displacement of the block only occurs between the time the earthquake acceleration exceeds the yield acceleration in the out-of-slope direction and the time when the relative velocity drops to zero, quiet intervals exist during which there are no increments in the relative block displacement.

The value of relative displacement at the end of the base plane excitation is commonly called the *(calculated) permanent seismic displacement* or *(calculated) seismic deformation*. While the calculated displacements (and velocities) are properly referred to as relative displacements (and velocities), for simplicity the attribute *relative* is commonly omitted.

1.2 Conventional Newmark Analysis for Geotechnical Problems

In conventional geotechnical practice, the potential failure mass established in a limit equilibrium stability analysis is considered analogous to the sliding block on the plane shown in Figure 1. The yield acceleration of the potential failure mass is established as a function of the material and/or interface shear strength(s) and slope geometry using a pseudo-static limit equilibrium analysis.

In a formal *conventional Newmark analysis*, the acceleration time history of the base plane on which the block sits is generally evaluated from the results of seismic site response analysis as the average acceleration time history of the potential failure mass for which the pseudo-static yield acceleration has been calculated. Then, the seismically induced permanent displacement of the potential failure mass is calculated following the procedure schematically depicted in Figure 2.

Less formal, classical (simplified conventional) analyses may also be performed using acceleration time histories selected from a catalog of recorded or synthesized earthquake ground motions and scaled to the peak horizontal ground acceleration, *PHGA*, of the base plane motion. Alternatively, charts that summarize the results of previous classical and conventional Newmark deformation analyses as a function of the yield acceleration divided by the *PHGA* can be used.

The formal conventional Newmark seismic deformation analysis described above involves the following five simplifying assumptions: (i) the potential failure mass is rigid (noncompliant); (ii) the dynamic response of the failure mass is not influenced by (coupled with) the permanent displacement (slip) that occurs along the failure surface; (iii) permanent displacement accumulates in only one direction (the downslope direction); (iv) the vertical component of the ground motion does not influence the calculated permanent displacement; and (v) the yield acceleration of the potential failure mass is constant.

1.3 Application to Geosynthetic Systems

Over the past six years, conventional Newmark seismic deformation analyses have been used extensively for seismic design of geosynthetic liners and covers for landfills and other waste containment systems. However, little direct field evidence of the accuracy of these conventional Newmark analyses exists. In fact, due to the many simplifying assumptions inherent to the analysis, many engineers consider the permanent seismic displacement calculated in a conventional Newmark analysis to be merely an index of the seismic performance of the system rather than an engineering estimate of the anticipated seismic displacement of the geosynthetic liner or cover (Anderson and Kavazanjian 1995).

Experimental evidence on the accuracy of conventional Newmark analyses applied to geosynthetic interfaces is summarized in Table 1. This evidence includes both geotechnical centrifuge testing (Hushmand and Martin 1990; Zimmie et al. 1994; De 1996) and shaking table testing (Kavazanjian et al. 1991; Yegian and Lahlaf 1992; Zimmie et al. 1994; Yegian et al. 1995a,b; Yegian and Harb 1995; De 1996; Kramer and Smith 1997) under a variety of testing conditions. The results of these studies confirm that the response of a geosynthetically lined rigid block to cyclical base excitation is in accordance with the basic principles of the classical Newmark analysis. However, due to limitations of the testing (adhesion can not be evaluated; testing is limited to the evaluation of the peak friction angle), while previous studies indicate that the integration procedure depicted in Figure 2 is accurate, these experiments provide little insight about the overall impact of the simplifying assumptions made in a formal conventional Newmark analysis on the accuracy of the computed deformations.

In the current paper, the impact of the five simplifying assumptions discussed in Section 1.2 on the accuracy of the computed seismic deformations in a formal conventional Newmark analysis of geosynthetic cover systems is evaluated. The first two assumptions that the potential failure mass is noncompliant for displacement analysis purposes and that the displacement is decoupled from the seismic response are discussed together (Section 2.1). The remaining three assumptions are discussed separately (Sections 2.2 to 2.4).

2 PREVIOUS WORK

2.1 Effects of System Compliance and Decoupling

In a conventional Newmark seismic deformation analysis, the average acceleration time history of the potential failure mass is assumed to represent the motion of the base plane on which the sliding block in Figure 1 sits. The seismic response analysis typically used to calculate the average acceleration time history of the potential failure mass generally assumes that there is no relative displacement (no sliding) at the base of the potential failure mass. Therefore, it is implicitly assumed that the dynamic response of the potential failure mass and the displacement (sliding) of this mass are two separate process, i.e. they are *decoupled.* Furthermore, the use of the average acceleration time history of the potential failure mass as the base plane motion implicitly treats the mass as a noncompliant (i.e. rigid) mass for purposes of the displacement analysis (though the compliance of the failure mass is considered in the seismic response analysis).

Lin and Whitman (1983) studied the seismic deformation response of earth dams to gain insight into the decoupling and noncompliant assumptions of a conventional Newmark analysis. Lin and Whitman (1983) modeled the seismic response of an earth dam using a multi-degree-of-freedom system consisting of vertical stacks of lumped masses connected by springs and viscous dashpots. A horizontal sliding element was introduced within the model at varying locations to simulate shallow, intermediate, and deep sliding wedges.

Table 1. Summary of physical modeling involving geosynthetics.

Reference	Test type	Block		Interface	Excitation		Numerical simulation
		Stiffness	Base inclination		Type	Variables	
Hushmand and Martin (1990)	Centrifuge	Stiff	Horizontal	Geomembrane/Geomembrane Geomembrane/Geotextile	Harmonic Transient	Frequency 1.5 to 5 Hz Base acceleration 0.3 to 0.8g Normal stress up to 8.4 kPa	Conventional Newmark
Kavazanjian et al. (1991)	Shaking table	Stiff	Horizontal	Geomembrane/Geomembrane Geomembrane/Geotextile	Harmonic	Frequency 1.5 to 5 Hz Base acceleration 0.3 to 0.8g Normal stress up to 8.4 kPa	--
Yegian and Lahlaf (1992)	Shaking table	Stiff	Horizontal	Geomembrane/Geotextile	Harmonic	Frequency 2, 5, 10 Hz Base acceleration up to 0.4g Normal stress up to 13.6 kPa	--
Zimmie et al. (1994)	Shaking table Centrifuge	Stiff	Horizontal	Geomembrane/Geotextile Geonet/Geotextile	Harmonic	Frequency 5 to 40 Hz Base acceleration 0.04 to 4g Normal stress 2.1 to 84 kPa	--
Yegian et al. (1995a)	Shaking table	Stiff	Horizontal	Geomembrane/Geomembrane Geomembrane/Geotextile	Harmonic Transient	Frequency 2, 5, 10 Hz Base acceleration up to 0.6g	--
Yegian et al. (1995b)	Shaking table	Stiff	Horizontal	Geosynthetics/Sand Geomembrane/Geotextile	Harmonic Transient	Frequency 2, 5, 10 Hz Base acceleration up to 0.6g	--
Yegian and Harb (1995)	Shaking table	Stiff	Horizontal 1V:3, 4, 6, 10H	Various geosynthetics Geosynthetics	Harmonic	Frequency 1, 2, 5 Hz Base acceleration up to 1g	Yegian et al. (1991) simplified approach
De (1996)	Shaking table Centrifuge	Stiff	Inclined (2 and 4°)	Geomembrane/Geomembrane	Harmonic	Frequency 2.5 to 20 Hz Base acceleration 0.15 to 0.22g, duration = 2 s Normal stress up to 207 kPa	Goodman and Seed (1966) simplified Conventional Newmark
Kramer and Smith (1997)	Shaking table	Compliant	Inclined	Geomembrane/Geotextile	Harmonic Transient	Frequency is a function of fundamental period of model	Conventional Newmark, Modified Newmark

After subjecting the model to both simple harmonic and simulated earthquake motions, Lin and Whitman (1983) concluded that the decoupled approach generally provides a conservative estimate of permanent displacements (i.e. the permanent displacements calculated assuming decoupling are generally larger than those that would be obtained without assuming decoupling). This conservatism was the greatest at the fundamental period of the system. Around the fundamental period of the system, Lin and Whitman (1983) found that the magnitude of overprediction introduced by the decoupling assumption was negligible for shallow wedges (e.g. for cover veneer failures), approximately 20% for wedges of intermediate depth, and approached 100% for deep wedges.

Gazetas and Uddin (1994) used two-dimensional finite element analyses with slip elements to investigate the decoupling assumption. By comparing the permanent displacements computed with slip elements with those calculated by a decoupled procedure, based on the average acceleration time history (obtained by repeating the finite element analysis without the slip elements) for hypothetical and actual rockfill dams, Gazetas and Uddin (1994) concluded that the decoupling assumption generally does not have a significant influence on the calculated permanent displacements. However, an exception was noted for narrow band input motions that coincided with the fundamental period of the dam, in which case the decoupled procedure overpredicted permanent displacements by 100% or more.

Kramer and Smith (1997) developed a two degree-of-freedom analytical model to take into account both slip at the base of the potential failure mass and mass compliance. Kramer and Smith (1997) demonstrated the accuracy of their model in a limited number of shaking table tests (see Table 1).

Analyses performed by Kramer and Smith (1997) indicated that, if the fundamental period of the potential failure mass is very low with respect to the predominant period of base excitation, as in the case of a cover soil veneer, the combined effect of neglecting the slip at the base of the potential failure mass and mass compliance is not large and is generally conservative. Consistent with the findings of Lin and Whitman (1983) and Gazetas and Uddin (1994), when the fundamental period of the potential failure mass was close to the predominant period of the base motion, displacements calculated in a conventional Newmark analysis overpredicted the displacement calculated for the two degree-of-freedom system by up to 100%.

