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SIMPLIFIED THEORETICAL ANALYSIS OF THE SEISMIC RESPONSE OF ARTIFICIALLY COMPACTED GRAVELS

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ABSTRACT

Despite extensive use of gravely materials for the construction of big earthworks, theoretical prediction of their seismic response is often based on very simple schemes, unable to reproduce most of the important features observed at sample scales. Theoretical models capable of simulating the effects of artificial compaction on the stress-strain response of these soils under complex static and dynamic loading conditions would be particularly useful for designing more cost effective solutions in the construction of large embankments. This paper is aimed to fill this gap by reporting the results of a simplified theoretical study on the seismic response of an artificial deposit of gravels compacted at different densities. A previously defined critical state multiple yielding elasto-plastic constitutive model, validated with the results of a large variety of triaxial tests on gravels (measurement from small to large strains, samples compacted at different initial soil densities, monotonic and cyclic loading conducted at largely different stress levels), is here adopted to calculate the shear stiffness and the damping an equivalent visco-elastic model. These results form the input of a finite differences one dimensional analysis implemented to study the propagation of shear waves into horizontally layered gravel deposits subjected to variable motion of their underlying bedrock. Analyses are performed in the frequency and time domains by varying the maximum amplitude of the base acceleration to evaluate the filtering and amplification effects of the deposit. The results of this study are parametrically reported in terms of free surface accelerations and amplification ratios, to show how artificial compaction affects the response of gravel.

INTRODUCTION

Thanks to their excellent mechanical properties and to a widespread availability, gravely materials are largely adopted in different fields of civil engineering. Earthfill and rockfill dams, highway and railway embankments, marine constructions are some examples of the large variety of possible applications. Since early stages of civil constructions practice noticeable importance was placed on the compaction of granular materials for the improvement of their mechanical response (Kerisel, 1985). However, only in the first half of last century, a scientific approach based on systematic experimental investigations was introduced to quantify and control the effects of compaction in the construction of large dams (Marsal, 1973). Significant improvements on the performance of earth structures could be finally obtained with the development of more powerful and effective machineries and with the optimisation of compaction procedures (Veiga Pinto, 1991). In spite of these progresses, design analyses are still performed with very simple schematisations of the stress strain response (e.g. linear elastic constitutive models) which neglect most of the mechanical characteristics of gravely soils. The surprising difference with the sophisticated approaches currently adopted for finer soils basically derives from a large confidence placed on the capacity of gravely materials and (much more) on the difficulties of obtaining accurate experimental data due to the uncommonly large dimensions of the required laboratory equipments. Accurate analyses focusing on the stress strain response of gravels under complex static and dynamic loading conditions would on the contrary lead to more reliable predictions of the structural performance of earthworks and to the design of more cost effective solutions.

The characteristics of gravels response under cyclic loading are first summarised in an experimental study of Seed et al. (1986) aimed to define the parameters of an equivalent visco-elastic model for dynamic analyses. By performing cyclic tests with variable strain amplitudes on several gravely materials compacted at different relative densities, the authors evaluate the soil response in terms of secant shear modulus and equivalent damping ratio variations with shear strain amplitude. A qualitative dependency of these curves on the soil density and on the stress applied at the beginning of cyclic tests is established by a comparison of different results.

A large number of studies were subsequently conducted with more accurate laboratory equipments focusing on the small strain stiffness of cohesionless soils. With particular reference to gravely soils, Park and Tatsuoka (1994) observe a dependency of normal stiffness on the orientation of loading directions compared with the planes of soil deposition. A further anisotropy factor is represented by the observed dependency of stiffness on the different stress components (Jiang et al., 1997). Modoni et al. (2000) simultaneously performed small strain (< 10^{-3} %) cyclic triaxial and pulse wave transmission tests on a gravel compacted at different initial void ratios. Although larger moduli are obtained by the dynamic tests compared with the static ones, results obtained with the two testing methods show a similar dependency of stiffness on the stress components and on the soil initial void ratio. Theoretical formulation of this dependency is found in a model introduced by Tatsuoka and Kohata (1995), simulating the inherent and stress induced anisotropy, and in a relation proposed by Hardin and Richart (1963) expressing the dependency of stiffness on initial void ratio.

Among the very few attempts of modelling the response of gravel at larger strain levels there is a study conducted by Balakrishnaiyer and Koseki (2001), who interpreted the results of their large amplitude cyclic tests with a model previously defined for sands by Masuda (1999). A drag and scaling rule was defined by this model to relate the unloading and reloading stress strain curves to the primary loading backbone curve. Clear dependency of soil response on its initial density is however not taken into account. With the aim of simulating the stress strain response of gravel from small strain to failure under the most general conditions (different soil void ratio, initial stress level, complex monotonic and cyclic loading paths), a critical state multiple hardening elasto-plastic model has been introduced by Modoni et al. (2008). It consists of an adaptation to gravely soils of models previously defined for sands (Muir Wood et al. 1994, Manzari and Dafalias, 1997) whose main concept is the combined dependency of soil response on the current void ratio (e) and mean effective stress (p'), both considered by the state variable ψ (Jefferies, 1993) expressing the distance from the critical state line in the e-p' plane. The model, validated by a large collection of experimental results, is also extended to cyclic loading by introducing multiple yielding, and by modifying hardening and flow rules.

