ANALYSIS OF SITE LIQUEFACTION AND LATERAL SPREADING USING CENTRIFUGE TESTING RECORDS

AHMED-W. ELGAMAL, MOURAD ZEGHAL, VICTOR TABOADA and RICARDO DOBRY

ABSTRACT

Dynamically-induced liquefaction of a saturated loose sand stratum is studied in centrifuge model simulations. A laminated container subjected to base excitation is employed to replicate one-dimensional shear beam conditions. Level site and mild infinite slope simulations are conducted at Rensselaer Polytechnic Institute (RPI) and California Institute of Technology (CalTech). The observed acceleration, pore pressure and lateral displacement responses show a high level of consistency, and are used herein to estimate the corresponding dynamic shear stress-strain histories. These histories shed light on the mechanisms of: (1) propagation of liquefaction through the stratum, and (2) soil response during liquefaction, and its effect on ground surface acceleration and lateral deformation.

Key words: centrifuge, dilative behavior, infinite slopes, lateral spreading, liquefaction, sites (IGC: D6/D7/E8)

INTRODUCTION

Recent major seismic events such as the 1989 Loma Prieta and the 1995 Hyogoken-Nambu earthquakes, continue to demonstrate the damaging effects of liquefaction-induced loss of soil strength and associated lateral spreading (Seed et al., 1990; Bardet et al., 1995; Comartin et al., 1995). Experimental laboratory research on soil liquefaction has provided valuable insight concerning excess pore-pressure buildup in saturated loose granular soils (National Research Council 1985). However, for engineering applications, there remains a need to further understand and identify the mechanisms of seismically induced soil deformation due to liquefaction, and associated stiffness and strength degradation.

In-situ seismic response records of site liquefaction are scarce. The Wildlife Refuge site seismic records (Imperial County, CA) during the 1987 Superstition Hills earthquake are currently the only such data set (surface and downhole accelerations; and pore-pressures) in the United States, and possibly worldwide (Holzer et al., 1989).

Centrifuge model tests provide an alternative source of information about the mechanisms of liquefaction and lateral spreading. In this regard, the VELACS (Verification of Liquefaction Analysis by Centrifuge Studies) project is unique (Arulananand and Scott, 1993, 1994). Within this project, tests were conducted on ten saturated soil (and soil-structure) centrifuge models with well defined boundary conditions, soil properties, and input dynamic motions. The recorded experimental data included accelerations, displacements, settlements and excess pore pressure (EPP) histories.

In this paper, a simple identification procedure was employed to investigate the recorded dynamic response of VELACS centrifuge models 1 and 2 (Fig. 1). These two models were proposed (Dobry and Taboada, 1994) to

![Diagram of models 1 and 2 in laminated container]

Fig. 1. Configuration of models 1 and 2 in laminated container

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simulate the dynamic response of level and mildly sloping loose sand sites respectively. Herein, the involved shear stress and shear strain histories are evaluated directly from the recorded accelerations, and displacements along the stratum height (Taboada and Doby, 1993a, b; Scott et al., 1993). These histories are then utilized to assess the involved soil response mechanisms including lateral spreading.

In the following sections, a brief description of the analyzed centrifuge model results is given. Thereafter, the employed simple identification procedure is described, and the involved stress-strain histories are presented and examined in detail. This identification procedure, originally proposed in basic form for shake-table studies (Koga and Matsuo, 1990), was further developed and used earlier for analyses of downhole site response (Zeghal and Elgamal, 1993, 1994; Zeghal et al., 1995).

SOIL PROPERTIES

Nevada sand was used at a relative density in the range 40% – 45% (mass density = 1973 kg/m$^3$), with a permeability coefficient $k = 6.6 \times 10^{-5}$ m/sec (Arulmoji et al., 1992). The general characteristics of cyclic soil response may be inferred from triaxial and torsional laboratory tests on this soil (Arulmoji et al., 1992). In an undrained cyclic triaxial test with an imposed static shear stress, the soil is seen (Fig. 2) to retain much of its stiffness and strength during cyclic loading until $r_u$ approaches unity corresponding to initial liquefaction ($r_u = p / \sigma_v$ is excess pore pressure ratio, where $p$ is excess pore water pressure and $\sigma_v$ is initial effective vertical stress). Thereafter, the soil gradually accumulates significant permanent strains for each additional cycle of loading. In this process, stress spikes appear associated with tendency of the soil skeleton to dilate at large strains. This pattern of response will be shown later to agree with shear stress-strain histories obtained from the analyzed centrifuge-testing acceleration records.