2.2 Effect of Two-Way Sliding

By integrating only when the yield acceleration is exceeded on one side of the acceleration time history, conventional Newmark analysis ignores the cumulative effects of block two-way sliding. The effect of ignoring two-way sliding has rarely been addressed in the technical literature. The reason for this omission lies in the fact that the Newmark analysis is used primarily for seismic slope deformation analyses where the horizontal acceleration needed to initiate upslope sliding (here referred to as the *inslope yield acceleration*) is so great that it is reasonable to assume that only downslope sliding occurs. Consequently, the effect of two-way sliding has generally been ignored.

With the advent of geosynthetic cover systems with thin veneers of protective soil cover, the effect of two-way sliding cannot be dismissed a priori. The relatively low shear strength of many geosynthetic interfaces and, consequently, the relatively low in-slope yield accelerations of a thin veneer cover suggest that the effect of two-way sliding on the results of a Newmark deformation analysis warrants consideration. The effect of two-way sliding can easily be introduced into a Newmark analysis by integrating the base plane acceleration time history modified by the out-of-slope yield acceleration on one side and by the in-slope yield acceleration on the other side, keeping track of the sign (direction) of the relative displacement. This approach is used in analyses described in Section 3.4 to evaluate the impact of two-way sliding on the accuracy of conventional Newmark analyses.

2.3 Effect of Vertical Acceleration Component

The effect of the vertical component of the earthquake motion on the sliding block normal force and, consequently, on permanent displacement is ignored in a conventional Newmark analysis. While a vertical acceleration acting downwards on the block in Figure 1 increases both the normal force, N, and the frictional component of the resisting force, T, at the base of the block, a vertical acceleration acting upwards on the block decreases both N and the frictional component of T.

Since the yield acceleration of the sliding mass is directly proportional to the limiting value of T, and the limiting value of T depends upon the value of N, applying a vertical acceleration to the block may increase or decrease the yield acceleration, depending on the direction of the vertical acceleration. The magnitude of the change in yield acceleration depends upon the relative contributions of interface adhesion and friction to the yield acceleration and the magnitude and direction of the vertical acceleration.

The impact of the vertical acceleration on the yield acceleration was explored by Ling and Leshchinsky (1997) in the context of the seismic stability of geosynthetic landfill cover systems. Ling and Leshchinsky (1997) introduced a vertical pseudo-static force into the limit equilibrium analysis of a landfill cover veneer. The vertical and horizontal pseudo-static forces were assumed to act simultaneously in their analysis, with the horizontal force acting out-of-slope and the vertical force acting upwards to produce the maximum destabilizing effect. For vertical forces equal to 20 and 50% of the horizontal force, Ling and Leshchinsky (1997) found the influence of the vertical force on calculated pseudo-static factors of safety practically insignificant.

The pseudo-static approach of Ling and Leshchinsky (1997) can be used to introduce the vertical component of the ground motion into a Newmark deformation analysis. In a conventional Newmark analysis, the yield acceleration is defined as the horizontal acceleration which yields a pseudo-static factor of safety equal to unity. However, the acceleration which results in a pseudo-static factor of safety equal to unity can also be calculated either by assuming a direction for the resultant acceleration that is not horizontal or by applying both horizontal and vertical accelerations simultaneously to the potential failure mass.

Most of the computer programs commonly used in practice for limit equilibrium slope stability analyses provide for one or both of these approaches. While this appears to provide a straightforward manner in which to introduce the vertical component of the ground motions into both limit equilibrium stability and Newmark deformation analyses, it is not widely used. A major barrier to the adoption of this approach is the lack of a rational means of determining the appropriate value of the vertical pseudo-static force to use in the limit equilibrium analysis.

Figure 3 shows horizontal and vertical components of a typical earthquake ground motion. The frequency of these two ground motion components are significantly different, and the peaks of the two components are typically out of phase. Consequently, it is difficult, if not impossible, to establish a functional relationship between the two components of ground motion in terms of the dynamic forces acting on the failure mass and, therefore, between the horizontal and vertical pseudo-static forces used to calculate the yield acceleration.

In fact, if the relationship between the vertical and horizontal ground motions is assumed to be random, the vertical acceleration is as likely to decrease the yield acceleration as it is to increase the yield acceleration, and the net impact of the vertical ground motion on the permanent seismic deformation may be deduced to be negligible. Based upon this logic, a vertical pseudo-static force should not be applied when calculating the yield acceleration of a potential failure mass for use in a Newmark deformation analysis.

Yan et al. (1996a) developed a more formal approach of considering the vertical component of the earthquake ground motions in a Newmark analysis. Yan et al. (1996a) applied the dynamic equation of motion in both the horizontal and vertical directions to calculate the permanent seismic displacement of a rigid block sliding on a plane. In this approach, the vertical and horizontal acceleration components of the ground motions are applied simultaneously to the base plane. The Yan et al. (1996a) model also allows two-way sliding, with out-of-slope and in-slope yield accelerations applied as explained in Section 2.2.

The results of a parametric study presented by Yan et al. (1996a) using actual acceleration time histories with the phase of the vertical and horizontal motions retained as-

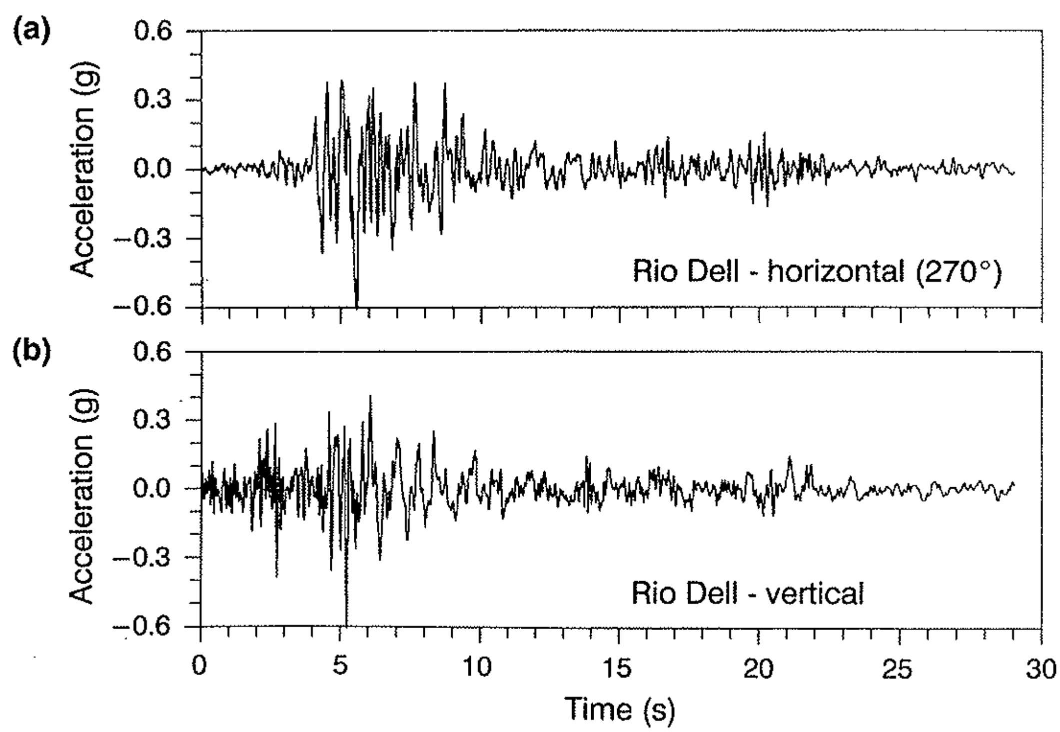

Figure 3. Rio Dell, California, accelerogram: (a) horizontal component; (b) vertical component.

recorded indicate that including the vertical ground motion in the analysis can both increase and decrease calculated permanent seismic displacement. An analysis of a geosynthetic cover with an inclination of 1V:3H (1 vertical:3 horizontal) and an interface friction angle of 8° indicated that the influence of including the vertical ground motion component in the analysis on the permanent seismic displacement was of the order of ± 10%. The relative insensitivity of the calculated displacement to the vertical motion was attributed to the random phasing and zero mean processes nature of the horizontal and vertical components of the ground motion. Two-way sliding was also shown to have little impact on the calculated displacements as the in-slope yield acceleration was significantly higher than its out-of-slope counterpart for this example.

2.4 Degrading Yield Acceleration

In the classical paper by Newmark (1965), it was recognized that the assumption of a constant shear strength on the block/plane interface, and therefore the implicit assumption of a constant yield acceleration, might not always be appropriate. Newmark suggested that modification of the method to include cyclic degradation of the shear strength, i.e. degradation of the yield acceleration, might be warranted. However, this suggestion has been largely ignored by the geotechnical and geosynthetics communities and, consequently, is not implemented in most computer programs for Newmark analysis. Faced with the necessity of choosing a constant value of yield acceleration for use in seismic displacement analysis of geotechnical systems, most engineers use a yield acceleration evaluated from residual, or large deformation, shear strength parameters to provide a conservative basis for permanent seismic displacement assessment.