The approach followed in the present study for the seismic analyses is the same originally suggested by Idriss and Seed (1968), based on one dimensional propagation of seismic shear waves into a horizontally unlimited embankment of gravely soils. In these analysis the soil response is simulated with equivalent linear visco-elastic models whose characteristic curves (G-γ and D-γ) are built by repetitive application of the previously recalled elasto-plastic model (Modoni, 2008). This latter, calibrated with a large number of experimental results, is thus used for the simulation of experimental tests where the initial conditions (namely initial stress and soil density) are continuously varied in their typical ranges. This procedure is aimed to quantify by a comparative analysis the effects of compaction on the seismic response of the gravely soil deposit, without introducing the noticeable complexity of implementing an original elasto-plastic model into the numerical code.

ELASTO - PLASTIC COSTITUTIVE MODEL

This theoretical analysis is carried out by assuming the gravel as a continuum and that each deformation increment is given by a sum of an elastic and a plastic component:

$$\delta\varepsilon = \delta\varepsilon^{e} + \delta\varepsilon^{p} \qquad (1)$$

In particular, the elastic stiffness is modelled with a cross-anisotropic model defined by Tatsuoka and Kohata (1995). Considering the vertical axis coincident with the direction of soil deposition, the stress strain relation is expressed by the following matrix relations:

$$\begin{bmatrix}\varepsilon_x\\ \varepsilon_y\\ \varepsilon_z\\ \gamma_{xy}\\ \gamma_{yz}\\ \gamma_{yz}\end{bmatrix} = \begin{bmatrix} \frac{1}{E_h} & \frac{-\nu_{hh}}{E_h} & \frac{-\nu_{vh}}{E_v} & 0 & 0 & 0\\ \frac{-\nu_{hh}}{E_h} & \frac{1}{E_h} & \frac{-\nu_{vh}}{E_v} & 0 & 0 & 0\\ \frac{-\nu_{vh}}{E_v} & \frac{-\nu_{vh}}{E_v} & \frac{1}{E_v} & 0 & 0 & 0\\ 0 & 0 & 0 & (1+\nu_{hh})/E_h & 0 & 0\\ 0 & 0 & 0 & 0 & 1/2G_{vh} & 0\\ 0 & 0 & 0 & 0 & 0 & 1/2G_{vh} \end{bmatrix} \begin{bmatrix}\Delta\sigma'_x\\ \Delta\sigma'_y\\ \Delta\sigma'_z\\ \Delta\tau_{xy}\\ \Delta\tau_{yz}\\ \Delta\tau_{zx}\end{bmatrix} \qquad (2)$$

where:

$$\begin{aligned} E_v &= E_1 \cdot f(e) \cdot \sigma'^{\,m}_{v} \cdot p_r^{\,1-m}\\ E_h &= E_1 \cdot (1-I_o) \cdot f(e) \cdot \sigma'^{\,m}_{h} \cdot p_r^{\,1-m}\\ \nu_{hh} &= \nu_o\\ \nu_{vh} &= \nu_o \cdot (1-I_o)^{-1/2} \cdot \left(\frac{\sigma'_v}{\sigma'_h}\right)^{n}\\ \nu_{hv} &= \nu_o \cdot (1-I_o)^{1/2} \cdot \left(\frac{\sigma'_h}{\sigma'_v}\right)^{n}\\ G_{vh} &= \frac{E_1 \cdot (2 - Io) \cdot f(e)\,(\sigma_v + \sigma_h)^m}{4(1+\nu_o)}\\ f(e) &= \frac{(2.17 - e)^2}{(1+e)} \end{aligned} \qquad (3)$$

p_r is a reference pressure (1 kPa), f(e) is a function of void ratio defined by Hardin and Richart (1963). I_o is a parameter defining inherent anisotropy which must be quantified, similarly to E_1, ν_o, m and n, by fitting of experimental results.

The plastic stress-strain relation is calculated with a model based on the critical state, whose locus in the void-ratio – stress invariants space is defined by the following classical relations:

$$\frac{q}{p'} = M \qquad e_{cs} = \Gamma - \lambda \cdot \ln p' \tag{4}$$

where Γ, λ and M are commonly acknowledged soil parameters. In particular, stress-strain relations depend on the distance in the e-p' plane from the critical state locus, expressed by the state variable ψ (Jefferies,1993):

$$\psi = e - e_{cs} \tag{5}$$

The dependency of the hardening functions on ψ for primary loading and for unloading-reloading paths is defined respectively by eqs. 6 and 7.

$$\frac{S}{S_{u.b.}} = I \frac{\varepsilon^p{}_q \cdot \left|\varepsilon^p{}_q\right|^{c-1}}{B + \left|\varepsilon^p{}_q\right|^c} \tag{6}$$

where ε^p_q represents the plastic distortional strain, S is a function of the stress invariants ratio (η=q/p'):

$$S = \frac{3 \cdot \eta}{6 + \eta} \tag{6.a}$$

$S_{u.b.}$ is an upper bound for S and is currently expressed as a function of the critical state friction angle ϕ'_{cs} and of the state variable ψ:

$$S_{u.b.} = \sin \phi'_{cs} \cdot (1 - k\psi) \tag{6.b}$$

B, c, K and I are soil parameters that must be defined with experimental results. In particular, two different values must be set for I in order to simulate the response on compressive ($I=I_{com}$ for $\varepsilon^p_q > 0$) and extensive ($I=I_{ext}$ for $\varepsilon^p_q < 0$) loading.

