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(*formerly* Journal of Geotechnical Engineering)

tems at several sites. Some of the reductions in first-mode frequency with increases in peak velocity (v_{max}) are clearly related to softening of structural components (e.g., the bearings in base-isolated buildings), while others may be associated with soil nonlinearity (both primary and secondary). By using in our analysis fixed-base structural vibration parameters based on CEM system identification analyses and soil shear wave velocities that incorporate primary nonlinearities, one accounts for a large percentage of the softening evident in the discusser's Figs. 16 and 17. All that is missing is secondary soil nonlinearity, which, as noted above, does not appear to have introduced significant bias.

A comment on the philosophy of these papers may also be relevant here. It would be possible to perform fully nonlinear dynamic finite-element analysis for the structures in the database using procedures such as those reported by Borja et al. (1992). It is expected that such analyses would provide a better fit to the data. However, such findings would be of use to only a small percentage of practitioners. Since SSI is sometimes misunderstood, and is neglected in the design of most buildings in the United States, the writers believed that more benefit would be gained through the use of simple methods that could easily be used by practicing engineers.

Finally, the writers agree with the discusser in stating the limitations of the database, namely, there are a small number of case histories with strong accelerations. Data from recent earthquakes in Taiwan and elsewhere may allow for additional evaluations of SSI effects under stronger levels of shaking.

KINEMATIC INTERACTION

The comments by the discusser point to the need for clarification of our treatment of kinematic interaction. Torsional motion of base slabs, which can result from kinematic interaction, were considered in the analysis as discussed in the following paragraph. Reductions of foundation-level translations at the centroid of the foundation slab from kinematic interaction were neglected, as such effects are typically modest near the fundamental period of most structures.

For buildings with accelerometers located near the building perimeter (both at the base level and roof), analyses of vibration parameters were repeated using different pairs of roof/base instruments to check the significance of torsion on identified parameters for translational response. Variations in identified first-mode vibration parameters for opposite sides of regular buildings were typically small (i.e., similar to random disturbance errors in the identification), and did not significantly affect the identified period lengthening and foundation damping. Variations in identified first-mode parameters were more significant for irregular buildings, but these variations tended to be similar for different cases of base fixity. Hence, even for irregular buildings the variations of period lengthening and foundation damping were typically comparable to random disturbance errors in the identification. It should also be pointed out that, wherever possible, identified parameters were based on the measured response near the building centroid where torsional effects are minimized.

The writers appreciate the discusser's insightful comments on conditions where torsional effects resulting from SSI can significantly contribute to shear in end columns. However, as noted above, the writers do not believe that inertial interaction effects identified from their system identification analyses are significantly biased by torsional responses.

APPENDIX. REFERENCE

Schnabel, P. B. (1973). "Effect of local geology and distance from source on earthquake ground motions." PhD dissertation, University of California, Berkeley, Calif.

PROBABILISTIC EVALUATION OF EARTHQUAKE-INDUCED SLOPE FAILURE[a]

Discussion by Claudio Cherubini,[3] Francesco Santoro,[4] and Marco Stigliano[5]

The paper itself is very interesting even if it seems to be useful to point out some questions.

The first observation concerns the value of σ_F that is equal "at least" to 0.25, according to various authors. Using the PEM technique (Harr 1987), concerning a definite slope (Fig. 3) whose geometric and mechanic characteristics (together with the values of the standard deviation of cohesion and friction angle) are summarized in Table 3, it has been possible to get, in the case of static evaluations, values of standard deviation of safety factor almost equal to 0.068 and to 0.170, respectively, in the case of minimum and maximum variability of c and ϕ.

If one considers the reduction of the variance, in order to take into account the spatial variability, the values of standard deviation resulting from the previously described calculations, which correspond to values of coefficient of variation equal to 20.8% for the cohesion and 13.7% for the friction angle, seems to fit the problem much better [for some values of coefficients of variation of cohesion and of friction angle, see Becker (1996) and Cherubini (1997)].

The most commonly used measure of the amplitude of a particular ground motion is the peak horizontal acceleration (PHA). The last, for a given component of motion, is simply the largest (absolute) value of horizontal acceleration obtained from the acceleration of that component. Generally, PHA refers, as in this case, to bedrock upper limit. The amplification function A is evaluated as the ratio between the amplitude of the free-field acceleration for the studied slope and the amplitude of the acceleration at the bedrock that transmits the seismic action. It follows, therefore, that the free-field PHA is obtained from the product of PHA and A.

The second observation concerns the different values of the reliability index and of the angle θ (the slope of sliding plane, Fig. 3), which characterizes the minimum conditions in the static and dynamic cases.

[a]November 1998, Vol. 124, No. 11, by John T. Christian and Alfredo Urzua (Technical Note 16597).

