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FINITE ELEMENT ANALYSIS OF REINFORCED SOIL RETAINING WALLS SUBJECTED TO SEISMIC LOADING

Adel M BELAL[1] And K. P GEORGE[2]

SUMMARY

A typical geogrid reinforced soil retaining wall constructed with and without facing units was analyzed for seismic response. The walls are proportioned using the Pseudo-Static design method. A finite element method—ABAQUS-code—was employed using Drucker-Prager model to characterize sand and nonlinear elastic reinforcement material. This paper presents the wall responses to a typical seismic spectrum. Of particular interest in this study are: (1) the acceleration response, (2) the wall displacement, (3) the tensile stress in the reinforcement, and (4) the slippage at the soil-reinforcement interface. Probable failure modes were also sought in this study. Specifically, three possible failure mechanisms were investigated, namely, wall displacement, tensile stress in reinforcement, and slippage between soil and reinforcement. Having designed for peak acceleration of 0.25g in conjunction with a factor of safety of two, the walls withstood a base excitation of 0.5g ground motion.. While imposing surcharge loads of different magnitudes, however, those responses begin to accumulate over the duration of the simulated seismic event, indicating imminent failure in one mode or another. Slippage at the interface seems to the probable failure mode of the wall without facing whereas the wall with facing would fail by breakage of the reinforcement.

INTRODUCTION

Analysis and design of geosynthetic reinforced soil under static conditions has been recently introduced by the National Concrete Masonry Association (NCMA) in North America [Simac, et al., 1993]. Though lateral displacement is arguably the most important performance feature of these structures under seismic loading, the NCMA method embodies a pseudo-static limit equilibrium method.

Many numerical analyses, laboratory modeling, and full scale field tests have been performed to better understand mechanisms, behavior and failure modes of reinforced soil walls. The first known investigation into the behavior of reinforced earth walls under dynamic load was carried out by Richardson and Lee [1975] using a shaking table, providing preliminary data for the development of a semi-empirical design method. Subsequent shaking table studies were conducted by various researchers [Wolfe, et al., 1978; Rea and Wolfe, 1980; Sommers and Wolfe, 1984]. Fairless [1989] tested six one-meter tall reinforced earth wall models under normal gravity on a shaking table. He concluded that seismic shaking and permanent displacement of reinforced walls cause quite dramatic increases of the forces in the reinforcing strip; theorizing that the reinforced wall would not collapse if the reinforcing strip did not break. The outward displacement at failure was about 4% of the wall height. Full scale tests were conducted refining the results of model studies [Richardson, et al., 1975; Reid, 1995]. Recently, a few finite element model studies have been reported [Bachus, et al., 1993; Cai and Bathurst, 1995].

Employing a finite element model, the seismic response of a geosynthetic reinforced soil retaining wall system with and without facing units is investigated. Highlighted in this study are: (1) the acceleration response in the wall, (2) the wall displacement, (3) the tensile stress developed in the reinforcement, and (4) the relative

[1] *Egyptian Military, Cairo, Egypt; e-mail: abelal@hotmail.com*
[2] *Professor, The University of Mississippi, USA; e-mail: cvkpg@cotton.vislab.olemiss.edu*

displacement between soil and the reinforcement. Three probable failure modes, namely, wall displacement, breakage of reinforcement, and slippage between soil and the reinforcement are also investigated.

FINITE ELEMENT IMPLEMENTATION

The finite element package ABAQUS Explicit, version 5.6 [ABAQUS Manual, 1990] was used to perform two dimensional, nonlinear finite element analyses. The model of the wall without facing units, includes 744 elements and 930 nodes, and the wall with facing units 775 elements and 984 nodes (Figure 1). The wall designed in accordance with Pseudo-Static design procedure is 325 cm high and comprises 16 concrete masonry units connected together by a cementing material, a uniform granular backfill, and five layers of HDPE geogrid reinforcement extending 244 cm into the backfill soil. The modeled width of the backfill soil extends a distance of 244 cm beyond the back face of the wall. The elements are discretized into 4-node quadrilateral elements.

