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Evaluation of dynamic site response for seismic design of pipelines

I.V. Constantopoulos[a], F. Lagasco[b], F. Pelli[b]
[a]*Université Libre de Bruxelles, Belgium*
[b]*D'Appolonia S.p.A., Genova, Italy*

Abstract

The design of pipelines often requires evaluations of the seismic site response based on limited information. In particular, the maximum acceleration at the ground surface is a crucial parameter as it controls the seismic stability of the slopes and the triggering of liquefaction in cohesionless soils. A parametric seismic response study is presented, where the effects of soil type, G_{MAX}, soil deposit depth, maximum acceleration and acceleration time history at the bedrock on the maximum acceleration at the ground surface are investigated. Emphasis is placed on the effects of G_{MAX}, and the results of field measurements carried out both on shore and offshore are evaluated. The results of the study can provide preliminary indications on the seismic response of sites characterized by soil conditions similar to those investigated.

1 Introduction

Due to the presence of a soil deposit, the amplitude and the frequency contents of the earthquake motion at the ground surface may be considerably different than at the bedrock. Crucial design parameters such as the maximum acceleration at the ground surface are controlled by the depth of the soil deposit and soil stiffness. The magnitude of the maximum acceleration and the acceleration time history at the bedrock also affect the seismic response.

Seismic response analyses can be carried out keeping into account the local soil conditions as determined by means of detailed geotechnical and geophysical investigations. However, for structures extending hundreds of kilometers, such as pipelines, a detailed knowledge of the soil deposit characteristics along the route is seldom available. Therefore evaluations of the site seismic response based on limited information are often required. In particular, the seismic design of pipelines is based mainly on the maximum acceleration at the ground surface, which controls the seismic stability of the slopes and the triggering of liquefaction in cohesionless material.

The study presented herein includes a parametric response analysis, and was carried out to investigate the effects of soil type, soil shear stiffness at small strain G_{MAX}, depth of the soil formation, maximum acceleration and acceleration time history at the bedrock, on the maximum acceleration at the ground surface. The purpose was to develop a simplified methodology for approximate evaluations of the maximum acceleration at the ground surface, based on limited information such as the earthquake characteristics at the rock outcrop and a general knowledge of the site.

2 Soil Shear Modulus At Small Strain

A simplified relationship for G_{MAX} in cohesionless soil was first proposed by Seed and Idriss[1], and it is here re-formulated as follows:

$$G_{MAX} = 126.3 K_{2\max} \sqrt{\sigma'_v (1+2(K_o)_{NC} (OCR)^{\beta})} \tag{1}$$

where G_{MAX} and σ'_v (i.e., vertical effective stress) are in kPa, K_{2max} is an empirical constant and K_o is the coefficient of lateral earth pressure at rest. $(K_o)_{NC}$ = 0.4, where NC stands for Normally Consolidated, and $\beta = 0.45$ can be assumed (Lunne and Christoffersen[2]). Typical values of K_{2max} for loose to extremely dense sands (30 to 86) and for gravels (60 to 180) are given in the literature[1,3] .

G_{MAX} in wet clays is usually related to the undrained shear strength S_u. Data presented by Seed and Idriss[1] indicate G_{MAX}/S_u values ranging between 1500 and 3500 where data presented by Mayne and Rix[4] values ranging between 220 and 2500 are inferred. Data reported by Hardin[5] and summarized by Prakash[6] show that G_{MAX} in clay can be expressed as follows:

$$G_{MAX} = \alpha\, OCR^{k} \sqrt{p_a \gamma' z \left(\frac{1+2K_o}{3}\right)} \tag{2}$$

where k is taken as 0.25, γ' is the submerged unit weight and z is depth. $(K_o)_{NC}$ was given a value of 0.6, whereas K_o for overconsolidated soil can be defined as a function of $(K_o)_{NC}$ and OCR according to Lambe and Whitman[7]. The parameter α was defined as a function of the soil void ratio e, for the lower bound (α_{LB}), the upper bound (α_{UB}) and an intermediate case (α_{INT}) as $\alpha_{UB} = 1050/e$ and $\alpha_{LB} = \alpha_{INT} = 550/e$. Based on the selected field G_{MAX}

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values, the void ratio e was varied with depth between 1.4 and 0.65 for the upper bound and the intermediate case, and between 2.0 and 1.2 for the lower bound.

