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LIMITING VALUES ON EARTHQUAKE GROUND MOTION DUE TO STRENGTH OF NEAR-SURFACE MATERIAL

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ABSTRACT

The need to determine upper bounds on ground surface acceleration, especially for constructions with very low annual frequencies of exceedance, is essential. In this paper a short description regarding the model proposed by Pecker (2005) for the determination of upper limits on ground surface motion along with its basic principles and assumptions is presented and explained. It is based on the notion that if the shear stress induced to the ground by the shear waves exceeds the shear strength of the soil, then failure occurs. At this failure state, the acceleration transmitted to the ground has reached an upper limit.

The model was applied to several sites from the U.S.A. and Japan where major earthquake motions were recorded. Information from geotechnical site investigations was used in order to estimate the model input parameters. In cases where some of the required data was unavailable, parametric studies were performed. The limiting values on ground surface acceleration were estimated according to the model introduced by Pecker and compared with the actual acceleration records of the sites as well as the results of 1D elastic perfectly plastic numerical analysis.

Keywords: maximum ground surface acceleration, upper limits, soil shear strength

INTRODUCTION

Constructions such as the nuclear waste repository in the Nevada desert and the nuclear power plants in Switzerland are huge risk projects that demand a thorough grip of the seismic hazard. The ground motion that such constructions should be subjected to should correspond to very low annual frequencies of exceedance like 10^{-8} to 10^{-7} and thus, the analysis results in values as extreme as 20g (Bommer et al., 2004). These values indicate the need to establish upper bounds on earthquake ground motion.

At that point an essential distinction has to be made: When the source and the travel path (Bommer et al., 2004) are taken into consideration, then the term 'bounding ground motion' seems to be more appropriate for describing the maximum ground acceleration at a rock site. On the contrary, when the shear strength of the soil at the site is regarded as a physical constraint for ground acceleration then the most suitable term is 'upper limit'.

The present paper deals only with upper limits, based on the fact that as shear waves propagate from the source to the earth's surface, they induce shear stress to the soil. According to the limit strength theory, if the shear stress exceeds the shear strength of the soil, then failure occurs. At failure state, the acceleration transmitted to the ground has reached an upper limit. Even though the acceleration is

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limited by the maximum shear stress reached by the soil, displacement is not, hence, great displacements can occur at the surface as the soil keeps shearing under constant shear stress.

Pecker (2005) developed an analytical model to estimate the upper limit of ground surface acceleration based on the limited capability of the ground to transmit shear stress. The method is based on a simple soil constitutive behaviour (linear elastic – perfectly plastic) and the input parameters can be easily obtained from common laboratory tests.

BRIEF PRESENTATION OF THE MODEL

The model was first introduced by Pecker (2003), providing estimation of maximum ground motion for a soil profile with zero shear strength at the surface. The model was further developed including soil profiles with non-zero strength at the surface (Pecker, 2004). The most thorough reference is Pecker (2005) which provides full presentation of the model along with some applications to actual sites. The detailed derivation of the model can be found there and will not be presented in detail herein. Thus, only a brief presentation will be provided in the following paragraphs.

Constitutive soil model

The soil layer is assumed to be isotropic, linear elastic perfectly plastic. Thus, only two parameters that might vary with depth (z) are sufficient to fully define the simplified constitutive soil behaviour:

1. the maximum shear strength $\tau_{max}(z)$
2. the yield shear strain $\gamma_f(z)$

Even though the yield strain is a straightforward parameter, the choice of the appropriate $\tau_{max}(z)$ is rather ambiguous since it depends on the soil type and on the degree of saturation. Pecker (2005) has provided discussion on how to make this choice: The drained shear strength is the unchallengeable for a dry granular soil. Similarly, the cyclic undrained shear strength $c_{u,c}$ seems to be the right choice for a saturated clayey soil. The debate arises for the case of saturated granular soils. There are three choices namely the cyclic undrained shear strength, the monotonic undrained shear strength and the drained strength. Since the object of research is the maximum response, it seems more appropriate to choose the maximum strength. So, the first one is more appropriate for the case of loose saturated soils that are likely to liquefy. However, for dense soils the monotonic undrained shear strength is probably greater than the other two and thus more appropriate.

Soil profile

It is assumed that a soil layer of thickness h overlies a semi-infinite medium (bedrock). The bedrock is considered to be rigid, so there is no radiation damping and the elastic wave energy is trapped inside the soil. The soil layer is horizontal and its stiffness increases with depth.