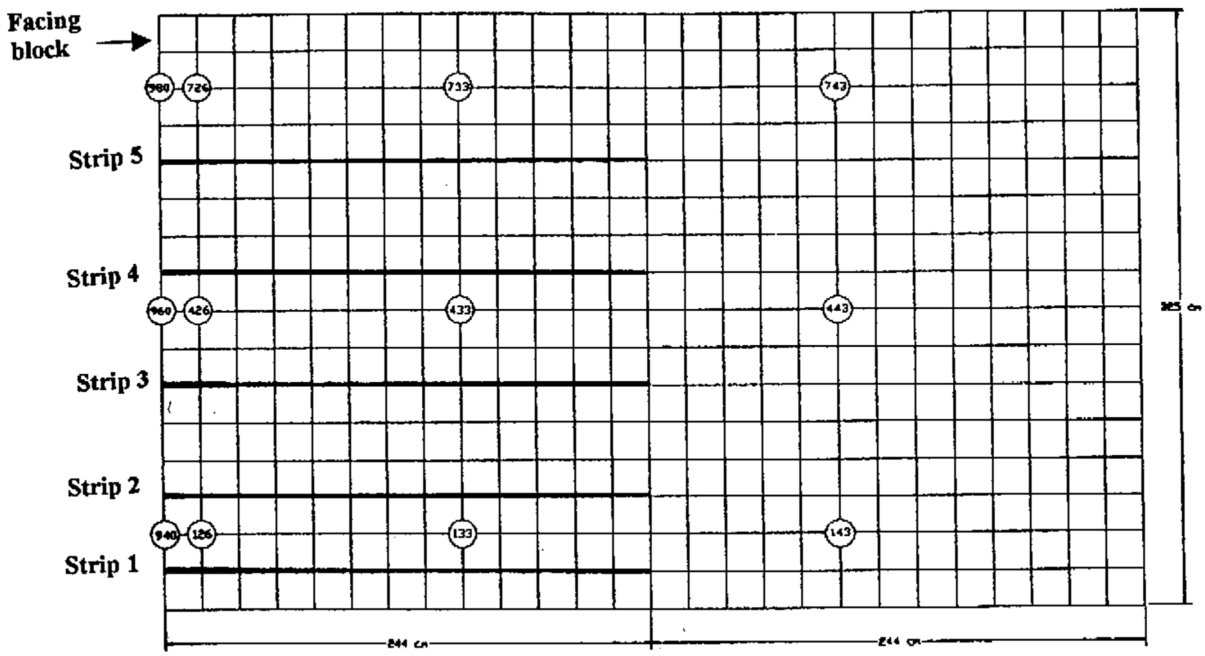

Figure 1. Finite element mesh of reinforced earth wall. Numbers in circle refer to nodes.

Material Models and Properties

The soil is characterized employing the Drucker-Prager model [Drucker, et al., 1957]. Soil properties selected include: Young's modulus = 82680 kPa; Poisson's ratio = 0.3; soil cohesion = 0.0; unit weight = 1628 kg/m^3 ; angle of internal friction in plane strain = 42.5°; angle of internal friction in direct shear = 37.5°; soil dilation angle = 10° [Jewell and Milligan, 1989]. Other material parameters for Drucker-Prager model are derived from soil cohesion, angle of internal friction and dilation angle [ABAQUS Manual, 1990].

Tensar geogrid SR2 used in the wall exhibits slightly nonlinear stress-strain properties with a breaking load of 63 kN/m [Netlon Ltd, 1984]. The properties of the modular facing blocks are assigned the following values, [Cai and Bathrust, 1995]: Young's modulus = 20,685,000 kPa; Poisson's ratio = 0.2, unit weight = 2170 kg/m^3. The interface between soil and reinforcement, and that between soil and concrete blocks are modeled by the contact pair option in ABAQUS. This option allows sticking, sliding or separation to occur between the contact elements, obeying the Mohr-Coulomb criterion.

Loading and Boundary Conditions

Two cases were studied, where the peak horizontal acceleration is 0.25g or 0.5g. Each acceleration history was applied for 20 seconds for want of more computer time. Finite element analyses were carried out for sand reinforced by geotextile material. Peak horizontal accelerations and the model loading conditions were changed to investigate various responses. In order to investigate near-failure behavior of the wall, surcharge loads of 21, 34 and55 kPa, respectively, in conjunction with peak acceleration of 0.25g were applied.

