Technical Note

SHEAR MODULUS G_s

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A study group of the New Zealand National Society for Earthquake Engineering has recently completed recommendations for the seismic design of storage tanks in a form suitable for use as a code.

A knowledge of site response is an integral part of seismic analysis, unfortunately providing guidleines on assigning relevant soil parameters (shear modulus and damping in particular) cannot easily be resolved in a code format. However, as shear modulus (G_s) is referred to directly in the recommendations, it was decided to provide this technical note to enable some guidelines for its assessment to be given. It is an involved problem which requires a great deal of judgement on the designer's behalf if a realistic value of G_s is to be attained.

Most available data on G_s has been developed for either sands or saturated clays although there has been a limited amount of work done on gravelly soils. Because most soils have curvilinear stress-strain relationships, it will be appreciated that the shear modulus is not constant but is usually expressed as the secant modulus determined for a specific value of shear strain.

SHEAR MODULUS VALUES FOR SANDS

Studies by a number of investigators have shown that the modulus values for sands is strongly influenced by confining pressure (σ'_m) strain amplitude (γ_s) and void ratio (e) (or relative density D_R). It has been found that shear modulus and the confining pressure can be conveniently related by the equation 1 (Seed and Idriss, 1970).

$$G_s = 220\ K_2 . (\sigma_m')^{\frac{1}{2}}\ \text{kPa} \qquad (1)$$

$$\sigma_m' = \left(\frac{\sigma_1' + \sigma_2' + \sigma_3'}{3}\right)$$

The effect of void ratio and strain amplitude are expressed through their influence on the parameter K_2. Figure 1 shows the values of $(K_2)_{max}$ with their attenuation with γ_s for various void ratios.

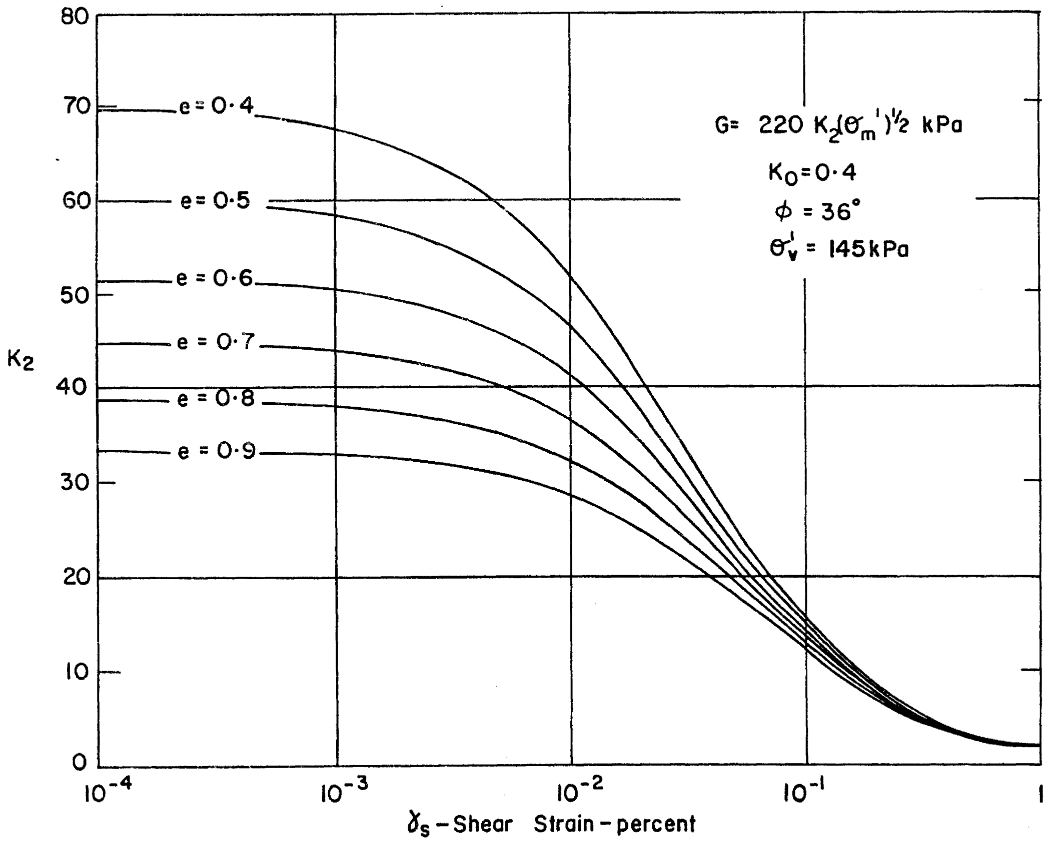

FIG. 1 - SHEAR MODULI OF SANDS AT DIFFERENT VOID RATIOS (after Seed and Idris (1970))

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Seed et al (1986) has described a useful method by which values for the modulus coefficient K_2 at low strain amplitudes (10^{-4}%) can be estimated from the standard penetration resistance of the sand. The value of $(K_2)_{max}$ arrived at from equation 2 can then be used in Figure 1.

$$(K_2)_{max} = 20\ N_{60}^{1/3} \qquad (2)$$

where N_{60} = SPT blow count standardised to an energy ratio of 60%.

It should be noted that equation 2 has two main assumptions inherent in it:

(i) The sand deposit is normally consolidated so that $\sigma_m \approx 0.65\ \sigma_1'$.

(ii) The water table is close to the surface so that σ_1' is calculated using buoyant unit weight.