TESTING PROCEDURE

At Rensselaer Polytechnic Institute (RPI), a 3.0 m radius Acutronic model 665-1 centrifuge was employed (Elgamal et al., 1991). A Shaevitz centrifuge, 1.0 m in radius, was employed at California Institute of Technology (CalTech). Tests were performed in one dimensional (1D) laminated containers (Hushmand et al., 1988; Van Laak et al., 1994a). These containers (Fig. 1) allow relative slip between laminates in order to simulate 1D shear response. Instrumentation included pore pressure transducers, accelerometers, and LVDTs (Linear Variable Differential Transducers) for measurement of displacements.

The soil models were built at 1 g, with transducers installed at the specified locations; and saturated with water under vacuum (Taboada and Doby, 1993a, b; Scott et al., 1993). Thereafter, the model containers were placed on the centrifuge, and spun to a 50 g gravitational field. At this gravitational field, the models depicted in Fig. 1 represent a prototype 10 m thick sand layer of infinite lateral extent; with model 1 being horizontal and model 2 inclined at a slope of 2°. In view of the scaling laws applicable to centrifuge experiments (Tan and Scott, 1985), the prototype permeability is 50 × $k$, and corresponds to that of coarse sand or fine gravel.

One-dimensional (1D) shaking was finally imparted along the model base using electro-hydraulic shakers (Van Laak et al., 1994a; Hushmand et al., 1988). Acceleration, pore pressure, and displacement histories over the model height were recorded by the installed sensors (Fig. 1). The results of these tests have been thoroughly documented in Taboada and Doby (1993a, b) and Scott et al. (1993); and further discussed by Doby and Taboada (1994). In the following sections, prototype units (Tan and Scott, 1985) will be employed when discussing these results.

EVALUATION OF SHEAR STRESS-STRAIN HISTORIES

Abdel-Ghaffar and Scott (1978, 1979) conducted a pioneering study to identify the dynamic soil properties of an earth dam using recorded earthquake accelerations. A shear wedge model was used to describe the embankment seismic response. In VELACS models 1 and 2, lateral response (relative to the base) may be idealized using a
one-dimensional shear beam (Fig. 3):

\[ \frac{\partial \tau}{\partial z} = \rho \ddot{u}, \quad \text{with the boundary conditions} \]

\[ u(h, t) = u_2, \quad \tau(0, t) = 0 \quad (1) \]

where \( t \) is time, \( z \) is depth coordinate, \( \tau = \tau(z, t) \) is horizontal shear stress, \( \ddot{u} = \ddot{u}(z, t) \) is absolute horizontal acceleration, \( u = u(z, t) \) is absolute horizontal displacement, \( u_0 = u(t) \) is input horizontal displacement, \( \rho \) is mass density, and \( h \) is soil stratum depth.

Integrating the equation of motion and using the surface stress-free boundary condition (Eq. (1)), shear stress at any level \( z \) may be expressed as:

\[ \tau(z, t) = \int_0^z \rho \ddot{u} \, dz \quad (2) \]

Employing linear interpolation between downhole accelerations, the discrete counterpart to stress at level \( z_i \) (Eq. (2)) reduces to:

\[ \tau_i(t) = \tau_{i-1}(t) + \rho \frac{u_{i-1} + u_i}{2} \Delta z_{i-1}, \quad i = 2, 3, \ldots \quad (3) \]

in which subscript \( i \) refers to level \( z_i \), \( \tau_i = \tau(z_i, t) \), \( \tau_{i-1} = \tau(0, t) = 0 \), \( u_{i-1} = \ddot{u}(z_{i-1}) \), and \( \Delta z_i = \) spacing interval as shown in Fig. 3. Midway between levels \( z_{i-1} \) and \( z_i \), shear stress may be expressed as:

\[ \tau_{i-1/2}(t) = \tau_{i-1}(t) + \rho \frac{3u_{i-1} + u_i}{8} \Delta z_{i-1}, \quad i = 2, 3, \ldots \quad (4) \]