The shear wave velocity V(z) at any depth z (which corresponds to the yield strain and should not be confused with the small strain shear velocity obtained by seismic geophysical tests) can be estimated according to the parameters controlling the constitutive soil behaviour as follows:

$$V(z) = \sqrt{\frac{G(z)}{\rho}} = \sqrt{\frac{\tau_{\max}(z)/\gamma_f}{\rho}} \tag{1}$$

The distribution of the shear wave velocity V(z) of the soil layer at depth z is assumed to comply with the following formula and is shown in Fig. 1:

$$V(z) = V_s \cdot \left(\frac{z+d}{d+h}\right)^{p/2} = V_s \cdot \left(\frac{z+d}{H}\right)^{p/2} \tag{2}$$

where d is a positive parameter, p is a real positive parameter ($0<p<2$) and V_s is the shear wave velocity at the bedrock-soil interface.

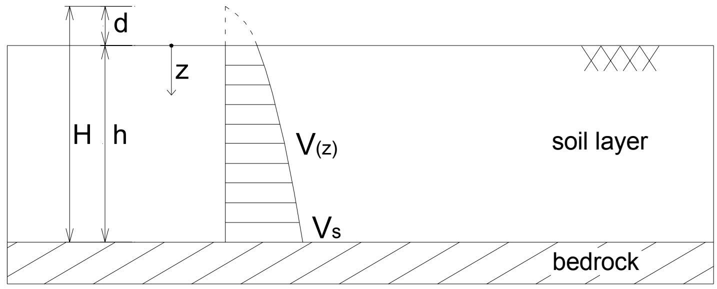

Figure 1. Cartoon of model parameters

Input motion

The displacement u of the soil depends on both depth z and time t, each of affect it independently according to the equation:

$$u(z,t) = X(z) \cdot y(t) \tag{3}$$

A solution for y(t) can be obtained based on the input motion $\ddot{u}_g(t)$ that the bedrock transmits to the soil layer by using Duhamel's integral. However, since only the maximum response needs to be determined for the particular problem, a spectral analysis is adopted. Thus, the pseudo-acceleration response spectrum of each site is also required.

Determination of the maximum ground surface acceleration

The model provides the shape of the X(z) distribution with depth for each of the N modes considered as well as the mode participation factors. Since the eigenfrequency ω_i of the i mode is known, the spectral pseudo-acceleration $S_a(\omega_i,\xi_i)$ that corresponds to this mode can be determined. The estimation of the above parameters leads to the determination of the maximum shear strain distribution with depth $\gamma(z)$ and of the input acceleration S^*_{ai}.

The maximum shear strain is compared with the yield strain of the soil and parameter μ is estimated, which shows how much $\gamma(z)$ must be increased in order to reach the yield strain value $\gamma_f(z)$. Finally, the maximum ground surface acceleration $\ddot{u}_{max}$ is determined according to the new input acceleration which is μ times the initial value S^*_a.

APPLICATION TO ACTUAL SITES

The model by Pecker described above provides the maximum ground surface acceleration depending on the site soil properties and the design spectrum of the site. In order to examine if the model represents reality sufficiently, the estimated surface acceleration $\ddot{u}_{pre}$ for a site obtained by the model should be compared with the maximum acceleration $\ddot{u}_{rec}$ ever recorded at the specific site's surface. As Bommer et al. (2004) point out 'the maximum recorded motions provide lower bounds on the upper limits'.

Since the model derives the upper limit for ground motion, its reliability should be tested on sites where large earthquakes have occurred. In this way, it is more likely that the input motion has forced the soil to fail and thus, the $\ddot{u}_{rec}$ is mainly controlled by the soil strength rather than by the source and the travel path. Finally, it should be kept in mind that both $\ddot{u}_{rec}$ and $\ddot{u}_{pre}$ are affected by the frequency content of the input motion. Thus, a similarity between the earthquake motion that produced $\ddot{u}_{rec}$ and the pseudo-acceleration response spectrum used for the calculation of $\ddot{u}_{pre}$ should exist.

Despite the small number of required model input parameters, finding sites that fulfilled the aforementioned requirements was a difficult task. The greatest obstacle was the lack of geotechnical data in sites where strong motion had occurred. Thus, twenty-six stations were chosen for analysis according to the geotechnical information provided and the strong motion recorded there. Fifteen of those were located in Japan and the rest in California, U.S.A.