The hardening functions for each unloading and reloading step are written in a similar form as for primary loading, by resetting stress and strain variables at the last reversal point.

$$\Delta\left(\frac{S}{S_{u.b.}}\right) = (I_{com} + I_{ext}) \frac{\Delta\varepsilon^p{}_q \cdot \left|\Delta\varepsilon^p{}_q\right|^{c-1}}{B + \left|\Delta\varepsilon^p{}_q\right|^c} \tag{7}$$

With the above relations, the effects of stress history are accounted in the model by a hierarchy of nested yield surfaces defined by the $S/S_{u.b.}$ values at load reversal points (see Modoni, 2008).

The flow rules for primary loading and for unloading-reloading paths are defined by the following equations:

$$D^p = J \cdot M \cdot \frac{\Delta\varepsilon^p{}_q}{\left|\Delta\varepsilon^p{}_q\right|} \cdot \left(1 + k_D \cdot \psi \cdot |\psi|^{\beta-1}\right) - \eta \tag{8}$$

where D^p is the ratio between the increments of strain invariants ($\delta\varepsilon^p_p / \delta\varepsilon^p_q$), M is the stress invariants ratio at critical state, k_D, β and J are soil parameters to be found by fitting of experimental results. In particular J assumes different values for primary loading ($J_{p.l.}=1$), and for unloading and reloading ($J_{u.r.}<1$).

Calibration of all previously defined parameters (Tab.1) has been accomplished with the results of a large number of monotonic and cyclic triaxial tests performed on a crushed sandstone gravel (Chiba gravel) compacted at different initial void ratios (between 0.2 and 0.35) and tested at different initial confining stresses (from 50 to 650 kPa). In particular, the elastic model parameters have been evaluated by a mixed procedure simultaneously combining small strain triaxial unloading-reloading cycles ($\Delta\varepsilon<10^{-3}\%$) together with shear and compression pulse wave transmission tests (Modoni et al., 2000). The parameters for plastic strain model have been fixed by best fitting of the triaxial tests results (Modoni, 2008). In Fig.1 a sample is reported where experimental results and theoretical simulations are compared for a cyclic test conducted on Chiba gravel (σ'_h=const=490 kPa - e_o=0.311).

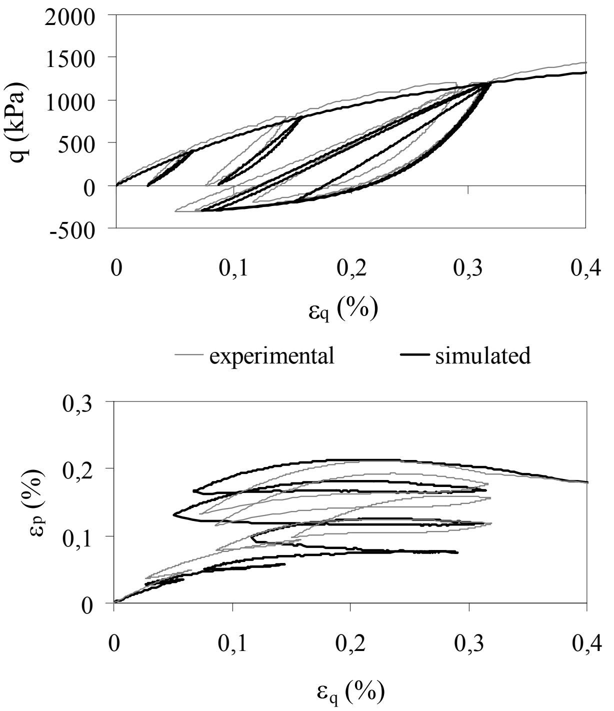


Fig. 1. Comparison between experimental and theoretical stress strain relations for a cyclic triaxial test (e_0 = 0.311; σ_h' = 490 kPa).

Tab. 1.Parameters of the elasto-plastic model for Chiba gravel

ELASTIC		E_1	I_o	m	n	ν_o
		18480	0.41	0.5	0.25	0.17
PLASTIC	ϕ'_{cs}	Γ	λ	k	B	C
	42.8	1.52	0.17	1	0.08	0.9
		I_{com}	I_{ext}	k_D	β	$J_{u.r.}$
		1	0.85	0.7	0.3	0.5

EQUIVALENT VISCO-ELASTIC MODEL

The study of shear wave propagation into a gravely soil deposit is carried out by adopting an equivalent visco-elastic continuum model (Idriss and Seed, 1968) for the simulation of the gravel stress-strain response. In this model the non linear irreversible soil response is schematised by a small strain value of shear stiffness G_o and by two curves expressing respectively the attenuation of G_{eq} and the increase of an equivalent damping D_{eq} with increasing distortional strains.

The initial modulus G_o has been put equal to the elastic shear modulus G_{vh} relating small shear strains γ to the shear stress τ in the vertical plane (eq.2). The modulus has been then calculated with eq.3 by assigning the parameters listed in Tab.1. A sample of G_o variation with depth in fully dry, horizontally layered gravel deposits compacted at different initial void ratios is reported in Fig.2. The vertical and horizontal stress components at different depths are calculated considering a specific density of soil particles G_s=2.7 and an in situ earth pressure coefficient k_o=1-sinϕ'_{cs}. From this plot a marked dependency of G_o values on the initial stress components (both proportional to the depth) as well as on the initial void ratio can be derived. These values are know to affect the propagation of waves into soil.

