Discussion of "Subsurface Characterization at Ground Failure Sites in Adapazari, Turkey" by Jonathan D. Bray, Rodolfo B. Sancio, Turan Durgunoglu, Akin Onalp, T. Leslie Youd, Jonathan P. Stewart, Raymond B. Seed, Onder K. Cetin, Ertan Bol, M. B. Baturay, C. Christensen, and T. Karadayilar

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B. Sadık Bakır[1] and M. Tolga Yılmaz[2]

[1]Associate Professor, Dept. of Civil Engineering, Middle East Technical Univ., Ankara, Turkey 06531. E-mail: bakir@metu.edu.tr

[2]Instructor, Dept. of Engineering Sciences, Middle East Technical Univ., Ankara, Turkey 06531. E-mail: mtyilmaz@metu.edu.tr

Valuable cases were presented regarding seismic performance of the shallow mat foundations of building structures in Adapazari, Turkey, during the 17 August 1999 Kocaeli (İzmit) earthquake. The authors attributed the occurrence of displacements of various forms and levels of the mats essentially to the liquefaction or cyclic softening of the saturated fine surface soils of ML/CL type, which dominated those sites. Subsequently, through contrasting the presumed field liquefaction to the analysis results, they evaluated the predictive capability of field-penetration-testing-based liquefaction triggering procedures. It was concluded that in Adapazari, the soils, though they contained significant amounts of clay-size particles and had grain-size distributions within ranges that were believed not to be susceptible to liquefaction, yet liquefied.

Among others, the major drawback of the paper under discussion appears to be *a priori* reasoning of soil liquefaction to explain the observed displacements without a preliminary consideration of whether the bearing capacity failure of the foundations were likely in the cases presented when the seismic demand on building-foundation systems was taken into account. We question this issue, as well as the likelihood of liquefaction of the fine surface deposits encountered in Adapazari during 1999 Kocaeli earthquake.

Liquefaction of Adapazari ML/CL Deposits

One of the arguments put forward in the paper to support the hypothesis of liquefaction of the fine surface deposits in Adapazari was the boils observed at a few of the case sites. Independent reconnaissance teams that visited Adapazari following the earthquake reported that the grab samples from sporadic boils encountered in the city were classified as silty sand (SM), with few exceptions (Yoshida et al. 2001; Sucuoğlu et al. 2000; Bray and Stewart 2000). Still, for the few occurrences in which the ejecta were classified as silt, the sample is likely not to be representative of the soil that actually liquefied, due to mechanisms explained by Fiegel and Kutter (1994). As the sand layers, if existed, are generally deeper and appear to be too dense to have liquefied in the presented soil profiles, a rational explanation concerning the fine sediment ejecta is that such material could have been carried to the surface by the liquefied loose sand which is commonly found embedded in fine surface deposits in the form of seams or pockets in Adapazari. Therefore, the boils observed couldn't possibly constitute reliable evidence indicating liquefaction of fine layers, unless the potential misleading mechanisms are rigorously confirmed not to exist for each case.

The authors discussed conformance of the ML/CL soils which they presumed to have liquefied in the City to the Chinese criteria, originally introduced by Wang (1979) based on observations during Chinese earthquakes. As pointed out by Perlea (2000), the major deficiency of the Chinese criteria remains that the liquefaction susceptibility is not related to the severity of shaking; hence, any soil complying with them should be considered categorically vulnerable to liquefaction, irrespective of the level of shaking. Interestingly, although appearing to be well accepted by the engineering community, a rigorous examination (and confirmation) of the Chinese criteria through comprehensive testing is lacking as yet. Andrews and Martin (2000) presented additional cases regarding "liquefaction" of silty soils, and suggested modification to the Chinese criteria. However, it is to be noted that in their work, which was based on the particle-size distribution analysis of soil-boils, the ejecta contained consistently greater percentages of sand-size particles than the ML/CL soils of Adapazari case sites. This implies that the coarse fraction of the soil could still have dominated the liquefaction trend, and hence any extension of their results to the Adapazari cases would remain questionable.

Results from a series of stress-controlled cyclic tests that were conducted over silty and clayey soil specimens collected from Adapazari (Bray et al. 2004), were referred to in the paper as the experimental evidence relating to the liquefaction or cyclic softening of such soils during the 1999 Kocaeli earthquake. The study shows that, under cyclic stress ratios estimated to have existed in Adapazari during the Kocaeli earthquake, these soils can accumulate significant strain amplitudes that meet the well-accepted strain-based criteria utilized for the liquefaction assessment in sands. Nevertheless, applicability of any such criteria, which is specifically associated with sand, to ML/CL soils would be questionable, since the pore pressure build-up and deformation characteristics of fine-grained soils under cyclic loading can be remarkably different from those of sands (Perlea 2000; Guo and Prakash 1999). Furthermore, from the typical plot of stress-strain response provided in that study, the strain accumulation per load cycle is observed to remain practically constant throughout the test, starting from the very first cycle. Such behavior, however, does not conform to the conventional definition of dynamic liquefaction, since (either sudden or gradual) no transition regarding strain accumulation rate is involved. Yılmaz et al. (2004) investigated the undrained shear and deformation behavior of Adapazari ML/CL soils through a series of parallel standard monotonic and stress-controlled cyclic triaxial tests performed over anisotropically consolidated samples retrieved from sites where significant foundation displacements occurred. They have concluded that, while these soils did not display any trends that could be

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Table 1. Static Factors of Safety of Case Building Foundations and Corresponding Pseudo-static Yield Accelerations for Drained Soil Condition, Based on Formulation by Paolucci and Pecker (1997)

Case	FS_{net} $\Phi'=25°$	FS_{net} $\Phi'=30°$	a_y (g) $\Phi'=25°$	a_y (g) $\Phi'=30°$
C2	10.6	24.9	0.32	0.41
F1	5.2	12.1	0.19	0.26
I2	7.1	16.5	0.24	0.32
I3	6.1	14.1	0.21	0.28
G1	12.7	39.5	0.33	0.41
G3	3.4	7.8	0.11	0.17

interpreted as liquefaction under means of loading comparable to that of 1999 Kocaeli earthquake, their plastic strain accumulation characteristics critically depended on the mode of loading (one-way or two-way), as well as the relative levels of the applied load with regard to the monotonic strength. More specifically, while plastic strains rapidly accumulated at virtually constant rates per cycle in case the monotonic strength was exceeded, they tended to remain insignificantly small otherwise.

Seismic Bearing Capacity on Adapazari ML/CL Deposits

The natural water content to liquid limit ratios of the critical soil layers consisting of low plastic silts and clays at the presented building sites, listed in Table 2 of the authors' paper, are observed to vary between 0.9 and 1.2. Depending on the depositional characteristics and stress history, the saturated recent deposits with consistencies around liquid limit, such as those encountered in Adapazari, can display drastically low levels of undrained strength in the remoulded state; Sharma and Bora (2003) report that the remolded strength can be as low as around 2 kPa for such soils. Concerning this issue, we would like to call attention to the results of the only available shear vane tests series from Adapazari (Site A, boring SPT-A4) presented by Bray et al. (2001), which indicate sensitivities reaching 4, and remolded shear strength as low as 4 kPa. Hence, recognizing that these soils would have been virtually transformed to the remolded state following the first strong pulse during shaking, their inherent weakness under seismic loading is obvious.

To examine whether the foundations of the case buildings were likely to experience bearing-capacity failures during the 1999 Kocaeli earthquake, drained analyses are performed utilizing the formulation presented by Paolucci and Pecker (1997) for the plane-strain condition and associative flow rule. The solution incorporates the impact of seismic demand on the bearing capacity of a surficial foundation through inclination and eccentricity of the load acting on foundation. Inertial load acting on the soil body due to the horizontal component of seismic loading is also accounted for. In the application, soil is presumed to have a buoyant unit weight of 8 kN/m^3, groundwater table is assumed to be located at the foundation level, and the surcharge due to embedment as well as the weight of the foundation is ignored, for simplicity. Bearing capacity is calculated using the N_γ factor provided by Meyerhof (1963). Regarding buildings, a story mass of 1 Mg/m^2 is assumed and the total mass is lumped at two-thirds of the building height. Shear strength of the fine deposits constituting critical soil layers at the building sites in Adapazari can be expressed presumably through drained internal angle of friction (Φ') alone, the probable levels of which appear to be rather low considering the reported values of N_{60} that range between 2 and 8 (Table 2 of the paper). Hence, for the estimated representative values of 25° and 30° of Φ', the static factors of safety (FS_{net}) and the corresponding lateral seismic accelerations (a_y) causing bearing failure of foundation for the case buildings are presented in Table 1 here. Considering that the peak ground accelerations were estimated to be around 0.4 g over deep alluvium sites in Adapazari during the 1999 Kocaeli earthquake (Bakır et al. 2002), the ultimate capacities of these foundations appear to have been significantly exceeded. It is also to be noted that consideration of such other factors as the undrained and nonassociative soil behavior, spectral amplification including soil-structure interaction, and simultaneous inertial acceleration in the perpendicular direction, can result in significantly lower a_y values. Similar conclusions can be reached if the soils are modeled as purely cohesive with remolded shear strengths, as discussed above.