Maximum recorded surface acceleration

The strong motion recordings from California were retrieved from the PEER, COSMOS and USC websites. The geological and geotechnical data for sites very near the stations were found at the ROSRINE website. All data for the Japanese sites are taken from the Kyoshin Network website. Tables 1 and 2 show the station names and maximum PGA values recorded in Japan and California respectively. The events chosen have moment magnitudes between 5.9 and 8.0 while PGA values vary from 0.26g to 1.15g.

Table 1. Stations in Japan

STATION CODE	STATION NAME	MAX PGA (g)	EARTHQUAKE (DATE)-MAGNITUDE
MYG007	Toyosato	0.26	Miyagi-oki (26/05/2003) -M_w7.0
IWT001	Taneichi	0.27	Miyagi-oki (26/05/2003) - M_w7.0
IWT006	Yamada	0.35	Miyagi-oki (26/05/2003) - M_w7.0
IWT014	Ishidoriya	0.26	Miyagi-oki (26/05/2003) - M_w7.0
IWT016	Kawai	0.39	Miyagi-oki (26/05/2003) - M_w7.0
HKD020	Minatomachi	1.15	(14/12/2004) - M_w6.1
HKD038	Minamifurano	0.20	Tokachi-oki (26/09/2003) - M_w8.0
HKD067	Nakashibetsu	0.34	(28/01/2000) - M_w6.8
HKD071	Atsutoko	0.42	Hokkaido (29/11/2004) - M_w7.1
HKD073	Nemuro	0.35	Hokkaido (29/11/2004) -M_w7.1
HKD074	Nosappu	0.56	Hokkaido (29/11/2004) -M_w7.1
HKD079	Shibeccha	0.19	(06/12/2004) -M_w6.9
HKD084	Akan	0.36	Tokachi-oki (26/09/2003) - M_w8.0
HKD086	Chokubetsu	0.80	Tokachi-oki (26/09/2003) - M_w8.0
HKD090	Hombetsu	0.48	Tokachi-oki (26/09/2003) - M_w8.0

Table 2. Stations in California, U.S.A.

STATION CODE	STATION NAME	MAX PGA (g)	EARTHQUAKE (DATE)-MAGNITUDE
ARL	Arleta	0.59	Northridge (17/01/1994) - M_w6.6
HLV	Halls Valley	0.13	Loma Prieta (17/10/1989) - M_w6.9
IBM	Santa Teresa Hills	0.28	Loma Prieta (17/10/1989) - M_w6.9
JST	Joshua Tree	0.32	Desert Hot Springs (22/04/1992) - M_L6.1
KAG	Kagel Canyon	0.43	Northridge (17/01/1994) - M_w6.6
NWH	Newhall Fire Station	0.59	Northridge (17/01/1994) - M_w6.6
YRM	Yermo	0.25	Landers (28/06/1992) - M_w7.3
PIC	Portrero 1 (SMA site)	0.42	Northridge (17/01/1994) - M_w6.6
LBM	Bell Bulk Mail Facility	0.45	Whittier Narrows (01/10/1987) - M_w5.9
SMT	Superstition Mountain	0.91	Superstition Hill (24/11/1987) - M_w6.5
RRS2	Rinaldi 2	0.84	Northridge (17/01/1994) - M_w6.6

Selection of input parameters

The input parameters for the model regarding the geotechnical properties of the soil layer are the shear strength distribution with depth and the soil yield strain. The shear strength distribution is needed in order to define the shear wave velocity profile V(z). Even though these parameters are easily defined by common laboratory tests, the data required prove difficult to find. Furthermore, the pseudo-acceleration response spectrum corresponding to each station needs to be determined.

Shear strength distribution

Due to the lack of data, the shear strength distribution was obtained based on Standard Penetration Tests that are more commonly performed and are easier to retrieve as opposed to explicit strength values. In order to overcome the scatter in the results of the SPT and the uncertainty in the corrections, the analysis was also performed for a 30% increase and decrease of the measured N_{SPT} values.

For dry granular soils, the angle of shearing resistance can be determined according to Stroud (1988) based on N_{SPT} values.

As has been mentioned above, for saturated granular soils, the most appropriate shear strength seems to be the monotonic undrained strength for dense sands and the cyclic undrained strength for loose sands. Even though SPT results provide a good estimate of the soil density and thus, of the kind of shear strength that should be used is known, no correlation was found linking kind of strength to N_{SPT} values. Since no other option was available, the analysis was performed according to the monotonic drained strength.