The influence of a degrading yield acceleration on the seismic deformation potential of geosynthetic interfaces was studied by Matasovic et al. (1997). Matasovic et al. (1997) developed a trilinear model for degradation of yield acceleration as a function of displacement. In this model, shown in Figure 4, it was assumed that degradation of the initial "peak" value of yield acceleration, k_{y1}, starts when the calculated permanent displacement reaches an initial threshold displacement, δ_1. For a geosynthetic interface, this initial threshold displacement value corresponds to the peak of the interface shear force-displacement curve, as indicated in Figure 4a. After reaching the initial

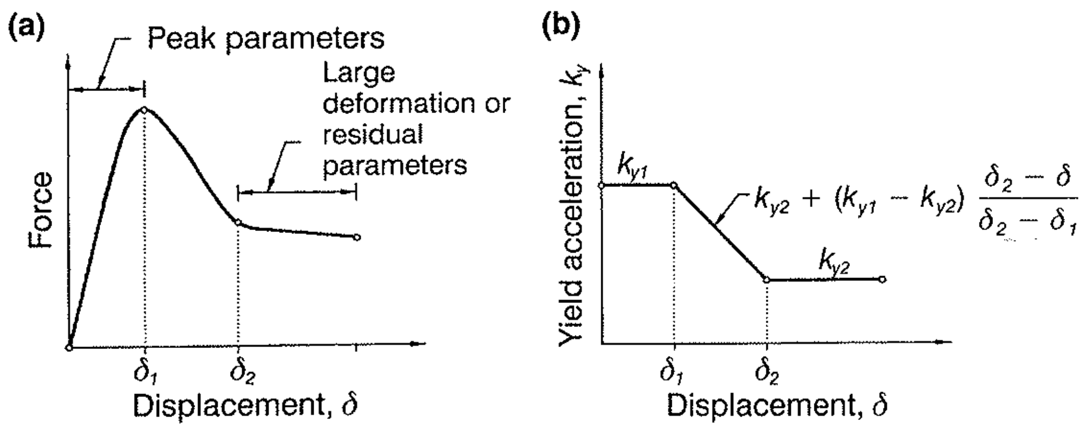

Figure 4. Composite landfill cover interfaces: (a) measured force-displacement curve; (b) yield acceleration degradation model by Matasovic et al. 1997.

threshold displacement, the yield acceleration degrades linearly with increasing interface displacement until the ultimate "residual" yield acceleration, k_{y2}, is reached at a second threshold displacement, δ_2. This second threshold displacement value corresponds to the displacement at which the large deformation, or residual, shear strength of the interface is reached.

Figure 5 illustrates how the degrading yield acceleration model impacts the results of a Newmark deformation analysis. Figure 5 illustrates the difference between the results of a conventional Newmark analysis with a constant yield acceleration based upon residual shear strength parameters and a Newmark analysis using the degrading yield acceleration model. Analyses performed by Matasovic et al. (1997) support the logical inference that cumulative displacement calculated with the degradation model illustrated in Figure 4 is consistently lower than the cumulative displacement calculated by

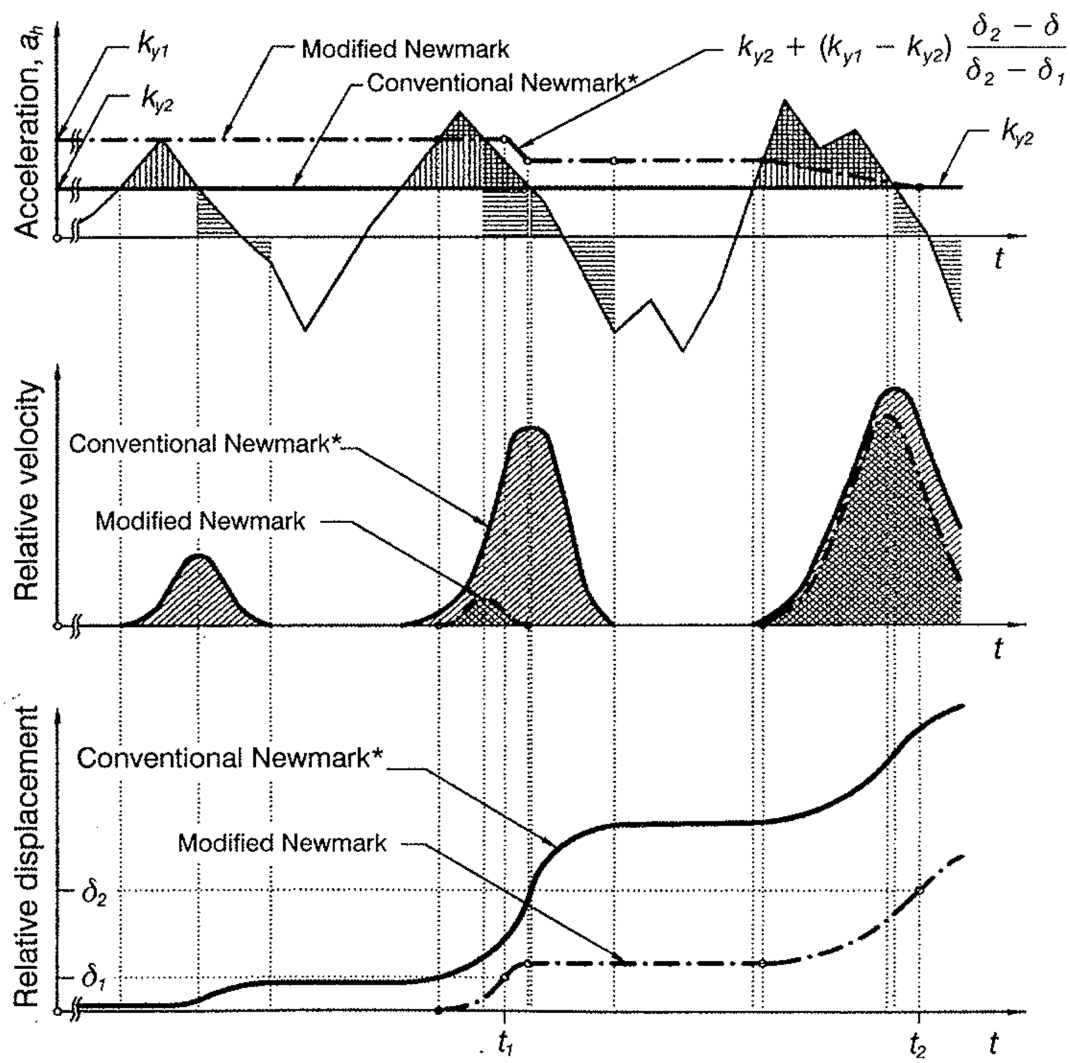

Figure 5. Comparison of classical and modified Newmark analysis integration schemes.

Note: Yield acceleration evaluated using large deformation shear strength parameters ($k_y = k_{y2}$).

the conventional Newmark procedure with a constant yield acceleration based upon residual strength parameters.

3 NEWMARK SEISMIC DEFORMATION ANALYSIS FOR GEOSYNTHETIC COVERS

3.1 General

A review of previous studies on the accuracy of conventional Newmark seismic deformation analysis indicates that the simplifying assumptions of decoupling the seismic response of the potential failure mass from the permanent displacement of the mass and of neglecting failure mass compliance in the deformation analysis introduce an acceptable level of conservatism (no more than a factor of 2) into a conventional Newmark analysis for a geosynthetic cover system. Therefore, the balance of this study focused on the individual and combined effects of other parameters, i.e. two-way sliding, the vertical component of the ground acceleration, and degradation of the yield acceleration, on the results of Newmark seismic deformation analyses applied to geosynthetic cover systems.

To study these effects, a computer program with provisions for these effects developed by Yan et al. (1996b) was used. This computer program incorporates the effects of two-way sliding, formally takes into account the vertical acceleration component of the ground motion, and employs the displacement-dependent yield acceleration model proposed by Matasovic et al. (1997). A series of parametric studies was conducted using this computer program to investigate the impact of these factors on the results of a Newmark deformation analysis.

3.2 Model Parameter Development

In order to develop the parameters for the degrading yield acceleration model for the parametric studies described herein, the results of a series of three direct shear interface tests on a compacted soil-nonwoven geotextile-textured geomembrane-compacted soil sandwich was used. The testing conditions were established to simulate the "typical" geosynthetic cover system schematically shown in the inset of Figure 6.

This "typical" geosynthetic cover system consists of (top to bottom): vegetative cover soil (silty sand); double-sided geocomposite drainage layer; textured, 1 mm thick geomembrane; and compacted foundation soil. The testing was performed in accordance with the ASTM D 5321 standard test method. Prior to the testing, the geomembrane and geotextile surfaces were wetted with water. The testing was carried out under normal stresses, σ_n, of 6.9, 20.7, and 69.0 kPa, corresponding to approximate vegetative layer thicknesses of 0.4, 1.2, and 4.0 m, respectively. The displacement rate used in the tests was 1 mm/minute. The testing results are presented in Figures 6 and 7.

