CISM COURSES AND LECTURES NO. 494
INTERNATIONAL CENTRE FOR MECHANICAL SCIENCES

ADVANCED EARTHQUAKE ENGINEERING ANALYSIS

EDITED BY

ALAIN PECKER

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ALAIN PECKER
ECOLE POLYTECHNIQUE, PALAISEAU, FRANCE

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PREFACE

This book contains lecture notes, albeit not covering all materials, delivered at the course on Advanced Earthquake Engineering Analysis that took place at CISM Udine in July 3-7, 2005.

During the last decade, the state of the Art in Earthquake Engineering Design has made significant steps towards a more rationale analysis of structures. Scientists have long recognized that earthquake design is guided by displacements and deformations rather than forces. However due to the historical background of structural engineers in static analyses, effects of earthquake on structures have been viewed as forces acting on the structures. All presently available design building codes are developed along these lines.

Our knowledge of ground motion characteristics, earthquake geotechnical engineering, structural behavior (design and numerical analyses) and model tests have advanced to a point where it is possible to anticipate a significant move from force based design approaches to displacements based design approaches. Although displacement based design approaches constitute the kernel of most research programs, they have not yet been incorporated in the State of Practice.

The purpose of the course was to review the fundamentals of displacement based methods, starting from engineering seismology, earthquake geotechnical engineering, to focus on design, analysis and testing of structures with emphasis on buildings and bridges.

The five main topics presented during the course are detailed below. Each topic started with the fundamentals and then focused on advanced, State of the Art, subjects. The lectures were heavily illustrated with examples drawn from actual projects in which the lecturers are deeply involved.

- Engineering Seismology: measurement and characterisation of seismic motions; prediction of ground motions with empirical attenuation relationships; physics behind earthquake signals; site effects.
- Geotechnical Engineering : non linear soil behavior under cyclic loading ; non linear site response analyses; soil-structure interaction including non linear effects; earthquake resistant design of foundation; performance based design.
- Seismic Analyses: introduction to structural response and computer modelling; overview of linear elastic analysis methods; non linear inelastic analyses; pushover analyses; non linear time history analysis.
- Seismic Design: need for displacement based design; fundamentals for direct displacement based design and assessment; strength and deformation capacity.

- Seismic Testing: introduction to seismic testing; shake table tests; pseudo dynamic tests; centrifuge testing; special topics and illustration.

The coordinator of the course wishes to express his gratitude to P.Y. Bard, G.M Calvi, R. Pinho, N.Priestley and P. Sollogoub for their active participation in the elaboration, preparation and delivery of the course. He and the lecturers are also indebted to Pr. Jean *Salençon* who, as non resident Rector at CISM, suggested the topic of the course and provided several constructive suggestions during its preparation. They also want to express their gratitude to the Secretariat staff of CISM for their efficient handling of administrative matters before and during the course.

CONTENTS

Soil Behaviour under Cyclic Loading

Alain Pecker[1,2]

[1] Solid Mechanics Laboratory, CNRS UMR 7649, Department of Mechanics, Ecole Polytechnique, Palaiseau, France
[2] Géodynamique et Structure, Bagneux, France

Abstract. Fundamental characteristics of soil behaviour during earthquakes are reviewed; field and laboratory evidences of non linearities and energy dissipation mechanisms are presented. Different kinds of soil constitutive models are discussed with special emphasis on the equivalent linear viscoelastic model commonly used in engineering practice.

1 Field Observation of Soil Behaviour during Earthquakes

Field observations made on sites during earthquakes clearly point out the influence of the geotechnical nature of the soil profile on the recorded ground surface motion. This fact has been acknowledged more than twenty years ago with the pioneering work of Seed *et al* (1976) and lead to the consideration, in design practice and in seismic building code, of different spectral shapes to specify the seismic action.

