Abstract: In this paper, we evaluate the additive effects of topography, soil nonlinearity, and soil-structure interaction (SSI) along the crest of an idealized 40 m high cliff-type topographic feature with slope inclination 30°, where excessive damage was observed during the Athens 1999 earthquake. The objective of this paper is to investigate the relative contribution of topographic amplification, and kinematic SSI as a function of the incident motion frequency content and geotechnical site conditions for a surface and an embedded structure located at the cliff crest. For this purpose, we perform elastic parametric and nonlinear site-specific two-dimensional finite element simulations using three profiles and six input motions. We illustrate the role of SSI in altering the response at the location of peak topographic amplification potential behind the crest, the effects of incident motion incoherency on the transient structural response, and the beneficial contribution of structural embedment. We finally suggest that empirical models for base-slab averaging of shallow foundations, developed as a function of site conditions, structural dimensions and center line location, could be combined with topographic amplification factors to predict realistic design spectra for structures located on irregular topographic features.

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Introduction

During the Ms 5.9 Athens earthquake, an event that resulted in the loss of 150 lives, the collapse of 100 residential and industrial buildings, and the severe damage of another 6,000, the small community of Adàmes suffered extensive structural damage. Despite its 8–10 km distance from the projection of the causative fault and the rather uniform quality of local buildings (comprising two- to four-story concrete reinforced structures built in the 1970s and early 1980s), the modified mercalli intensity (MMI) in the 1,200 m long and 300 m wide town ranged from VII, to VIII, by contrast to numerous other towns located at equal or smaller distances from the source where the motion intensity did not exceed a mere VII.

The location of the town next to the crest of the Kifissos canyon and the observed high damage intensity brought forward topography effects to justify the macroseismic observations. Indeed, the aggravation of seismic motion in the vicinity of irregular topographic features has been the focus of multiple studies in the past, and has been verified based on observational evidence, as well as analytical and numerical simulations. Among others, Celebi (1987) and Pedersen et al. (1994) quantified directly crest-to-base amplification ratios using dense instrumented arrays, Boore (1972) modeled the effects of a ridge using finite differences, Smith (1975) utilized finite-element methods, Sanchez-Sesma and Campillo (1991) used boundary-element methods, and Bouchon (1973) and Bard (1982) used a discrete-wave number method to simulate topographic wave diffraction. Published results on the response of more complex configurations include Bard and Tucker (1985), who investigated the antiplane response of a ridge chain with irregular subsurface layering, and Sánchez-Sesma (1983) and Bouchon et al. (1995) who investigated the response of three-dimensional homogeneous ridges, Geli et al. (1988) who evaluated the effects of compositional layering and complex topography, Deng (1991) who developed a numerical model in which the geometry and geology of the configuration are simulated, and Ashford et al. (1997) who used this numerical model to illustrate the significance of steep slopes on site amplification. More recent publications on semi-analytical and numerical techniques and their application for studies of topographic amplification of seismic motion include the work of Ortiz-Alemán et al. (1998), Paolucci (1999), Hayashi et al. (2001), Savage (2004), and Fu (2005).

Along the crest of the Kifissos canyon, however, the observed damage distribution was nonuniform despite the two-dimensional nature of the feature and the rather uniform structural quality characterizing the area. In particular, in the parallel to the river axis direction, the damage was concentrated in two zones, one next to the crest and one at a distance about 200–300 m from it. Some scattered—yet less intense—damage was observed at intermediate locations. It seems, therefore, that focusing of seismic energy at the vertex certainly played a significant role, but was not the only phenomenon involved. Prompted by the observational evidence, Gazetas et al. (2002), Assimaki and Gazetas...
(2004), Assimaki (2004), and Assimaki et al. (2005a,b) predicted quantitatively the observed damage distribution, and illustrated the additive role of topography, stratigraphy, soil heterogeneity, and material nonlinearity in aggravating the seismic input motion in the vicinity of the crest.

This paper is a continuation of these studies, illustrating how the free-field motion near the vertex was further modified on account of soil-structure interaction (SSI). Observational evidence on the additive role of SSI, soil layering, and material nonlinearity for single-faced slope during the Coalinga Earthquake has already been illustrated by Stewart and Sholtis (2005), who used theoretical (Elsabee and Murray 1977) and empirical (Kim and Stewart 2003) models to decouple topography and SSI effects for a structure located at the base of a similar topographic feature. Through a series of elastic parametric and nonlinear site-specific two-dimensional finite-element simulations, we explicitly investigate here the relative contribution of topographic amplification and kinematic SSI as a function of the incident motion frequency content and geotechnical site conditions for a surface and an embedded structure located at the cliff crest. We illustrate the role of SSI in altering the response at the location of peak topographic amplification potential behind the crest, the effects of incident motion incoherency on the transient structural response, and the beneficial contribution of structural embedment. We finally suggest that empirical models for base-slab averaging of shallow foundations, developed as a function of site conditions, structural dimensions, and center line location, could be combined with topographic amplification factors to predict realistic design spectra for structures located on irregular topographic features.

Site and Ground Motion Characteristics

A typical cross section of the Kifissos canyon is shown in Fig. 1(a), along with the slightly idealized geometry used for the purpose of this study. The site conditions in the vicinity of the Kifissos canyon were investigated at ten boreholes by means of standard penetration blow count ($N_{SPT}$) measurements and laboratory testing for the definition of the variation of plasticity index ($I_p$) with depth. Eight of these were performed down to a depth of about 35 m and two reached almost 80 m. Some indirect evidence for greater depths was "extrapolated" from two 150 m deep boreholes drilled for the under-construction Olympic Village, 1.5 km west to northwest of Adàmes. The overall picture emerging from this investigation is shown in Fig. 1(b), where low-strain shear-wave velocity profiles are constructed for three characteristic locations in Adàmes, referred to in the ensuing as Profiles A, B, and C. All profiles comprise alternating soil layers of silty-gravely sands and sandy-gravely clays in the top 20–30 m. The approximate average velocity $V_{s,30}$ of the 30 m surface soil layers for the three profiles are: 500 m/s for Profile A, 400 m/s for Profile B, and 340 m/s for Profile C, indicative of very stiff (Profile A), just stiff (Profile B), and moderately stiff (Profile C) soil formations according to the European Seismic Code (EC8 2000). These profiles are used in the ensuing to illustrate topography-soil-structure interaction effects on the cliff crest of the Kifissos canyon during the Athens 1999 earthquake.

