Engineering Structures 78 (2014) 57–68

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Seismic performance-based design of flexible earth-retaining diaphragm walls

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ARTICLE INFO

Article history:
Available online 10 July 2014

Keywords:
Subgrade reaction
Inelastic Winkler model
Cumulative pressure
Ratcheting
System uncertainty
Ground motion selection
Multiple stripe analysis
Statistical uncertainty

ABSTRACT

Performance-based design differs from conventional design in that target performance is clearly stated and explicitly checked. A faithful application of this design approach requires refined analysis methods to quantify performance and explicit consideration of all relevant uncertainties. The paper presents a design procedure that satisfies these requirements. The procedure is illustrated with reference to flexible earth-retaining diaphragm walls, which represent a challenging test for such a procedure given the influence of dynamic soil–structure interaction on the response in terms of internal forces and deformations. In order to remain affordable and feasible in practice, yet accounting for the aforementioned sources of uncertainties and the important physical aspects of the system response, the procedure is based on small-sample Monte Carlo simulation and a recently developed nonlinear dynamic model of such soil–wall systems. The latter strikes a balance between model setup and analysis time, on one hand, and accuracy on the other, being capable of re-producing the cumulative build-up of soil pressure in cyclic deformation and the accumulation of permanent displacement of the wall, with an overall modeling and analysis effort in the order of few tens of minutes (on a standard consumer-grade laptop). Performance is measured in terms of mean annual rate of exceedance curves from which design values are obtained. [This paper is an extended version of the contribution presented at the mini-symposium on Performance-based design held at the ICOSSAR conference in June 2013, NY.]

1. Introduction

Current formulations of seismic performance-based design (PBD) specify performance requirements as a number of target performance levels associated with predefined mean return periods of the seismic action. Modern seismic design codes incorporating these concepts, such as e.g. the Eurocode 8 [1], do not require an explicit check of the attainment of the target, and, while leaving freedom to the user on the way to accomplish the task, they actually contain a number of “deemed to satisfy” rules that easily become the default choice for any operator. As a result, these formulations do not provide a reliable quantitative measure of the target performance attainment, and most often they result in a generally higher, but definitely non-uniform (over different structures/systems designed to the code) and uncontrolled performance. Given the many uncertainties involved, on both action and system, the only option to control compliance with the performance requirements is to account for them explicitly and provide a probabilistic measure of performance. Implementation of such an approach involves abandoning most of the conventionalism in the current procedures, e.g. by employing more refined analysis tools. The advantage is a more transparent and responsible design, and higher flexibility in choices, as long as target performance is achieved. On the downside, it also entails a more demanding modeling and computational effort, and the risk of a drift towards possible excessive reliance on analysis. The latter risk should be avoided through a thorough education to conceptual design and the development of handy checks for the designer. In general, consistent application of a truly performance-based design approach will require a higher technical background. For what concerns the effort, research of the recent past has made the implementation of this approach a more affordable task, simplifying to the extent of the possible reliability methods, so that they can now be applied in practice with confidence and a background and computational resources within reach of the professional engineer [2,3].

This paper presents an application of these methods, which rely on so-called small-sample simulation, conditional on a ground motion intensity measure (IM), to the PBD of flexible earth-retaining structures. These structures represent a challenging test case since

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http://dx.doi.org/10.1016/j.engstruct.2014.06.027

soil–structure interaction (SSI) phenomena and lack of symmetry play an important role in their response for both static and seismic conditions. A crucial aspect is the so-called ratcheting [4], or progressive build-up of soil pressures behind the wall with associated increase in internal forces and displacements.

Results are provided in terms of mean annual frequency (MAF) of exceedance of predefined response thresholds. From these curves design values are derived which duly account for all uncertainties, in action and system. The usually time-consuming task of deriving these curves through inelastic response history analysis (IRHA) is made affordable employing an inelastic dynamic model of these soil–structure systems developed previously [5], which describes all relevant physical aspects of their response.

2. Theory

2.1. Model of soil–wall system

2.1.1. Introduction

Seismic analysis of earth-retaining structures can be carried out in principle with satisfactory accuracy with inelastic large-displacement finite element or difference formulations (implemented e.g. in codes like DYNAFLOW or FLAC). The drawback of these tools is that they require advanced competence and time for both model setup and analysis [6,7]. At the other side of the spectrum, state of the practice has not evolved much from the pseudo-static methods devised by Mononobe–Okabe [8,9], as modified e.g. by Seed and Whitman [10], to approximately account for wall flexibility. Elastic or visco-elastic solutions have been proposed [11,12], which albeit idealized have shed light on important aspects of the dynamic response such as the soil dynamic response and the relative flexibility of soil and wall.