For clayey soils, the cyclic undrained shear strength $c_{u,c}$ is adopted, which is related to the monotonic undrained shear strength $c_{u,m}$ and to the N_{SPT} values by the equation (Pecker, personal communication):

$$c_{u,c} = 0,9 \cdot c_{u,m} = 0,9 \cdot (7 \cdot N_{SPT}) \tag{8}$$

Water table

One additional obstacle in determining the shear strength is that the water level is not given. Thus, two analyses were performed, one assuming that the water level is below the depth considered and one assuming it at ground surface.

Yield strain

No data were available regarding the yield strain, so it was decided to adopt yield strains from the literature since their values do not greatly affect the results (Pecker, 2005). The variation and the median values of yield strain that were used in the analysis for granular soils and for soft and stiff clayey soils are shown in Table 3. The values were given as a rough estimation by Dr Coop, Imperial College (personal communication).

Table 3. Yield strain

Soil type	minimum γ_f	median γ_f	maximum γ_f
Soft clay	4.5%	6.0%	7.5%
Stiff clay	0.75%	2.0%	3.0%
Sand	1.5%	2.2%	3.0%

Distribution of shear wave velocity with depth

The shear wave velocity profile used in the model should be subjected to an exponential distribution given by equation (2). However, the depth under consideration may consist of different soil layers, resulting in an estimated shear wave velocity profile that deviates from the theoretical exponential distribution. A least-square method was adopted in order to define the values of parameters p and d which produce the best fit of the measured shear wave velocity profile with the theoretical one. It is worth mentioning that most sites under consideration provided a good fit with the theoretical distribution. Problems only arose with some sites containing thin layers of weak soil between soils of greater strength. These sites were excluded from the dataset. Sites excluded from the dataset also included all of those for which information was found indicating possible liquefaction phenomena.

Pseudo-acceleration response spectrum
In lieu of a pseudo-acceleration response spectrum, the design response spectrum of each country was adopted (UBC '97, Japanese code) customised to the requirements of each site as suggested by Pecker (2003b). The 20% damping ratio that is more appropriate for a soil near the failure condition (Pecker, 2005), was imposed to the design response spectra by scaling factors applied to the ordinates of the spectral plateau according to Bommer and Mendis (2004):

$$D = \sqrt{\frac{10}{5+\xi}} = \sqrt{\frac{10}{5+20}} = 0.632 \tag{9}$$

Analyses
Table 4 illustrates the set of analyses performed in order to overcome the uncertainty in the input parameters and to examine the sensitivity of the model to them.

Table 4. Set of analyses

	N_{SPT}			Yield strain (%)			Water table	
	-30%	given	+30%	min	med	max	at the bedrock	at the surface
Analysis 1		✓			✓		✓	
Analysis 2	✓				✓		✓	
Analysis 3			✓		✓		✓	
Analysis 4		✓		✓			✓	
Analysis 5		✓				✓	✓	
Analysis 6		✓			✓			✓

Results
Effect of water level
Figure 4 illustrates the values of ground surface acceleration predicted by the model for analysis 1 and analysis 6 in comparison with the maximum acceleration values recorded at the site. The 1:1 line constitutes the threshold above which the PGA limit values predicted by the model are actually smaller than the recorded ones.

In analysis 1, the majority of sites lie well below the line, in good agreement with the model. One site (IBM) lies far away beneath the line. This is an indication that the site may not have received the maximum motion it can sustain or that the model is very conservative in the particular case. Four sites, namely HKD071, HKD074, HKD084 & HKD090 lie slightly above this line.

If the water table is assumed to be at the surface -analysis 6- then the results change notably since the strength of the soil layer is highly reduced, especially in sites where the granular soil dominates. Indeed, almost half of the sites under consideration show that the model underestimates the motion they can sustain. So, at first sight, it can be concluded that the model fails to predict the upper limit at these sites.

