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Probabilistic Evaluation of the Adaptation Time for Structures under Seismic Loads

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Abstract

In this paper, a probabilistic approach for the evaluation of the adaptation time for elastic perfectly plastic frames is proposed. The considered load history acting on the structure is defined as a suitable combination of quasi-statical loads and seismic actions. The proposed approach utilizes the Monte Carlo method in order to generate a suitable large number of seismic acceleration histories and for each one the related load combination is defined. Furthermore, for each load combination the related adaptation time is determined, if any, as the optimal one for which the structure is able to shakedown under the unamplified applied actions. A known generalized Ceradini's theorem is utilized. The adaptation time values obtained with reference to all the generated seismic acceleration histories for which the shakedown occurs allows us to define the related cumulative conditioned probability function and, therefore, to identify the optimal adaptation time as the one with a probability not lower than a suitably assigned value.

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1. Introduction

An elastic plastic structure, subjected to time-dependent variable loads, exhibits an elastic shakedown behavior if, after a first (transient) phase characterized by a time interval (adaptation time), during which it can suffer some limited amount of plastic deformations, in the subsequent phase no further plastic strains are generated and it has the ability to eventually respond in a purely elastic manner to any subsequent load condition.

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For an assigned structure and a known load history, the main interest within this topic is the determination of the maximum limit load multiplier for which the structure shakes down. Several methods devoted to the reaching of such a multiplier value have been proposed; they are based on the relevant (static or dynamic) shakedown theory and it has been shown that the obtainable value depends on the adaptation time.

In the present paper, a special approach devoted to the search of the optimal value of the adaptation time is proposed. We will assume that the structure be subjected to a combination of a quasi-static load history and a seismic action. For such a load model, the dynamic shakedown problem is based on the Ceradini's theorem [1,2]. The quasi-static load history will be considered as unknown, time-dependent and acting for the entire lifetime of the structure, although defined within an assigned (deterministic) admissibility load domain. The seismic action will be modelled as a random process, acting in a limited time interval and starting at any instant of the lifetime of the structure. In order to probabilistically describe the seismic load history, reference will be made to a Monte Carlo approach [3], which allow us to generate a sufficiently large number of time acceleration histories. For each realization a related load combination is defined. Making reference to the load combinations for which the structure shakes down, an appropriate minimum adaptation time problem is proposed [4]. The solution to the minimum problem, obtained for all the generated samples, provides the adaptation time cumulative conditioned probability function. The optimal value can be chosen as the one with a probability not lower than a suitably assigned value. Some applications related to plane steel frames conclude the paper.

2. Fundamentals and position of the problem

Let us refer to a plane frame constituted by Euler-Bernoulli beams. The material is modelled as elastic perfectly plastic and the limit behavior of the elements is evaluated at the end cross sections, where rigid perfectly plastic hinges are placed, and it is described by an appropriate piece-wise linearized yield surface, function of the axial force N and of the bending moment M. The actions are represented by nodal loads, by dead loads applied on the beams and by seismic actions. As an example, in Fig. 1a a two spans-three floors frame is represented.

Let us assume that the nodal and the gravity loads, even if they are quasi-static actions, are time dependent too and they permanently act on the structure during its lifetime; on the contrary, the seismic loads act on the structure just for a short time interval randomly situated within the structure's lifetime.

For later use, it is convenient to report the equations governing the purely elastic response of the structure to the quasi-static load history at typical time $t \in (0,+\infty)$:

$$\boldsymbol{d}_s(t) = \boldsymbol{C}\boldsymbol{u}_s(t), \quad \boldsymbol{Q}_s(t) = \boldsymbol{D}\boldsymbol{d}_s(t) + \boldsymbol{Q}_s^*(t), \quad \boldsymbol{C}^T\boldsymbol{Q}_s(t) = \boldsymbol{F}(t), \tag{1}$$

where $\boldsymbol{d}_s(t)$, $\boldsymbol{Q}_s(t)$ and $\boldsymbol{Q}_s^*(t)$ are displacement, generalized stress and perfectly clamped generalized stress vectors evaluated at the beam ends, respectively, $\boldsymbol{u}_s(t)$ is the structure node displacement vector, $\boldsymbol{C}$ is the compatibility matrix, $\boldsymbol{C}^T$ the equilibrium one and $\boldsymbol{D}$ the block diagonal matrix collecting the beam element stiffness. The solution to problem (1) is given by:

$$\boldsymbol{u}_s(t) = \boldsymbol{K}^{(-1)}\boldsymbol{F}^*(t), \quad \boldsymbol{Q}_s(t) = \boldsymbol{D}\boldsymbol{C}\boldsymbol{u}_s(t) + \boldsymbol{Q}_s^*(t) = \boldsymbol{D}\boldsymbol{C}\boldsymbol{K}^{(-1)}\boldsymbol{F}^*(t) + \boldsymbol{Q}_s^*(t), \tag{2}$$

where $\boldsymbol{K} = \boldsymbol{C}^T\boldsymbol{D}\boldsymbol{C}$ is the frame external stiffness square matrix and $\boldsymbol{F}^*(t) = \boldsymbol{F}(t) - \boldsymbol{C}^T\boldsymbol{Q}_s^*(t)$ is the equivalent nodal force vector.

Analogously, the equations governing the purely elastic response of the structure subjected to seismic actions are:

$$\boldsymbol{d}_e(\tau) = \boldsymbol{C}\boldsymbol{u}_e(\tau), \quad \boldsymbol{Q}_e(\tau) = \boldsymbol{D}\boldsymbol{d}_e(\tau) + \boldsymbol{Q}_e^*(\tau), \quad \boldsymbol{M}\ddot{\boldsymbol{u}}_e(\tau) + \boldsymbol{B}\dot{\boldsymbol{u}}_e(\tau) + \boldsymbol{K}\boldsymbol{u}_e(\tau) = -\boldsymbol{M}\boldsymbol{\tau}a_g(\tau), \tag{3}$$

for all $\tau \in (0,T_e)$, being T_e the length of the seismic time history, $\boldsymbol{\tau}$ the influence vector, $a_g(\tau)$ the horizontal ground acceleration and where $\boldsymbol{d}_e(\tau)$, $\boldsymbol{Q}_e(\tau)$ and $\boldsymbol{u}_e(\tau)$ are displacement, generalized stress vectors evaluated at the beam element ends and structure node displacement vector due to the earthquake, respectively. $\boldsymbol{M}$ and $\boldsymbol{B}$ are lumped mass and damping matrices. As usual, the solution to problem (3) can be obtained by means of a modal analysis, well known procedure here skipped for the sake of brevity.

In order to characterize the special acting load combination, we assume that the quasi-static loads are variable and time-dependent, although defined within a given deterministic load domain, while the ground acceleration is a