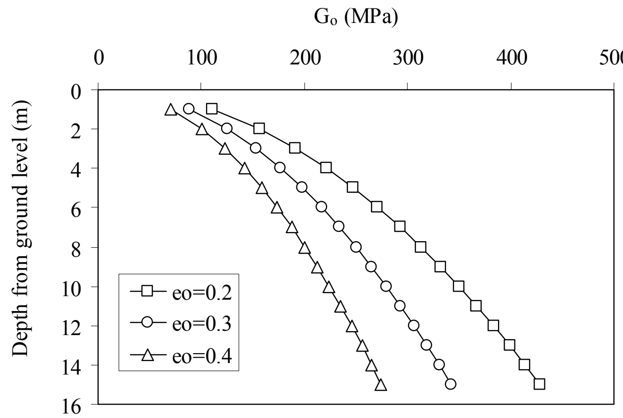


Fig. 2. Variation of initial shear stiffness G_o with depth for different initial soil void ratio.

The curves expressing equivalent shear modulus and damping variation have been calculated by simulating cyclic triaxial tests of different distortional strain amplitudes with the previously defined elasto-plastic models. All cyclic tests start from isotropic stress states and consist of constant p' shearing ($\Delta q/\Delta p'=\infty$). Loading with assigned distortional strain amplitude ($\Delta\varepsilon_q/2$) is initially performed. Then double strain amplitude ($\Delta\varepsilon_q$) unloading and reloading cycles are performed along the same stress path (Fig.3.a) and the corresponding q-ε_q curves are calculated. The values of equivalent shear stiffness and damping ratio from each cycle are then obtained as follows:

$$G_{eq} = \frac{1}{3}\frac{\Delta q}{\Delta \varepsilon_q} \quad (9.a)$$

$$D_{eq} = \frac{A_{cyc}}{4 \cdot \pi \cdot A_{f.l.}} \quad (9.b)$$

where Δq represents the double amplitude variation of deviator stress, A_{cyc} the area enclosed by the stress-strain hysteretic loop and $A_{f.l.}$ the product (1/8)*Δq*$\Delta\varepsilon_q$.

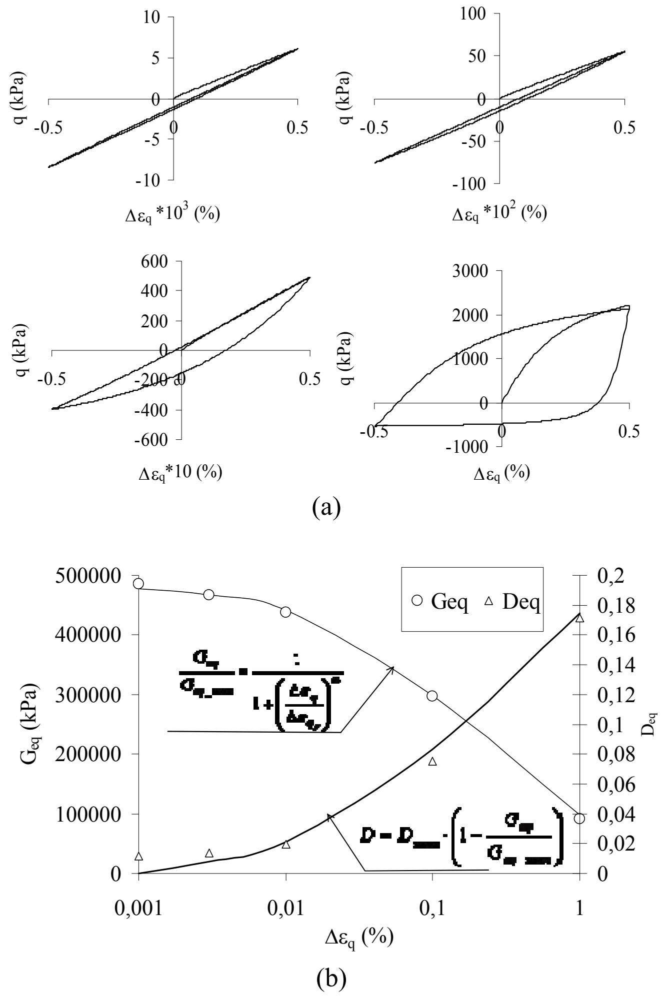


Fig. 3. Stress-strain response of Chiba gravel on different amplitudes cyclic tests (a); shear stiffness decay and damping ratio increase with cyclic strain amplitude (b) (tests have been performed at e_0 = 0.215 and p' = 600 kPa).

An example of simulations performed with e_o = 0,215 and p'= 600 kPa is shown in Fig.3. The different q-ε_q curves for cyclic tests of different amplitudes ($\Delta\varepsilon_q$ = 0.001%, 0.01%, 0.1%, 1%) reported in Fig.3.a show the typical effects of non linearity and irreversibility of soil response (i.e. progressively less steeper and open stress-strain loops). These effects are expressed by the two curves $G_{eq}(\Delta\varepsilon_q)$, and $D_{eq}(\Delta\varepsilon_q)$ of Fig.3.b.

The influence of mean effective stress and soil density on these responses have been singularly evaluated in Fig.4.a and 4.b, by comparing curves obtained from different simulations. In both figures the shear moduli calculated for different strain levels have been normalised with their value obtained for $\Delta\varepsilon_q$ = 10^{-3}%, typically assumed as a threshold for the transition from elastic to plastic responses.