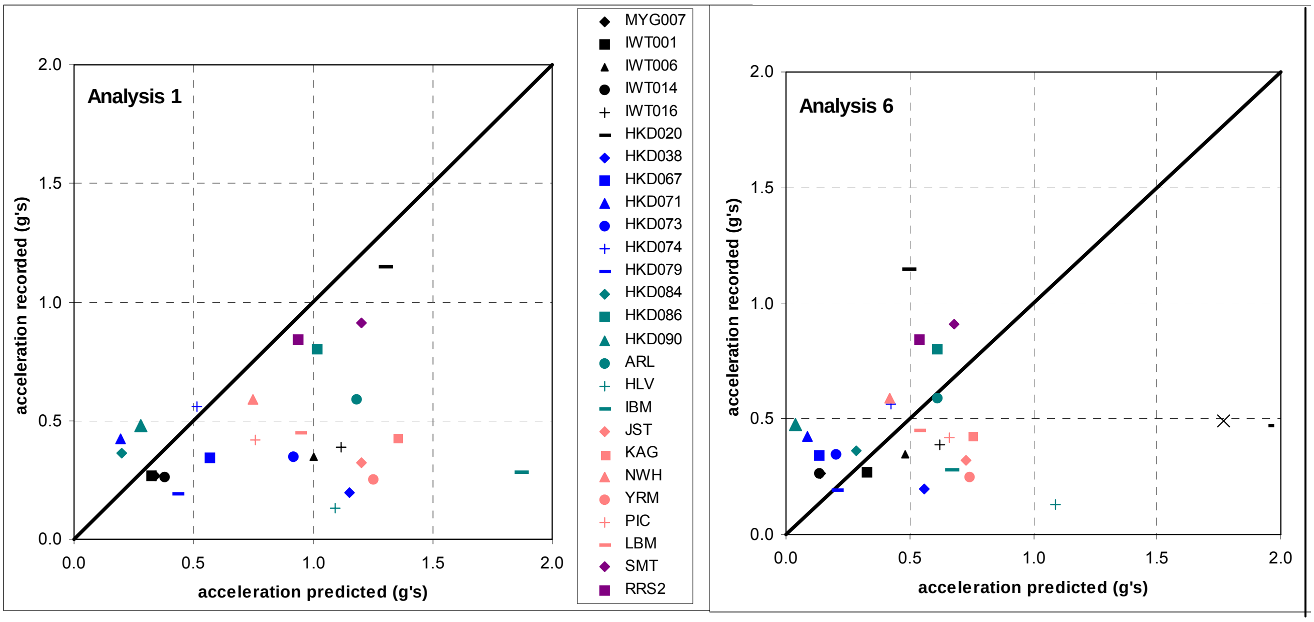

Figure 4. Results of analyses 1 and 6 (effect of water table)

necessary to make assumption as to the level of water. If it is assumed that the water level in the actual sites is below bedrock, then according to analysis 1 the results are in good agreement with the model predictions since greater accelerations were recorded only in four sites. However, if the water table is near the surface, then according to analysis 6, 13 sites are found to lie above the limit. But a closer look at these sites may reveal further issues. Indeed, sites HKD020 and HKD090, which lie far from the line in analysis 6, consist of saturated granular soil with high SPT values which is evidence of high density, which is the case for most of these 13 sites. As has been stated above, for saturated dense granular soils, the drained shear strength was used in the analysis, even though the undrained shear strength seemed to be more appropriate. For dense soils, the undrained strength will be much greater than the drained and thus, if the first one was used then the predicted PGA values would have been much greater than the ones shown in analysis 6. So, the discrepancy between the model and the recordings does not mean that the model malfunctions but is a great indication that the undrained shear strength should be used in order to predict the sustainable motion of dense saturated granular soils.

It is important to state that analysis 6 is an extreme case and that, in general, the water table would probably lie at a level between the two extreme cases described in analyses 1 and 6. Thus, the results would also lie roughly between the values obtained from the two analyses, which would bring most sites closer to the line. It is also worth mentioning that in the case of HKD090 site, the P-wave and V-wave velocity distributions seem to indicate that the water table lies at a relatively shallow depth of 3m. The level of the water was inferred by un unusually abrupt increase in the V_p values in conjunction with very low V_s and SPT values (Kramer, 1996, p.193). This may imply that if the undrained strength had been used, which for a nearly saturated sandy soil is the correct one, then the model may have yielded a result nearer to the recorded value.

Effect of SPT and γ_y uncertainty

Four more analyses were performed accounting for errors in SPT values and yield strain. The effect of these uncertainties in the results is shown in Fig. 5.