Each analysis begins with initializing of gravity stresses by inducing an acceleration of 981 cm/sec^2 in the positive vertical direction over 1 sec time period which remains over the entire duration of the analysis. The acceleration-time history employed is the horizontal component of the Northridge earthquake of 1994, with a peak horizontal acceleration of 0.25g, as shown in Figure 2. By scaling the peak horizontal acceleration, another input acceleration time history of 0.5g was obtained.

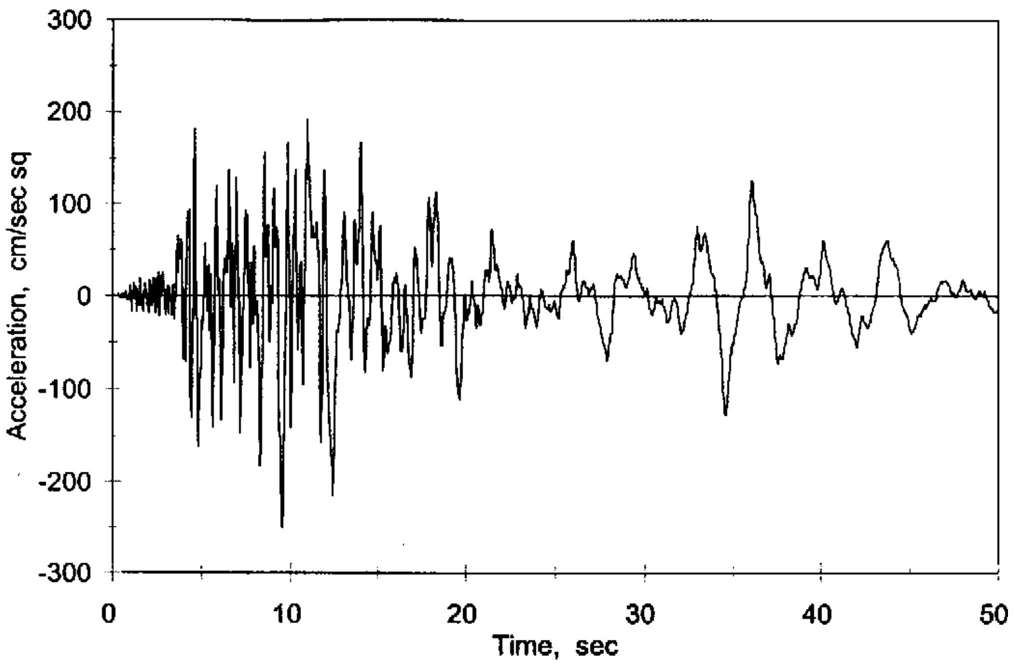

Figure 2. Time history of reference input acceleration.

The boundary conditions simulated in the problem follow: Only vertical displacement is permitted along the rear wall face. The gravity is applied at the bottom in the vertical direction whereas the seismic loading is simulated by applying the acceleration-time history of Figure 2 at the bottom and rear face of the retaining wall.

Failure Criteria

Threshold values for the three failure modes, namely, wall displacement, tensile stress in reinforcement, and relative displacement between soil and reinforcement are selected based on the results reported by various researchers. Fairless [1989], in his seismic testing of reinforced earth walls, reported that the outward displacement at failure is about 4% of the wall height. Based on full scale model studies, Bathurst and Benjamin [1990] proposed a rather conservative displacement criterion of 2%, which is adopted here. Using a 2% criterion, the 325 cm wall can undergo an outward displacement of 6.5 cm. Reinforcement material, with an allowable stress of 20,700 kPa, is used in this study [Netlon Ltd.1984]. Based on Ingold's [1983] pull out test of grid reinforcement in sand, aided by practical considerations, an allowable slippage value of 0.6 cm is chosen.

