A better match could, however, be obtained when a considerably softened or weakened zone was assumed surrounding the piles (program PILAY 2) simulating disturbance to soil during pile installation. A loss of contact of the soil with the pile for a short length close to the ground surface also improved the predicted response.

Novak and El-Sharnouby (1984) performed tests on 102 model pile groups using steel pipe piles. A typical comparison of the theoretical and experimental horizontal response is shown in Fig.2. Plot ‘a’ shows the theoretical group response without interaction effects. Response shown in plot ‘b’ was obtained by applying static interaction factors to stiffness only. Plot ‘c’ was obtained with arbitrary interaction factor of 2.85 applied to stiffness only. Plot ‘d’ was obtained by using an arbitrary interaction factor of 2.85 on stiffness and 1.8 on damping respectively. Plot ‘e’ shows the experimental data. The plot which shows an excellent match with experimental data was obtained by *arbitrarily* increasing the damping factor by 45%. These reduction

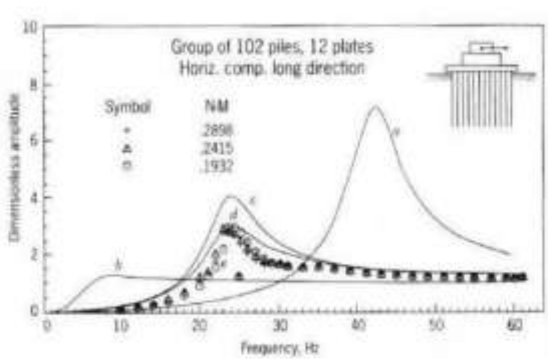


Fig. 2. Experiment horizontal response curves and theoretical curves calculated with static interaction factors. (Novak and El-Sharnouby, 1984)

factors had been arbitrarily selected to match the predicted response with the observed one.

The purpose of this study was to evaluate appropriate reduction factors for stiffness and radiation damping determined using the analytical approach developed by Novark and El-Sharnouby (1983).

EXPERIMENTAL TESTS DATA

The experimental tests conducted by Gle (1981), on pipe-piles of 12.75 and 14 inch outside diameter have been used in this study. Gle tested four different single steel pipe piles at two different sites in Southeastern Michigan. The soil profiles at these sites were predominantly composed of clayey soils. Each pile was tested at several vibrator-operating speeds. A total of eighteen dynamic lateral tests were conducted in clayey and silty sand media.

METHOD OF ANALYSIS

The method of analysis used in this study is as follows (Jadi (1999) and Prakash and Jadi (2001)):

Step 1. Field data obtained from lateral dynamic tests performed by Gle (1981) on full-scale single piles embedded in clayey soils, were collected.

Step 2. Theoretical dynamic response was computed for the test piles, using Novak and El-Sharnouby’s (1983) analytical solution for stiffness and damping constants, with no corrections.

Step 3. The soil’s shear modulus and radiation damping used for the response calculations were arbitrarily reduced, such that measured and predicted natural frequencies and resonant amplitude matched.

Step 4. The reduction factors obtained from step 3 were plotted versus shear strain at resonance without corrected G and ‘c’. Two quadratic equations were developed to determine the shear modulus reduction factors (λ_G) versus shear strain, (γ) and the radiation damping reduction factor (λ_C) versus shear strain (γ).

Step 5. For all the pile tests considered in this study, the empirical equations determined in step 4 were used to calculate shear modulus and radiation damping reduction factors. Predicted responses before and after applying the proposed reduction factors were then compared to the measured response.

Step 6. To validate this approach, the proposed equations were used to calculate shear modulus and radiation damping reduction factors for different sets of field pile tests. The new predicted response was then compared to the measured response, both for Gle (1981) tests and two other cases.

COMPARISON OF COMPUTED AND PREDICTED PILE RESPONSE IN NON LIQUEFYING SOILS

Jadi (1999) and Prakash and Jadi (2001) reanalyzed the reported pile test data of Gle (1981) for the lateral dynamic tests on single piles and proposed reduction factors for the stiffness and radiation damping obtained by using the approach of Novak and El-Sharnouby (1983) as:

$$\lambda_G = -353500\,\gamma^2 - 0.00775\,\gamma + 0.3244 \quad (1)$$

$$\lambda_c = 217600\,\gamma^2 - 1905.56\,\gamma + 0.6 \quad (2)$$

where, λ_G and λ_c are the reduction factors for shear modulus and damping and γ is shear strain at computed peak amplitude, without any correction.