The strong motion time histories used in the numerical simulations were selected from recordings at 35 recording stations where local soil conditions or adjacent structures could have altered the bedrock input motion (Gazetas et al. 2002). In addition, two historic time histories were included in the simulated ground-motions, to account for forward-directivity effects that must have affected ground motion in the town of Adàmes (lying in front of the rupture zone) and were not represented in the aforementioned time histories (recorded perpendicular to the rupture zone within a narrow region located 10 km away): the 1966 Ms 5.6 Parkfield, CA earthquake recorded at the Cholame...
Shandon No. 8 and Temblor stations. The ensemble of response spectra corresponding to the event-specific and long-period near-fault pulses are shown in Fig. 1(c). These acceleration time histories formulate an input motion database that represents the general strong motion characteristics of the Athens event, and possible directivity effects anticipated at the area of interest; for a complete description of the selection criteria and deconvolution process of the event specific characteristic motions described above and used for the purpose of this study, the reader is referred to Assimaki et al. (2005b). It should also be noted that in the simulations presented in the ensuing, a series of linear elastic parametric analyses using narrowband (Ricker) wave forms is initially conducted, opting to identify the relative contribution of the multitude of frequency-dependent mechanisms in the observed response, and followed by the ensemble of nonlinear transient analyses performed using the aforementioned database of broadband time histories.

**Finite-Element Model Discretization and Boundary Conditions**

The effects of topography, incident wave characteristics, soil nonlinearity, and soil-structure interaction are herein investigated by means of finite-element simulations. Despite the fact that numerous methods have been developed solely for the analysis of site effects—usually in the frequency domain—rendering the algorithms more efficient in terms of the associated computational time, they all imply elastic behavior of the material. Since the finite-element method allows the simulation of inelastic soil response, which will be accounted for in the ensuing, it has been selected for consistency throughout the study.

A schematic representation of the numerical model used in our investigation is shown in Fig. 2(b). In particular, the finite-element discretized model of the 2D problem consists of four-node quadrilateral and three-node triangular elements, the size of which is selected according to the frequency content of the incident motion and the shear-wave velocity of the medium considered, for the effective representation of the propagating wavelengths.

Successively, the input motion is prescribed in the form of effective forcing functions at the base and the lateral boundaries of the model, whereas spurious reflections from the boundaries are avoided by placing absorbing elements around the simulated domain. Thus, the seismic excitation is prescribed directly on the area of interest by means of time-dependent forces, which, for the lateral boundaries are evaluated from the 1D response of the corresponding soil columns. The difference between the 1D motion and 2D response in the far field is actually the scattered energy of the system, which propagates outwards from the irregularity, and is absorbed by the artificial boundaries.

In particular, for both elastic and nonlinear simulations described in the ensuing, the configuration is subjected to in-plane vertically-incident SV waves with amplitude \( I(t) \), prescribed at the base of the finite-element model as tractive time histories of the following form:

\[
\tau(t) = -\rho_a V_{sh} \dot{2}I(t) - \dot{u}(t)
\]

where \( \rho_a, V_{sh} \) correspondingly, the density and shear-wave velocity of the half-space underlying the finite-element discretized domain, \( \dot{u}(t) \) total displacement time history that results from the superposition of upgoing and downgoing waves at the base of the numerical model, i.e., the simulated soil-half-space interface. For 1D conditions (loading and geometric configurations), tractions prescribed at the base of the finite-element model in accordance with Eq. (1)—typically referred to as transmitting boundaries—ensure that downgoing seismic waves reflected from the surface are absorbed exactly at the base of the numerical domain; thus, reproducing the radiation damping effects of the underlying half-space. For 2D geometric configurations, however, diffraction and incidence of seismic energy at inclined interfaces or boundaries give rise to nonvertically propagating waves. In this case, multi-directional reflections of downgoing waves at the base of the numerical model cannot be equilibrated by prescribed tractions in the form of Eq. (1); in order for the scattered energy to be minimized, the soil-half-space interface of the computational domain needs to be defined at a distance equal to multiple times (typically on the order of four to five times) the dimensions of the irregularity that gives rise to wave diffraction near the surface.

The evaluation of effective forcing functions (also referred to as consistent boundary conditions) prescribed at the boundaries of the truncated numerical domain is based on a substructure approach (Kausel and Roesset 1975). According to this method, the free-field vibration problem can be decomposed into substructures (the far field and the 2D irregular topographic configuration), as shown schematically in Fig. 2(c) (note that the structure in this case represents any irregularity that alters the 1D nature of vertically propagating incident shear waves).

Since the excitation is identical in the far-field (top) and near-field (bottom) problems, differences in the interface displacements \( \Delta U = U_b - U_b^c \) are attributed to differences in the interface stresses \( \Delta S = S_b - S_b^c \). Therefore, subjecting the far field to forces equal to \( \Delta S \) (in absence of seismic excitation), displacements \( \Delta U \) will be produced, such that \( \Delta S \approx X \cdot \Delta U \), where \( X \) = frequency-dependent dynamic impedance matrix of the far field, i.e., the stiffness of the far field as “seen” by the interface. Substituting the forces and displacement differences at the boundaries, the equation of equilibrium for the near-field problem is formulated as follows:

\[
-S_b = -XU_b + XU_b^c - S_b^c
\]