For flexible diaphragm walls SSI becomes even more important, making the application of pseudo-static methods less reliable. Alternatives that are mid-way between FEM–FDM and limit analysis have found their way in professional practice, as reviewed e.g. in Faccioli and Paolucci [13]: in these approaches usually the seismic action is described as a predefined distribution of static forces and interaction is modeled by the subgrade reaction method (SRM).

In this paper, in view of the small-sample simulation procedure adopted to account for uncertainties, use is made of 1D inelastic dynamic model developed in [5], herein extended to account for construction stages. The model, shown in Fig. 1, has four building blocks, described in the following sections: a 1D inelastic shear soil column to reproduce free-field motion traveling from the bedrock to the surface; a soil–wall interface through which seismic motion is imposed to the structure; structural elements for the wall and the tie-back(s); a compliant base where motion is input to the system allowing for energy radiation. The figure reports numerical values for some of the input data in the application presented in Section 3. The discretization shown is coarse for clarity of illustration; in the application it is much finer. The model captures most relevant physical aspects of the response, such as the cumulative build-up of soil pressure on the wall sides, and associated permanent inelastic displacements but, notably, since it describes only a one-dimensional displacement field, it cannot give account of vertical displacement such as soil settlement behind the wall. As a consequence, the effect of soil–wall friction is only captured indirectly through the modification of pressure coefficients.

The model is developed within the open-source analysis package Opensees [14].

2.1.2. Soil: free-field motion (1D soil column)

A 1D soil column extends from the bedrock to the surface. This shear column is the typical scheme employed in 1D site response analysis. Current state of the practice employs equivalent linear formulation in the frequency domain [15], where soil stiffness and damping as a function of effective maximum shear strain are described by so-called modulus-reduction curves, of experimental derivation (implemented in computer codes such as SHAKE or EERA).

On the other hand, the limit of the equivalent linear approach is particularly severe in the particular case of interest here: residual displacements and progressive build-up of earth pressures cannot be predicted. For this reason an inelastic model in the time domain is the only option. Hysteretic laws that can be employed in this case can be as simple as a monotonic envelope, such as the hyperbolic model or a piece-wise model derived from modulus-degradation curves, to which an unload-reload rule, such as e.g. the Masing criterion, is attached [16]. This approach, however, is well known to under-predict response at higher intensities, since it overpredicts energy dissipation at medium-large deformations. More refined models that overcome this problem are by now available, such as the multi-surface plasticity at the base of the Cyclic-1D software [17] or a modified version of the Bouc–Wen (BW) law [18,19] employed in the code NL-DYAS [20,21], which yield substantially equivalent results [21].

In this paper the soil shear column is modeled as a spring-mass system, with masses equal to $m = \rho A \Delta z$, where A is the column cross-section area and Δz the layer thickness. Springs are described by the *two node link* element in Opensees. The force–deformation relationship in the shear degree of freedom is modeled with the plain BW law (not the modified version by Gerolymos and Gazetas [20]) implemented in the uniaxial material *BoucWen*:

$$f = \alpha k u + (1-\alpha) f_y \zeta \tag{1}$$

$$u_y \dot{\zeta} = \begin{cases} \dot{u}(1-|\zeta|^n) & \text{if } \zeta\dot{u} > 0 \\ \dot{u} & \text{otherwise} \end{cases} \tag{2}$$

where f is the resisting force, product of the column area and the shear stress, u is the deformation and ζ is an internal hysteretic variable governed by Eq. (2). The model has four parameters: initial stiffness k, hardening ratio α, yield deformation u_y and the exponent n, which regulates the sharpness of the transition between the elastic and post-elastic range. Eq. (2) is obtained from the classical one:

$$u_y \dot{\zeta} = \dot{u}\left[1 - |\zeta|^n (\beta + \gamma \operatorname{sgn}(\zeta \dot{u}))\right] \tag{3}$$

by setting $\beta = \gamma = 0.5$, which corresponds to the Masing unload–reload criterion, as shown in [21]. The corresponding limitation (over-estimation of damping) will be easily removed by implementing the modified version of Bouc-Wen by Gerolymos e Gazetas [20] within Opensees, but this is outside the scope of this contribution and does not influence its generality.

Natural soil deposits exhibit variations of strength and stiffness (which are related) with the confining pressure $\sigma'_0(z)$, as a function of their plasticity index PI. Coarse grained soils with low PI are characterized by strength and stiffness that increase with depth, while fine grained ones with higher PI tend to have both strength and stiffness with no or lower variability with depth [22]. The model therefore allows for depth-dependent values in input for k and f_y:

$$k(z) = G_0(z) \frac{A}{\Delta z} \tag{4}$$

$$f_y(z) = A \tau_y(z) \tag{5}$$

where G_0 is the low-strain value of the soil shear modulus and τ_y the shear strength of soil, all at depth z.

The stiffness G_0 can be obtained either from correlations with SPT/CPT data or as a function of mass density and shear-wave velocity $G_0(z) = \rho V_{s0}(z)^2$ (in both cases important model uncertainties in