SEISMIC STABILITY MODELING AND ANALYSES OF GEOFOAM EMBANKMENTS

Karthikeyan Radhakrishnan
CH2M HILL, Santa Ana, CA 92707

Dawit Negussey
Syracuse University, Syracuse, NY 13244

ABSTRACT

Use of geofoam for construction of roadway embankments has become more common and is gaining wider acceptance. There is also growing concern in seismically active areas about the stability and serviceability of geofoam embankments under strong motion earthquake shaking. The fundamental frequencies of geofoam and soil embankments were evaluated using finite element models. Different embankment heights and foundation support conditions were simulated. Harmonic excitations of different frequencies were applied at the base of the embankments. Frequencies that resulted in maximum displacements were selected as best estimates of the natural frequency for each embankment height. Actual earthquake records for a soft and also a hard site were applied as base excitations to analyze the local site effects on geofoam and soil embankments. Locations of maximum horizontal displacement and settlement as well as associated magnitudes of permanent deformation were determined for each embankment case. Results of the numerical modeling and simulation are summarized and discussed.

BACKGROUND

Embankments constitute a key element in highways and other transport modes as well as for flood control and protection networks. The stability and operability of embankments must be assured under both static and seismic design conditions. Following the North Ridge earthquake, several key roads in Los Angeles became unusable. The Kobe earthquake in Japan presented similar difficulties to the transport network. For either new embankment construction or expansion of existing roadways, the use of geofoam has been gaining wider acceptance to overcome site specific constraints. But there is a growing concern about the performance and safety of geofoam embankments in seismically active areas. Hotta et al (1996) reported on assessment of damages to EPS geofoam embankments due to earthquakes in Japan. They indicated geofoam embankments remained serviceable and were not damaged by the reported earthquakes. Nomaguchi (1996), summarized observations from model vibration experiments conducted on geofoam blocks. The results of the experiments indicated dynamic friction coefficient of 0.5 between blocks. Kuroda et al (1996), compared results of shake table tests and numerical models. They found distinct element models (DEM) to be better than finite element models (FEM) in simulating dynamic behavior of geofoam.

Earthquake induced slope failures can be due to either inertial forces or foundation weakening (Kramer, 1996). Negussey and Sun (1996) demonstrated that replacement of backfill soil for retaining walls with geofoam offered significant advantages by reducing settlements and lateral loads. The main reason for such benefits was due to the very low, 50 to 100 times less than soil, unit weight of the geofoam. Similarly for seismic response of geofoam embankments, associated inertial forces should be comparably much less than for soil embankments. This proposition is supported by the performance of geofoam filled areas during earthquakes in Japan, as noted above.

Geofoam embankments impose much less stress on foundation soils than earth embankments, and thus would have better stability in cases of subsoil weakening due to earthquake shaking. Hence, conceptually, geofoam embankments should have greater stability against both inertial and weakening instabilities. This hypothesis is investigated in numerical model simulations. Slope stability assessments under earthquakes consider inertial forces and foundation weakening conditions. Pseudo-static limit equilibrium analyses are commonly used to evaluate safety factors against inertial instabilities. Whereas, assessment of foundation weakening related instabilities require consideration of pore pressure generation and soil behavior under cyclic loading. For both approaches, local site characteristics are important in determining the response of an embankment to an earthquake. The amplitude, frequency content and duration of the ground motion can be affected by local site conditions (Kramer, 1996).

In seismic stability analyses by pseudo-static methods, the additional seismic force is approximated by an equivalent static horizontal force applied to the embankment. Geofoam embankments represent top heavy (due to pavement) low mass structures whose response would depend on the frequency of applied loads. In pseudo-static analyses, there are no provisions for representing time dependent effects and displacements; hence the need for finite element or finite difference modeling to investigate the seismic stability and performance of geofoam embankments.

ANALYSES AND RESULTS

The seismic response comparison of geofoam and soil embankments is made by considering trapezoidal configurations with 2 horizontal to 1 vertical side slopes. The top, short width, of the embankments was constant to represent a 50m wide roadway of 1.2m thick pavement structure. Three embankment heights of 6m, 12m or 24m were simulated as resting on either a soil or rock foundation, as shown in Figure 1.