Wall subjected to horizontal acceleration

Distribution of horizontal acceleration

Comparing the response spectra from bottom to top (for example nodes 133, 433 and 733, Figure 1), the peak responses at node 433 and 733 are magnified by 60% and 85 % with respect to that at node 133. Identical results are obtained with 0.5g peak horizontal acceleration as well. Another observation is that not only is the peak acceleration enhanced from bottom to top, but also the spectrum frequency substantially increases toward the top of the wall. The horizontal acceleration at the same level but different locations of the wall reveals the peak acceleration occurs at the same time (compare Figures 3.a and 3.b). This observation has some implications in the Pseudo-Static design method in which it is often assumed that wall forces and peak dynamic lateral pressure due to ground excitation do not occur simultaneously, an argument used to reduce the dynamic earth force. The justification for the reduction is that horizontal inertia forces will not reach peak values at the same time during a seismic event [Christopher, et al., 1989]. AASHTO [1996] interim guidelines propose that the dynamic active earth force be reduced by 40% when applied to an arbitrary selected portion of the reinforced soil mass. The north American practice is to reduce dynamic factor of safety against sliding and overturning to 75% of the static factor of safety in recognition of the transient nature of seismic loading [Bathurst and Alfaro, 1996].

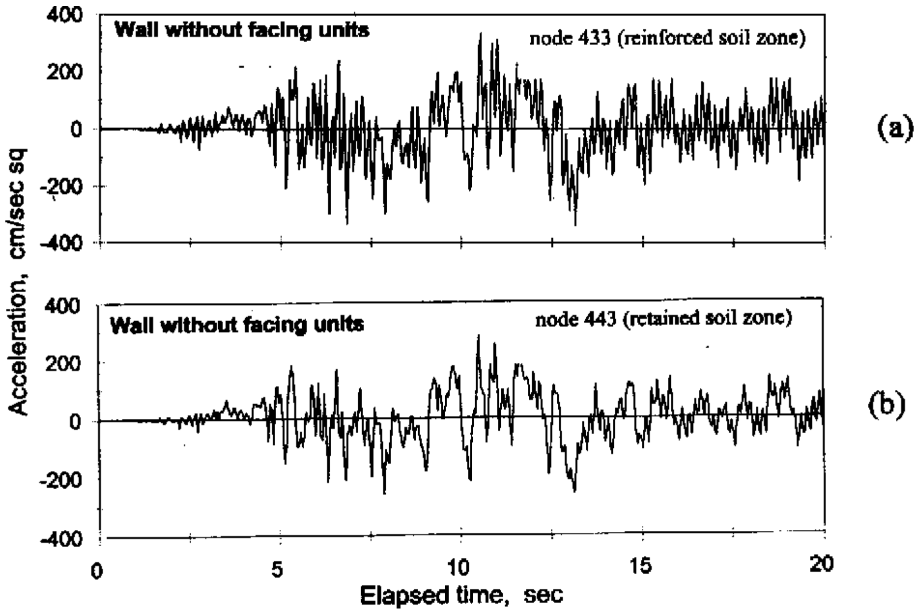

Figure 3. Horizontal acceleration at mid level. Input acceleration in Figure 2.

Lateral displacement of wall face

The displacement time history at the lower level of the wall with facing units, subjected to 0.25g excitation, graphed in Figure 4.b, shows that the displacements fluctuate around static displacement over the ground motion duration, and are not permanent at the end of the excitation. Graphed in Figure 4.a is the fluctuating displacement at the middle level of the wall. The increase in the displacement of the top, rightly so, results from the increasing acceleration along the wall height. A comparison of the displacements in the walls with and without facing units (the latter not shown here) reveals that the lateral displacements in the two cases are nearly the same and are well below the critical value. However, the amplification of the displacement of the wall top for the ones with facing units is relatively less than that for the wall without facing units, reaffirming the use of wall with facing units in seismic areas.

Tensile stress in the reinforcement

The time histories of tensile stress in the reinforcement layers reveal that the stress fluctuates around static stresses over the duration of ground motion with the magnitude of the fluctuating stress depending on the location of the reinforcement element, the top three strips undergoing large fluctuations. A typical stress history is shown in Figure 5.a The tensile stress distribution along reinforcement layer differs for the two cases. For the wall without face the stress tends to be zero at both ends whereas for the wall with face very high stress is observed behind the facing units (Figure 5.b). Also note that the stress in reinforcement is relatively large in the latter case. With the calculated tensile stresses well below the allowable, namely, 20,700 kPa, wall failure by breakage of the reinforcement is unlikely.

Soil-reinforcement slippage

The relative displacement between soil and reinforcement, not shown for brevity, reveals hardly any slippage between soil and the reinforcement at any level due to gravity nor after the excitation with 0.25g or 0.5g peak horizontal accelerations. There is no indication whatsoever of soil shearing over reinforcement surface or soil shearing over soil through the grid apertures, corroborating the results of Jewell, et al. [1984].