Figure 6 shows the load-deformation curves from the three laboratory interface shear tests. Figure 7 presents an interpretation of the test results of Figure 6 in terms of the interface shear strength parameters a (adhesion) and ϕ (friction angle) for both peak and large deformation (residual) conditions. The threshold displacements δ_1 and δ_2, indi-

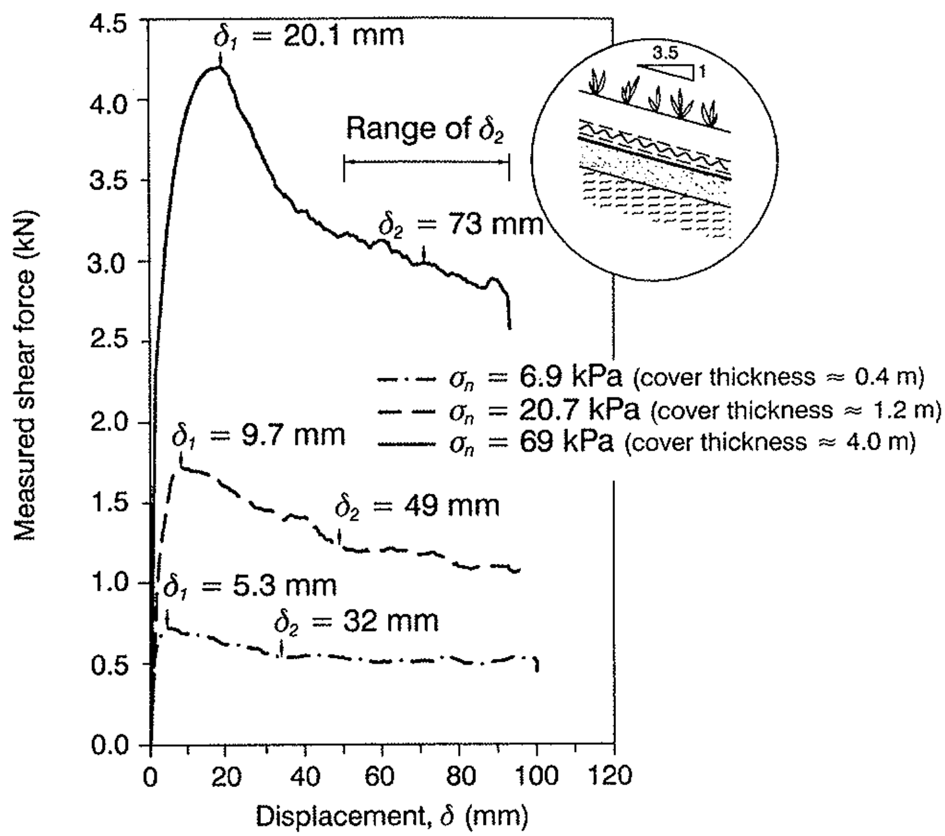

Figure 6. **Compacted soil-geotextile interface direct shear testing results.**

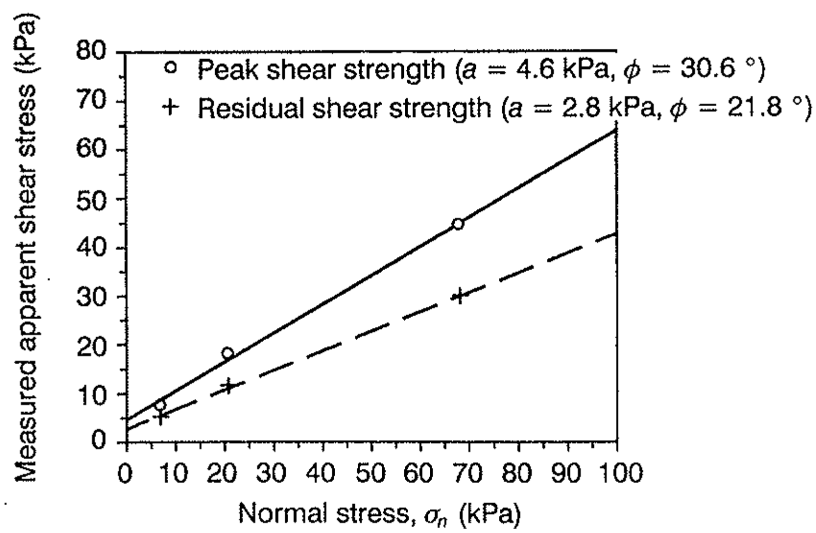

Figure 7. **Interpretation of compacted soil-geotextile interface direct shear testing results.**

cated in Figure 6 and used to develop the peak and large deformation failure envelopes in Figure 7, were established by visual inspection.

Table 2 presents the static factors of safety, FS_{STAT}, and the corresponding peak and large deformation yield accelerations for the three different normal stresses considered in the analysis. Yield accelerations were calculated for a 1V:3.5H slope using the infinite slope equations for veneer-type failures proposed by Matasovic (1991). Yield accelerations k_{y1} and k_{y2} were calculated for the out-of-slope direction, while yield accelerations k_{y1}^{in} and k_{y2}^{in} were calculated for the in-slope direction. The data in Table 2 are plotted in Figure 8 in the form of the yield acceleration degradation model of Matasovic et al. (1997).

3.3 Representative Ground Motions

The parametric study presented herein was carried out using a set of five horizontal acceleration time histories for earthquakes varying in moment magnitude, M, from 5.2 to 8.0. The five horizontal time histories used in the parametric analyses are summarized in Table 3. This suite of horizontal time histories is shown in the form of acceleration response spectra in Figure 9.

Table 2. Summary of the cover system parameters (1V:3.5H infinite slope).

Cover thickness (m)	Normal stress, σ_n (kPa)	Peak parameters			Residual parameters		
		FS_{STAT}	k_{y1} (g)	k_{y1}^{in} (g)	FS_{STAT}	k_{y2} (g)	k_{y2}^{in} (g)
0.4	6.9	4.56	0.87	-1.91	2.94	0.50	-1.26
1.2	20.7	2.90	0.46	-1.34	1.91	0.23	-0.94
4.0	69.0	2.32	0.32	-1.14	1.55	0.14	-0.82

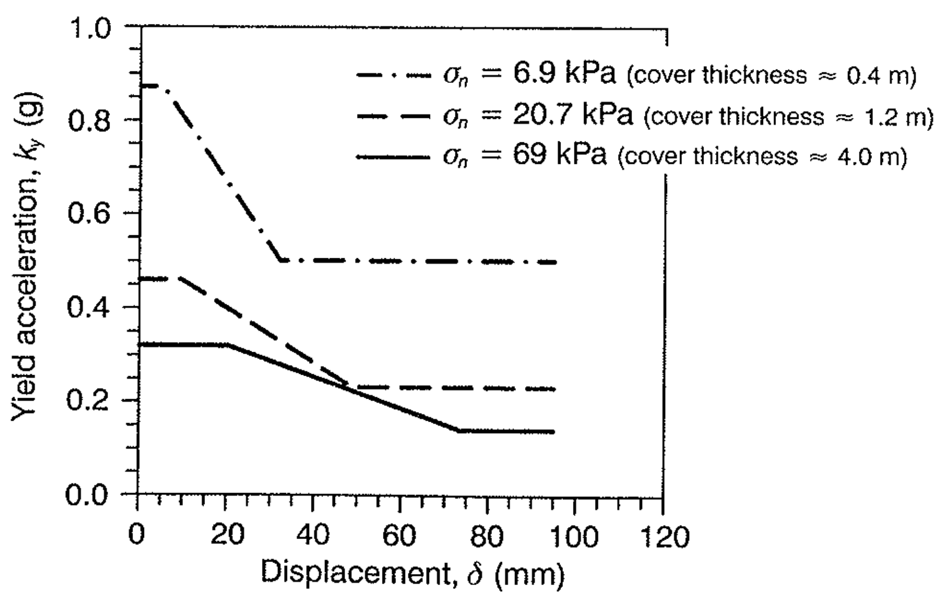

Figure 8. Yield acceleration degradation model for three landfill cover thicknesses analyzed by the modified Newmark analysis.

Table 3. Summary of accelerograms.

Accelerogram	Earthquake magnitude, M	Accelerogram duration, D_s (s)
Lima, Peru	8.0	47.8
Rio Dell, California (horizontal component)	7.0	15.5
Rio Dell, California (vertical component)	7.0	19.0
Corralitos-Eureka Canyon Road, California	6.9	6.9
C4, Synthetic (Silva 1995)	6.7	6.7
Golden Gate Park, California	5.2	3.2

Notes: Magnitudes are based upon the Moment Magnitude scale. Durations are based upon the significant duration as defined by Trifunac and Brady (1975).

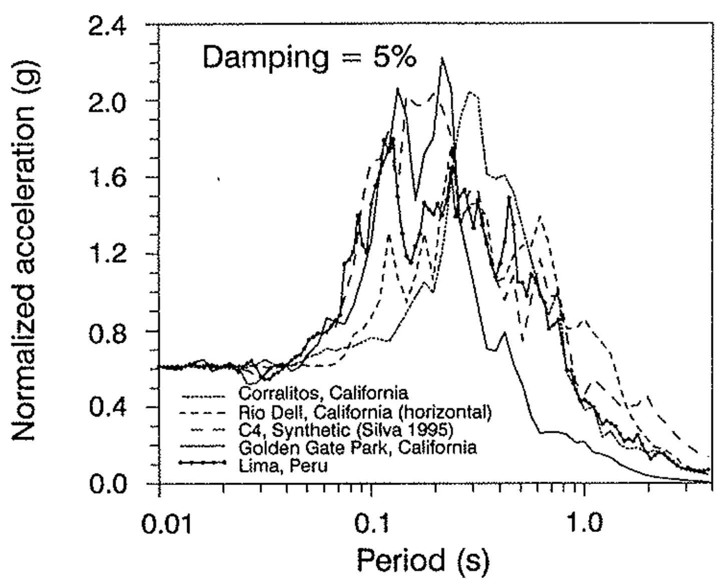

Figure 9. Acceleration response spectra of accelerograms considered in this study.