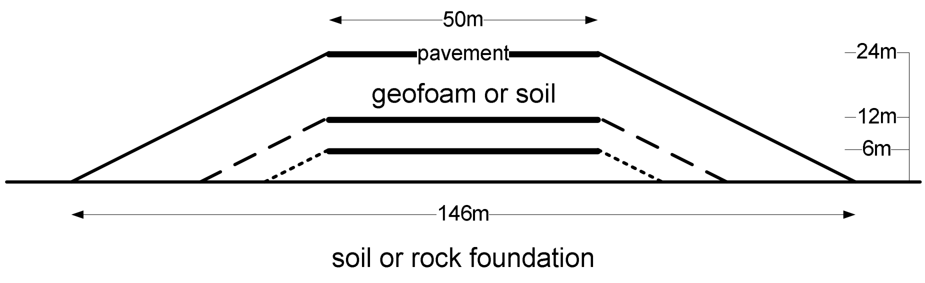

Figure 1: Geofoam or soil embankment sections

The analysis and modeling was based on the following assumptions and simplifications:

1. The soil cover over the geofoam embankment side slopes does not provide lateral support and was not considered.
2. Bonding was assumed between the pavement / geofoam and geofoam / foundation interfaces.
3. Only horizontal ground motion transverse to the embankment alignment was considered.
4. The geofoam embankment was assumed to be monolithic and discrete behavior of geofoam blocks was not simulated.

Evaluation of Fundamental Frequencies using Finite Element Modeling (FEM)

Evaluation of fundamental frequencies is important to better understand the response of embankments under ground motions of varying frequencies. Mode shapes and corresponding frequencies of geofoam embankments of different heights were modeled in ANSYS, a finite element program. The material properties assumed for geofoam, pavement and soil in the analysis are shown in Table 1. The embankment sections, as shown in Figure 1, were simulated as plane strain conditions. The bottom boundary of the embankment, at the interface with the foundation, was restrained against displacement in both horizontal (x) and vertical (y) directions, like a cantilever beam. Harmonic base excitations of 0.5g amplitude and frequencies ranging from 0.1 – 10 Hz were applied. At each level of frequency, displacement responses at different points along the depth of the embankment and pavement were monitored. Fundamental frequencies for each embankment case were determined by modal and harmonic analyses. The frequency that yields the peak displacement response was taken as the fundamental frequency.

Table 1: Material properties for FEM

Material	Depth (m)	Density, ρ (kg/m^3)	Elastic Modulus, E (MPa)	Poisson's Ratio, μ
Soil	6, 12, 24	1800	37	0.30
EPS Geofoam	6, 12, 24	20	8	0.25
Pavement Structure	1.2	2250	70	0.30

Figure 2 compares the fundamental frequencies of soil and geofoam embankments. Geofoam embankments without pavement had higher fundamental frequencies of up to 13Hz. Geofoam embankments with overlying pavement structure had lower fundamental frequencies of around 2.5Hz. The fundamental frequency of the 6m high soil embankment was about 4Hz. Soil embankments also produced secondary responses of significant amplitude at frequencies in the range of 1 - 5Hz. For the soil embankments, ground motions of 1 – 5Hz might match their fundamental frequencies. For geofoam embankments, ground motion frequencies in the 1-3Hz range may be critical for assessing safety and stability. Based on review of ground motion records and local site conditions, expected base excitation frequencies of likely earthquakes can be calculated to compare with fundamental frequencies and identify possible resonance conditions. Fundamental frequencies of embankments vary largely with the height and shape of the embankments. Hence for the same earthquake motion, different kinds of embankments may have different responses and hence different levels of safety.