3 Selected Field Data

In order to calibrate and to verify the appropriateness of the selected G_{MAX} boundaries adopted in the parametric response analyses, and to investigate their range of applicability, shear wave velocity (V_s) field measurements carried out at 9 sites (29 locations in total), both on shore (by cross hole tests) and offshore (by seismic cone penetration tests) were collected.

In Figures 1a,b,c the boundary G_{MAX} curves selected for the parametric study are compared with the available field data for clays, sands and gravels respectively. Normally consolidated conditions were adopted for the lower bound curves, whereas overconsolidation ratios variable with depth and associated with a surface preload of 450 kPa were imposed for the upper bound. A preload of 450 kPa was also adopted for the intermediate line in sand. For clays, the intermediate line was defined considering a preload of 225 kPa. These limits proved to fit well the field data collected for verification and presented later in this section.

For the clays (Figure 1) data from 5 Sites (15 locations) are reported. Note that Sites 1 and 2 and some of the data of Site 6 between 0 and 40 meters plot between the lower bound and the intermediate line. The values for Site 6 plot higher than the others, which may be due to higher OCR's and silt contents if compared with Sites 1 and 2. Sites 3 and 4 plot above the intermediate line, due to the higher OCR's and older age of the deposits. Below 60 meters of depth, Site 6 plots above the intermediate line, probably due to age and to the high silt content that tend to increase with depth. The lower range (lower to intermediate boundaries) seems appropriate for soft, relatively recent deposits, normally consolidated or moderately overconsolidated. The higher range (intermediate to higher boundaries) is appropriate for older, overconsolidated deposits. Aged silts also appear to be stiffer than relatively young normally consolidated clays.

For the sands (Figure 2) the three boundary lines were obtained assuming K_{2max} = 30, 55 and 86 for lower, intermediate and upper boundary respectively. The G_{MAX} data recorded at 2 Sites (11 locations) are shown. Site 7 falls quite reasonably in the upper range, that accordingly is representative of dense to very dense sands[3]. Site 6 plots mostly in the lower range, although the data are scattered and cover the whole area between lower and upper bounds. These differences are probably due to different relative density and grain size. The lower and upper ranges defined in Figure 2 can be taken as representative of "soft" (loose, silty, normally consolidated, recently deposited) and "stiff" (dense, overconsolidated, old) sandy deposits, respectively.

In Figure 3 curves computed for gravel assuming K_{2max} = 60, 80 and 180 are shown. In this case data from only one site (9 locations) are available, where the soil is composed of sand and gravel. The data show extensive scattering but mostly keep within the given upper and lower bounds. A lower bound as given by the intermediate curve may be acceptable in many instances where dense to very dense gravelly soils are considered. The data available, however, do not allow to investigate the latter aspect in further detail.

The considerations presented were found to be not applicable to 2 of the selected sites, one in clay and one in sand and gravel. The high G_{MAX} values measured in the clay site are believed to be due to high carbonate content in excess of 40 per cent and often higher than 60-80 per cent. In the other cases sand cementation is the most likely cause of high G_{MAX}.

4 Other Soil Parameters

The dependency of the soil shear modulus and damping ratio on shear strain can be taken into account in a simplified manner, by defining G/G_{MAX} and D/D_{max} curves as a function of the effective shear strain γ_{eff}. For this study the curves given by Hardin and Drnevich[8] have been adopted for sands and gravels (taken as "sand") as well as for clays. In order to prevent numerical problems during the response analyses, the G values were not allowed to decrease below 10 per cent of G_{MAX}. The values of D_{max} were also estimated based on Hardin and Drnevich[8]. These assumptions are deemed sufficiently accurate for the purposes of the present study, also in consideration of the relatively wide uncertainty range characterizing other parameters, such as G_{MAX}, for each soil type.