Figure 3 shows prediction and performance of Gle's pile. It may be noted the reduction factors for shear modulus and damping had been developed from tests by Gle. Therefore, this match is obvious.

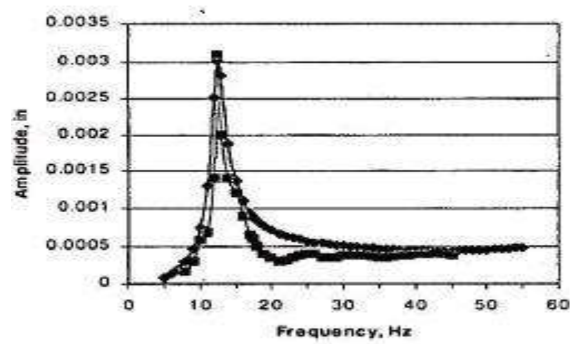


Fig. 3 Measured vs Reduced predicted lateral dynamic response using proposed equations for pile K16-7(θ=5°) at Belle River site. λ_G = 0.321, λ_c= 0.4 (Prakash and Jadi, 1999)

Figures 4,5 and 6 show plots of computed and measured response of several piles tested by Gle (1981).

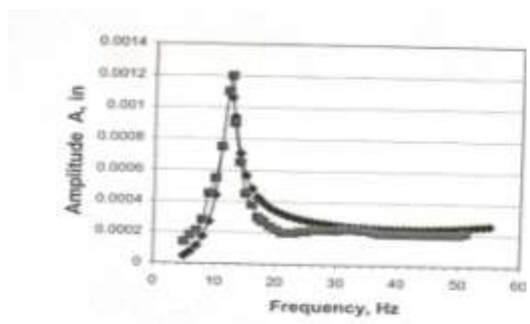


Fig 4. Measured and Reduced predicted lateral dynamic response for pile for lateral dynamic load test for pile L1810 θ=2.5^0 , Belle river site (Jadi, 1999)

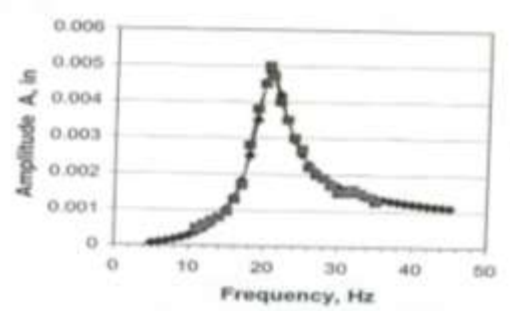


Fig 5. Measured and arbitrarily reduced predicted lateral dynamic response for pile LF16, θ=10^0, St. Clair Site (Jadi, 1999)

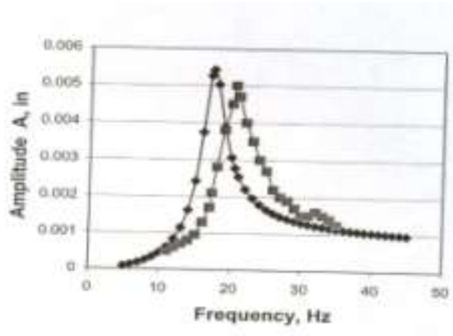


Fig 6. Measured and reduced predicted lateral dynamic response for pile LF16, θ=10⁰, St. Clair Site λ_G = 0.321, λ_c= 0.4 (Jadi, 1999)

Plots of the measured resonant frequencies and amplitudes versus corresponding predicted values determined with proposed reduction factors were constructed. Fig. 7 shows the measured natural frequencies of all test piles versus predicted natural frequencies. Fig. 8 shows measured resonant amplitudes for all test piles versus predicted resonant amplitudes. These figures show that all points fall into the zone of the 45 degree line, providing that predicted resonant frequencies and amplitudes are comparable to the measured values.