All recent major earthquakes recorded worldwide (Mexico, 1985; Loma Prieta, 1989; Northridge, 1994; Kobe, 1995) have all confirmed the following observation: in general, soft alluvial deposits tend to amplify the incident ground motion, especially in the low frequency range. To illustrate that proposal, let us consider the records from the Loma Prieta earthquake; on rock sites, in the town of San Francisco, all records exhibit a peak acceleration of the order of 0.1g; records from the same earthquake, at the surface of alluvial sites, exhibit peak accelerations two to three times larger (Table 1, AFPS, 1990) and acceleration response spectra with significant amplification in the low frequency range. Since the epicentre of the earthquake is remote from the recording stations (approximately 85 km), this amplification cannot be attributed to a source directivity effect; accordingly, all the recording sites being close to one another (within a radius of a few kilometres), it cannot either be attributed to a path affect between the focus and the site. The only parameter that can affect the ground recorded motion is the geotechnical nature of the soil profile, i.e. the mechanical characteristics of the soil layers close to the ground surface.

However, the previously reported observations, and several others like in Mexico City, must not be misleading: alluvial deposits do not invariably amplify the incident ground motion. Looking again at observations made in San Francisco, but during the 1957 earthquake, it appears that although the recorded accelerations on the rock sites were again of the order of 0.1g (because of the smaller magnitude but closer distance to the town), the recorded accelerations on the soil sites were between 1.5 and 2 times smaller than on the rock (Table 1).

Table 1. Recorded peak ground acceleration in San Francisco

Recording station	Soil profile	Peak ground surface acceleration (g)	
		1957	1989
Golden Gate Park	Rock	0.13	
Market/Guerrero St	Rock	0.12	
State Building	Sand + clayey sand (60 m)	0.10	
Mason/Pine St	Rock	0.10	
Alexander Building	Clayey silt + Sand (45 m)	0.07	0.17
Southern Pacific B.	Soft clay	0.05	0.20
Rincon Hill	Rock	0.10	0.09
Oakland City Hall	Clay, Sand (30 m) + Stiff clay (270 m)	0.04	0.26

The previous observations clearly show that the response of a soil deposit depends on the frequency content of the incident motion: the 1957 event has a smaller magnitude and a closer distance to the recording sites; its frequency content is richer in high frequencies than the larger, far away event. Another factor, which has a significant impact on the ground response, is the level of shaking induced by the earthquake; the higher this level, the larger the strains induced in the ground. These large strains induce a non linear response of the soil as shown in Figure 1. This figure depicts the Fourier amplitude spectra of the main shock (solid line) and of the aftershocks (shaded area), scaled by the same quantity at a rock outcrop, of Loma Prieta earthquake records at Treasure Island. Obviously would the soil profile behave linearly, the amplification with respect to the rock outcrop would be independent of the generating event. In this case smaller amplifications occur for the strongest event, which is consistent with the larger strains induced in the profile creating softening of the soil deposit.

In order to be able to predict such phenomena a deep understanding of the soil behaviour under cyclic loading is mandatory. It can be stated that nowadays, although many aspects still remain to be clarified, our knowledge of soil behaviour has advanced to a point where constitutive modelling can be reliably employed to allow for accurate prediction in engineering practice.

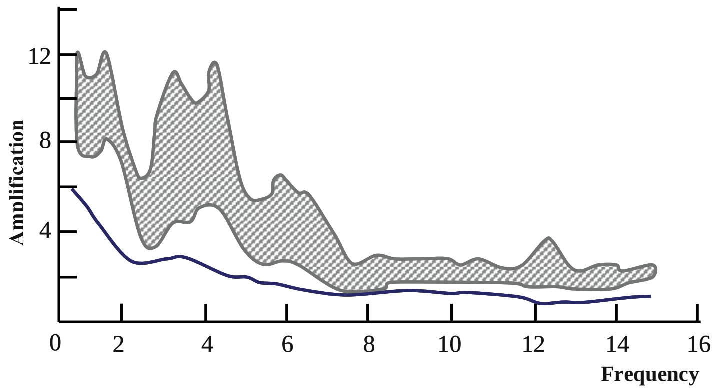


Figure 1. Fourier amplitude spectra of the main shock (solid line) and of the aftershocks (shaded area) of the Loma Prieta records at Treasure Island (after Jarpe *et al*, 1989)

2 Experimental Description of Soil Behaviour

It is commonly admitted for site response analyses, or for soil structure interaction problems, to consider that the seismic horizontal motion is caused by the vertical propagation of horizontally polarized shear waves. Under those conditions, a soil element within the soil profile is subjected to stress cycles similar to those presented in Figure 2.