The plot of Fig.4.a shows a significant influence of the mean effective stress on the non linearity of stress-strain response (G/G_o curves) and on the energy dissipation capacity (D curves) of soil. This result shows the importance of assigning different curves at the different layers of a gravel deposits mostly for particularly large heights of the embankment. It is also noticed that the trend of curves steepness obtained for higher p' is consistent with the experimental observation of Seed et al. (1986) on granular materials. In particular the calculated D-$\Delta\varepsilon_q$ curves fall within the ranges observed by Seed et al. (1986) on gravely soils (reported with shaded area), while the calculated shear modulus attenuation curves present a slightly steeper trend compared with observations.

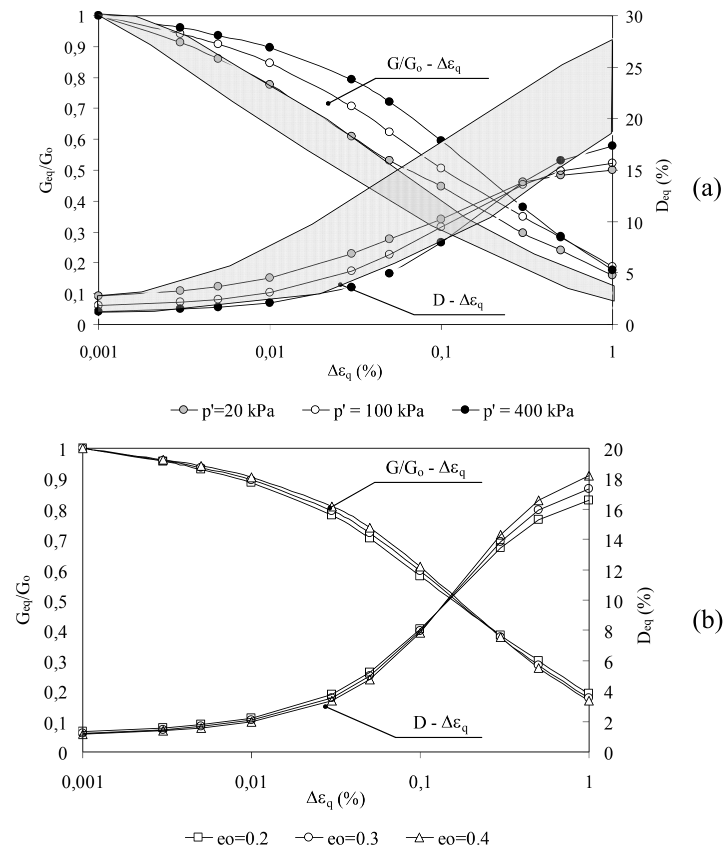


Fig. 4. Equivalent shear stiffness and damping as functions of distortional strain for different mean effective stress(a – e_o=0.3) and soil void ratio (b – p'=400 kPa).

By comparing the different curves obtained for fixed p' (= 400 kPa) and variable initial void ratio (Fig.4.b) a negligible effect can be retrieved for this latter. This result is consistent with the limited influence of relative density experimentally observed on gravels by Seed et al. (1986).

In order to link the above reported dependencies to the elasto-plastic model, the G_{eq}/G_o–$\Delta\varepsilon_q$ and the D_{eq}–$\Delta\varepsilon_q$ curves calculated for a larger number of simulations (by systematically varying e_o in the range 0.1-0.5 and p' in the range 20-800 kPa) have been parametrically expressed with the following empirical functions proposed by Hardin and Drnevich (1972):

$$\frac{G_{eq}}{G_o} = \frac{1}{1+\left(\frac{\Delta\varepsilon_q}{\Delta\varepsilon_{q_r}}\right)^{\alpha}} \qquad (10.a)$$