Figure 9 indicates that the selected suite covers a broad range of frequency contents and predominant periods. Table 3 indicates that these time histories encompass a broad range of duration, expressed in terms of significant duration, D_s, as defined by Trifunac and Brady (1975). In addition to these horizontal time histories, the vertical component of the Rio Dell time history from the $M = 7.0$ Petrolia earthquake was used to investigate the influence of the vertical component of the ground motion on the results of the Newmark deformation analyses.

3.4 Parametric Analyses

An initial set of Newmark deformation analyses was carried out using the two components of the Rio Dell record, shown graphically in Figure 3, to investigate the effects of two-way sliding and the vertical acceleration component on the computed deformation. The in- and out-of-slope yield accelerations listed in Table 2, corresponding to the residual shear strength parameters evaluated at a normal stress of 69 kPa, were employed in these analyses. The out-of-slope yield acceleration k_{y2} = 0.14g, corresponding to 4 m of vegetative cover soil, and the horizontal component of the Rio Dell accelerogram were used to establish benchmark seismic deformation values representing the results of a conventional Newmark analysis.

The calculations were performed for *PHGA* values of up to 1.1g, representing the range of *PHGA* values considered likely to be encountered in practice. The effect of two-way sliding was investigated using the out-of-slope yield acceleration of k_{y2} = 0.14g and the in-slope yield acceleration of k_{y2}^{in} = 0.82g presented in Table 2. The difference between permanent displacements calculated with and without two-way sliding was, even at the extreme *PHGA* level of 1.1g, negligible for all practical purposes.

Next, the effect of the vertical acceleration component on the calculated displacement was investigated. The same Rio Dell accelerogram and the same set of yield accelerations were used to establish the benchmark displacement values. To provide an upper bound on the impact of the vertical component of the ground motions, the vertical component of the Rio Dell accelerogram was scaled such that the peak vertical ground acceleration, *PVGA*, was equal to 1.7 times the *PHGA*. While the *PVGA* is typically assumed to be 0.5 to 0.67 times the *PHGA*, the 1.7 value was chosen as the upper bound on the ratio of *PVGA* to *PHGA* based upon examination of the most extreme strong ground motion records from the Loma Prieta, Northridge, and Petrolia California earthquakes.

In a conventional Newmark analysis, the accelerogram is integrated in two different trials, once assuming sliding in the direction of the top half of the accelerogram and once assuming sliding in the direction of the bottom half of the accelerogram. The largest permanent displacement calculated in the two trials is considered the calculated displacement. If the vertical component of the ground motion is included in the analysis, there are four possible combinations of vertical and horizontal time histories and, therefore, four trials are required to exhaust all possibilities.

The results of the analysis in which the vertical component of the ground motion is considered are summarized in Figure 10. Figure 10 indicates that the computed displacement can be both increased and decreased by including the vertical motion in the analysis. Consistent with conventional analysis, the results of the trial that produces the largest calculated displacement is considered the calculated displacement. Even with this assumption, Figure 10 indicates that the effect of the vertical acceleration component on the permanent displacement calculated in a Newmark analysis is relatively small and can be neglected for practical purposes, which is consistent with the previous studies discussed in Section 2.3.

Based on the review of previous studies (Section 2.4), the effect of degrading yield acceleration appears to be the most important effect for geosynthetic covers. Therefore, the effect of a degrading yield acceleration was investigated using all five time histories. Conventional Newmark deformation analyses using a constant yield acceleration were carried out for *PHGA* values of up to 1.1g using constant yield accelerations based upon the peak and residual shear strength parameters and using the degrading yield acceleration model. The analyses were carried out for the three cover configurations listed in Table 2. Results of these analyses are presented in Figures 11 to 15.

Figures 11a, 12a, 13a, 14a, and 15a, presenting the results for the 0.4 m thick vegetative cover soil layer (σ_n = 6.9 kPa), indicate that the seismic displacement response of the geosynthetic cover system calculated using the Newmark analysis with a degrading yield acceleration (modified Newmark analysis) is the same as for a conventional New-

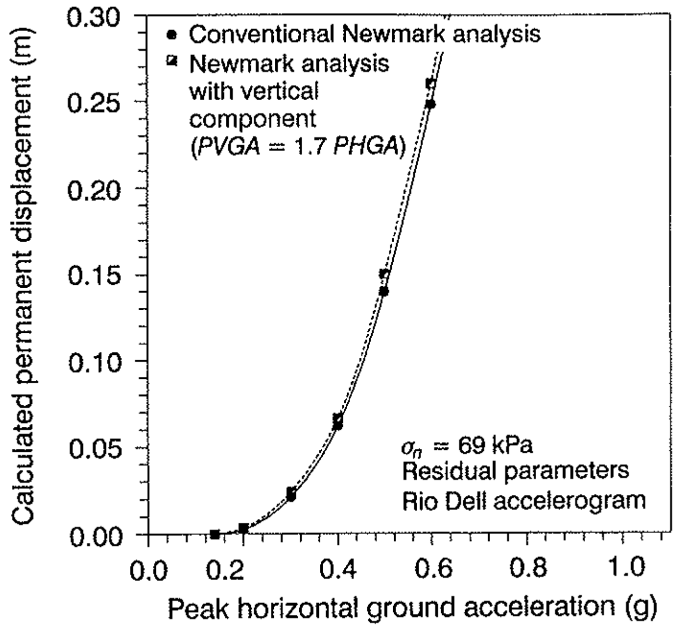

Figure 10. Influence of the vertical acceleration component on the results of a conventional Newmark analysis.

mark analysis based on peak shear strength parameters. This relative insensitivity of the calculated displacement to degradation of yield acceleration is readily explained by the fact that, due to the low normal stress, the initial (peak) value of yield acceleration is relatively high (k_{yI} = 0.87g).

In Figures 11b, 12b, 13b, 14b, and 15b, presenting the results for the intermediate normal stress level (σ_n = 20.7 kPa) corresponding to 1.2 m of vegetative cover soil, the effect of a degrading yield acceleration on the calculated displacement becomes apparent. Figures 11b, 12b, 13b, 14b, and 15b indicate that the bifurcation between the results of a Newmark analysis based upon the peak strength and a Newmark analysis using the degrading yield acceleration model starts when the permanent displacement value calculated using the modified Newmark analysis reaches the initial threshold displacement δ_I = 9.7 mm. Beyond the initial threshold displacement, the residual shear stress parameters rapidly start to influence the displacement response in the degrading yield acceleration case.

Figures 11b, 12b, 13b, 14b, and 15b also show that, for all five accelerograms, a relatively high *PHGA* value (0.7g or more) is required to induce a displacement response that converges on the displacement calculated using a constant yield acceleration based upon the residual shear strength parameters. The same trend in calculated displacement can be seen for the higher normal stress of 69 kPa, corresponding to 4 m of vegetative cover soil, presented in Figures 11c, 12c, 13c, 14c, and 15c.

Figures 11 through 15 demonstrate that a conventional Newmark analysis based solely on the residual shear strength parameters always produces conservative results (i.e. greater calculated displacements) compared to the degrading yield acceleration model (modified Newmark analysis).