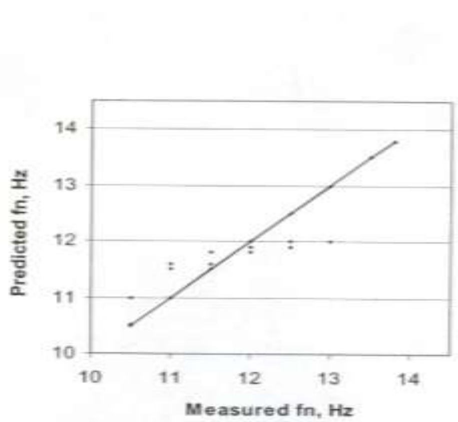


Fig 7. Measured natural frequency versus predicted natural frequency computed with proposed shear modulus reduction factor (Jadi, 1999)

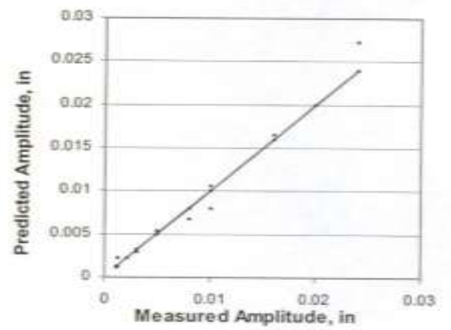


Fig 8. Measured resonant amplitude versus predicted resonant amplitude computed with proposed radiation damping reduction factor (Jadi, 1999)

CHECK WITH DIFFERENT TEST DATA

In order to confirm the validity of the proposed method dynamic response of different sets of experimental data from other sites were also checked. Two series of experimental data were analyzed. Blaney (1983) carried out two lateral dynamic tests on the single pile, embedded in the clayey soils. The first test was performed with a 'WES' (Waterways Express Station) vibrator. For the second test an 'FHWA' (Federal Highway Administration) vibrator was used.

Figure 9 represents the predicted response computed by applying suggested shear modulus and radiation damping reduction factors and measured lateral dynamic response of the same pile.

Resonant amplitudes matches, but computed natural frequency is about 40% higher.

Figure 10 also confirms the same observation. However these figures show that the predicted response with proposed reduction factors compares much better to be measured response, as compared to the predictions by Blaney (Jadi, 1999).

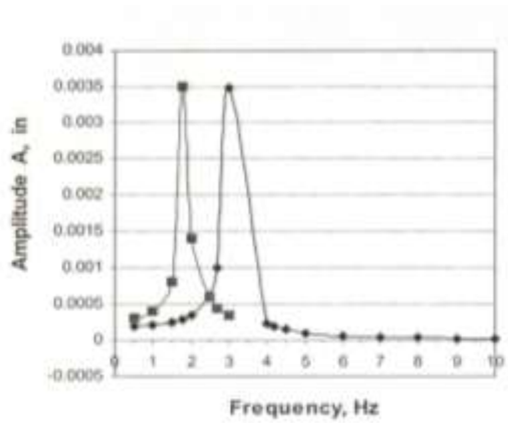
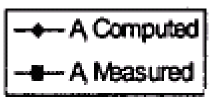


Fig 9 Measured vs reduced predicted lateral dynamic response for pile 1 using proposed reduction factors, WES vibrator, $\lambda_G = 0.31$, $\lambda_c = 0.5$ (Jadi, 1999)



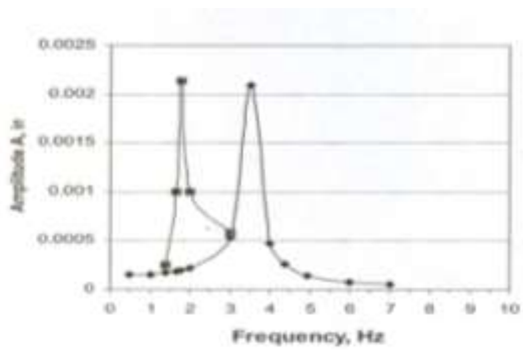


Fig 10 Measured vs reduced predicted lateral dynamic response for pile 1 using proposed reduction factors, FHWA vibrator, $\lambda_G = 0.32$, $\lambda_c = 0.54$ (Jadi, 1999)

The second experimental data considered for validation of the proposed method, consisted of Novak and Grigg's (1976) lateral dynamic test. This test was performed on a small single pile embedded in a very fine silty sand layer. Fig 11 shows measured and predicted lateral dynamic response for the 2.4 inch diameter pipe pile without corrections. Fig 12 shows the measured and predicted lateral dynamic response computed with proposed reductions for the same pile. As can be seen, the predicted response with proposed reduction factors becomes much closer to the measured response.