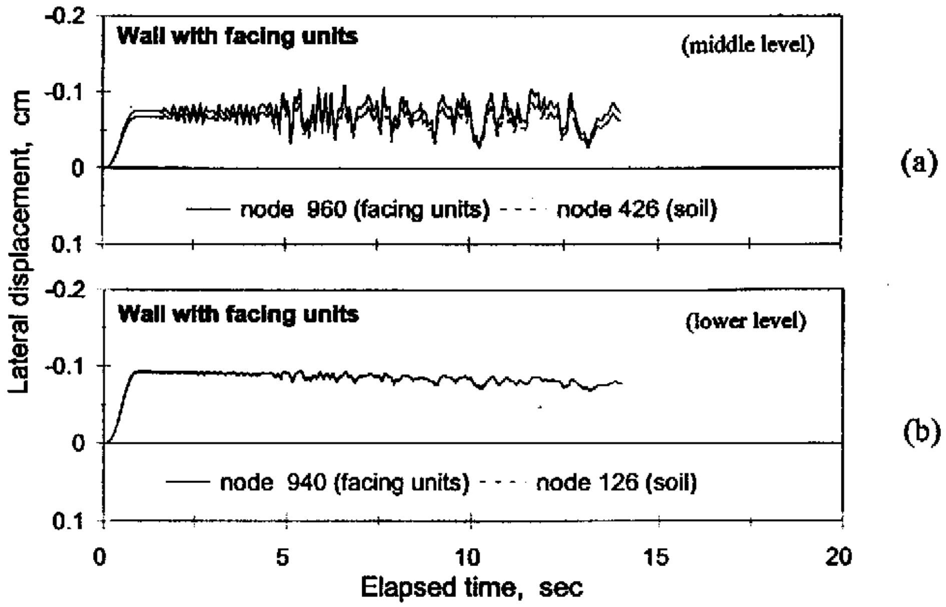

Figure 4. Displacement at two levels. Input acceleration in Figure 2.

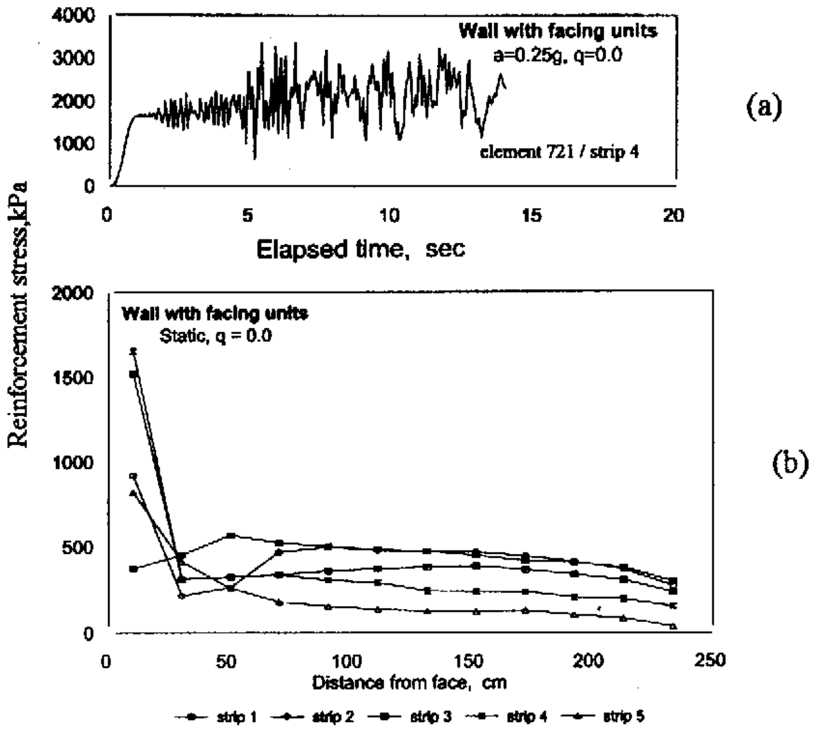

Figure 5. (a) Tensile stress history in strip 4. Input acceleration in Figure 2. (b) Stress distribution for static case (b) Stress distribution for static case