5 Parametric Seismic Response Analyses

The analyses were carried out using the computer program SHAKE[9] which solves the vertical propagation of shear waves in horizontally layered media. In this program the soil non linearity is taken into account by an approximate, linear-equivalent procedure.

The use of artificial acceleration time histories appeared inappropriate, as the frequency content of the applied motion influences sensibly the seismic response of the soil column. Therefore three different real acceleration time histories (San Fernando CA (1971), Parkfield CA (1966), and Sitka-Alaska (1972)) recorded on rock were used for the analyses, and their spectra are shown in Figure 4. In the same figure the spectrum on rock recommended by the API[10] is shown for comparison.

The parametric study was carried out for three different thicknesses of the soil deposit, 40, 80 and 120 meters respectively. Three different peak ground accelerations at the rock outcrop, 0.2, 0.3 and 0.4, where used for the analyses by linearly scaling the acceleration time histories to these values.

6 Discussion of the Results

The main results obtained from 243 SHAKE runs are presented in Tables 1, 2 and 3 for clayey, sandy and gravelly sites, in terms of ranges of peak ground acceleration at the surface, PGA_s given the peak ground acceleration at the outcrop PGA_0 based on the three selected acceleration time histories. It can be observed that PGA_s increases slightly non-linearly with PGA_o, and decreases with deposit depth H. PGA_s increases notably when passing from clay columns to sand and to gravel columns. Moreover, with respect to the influence of the variability of the selected parameters (deposit depth, H; PGA_o values; soil stiffness, G_{MAX}; input acceleration time history, acceleration time history) on PGA_s the following may be noted:

- the PGA_s scattering due to different input acceleration time histories is considerable; the difference among results given by the three acceleration time histories, having fixed the other parameters, may differ up to 25 percent for clay and up to 20 percent for sand and gravel;

- uncertainty on PGA_o has a sensitive influence on PGA_s; variability on PGA_s ranges between 20 to 60 percent when comparing values obtained with PGA_o equal to 0.2 and 0.4, keeping the other parameters fixed;

- the deposit depth H has a relatively low influence on PGA_s variability, in particular for sand and gravel; the percentual variation of PGA_s comparing values obtained for H equal to 40 and 120 meters is generally within 20 percent;

- the variability of G_{MAX} between lower and upper bound, dominates the data scattering, in particular for clay; the percentual variation of PGA_s comparing values obtained for the lower and upper bound of G_{MAX}, varies between 75÷130 percent for clay, 20 ÷ 60 percent for sand, 10 ÷ 25 percent for gravel.

7 Conclusions

The results of the parametric seismic response study presented herein can provide useful indications on site response, given that some knowledge of soil type, depth of soil deposit and soil stress history are available. Most emphasis was placed on soil type and on the shear modulus at small strain G_{MAX}. The variability of G_{MAX} was found to dominate data scattering. Further work in exploring the effects of other parameters, such as the acceleration time history at the bedrock, is needed. The acceleration time history at the bedrock has a considerable effect on the response, and therefore its selection for a specific site should be given careful consideration. During this study it was found that the presence of considerable carbonate content in clays and of limited cementation in sand may lead to considerably high G_{MAX} values. The results presented in this study do not apply to such cases.

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Table 1. PGA_s versus PGA_0 for clayey soil

LAYER THICKNESS H (m)	PGA_o/g (-)	PGA_s/g (-)	RANGE OF PGA_s/g	RELATIVE SCATTER (%)
40	0.2	0.18	0.12 ÷ 0.24	71
	0.3	0.24	0.14 ÷ 0.34	84
	0.4	0.28	0.15 ÷ 0.40	91
80	0.2	0.16	0.08 ÷ 0.24	96
	0.3	0.20	0.10 ÷ 0.31	105
	0.4	0.24	0.12 ÷ 0.36	101
120	0.2	0.16	0.09 ÷ 0.22	83
	0.3	0.19	0.11 ÷ 0.28	88
	0.4	0.22	0.12 ÷ 0.32	91