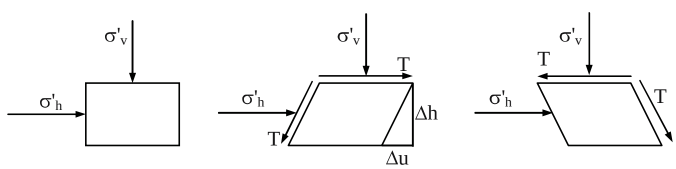
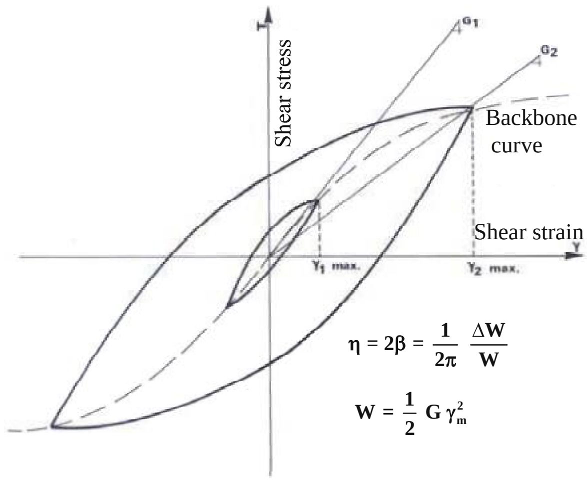


Figure 2. Idealized stress cycle during an earthquake

Initially, for a horizontally layered profile, the soil element is in equilibrium under the vertical effective stress σ'_{v} and the horizontal effective stress $K_0\sigma'_{\mathrm{v}}$ where K_0 is the at rest earth pressure coefficient. When the wave travels trough the soil profile an additional shear stress $\tau(t)$ is superimposed on the horizontal faces of the soil element and, hence on the vertical ones to maintain equilibrium conditions. Under the action of this stress the soil element undergoes a shear strain, which for an elastic material is accompanied by a zero volumetric strain. The shear strain, also called distortion, is defined by (eq.(1)):

$$\gamma = \frac{\Delta u}{\Delta h} \tag{1}$$



$$\eta = 2\beta = \frac{1}{2\pi}\,\frac{\Delta W}{W}$$

$$W = \frac{1}{2}\,G\,\gamma_m^2$$

Figure 3. Shear stress-shear strain curves for constant amplitude cyclic loading

When cycles of constant amplitude are reproduced in the laboratory on a soil sample, the stress strain curves depicted in Figure 3 are obtained. In the (τ,γ) plane the behaviour is characterized by an hysteresis loop, the surface and inclination of which depend on the strain amplitude. The larger the shear strain the wider the hysteresis loop and the flatter it is on the horizontal axis. Furthermore, experimental evidence shows that the shape of the loop is not affected by the loading rate. As soon as the cycles have no longer a constant amplitude, the description of the behaviour becomes more complex. One example is depicted in Figure 4.

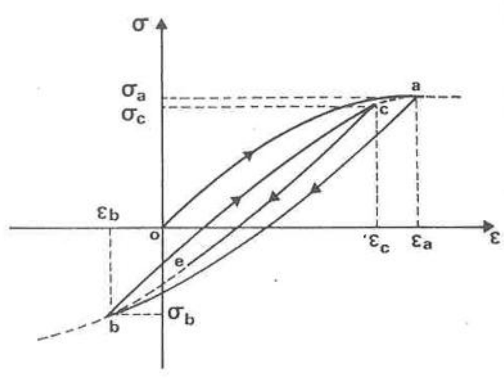


Figure 4. Arbitrary cyclic loading

Until point *b* is reached, the stress strain curve is identical to the one depicted in Figure 3 (first loading curve, also called backbone curve, followed by the unloading curve); at point *b*, such that $\sigma_b < \sigma_a$ the sign of the loading is reversed; the path is given by the curve *bc*, then eventually by *ce* if the loading sign is again reversed at *c*. If, on the contrary, the loading is continued beyond *c* the path is given by *ca* and then follows the backbone curve.

The shear stress strain behaviour described above is accompanied by volumetric strains (Figure 5).