In summary, the results suggest that all of the three possible failure modes can be ruled out when excited by 0.25g or 0.5g horizontal accelerations. Post earthquake observations on existing walls substantiate this finding, as in the case of Loma Prieta earthquake in 1989, Northridge earthquake [Sandri, 1994] or South Hyogo earthquake [Nishimura, et al., 1995]. The walls surveyed in each area withstood the respective earthquakes though the horizontal acceleration peaked at levels of one to four times the design value. A specific instance of satisfactory performance is reported in the survey study of Watsonville wall (geogrid) by Collins, et al. [1982]. Designed for 0.1g, the wall suffered no cracks or excessive deformation despite the actual horizontal acceleration peaked at 0.4g.

Having determined that the wall, in this case designed to withstand 0.25g, would not fail even under a 0.5g peak horizontal acceleration spectrum, surcharge loads of various magnitudes were applied repeating the analyses. Previous researchers, [Bathrust and Benjamin, 1990, and Al-Hussaini and Johnson 1978] have resorted to surcharging techniques to induce failure in reinforced earth walls. Surcharge loadings of 21, 34 and 55 kPa are applied in conjunction with 0.25 peak horizontal acceleration.

Analyses Under Horizontal Acceleration and Surcharge

Lateral displacement of the wall face

Typical displacement-time history of the wall face caused by 0.25g peak horizontal acceleration in conjunction with 34 kPa surcharge is shown in Figure 6. Clearly, the displacement is cumulative over the duration of the ground motion. Note the maximum displacement of the wall without facing units exceeds that of the wall with facing units by 300%. This cumulative pattern of facing element lateral displacement resembles the experimental observations reported for other types of reinforced soil structures, though peak accelerations were scaled differently [Fairless, 1989]. The results also concur with those of Cai and Bathurst's [1995] finite element analysis, with different peak horizontal accelerations. Furthermore, the seismic induced permanent displacement is dominant at the bottom 1/3 to 1/2 of the wall height from the base, in agreement with the full-scale test results on a geotextile reinforced soil structure under static loading [Thamm, et al., 1990]. In accordance with the criterion established in a previous section, the wall without facing units subjected to surcharge loads 21, 34, or 55 kPa would fail after 15, 8 and 5 seconds, respectively, for the lateral displacement exceeding the 6.5 cm criterion. With the same displacement criterion, the wall with facing units would also fail somewhere in the lower third of the wall.

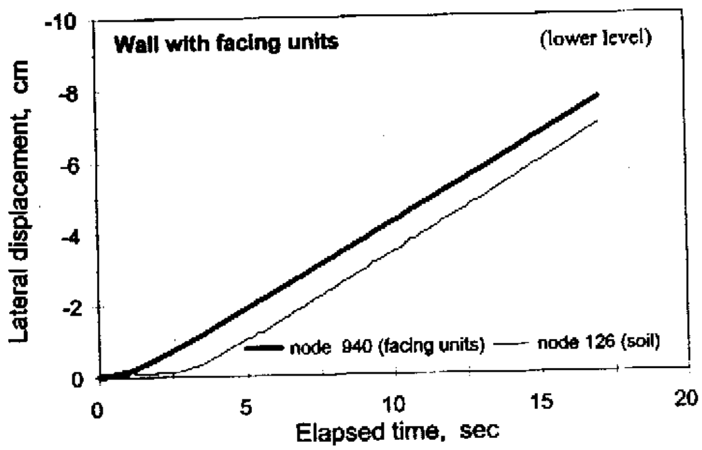

Figure 6. Lateral displacement at lower level. Input acceleration in conjunction with surcharge 34 kPa

Reinforcement tensile stresses

Figures 7.a shows the time history of the tensile stress in element 685 of strip 1, when excited by 0.25g peak acceleration in conjunction with 34 kPa surcharge. The tensile stress, similar but of a larger magnitude, is observed with the same excitation and 55 kPa surcharge (Figure 7.b). In both cases the stresses well exceed the allowable suggesting reinforcement rupture. Comparing the results in Figures 7.a and 7.b, it is noticed that for the same ground motion, a short duration seismic event at large external loads can generate the same tensile stress as a relatively small load of long duration. Cai and Bathurst [1995] reported an analogous result in their study. It is desirable, therefore to have both the peak acceleration and the duration of the event taken into account in designing walls. It is noteworthy that the maximum tensile stress in the reinforcement in the wall with facing is significantly higher (enhanced 400%) than that for the case without facing units.