Denoting with sub-index \( a \) the degrees of freedom associated with the near-field problem, and sub-index \( b \) the degrees of freedom associated with the far field, the equations of motion for the 2D configuration are given in the partitioned matrix form as follows:

\[
\begin{bmatrix}
\tilde{K}_{aa} & \tilde{K}_{ab} \\
\tilde{K}_{ba} & \tilde{K}_{bb}
\end{bmatrix}
\begin{bmatrix}
U_a \\
U_b
\end{bmatrix}
= \begin{bmatrix}
0 \\
-S_b
\end{bmatrix}
\]

(3)

where the stiffness matrix is frequency dependent. Substituting Eq. (2) in Eq. (3), one obtains

\[
\begin{bmatrix}
\tilde{K}_{aa} & \tilde{K}_{ab} + X \\
\tilde{K}_{ba} & \tilde{K}_{bb} + X
\end{bmatrix}
\begin{bmatrix}
U_a \\
U_b
\end{bmatrix}
= \begin{bmatrix}
0 \\
XU_b^c - S_b^c
\end{bmatrix}
\]

(4)

The infinite domain of the far-field problem is, therefore, truncated by substitution with dashpots (i.e., infinite 1D elements), while the equivalent spring stiffness implied by \( X \) vanishes for the infinite domain. Finally, the stresses \( X \cdot U_b^c - S_b^c \) correspond to the far-field motion, and describe the effective forces that need to be applied on the lateral boundaries of the 2D configuration to successfully simulate the effects of the far-field truncated domain.

For the 2D wave-propagation problem investigated in this paper, the far-field motion corresponds to the response of a one-
dimensional soil column (different for the far field behind the crest and in front of the toe) to the incident motion that will be prescribed at the base of the two-dimensional configuration. Successively, the effective forces to be applied on the lateral boundaries of the two-dimensional model are determined as follows for the case of SV wave vertical incidence (Fig. 2(d)): (a) $S_b^c$ corresponds to the vertical reaction preventing vertical motion at the far-field boundary; and (b) $X \cdot \dot{U}_b^c$ is the product of the far-field response and far-field impedance, expressed as $X \cdot \dot{U}_b^c = V_p \cdot \dot{U}_b^c$, where $\dot{U}_b^c$ = velocity time history at the 1D column nodes, and $V_p$ = P-wave velocity at the corresponding location.

It should be noted, however, that the substructure approach described above is based on the principle of superposition and is, thus, exact only to linear elastic problems. Upon the incidence of strong seismic motion associated with nonlinear, hysteretic mate-
schematic illustration in Fig. 2. The center line of the simulated structure is located at a distance $x/\lambda_0 = 0.2$ from the crest ($\lambda_0$ is the dominant propagating wavelength of Ricker wavelets), where peak topographic aggravation has been determined for the free-field response of a homogeneous half-space (Ashford et al. 1997; Assimaki et al. 2005a). Note that the two-dimensional ground surface response of the topographic configuration, in absence of the structure, is referred to as free-field motion, not to be confused with the far-field motion that is defined where purely vertical seismic wave propagation is assumed.

The dimensionless parameters that govern the response of this system are: (1) the horizontal dimension of the structure $L$ normalized by the dominant propagating wavelength ($L/\lambda_0$); and (2) the structure-soil impedance ratio $r=(pV_s)_m/(pV_s)_\text{soil}$ (where $p=$ density, and $V_s=$ shear-wave velocity of the material), which controls the amount of energy reflected at their interface. The parametric numerical models investigated are summarized in Table 1.

Results are shown in the following figures for the time-domain and frequency-domain characteristics of the response behind the crest and at the base of the cliff, as a function of the frequency characteristics of the incident motion and the material stiffness of the structure. In particular:

1. Fig. 3 shows the spatial distribution of peak-surface horizontal and vertical acceleration normalized by the peak far-field acceleration behind the crest; the normalized response is illustrated for the alternative values of structure-soil impedance ratio ($r$) considered and for wavelengths $D/\lambda_0=1.25$ (top) and 0.375 (bottom) (i.e., dominant propagating wavelengths approximately equal and 2.5 times longer than the horizontal dimension of the structure, respectively). More specifically, the former represents a wave form that may encompass the geometric constraints imposed by the rigidity of the foundation, a fact readily verified by the reduction of peak 2D response at the location of the structure and aggravation of the response in the region between the edge of the structure and the crest ($\max(a_{2D}/a_{\text{ffc}})=2.2$ versus $\max(a_{2D}/a_{\text{ffc}})=1.4$ estimated for the free-field problem, and $\max(a_{2D}/a_{\text{ffc}})=1.5$ versus $\max(a_{2D}/a_{\text{ffc}})=0.8$ estimated for the free-field problem in the vicinity of the crest). On the contrary, the latter represents a long pulse whose shape is minimally affected by the presence of the structure; hence, the topographic amplification of horizontal and vertical acceleration behind the crest is of the same order of magnitude as estimated for the 2D configuration in absence of the structure (see Assimaki et al. 2005b).

2. Fig. 4 shows seismograph synthetics of the horizontal and vertical surface acceleration components, for the free-field and stiff ($r=5.0$) structure investigated, subjected to a train of incident Ricker pulses with dominant wavelengths $D/\lambda_0=1.25, 0.625, \text{ and } 0.375$. Note that the traces of illustrated seismogram synthetics correspond to acceleration time histories computed at distinct points along the surface of the 2D configuration, and plotted on a common absolute time axis. The regions $d_1$ and $d_2$ on the right illustrate the altering
Fig. 3. Spatial distribution of horizontal (a); vertical (b) peak ground surface acceleration, for $D/\lambda_0=1.25$ (top) and $D/\lambda_0=0.375$ (bottom), as a function of the structure-soil impedance ratio $r$. Note that $a_{2D,h}^H$=horizontal 2D peak ground acceleration; $a_{2D,v}^V$=vertical 2D parasitic acceleration at ground surface; and $a_{fc}$ is the peak far-field acceleration behind the crest.