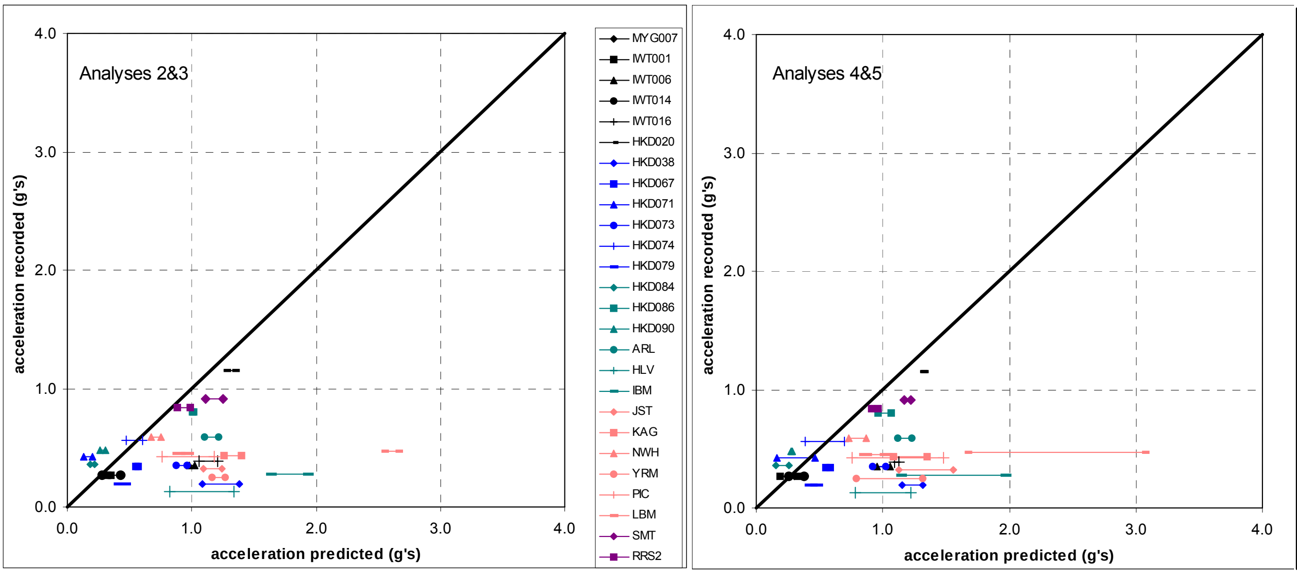

Figure 5. Results of analyses 2,3,4,5 (effect of parameter uncertainty)

In analyses 3 and 4, according to Fig.5, the ±30% fluctuation of N_{STP} produces a small fluctuation in the upper limits since an increase in SPT values corresponds to an increase in strength. The two analyses yield results that vary by 0.15g on average for most sites ranging from almost zero up to 0.5g for specific sites. The difference in the results is relatively small and thus, it can be concluded that the 30% variation in the SPT values does not affect the predicted acceleration significantly for most sites.

Analyses 4 and 5 correspond to the minimum and maximum yield strain. The effect of yield strain is two-fold. As the yield strain increases, the shear modulus decreases and thus, the shear wave velocity decreases. On the other hand, an increase in the yield strain results in a rise of shear strain values that causes the soil to fail. Conclusively, yield strain variation affects the upper limits in a contradictive way but in general, increase in yield strain tends to increase the predicted PGA. Pecker (2005) states that the model is not sensitive to yield strain variations. In the present analyses the results varied on average by 22%, reaching values up to 50%. This high proportion can be explained by two facts: firstly, the variation of yield strain in the three types of soils considered was very large in order to eliminate the uncertainty regarding this input parameter. Since the input parameter shows great variation, significant variations were expected in the results despite the small sensitivity of the model. Secondly, sites as SAT, LCN and IBM show considerably high variation which results not only by the change in yield strain values but mostly by the fact that different kinds of soil exist at the specific sites and the variation in yield strain differs according to each kind of soil. In other words, the great difference in results for some sites emanates mainly from the change in the p and d parameters that control the distribution of shear wave velocity due to the different variation that the yield strain has for each kind of soil.

NUMERICAL APPROACH

In an attempt to compare results obtained by the aforementioned analytical model with numerical method results, certain analyses were performed on a fictitious simplified soil profile and results were obtained using both the theoretical model of Pecker (2005) and a 1-D finite element model.