In view of the foregoing, and contrary to the early response of the engineering community, the hypothesis of liquefaction of the fine surface deposits of Adapazari appears to the discussers to be a premature reasoning to explain the observed foundation displacements of building structures associated with the 1999 Kocaeli earthquake. As clearly indicated by the field test data, these saturated fine soils are already rather weak; and the analysis results presented here reveal that the case buildings were indeed likely to sustain foundation bearing failures (and hence foundation displacements of various forms and levels) due to the earthquake, with no required involvement of a mechanism, such as liquefaction, leading to soil weakening. Seismically induced bearing failures of mats on saturated fine soils, in fact, are not uncommon, well-known examples of which were reported from Mexico City associated with the 1985 Michoacan earthquake (Mendoza and Auvinet 1988; Zeevaert 1991).

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S. Feyza Cinicioglu, M.ASCE[1]; Ilknur Bozbey[2]; and Sadik Oztoprak[3]

[1]Professor, Istanbul Univ., Dept. of Civil Engineering, Avcilar, Istanbul, Turkey 34850. E-mail: feyzac@istanbul.edu.tr
[2]Doctor, Istanbul Univ., Dept. of Civil Engineering, Avcilar, Istanbul, Turkey 34850. E-mail: ibozbey@istanbul.edu.tr
[3]Assistant Professor, Istanbul Univ., Dept. of Civil Engineering, Avcilar, Istanbul, Turkey 34850. E-mail: oztoprak@istanbul.edu.tr

The Kocaeli 1999 earthquake caused severe ground failure in Adapazari. Following the event, numerous surveys were made in the area to characterize the subsurface conditions in relation to the observed ground failures and resulting structural damages. This paper can be considered as one of the most extensive of its kind to reflect the results of a comprehensive field-testing program. The authors should be commended for this work, which helps grant insight on the causes of the observed events. However, the main focus of the paper seems to be diverted solely to the liquefaction phenomenon, which casts shadow on the other mechanisms that may well be the more probable cause of some of the observed performances. In this case, considering that all of the selected cases in the paper are supported by shallow foundations, the possibility of seismic bearing-capacity reduction during earthquake loading cannot be overlooked.

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stresses (Marcuson 1978; Youd et al. 2001). The mechanism described in this sense indicates a homogeneous reaction of the soil element to the shear stresses induced by the earthquake forces. The result is total collapse of the soil skeleton. On the other hand, bearing-capacity degradation during earthquake loading occurs by formation of slip planes along which shear stresses reach the shear strength of the soil. Richards et al. (1990) explained the mechanism that led to the bearing-capacity degradation in a general way by fluidization theory for half space. Accommodating this concept into the bearing-capacity mechanism, Richards et al. (1993) showed that the seismic degradation of bearing capacity depends primarily on the shear transfer at the soil-structure interface and the inertial forces within the soil mass. Both of these effects induce shear stresses, using up the reserve strength of the soil to carry the footing load. Shear flow during fluidization occurs gradually as a seismic transition of foundation bearing-capacity failures from general to local and to punching shear. The occurrence of fluidization under the footing may not necessarily result in general failure except for the general fluidization case at which the only remaining bearing capacity is the depth effect corresponding to buoyancy (Richards et al. 1993). Although the above mechanism is described for a single pulse, it is postulated that repeated cycles may cause bearing-capacity failures at lower acceleration levels due to accumulated shear strains. Additionally, for soils under structures, the fluidization mechanism described for free field is more easily activated by the presence of shear stresses. It should be admitted that preexistence of shear stresses may also act to trigger the liquefaction under cyclic stresses only if the soils encountered are liquefiable.

Although originally developed for dry cohesionless soils, the mechanism described in Richards et al. (1990, 1993) has conceptual allowance to cover a large range of soils. Richards et al. (1993) extended the Coulomb failure mechanism to the dynamic earthquake situation and derived seismic bearing-capacity coefficients. There are several other studies in the literature which indicate the occurrence of bearing-capacity loss (Shi and Richards 1995; Kumar and Rao 2002; Fishman et al. 2003).

Remembering that the ground failures examined in Adapazari are restricted to the places beneath and adjacent to the buildings, the discussers considered bearing-capacity degradation calculations as essential. The discussers believe that the comparison of the results can be used as a means of defining the causes of the ground failures other than liquefaction, or at least such a comparison should be made for the sake of completeness. Based on this view, some of the cases in the paper were reevaluated.

Building Site F

In the authors' view, it is probable that the observed downward movement and translation experienced by the building was primarily due to liquefaction of the upper brown silty soils. On the other hand, they did not find it probable for the deeper highly stratified sandy silt to liquefy. Although SPT-based analysis indicated liquefaction, the authors based their argument to the effect of corrections applied for thin layers. According to this finding, 90 cm downward movement of the building was attributed solely to the liquefaction of the 1.70-m-thick silts beneath the foundation of the building, which implies almost 50% volume displacement. However, there was no surface heaving or other liquefaction evidence around the building, but instead the building moved leaving a gap behind. The discussers are dubious for the occurrence of liquefaction in the mentioned critical layer (ML). Cyclic

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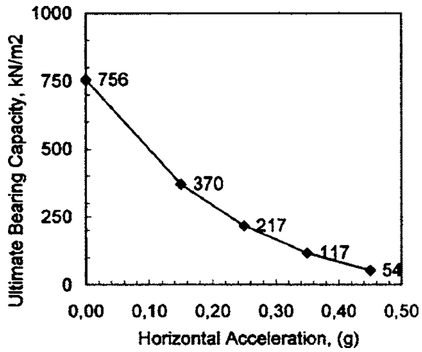

Fig. 1. Variation of ultimate bearing capacity with horizontal acceleration

behavior of silts and clays are different from sandy soils in terms of pore pressure buildup and deformation characteristics (Singh 1994). Additionally, depending on the intensity of applied initial stress (preexistence of Building F), the soil can fail before the pore pressure develops to a level to affect the soil stiffness and strength significantly (Yilmaz et al. 2004). This reminds the discussers of the possibility of bearing-capacity reduction due to formation of slip planes beneath the foundation of a building. According to the discussers, both the vertical and lateral displacement of Building F can be attributed to unidirectional sliding displacement, a phenomenon that usually accompanies seismic bearing-capacity degradation. In this context, seismic bearing capacity for Building F was calculated according to Richards et al. (1993). Width of the foundation was taken as 7.7 m, depth as 1 m, and saturated unit weight of soil as 17 kN/m^3. The internal friction angle of the subsoil was assumed to be 30° based on SPT values. Fig. 1 demonstrates the possible degrees of bearing-capacity reduction for various acceleration levels, covering those mentioned by the authors to have occurred in the region. According to this calculation for acceleration levels 0.35 and 0.45 g, the ultimate seismic bearing capacity reduces to 117 and 54 kN/m^2, respectively, which might be well below the design bearing capacity. Moreover, the results correspond to the effect of a single pulse, and repeating cycles may accelerate degradation tendency.

Building Site C

The dissimilar performances of three nearly identical reinforced concrete 5-story apartment buildings were attributed by the authors to different subsoil conditions encountered beneath these buildings. Sediment ejecta between buildings C1 and C2 seems to be an evidence of liquefaction; however, the discussers would like to extend their argument to consider soil-structure interaction mechanism for Site C. Although the structural properties of individual buildings are almost identical, there seems to be a significant difference when the possible failure wedge mechanisms for each of the buildings are considered. In this context, the possibility of bearing-capacity reduction should still be considered for buildings C1 and C2. On the other hand, for building C3, in addition to nonsusceptibility to liquefaction, the presence of stiff layer at a shallow depth may have increased both the static and seismic bearing capacity. The discussers claim that the soil structure interaction concept should always be treated as an inherent part of pre- and postearthquake assessment studies.

Building Site G

There are three buildings investigated in Site G, two of which, with high aspect ratios, experienced bearing-capacity failures with excessive tilt. There was no significant tilting in the third building with a lower aspect ratio. The authors judged liquefaction of the brown silty soils located below the foundations to be primarily responsible for the bearing failure experienced by buildings G2 and G3. In the discussers' view, the high aspect ratios of buildings G2 and G3 seem to be dominant in the observed result. The rocking settlement was described in Day (2002) as being caused by the dynamic structural loads, which momentarily increase the foundation pressure acting on the soil. Since the geometry of the structure is more effective, rocking action may be observed in both cohesionless soils and cohesive soils even when there is no liquefaction (Day 2002; Seed 1991). According to the discussers, G2 and G3 simply toppled over, since the center of gravity fell outside the kern.

Soil Gradation

The discussers would like to extend their reservations on the authors' view in relation to liquefaction of silts encountered in Adapazari. The authors argued that it was the percent of clay minerals present in the soil and their activity that were important, and nonplastic particles smaller than 5 μm would respond as cohesionless materials in terms of liquefaction. There are numerous findings in literature that indicate the contrary, such as follows:

- "Impacts of fines content (both nonplastic and plastic) on liquefaction resistance are found to be profound." (Chu et al. 2004);
- "Finer silts with flaky or platelike particles generally exhibit sufficient cohesion to inhibit liquefaction." (Kramer 1996);
- "Unlike sands and lab constituted samples of silts, undisturbed samples of silts develop strains before a significant pore pressure increase is recorded." (Singh 1994); and
- "The criteria used to define liquefaction of sand may no longer be applicable for silt clay mixtures." (Yilmaz et al. 2004).

The discussers believe that, while there is no generally agreed upon tool or technique to evaluate the liquefaction susceptibility of fine-grained soils, to claim liquefaction as the main cause of damage and destruction is a misleading approach. With this view, Fig. 18(a) in the paper should be treated with caution. The discussers would like to end the discussion with a quotation from Richards et al. (1990). "A phenomenon is better understood if it can be considered in more than one way."