Fig. 4. Seismogram synthetics of surface horizontal (top) and vertical (bottom) acceleration, illustrating the altering of topographic amplification of seismic motion due to kinematic soil-structure interaction effects behind the crest of a single-phased slope: response of the topographic irregularity in the absence of structure $r=0$ (left) and response of the topography-soil-structure configuration for structure-soil impedance contrast $r=5.0$ (right), subjected to a train of vertically propagating SV-Ricker waves with $D/\lambda_0=0.325$, 0.675, and 1.25.
of the horizontal and parasitic vertical wave field due to the
presence of the structure, an effect more pronounced upon
the incidence of the last pulse, whose wavelength is compa-
rable to the horizontal dimensions of the structure.

3. Vertical acceleration time histories recorded at the left corner
(Point A), the right corner (Point B), and the center line of the
structure for \( r = 0.5 \) and 5.0 are next plotted in Fig. 5.
Note that the observed phase differences of structural re-
sponse along the soil-structure interface correspond to the
rocking motion imposed by diffracted up-slope-propagating
surface waves (see Assimaki and Gazetas 2004), while both
the wave form amplitude and phase difference are consid-
erably affected by the soil-structure impedance contrast [e.g.,
\( a_{2D}(r=0.5) = 1.5 a_{2D}(r=0.5) \) at Point A].

4. Finally, kinematic soil-structure interaction effects in the
frequency domain are illustrated for a broadband Ricker
wavelet series comprising pulses with \( L/\lambda_0 = 0.375, 0.625, \)
and 1.25. Fig. 6 plots the crest-to-base, center line-to-far-
field, and crest-to-far-field transfer functions for the different
soil-structure impedances (\( r \)) investigated, as well as for the
free field (\( r = 0 \)). As can be readily seen, kinematic interac-
tion effects are more pronounced in the frequency region
that corresponds to wavelengths comparable to the horizontal di-
ensions of the structure (e.g., this frequency region is in the
vicinity of \( f = 10 \) Hz for \( V_s = 200 \) m/s and \( D = 20 \) m).
Nonethe-
less, altering of topographic amplification due to kine-
matic SSI is also observed in the low-frequency region
\( (f \in [0.5 \, 1.5] \) Hz), both in terms of frequency content and
amplitude of the transfer function.

The elastic parametric investigation illustrates clearly that the
presence of a structure at the location of peak topographic ampli-
fication affects the time- and frequency-domain characteristics of
the response, not only at the location of the structure, but also
along the level surface behind the crest and along the slope to-
ward the cliff toe. The observed topography-soil-structure effects
are frequency dependent, and are maximized for wave lengths
comparable, or shorter, than the horizontal dimension of the struc-
ture, which encompasses the geometric constraints imposed by
the rigidity of the foundation. The rocking structural motion is
more pronounced for soft structures founded at the location of
peak amplification potential behind the crest—and results in
aggravation of the free-field parasitic acceleration component—
while base-slab averaging effects reduce its intensity as the rela-
tive structural stiffness increases. Finally, surface waves that
travel toward the crest and comprise the incoherent incident mo-
tion that results in rocking response of the structure, are subse-
quently backwards scattered towards the toe and destructively
interfere with the low-frequency components of incident motion
\( (2-5 \) Hz), while enhancing the short incident wavelengths that
correspond to the lateral structural dimension \( (10-15 \) Hz); note
that the backscattered energy emanating from the structure also
results in the observed reduction of relative amplitude between
the crest and the far field (see Fig. 6). The ensemble of these
phenomena is clearly illustrated in the time domain by Fig. 4,
which depicts the incident energy distribution in space, wave
front and particle-motion direction (in-plane horizontal versus
vertical) for the stiff structure case \( (r=5.0) \) investigated. For fur-
ther details on the combined effects of kinematic SSI and topog-
ographic amplification, the reader is referred to Assimaki (2004).
In the ensuing, we shall investigate in detail the effects of kine-
matic and inertial SSI combined with the free-field topographic
amplification and strong motion site response by means of dy-
namic nonlinear simulations.

**Nonlinear Soil-Structure Interaction on the Cliff Crest**

It is well known that hysteresis, rate dependence, and irreversible
deformations are dominant features of cyclic soil behavior. In
particular, for the case of repeated loading on structures, typical
effects comprise the formation of a yielding zone and associated
inelastic deformation beneath the foundation. For the case of a
half-space, this phenomenon results in dynamic inhomogeneity
within an otherwise elastic medium, which tends to reduce the
effective dynamic impedance (radiation damping) of the semi-
infinite domain and create resonant frequencies at which the
structural motion is amplified (Borja et al. 1993, 1994). This ob-
servation is consistent with known results for the problem of
finite-size foundations on a uniform layer over bedrock, and the
problem of finite-size foundations on a finite region embedded in
a half-space.

On the other hand, the soil response to relatively strong seismic
motion is usually associated with hysteretic deformations—
dominant features of the cyclic soil behavior. Since material
yielding is expected to be initiated by seismic wave propagation,
the response of a structure on a homogeneous soil profile that
exhibits instantaneous strain softening is expected to resemble the
The multiyield plasticity constitutive soil model is a kinematic hardening model based on a relatively simple plasticity theory (Prévoz 1995) and is applicable to both cohesive and cohesionless soils. The formulation of the mathematical model follows the concept of a “field of work-hardening moduli” (Iwan 1967; Mróz 1967; Prévoz 1977), evaluated by defining a collection \( f_1, f_2, \ldots, f_n \) of nested yield surfaces (von Mises for cohesive materials and Drucker-Prager/Mohr-Coulomb for frictionless materials) in the stress space [Fig. 7(a)]. The yield surfaces define regions of constant shear moduli in the stress space, by means of which the model discretizes the smooth elastic-plastic stress-strain curve into \( n \) linear segments. When the stress point reaches the yield surface \( f_m \), all the yield surfaces \( f_1, \ldots, f_m \) are tangent to each other at the contact point, as shown in Fig. 7(b). The outermost surface \( f_p \) corresponds to zero shear strength.