$$D_{eq} = D_{\max}\left(1 - \frac{G_{eq}}{G_{eq\_\max}}\right) \qquad (10.b)$$

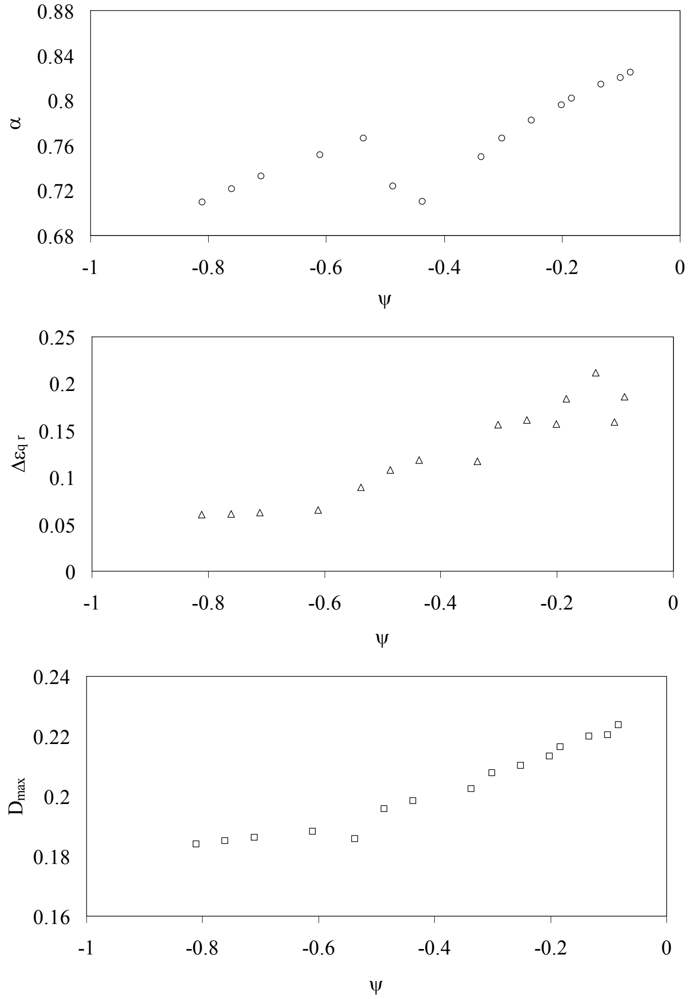


Fig. 5. Dependency of coefficients α, $\Delta\varepsilon_{q\,r}$ (rel. 10.a) e D_{max} (rel.10.b) from the state variable ψ for Chiba gravel.

The three parameters α, $\Delta\varepsilon_{qr}$ and D_{max} introduced in eqs. 10 have been calculated for different couples of initial void ratios and mean effective stresses by best fitting of the simulated curves (see Fig.2.b). Figs. 5 shows that the dependency of these parameters can be expressed rather uniquely as a function of the state variable ψ grouping void ratio and mean effective stress (eq.5).

FINITE DIFFERENCE ANALYSIS

The procedure described in the previous paragraphs is here adopted to assign the stress strain properties of gravel in the dynamic analysis of a horizontally layered deposits. One dimensional vertical propagation of shear waves is studied by means of a finite differences numerical code (EERA, Bardet et al., 2000) on an imaginary 15 m thick horizontally unlimited deposit subjected to motion of the underlying bedrock (Fig.6).

To his aim the gravely soil profile has been subdivided into a sequence of 1 m thick 15 layers, whose stress strain properties, consisting of initial shear stiffness modulus G_o, $G_{eq}/G_o(\Delta\varepsilon_q)$ and $D_{eq}(\Delta\varepsilon_q)$ curves, have been singularly calculated with the previously defined elasto-plastic models. A parametric variation of the initial void ratio among typical values attainable by compaction and different acceleration time histories at the bedrock have been considered to analyse the response of gravels under the most variable conditions.

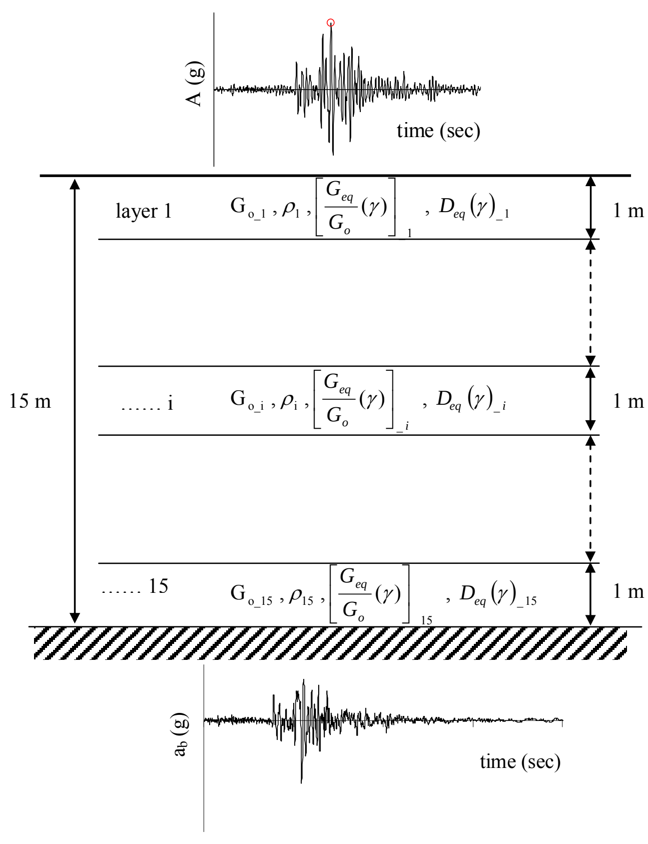


Fig. 6. One dimensional layout of the layered deposit.

A systematic study on the effects of the input motion on the response of soil deposit is conducted by considering different harmonic acceleration time histories at the bedrock. In particular, a parametric variation of amplitude A_b (between 0.1 and 1 g) and frequency f (between 1 and 15 Hz) has been assigned at the base acceleration:

$$a_b = A_b \cdot \sin(2\pi \cdot f \cdot t) \qquad (11)$$

The corresponding free surface accelerations A_s and amplification ratios (A_s/A_b) are then calculated for all assigned couples of A_b and f values. An example of calculation performed with a soil void ratio $e_o = 0.2$ is plotted in Fig. 7.

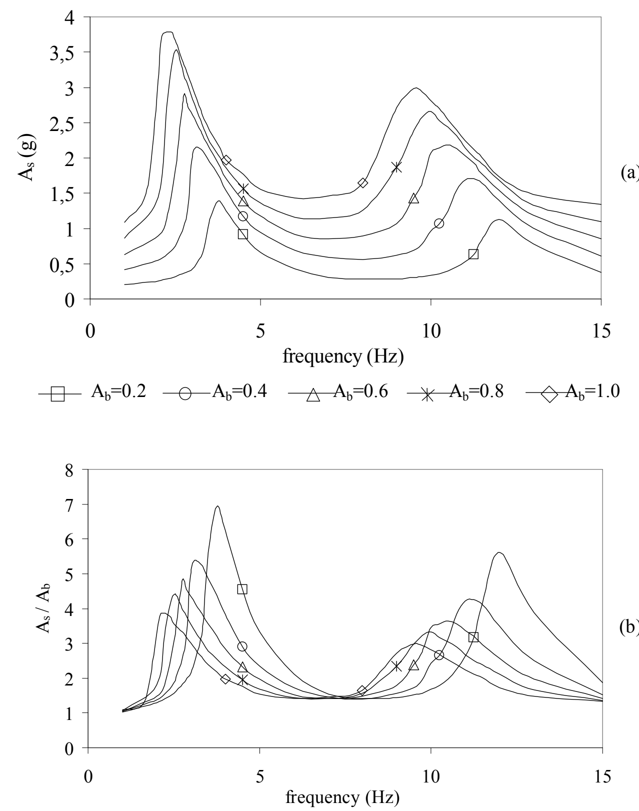


Fig. 7. Free surface maximum accelerations (a) and amplification ratios (b) calculated on a soil deposit (e=0.2) subjected to base input acceleration with harmonic time histories (eq. 11)

The results clearly show that the free surface accelerations (Fig.7.a) reach maximum values for typical fundamental frequencies dependent on the characteristics of the soil deposit and on the input acceleration. It is particularly worth observing the reduction of fundamental frequencies observed for larger A_b, due to the non linearity of soil response (i.e. decay of shear stiffness with increasing strains). The Fig.7.b, expressing the same results in terms of amplification ratios (A_s/A_b), shows the positive effect of the energy dissipation produced by the hysteretic soil response. Lower amplification ratios are in fact observed for larger input accelerations, possibly due to the higher values of equivalent damping ratios obtained on larger strains cycles (see Figs. 4).

The effects of soil density on the seismic amplification are evaluated by repeating the previous analysis for different soil initial void ratios. The results of this analysis performed for different e_o values are presented in Fig.8 where the maximum ratios A_s/A_b for each assigned value of A_b, is plotted as a function of the corresponding fundamental frequency (i.e. the coordinates of tip points of the curves in Fig.7). In this analysis a void ratio variation between 0.1 and 0.5 has been considered, slightly extending the range practically achievable by compaction of the considered gravel (Chiba). The plot clearly shows the influence of density on the dynamic soil response both on the fundamental frequencies and on the amplification ratios. Concerning the former aspect, the higher fundamental frequencies observed for denser soils can be explained considering their direct dependency on shear stiffness and the increase produced on this latter by compaction (see Fig. 2). Similarly, the reduction of amplification ratio with soil density can be justified as an effect of the increase of stiffness produced by compaction. However, the overall seismic response of the soil deposit is governed by the characteristic of the input motion (distribution of energy with frequencies).

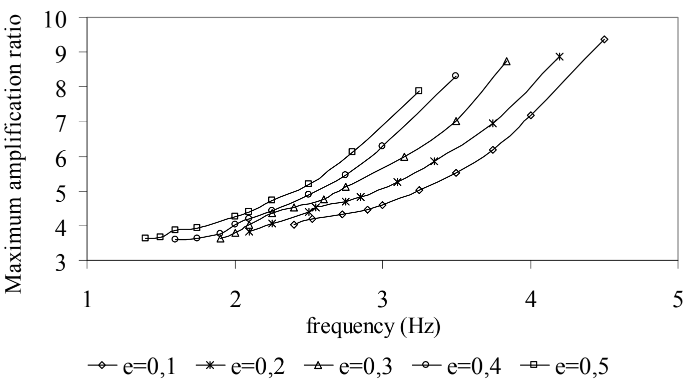
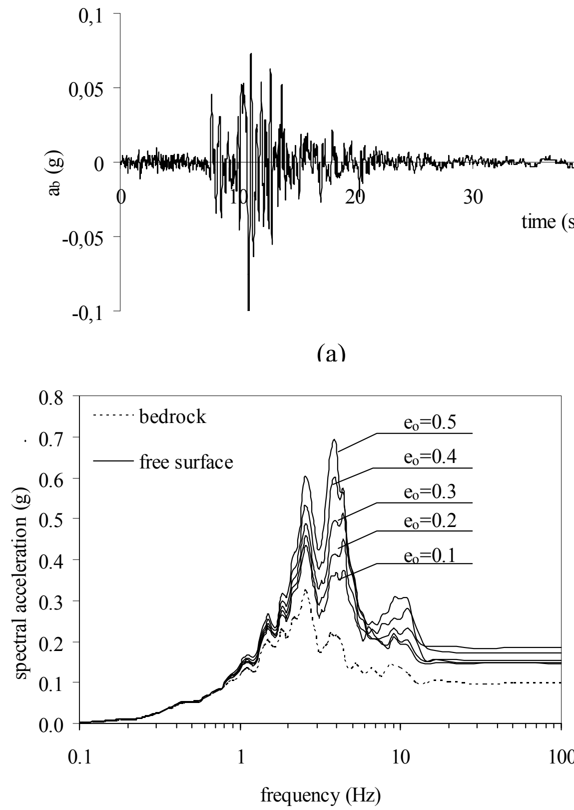


Fig. 8. Free surface maximum accelerations (a) and amplification ratios (b) calculated on a soil deposit (e=0.3) subjected to base input acceleration with harmonic time histories (eq. 11).