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Jonathan D. Bray[1]; Rodolfo B. Sancio[2]; Thaleia Travasarou[3]; Turan Durgunoglu[4]; Akin Onalp[5]; T. Leslie Youd[6]; Jonathan P. Stewart[7]; Raymond B. Seed[8]; Onder K. Cetin[9]; Ertan Bol[10]; M. B. Baturay[11]; C. Christensen[12]; and T. Karadayilar[13]

[1]Professor, Dept. of Civil and Environmental Engineering, Univ. of California, Berkeley, CA 94720-1710.

[2]Project Engineer, Golder Associates, Inc., 15603 W. Hardy Rd., Houston, TX 77060.

[3]Project Engineer, Fugro-West, Inc., 1000 Broadway, Suite 200 Oakland, CA 94607-4099.

[4]Professor, Bogaziçi Univ., Istanbul, Turkey.

[5]Professor, Istanbul Kultur Univ., Istanbul, Turkey.

[6]Professor, Brigham Young Univ., Provo, UT 84602-4081.

[7]Associate Professor, Univ. of California, Los Angeles, CA 90095-1593.

[8]Professor, Dept. of Civil and Environmental Engineering, Univ. of California, Berkeley, CA 94720-1710.

[9]Assistant Professor, Middle Eastern Technical Univ., Ankara, Turkey.

[10]Doctoral Candidate, Sakarya Univ., 54040 Adapazari, Turkey.

[11]Senior Staff Engineer, GeoSyntec Consultants, Walnut Creek, CA 94597.

[12]Doctoral Candidate, Brigham Young Univ., Provo, UT 84602-4081.

[13]Engineer, ZETAS Earth Technology Corp., 81150 Istanbul, Turkey.

The writers appreciate the discussers' interest in our study and their differing points of view on the contributing mechanisms involved in the building foundation deformations observed in Adapazari after the 1999 Kocaeli earthquake. The writers believe that these differing opinions may be resolved by providing additional information on soil properties and dynamic analyses that were developed in follow-up investigations that were not presented in the original paper.

Soil Index Properties of Shallow Soils

CPT profiles performed in Adapazari indicate that the soils are stratified. Soil behavior indices (I_c) can vary significantly; however, they generally indicate that the shallow soils are silty (i.e., silt, clayey silt, or silty sand). Careful examination of retrieved soil samples was required to confirm the soils' fines content and the characteristics of the fine-grained fraction of the soil. Hence, hundreds of sieve, hydrometer analyses, and Atterberg limits were performed (Sancio, 2003). As an example of what was typically discovered, the characteristics of the shallow soil at Site F are discussed in greater detail.

Fig. 1 shows the frequency distribution of 20 specimens of silt (ML) obtained from the critical shallow layer at Site F (at depth of 1.5 to 3.2 m). These specimens were recovered by continuous sampling at six different borings using a thin-walled fixed piston sampler during the field program that was carried out for laboratory testing in 2002. Each specimen was extruded from one half of the 45-cm-long sampling tubes. As shown in Fig. 1, all specimens have fines content greater than 50%, and most specimens have $24 < \mathrm{LL} < 36$. Sand lenses were not identified within the critical layer of brown silt at Site F. Similar observations were made at sites G and J, which are located about 300 m from Site F and correspond to a soil profile Type 3 according to the classification described by Sancio et al. (2002), shown in Fig. 2.

Of the sites investigated in Adapazari, a shallow deposit of clean, loose sand (SP) was only found at Site E (see Table 2 of the original paper), even though 46 soil borings with closely spaced SPTs were performed throughout the city after advancing 135 CPTs to locate soil layers that could be susceptible to liquefaction. At the 12 building sites discussed in the paper, a few silty sand specimens were also retrieved. The critical layer at Site H is classified as silty sand (SM), with 15% fines content. Two specimens retrieved below the critical silt layer at Site B are classified as silty sand (SM), with fines contents of 14% and 21%, but these sandy soils have higher CRRs. Two other specimens retrieved below the critical silt layer at Site C are classified as silty sand (SM), but their fines contents are 28% and 45%, so the silt matrix probably controls the cyclic response of these sandy soils, and again these sandy soils have higher CRRs than the looser silts at this site.

It may be concluded from the available data that sand lenses are not common within the shallow brown silt layer that is found

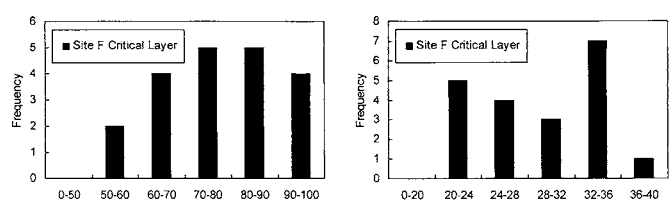

Fig. 1. Frequency distribution of fines content (particles <75 μm) and liquid limit for 20 specimens from the critical layer of Site F

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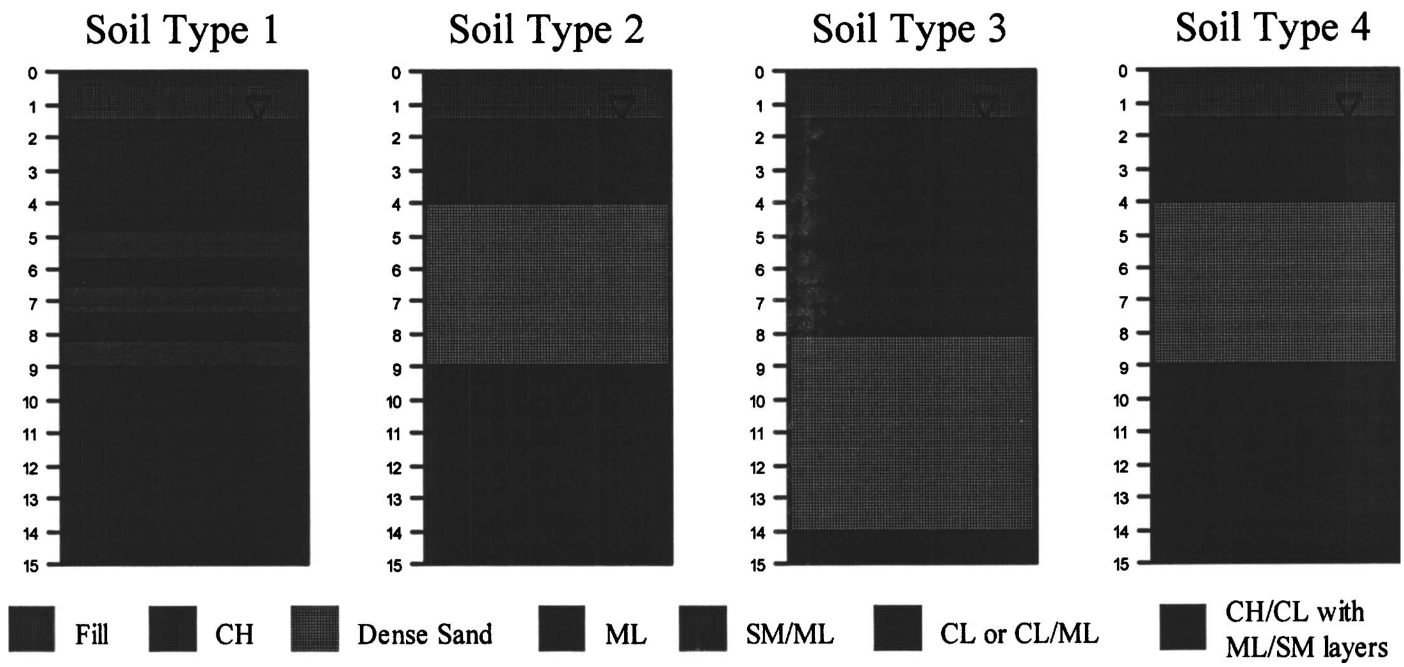

Fig. 2. Generalized subsurface soil profiles in downtown Adapazari (Sancio et al. 2002)

throughout much of Adapazari. A much larger data set of sites throughout the city by Turkish researchers provides further support to this finding (Önalp et al., 2001; Bol and Önalp, 2004). Thus, the writers cannot agree with the suggestion by Bakir and Yilmaz that sand lenses within the shallow silt were the source of the observed liquefaction. It is the writers' opinion that in Adapazari sieve analyses of ejecta typically underestimated the amount of fines present in the soil, because fines were taken away with the water and did not settle out as rapidly as the coarse-grained fraction. The color of the ejecta in all cases matched that of the brown silty soils that were located at depths less than 4.5 m.

Liquefaction of Silts and Clays

As noted by Youd et al. (2001), the procedures delineated therein are applicable to "granular soils" and liquefaction refers to "… the phenomena of seismic generation of large pore-water pressures and consequent softening of granular soils." The liquefaction triggering analyses presented in the writers' paper were performed in accordance with the procedures recommended by the state-of-the-practice consensus document prepared by Youd et al. (2001), without adopting the Chinese criteria, which has been found to be misleading (Bray et al. 2004). Table 2 in the original paper was presented for the use of others as data points for the development of liquefaction triggering procedures for silty soils.