The material hysteretic behavior and shear stress-induced anisotropic effects are simulated by a kinematic rule. Upon contact, the yield surfaces are translated in the stress space by a stress point, and the direction of translation is selected such that the yield surfaces do not overlap, but remain tangent to each other at the stress point.

For the case of nonpressure sensitive materials in particular, a kinematic hardening rule is employed, according to which the yield surfaces do not change size, but are translated in stress space by the stress point. For the case of pressure sensitive materials, a purely kinematic rule is adopted. The dependence of the moduli on the mean effective normal stress \( (p') \) is assumed to be of the following form:

\[
x = x_p (p'/p_p)^n\]

where \( x \) = shear \( (G) \), bulk \( (B) \), or plastic \( (H) \) moduli; \( p_p \) = reference mean effective normal stress; and \( n \) = so-called power exponent. The multiyield plasticity constitutive model parameters required for the formulation of the problem under investigation are briefly described in the ensuing:

1. State parameters: Mass density of the solid phase \( \rho_s \).
2. Low strain elastic parameters: Low strain moduli \( G_0 \) and \( B_0 \); power exponent \( n = 0.5 \), and the reference effective mean normal stress \( p_p = 1 \) kPa.
3. Yield and failure parameters: These parameters describe the position, size, and plastic modulus corresponding to each yield surface \( f_i \), the total number of which is also an input model parameter of the multiyield plasticity constitutive law; in particular for the analyses illustrated in the ensuing, a total number of 200 yield surfaces was selected to ensure adequate representation of the material response subjected to transient high-frequency input motion. The shear stress-strain relations that were implemented for the simulations and were linearly discretized in the deviatoric stress space via the multiyield

Fig. 6. Crest-to-base (top), center line-to-far-field, and crest-to-far-field transfer functions of the horizontal acceleration component, as a function of the soil-structure impedance ratio \( r \).
surface representation, correspond to the modified hyperbolic expression proposed by Hayashi et al. (1992). The necessary parameters for this model are: (1) the initial gradient \( G_0 \); (2) the undrained strength \( s_u \) and reference shear strain \( \gamma_{\text{max}} \) at failure; (3) the parameter \( a \), defined experimentally as a function of the maximum grain size \( D_{\text{max}} \) and uniformity coefficient \( C_u \); and (4) the coefficient of lateral stress \( K_0 \), necessary for the evaluation of the initial positions of the yield surfaces.

The calibration of elaborate constitutive soil models, especially of the complexity degree of multiyield plasticity formulations such as the ones by Mroz (1967), Prévost (1977, 1985), etc., requires the availability of stress-strain curves obtained by means of triaxial or simple shear soil tests, procedures which are, however, frequently prohibited by budget constraints. Even further, the usual paucity of field information—as is the case in this study—as well as the randomness and spatial variability of natural deposits, require parametric studies and/or Monte Carlo simulations to be conducted. Therefore, the generation of stress-strain curves required for the analysis—from the necessarily limited amount of field information—is a common and significant problem.

The lack of field and laboratory experimental data in the Adàmes region necessitated the evaluation of the required yield and failure parameters by means of nonlinear least-squares fit of the predicted modulus degradation curves to the corresponding curves published by Vucetic and Dobry (1991), as a function of the plasticity index at the atmospheric confining pressure level; successively, the depth dependency of nonlinear material properties was accounted for by means of Eq. (6). In the objective function of the nonlinear optimization, the stress-strain curve coefficient \( a \) and reference strain at failure \( \gamma_{\text{max}} \) were selected as the free parameters, while the undrained shear stress \( s_u \), coefficient of lateral earth pressure \( K_0 \), and mass density \( \rho \) were defined from existing field data and empirical correlations as follows:

\[
\frac{s_u}{\rho} = 0.11 + 0.0037 I_p[\%] \quad \text{(Skempton and Henkel 1953)}
\]

\[
\rho = 2.0 [\text{t/m}^3] \quad K_0 = 0.5
\]

where \( p' = \) in situ overburden pressure and \( I_p[\%] = \) plasticity index of normally consolidated clay. The best-fit modulus degradation and damping curves for the site conditions in Adàmes evaluated by means of the Hayashi et al. (1992) stress-strain formulation and the multiyield plasticity model unload-reload constitutive law are depicted in Fig. 7(c). It should be noted that the sensitivity of the computed site response was also investigated and found to be within acceptable limits for variation of the free parameters within \( \pm 15\% \) of the global optimum solution. For further details on the material model parameters and field or laboratory suggested investigation techniques, the reader is referred to Popescu (1995), while for further details on the sensitivity analysis conducted on the nonlinear material properties, the reader is referred to Assimaki (2004).

**Soil-Structure Interaction for 1D Ground Surface Conditions**

Two types of concrete structures were investigated in the ensuing, with shear-wave velocity \( V_{s, \text{STR}} = 2800 \, \text{m/s} \) and dimensions
typical of four-story residential buildings prevailing in the area of interest: (1) a surface structure \((D=25.0\, \text{m}, \text{height}=16.0\, \text{m})\); and (2) a structure with 20% embedment depth (basement) \((D=25.0\, \text{m}, \text{height}=20.0\, \text{m})\). The structures were simulated as plane-strain shear beams, namely, as infinitely long asperities on the surface of the topographic irregularity in the off-plane direction, while the equivalent block density in both cases was defined as 0.4 \(\text{tn/m}^3\), resulting in the self-weight of the surface and embedded structures to be 160 \(\text{kPa/m}\) and 200 \(\text{kPa/m}\), respectively.

It should be noted that while the selected numerical model of structural configuration as a linear elastic continuum of equivalent reduced density may not realistically capture the ensemble of inertial interaction effects that arise in the occurrence of a strong intensity seismic event, the relatively stiff site conditions and low static loads of typical structures in the region of Adâmes retain kinematic interaction phenomena as the predominant factors of topographic amplification altering, and, thus, render the selected structural numerical representation adequate for the purposes of this study. Similarly, despite the limitations involved in the conduction of plane-strain in lieu of three-dimensional analyses for the investigation of inertial and kinematic interaction effects, increasing the spatial dimensions of the configuration was considered beyond the purposes of this study, whose objective is the understanding of wave diffraction and scattering phenomena in the vicinity of the crest that alter the nonlinear topographic amplification pattern already studied by the authors and co-workers for the case of a two-dimensional configuration (see Assimaki 2004; Assimaki et al. 2005a,b).

For the ensemble of foregoing simulations, a realistic representation of the structural construction process was ensured by applying the structural loading gradually, once equilibrium due to self-weight of the soil is established, and in particular for the embedded structure, simulating both the soil excavation and the gradual increase of structural weight. Finally, to avoid spurious reflections from the boundaries of the numerical model, repeatable boundary conditions are imposed to the far field, namely, the leftmost nodes of the finite element model are slaved to the rightmost nodes at the side boundaries. Slaving is imposed both in the horizontal and vertical directions, simulating an identical response of the corresponding nodes, i.e., the infinite extent of the computational domain in the lateral dimension. The effectiveness of the approximation is ensured by placing the structure an adequate distance from the lateral boundaries, so that the imposed periodicity of the discretized domain will not alter the computed response (see enlarged detail of finite-element mesh in the vicinity of the structure in Fig. 8(a)). Typical results of the response computed for the surface and embedded structures founded on Profile C and subjected initially to a quasi-static load of the form

\[
P(t) = 180 \sin \left( \frac{\pi t}{2} \right) \text{(kPa/m)}
\]

are applied at the center of gravity of the structure, after establishing equilibrium in the numerical solution due to the soil and structure self-weight initiation. Note that the selection of horizontal load amplitude comparable to the weight of the structures is equivalent to a quasistatic seismic inertial loading of peak ground acceleration \(a_g = 1.0\, \text{g}\).

For this configuration, Fig. 8(b) depicts the time history of horizontal (left) and vertical (right) displacement at various locations along the surface, for the surface (top) and embedded (bottom) structures founded on Profile C. Successively, Fig. 8(c) illustrates the shear stress-strain hysteresis loops at selected depths along the edge (left) and the center line (right) of the surface (top) and embedded (bottom) structures. Interpretation of these results leads to the conclusion that a nonsymmetric, valley-shaped region of peak displacement is formed below the structure (Fig. 8(b) shows maximum displacement at the center line and nonsymmetric time histories for the left and right edges of the structure), which gives rise to plastic drift in the vertical direction, due to the rocking structural response.

The material softening observed both for horizontal and vertical quasi-static loading conditions (the latter can be found in Assimaki (2004)) is also characteristic of the soil response in the immediate vicinity of the foundation upon the incidence of coherent, vertically propagating in-plane waves, and results in spatially varying amplitude of the parasitic vertical response (i.e., displacement, velocity, and acceleration). Finally, this phenomenon is further intensified for the case of seismic loading, comprising both horizontal and vertical components, characteristic of seismic motion in the vicinity of topographic discontinuities, where diffraction of seismic waves results in parasitic vertical components that carry a significant amount of the incident seismic energy.

**Topography-Soil-Structure Interaction Simulations**

The structural response to coherent incident seismic motion at the base of the numerical domain was successively evaluated to illustrate the role of topography in altering the soil-structure interaction effects observed for the site conditions and input seismic motions of interest. The geometry of the configurations used in the numerical simulations of nonlinear topography-soil-structure interaction is schematically shown in Fig. 9. In accordance with previous publications by the authors and co-workers (Assimaki and Gazetas 2004; Assimaki 2004; Assimaki et al. 2005a,b), the underlying soil profile comprises horizontally stratified soil layers that correspond to Profiles A, B, and C, and the configuration is subjected to vertically incident in-plane shear waves corresponding to the six acceleration time histories.

Results are next presented in Figs. 10(b-d) and 11, where the frequency- and spatial-domain response is averaged for each configuration (surface and embedded) and Profiles A, B, and C over the multitude of input motions used. In each figure, the symbols used correspond to the motion computed at the following nodes of the numerical model; also schematically illustrated in Fig. 10(a) for the case of the surface structure: \(a_{2D\,\text{SSI}}\) at the center line of the structure and at the soil-structure interface; this location corresponds to the ground surface for the surface structure and to the embedment depth for the embedded structure; \(a_{1D\,\text{SSI}}\) at the same node for the soil-structure interaction simulations on horizontally stratified layered media; \(a_{2D}\) on the surface behind the crest at the location of the structural center line in the absence of the structure (see Assimaki et al. 2005b); and \(a_g\) is the ground surface motion computed for a layered medium in the absence both of the topographic irregularity and structure. The main conclusions drawn from the ensemble of simulations are summarized below:

1. Fig. 10(b) compares kinematic SSI effects for ground level and two-dimensional topographic conditions by illustrating the spectral content of the response recorded at the center of the soil-structural interface, normalized by the corresponding motion at the same location in the absence of the structure. In particular, in the period range 0.05 \(s < T < 0.1\, \text{s}\) where the incident wavelengths are comparable to the horizontal structural dimensions, kinematic SSI phenomena dominate the re-
Fig. 8. (a) Location of finite element model nodes (denoted as receivers) where displacement time histories and stress-strain hysteresis loops have been depicted during the numerical simulation of nonlinear soil-structure interaction effects for a surface and an embedded structures located on Profile C, in the absence of the topographic irregularity (enlarged detail of finite element mesh in the vicinity of the structure); (b) displacement time histories of horizontal (left) and vertical (right) displacement at the center line (CL), immediate vicinity of the structure’s leftmost and rightmost sides (±15.0) and free field (x = ±20.0) along the surface, for a rigid surface (top) and embedded (bottom) structures founded on Profile C and subjected to quasistatic transverse loading of sinusoidal form at their center of gravity; and (c) shear stress-strain hysteresis loops at selected depths along the leftmost side (x = −15.0) (left) and center line (x = 0.0) of a rigid surface (top) and embedded (bottom) structures founded on Profile C and subjected to quasistatic transverse loading of sinusoidal form applied at their center of gravity.
sponse by reducing the amplitude of motion at the location of the structure, especially for the 2D topographic configuration that is characterized by surface waves traveling in the vicinity of the crest. On the other hand, in the 0.2 s < T < 0.35 s period range that corresponds to the dominant period of seismic excitations, kinematic SSI effects are more pronounced for the ground level configuration, in which the structural response is only attributed to the incident seismic energy in the form of vertically propagating seismic waves that have not undergone mode conversion—and therefore no frequency-content altering—prior to the initiation of the structural response. Finally, kinematic SSI is shown to have minimal effects for the surface structure in the long period range (T > 0.5 s), and for the embedded structure in the entire T > 0.2 s range;