The comparative analysis presented herein validates the effectiveness of the proposed reduction factors for piles embedded in clayey soils.

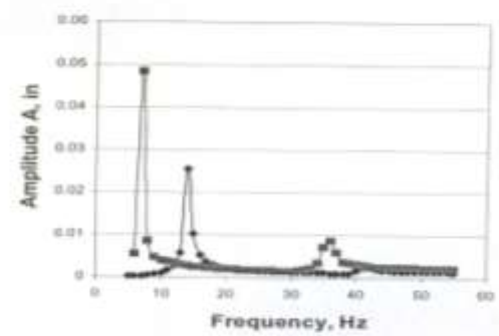


Fig 11 measured vs predicted lateral dynamic response for the 2.4" pile tested by Novack and Grigg, 1976 without correction factors.

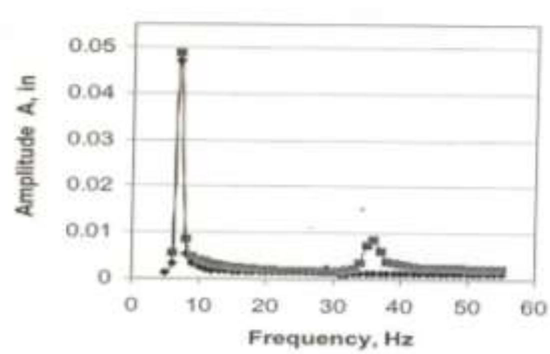


Fig 12. Measured vs reduced predicted lateral dynamic response for the 2.4" test pile using proposed reduction factors $\lambda_G = 0.044$, $\lambda_c = 0.34$ (Jadi, 1999)

COMMENTS ON PREDICTIONS

Novak and El Sharnouby (1984) have attempted to match the observed with predicted response by adjusting, arbitrarily, the group stiffness and damping values. No guidelines were developed to modify these values.

Woods (1984) used Pilay program with modified stiffness to match prediction and performance.

Jadi (1999) developed rational correction factors to both stiffness and damping to match the computed and predicted responses. She was reasonably successful in her efforts. Her approach is more scientific but based on a limited data. More studies are needed to develop relationships for the reduction factors for different modes of vibration, and different soils.

PILES IN SOILS SUCEPTIBLE TO LIQUEFACTION

Excess pore pressures during seismic motion may cause lateral spreading resulting in large moments in the piles and settlements and tilt of the pile cap and the superstructure. Excessive lateral pressure may lead to failure of the piles which was experienced in the 1964 Niigata and the 1995 Kobe earthquakes (Finn and Fujita, 2004).

Damage to a pile under a building in Niigata caused by about 1 m of ground displacement is shown Figure 13 (Yasuda et al., 1999). Displacement of Quay wall and damage to piles supporting tank TA72 (Fig. 14 and 15) during 1995 Kobe earthquake has been reported by Cubrinovski and Ishihara (2004).

The quay wall moved approximately 1 m towards the sea. The seaward movement of the quay wall was accompanied by lateral spreading of the backfill soils resulting in a number of cracks on the ground inland from the waterfront. This observation indicates that liquefaction and resulting lateral spreading of the backfill soil seriously affected the pile performance. It is observed from the above failures that the failure of pile section occurs not at the junction of the pile with the cap, where moment is maximum, but at depth of 6 to 8 m. It appears the pile and the cap joint may have suffered a potential damage resulting in the moment decrease at the top and transfer of the moment at appropriate depths below where the pile section failed. This behavior has also been observed in failures of piles under static lateral loads.

Therefore, prediction of quality of the pile cap joint is an unanswered question.