Cyclic triaxial and simple shear testing of Adapazari soils (Sancio 2003; Bray et al. 2004) show remarkable similarities to the results of cyclic testing of clean sands (e.g., Seed 1979). For example, Fig. 3 shows the development of excess pore-water pressure and axial strain in a specimen of silt (ML) with PI=8 (Sample F4-P7B). This silt liquefied (as defined by r_u=100%) in 5 cycles of loading at a CSR=0.40. Table 1 summarizes the data for a series of cyclic triaxial tests performed at a frequency of 0.005 Hz, wherein it may be noticed that the onset of r_u=100% is roughly coincident with the development of 3% axial strain or 5%

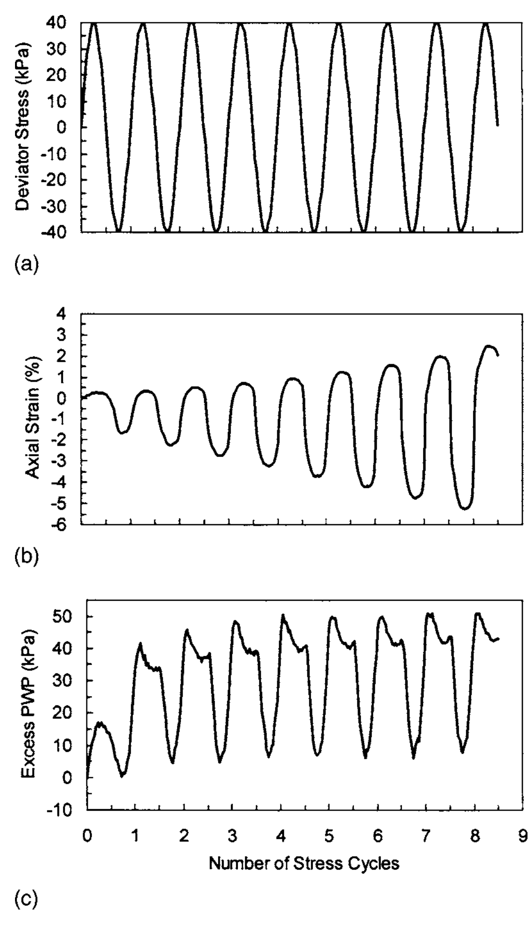

Fig. 3. Results of a cyclic triaxial test (loading frequency of 0.005 Hz) on Specimen F4-P7B (ML, PI=8, e=0.89, σ'_m=50 kPa): (a) deviator stress versus. number of stress cycles, (b) axial strain versus number of stress cycles, and (c) excess pore water pressure versus. number of stress cycles

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Table 1. Soil Index Properties and Results of Cyclic Triaxial Tests Loaded at Frequency of 0.005 Hz

Test	e_{test}	LL (%)	PI (%)	FC (%)	2μ (%)	5μ (%)	USCS	σ'_m (kPa)	σ_d (kPa)	CSR ($\sigma_d/2\sigma'_m$)	N_l-r_u=100%	N_l-3% SA ε_a	N_l-5% DA ε_a
F7-P1A	0.84	34	8	88	16	22	ML	40	32.0	0.40	8.5	5	8.5
F7-P3A	0.76	27	NP	77	12	16	ML	40	40.0	0.50	4	3	4
F6-P4B	0.92	35	9	99	17	22	ML	40	24.0	0.30	14	15	18
F8-P3B	0.70	24	NP	58	6	8	ML	40	32.0	0.40	7	6	7
F4-P7B	0.89	32	8	69	16	22	ML	50	40.0	0.40	5	4	6
A6-P6B	0.87	36	12	–	24	31	CL	50	40.0	0.40	2.5	3	3
I6-P7	1.03	41	14	90	18	25	ML	50	40.0	0.40	12	7	8

Note: e_{test}=void ratio as tested; LL=liquid limit; PI=plasticity index; FC=fines content (amount smaller than 0.075 mm); 2μ=amount smaller than 0.002 mm (from hydrometer test); 5μ=amount smaller than 0.005 mm (from hydrometer test); USCS=Unified Soil Classification System; σ'_m=mean effective stress; σ_d=applied cyclic deviator stress; CSR=cyclic stress ratio applied in cyclic triaxial test; N_l-(r_u=100%)=number of cycles until excess pore water pressure equals the initial mean effective stress; N_l-3% SA=number of cycles to reach 3% axial strain; and N_l-5% SA=number of cycles to reach 5% double amplitude axial strain.

double amplitude strain in the cyclic tests on Adapazari silts. Uniform pore-water pressures throughout the test specimen and reliable measurement at the ends of the specimen were achieved when loading was performed at a rate of 0.005 Hz. Similar responses in terms of the induced strain were observed in cyclic tests performed at a loading rate of 1 Hz. The responses observed in cyclic tests of Adapazari silts at effective confining stresses less than or equal to 100 kPa is similar to that of moderately dense materials described by Youd et al. (2001), i.e., "liquefaction leads to transient softening and increased cyclic shear strains, but a tendency to dilate during shear inhibits major strength loss and large ground deformations."

The writers thus conclude that liquefaction of the shallow silty soils of Adapazari occurred. If the amount of soil finer than 5 microns criterion of the Chinese criteria is ignored, the well-established SPT and CPT liquefaction triggering procedures described in Youd et al. (2001) indicate that liquefaction should have occurred. Sediment ejecta, underground pipe breaks, and ground failure indicative of soil liquefaction were frequently observed, and were more common in areas containing shallow deposits of loose saturated silts as opposed to areas containing shallow deposits of high-plasticity clays (Sancio et al. 2002). Over 100 cyclic triaxial tests and 10 cyclic simple shear tests performed on carefully retrieved, relatively "undisturbed" soil specimens indicated that soils with plasticity indices less than or equal to 12 liquefied at a relatively small number of cycles of loading at the high cyclic stress ratios representative of the 1999 Kocaeli earthquake shaking in Adapazari (Sancio 2003 and Bray et al. 2004). It is unlikely that the development of transient positive excess pore-water pressures with the resulting soil softening and temporary loss of soil strength in the shallow, loose silty soils of Adapazari did not contribute significantly to the observed building foundation deformations.

Soil Static Strength

In the discussion by Cinicioglu et al., the internal friction angle of the silt material was estimated to be 30° based on SPT N-values. Without an explanation, Bakir and Yilmaz assumed that the friction angle of the foundation soil was between 25 and 30°. The value of the friction angle for the Adapazari silt is provided herein to address this issue.

Fig. 4(a) shows the stress path obtained from two strain-controlled anisotropically consolidated undrained triaxial com-

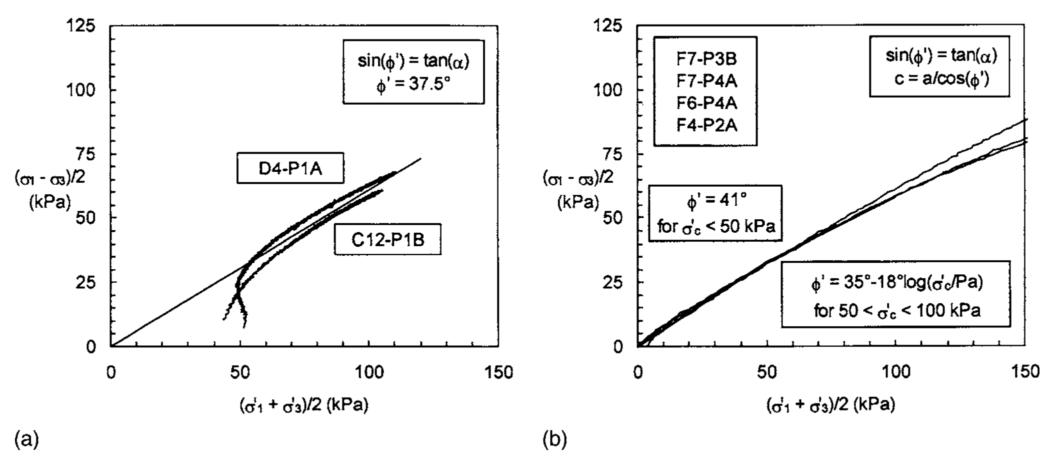

Fig. 4. (a) Stress paths of two specimens of shallow brown silt from sites D and C obtained from anisotropically consolidated triaxial compression tests, and (b) stress paths of four specimens from the critical layer at Site F obtained from stress-controlled triaxial compression after cyclic loading