2. Fig. 10(c) illustrates the effects of soil-structure interaction on the topographic motion aggravation by comparing the ground surface motion normalized by the corresponding far-field surface response. As can be readily seen, the presence of the surface structure aggravates the topographic amplification effects in the short-to-intermediate period range due to wave diffraction effects at the soil-structure interface, while the embedded structure primarily affects the short-period components (T < 0.3 s) and especially in the softer Profile C case, results in the most pronounced topographic amplification phenomena for T > 0.5 s;

3. Fig. 10(d) plots the peak-ground acceleration recorded at the surface of the topographic configuration for the surface, embedded, and free-field cases, normalized by the corresponding value in the far field. Compared to the free-field motion, the coherent response along the soil-structure interface prevents—for the most part—the erratic spatial distribution of peak horizontal acceleration. As a result, the level of topographic aggravation close to the vertex is shown to be of the same order of magnitude for the two problems. Topographic amplification, however, is primarily restricted in the vicinity of the foundation, and strongly decreases thereafter. Nonetheless, the overall zone of influence of topography effects behind the crest is practically the same as in the free-field problem. Finally, the peak vertical acceleration is shown to be on the same order of magnitude for all the cases analyzed. The sharp peaks observed at the corners of the surface structure case correspond to the rocking response of the structure, associated with the inclined, reflected, and diffracted waves that comprise the structural seismic input. For the embedded structure case, sharp peaks are not observed, since the response of the configuration is evaluated at ground surface, and therefore refraction and reflection of incident waves at the base of the embedded structure is already incorporated in the illustrated results;

4. Fig. 11(b) compares the modified amplification factor (i.e., $SA_{2DSSI}/SA_{1D}$) that includes topographic and soil-structure interaction effects, with the product of the average free-field topographic aggravation factor (i.e., $SA_{2D}/SA_{1D}$), the average soil-structure interaction factor (i.e., $SA_{1DSSSI}/SA_{1D}$) for ground-level conditions, and the topography-SSI modification factor, defined as

$$modF = \frac{(SA_{2DSSI}/SA_{2D})}{(SA_{1DSSSI}/SA_{1D})}$$

at the crest for Profiles A, B, and C; the spectrum of the latter is shown in Fig. 11(a). Note that the excellent comparison was not granted, due to the nonlinear transient nature of the solution and the averaging process over a series of incident motions of different peak amplitude and frequency content. Recently, Assimaki et al. (2005b) have introduced a tentative scheme by which topographic amplification effects could be included into contemporary building codes by multiplication of the far-field response spectra with the expected amplification factor (TAF) as a function of normalized frequency and normalized distance from the crest. Based on the aforementioned results, similar simplified curves seem possible to be generated to account for the 2D modified soil-structure interaction input motion (base-slab averaging and inertial interaction effects) as a function of site class, normalized dimension of the structure relative to the topographic feature’s characteristic length, and location of the structural center line from the crest. Obviously, for this purpose, numerical simulations based on multiple configurations should be combined with recorded data from weak and strong motion events, collected through structural instrumentation research programs. A typical example of such a program is the Advanced National Seismic System, a national seismic network operating under the auspices of the National Earthquake Hazards Reduction Program of the United States Geological Survey (http://earthquake.usgs.gov). This network comprises densely installed urban and regional stations that provide accurate and timely data and information products for seismic events, including their effects on buildings and structures, by means of
modern monitoring methods and technologies.

5. Finally, Fig. 12 shows the peak shear modulus reduction extracted from the lower secant modulus during the incremental nonlinear simulations for the two configurations and two input motions, namely the Cholame and high-frequency incident motion MNSA acceleration time histories; the wave forms and spectral content are depicted in Fig. 12. As can be seen, the effects of structural embedment are more pronounced in the MNSA, and comprise stress relief in the immediate vicinity below the structure and increased zone of yielding in the immediate vicinity behind the structure towards the far field. Nonetheless, this zone of reduced stiffness is observed for both incident motions when compared to the corresponding free-field response, causing sharp reduction of the peak ground acceleration behind the structure [Fig. 10(d)]. Results are compared to the topographic amplification factors estimated by Assimaki et al. (2005a) and the Eurocode 8 (EC8 2000) for the free-field 2D configuration, and illustrate the beneficial effects of SSI for the location of the structure. It should be noted, however, that the topographic aggravation factor is also space dependent, and while reduced response is illustrated at the structural center line due to SSI effects, scattered waves from the soil-structure interface aggravate, correspondingly, the free-field response, either towards the toe or towards the far field behind the crest, depending on the soil-structure impedance contrast.

Conclusions

This paper has presented the additive role of kinematic and inertial soil structure in aggravating the surface ground motion near the crest of cliff-type topographies, within the context of a case study from the Athens, 1999 earthquake. It has been shown that the most important effect of soil-structure interaction for stiff soil
Fig. 10. (Continued).
conditions is the kinematic incompatibility of the structural response following the strongly differential surface motion in the vicinity of the vertex. As a result, frequencies that correspond to wavelengths comparable or shorter than the horizontal dimension of the structure are filtered. At the same time, however, spatial variability of the parasitic component imposes a rocking motion at the foundation of the structure, and its subsequent response further aggravates the vertical ground motion. This phenomenon is particularly important for relatively soft structures, for which the reflection coefficients of inclined seismic energy at the foundation level are very low. As a result, refracted waves are forced to propagate in a rotational path within the structure, imposing loading conditions that are not taken into account in current design practice.

The numerical simulations have illustrated the response of structures founded next to the crest of the Kifissos canyon and on the slope of the cliff, namely, two- to four-story concrete residential buildings that are relatively stiff and light weighted to produce minimal inelastic effects in the absence of the topographic irregularity. Nonetheless, site-specific simulations and a series of parametric studies focusing on structural flexibility have shown that the nonlinear ground surface response of an irregular topographic feature is altered upon the presence of a structure:

- Stiff structures founded on the crest result in filtering of the high-frequency components of surface horizontal response, due to kinematic interaction effects, namely geometric incompatibility of the structure to follow the strongly differential surface response near the crest. Surface waves that arrive at the crest from the toe of the cliff are reflected backwards, a fact that intensifies the differential motion along the slope.
- The vertical component imposes a strong rocking motion at the foundation level of the structure. Its rotating response is particularly important for relatively soft structures, since the inclined incident waves at the base are trapped in the superstructure and further amplified. For soft soil conditions, substantially high strain would be subsequently imposed on the underlying profile as a result of the structural response.

For the cliff-crest SSI configurations studied, results showed that the free-field topographic aggravation factor (TAF), namely the factor that should be used to modify the far-field ground surface conditions to account for the effects of topographic irregularities, could be potentially further modified by a base-slab averaging factor and a topography-SSI modification factor, both defined as functions of the site conditions, normalized structural dimensions, and structural location on the ground surface, to approximate the input motion for structures founded on irregular topographic features and subjected to incoherent incident waves due to scattering. For the site conditions, topo-

Fig. 11. Comparison of modified TAF due to soil-structure interaction effects, with the product of the average TAF and the average soil-structure interaction modified input motion at the crest for Profiles A, B, and C: (a) spectrum of the 2D-to-1D modification factor that represents the altering of SSI effects due to the presence of a single-faced slope topographic irregularity; (b) modified topographic aggravation factor accounting for SSI effects at the crest, as the product of the free-field topographic aggravation factor (TAF), the base-slab averaging factor for level ground surface conditions (SA_{1D,SSI}/SA_{0}), and the 2D-to-1D base slab modification factor (modF); results are compared with the average TAF by Assimaki et al. (2005a) for free-field conditions, and the frequency-independent TAF as indicated in the Eurocode 8 (EC8 2000) seismic code provisions for slope inclination $i \geq 30^\circ$.
graphic, and structural configurations investigated in this study, comparison with the frequency-dependent topographic amplification factors estimated by Assimaki et al. (2005b) in the absence of SSI effects, as well as the frequency-independent topographic amplification factors suggested by the Eurocode 8 (EC8 2000) for slope inclination $i \geq 30$ deg (TAF=1.4), results show that the effects of SSI are beneficially altering topographic amplification phenomena in the vicinity of the crest; however, scattered waves from the soil-structure interface aggravate, correspondingly, the free-field response, either towards the toe or towards the far-field behind the crest, depending on the soil-structure impedance contrast.

It should also be noted that the erratic response observed along the slope and in the vicinity of the crest would be potentially very important for long, flexible structures that are prone to strong differential input motions, while additional complexity would be introduced by topographic features that involve spatial variability of surface motion in both plane directions, i.e., three-dimensional irregularities. In addition, relatively large structures founded on soft soil formations would impose strain softening of the material around and below the foundation solely upon static loading, in contrast to the particular case study. As a result, additional resonant frequencies would arise for the soil-structure system, associated with the vertical and horizontal dimensions of this area. Upon incidence of inclined and surface waves, as is the case in the vicinity of topographic irregularities, valley-type resonant effects would be superimposed to the already aggravated input motion. Even further, for structures that would potentially span canyon-type topographic features, such as bridges, which comprise pier foundations at the slope and abutments behind the crest,

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**Fig. 12.** (Color) (a) Field plot of percentage of peak shear modulus reduction—evaluated on the basis of the largest hysteresis loop at each point—for a rigid structure founded on Profile C and subjected to the Cholame (top) and MNSA (bottom) rock-outcrop seismic motions. (b) Acceleration time histories and elastic response spectra for 5\% structural damping for the two seismic events (Cholame and MNSA) used in this paper to illustrate altering of topographic amplification as a result of the effects of kinematic soil-structure interaction.
results show that special consideration should be taken with respect to the incident motion at the abutment due to the presence of a structure at the cliff.

Despite the extensive parametric study presented in a series of papers by the authors and co-workers on the effects of topography—completed in this paper by the investigation of the additive effects of SSI on the modification of topographic seismic amplification—the authors acknowledge that their results correspond to a certain range of site conditions, a given topographic geometry and structural configuration, and consequently, a frequency range of confidence between 1 and 8 Hz; the latter can be readily seen in the ensemble of the foregoing results, which clearly show that the frequency domain resolution is substantially reduced and yields a steep amplitude decrease in the response spectral amplitude for frequency components beyond this range. Even further, the simulations presented, especially the nonlinear strong motion site response analyses, have not been validated against strong motion recordings. Therefore, the parametric investigation and topography-SSI modification factors presented above should be interpreted primarily from a qualitative viewpoint and generalization to other geometric configurations and site conditions should be applied with caution. For the development of quantitatively accurate factors to be tentatively included in design code provisions and account for topography-SSI effects, a statistically significant number of similar analyses should be conducted to account for the effects of slope geometry, site conditions, and structural nonlinearity, and validated by a series of weak and strong motion recordings.

References


Sanchez-Sésma, F. J. (1983). "Diffraction of elastic waves by three-