For GRAVELLY SOILS the limited amount of study done has shown that grain size also has a major influence on K_2. Values of modulus coefficient for gravels are generally greater than for sands at equal relative densities. For relatively dense well graded gravel, $(K_2)_{max}$ will generally be in the range of about 80-180 compared with a range of 55-80 for sand, indicating that the gravel would be a somewhat stiffer deposit. However, the modulus attenuation for gravels are quite similar to those for sands.

The designer must make a decision on representative values of shear strains (γ_s) and confining pressure ($K_o \Rightarrow \sigma_m'$) that will be appropriate to the problem. A soil investigation will provide one of either relative density, void ratio or N_{60} from which $(K_2)_{max}$ can be determined. Using this value with the selected values of γ_s, permits K_2 to be chosen from Figure 1, which together with the value of effective mean principal stress (σ_m'), can then be used in equation 1 to produce a value of G_s. If necessary, the analysis could be iterated to converge on a value of γ_s.

G_s could alternatively be measured directly using a wide variety of field and laboratory tests. However, this approach is not generally recommended for use in design since it is difficult and very expensive.

SHEAR MODULUS FOR SATURATED CLAYS

An accurate determination of the shear modulus for saturated clay is enormously complicated by the effect of sample disturbance and strain amplitude.

Due to sample disturbance, laboratory tests will under-estimate the modulus by a factor of between 2.5 and 5.0. Insitu measurements do eliminate the problems of sample disturbance but no insitu techniques are available that will induce large controlled shear strains in natural deposits.

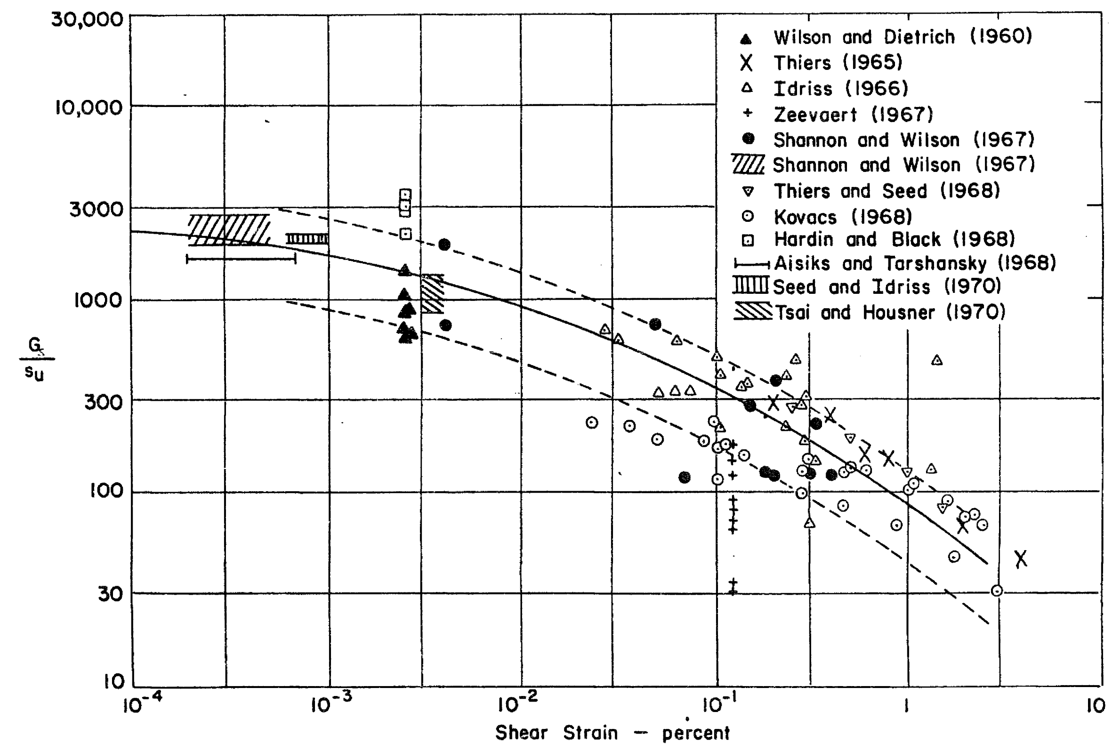

FIG. 2 - IN-SITU SHEAR MODULI FOR SATURATED CLAYS (after Seed and Idriss (1970))

Test data obtained by a large number of investigators has been presented in chart form by normalising G_s with respect to s_u and expressing the relationship G_s/s_u as a function of γ_s. For the test data obtained in the U.U laboratory tests the measured moduli were multiplied by a factor of 2.5 to make an approximate allowance for the effects of sample disturbance. No correction was made to the insitu test data.

Whilst there is considerable scatter in the data, most does fall within ±50% of the average value and thus the average values are likely to provide reasonable estimates of the insitu modulus for many clays given a representative value of undrained shear strength.

CONCLUSION

It is evident that the evaluation of G_s is dependent upon a number of factors, many of which are highly indeterminate in themselves. For a critical facility it would be appropriate to check the sensitivity of the analysis over an appropriate range of values for G_s.

Direct measurement of G_s is not recommended because of the difficulties involved, the expense and doubtful increase in accuracy over the values developed from the relationships presented in this paper.

REFERENCES

1. Seed, H., Bolton and Idriss, I.M. (1970) "Soil Moduli and Damping Factors for Dynamic Response Analysis", Report No. EERC 70-10 (Earthquake Engineering Research Centre, University of California, Berkeley).

2. Seed, H., Bolton, Wong, R.T., Idriss, I.M. (1986): "Moduli and Damping Factors for Dynamic Analysis of Cohesionless Soils", Journal of Geotechnical Engineering, Vol. 112, No. 11, November 1986.