in which \( \tau_{i-1/2} \) is shear stress at level \( (z_{i-1} + z_i)/2 \). These stress estimates (Eqs. (3) and (4)) are second order accurate (Zeghal and Elgamal, 1993). A corresponding second-order accurate shear strain \( \gamma_i \) at levels \( z_i \) and \( (z_{i-1} + z_i)/2 \) may be expressed as (Pearson, 1986):

\[ \gamma_i(t) = \frac{1}{\Delta z_{i-1} + \Delta z_i} \left( u_{i+1} - u_{i-1} \frac{\Delta z_{i-1}}{\Delta z_i} + (u_i - u_{i-1}) \frac{\Delta z_i}{\Delta z_{i-1}} \right), \quad i = 2, 3, \ldots \quad (5) \]

\[ \gamma_{i-1/2}(t) = \frac{u_{i-1} - u_{i-1}}{\Delta z_{i-1}}, \quad i = 2, 3, \ldots \quad (6) \]

in which \( u_i = u(z_i, t) \) is either displacement furnished by LVDT recordings, or absolute displacement evaluated through double integration of the corresponding acceleration histories. A strain history that combines accelerometer and LVDT recordings was used herein in order to include permanent drift in model 2 (Appendix 1).

The stress histories of Eqs. (3) and (4) were adjusted to include additional stresses imposed by the employed laminated containers and testing configuration. Appendix 1 discusses the employed stress and strain histories, and the various approximations associated with conducting the above analysis procedures.

**LEVEL SITE RESPONSE: MODEL 1**

This test (Fig. 1) was proposed and tested by RPI (Taboada and Doby, 1993a), and also performed by the University of California, Davis (Arunalanand and Zeng, 1993a), and the University of Colorado at Boulder (Stadler et al., 1993). In this section, the RPI test results are analyzed.

Figures 4 and 5 display the recorded lateral accelerations, and pore-pressures respectively. The input acceleration time history consisted primarily of superposed 2 Hz and 6 Hz signals (AH1, Fig. 4), and lasted for nearly 11 s (from 1 s to 12 s) with a peak value of about 200 cm/s². At the free surface, accelerations virtually disappeared after about 3 s of shaking due to liquefaction (AH3, Fig. 4). At 2.5 m depth, a similar mechanism followed at about 5 s (AH4, Fig. 4). Excess pore-pressures were found to reach \( r_u = 1.0 \) down to a depth of 5 m below ground surface. However, at this 5 m depth, accelerations did not vanish after liquefaction. Instead, a number of spikes were observed after about 8 s of shaking (AH5, Fig. 4).

**Stress and Strain Histories**

Figures 6 and 7 display model 1 shear strain and stress histories at 1.25 m, 2.5 m, 3.75 m, 5.0 m, and 7.5 m depths (evaluated from the recorded accelerations). It might be postulated that a liquefaction front (\( r_u = 1.0 \)) propagated from the top to a depth of 3.0 m approximately. This is inferred from: (1) the excess pore-pressure histories (Fig. 5) which reached \( r_u = 1.0 \) first at the range 1.25 m–2.5 m depth, followed by the 5 m depth, and (2) the corresponding acceleration and stress histories where minimal magnitudes were observed as \( r_u \) approached 1.0 (Figs. 4 and 7).

Representative shear stress-strain cycles at the different elevations during selected time windows are shown in Fig. 8. This time-window display (Fig. 8), clearly shows how soil stiffness and strength degradation, as well as the development of larger strains, propagate from the top downwards (down to a depth of about 5 m). At 5 m depth, appreciable soil strength remained despite \( r_u = 1.0 \) at this elevation, leading to a form of cyclic mobility after 8 s (Castro, 1975). The resulting stress-strain response (Fig. 8) remained stiff, and included the additional 6.0 Hz input excitation, which appeared as a superposed high frequency oscillation.

At 7.5 m depth, High EPPs after 8.0 s were accompanied by a number of acceleration and stress spikes (Figs.
Fig. 4. Model 1 acceleration histories at free surface (AH3), 2.5 m depth (AH4), and 5 m depth (AH5), and the associated input acceleration (AH1).

Fig. 5. Excess pore pressure histories at 1.25 m (P5), 2.5 m depth (P6), 5 m depth (P7), and 7.5 m depth (P8).

Fig. 6. Model 1 shear strain histories at 1.25 m, 2.5 m, 3.75 m, 5 m, and 7.5 m depth (evaluated through double integration of the acceleration histories).

Fig. 7. Model 1 shear stress histories at 1.25 m, 2.5 m, 3.75 m, 5 m, and 7.5 m depth.
4 and 7) that might be indicative of mild dilative soil response as \( r_e \) approached 1.0 (see Fig. 2). However, at this elevation no evidence of appreciable strength or stiffness degradation was observed (Fig. 8).

Large cyclic strains (of about 0.75%) associated with liquefaction (Figs. 6 and 8) appeared first at 1.25 m (2–3.5 s), and thereafter moved downwards to 3.75 m; whereupon the 1.25 m depth became isolated from base shaking (see also Fig. 4). At 7.5 m depth (Fig. 6), smaller cyclic strains of about 0.2% prevailed. Thus, significant generation of EPPs at this elevation would not be expected. Indeed, the P8 EPP record (at 7.5 m depth) appeared (Fig. 5) to achieve pore-pressures that were only slightly higher than those of P7 above, thus achieving an \( r_e \) value of only 0.8 at this elevation.

**INFINITE SLOPE: MODEL 2**

This test (Fig. 1) was proposed and conducted by RPI (Taboada and Doby, 1993b), and also performed by the University of California Davis (Arunanandan and Zeng, 1993b), and CalTech (Scott et al., 1993). No lateral displacements were measured in the Davis test. Hence, in this study, the RPI and CalTech test results were analyzed.

In view of the model inclination (\( \alpha = 2^\circ \)), shear stress-strain histories of model 2 were influenced by the presence of a driving static shear stress component induced by gravity. This static stress may be expressed as \( \tau_s = \rho g z \sin(\alpha) \), and acts along with an additional static
stress state dictated by the employed testing scheme (Appendix 1).

The recorded EPPs in these tests were very similar to those of model 1 (Fig. 5). However, unlike model 1, the acceleration records showed a directional bias (Fig. 9), with no significant reduction in acceleration amplitudes due to liquefaction. In this regard, a number of large acceleration spikes occurred exclusively in the positive upslope direction (Fig. 9); and were accompanied by large negative downslope displacements (Figs. 10 and 11). The comparison of RPI and CalTech results as depicted in Figs. 9 through 11, shows a high level of agreement.

The corresponding identified stresses and strains are shown in Figs. 12 through 15. In general, the relatively minor differences in response between RPI and CalTech might be attributed mainly to the difference in input motion (Fig. 9). Nevertheless, as noted for the accelerations, lateral displacements and EPPs, a high level of agreement is also seen in the stress-strain response values and patterns.

In Fig. 9, the observed bias in acceleration magnitudes was found to be a consequence of downslope displacement; and was a reflection of the involved high stress peaks (Fig. 13) at large strains (Figs. 14 and 15). The resulting permanent strains (Figs. 12, 14 and 15) occurred primarily within the zone bound by depths 2.5 m–5.0 m.

The magnitude of initial yield shear stress $S_y$ is seen to increase with depth (Figs. 14 and 15), from $S_y = 2–3$ kPa at 1.25 m to 15–17 kPa at 7.5 m depth. Again, the agree-

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**Fig. 11. Lateral displacement profiles at various times during shaking for RPI and CalTech model 2 (Dobry and Taborda, 1994)**

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**Fig. 10. RPI and CalTech Model 2 displacement histories at free surface (LVDT3), 2.5 m depth (LVDT4), 5 m depth (LVDT5), and 7.5 m depth (LVDT6)**

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**Fig. 12. RPI and CalTech Model 2 shear strain histories at 1.25 m, 2.5 m, 3.75 m, 5 m, and 7.5 m depth**
Fig. 13. RPI and CalTech Model 2 shear stress histories at 1.25 m, 2.5 m, 3.75 m, 5 m, and 7.5 m depth

Fig. 14. RPI Model 2 shear stress-strain histories with superposed static stress due to inclination

Fig. 15. CalTech Model 2 shear stress-strain histories with superposed static stress due to inclination
ment between RPI and CalTech results was quite satisfactory in this regard. In the stress-strain histories, the following additional response mechanisms may be noted:

1. At the end of shaking, large localized permanent strains (7% CalTech, 12% RPI) had accumulated at 3.75 m depth. The average increment of accumulated strain per loading cycle was in the range 0.7%–1.2%. In general, accumulation of permanent strains was more evenly distributed along the height in the CalTech test, as compared to the more localized pattern observed at RPI. These patterns of deformation might be somewhat influenced by the employed laminated container configurations (a larger number of laminates constituted the RPI box).