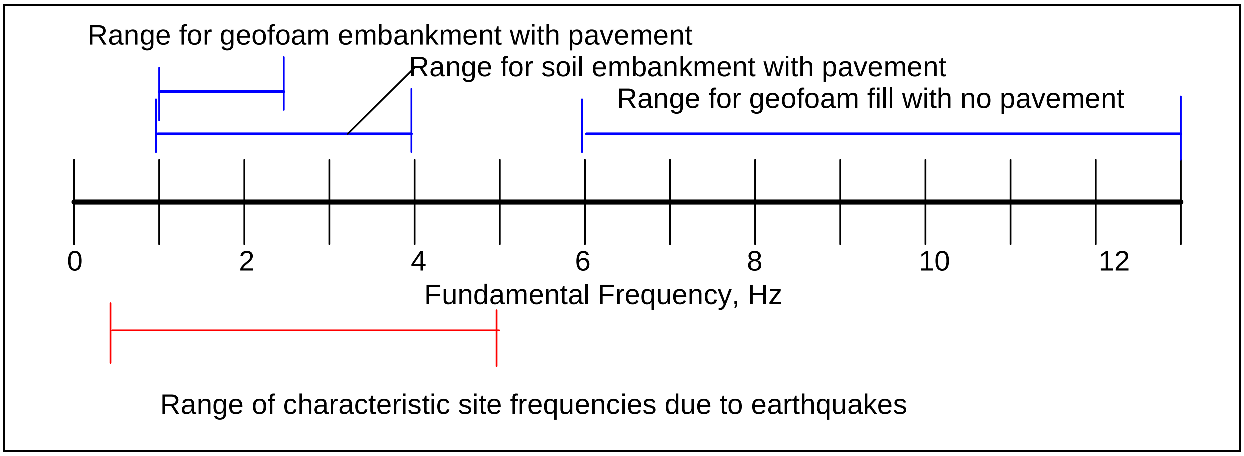

Figure 2: Fundamental frequency ranges for geofoam and soil embankments.

The results obtained from FEM were checked against closed form equations given by Kramer, S.L. (1996) for soil embankments and Horvath, J, S (2004) for geofoam embankments. Table 2 shows the comparison of results of fundamental frequencies from FEM and the referenced equations.

Table 2: Fundamental frequency (Hz.) corresponding to first mode shape

Embankment Height (m)	Geofoam Embankment FEM (Equations)	Soil Embankment FEM (Equations)
6	2.5 (2.0)	4.0 (4.7)
12	1.5 (1.9)	2.5 (2.6)
24	1.5 (1.8)	1.5 (1.4)

Displacement Analyses using Finite Difference Modeling

The magnitude and location of maximum deformations during earthquakes and permanent deformations after earthquake are important for assessing the stability and serviceability of embankments. A finite difference program, Fast Lagrangian Analysis of Continua (FLAC, 2008), was used to perform displacement analyses on soil and geofoam embankments. A finite difference embankment model as shown in Figure 3, was used for soil and geofoam embankments with three different heights (6m, 12m and 24m). Two types of foundation support, soil or bedrock, were used for each embankment case.

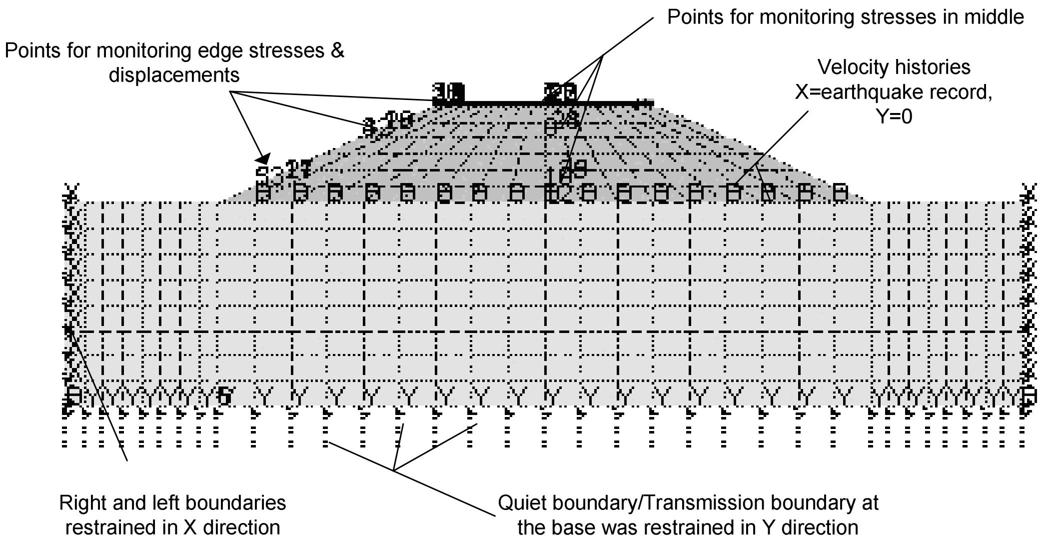

Figure 3: FLAC Embankment Model

The frequency and amplitude of ground motions depend on site conditions underlying soil embankments or the top heavy geofoam embankments. Earthquake records from two sites that have different subsoil conditions were used as input for FLAC model embankments. Both records were generated by the magnitude 7.1 Loma Prieta Earthquake of 1989.

Velocity time histories of Gilroy -1 (G1) recorded on a rock outcrop, and Gilroy-2 (G2) recorded on a deep soft soil deposit (San Francisco Bay mud) but at about the same 100km distance from the Loma Prieta Earthquake epicenter were used. These data were obtained from the Pacific Earthquake Engineering Research Center (PEER, 2005). The predominant frequencies of the G1 and G2 records were 2.6 Hz and 1.9 Hz, respectively. Because the time histories were recorded near the ground surface, velocity time histories were applied at the base of the embankments along the embankment and foundation interface. Free field boundary conditions were applied at both vertical edges of the foundation. A quiet boundary condition was provided along the bottom boundary. The material properties of foundation soils were assigned to correspond roughly to the site conditions of G1 and G2. The foundation was modeled with larger dimensions compared to the embankment to reduce the effects of boundary reflections. Based on resonant column and cyclic uniaxial test results reported by Athanasopoulos et al (1999), a damping ratio of 2% corresponding approximately to 1% cyclic shear strain amplitude was used for the geofoam embankments. A damping ratio of 5% was used for soil embankments.

Modeling Results and Observations

Figure 4 shows the variation of horizontal displacements in different embankments. For a 24m (79 feet) high earth embankment on a deep soil foundation about 22 cm (9 inches) of permanent displacement would be expected for an earthquake comparable to the Loma Prieta Earthquake. Under similar conditions, a 24m high geofoam embankment would experience only about 2.5 cm (1 inch) permanent deformation.