Soil-reinforcement slippage

Even with 55 kPa surcharge and gravity load only, the relative displacement turns out to be small in the order of 0.01 cm and hence failure by slippage does not arise. When subjected to 0.25g ground acceleration, however, significant increase in slippage is observed, with increase in surcharge (from 21 to 55 kPa) and/or increase in ground motion duration as well. The relative displacement even at 21 kPa surcharge load surpasses the 0.6 cm criterion, indicating imminent wall failure, initiating as early as 3 seconds after the seismic event. It is noteworthy that in the wall with facing units, the slippage is pronounced in the bottom strip, decreasing toward the top strip.

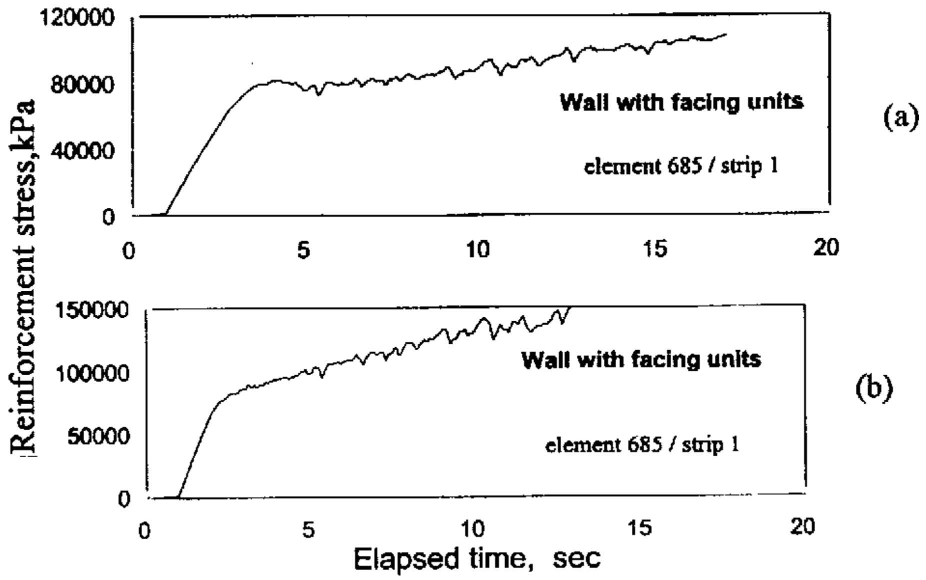

Figure 7. Tensile stress distribution in reinforcement. Input acceleration in conjunction with (a) 34 kPa surcharge (b) 55 kPa surcharge

CONCLUSIONS

For geogrid reinforced retaining walls with and without facing units, when subjected to 0.25g or 0.5g horizontal acceleration, the acceleration response is amplified along the height. The predicted horizontal accelerations at different locations in the wall, however, occur at the same time across the entire wall. These observations have important implications in the Pseudo-static design method in which it is often assumed that wall inertia forces and peak dynamic lateral pressure do not occur simultaneously, an argument used to reduce the dynamic earth force. Other responses, for example, the outward displacement as well as the tensile stress in the reinforcement layers fluctuate around the static values, with their maximum values well below the allowable, signaling no failure. The slippage is too small to be concerned with as well.

After superimposing the surcharge loads, however, the lateral displacement is cumulative and dependent on the duration of the base excitation. The lateral displacement of the wall without facing units is indeed larger than that for the wall with facing units, by about 300%. The tensile stress time history reveals that a short time duration seismic event at large external loads can generate the same tensile stress as a relatively small load of long time duration, cf. with the results of Cai and Bathrust [1995]. As can be expected, the peak reinforcement stress in the wall with facing units far exceeds that without facing units, by about 400%. With prolonged ground motion, the slippage increases, the bottom-most strip experiencing a substantial increase. Notably, the slippage at this level for the wall without facing units is generally higher than that with facing units.

Slippage at the interface seems to be the probable failure mode of the wall without facing units whereas the wall with facing would fail by breakage of the reinforcement. Another noteworthy conclusion is that the wall with facing units inhibits lateral deformation during a seismic event.

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