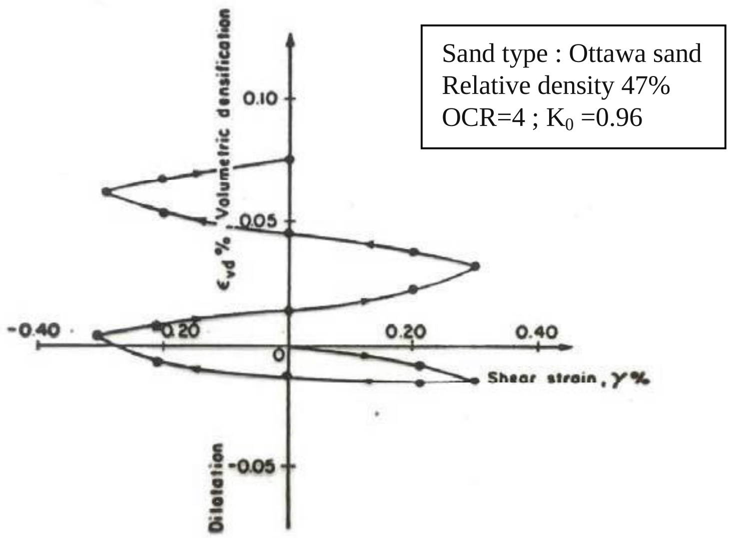


Figure 5. Volumetric strains under cyclic loading

These volumetric strains point out that the behaviour is no longer elastic, even non linear elastic. In a dry material they induce a hardening behaviour. Therefore, even for symmetric closed cycles with identical strain amplitudes, the hysteresis loop measured during, for example, the fourth cycle is different from the loop obtained during the first cycle. The latter is less inclined on the horizontal axis and exhibits a smaller area. For an impervious saturated material, strains occur under constant volume conditions because the pore water does no have time to drain from the sample. The volume change tendency is compensated by a pore pressure build up, hence by a decrease of the effective stresses.

The (over)simplified examples described above illustrate the complexity of soil behaviour, which is highly non linear and inelastic. The constitutive model adopted in practice, and described hereafter, only takes into account the deviatoric behaviour (Figure 3); the volumetric changes are often neglected, at least for soil structure interaction analyses, except in the case of true elasto-plastic constitutive laws.

3 Modelling of Soil Behaviour

A complete description of the behaviour is obtained if, starting from an equilibrium state characterized by a stress field $\underline{\underline{\sigma}}$ and an associated strain field $\underline{\underline{\varepsilon}}$ it is possible, for any strain increment $d\boldsymbol{\varepsilon}$ (or any stress increment $d\underline{\underline{\sigma}}$), to determine the new stress field (respectively strain field) corresponding to a new equilibrium state. In the most general situation, time is also a variable that must be taken into consideration in the constitutive law; however, for most soils, this parameter can be neglected because soils are not rate dependent materials in the range of loading rates induced by earthquakes. The development of a complete constitutive relationship is the ultimate goal of soil modelling; however, in view of the complexity of the behaviour this task constitutes a real challenge and it can be stated that, at the present time, there does not exist a universal constitutive model. Every model available in the literature has its own advantages but also its drawbacks and limitations.

Facing that challenge the earthquake geotechnical engineer often favours, in engineering practice, a more straightforward approach, traditional in soil mechanics. According to this approach, the loading path to which the soil element will be subjected during an earthquake is anticipated and reproduced in the laboratory, or possibly in the field. The parameters measured during those tests are then directly used in the computations. For instance, in soil mechanics, the settlement of a finite thickness compressible clay layer under a wide spread load is studied from a one dimensional compressibility test with zero radial strain.

It must be realized that this approach is not similar to the development of a constitutive model, even if the measured stress-strain curves are represented by mathematical relationships. This kind of modelling remains valid only for the stress paths for which they were established, or for similar stress paths. Its extrapolation to fundamentally different stress paths is erroneous and not permissible. Furthermore, more than often, this approach is only an imperfect modelling of the actual physical phenomena; for example, the equivalent linear visocelastic model does not take into account the volumetric strains (settlements) that the soil experiences under shear loading. In addition, the stress paths duplicated in the experiments represent ideal, somewhat crude, representations of the actual paths. This kind of approach is a good compromise between the actual phenomenon to be modelled and its easy implementation. When used with care it can be a very powerful tool.