Table 2. PGA_s versus PGA_0 for sandy soil

LAYER THICKNESS H (m)	PGA_o/g (-)	PGA_s/g (-)	RANGE OF PGA_s/g	RELATIVE SCATTER (%)
40	0.2	0.24	0.17 ÷ 0.30	59
	0.3	0.30	0.21 ÷ 0.40	64
	0.4	0.35	0.22 ÷ 0.47	70
80	0.2	0.25	0.19 ÷ 0.31	49
	0.3	0.29	0.22 ÷ 0.36	46
	0.4	0.34	0.24 ÷ 0.42	52
120	0.2	0.23	0.20 ÷ 0.28	35
	0.3	0.29	0.22 ÷ 0.36	46
	0.4	0.31	0.21 ÷ 0.41	62

Table 3. PGA_s versus PGA_0 for gravelly soil

LAYER THICKNESS H (m)	PGA_o/g (-)	PGA_s/g (-)	RANGE OF PGA_s/g	RELATIVE SCATTER(1) (%)
40	0.2	0.28	0.24 ÷ 0.34	35
	0.3	0.32	0.27 ÷ 0.36	28
	0.4	0.35	0.29 ÷ 0.43	39
80	0.2	0.26	0.20 ÷ 0.32	48
	0.3	0.32	0.26 ÷ 0.38	37
	0.4	0.36	0.29 ÷ 0.43	41
120	0.2	0.29	0.24 ÷ 0.35	36
	0.3	0.31	0.24 ÷ 0.37	39
	0.4	0.36	0.30 ÷ 0.42	34

Note (1) Relative scatter refers to: $\dfrac{PGA_S^{UB} - PGA_S^{LB}}{PGA_S^{INT}}$

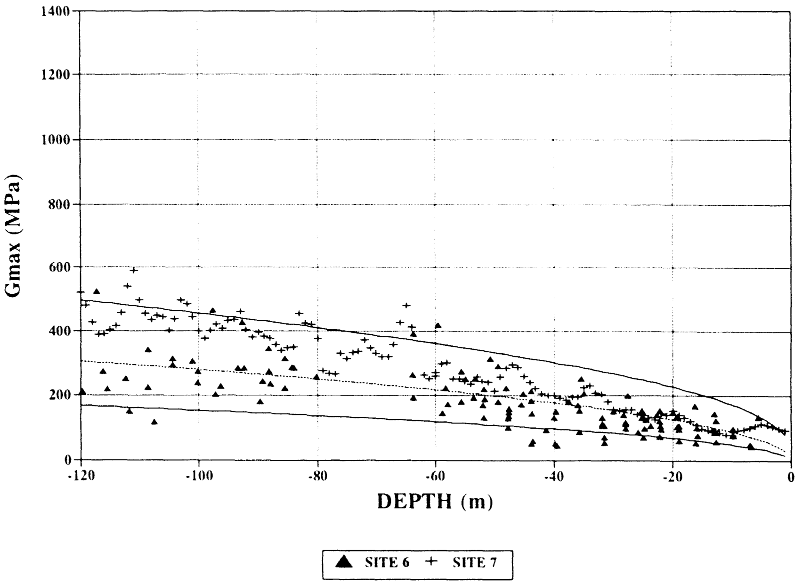

Figure 1: Selected G_{MAX} curves for clay.