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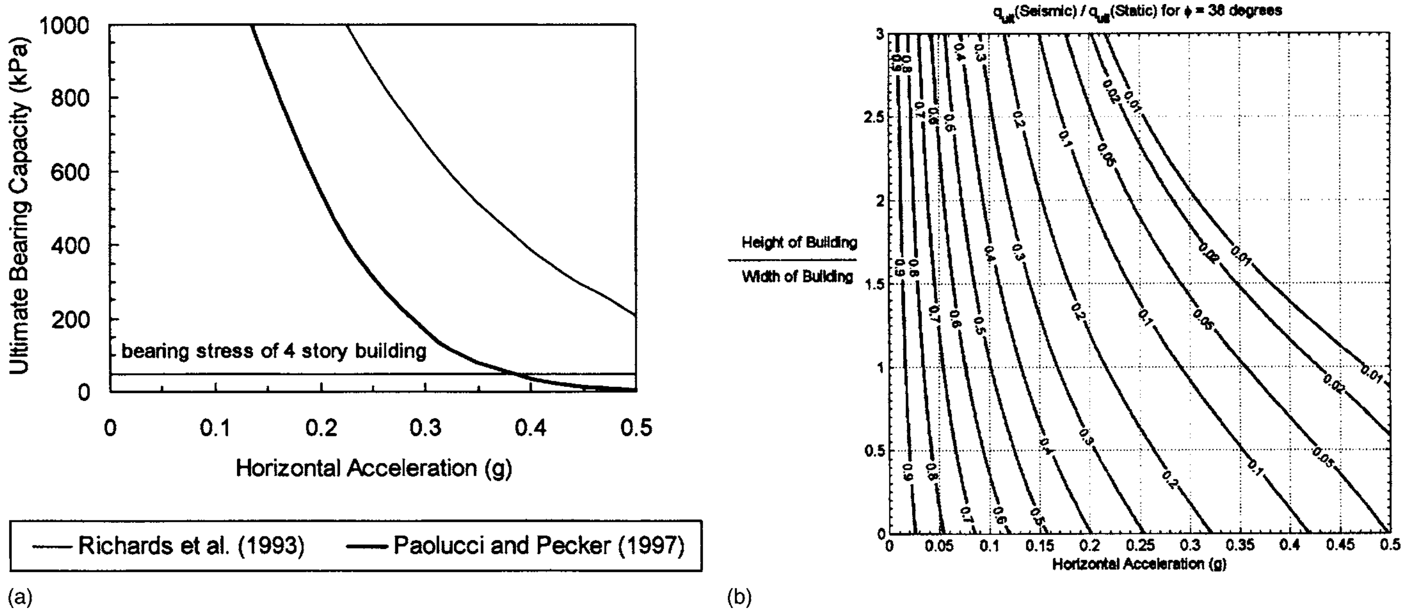

Fig. 5. Ultimate bearing capacity under seismic conditions calculated for the building at Site F (H/B=1.35) using the procedures by Richards et al. (1993) and Paolucci and Pecker (1997), and (b) relationship between seismic and static bearing capacity as a function of the building's height to width ratio and the horizontal peak acceleration

pression tests performed on specimens recovered from the critical layer of Site D (D4-P1A) and Site C (C12-P1A). As shown in this figure, the friction angle described by the stress path is approximately 38°. Fig. 4(b) shows the stress path of four stress-controlled undrained triaxial compression tests. These tests were performed on specimens that had been initially cyclically loaded. The effective stress at the end of the cyclic test and at the beginning of the monotonic triaxial compression test was zero. For these tests, the failure envelope is curved and may be described by $c'=0$ with $\phi'=41°$ for $\sigma_c'<50$ kPa and $\phi'=35° -18°\cdot\log(\sigma_c'/P_a)$ for $50\text{ kPa}<\sigma_c'<100$ kPa, where σ_c' is the effective normal stress and P_a is the atmospheric pressure (i.e., 101 kPa). These relatively high values of friction angle are developed through dilation and are considered representative of the effective strength parameters of the shallow silts of Adapazari.

Static and Seismic Bearing Capacity

Both discussions claim that the excessive building movements observed in Adapazari were primarily due to bearing capacity failures resulting from high inertial dynamic loading of the buildings during the 1999 Kocaeli earthquake. As opposed to the writers, the discussers do not believe that these building movements required liquefaction or significant softening of the foundation soils. The writers agree with the discussers that all possible mechanisms should be explored, and the writers did investigate the buildings' seismic bearing capacity. In fact, a number of the writers first thought that much of the observed building damage was due to bearing failures in clayey soils. However, as described in Sancio et al. (2002), building movements and damage were generally less severe at sites where buildings were directly underlain by clays (i.e., Soil Type 4 of Fig. 2), and damage was more prevalent at sites where liquefaction was observed or calculated to have occurred (i.e., Soil Types 1, 2, and 3 of Fig. 2).

In their discussion, Cinicioglu et al. use a soil friction angle of 30° and the seismic bearing capacity procedure by Richards et al. (1993) to provide a curve showing how the ultimate bearing capacity reduces significantly with increasing horizontal acceleration. Bakir and Yilmaz used the procedure by Paolucci and Pecker (1997) to calculate the horizontal acceleration required to reduce the seismic bearing capacity factor of safety to unity. They used a value of soil friction angle between 25 and 30° and estimated that the total mass of the building is concentrated at two-thirds of the building height. Both seismic bearing capacity procedures were developed for plane strain conditions (i.e., where the length of the mat is at least five times the width of the mat). None of the mat foundations of the buildings studied in this investigation meets this criterion. Additionally, both of these procedures require validation against observations from well-documented field case histories. Use of these procedures suggest that bearing failures of buildings with shallow foundations should be widespread in areas of strong ground shaking even at sites with soils that do not undergo severe strength loss. However, widespread bearing failures have not been documented after major earthquakes at sites that did not undergo severe strength loss due to liquefaction of sandy soils or softening of weak sensitive clays. For example, the cases reported by Mendoza and Auvinet (1988) in Mexico City had soils within the depth of influence of the mat foundations with water contents between 250% and 380% and undrained shear strengths of about 25 kPa. These unfavorable site conditions do not exist in Adapazari.

For the sake of comparison, the writers' used the procedures recommended by the discussers to explore the seismic bearing capacity issue. For the writers' study, the applied vertical load on the mat foundations of the buildings in Adapazari was calculated using the unit weights of the construction materials commonly used in Turkey. The weight of the mat foundation was calculated using a total thickness of 120 cm, of which the lower 30 cm correspond to reinforced concrete with unit weight of 24.5 kN/m^3. The honeycomb structure of the mat formed by the crossbeams is filled with soil. Hence, it is estimated that 80 cm correspond to soil of unit weight equal to 18 kN/m^3 (i.e., considering the weight of the crossbeams). The upper 10 cm consist of a concrete

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slab of unit weight equal to 23.5 kN/m^3. All this considered, the load applied by the mat is 24 kPa. The dead load of each slab (including in-fill walls and flooring) is 5.8 kPa. The roof is considered to have a dead load of 4.5 kPa. Permanent live loads are considered to be 1 kPa per floor. The total bearing stress applied by a four-story structure such as the building at Site F is then equal to 50 kPa (i.e., DL=24 kPa+3×5.8 kPa+4.5 kPa and LL=4×1 kPa).

The total height of the four-story building at Site F is 10.4 m, or approximately 2.6 m per story. The center of mass may then be estimated at two-thirds of the total height. The width of the mat foundation is 7.7 m, so that its height to width ratio is 1.35. Fig. 5(a) shows the values of the ultimate seismic bearing capacity for the building of Site F as a function of horizontal acceleration using the procedure by Richards et al. (1993) and Paolucci and Pecker (1997). A soil friction angle of 38° was used in these calculations. As shown in Fig. 5(a), the ultimate seismic bearing capacity is greater than the applied load for horizontal accelerations less than 0.38 g using the procedure by Paolucci and Pecker (1997). The procedure of Richards et al. (1993) does not calculate bearing failure for horizontal accelerations up to around 0.6 g for this case.

The Paolucci and Pecker (1997) procedure indicates that, as might be expected, a building's ultimate seismic bearing capacity relative to its static bearing capacity increases as the building's height (H) to width (B) ratio decreases [Fig. 5(b)]. Unlike Site F, many buildings in Adapazari had $H/B<1$, and for these cases, a building's seismic capacity increases approximately proportionally to the width of its mat, but the applied building pressure remains constant. Hence, it is difficult to explain the large movements observed at the commonly stout ($H/B<1$) buildings in Adapazari as inertially loaded bearing failures. For these cases, the static factors of safety against bearing of the stout buildings studied in Adapazari using the Terzaghi bearing capacity equation (Vesic 1975) are greater than 60. A significant loss of soil stiffness and strength is required to explain the large building movements at these sites. However, the work of Paolucci and Pecker (1997) does help explain how overturning and excessive tilt of buildings in Adapazari occurred primarily for buildings with height to width ratios greater than 2. In these cases, inertial loading from the building, which reduced its bearing capacity, combined with liquefaction and softening of the underlying soils both contributed to the observed foundation failures.

These bearing capacity calculations presented are based on the assumption that the building mat is founded on homogeneous soil rather than the highly stratified Adapazari alluvium. Dense sand layers that are commonly found in Adapazari at depths between 4.5 and 9 m (e.g., Soil Type 2 in Fig. 2) have significantly greater shear strengths than the shallow silt used for the calculations described above. Yet, bearing capacity failures occurred at a number of sites where these dense sand layers were found just 3 or 4 m below the base of the mat foundations in Adapazari, such as at Site B and Site D. If bearing capacity failures were driven by large inertia forces rather than softening of the shallow silts, then building movements and damage should have been significantly lower at sites with Soil Type 2 than at sites with Soil Type 1, but this was not the case (Sancio et al., 2002; Sancio, 2003).

It should be noted that the horizontal acceleration to be used in Fig. 5 is not the ground acceleration, but rather the spectral acceleration corresponding to the period of the building. This value should consider such effects as lengthening of the fundamental period and increased damping due to structural nonlinearity and soil-structure interaction (SSI), which could be estimated only through advanced nonlinear SSI dynamic analyses that would involve a considerable amount of uncertainty.