The simplified soil profile used for the comparisons consists of a single clay layer, 20m thick, overlying bedrock. The soil has a constant yield strain of 20% so as to comply with the soil profile of Pecker's model. The soil strength increases linearly with depth according to this formula: $c_u=10+10z$ (kPa).
The soil layer is discretised into a finite element mesh of one-dimensional shear elements whose constitutive relation is described by an elastic-perfectly plastic law (Owen & Hinton, 1986), assuming

that the strain energy is transmitted in the form of pure shear waves propagating in the axial direction, while the dynamic loading is introduced by means of the fixed end support excitation (Clough & Penzien, 1993). A Newmark integration scheme with δ=0.5 and α=0.25, enhanced with the typical Newton-Raphson algorithm (Bathe, 1996), is employed to solve the non-linear system of dynamic equilibrium equations. Using the return mapping integration algorithm (Simo & Hughes, 1998) the stresses are updated, while convergence is checked throughout the analysis by the energy norm. The algorithm was implemented in *nemesis* (nemesis, 2006), which is a general-purpose experimental finite element code to be found at www.nemesis-project.org.

In order to comply with the assumptions made in Pecker's model, damping was excluded from all analyses. Also, the F.E. analysis was terminated as soon as the first plastification occurred.

Seventeen strong motions were applied as input to this model. Eleven of them were actual earthquake recordings, most of which correspond to the large events from California studied previously, and were retrieved from the PEER database. The remaining six motions used were synthetic accelerograms created through the EQUSIMU Software for Artificial Ground Motion Generation (Joint BG/GR ERAN Project website) for large magnitude events and different durations. The input motions can be seen in Table 5.

Table 5. Input motions used for numerical analyses in *nemesis*

#	Name	Earthquake name	Date	Name of station	Database
1	synth01	Synthetic: M=6.0, t=20s	-	-	EQUSIMU
2	synth02	Synthetic: M=6.0, t=50s	-	-	EQUSIMU
3	synth03	Synthetic: M=6.5, t=20s	-	-	EQUSIMU
4	synth04	Synthetic: M=6.5, t=50s	-	-	EQUSIMU
5	synth05	Synthetic: M=7.0, t=20s	-	-	EQUSIMU
6	synth06	Synthetic: M=7.0, t=50s	-	-	EQUSIMU
7	chichi	Chi-chi	09/20/99	TCU065	PEER
8	coalinga	Coalinga	05/02/83	USGS STATION 1162	PEER
9	ducze	Ducze	11/12/99	BOLU, 090 (ERD)	PEER
10	gilroy	Loma Prieta	10/18/89	GILROY ARRAY #3	PEER
11	kobe	Kobe	01/16/95	TAKARAZU	PEER
12	landers	Landers	06/28/92	YERMO FIRE STATION	PEER
13	plamsprings	Palm Springs	07/08/86	N PALM SPR	PEER
14	northridge	Northridge	01/17/94	USC STATION 90056	PEER
15	sanfernando	San Fernando	02/09/71	CDMG STATION 279	PEER
16	superstition	Superstition Hills	11/24/87	USGS STATION 286	PEER
17	whittier	Whittier Narrows	10/01/87	CDMG STATION 24436	PEER

The results to be compared concern the maximum PGA value reached during the analysis as well as the failure depth. The values calculated using the theoretical and numerical model are illustrated in Fig. 6 and 7 respectively.