Before describing the experimental observations and their mathematical modelling, it is important to realize that, given the time scale of earthquake loading, most soils behave under undrained conditions during the earthquake. The soil permeability is not large enough (with respect to the rate of loading) to allow for drainage and dissipation of excess pore pressures. Consequently, the approach described previously is implemented in terms of total stresses; again, this implementation is an oversimplification of the actual situation as soil behaviour is governed by effective stresses.

Finally, in the rest of the chapter we will restrict ourselves to the description and modelling of the pre-failure behaviour of soils. The modelling of soil behaviour at failure is a matter of specific approaches. When a true constitutive law is available, that distinction is not required; the constitutive model allows for an accurate modelling of soil behaviour from very small strains (quasi elastic behaviour) to very large strains associated with failure.

For a more detailed description of soil behaviour, the reader can refer to Hardin (1978), Pecker (1984) or Prevost (1998).

As evidenced by the experimental observations described in paragraph 2, the soil cannot be modelled with a linear constitutive relationship, at least as soon as strains become significant. The strain thresholds for which non linearities appear are usually very small (10^{-6} to 10^{-4}). It is however fundamental to make a distinction between recoverable, or quasi elastic, strains and irrecoverable strains that develop for larger thresholds (10^{-4} to10^{-3}). The values of these two thresholds, which will be denoted γ_s and γ_v, depend on the nature of the material that can be roughly characterized by its plasticity index (Vucetic, 1984). Table 2 and Figure 6 delineate both domains as well as the mathematical description that can be used in numerical analyses.

Table 2. Strain thresholds for cyclic loading

CYCLIC SHEAR STRAIN AMPLITUDE γ		BEHAVIOUR	ELASTICITY and PLASTICIY	CYCLIC DEGRADATION in STAURATED SOILS	MODELLING
Very small	$0 \leq \gamma \leq \gamma_s$	Practically linear	Practically elastic	Non degradable	Linear
Small	$\gamma_s \leq \gamma \leq \gamma_v$	Non-linear	Moderately elasto-plastic	Practically non-degradable	Equivalent linear
Moderate to large	$\gamma_v \leq \gamma$	Non-linear	Elasto-plastic	Degradable	Non-linear

Strain amplitudes induced by major earthquakes in the European context are capable of creating significant non linearities, and possibly irrecoverable deformations ($\gamma \geq \gamma_s$ or γ_v). As indicated in Table 2 a different behaviour corresponds to each domain and must be characterized by specific constitutive parameters.

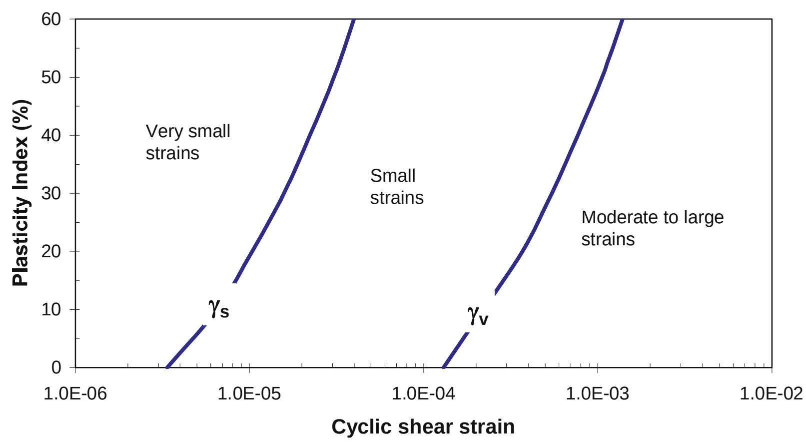


Figure 6. Threshold values for cyclic shear strains

3.1 Mathematical Description for Small Strains $\gamma \leq \gamma_s$: Elastic Model

Typically for strains smaller than $10^{-6} – 10^{-4}$ soils behave elastically. Some saturated materials may however exhibit some viscous energy dissipation. The natural soil constitutive model to use will therefore be the linear elastic, possibly viscoelastic, model. For an isotropic material the shear modulus G (equivalent to the Lamé coefficient μ of continuum mechanics) and the bulk modulus B completely describe the model. Alternatively, one can use the elastic wave propagation velocities V_S (shear wave) and V_P (dilatational wave), which are related to the previous quantities: