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Geotechnical design with apparent seismic safety factors well-bellow 1

George Gazetas*, Ioannis Anastasopoulos, Evangelia Garini

National Technical University, Athens, Greece

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ABSTRACT

The paper demonstrates that whereas often in seismic geotechnical design it is not realistically feasible to design with ample factor of safety against failure as is done in static design, an "engineering" apparent seismic factor of safety less than 1 does not imply failure. Examples from slope stability and foundation rocking illustrate the concept. It is also shown that in many cases it may be beneficial to under-design the foundation by accepting substantial uplifting and/or full mobilization of bearing capacity failure mechanisms.

1. Factors of safety in geotechnical engineering

In engineering practice the unavoidable uncertainties (in loads, geometry, methods of analysis) and the associated severe risks from failure dictate the use of factors of safety (FoS), which by definition are greater than 1. In foundation design ample factors of safety (of the order of $2-3$) are imposed on the static loads to avoid bearing capacity failure of shallow and deep foundations.

Historically, in seismic design the factors of safety were somewhat lower (by up to 50%), in view of the small probability of seismic occurrence during the lifetime of the facility. Thus, for foundation bearing capacity, a factor of safety of 2 under seismic conditions was deemed sufficient instead of the traditional 3 under non-seismic loads. In view of the un-realistically small levels of seismic acceleration of times past (seismic coefficients of the order of $0.05-0.15$ prevailed even in regions of very high seismicity), keeping the factors of safety substantial (e.g., ≈ 2) was a prudent, easily satisfied requirement.

With the advent of the accelerograph, the levels of design acceleration increased significantly; this eventually necessitated the adoption of (explicit) factors of safety close to 1 (see for instance EC8-5).

It will be argued in this paper that the nature of the seismic factors of safety (F_E) is fundamentally different from the static F_S, and that accepting seismic "engineering" F_E (well) below 1 may even lead to a safer overall structure.

2. Earthquake engineering: the realm of "capacity design"

Structural earthquake engineering has long ago embraced the philosophy of "capacity design". The main idea is to design the various constituent members of a structure in such a way that members crucial for its stability, the columns, are stronger than the less critical members, the beams; and that the plastification of members should result from exceedance of their moment, not their shear capacity, thus avoiding brittle failures. Hence, against the design motion, flexural yielding is *directed* to take place in beams, dissipating energy without endangering the overall structural safety [1,8,27,32,33].

"Capacity Design" for foundations has taken a slightly different turn: the overturning moment to be carried by the supporting below-ground members is increased over the calculated bending moment capacity of the superstructure (by applying an "overstrength" factor of about $1.3-1.5$). Thus, the "hidden" safety factor utilized in the strength calculation of the concrete cross section is removed. The aim is to ensure that:

- No plastic "hinging" develops below the ground surface; i.e. piles, caps, footings remain structurally nearly elastic
- No mobilization of bearing capacity failure mechanism takes place.

Therefore, since the subsequently utilized explicit seismic factors of safety are kept just above 1, the F_E would be certainly larger than 1. This approach is imposed on foundation design mainly (but not only) because post-seismic inspection and repair below ground is hardly feasible — unlike the above ground structural damage [9]. The past

* Corresponding author. Tel.: +30 210 772 4075, +30 210 772 4076; fax: +30 210 772 2405.
E-mail addresses: gazetas@ath.forthnet.gr, gazetas@central.ntua.gr (G. Gazetas).

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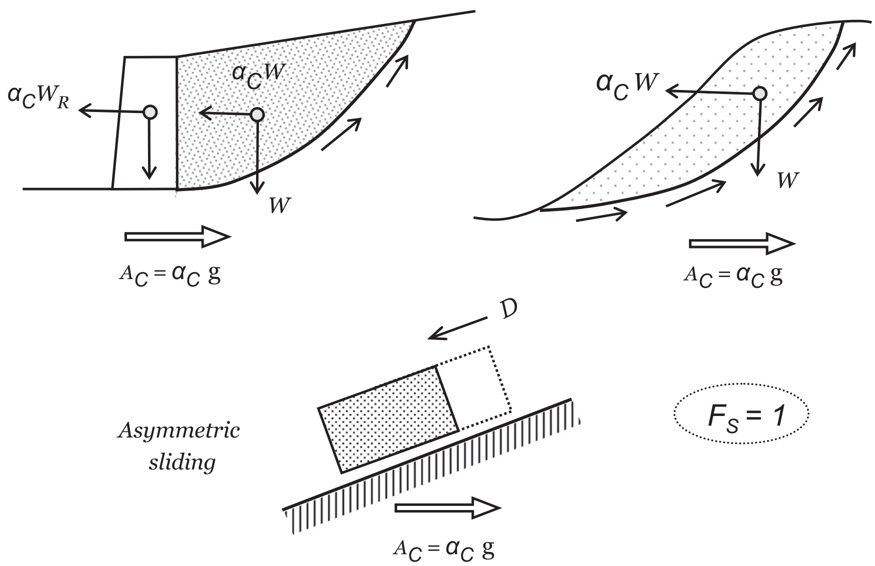