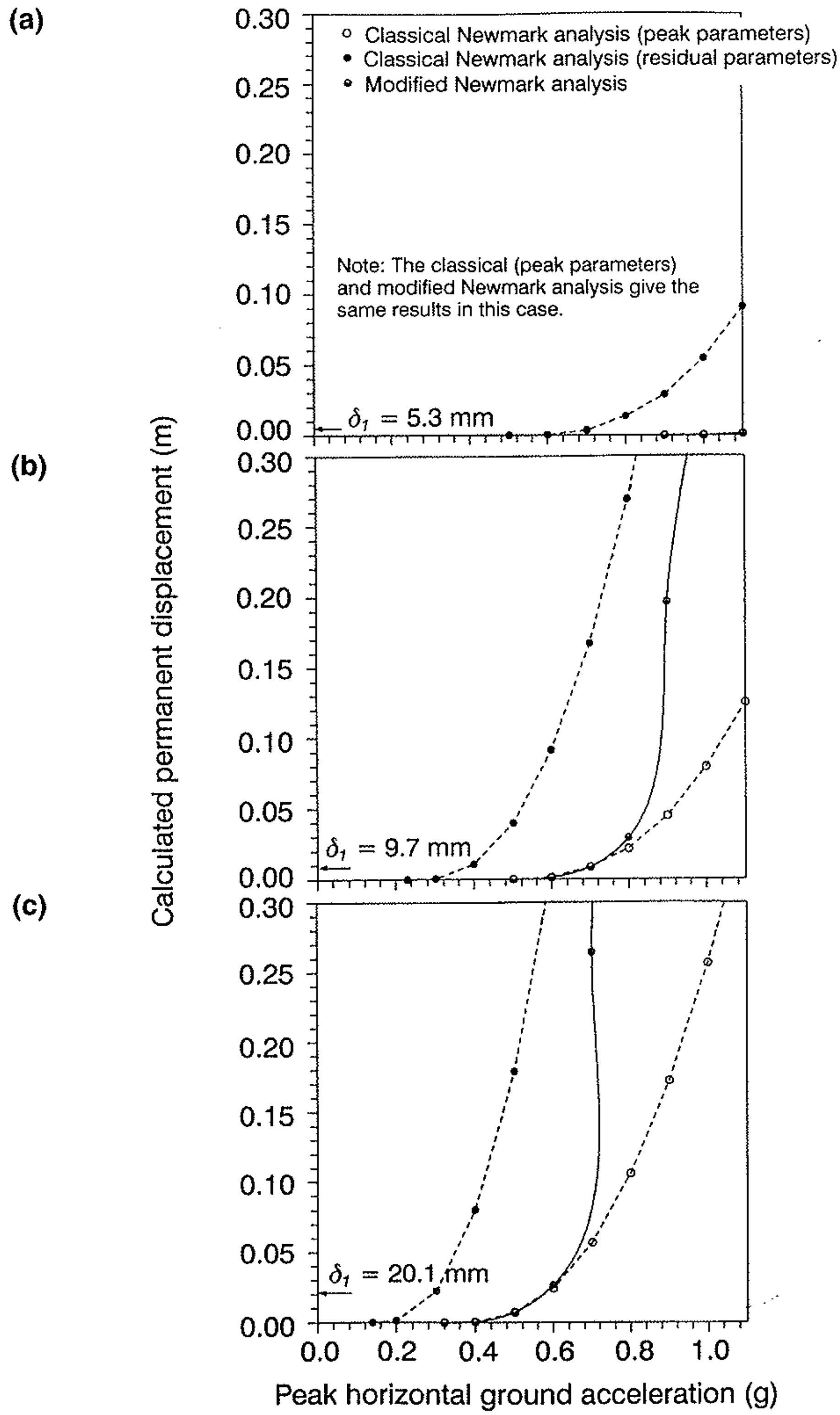

Figure 11. **Lima, Peru, accelerogram - comparison of classical and modified Newmark analysis at different normal stress and peak horizontal ground acceleration levels: (a) σ_n = 6.9 kPa; (b) σ_n = 20.7 kPa; (c) σ_n = 69.0 kPa.**

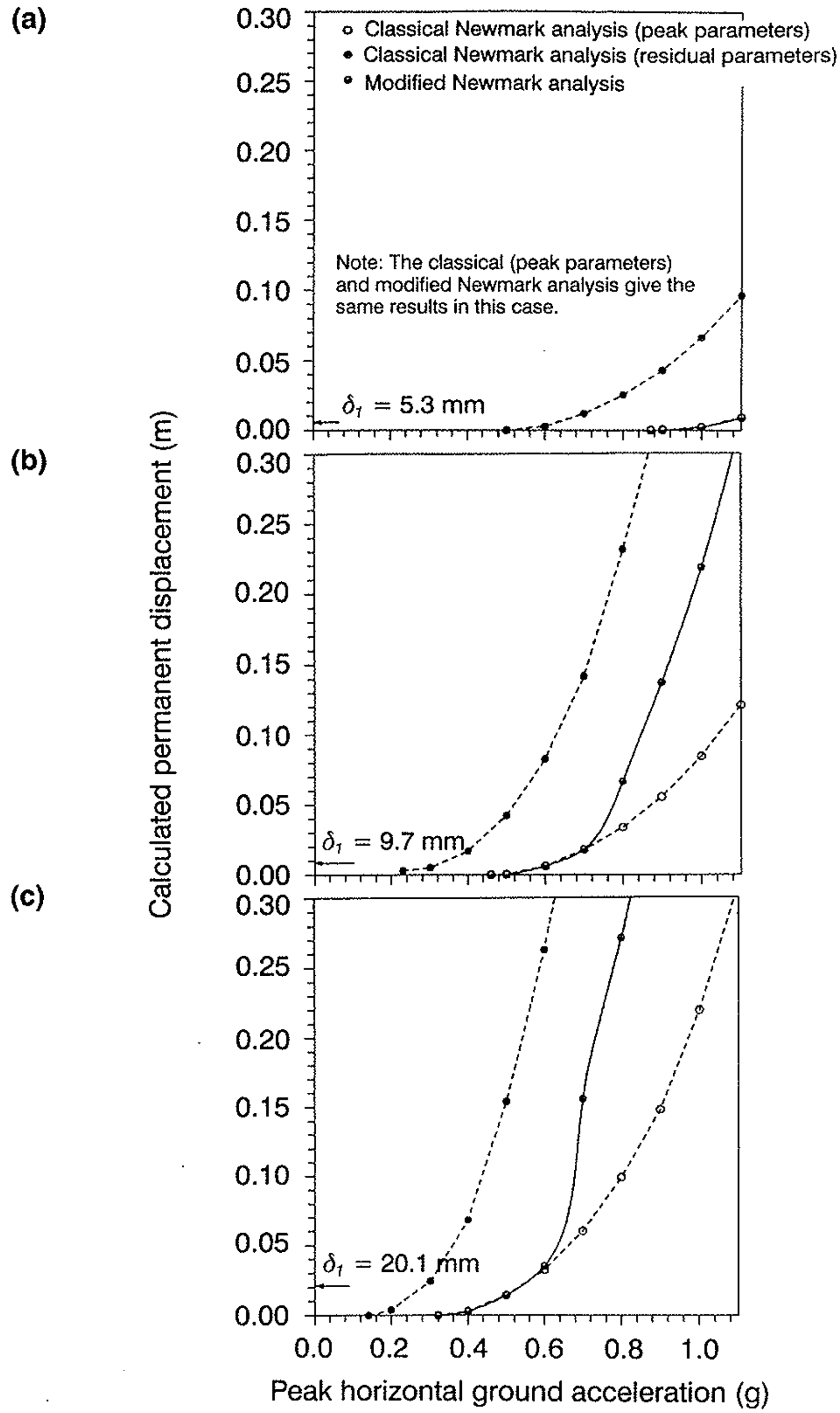

Figure 12. Rio Dell, California, accelerogram - comparison of classical and modified Newmark analysis at different normal stress and peak horizontal ground acceleration levels: (a) σ_n = 6.9 kPa; (b) σ_n = 20.7 kPa; (c) σ_n = 69.0 kPa.

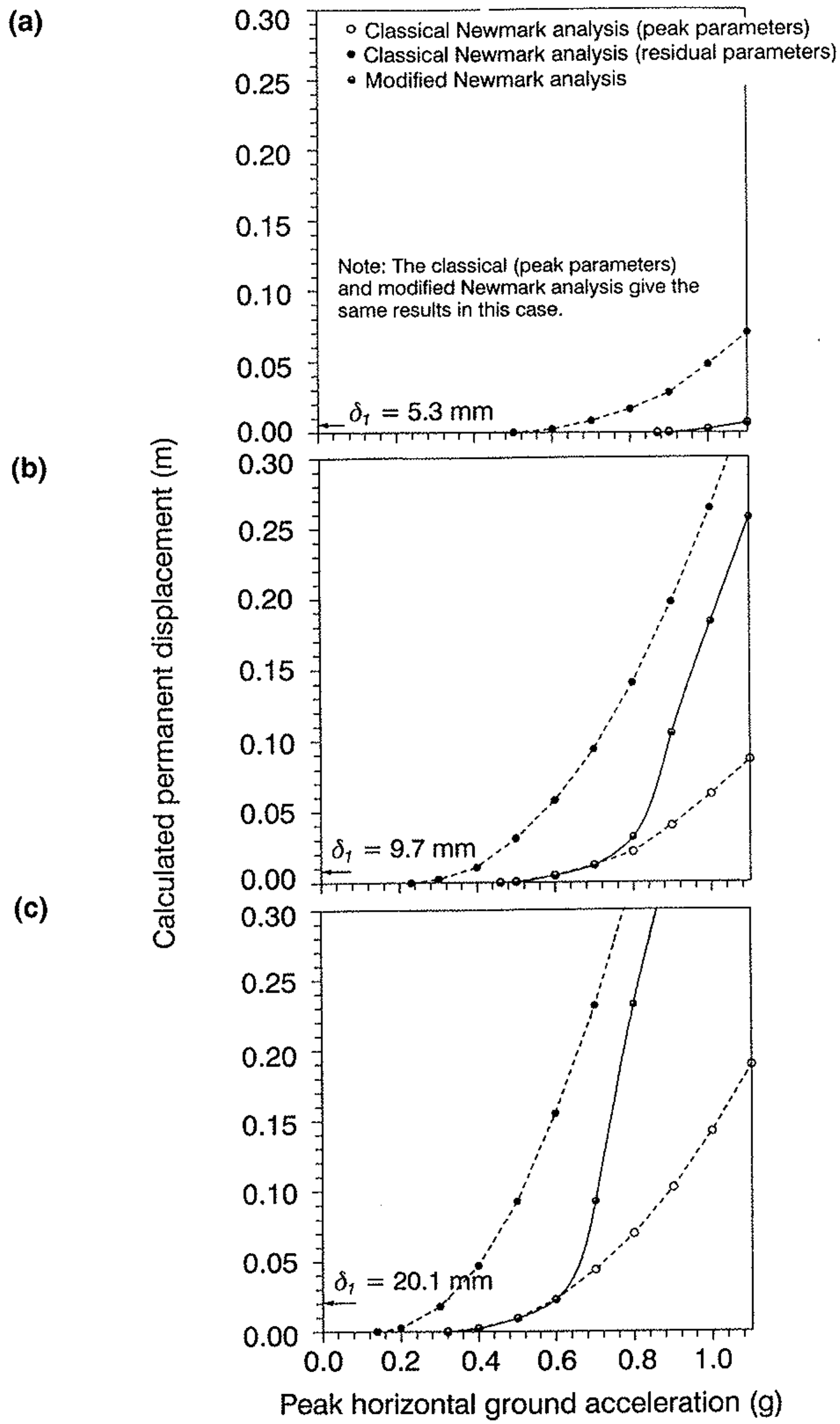

Figure 13. Corralitos-Eureka Canyon Road, California, accelerogram - comparison of classical and modified Newmark analysis at different normal stress and peak horizontal ground acceleration levels: (a) σ_n = 6.9 kPa; (b) σ_n = 20.7 kPa; (c) σ_n = 69.0 kPa.