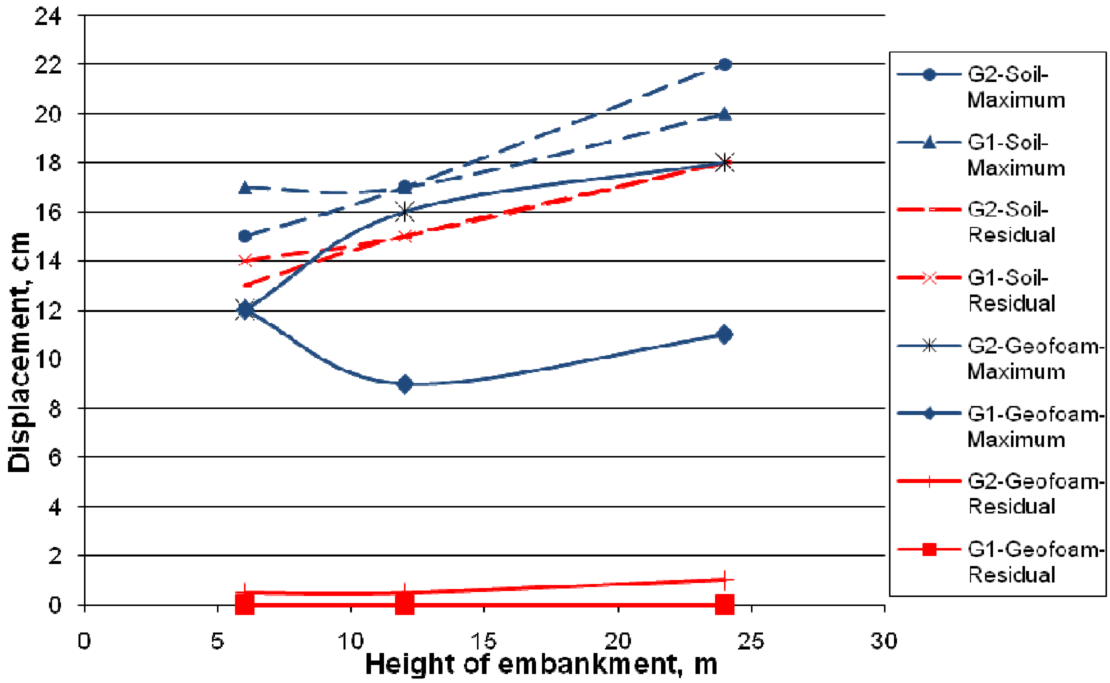

Figure 4: Summary of Maximum and Residual Horizontal Displacements

The location of maximum horizontal displacement was at the base for the soil embankments, and at the top for geofoam embankments, as would be expected for a top heavy structure. The residual displacements after the earthquake motion were small for the geofoam embankments. Horizontal maximum and residual displacements were higher for the soil embankments and occur at the embankment and foundation interface. The displacements of the soil embankment were higher at the soft soil site than at the rock site. Horizontal displacements in both soil and geofoam embankments increased with height.

The settlements for soil embankments were also much higher than for geofoam embankments as shown in Figure 5. A 24m (79 feet) high soil embankment on soft soil subsurface conditions would experience about 18 cm (7 inches) of vertical deformation under the Loma Prieta Earthquake. Under similar conditions, the geofoam embankment would experience settlement of less than 4 cm (2 inches). The settlements of the soil embankments increased significantly with height, while for the geofoam embankments settlement increases with height were very small. These observations indicate geofoam embankments on weak soil foundations would develop only minor settlements and hence, foundation weakening instabilities can be reduced or avoided by using geofoam embankments in place of earth embankments.

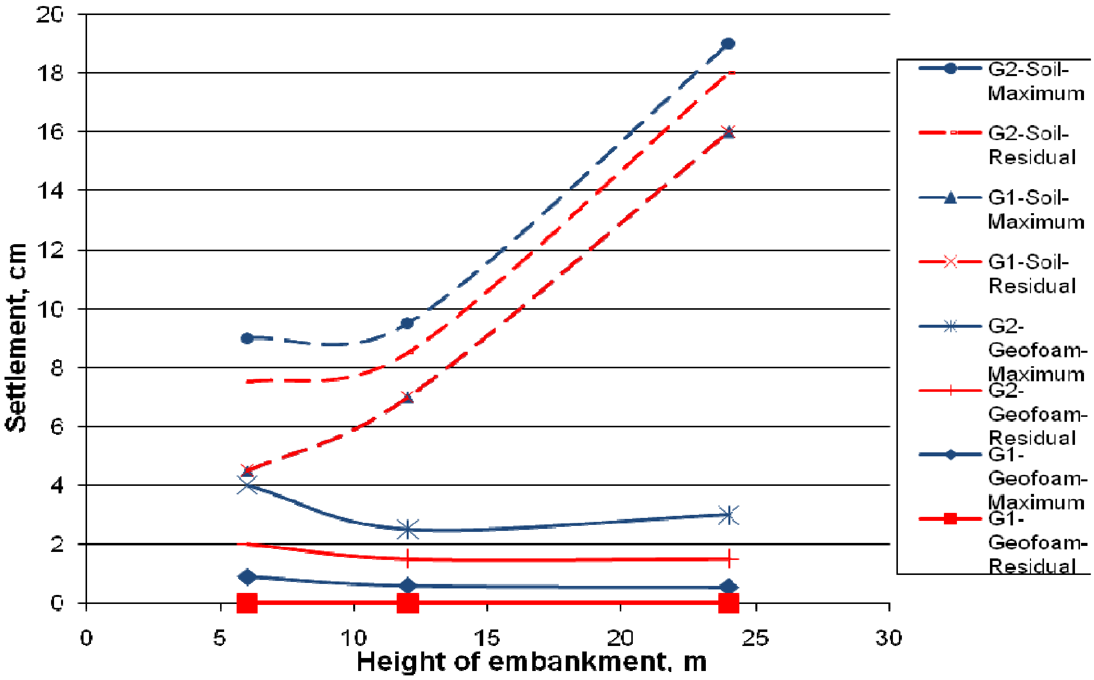

Figure 5: Summary of Maximum and Residual Settlements

The maximum horizontal stresses at the slope boundary and interface between the pavement structure and embankment were monitored. The maximum horizontal stress increases during earthquake motions for soil embankments were 2 times higher than for geofoam embankments of low heights and about 5 times for high embankments. The stresses were higher for earth embankments on soft soil foundation compared to earth embankments on rock foundations. The stresses for geofoam embankments on rock foundations were higher than for cases on soft soil foundations. The site soil conditions affect the response of both soil and geofoam embankments. Horizontal and shear stresses at the center of the embankment were much higher for soil embankments than for geofoam embankments. Geofoam embankments impose much less inertial forces and, hence, reduced stresses due to the very low density of as opposed to soils.

