**Performance based design of retaining structures – Part 1; Theory (Taylor, 2008)**

*Sarma & Yang (1987) attempted to add a theoretical basis to the common practice of applying a factor of 1/2 - 1/3 to Peak Ground Acceleration, by considering the acceleration required to generate 95% of the energy in an earthquake...........a best fit correlation of $A_{95}$ = 0.675 PGA was obtained. In contrast, Japanese practice adopts the proposal by Noda et al (1975) where $k_h$ for PGA > 0.2g is determined as follows:*

$k_h = 1/3\ (PGA)^{1/3}$

and

*To estimate the displacement of a slope failure mass for the case where the ground motion acceleration exceeds the critical or "yield" acceleration $k_c$, an application of Newmark's sliding block model is often used........Typically negligible displacements occur when $k_c$ is exceeded by PGA by up to two times. Only beyond this level do significant displacements typically occur, and for this reason it is common to take a design coefficient of acceleration $k_h$ = 1/2PGA (eg EN 1998).*

**FHWA Circular No 3; Geotechnical EQ Engineering for Highways.**

In the seismic coefficient - factor of safety approach to pseudo-static limit equilibrium analysis, a seismic coefficient is used to represent the effect of the inertia forces imposed by the earthquake upon the potential failure mass. An allowable factor of safety is associated with the seismic coefficient in such a way that the behavior of the slope is within the range considered acceptable, i.e., the slope or embankment will experience acceptable deformation in the design earthquake. The seismic coefficient, $k_s$, is a dimensionless constant. The main drawback of the seismic coefficient - factor of safety approach lies in the difficulty of directly relating the value of the seismic coefficient to the characteristics of the design earthquake. Therefore, a considerable amount of conservatism is usually built into seismic coefficient - factor of safety analyses. Use of either the peak ground acceleration PGA, or the peak average horizontal acceleration of the failure mass, $k_{max}$, as the seismic coefficient (expressed as a function of gravity, i.e., $k_s = k_{max}/g$) in conjunction with a pseudo-static factor of safety of 1.0 has been shown to give excessively conservative assessments of slope performance in earthquakes. However, little guidance on selection of the seismic coefficient as a fraction of the peak acceleration is available to the engineer.

In the seismic coefficient-factor of safety approach to pseudo-static stability analyses, the engineer attempts to select a seismic coefficient and allowable factor of safety such that the cumulative permanent deformation in the design earthquake is small enough to be acceptable. The seismic coefficient is always less than the peak average acceleration of the failure mass and the factor of safety is typically between 1.0 and 1.2. The reason the seismic coefficient is always less than the peak average acceleration is as follows: Earthquakes produce ground motions that in turn induce inertia forces of an alternating nature in slopes or embankments. The alternating inertia forces are of short duration and change direction many times. Therefore, even though the factor of safety during a cycle of earthquake loading may fall below one, it will usually remain below one for only a very brief period of time, until the load reverses. During the interval when the factor of safety is below one, permanent displacement will accumulate. However, only limited displacements will occur during the interval because of its short duration. Therefore, even though the seismic coefficient is less than the peak average acceleration of the failure mass, the cumulative deformation that occurs over the entire earthquake will be small provided the seismic coefficient and factor of safety are selected appropriately.

A major difficulty in the application of the seismic coefficient-factor of safety approach to seismic stability analysis arises from the fact that there are many different views on how to define the seismic coefficient (Seed and Martin, 1966; Seed, 1979; Marcuson, 1981; Hynes and Franklin, 1984). In many building codes, empirical values based on judgement and experience are used ($k_s$ = 0.1 to 0.25 is typical in the United States; $k_s$ = 0.15 to 0.25 is typical in Japan). Seed (1979) reports that clay slopes and embankments with a pseudo-static factor of safety of 1.15 using a seismic coefficient of 0.15 have experienced "acceptable" deformations in earthquakes of magnitude as great as 8.5 subjected to peak acceleration levels as great as 0.75 g. Seed's definition of acceptable deformation appears to include deformations of up to one meter in some cases. Seed (1979) also recommends that, for earthquakes of magnitude 6.5 or less, a seismic coefficient of 0.10 combined with a factor of safety of 1.15 should be used. Seed's definition recognizes the importance of earthquake magnitude in determining the seismic coefficient. Unfortunately, this definition provides no guidance on selection of an appropriate value for $k_s$, for earthquakes with peak acceleration levels less than 0.75 g.

Other investigators have attempted to relate the seismic coefficient to the peak horizontal ground acceleration without considering earthquake magnitude. Figure 51 shows the results of Newmark seismic deformation analyses performed by Hynes and Franklin (1984) using 348 strong motion records (all soil/rock conditions; $4.5 < M_w < 7.4$) and 6 synthetic records. Based upon this data and their experience with seismic response analyses of slopes and embankments, Hynes and Franklin (1984) concluded that slopes and embankments designed with a yield acceleration $k_y$ equal to half the peak ground acceleration $a_{max}$ (i.e., a factor of safety of 1.0 for $k_s = 0.5 \cdot a_{max}/g$) would experience permanent seismic deformations, $u$, of less than one meter in any earthquake, even for embankments where amplification of peak accelerations by a factor of three occurs. In the absence of amplification, or if amplification is taken into account in determining the peak acceleration, the Hynes and Franklin "upper bound" curve presented in figure 51 suggests that deformations will be less than 0.3 m for yield accelerations greater than or equal to one-half the peak acceleration for all cases. Therefore, based upon the work of Hynes and Franklin, it appears that a value of $k_s$ equal to $0.5 \cdot k_{max}/g$ will limit permanent seismic deformations to less than 0.3 m, where $k_{max}$ is peak horizontal average acceleration of the potential failure mass. The value of $k_{max}$ can be estimated using the methods presented in chapter 6 of this document.

**FHWA – Circular No 7, Design of Soil Nail Walls**

*In general, it is acceptable to select a seismic coefficient for soil nail walls between:*

*$k_h = 0.5\ A_m$ to $0.67\ A_m$ (Equation 5.20)*

*This range has provided wall designs that yield tolerable deformations in highway facilities (Kavazanjian et al., 1997). Equation 5.20 is generally conservative and provides slightly larger values of the seismic coefficient than the values resulting from Equations 5.18 and 5.19.*

**Transfund Guidelines for the Design & Construction of GRS Walls**

*"For the analysis of external stability and internal sliding stability of unrestrained GRS structures, the horizontal seismic coefficient is taken as the horizontal peak ground acceleration reduced by 40% as follows; $a_h$ = 0.6xPGA"*

**Eurocode 8: design of structures for earthquake resistance (Draft 2002)**

Section 4.1.2.3 (5)

The design seismic inertia forces on acting on the ground mass for pseudo-static analyses shall be taken as

$F_h = 0.5\ a_{gR}.\gamma_I.S.W/g$

Where:-
$a_{gR}$. = reference peak ground acceleration
$\gamma_I$ = importance factor
S = soil parameter
W = weight of sliding mass

**<u>Geotechnical Earthquake Engineering; Kramer (1996)</u>**

Paraphrase; *Although engineering judgement is required for all cases $k_h$ = 0.5xPGA should be appropriate for most slopes.*