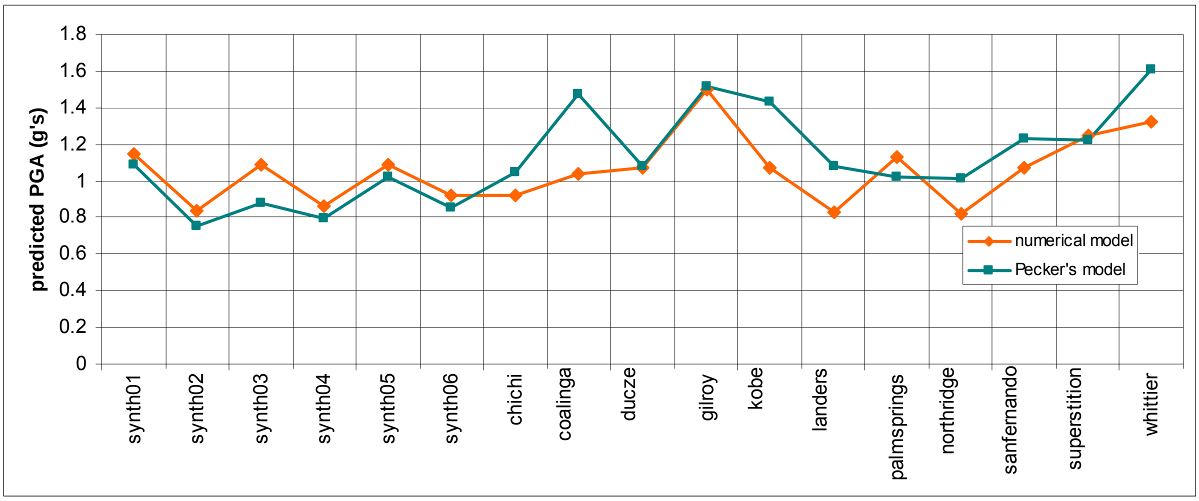

Figure 6. Comparison of PGA values as predicted by the two models

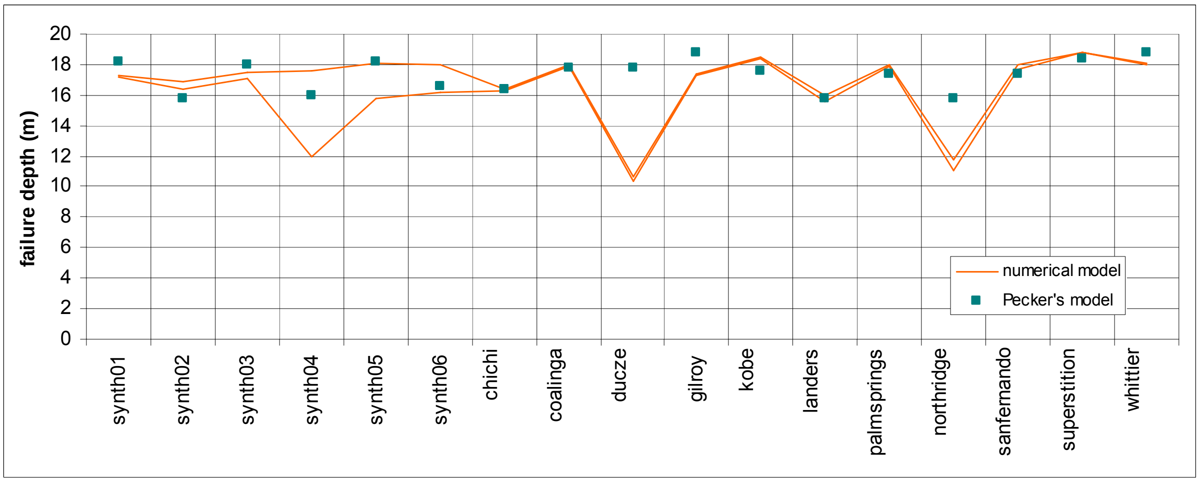

Figure 7. Comparison of the failure depth as estimated by the two models

It is seen from these figures that the results of the two models are in good agreement. The mean error estimated for the maximum PGA was found to be 13%. However, if the post-yield behaviour is taken into account, the PGA obtained from the numerical analysis increased.

CONCLUSIONS

The theoretical model of Pecker (2005) is applied to several sites from the U.S.A. and Japan where major earthquake motions were recorded. Information from geotechnical site investigations is used in order to estimate the model input parameters. The main obstacles regarding the available data and the assumptions necessary to overcome them are highlighted and accounted for through parametric studies. The limiting values on ground surface acceleration are estimated according to the model introduced by Pecker and compared with the actual acceleration records of the sites. The analyses show that the theoretical predictions are in good agreement with the recorded motions, since only four out of twenty-six sites exhibited acceleration values slightly higher that the maximum ones predicted, when the water table was assumed to be below bedrock. Greater discrepancies are observed when the water table is assumed to be at the ground surface. This can be attributed to inappropriate use of the drained strength instead of the undrained value, since most of these sites consist of dense granular soils. Furthermore, sensitivity analyses are performed in order to overcome the uncertainty concerning the SPT values and the yield strain. A ±30% variation in the N_{SPT} values yielded minor variations in the results. Small variations were also observed in the predicted motion due to changes in the yield strain, even though some sites showed great changes in the results mainly due to the different types of soil.

Also presented here are some numerical results produced by a 1D elastic-perfectly plastic model, that are similar to the theoretical results achieved using Pecker's model, under the same assumptions. However, following a finite element approach offers additional options for assessing the upper limit in similar studies, such as working directly in the time domain, when recordings are available. Furthermore by using a finite element analysis the soil profile modeled may consist of various soil layers whose properties need not comply with the particular distribution described by equation (2).

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