Comparing the results of displacement responses from both the rock and soil foundation sites, the following can be concluded.

1. Maximum and residual horizontal displacements occur at the top for geofoam embankments and may mostly result in repairable pavement cracks or sliding. The location of maximum horizontal residual displacement is at the bottom for soil embankments and may lead to instability

2. Settlements of geofoam embankments were much less than of soil embankments.

3. Residual or permanent displacements were much higher for soil embankments than for geofoam embankments.

4. Peak stresses during the earthquake motion were much less in the geofoam embankments as compared to the soil embankments. The peak stresses were less than the strength of the geofoam.

REMARKS

The inertial force due to the earthquake on geofoam embankment was much less than for earth embankment owing to the very low mass density of the geofoam. Limit equilibrium pseudo-static stability analysis are commonly used to evaluate the seismic stability of earth embankments. Geofoam embankments represent top heavy configurations and their seismic displacement response would not be characterized by limit equilibrium methods. The frequency content of earthquakes, site conditions and dynamic response, and the fundamental frequency of geofoam embankments are important considerations in seismic stability assessments. Fundamental frequencies of geofoam embankments with overlying pavement structures are less than that for soil embankments of the same shape and dimensions. Frequency matching for geofoam embankments with pavement surcharge and earthquakes may occur over a narrow band at the low end of the spectrum. Significant response amplitudes were observed for soil embankments over a broader range of frequencies that match typical earthquakes. The shape, size, and especially height and configuration of materials of embankments can affect the fundamental frequency and response of embankments to earthquakes. The analysis indicated, geofoam embankments developed lower displacements than soil embankments, irrespective of the foundation conditions. For soft soil foundation conditions, geofoam embankments can be constructed steeper sloped or as vertical stable embankments. Even for competent foundation soils, strong earthquakes and large amplitude ground motions; geofoam embankments may be more attractive to reduce permanent displacements.

References

1. Athanasopoulos, G.A., Pelekis, P.C., and Xenaki, V.C. (1999), “Dynamic properties of EPS geofoam: An experimental investigation”, Geosynthetics International, Vol.6, No.3, pp. 171-194.

2. FLAC (2008), Fast Lagrangian Analysis of Continua, User’s manual, Itasca consulting group, Minneapolis.

3. Horvath, J, S (2004), “Geomaterials research project – A technical note on recalculating the fundamental period of an EPS-block-geofoam embankment”, Res. Rpt. No. CGT-2004-1, Manhattan College, Bronx, NY.

4. Hotta, H., Nishi, T., Kuroda, S., (1996), "Report of results of assessments of damage to EPS embankments caused by earthquakes", International symposium on EPS construction method – EPS TOKYO '96, Japan, pp. 307-318.

5. Kramer, S.L.(1996), Geotechnical Earthquake Engineering, Prentice Hall Inc., New Jersey.

6. Kuroda, S., Hotta, H., and Yamazaki, F. (1996), "Simulation of shaking table test for EPS embankment model by distinct element method", International symposium on EPS construction method – EPS TOKYO '96, Japan, pp. 83-92.

7. Negussey, D. and Sun, M.C. (1996), "Reducing lateral pressure by geofoam (EPS) substitution", Proceedings of International Symposium on EPS Construction Method – EPS TOKYO '96, Japan, pp. 202-211.

8. Nomaguchi, A. (1996), "Studies on earthquake resisting performance of EPS embankment", International symposium on EPS construction method – EPS TOKYO '96, Japan, pp. 382-393.

9. Pacific Earthquake Engineering Research Center (PEER, 2005), Loma Prieta Earthquake Record, Strong Ground Motion Database, http://peer.berkeley.edu/