2. Excursions of dilative soil behavior at large cyclic strains (in the downslope direction) were observed in the form of hardening shear stress spikes. These dilative excursions (at about 8 s and 10 s) were somewhat less pronounced at CalTech (Figs. 14 and 15), and the net accumulated strain increment per cycle was accordingly lower (about 0.5% at CalTech vs. 1.0% at RPI). The hardening dilative stress phase explains the fact that no sustained loss of soil stiffness was inferred from the acceleration records (i.e., no isolation of accelerations appeared in Fig. 9 of model 2, in contrast to Fig. 4 of model 1).

In general, the soil behavior identified from the recorded dynamic response (Figs. 12 through 15) appears to agree in character with the laboratory stress-strain relations of Arulmoli et al. 1992 (Fig. 2).

RELEVANCE TO CASE HISTORIES

The acceleration responses of models 1 and 2 may be viewed as a reflection of the corresponding soil behavior during liquefaction in terms of: (1) isolation of upper layers from further base accelerations at small cyclic strains, for a level site (model 1), and (2) persistence and transmission of surface accelerations during liquefaction due to the dilative soil response at large strains in mildly sloping sites (model 2). Aside from minor fluctuations due to this dilative response, the pore pressures of models 1 and 2 are otherwise very similar (Fig. 5).

The above identified response mechanisms are generally in close agreement with those observed at:

1. The 1964 Niigata (Ishihara, 1985) and the 1989 Treasure Island (Hryciw et al., 1991; Finn et al., 1993) liquefaction case histories. As observed in model 1, after liquefaction the ground surface was isolated from further strong base excitation.

2. The Wildlife refuge site, CA during the 1987 Superstition Hills earthquake (Elgamal and Zeghal, 1992; Zeghal and Elgamal, 1994). As noted in model 2, the soil response during liquefaction was clearly associated with acceleration and stress spikes at large shear strains due to soil dilation.

CONCLUSIONS

The recorded accelerations and displacements of two VELACS centrifuge tests were analyzed. A simple identification technique was employed to estimate shear stress and strain histories at different depths within the soil stratum. These histories were employed to assess the mechanisms of soil liquefaction and lateral spreading in level and mildly inclined soil layers of infinite horizontal extent. It was observed that:

1. The presented results of models 1 and 2 showed remarkable consistency. These models were tested by two different research groups, using different laminated containers, shaking tables, and centrifuge machines. The agreement attests to the overall reliability of the basic testing technique.

2. Minor variability in employed input motions at RPI and CalTech (model 2) had a correspondingly small effect on the overall observed response.

3. Level and mildly sloping models had similar pore pressure buildup patterns.

4. The upper layers of a liquefied stratum \( r_e = 1 \) become isolated from further seismic excitation, in the absence of large shear strains that induce a tendency for soil dilation. This dilation during large cyclic strain excursions results in an instantaneous reduction in pore pressures, with an associated regain in shearing resistance. Such dilative response is manifested in the form of recorded acceleration spikes (model 2).

5. During each seismically-induced shear stress cycle after liquefaction, increments of permanent lateral ground deformation may accumulate in the direction of an existing slope. These permanent deformation increments (per cycle) may be highly constrained by the soil large-strain dilative response characteristics. Such a lateral spreading mechanism agrees well with that observed in undrained cyclic-loading laboratory tests, with a locked-in driving shear stress.

6. Centrifuge experiments such as those of models 1 and 2 may serve as calibration tests along with other conventional cyclic laboratory tests. Using the centrifuge testing records, stress, strain and excess pore-pressure histories may be obtained at different elevations within the liquefied soil stratum.