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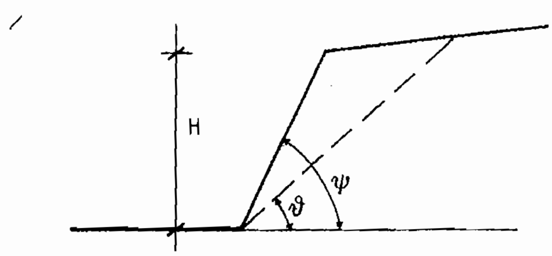

H = 10 m	c = 6 kPa	Amp.factor	A = 1.5
ψ = 35°	φ = 26°	PH.A	= 0.10 g

FIG. 3. Schematic Representation of Studied Slope

TABLE 3. Summary of Data and Results

Number of case (1)	Type (2)	σ_c (kPa) (3)	σ_φ (degrees) (4)	Estimation of F (5)	Standard deviation of F (6)	β_{min} (7)	θ (degrees) (8)
1	Static	0.50	1.2	1.503	0.068	7.367	27.04
1	Dynamic	0.50	1.2	1.091	0.049	1.852	25.35
2	Static	0.75	2.4	1.504	0.123	4.080	26.02
2	Dynamic	0.75	2.4	1.092	0.090	1.026	25.17
3	Static	1.00	3.6	1.509	0.181	2.807	25.71
3	Dynamic	1.00	3.6	1.094	0.131	0.717	25.12
4	Static	1.25	4.8	1.515	0.240	2.145	25.58
4	Dynamic	1.25	4.8	1.097	0.173	0.560	25.09

Examining the results given in Table 3, it appears that β_{min} dramatically decreases, as one could expect, changing from the static case to the dynamic case. The ratio between the static and the dynamic value of β_{min} remains almost constant, changing from 3.97 in case 1 to 3.80 in case 4.

It is important to notice that in case 1, which is characterized by the smallest c and ϕ variability, the failure surfaces are characterized by quite different values of failure angle (θ = 27.04° in the static case; θ = 25.35° in the dynamic case). This difference becomes smaller when the standard deviations σ_c and σ_ϕ increase (cases 2, 3, and 4).

APPENDIX. REFERENCES

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Harr, M. E. (1987). *Reliability-based design in civil engineering*, McGraw-Hill, New York.

Closure by John T. Christian[6] and Alfredo Urzua[7]

The writers thank the discussers for their interest in the technical note and comment as follows on the points raised in their discussion.

The discussers properly observe that the variation in the factor of safety due to the uncertainty in the cohesion and friction alone would usually be less than 0.25, the precise value depending on the values assumed for the parameters for the determinate and indeterminate variables. They also point out that the inclination of the critical failure plane in a homogeneous and isotropic material depends on the values of the other parameters.

There are many more uncertainties for landslides than the strength parameters alone, for as Morgenstern (1997) wrote:

> If the major uncertainty in landslide risk assessment was parameter uncertainty, the use of probabilistic methods in practice would be much more advanced. Unfortunately, the major uncertainty is model uncertainty. . . . Examples of components of model uncertainty include strain-weakening and progressive failure, influence of loosening on the strength of blocky material, problems of characterizing fissured clays, time-dependent softening processes and others associated with drainage, both inflow and outflow.

For Karast terrains the writers would add the heterogeneous characteristics of the limestone, the presence and configuration of solution cavities, and the variable ground-water regime. All in all, the writers believe that the value of 0.25 for the standard deviation of the factor of safety is reasonable and that the value should certainly be larger than that due to strength parameters alone.

It should also be noted that the order of computation in the present case is the reverse of the usual: One has estimates of the rates of failure and computes the corresponding factors of safety. The writers also emphasize that (8) is not limited to the case of the single-plane wedge failure in Fig. 1. It can be derived for the infinite slope case and for a single block on an inclined plane. Similar, but more complicated, relations can be derived for other modes of failure. The point is that one is not interested so much in the details of a particular failure mode as in the effect of the seismic accelerations on the factor of safety.

The exact definition and location of the horizontal acceleration given in code-related reports on seismic hazard are often ambiguous. Particularly in areas of light to moderate seismicity, the acceleration is often taken as the effective value observed on level, firm ground. Geotechnical earthquake engineers, faced with an embankment or slope, would amplify that value, and that is what the factor A represents. The writers do not agree with the discussers that the values of acceleration given by Earth Scientific Consultants (1994) should be taken as representing the value transmitted by the bedrock to the overlying softer rocks.

APPENDIX. REFERENCE

Morgenstern, N. R. (1997). "Toward landslide risk assessment in practice." *Landslide risk assessment*, D. M. Cruden and R. Fell, eds., Balkema, Rotterdam, The Netherlands, 15–23.

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