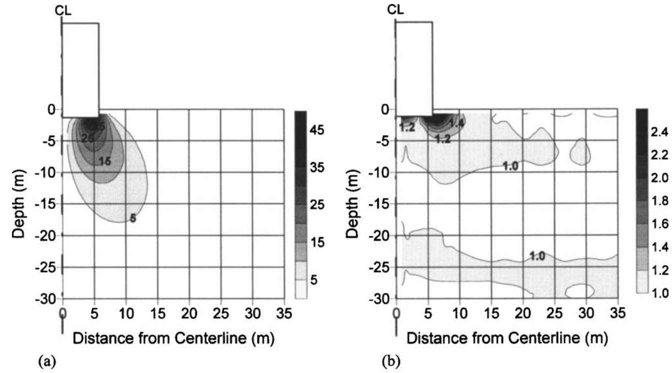

Fig. 6. Additional cyclic stresses imposed on the foundation soils due to the inertial soil structure interaction: (a) additional cyclic vertical stress in kPa, and (b) ratio of the cyclic horizontal shear stress due to the building to that calculated in the free-field, i.e. τ_{bldg}/τ_{ff}

To explore further whether the foundation soils experienced a significant increase in demand due to the inertial response of the building during strong shaking, a series of finite-element analyses was performed using the equivalent-linear finite-element program FLUSH (Lysmer et al. 1975). These total stress analyses do not capture the effects of pore-water pressure generation and the resulting soil-softening induced by the cyclic loading. The FLUSH analyses were used solely to evaluate the likely changes in the stress state of the foundation soils due to SSI effects.

Specific buildings from Adapazari were modeled using the approximate 3D method described in Lysmer et al. (1975), and these buildings and their foundation soils were subjected to a suite of nine acceleration-time histories that represented the likely range of ground motions during the Kocaeli earthquake at a depth of 74 m based on a series of SHAKE analyses of a deeper soil profile (Fig. 2 in the original paper). All input outcropping rock motions are recordings from the Kocaeli earthquake, scaled to a peak ground acceleration of 0.3 g. The results discussed in this closure are for a structure that is similar to the building at Site F in terms of geometry, mass, and stiffness characteristics. The analyzed structure, however, provides a slightly conservative evaluation of the building at Site F, because it is slightly taller (i.e., 5-story building) and heavier (i.e., bearing pressure of 77 kPa). The foundation soil is similar to that at Site F.

Fig. 6 shows the results in terms of cyclic vertical stress and cyclic horizontal shear stress across half of the finite-element mesh when the North–South component of the record at Gebze scaled to a PGA=0.3 g was used as the outcropping rock motion. Fig. 6(a) shows the distribution of the additional cyclic vertical stress induced in the foundation soils due to the rocking of the structure. This additional vertical stress directly below the edges of the mat foundation due to the earthquake loading is at most 60% of that due to the preseismic static dead and live loads, and at a depth of 3 m below the mat foundation, the additional vertical stress below its edge is only a fraction of that due to the static loads (i.e., about 30%). Hence, rocking of the building is not expected to contribute significantly to a large reduction in the building's bearing capacity. This relatively modest increase in vertical stress would not be expected to be sufficiently large to induce shear failure of the foundation soil in the absence of cyclic softening and strength loss.

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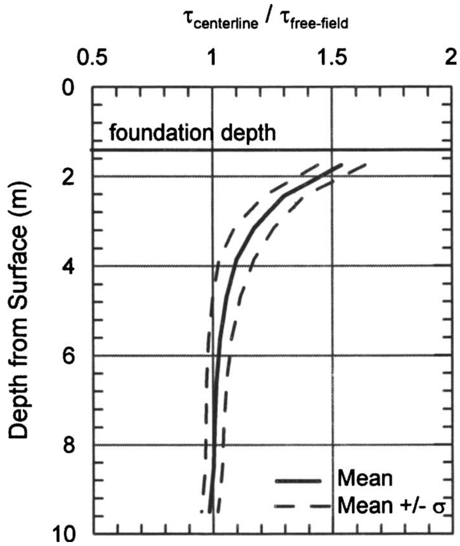

Fig. 7. Ratio of cyclic horizontal shear stress developed in the foundation soil along the centerline of the building over the cyclic horizontal shear stress in the free-field. The mean and +/−1 standard deviation are computed from the response to 9 different excitations at the base scaled to a peak ground acceleration of 0.3 g.

Fig. 6(b) shows the ratio of the cyclic horizontal shear stress calculated by FLUSH for this case relative to the cyclic horizontal shear stress calculated in the free-field away from the building. The regions where the building-induced cyclic shear stress is significantly greater than that of the free-field are confined to shallow depths below the foundation (typically less than 2 m) and are located in the immediate vicinity of the building edges. Such a localized distribution of additional cyclic shear stress is unlikely to produce global bearing failures during a seismic event without significant softening of the shallow foundation soils. Because bearing capacity failures of foundations typically occur on more deeply seated failure surfaces, the writers do not expect that these shallow shear stresses would have induced foundation failure directly. However, these stresses did represent additional seismic demand placed on the soil from the standpoint of liquefaction triggering, and would have contributed to more severe ground softening and strain accumulation than was observed in the free-field.

Fig. 7 shows the increase in cyclic horizontal shear stress due to the inertial response of the building along its centerline with respect to the cyclic horizontal shear stress calculated in the free-field for all nine rock motions. Only the upper 10 m of the soil deposit are shown. The results are plotted in terms of the calculated mean ratio of these responses with its plus and minus one standard deviation. Directly underneath the centerline of the building foundation, the total cyclic shear stress may be approximately 50% higher than that calculated in the free-field. However, this additional cyclic stress due to the building's dynamic response decreases rapidly with increasing depth below the building. It is only about 10% at a vertical distance of 2.5 m below the base of the building's mat foundation. These results are consistent with those presented by Rollins and Seed (1990) in their study of SSI effects due to building shaking. Thus, it is unlikely that the building's dynamic response would have overloaded the soil in the absence of significant strength loss from an increase in the soil's pore-water pressure.

Summary

The objective of the writers' paper was to share the results of a comprehensive field-testing program and present possible mechanisms that led to the observed foundation performance. Research on the foundation failure mechanism is ongoing. It is the writers' hypothesis that soil softening and liquefaction of the shallow silty soils in Adapazari during the 1999 Kocaeli earthquake was caused by an increase of the pore-water pressure in the soil due to cyclic loading, which in turn caused a decrease in the effective stress of these soils that resulted in a significant reduction of its available shear strength. Shear strains developed in these softened/liquefied soils as a result of the building loading (both static and dynamic), but these were typically limited due to the dilative response of the soil, which causes the soil to regain a portion of its preseismic shear strength. Strain accumulation occurred under repeated cyclic loading as the soil continued to resoften during episodes of significant pore-pressure generation. Volumetric compression due to dissipation of excess pore-water pressure also contributed to the observed building movements, but generally this was a minor effect relative to the movements caused by the undrained deviatoric straining of the softened foundation soils. Postseismic volumetric strains were found to be proportional to the maximum cyclic shear strain and ranged from 3 to 5% of the thickness of the liquefied layer (Sancio 2003).

Acknowledgments

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Discussion of "Development of Downdrag on Piles and Pile Groups in Consolidating Soil" by C. J. Lee and Charles W. W. Ng

September 2004, Vol. 130, No. 9, pp. 905–914.
DOI: 10.1061/(ASCE)1090-0241(2004)130:9(905)

Frank M. Clemente Jr., M.ASCE[1]
[1]Technical Director, Geotechnical Engineering, Earth Tech, OneWorld Financial Center, New York, 10281. E-mail: frank.clemente@earthtech.com

This noteworthy paper prompts the following comments in response to two statements by the authors.

The authors state the following:

> "Although the study of dragload on piles has been investigated and well documented in the literature, the investigation of downdrag has received less attention. There are only few reported settlement measurements from field monitoring (Bjerrum et al. 1969; Endo et al. 1969; Lambe et al. 1974; Okabe 1977), …"

The discusser produced a paper (Clemente 1979), which may be of interest to the authors. It included profiles of ground settlement versus time, profiles of settlement versus depth, and profiles of measured downdrag or dragload in prestressed concrete piles in a deep, 43 meter (140 f) thick deposit of soft clay consolidating under a 3.7 meter (12 f) embankment.

The authors state the following:

> "… the inner piles are shielded (or protected) by the outer piles. This suggests that sacrificial piles can be designed and built to protect pile groups in consolidating soils."

The quantity of sacrificial outer piles that may be required to shield the inner piles is likely to be significant and costly. An example of a similar concept (Okabe 1977) employed 14 cased outer piles to shield 24 inner piles.

Where costly sacrificial piles are contemplated, consideration can also be given to coating the piles with bitumen. If anticipated large downdrag and dragloads may lead to unacceptable pile settlement values or to potential structural damage of piles, bitumen coating, about 2 to 5 mm (1/16 to 3/16 in.) thick, if properly specified and controlled in the field, is an economical effective medium for alleviating downdrag and dragload if field preparations are faithfully made (Clemente 1981).