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REFERENCES

Liquefaction and Lateral Spreading

78-02, Earthquake Engineering Laboratory, Pasadena, California.

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APPENDIX I: ANALYSIS APPROXIMATIONS

The approximations involved in the above stress-strain evaluation process stem from three different sources:

- Instrument Configuration: Accuracy of shear stress and strain estimates (Eqs. (3) and (6)) is a function of: (1) instrument spacing, and (2) recorded acceleration wave-lengths ($\lambda = \upsilon_r / f$, where $\upsilon_r$ is shear wave velocity and $f$ is frequency). Dynamic energy of models 1 and 2 response was mainly within the 1.5–2.5 Hz frequency range, and acceleration wave-lengths were about 40–50 m. Within these ranges, approximation errors in shear stress and strain estimates were evaluated to be generally less than 6.0%. A detailed analysis of these approximations may be found in Zeghal and Elgamal (1995).

- Data Processing: In view of instrument and digitization inaccuracies, shear strain histories (evaluated using integrated accelerations) include baseline drifts in the form of spurious very low frequency components. These drifts and minor high frequency components (Eqs. (5) and (6)) were eliminated using low- and high-pass filters (Oppenheim and Shafer, 1989). Zero-phase time domain FIR (finite duration impulse response) filters with the characteristics mentioned in Table 1 were utilized. This filtering procedure introduces no phase shifts. As shown in Table 1 filter bandwidths were selected to be wide enough so as to conserve the shear stress and strain characteristics.

- Analysis Technique: In view of the relatively close spacing between instruments, and in order to maintain simplicity, first order linear interpolation between accelerations was employed to estimate stresses (Eqs. (3)–(4)); and second order interpolation between displacements was used to evaluate strains (Eqs. (5)–(6)). Both interpolation schemes yield second-order accurate shear stress and strain estimates (Zeghal and Elgamal, 1993).

- Estimation of Permanent Strains: When stresses were plotted against LVDT-derived strains, a time lag was observed in the resulting cyclic stress-strain components (Fig. 16). This lag resulted in a negative stress-strain slope (Fig. 16(d)) and might be attributed to: (1) difference (or lag) in response between central accelerometer and boundary LVDT locations (Fig. 1), (2) at high $\gamma_p$ values, partial dissociation of accelerometer from surrounding soil particles due to difference in inertial properties. In order to remedy this problem, shear strain histories that included permanent deformation were evaluated in the following fashion (Figs. 16(a)–(c)): (1) accelerometer derived strains $\gamma_p(t)$ were filtered to eliminate superfluous drift (Trifunac and Lee, 1973), (2) LVDT permanent drift histories $\gamma_d(t)$ were obtained by filtering out all cyclic strain components, and (3) cyclic strains $\gamma_c(t)$ derived from accelerations were superposed on the corresponding drift strains $\gamma_d(t)$ such that:

$$\gamma(t) = \gamma_c(t) + \gamma_d(t)$$  \hspace{1cm} (7)

A sample of the resulting stress-strain histories is shown in Fig. 16(e).

![Fig. 16. Sample shear stress and strain histories: (a) strain derived from accelerations; (b) strain derived from LVDTs; (c) employed strain; (d) stress vs. LVDT-derived strain and (e) stress vs. employed strain](image)

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<th>Table 1. Characteristics of employed filters</th>
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<td>Acceleration filter</td>
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<td>LVDT filter for $\gamma_d$</td>
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• Testing Scheme: Taboada and Dobry (1993a, b) and Dobry and Taboada (1994) reported the following effects of dynamic testing and model inclination:

—Additional shear stresses were imposed during shaking due to own weight of the laminates. According to the adopted shear beam approximations (Eq. (1)), the dynamic stresses (Eqs. (3) and (4)) are increased by a factor of 22% and 16% for RPI and CalTech laminated containers, respectively. In view of the 2° inclination of model 2, own weight of laminates also increased the initial static shear stress by 45% and 33% for RPI and CalTech laminated containers, respectively.

—In model 2, the imposed 2° inclination applies only to the saturated soil, while the water table remained level. This condition increased the initial static shear stress by 70% and 77% for RPI and CalTech laminated containers, respectively.

All stress histories reported in this paper have been adjusted to account for the above mentioned effects.