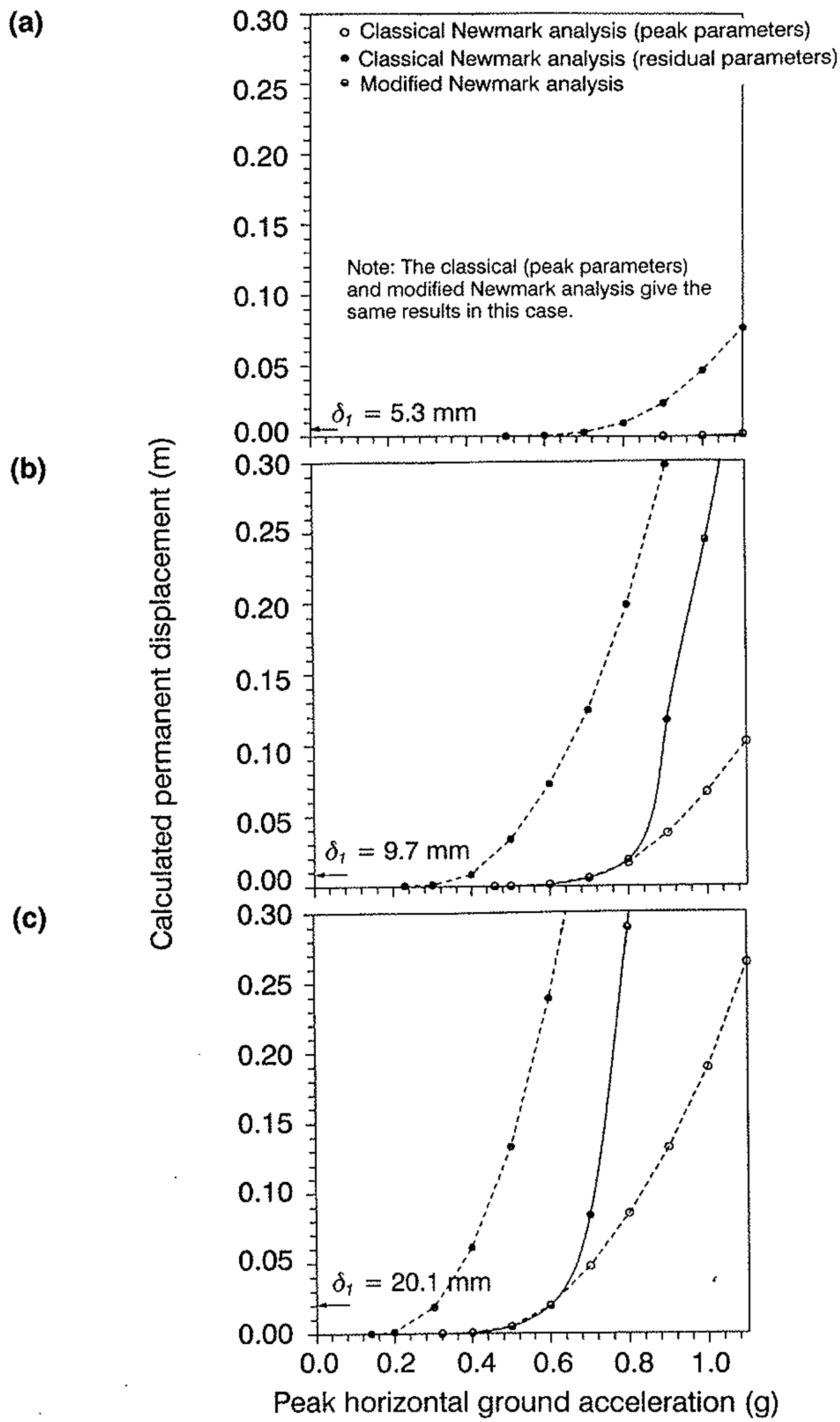

Figure 14. **C4-synthetic accelerogram - comparison of classical and modified Newmark analysis at different normal stress and peak horizontal ground acceleration levels: (a) σ_n = 6.9 kPa; (b) σ_n = 20.7 kPa; (c) σ_n = 69.0 kPa.**

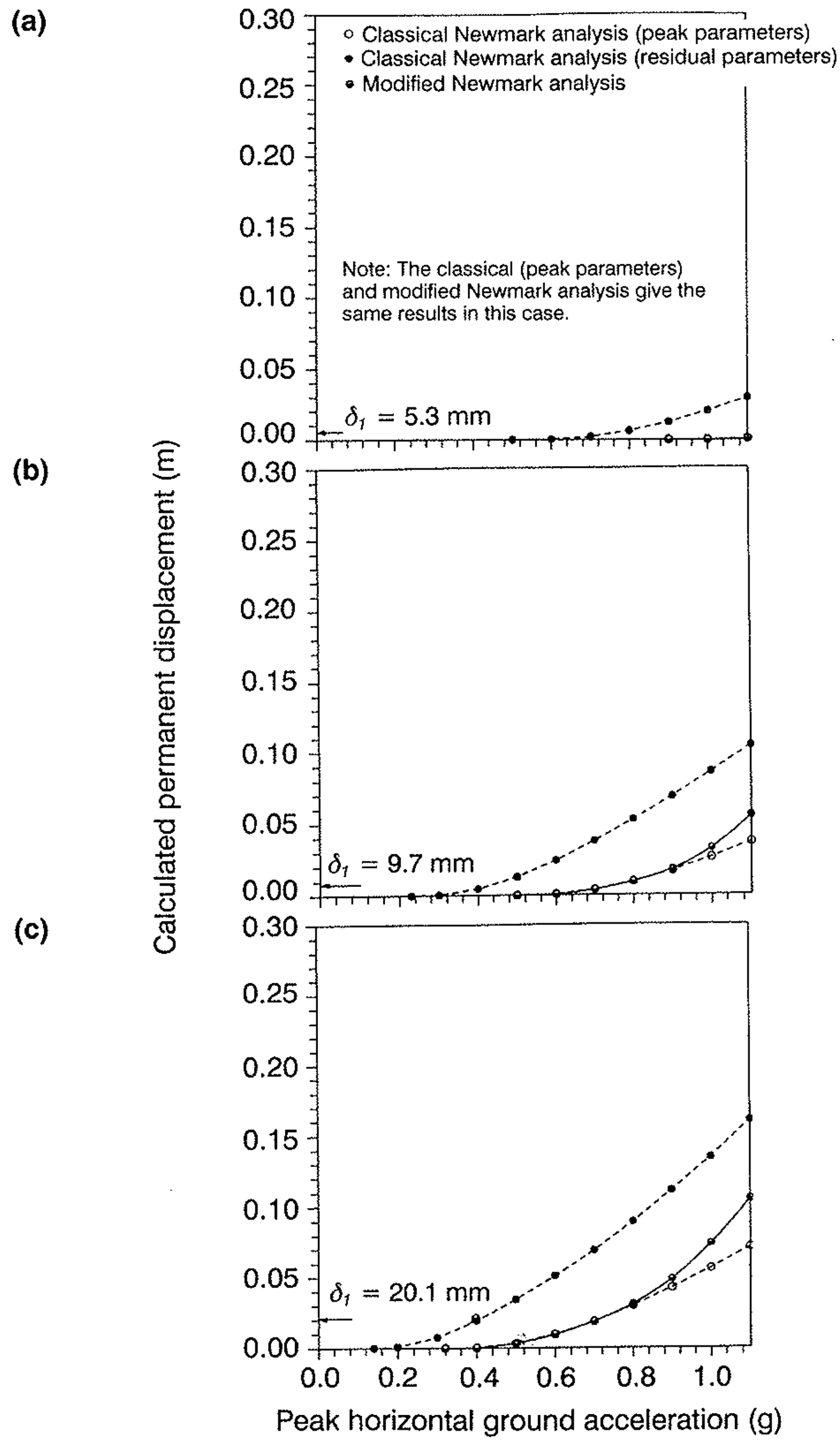

Figure 15. Golden Gate Park, California, accelerogram - comparison of classical and modified Newmark analysis at different normal stress and peak horizontal ground acceleration levels: (a) σ_n = 6.9 kPa; (b) σ_n = 20.7 kPa; (c) σ_n = 69.0 kPa.

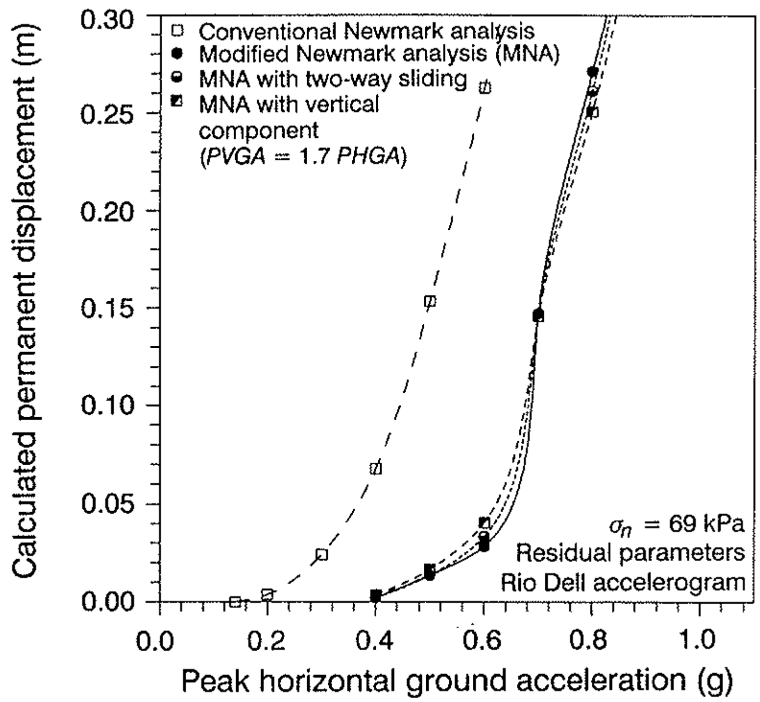

Figure 16. Influence of the second threshold displacement, δ_2, on the results of a modified Newmark analysis.