Fig: 13 A Pile Damaged by lateral Ground Displacement During 1964 Niigata Earthquake (Yasuda et al, 1999)

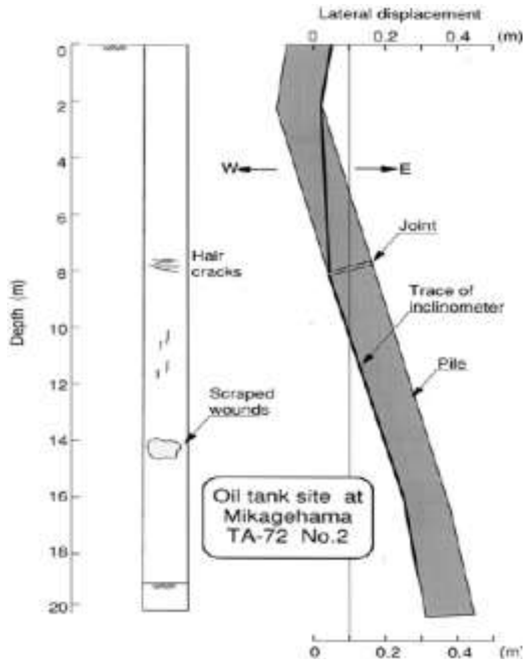


Fig14. Lateral Displacement and cracking of Pile no. 2 (Ishihara and Cubrinovski 2004)

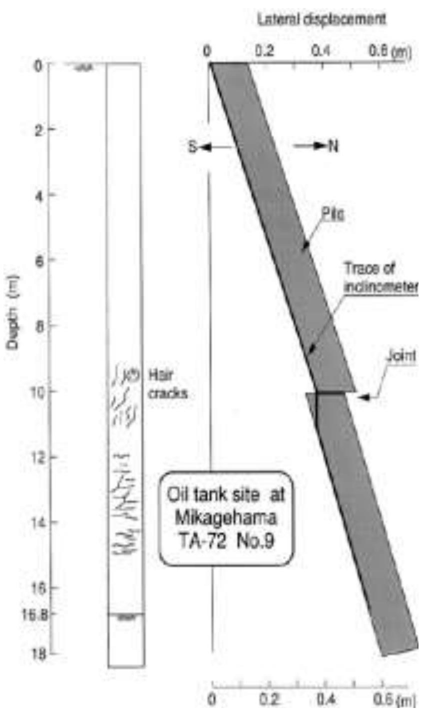


Fig. 15 Lateral Displacement and cracking of Pile no. 2 (Ishihara and Cubrinovski , 2004)

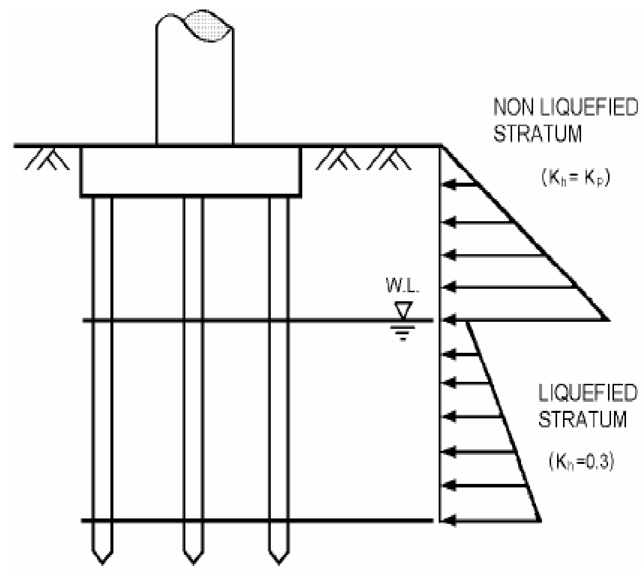


Fig. 16 Schematic Sketch Showing Pressure Distribution Against the Piles due to Lateral Soil Flow associated with Liquefaction (JWWA, 1997) (Ashford and Juirnarongrit, 2004 and Finn Fujita, 2004).

DESIGN METHODS

The methods currently in use for design of piles in liquefying soil are;

1. The force or limit equilibrium analysis and
2. The displacement or p-y analysis.

The Force or Limit Equilibrium Analysis

The method of analysis is recommended in several Japanese design codes for analysis of pile foundations in liquefied soils undergoing lateral spreading (JWWA, 1997; JRA, 1996). The method involves estimation of lateral soil pressures on pile and then evaluating the pile response. A schematic sketch showing lateral pressures due to non-liquefied and liquefied soil layers is shown in Fig.16. The non-liquefied top layer is assumed to exert passive pressure on the pile. The liquefied layer is assumed to apply a pressure which is about 30% of the total overburden pressure This estimation of pressure is based on back calculation from case histories of performance of pile foundations during the Kobe earthquake . The maximum bending moment is assumed to occur at interface between the liquefied and non-liquefied soil layer.