Fig. 1. Schematical configurations of geotechnical structures that can be modeled by a rigid block on top of a sloping plane. Definition of critical pseudostatic acceleration.

argument of greater uncertainty with soils is still being invoked but less convincingly [10,25,26,28,29,31].

3. Why is it not always feasible in geotechnical engineering to achieve FoS > 1?

The levels of acceleration recorded in the last 30 years, with huge values of both peak (ground) acceleration [PGA] and response spectral acceleration [SA] impose a heavy load on foundations, even when the accepted inelasticity (ductility) of the superstructure is large. As examples, we just mention that several records of Kobe (1995) and Northridge (1994) had PGA values exceeding 0.80 g and maximum SA exceeding 2.0 g. Even small magnitude events, e.g. the 1986 San Salvador M_S 5.7, produced peak acceleration of 0.75 g with proportionally large SA values at not-too-short periods. Calling for nearly-elastic response of the soil-foundation system is not only an expensive demand, but also one that in some cases could not be possibly satisfied (as for example when retrofitting and old structure to meet current code requirements). And in any case such a demand is incompatible with the design for high inelastic action (ductility) of the superstructure. After all it is the failure of the superstructure that could have the most severe consequences.

4. Under seismic base excitation FoS < 1 does not imply failure

The factor of safety (FoS) against any type of failure under static permanent loads, denoted hereafter as F_S, must be kept above 1 to avoid failure (actually "well" above 1 to cover uncertainties). Under seismic shaking, FoS is a function of time. Hereafter by seismic factor of safety we mean the *apparent min* FoS(t) *with respect to time*. We will call it "engineering" factor of safety, F_E.

$F_E < 1$ does not necessarily signify failure. For two reasons, that relate to the nature of seismic excitation:

(a) seismic loading is **cyclic** (and, in fact, with rapidly alternating cycles as well)
(b) the triggering seismic motion is an imposed oscillatory displacement at the base, i.e., it is a **kinematic** excitation, not an external "pre-determined" load on the superstructure.

Thanks to (a), the duration of $F_E < 1$ is limited (usually to tenths of a second) and the ensuing displacements are reversed before they reach the point of no return, due to the load reversal. Thanks to (b), the *actual loads* transmitted from the base upward to the critical-to-fail structure are limited by the *actual capacity* of the base of the structure or of the interface separating this structure from the base. In other words, as will be seen below, it is only the apparent "engineering" factor of safety, F_E, that (momentarily) drops below 1.

The consequence of $F_E < 1$ is a *finite* inelastic (permanent) deformation of the system: rotation, horizontal, vertical displacement of foundations, slippage of retaining walls and slope wedges.

4.1. Newmark's sliding block analog

In his seminal 1965 Rankine lecture, Newmark [23] proposed that the seismic performance of earth dams and embankments be evaluated in terms of permanent deformations which occur whenever the inertia forces on a potential slide mass are large enough to overcome the frictional resistance at the "failure" surface. He proposed the analog of a rigid block on inclined plane as a simple way of analytically obtaining approximate estimates of these deformations. Since then, the analog has seen numerous applications and extensions, three of which are shown in Figs. 1 and 2. (See also [30].)

The concept of the pseudo-statically determined "critical" or "yield" acceleration, A_C, is a key of the Newmark-type analysis. Figs. 1 and 2 illustrate the concept with two asymmetric and one symmetric geotechnical problems. In the first two, A_C is the pseudo-static "constant" base acceleration which induces inertia forces (*mass* × A_C) in the system that just lead to sliding failure: $F_S = 1$. In the second application A_C is the "constant" base acceleration that induces inertia forces in the superstructure the overturning moment and shear force of which just lead to a bearing capacity failure: $F_S = 1$ (under eccentric and inclined loading). The asymmetric and symmetric sliding block analogs (with an inclined and a horizontal base) are also shown in the two figures.

Newmark showed that when an embankment or dam is excited by an acceleration of peak amplitude A substantially exceeding the critical acceleration A_C of a prone-to-failure wedge, it will simply