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Discussion of "Design Method for Geogrid-Reinforced Unpaved Roads. I: Development of Design Method" by J. P. Giroud and Jie Han

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DOI: 10.1061/(ASCE)1090-0241(2004)130:8(775)

Michael R. Simac[1]; David J. Elton[2]; and Stephan M. Gale[3]
[1]Principal Engineer, Earth Improvement Technologies, P. O. Box 397, Cramerton, NC 28032. E-mail: simac.eit@earthlink.net
[2]Associate Professor, Dept. of Civil Engineering, Auburn Univ., Auburn, AL. E-mail: elton@eng.auburn.edu
[3]Principal Engineer, Gale-Tec Engineering, Inc., Wayzata, MN. E-mail: smg@gale-tec.com

We commend Giroud and Han on their paper, which adds significantly to the body of knowledge for engineering analysis of unpaved roads. The development of the truncated cone stress distribution represents a technical advancement and simplifies the calculations for this application. The proposed design method also incorporates the modulus of the aggregate base course and subgrade soil as a ratio, permitting quantification of geosynthetic improvement on that ratio, known as the stress distribution angle. The manner in which the geogrid improvement on that modulus ratio or stress distribution angle is formulated and presented in the design method is the focus of this discussion.

Selection of Aperture Stability Modulus for Geogrid Performance

Giroud and Han selected aperture stability modulus (ASM) of geogrid reinforcement as the only performance property upon which to calibrate this new design method for unpaved roads. The basis for this selection are two studies on geogrid reinforcement for paved roads (p. 779) that have far different performance (failure) criteria than the limiting rut depths (50–100 mm) stated as a limitation of the design method (p. 781). The two previous

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Table 1. Comparison of Aggregate Thickness for Unpaved Road Design Methods

	100 cycles of 80 kN axle load, rut 75 mm, CBR=1.0		
Method	Unreinforced	Geogrid	Geotextile
Giroud and Han with B11	**0.41 m**	**0.21 m**	**0.26 m**
Giroud and Noiray with B11	0.38 m	0.25 m	0.25 m
Barenberg with B11	0.43 m	0.26 m	0.26 m
Giroud and Han with B12	**0.41 m**	**0.13 m**	**0.26 m**
Giroud and Noiray with B12	0.38 m	0.25 m	0.25 m
Barenberg with B12	0.43 m	0.24 m	0.24 m

studies appear to be based on the testing of a proprietary polypropylene geogrid with formed ribs and junctions.

Aperture stability modulus (ASM, symbol "J") is an index test conceived to measure the in-plane stiffness of a geogrid reinforcement, by measuring the torsional load required to twist the geogrid through a particular in-plane angular distortion. The procedure attempts to quantify the combined effects of tensile modulus and junction strength. Several researchers, Webster (1992) and Kinney (2000), referenced by the authors, and GRI (2004) have attempted to define the test procedure. However, there is no consensus standardized test method for ASM (J) at this time, unlike the ASTM procedure for CBR, the design parameter for soil and aggregate course base strength.

There are no provisions in the design method to account for installation damage and other environmental factors known to affect synthetic polymer products used in soil reinforcement. This is unique to, and particularly important for, the Giroud-Han method since it attributes a significant amount of the aggregate savings to the ASM (J) of the geogrid reinforcement. Other unpaved road design methods do not rely as much on geosynthetic strength being present throughout the service life, but rather on the mere presence of the geosynthetic improving bearing capacity performance of the subgrade.

Use of ASM is a departure from the well-known standard practice of geosynthetic tensile strength established in Giroud and Noiray (1981), Bender and Barenberg (1978), and the long-standing work by the U.S. Forest Service (see Steward et al. 1977). Additionally, Berg et al. (2000, pp. 63–64) previously reviewed those same paved road studies and concluded that "there was not clear, quantifiable values for these properties (ASM) specifically related to performance." Instead, Berg et al. recommended generic base stabilization performance be based on "empirical evidence," starting with "tensile strength at specific strains."

We suggest that the proposed geogrid-reinforced unpaved road design method would be significantly more generic and applicable were it calibrated to average tensile strength at 2% to 5% strain in two directions, versus ASM. The numerical difference, for say 5%, is illustrated below:

B11	ASM=0.32	avg$T$=11.0 kN/m = (8.5+13.4)/2
B12	ASM=0.65	avg$T$=15.8 kN/m = (11.8+19.8)/2

This change in the ratio between the performance properties of B11 and B12 from 2.03 for ASM to 1.44 for average tensile strength should lead to better correlation with observed performance in the lab and field studies, as shown in Giroud and Han's Figs. 5 and 6.

Typical Aggregate Base Course Thickness for Design Method

The authors' Figs. 5 and 6 clearly show the decreased influence of the proprietary geogrid performance properties with increasing aggregate base course thickness. Therefore, it appears to be more appropriate to base the design method on the performance of the 0.25 m section, which is more representative of typical unpaved road aggregate base thickness, which range from 0.2 to 0.6 m. This would also produce a conservative approximation of geogrid reinforcement effects on aggregate base course thickness less than 0.25 m, a more desirable approach than a potentially unconservative aggregate base course thickness over 0.15 m, as currently proposed by the authors. It is important to maintain a degree of conservatism when attempting to aggressively advance design procedures relative to established design practices that have performed well. This is especially the case when the lab and field research data used to establish the Giroud and Han design method rarely went over 1,000 load cycles and most tests or observations endured less than 500 cycles.

Comparison of Unpaved Road Design Methods

To quantify the aggregate base course reduction proposed in the Giroud and Han design method using ASM, calculations were

Table 2. Comparison of Aggregate Thickness for Unpaved Road Design Methods

	1,000 cycles of 80 kN axle load, rut 100 mm, CBR=1.0		
Method	Unreinforced	Geogrid	Geotextile
Giroud and Han with B11	**0.38 m**	**0.20 m**	**0.25 m**
Giroud and Noiray with B11	0.53 m	0.40 m	0.40 m
Barenberg with B11	0.56 m	0.30 m	0.30 m
Giroud and Han with B12	**0.38 m**	**0.11 m**	**0.25 m**
Giroud and Noiray with B12	0.53 m	0.40 m	0.40 m
Barenberg with B12	0.56 m	0.28 m	0.28 m

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performed using comparable, current-practice design methods for typical stabilization applications. Those results are presented in Tables 1 and 2. For this comparison, the average tensile strength at 5% strain for B11 & B12 was used in both the Giroud and Noiray (1981) and Bender and Barenberg (1978) methods as the geosynthetic performance property. Since those methods do not distinguish between product type, geogrid, or geotextile, identical aggregate thickness result. Contrast this with the Giroud and Han method, which utilizes a lower bearing capacity factor for geotextiles of similar strength as a geogrid, and eliminates any benefit of the tension membrane effect for geotextiles.

Table 1 shows that both the unreinforced and reinforced aggregate thickness calculated by the three methods are similar, except for the Giroud-Han design using B12 geogrid, (high J, ASM). However, when significant load cycles are considered, like in Table 2, the unreinforced aggregate thicknesses are quite different. Although geotextile aggregate thickness compare favorably in Table 2, the unreinforced and geogrid aggregate thickness for Giroud-Han are significantly less than current practice would utilize.

The approximate 70% reduction in aggregate thickness due to incorporation of the B12 geogrid seems particularly aggressive when compared with the 48% reduction for B11 geogrid and the roughly 35% reduction due to geotextile reinforcement calculated by the new Giroud-Han method. These large differences in aggregate thickness seem particularly unusual when considering the relatively small difference in performance characteristics between the three geosynthetics compared.

Summary

These comments and comparative analyses are provided so that more consideration can be given to secant moduli (i.e., average tensile strength at 2% or 5% strain) versus ASM, as the geogrid performance property. It is also suggested that the design method be reformulated based on a typical aggregate thickness of 0.25 versus 0.15 m.

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Closure to "Design Method for Geogrid-Reinforced Unpaved Roads. I: Development of Design Method" by J. P. Giroud and Jie Han

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J. P. Giroud[1] and Jie Han[2]

[1]Consulting Engineer, JP GIROUD, Inc., 5837 North Ocean Blvd., Ocean Ridge, FL 33435.

[2]Associate Professor, Civil, Environmental, and Architectural Engineering Dept., Univ. of Kansas, 1530 W. 15th St., Lawrence, KS 66045-7609.

The writers of the paper are grateful to the discussers for their interest in the research work presented. All the points made in the discussion are addressed below.

Generic Nature of Design Method Presented in the Paper

The discussers suggest that the design method for unpaved roads presented in the paper would be more generic and applicable if it were calibrated using the tensile strength at 5% strain rather than the aperture stability modulus. In fact, the design method presented in the paper is generic. Eq. (11) and the two equations derived from it before the calibration of the method [i.e., Eqs. (14) and (32)] can be used for unreinforced and reinforced unpaved roads; and, in the case of reinforced unpaved roads, these equations can be used for both geotextile reinforcement and geogrid reinforcement. Since Eq. (32) is generic, it can be calibrated using any appropriate characteristic of the geosynthetic through the constant k. Therefore the design method presented in the paper [as expressed by Eq. (32)] does not have to be used with the aperture stability modulus if another relevant parameter can be identified.