Displacement or p-y Analysis

This method involves making Winkler type spring mass model shown schematically in Fig.17. The empirically estimated post liquefaction free field displacements are calculated. These displacements are assumed to vary linearly and applied to the spings of the soil-pile system as shown in Fig. 17 (Finn and Thavaraj, 2001). Degraded p-y curves may be used for this kind of analysis. In the Japanese practice the springs are assumed to be linearly elastic-plastic and can be determined from the elastic modulus of soil using semi-empirical formulas (Finn and Fujita , 2004). The soil modulus can be evaluated from plate load tests or standard penetration tests. Reduction in spring stiffness is recommended by JRA (1996) to account for the effect of liquefaction. Such reduction is based on FL (factor of safety against liquefaction).

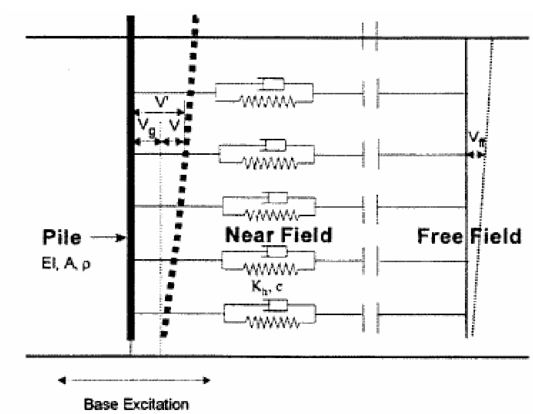


Fig.17. A Schematic Sketch for Winkler Spring Model for Pile Foundation Analysis (Finn and Thavaraj, 2001)

The U.S practice is to multiply the p-y curves by a uniform degradation factor 'p', which is commonly referred to as the p-multiplier. The typical values of 'p' range from 0.3 to 0.1%.The values 'p' seem to decrease with pore water pressure increase (Dobry et al 1995) and become 0.1 when the excess pore water pressure is 100%. Wilson et al(1999) suggested that the value of 'p' for a fully liquefied soil also depends on the initial relative density Dr. The values of 'p' range from 0.1 to 0.2 for sand at about 35% relative density and from 0.25 to 0.35 for a relative density of 55%. It was found that the resistance of the loose sand did not pick up even at substantial strains but the denser sand, after an initial strain range in which it showed little strength, picked up strength with increasing strain Fig.18 . This finding suggests that the good performance of the degraded p-y curves which did not include an initial range of low or zero strength ,must be test specific and the p-multiplier may be expected to vary from one design situation to another.

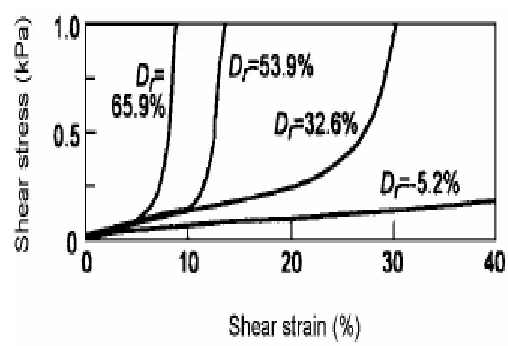


Fig.18 Post-liquefaction undrained stress-strain behavior of sand (Yasuda et al 1999)

Dilatancy effects may reduce the initial p-y response of the dense sands(Yasuda et al 1999). Ashford and Juirnarongrit (2004) compared the force based analysis and the displacement based analysis for the case of single piles subjected to lateral spreading problems. They observed that the force based analysis reasonably estimated the pile moments but underestimated pile displacements. The displacement analysis was found to make better prediction about the pile moment and the pile displacement.

EFFECTIVE STRESS APPROACH

Piles in saturated soils are subjected degradation of stiffness due to progressive buildup of pore water in the surrounding soil as a result of seismic loading. The degradation in the soil springs will depend on the magnitude of pore water pressure and can best obtained by following the effective stress approach.

CONCLUSIONS

Soil-pile behavior is strongly strain dependent. Many attempts have been made to obtain a match between observed and computed pile response. The proposed concept of reduction factors for shear modulus and damping by Jadi (1999) appears reasonable but more research is before this method can used in practice.

Behavior of pile in liquefying soil has beenbetter understood in the last decade but the design practice varies. Considerably more research is needed to refine design methods.

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