The combined effects of a degrading yield acceleration, two-way sliding, and the vertical acceleration component were also investigated using the Rio Dell accelerogram and the yield accelerations corresponding to the higher normal stress of 69 kPa (representing 4 m of vegetative cover soil). The results of these analyses, presented in Figure 16, further substantiate the conclusion that the influences of the two-way sliding and vertical acceleration component on calculated permanent displacements may be neglected for practical purposes.

An additional set of parametric analyses was performed to investigate the sensitivity of the degrading yield acceleration analysis to the threshold displacements δ_1 and δ_2. The selection of these parameters from the test results shown in Figure 6 was done subjectively, as mentioned in Section 3.2. The selection of δ_2 for the highest stress level in Figure 6 is particularly subject to different interpretation. In order to investigate the influence of δ_2 on the results of the modified Newmark analysis, the analysis for the normal stress level of 69 kPa (4 m of vegetative cover soil) was repeated for a reference *PHGA* value of 0.61g with δ_2 varied from 20.1 mm (equal to δ_1, corresponding to an instantaneous drop from the peak strength to the residual strength) to 93 mm (the limit of the testing apparatus). Results of this parametric study are shown in Figure 17.

Figure 17 indicates that, at the selected reference *PHGA* level of 0.61g, the results of the displacement calculations are strongly dependent on the cumulative deformation potential of the accelerogram employed in the analyses. Based upon the data in Figure 17 and in Table 3, the cumulative displacement appears to be related to the significant duration and magnitude of the earthquake record. The discrepancy between the displacements calculated for two extreme estimates of δ_2 is large. For the motion with

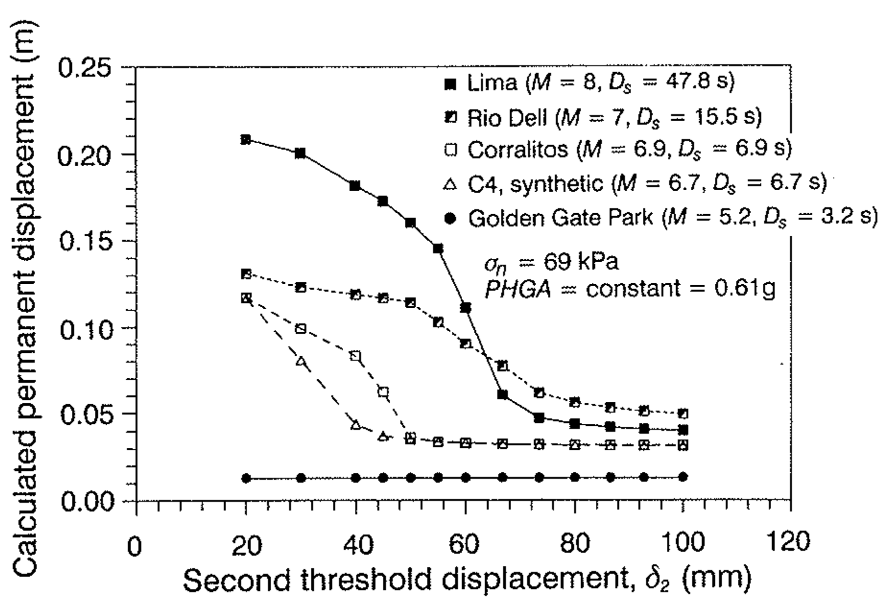

Figure 17. Influence of the vertical acceleration component on the results of a modified Newmark analysis.

the largest significant duration and magnitude (the Lima, Peru, record), the calculated displacement for the maximum value of δ_2 is 0.04 m while the calculated displacement of the minimum value of δ_2 is over five times as great, 0.21 m. For the motion with the smallest significant duration and magnitude (the Golden Gate Park, California record), the discrepancy between the calculated displacement at the extreme values of δ_2 is negligible. Figure 17 further indicates that, in the vicinity of the δ_2 value of 73 mm that was estimated subjectively, the calculated permanent displacement is relatively stable for all five accelerograms.

Figure 17 also illustrates the importance of selecting accelerograms of appropriate magnitude and/or duration for use in a Newmark deformation analysis. In several of the chart solutions for Newmark displacement analyses used in current practice (e.g. Hynes and Franklin 1984), the influence of earthquake magnitude and/or duration is ignored. Ignoring earthquake magnitude and/or duration can lead to an unrealistic assessment of seismic deformation potential. This is particularly true for small magnitude events if the "upper bound" displacement curve from a deformation chart which ignores earthquake magnitude is employed. As indicated in Figure 17, in such a case the result may be excessively conservative (i.e. the calculated displacement value may be too large).

4 CONCLUSIONS

Conventional Newmark seismic deformation analyses are based on the following five simplifying assumptions: (i) the potential failure mass is rigid (noncompliant); (ii) the dynamic response of the failure mass is not influenced by (coupled with) the permanent displacement (slip) that occurs along the failure surface; (iii) permanent displacement accumulates in only one direction (the downslope direction); (iv) the vertical component of the ground motion does not influence the calculated permanent displacement; and (v) the yield acceleration of the potential failure mass is constant.

Review of relevant previous studies indicates that the assumptions of decoupled seismic response and displacement and of a noncompliant failure mass are of relatively minor significance for assessing the deformation potential of geosynthetic covers for solid waste landfills and other facilities, resulting in overprediction of the permanent displacement by at most a factor of 2. The analyses presented herein also indicate that the effects of two-way sliding and of the vertical component of the earthquake ground motions are, for most practical purposes, negligible for geosynthetic cover systems. However, for strain softening materials such as the interfaces of typical geosynthetic cover systems, the effect of degradation of the yield acceleration from an initial peak value to an ultimate residual value may be an important factor impacting the accuracy of conventional Newmark analyses.

The analyses described herein for three representative geosynthetic cover configurations using the yield acceleration degradation model proposed by Matasovic et al. (1997) demonstrate that conventional Newmark deformation analyses which use a constant yield acceleration based solely on residual shear strength parameters may be excessively conservative, i.e. may lead to a calculated permanent displacement much greater than the value calculated taking into account degrading yield acceleration. The degree of conservatism depends to a large extent upon the value of the calculated seismic deformation compared to the threshold deformations at which the peak and residual strengths are mobilized. The analyses presented herein further demonstrate that selection of an acceleration time history of appropriate magnitude and/or duration is a key factor in performing a seismic displacement analysis.

In deciding on whether to base a geosynthetic cover design on an analysis using a constant yield acceleration or on an analysis using a degrading yield acceleration, a variety of other factors in addition to the value of the calculated permanent seismic displacement should be considered. Factors such as creep, the cyclic nature of earthquake loading, and transient seismic displacements may accelerate the degradation of the interface shear strength to the value corresponding to the residual shear strength parameters. Therefore, evaluation of the appropriate interface shear strength values for use in permanent seismic deformation analyses still requires more research.

Given the interface testing state of practice, two boundaries may be calculated for the permanent displacement: (i) an upper boundary using a constant yield acceleration based on residual strength parameters; and (ii) a lower boundary using a degrading yield acceleration as indicated in the current paper.

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NOTATIONS

Basic SI units are given in parentheses.

a	=	interface adhesion (Pa)
$a_h(t)$	=	horizontal acceleration time history of the block (m/s^2)
B	=	width of the block/plane interface (m)
D_s	=	significant duration of strong shaking (s)
FS_{STAT}	=	static factors of safety (dimensionless)
g	=	acceleration due to gravity (m/s^2)
$k_h(t)$	=	seismic coefficient (dimensionless)
k_y	=	yield acceleration (m/s^2)
k_{y1}	=	yield acceleration calculated on the basis of peak shear strength parameters (m/s^2)
k_{y1}^{in}	=	k_{y1} needed to initiate upslope sliding (m/s^2)
k_{y2}	=	yield acceleration calculated on the basis of residual shear strength parameters (m/s^2)
k_{y2}^{in}	=	k_{y2} needed to initiate upslope sliding (m/s^2)
L	=	length of the block/plane interface (m)
M	=	moment magnitude (dimensionless)
m	=	mass of the block (kg)
N	=	normal force (N)
$PHGA$	=	peak horizontal ground acceleration (m/s^2)
$PVGA$	=	peak vertical ground acceleration (m/s^2)
T	=	interface shear force (N)
t	=	time (s)
T_f	=	shear strength of the interface (N)
W	=	weight of the block (N)
δ	=	displacement (m)
δ_1	=	initial threshold displacement (m)
δ_2	=	second threshold displacement (m)
ϕ	=	interface friction angle (°)
σ_n	=	normal stress (Pa)

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