Calibration of the Design Method

In the paper, the calibration of the method using the aperture stability modulus starts after Eq. (32). Eq. (41), which is derived from Eq. (32), uses the aperture stability modulus because the writers consider it an appropriate way to characterize with a single parameter the properties that allow a geogrid to reinforce an unpaved road, i.e., mostly the in-plane stiffness of the geogrid and its ability to interlock with aggregate. Using only the tensile modulus (expressed by the tensile strength at 5% strain), as suggested by the discussers, does not account for the geogrid properties that ensure interlocking between the geogrid and aggregate. The irrelevance of the geogrid tensile strength at 5% strain for the design of geogrid-reinforced unpaved roads can be illustrated using the results of full-scale tests carried out by Watts et al. (2004). Fig. 1 gives the traffic benefit ratio (TBR) as a function of the tensile strength at 5% strain of the geogrids tested by Watts et al. (2004). The TBR is defined as the ratio of the number of passes necessary to reach a given rut depth for a section containing reinforcement and the number of passes necessary to reach the same rut depth for an unreinforced section with the same base thickness and subgrade properties. Inspection of Fig. 1

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shows that there is no correlation between the geogrid tensile strength at 5% strain and the performance of the tested unpaved road sections. Also, the writers of the paper calculated the geogrid tensile strain in the unpaved road trafficking tests by Watts et al. (2004) using profiles provided by Watts (personal communication, 2005). These profiles correspond to maximum rut depths (i.e., at end of testing) for Section B of the Watts et al. (2004) tests. It was found that the average geogrid strains under the dual wheels ranged between 0.1 and 1.2%. These values are significantly less than 5%.

Consensus Standardized Test Method

The discussers rightfully note that there is no consensus standardized test method for the aperture stability modulus at this time. The writers of the paper agree that it would be preferable to characterize the geogrid using a property that can be measured using a standardized test method. However, there is a condition that is more important than standardization: the geogrid property used in the design method must be able to quantify (preferably with a single parameter) the road base reinforcement mechanisms associated with geogrids (i.e., tensile reinforcement and interlocking). The writers have used the aperture stability modulus in equations beyond Eq. (32) in the paper because they do not know any other parameter that meets the above condition.

Departure from Other Design Methods

The discussers state that the use of the aperture stability modulus is a departure from the standard practice of using geosynthetic tensile strength established by Giroud and Noiray, Bender and Barenberg, and Steward et al. The design methods cited were developed in the late 1970s for geotextile-reinforced unpaved roads. At that time, geogrids were not available, and it is not surprising that these classical design methods do not use a parameter (i.e., the aperture stability modulus) that has been developed in the 1990s to characterize geogrids. Therefore, the design method presented in the paper is a departure from the classical design methods because it is intended to be so.

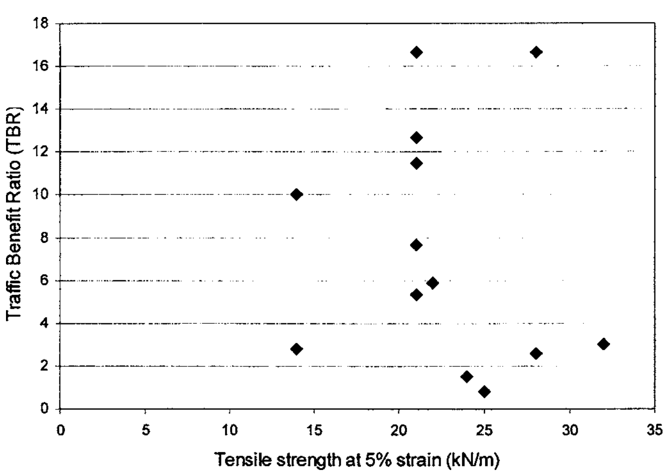

Fig. 1. Relationship between traffic benefit ratio and geogrid tensile strength at 5% strain

Typical Aggregate Base Thickness for Design Method Development

The discussers, referring to Figs. 5 and 6, state that it would have been more appropriate to base the design method on the performance of a base thickness of 0.25 m rather than 0.15 m. It appears that there is a misunderstanding on this point. The curves for the 0.15 and 0.25 m base thickness in Fig. 6 are both represented by Eq. (33) with $x=1.5$ [from Eq. (36)], $r=0.15$ m [from Eq. (3)], and $h=0.15$ m for the top curve and 0.25 m for the bottom curve. Therefore, the method is based on both the 0.15 and the 0.25 m base thickness. One may even argue that the method relies more on the base thickness of 0.25 m than on the base thickness of 0.15 m because three experimental data points are available for the 0.25 m base versus two for the 0.15 m base (as seen in Fig. 6). Clearly, on this point, there should be no disagreement between the discussers and the writers.

Installation Damage and Environmental Factors

The discussers mention, rightfully, that the design method presented in the paper does not account for installation damage and environmental factors that may affect geosynthetics. In soil reinforcement applications, installation damage and degradation due to environmental factors are generally accounted for by decreasing the properties of the geosynthetics, not by altering the design method. Therefore, the same approach can be used for the design method presented in the paper. In other words, no change in the equations is required to address installation damage and environmental factors. It is up to the users of the equations to reduce the properties of the geosynthetics to reflect the expected installation damage and degradation with time during the design life of the unpaved road.

Tensioned Membrane Effect

The discussers state that the design method presented in the paper eliminates any benefit of the tensioned membrane effect for the cases where the unpaved road is reinforced using a geotextile. As stated in the paper, the tensioned membrane effect is significant only when the ruts are deeper than approximately 150 mm, as indicated by Giroud et al. (1985). It is therefore conservative to neglect the tensioned membrane effect in most practical cases. This appears to be confirmed by the calculation results presented in Tables 1 and 2 by the discussers: the same values were obtained for B11 and B12 geogrids with the Giroud and Noiray method, the only design method that quantifies the tensioned membrane effect, to the best of the writers' knowledge.

Base Thickness Obtained with Different Unpaved Road Design Methods

The discussers performed numerical calculations using the design method presented in the paper and two design methods developed for geotextile reinforcement: the Giroud and Noiray (1981) method and the Bender and Barenberg (1978) method. The calculations were done for unreinforced unpaved roads and for unpaved roads reinforced using geotextile or geogrid. One geotextile and two different geogrids are used in the calculations. The calculations performed using the Giroud and Noiray (1981) method and the Bender and Barenberg (1978) method show little

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difference between geotextile and geogrid reinforcement and between the two geogrids. In contrast, the calculations performed using the method presented in the paper show a significant difference in the calculated base thickness between geotextile-reinforced unpaved road and geogrid-reinforced unpaved road, and between the cases of the two considered geogrids. The discussers conclude that the differences are not justified by the relatively small difference in performance characteristics between the geosynthetics involved. It should be noted that, with the term "performance characteristics," the discussers refer to the tensile strength at 5% strain. The difference between the characteristics of the geosynthetics involved in the calculations (one geotextile and two geogrids) is greater when the aperture stability modulus is used, which explains the difference in calculated base thickness. Clearly, the tensile strength at 5% strain does not appear to be a relevant parameter of the performance of geogrid-reinforced unpaved roads.

Current Practice

Finally, the discussers note that the aggregate base thickness is smaller when calculated with the method presented in the paper than with the Giroud and Noiray and Bender and Barenberg methods, for both unreinforced and reinforced unpaved roads. They conclude that the aggregate thickness calculated using the design method presented in the paper is significantly less than current practice would utilize. The writers are not surprised that a smaller aggregate base thickness was obtained using the method presented in the paper rather than the Giroud and Noiray method. To illustrate this point, it is interesting to consider a case where field data are available, such as the Tingle and Webster's 2003 study reported in Table 3 of the writers' companion paper (Giroud and Han 2004). The unreinforced base thickness was 0.51 m, whereas the calculated base thickness was 0.59 m with the method presented in this paper and 0.82 m with the Giroud and Noiray method. It is also interesting to note that the Giroud and Noiray method predicts with a good approximation the thickness reduction (0.14 m) that results from geotextile reinforcement in the Tingle and Webster case. However, the resulting predicted thickness in the case of geotextile reinforcement (0.68 m) is overestimated due to the overestimated thickness calculated for the unreinforced base (0.82 m). Clearly, in this well-documented case, the thicknesses predicted using the method presented in the paper are closer to the measured values than those predicted using the Giroud and Noiray method. However, no general conclusions can be drawn from the few cases discussed herein. A comprehensive study would be required to compare predicted base thicknesses, actually measured base thicknesses, and base thicknesses used in practice. Regarding the state of practice, it is interesting to note that the paper's writers have identified more than 20 geogrid-reinforced unpaved roads or areas, constructed between April 2004 and May 2005, that were designed using the method presented in the paper. (These are probably only a fraction of the unpaved structures designed using the method presented in the paper.) To the best of the writers' knowledge, there has been no negative response regarding the performance of these unpaved structures or any other unpaved structures designed using the presented method.

Comment on Giroud and Noiray Method

As the Giroud and Noiray method is often used as a reference for comparisons (as seen in the above discussions), it is appropriate to mention that there is an error in Eq. (26) of the paper by Giroud and Noiray (1981). The correct version is as follows:

$$h_0' = \frac{125.70 \log N + 496.52 \log P_A - 294.14s - 2412.42}{(c_u)^{0.63}}$$

where h_0'=thickness of unreinforced aggregate base (m); N=number of passes of axle; P_A=axle load (N); s=rut depth (m); and c_u=undrained cohesion of the subgrade soil (Pa).

Both the above equation and the following equation give 0.82 m for the unreinforced base thickness in the above discussion.

The above equation is equivalent to the following equation:

$$h_0' = \frac{0.19 \log N_s - 2.34(s - 0.075)}{(\mathrm{CBR}_{sg})^{0.63}}$$

where N_s=number of passes of a standard axle, with

$$N_s = N\left(\frac{P_A}{80{,}000}\right)^{3.95}$$

and the undrained cohesion of the subgrade is related to its CBR as follows:

$$c_u = 30{,}000\ \mathrm{CBR}_{sg}$$

Conclusion

The writers of the paper would like to thank the discussers for a constructive discussion that offered an opportunity to clarify some important points.

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