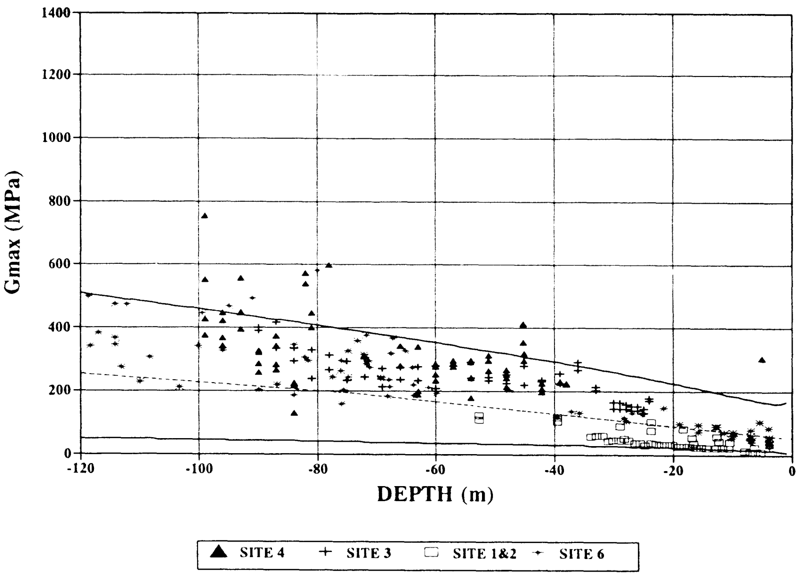

Figure 2: Selected G_{MAX} curves for sand.

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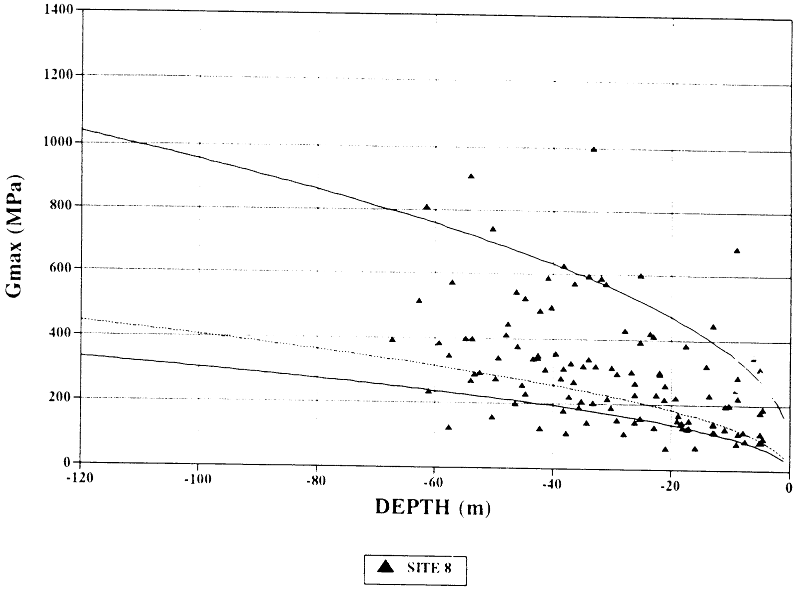

Figure 3: Selected G_{MAX} curves for gravel.

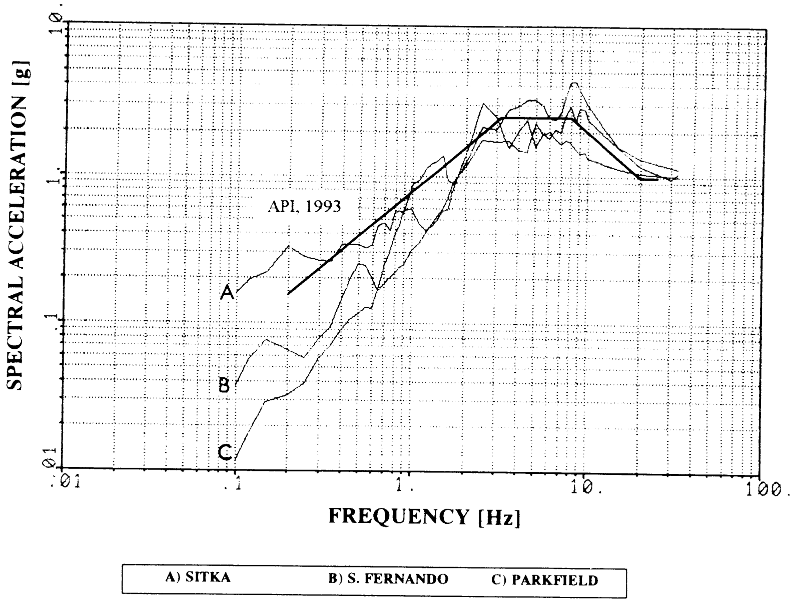

Figure 4: Real time history spectra on rock.