In order to extend the seismic analysis from the ideal harmonic input motion laws to more realistic ones, the acceleration time history recorded at Diamond Heights (E-W component) during the earthquake occurred in 1989 at Loma Prieta (CSMIP, 1991) has been assigned at the bedrock of the gravel deposit. In a first analysis the original time sequence has been normalised in order to give a peak acceleration of 0.1 g (Fig.9.a) and the spectral accelerations calculated at the bedrock and at the free surface are compared in Fig.9.b. This comparison show that progressively higher accelerations are obtained on looser soil deposits, with larger differences corresponding to the fundamental frequencies.

A more direct view on the effects of compaction is obtained by comparing the free surface acceleration time histories calculated by assigning the same input motion to deposits compacted at different e_o values. This analysis has been performed on the imaginary soil deposit by considering variable input motions at the bedrock, each of them obtained by proportionally scaling the original acceleration time history of Fig.9.a with a different factor. In Fig.10 the values of peak acceleration calculated at the free surfaces are plotted as functions of the peak bedrock acceleration for different eo values. The amplification given by the layered soil deposits can be clearly seen by the comparison of the calculated curves with the 1:1 line.



(b)

Fig. 9. Normalised acceleration time history (a_{b_peak}=0.1 g) at the bedrock (a) and acceleration spectra (b) calculated for Loma Prieta earthquake.

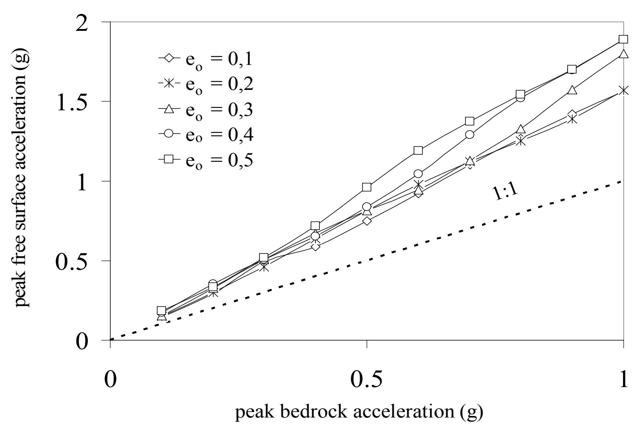


Fig. (10) – Seismic amplification effects of the gravel deposit for Loma Prieta earthquake.

As a general comment, larger amplifications are seen for soil deposits compacted in looser states (up to a_b=0.3g). More particularly, all curves follow an initial common trend, meaning that the effect of soil density is negligible for lower base accelerations. Progressively earlier detachment from this trend occurs on curves pertaining to soils in looser states. Therefore differences in the seismic amplification are enhanced by the combination of low soil density and high bedrock acceleration. It is however worth noting that these differences reach maximum values of the order of 25% (for a_{b_peak}= 1g), this results obviously dependent on the considered time history.

CONCLUSIONS

The stress strain behaviour of artificially compacted gravels as observed by laboratory investigations shows a particularly complex pattern, including non linear and irreversible relations at any stage of loading, dependency on the present and previously applied stresses, noticeable influence of the initial soil density. In the presented analysis the dependency of soil response on these factors has been simulated with an elasto-plastic model validated by a large number of experimental tests and calibrated for a crushed sandstone gravel. A one-dimensional seismic analysis has been then performed on a soil deposit imaginarily constituted with this gravel and subjected to variable motion laws at the underlying bedrock.

Calibration of equivalent visco-elastic parameters shows that small strain stiffness G_o is strongly influenced by the initial void ratios and by the depth from the top surface. Additionally, the curves expressing the decay of shear stiffness $[G_{eq}/G_o(\Delta\varepsilon_q)]$ and the increase of damping $[D_{eq}(\Delta\varepsilon_q)]$ show a significant influence of the mean effective stress but negligible dependency on the soil void ratio. These variations, which can be almost uniquely related to the state of gravel expressed by the variable ψ introduced in the elasto-plastic model, determine different seismic responses of the gravel deposit.

The finite difference dynamic analysis shows a behaviour governed by a complex interaction between input motion characteristics and soil stress-strain properties. Focusing more particularly on the effects of soil density, all the performed analysis show a beneficial effect of compaction. In particular, the parametric study conducted by assigning harmonic input accelerations with variable amplitudes and frequencies, reveals higher fundamental frequencies and lower amplification ratios for denser soils deposits.

This result is broadly confirmed by the analysis performed in the frequency and time domains for the truly recorded acceleration time history of Loma Prieta earthquake. The differences in terms of free surface peak accelerations among soil deposits compacted at different e_o are negligible for relatively small peak base acceleration (< 0.3g) and become more and more relevant with increasing a_b and e_o. However, the maximum differences for the considered case are in the order of about 25%.

As a final comment, the authors are aware that the simplification introduced in the adopted procedure could potentially affect the obtained results. Quantitative refinements will certainly come out by the implementation of the proposed elasto-plastic model into numerical codes capable of performing two or three dimensional analyses and by the inclusion of time effects (stress or strain rate dependency) on the stress-strain response of gravely soil (as recognised by Anh Dan et al., 2001). These improvements, which represent a future program of the research, do not limit the qualitative observation on the positive effects of compaction emerged from the present study.

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