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ABSTRACT

Guadalajara, México, is a large city located mainly over a thick deposit (up 100 m) of pumice, in a seismic zone. Then, besides the stiffness of pumice soils, it is important to predict their behavior under seismic movements.
Pumice soils are so crushable that SPT or CPT does not adequately characterize them. As a complement or alternative to SPT, CPT and other field testing, in recent years there has been a gradual increment in the use of surface wave analysis for soil characterization, by measuring shear wave velocity (V_S). ReMi is one of the surface wave analysis methods and have been used in different locations of Guadalajara for determining the stiffness of pumice soils, depth to bedrock, classify the soil according to IBC, and calculate fundamental periods. Also one-dimensional ground response seismic analysis of four different sites in Guadalajara, under two different seismic scenarios, is presented. From this seismic analysis there are five different response spectra and other parameters.

INTRODUCTION

Pumice soils are volcanic ashes from explosive eruptions. They might be found in several volcanic zones around the world, and they have been studied for geotechnical purposes at least in Mexico, El Salvador, USA, Italy, Tanzania, New Zealand and Japan.

Guadalajara, with around 4.5 million inhabitants, is perhaps the largest city in the world located mainly over pumice soil deposits with thickness up to around 100 m. These soils were originated in the Late Pleistocene rhyolitic center “Sierra La Primavera”, located on the southwest limit of the urban area. The different activity periods were around 145, 95, 75, 60 and 30 thousand years ago, and their mineralogy is composed mainly by silica (74%) and alumina (11%) (Mahood 1981).

In the tens of thousands years that have lapsed since the different volcanic eruptions of “Sierra La Primavera”, important weather changes have occurred. Rain has played an important roll in pumice soils conformation because of their erodability. So, pumice deposits of Guadalajara and surrounded area, have been partially eroded, transported and re-deposited.

PUMICE SOILS

Pumice is a frothy volcanic glass, with a dense network of fine inter connected holes, most of them open to the surface, but others isolated inside the particles (see Figure 1). All this result in light-weight, rough surface and easily crushable particles.

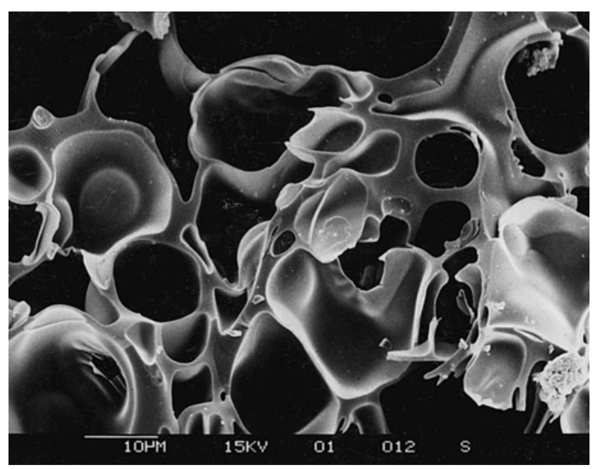

Fig. 1 Scanning electron micrograph showing the internal voids in an Italian pumice clast (from Esposito and Guadango 1998).

Light-weight of pumice soils causes, among other phenomena, high susceptibility to erosion (Esposito and Guadango 1998).

Rough surface in pumice gives as a result high shear strength. Friction angles (ϕ) from 40° to 48° are reported in several pumice soils from Mexico, Tanzania and New Zealand (Saborio 1998; Bucher 1998; Wesley *et al.* 1999; Pender *et al.* 2006).

At stresses greater than a few hundred kPa the stress-strain-strength behavior of pumice soils is dominated by particle crushing, in a similar way than carbonate sands behaves. Particle crushing causes changes in density and a reduction in shear strength (Pender *et al.* 2006). Allely and Newland (1959) found that by increasing cell pressure in triaxial test of a pumice soil from 52 to 550 kPa, there is a reduction of internal friction angle (ϕ) from 45° to 37°, respectively.

Compressibility of pumice sand is much higher than quartz sand, as it is shown from tests reported by Wesley *et al.* (1999) (see Figure 2). From New Zealand pumice, Wesley *et al.* (2006) and Pender (2006) reported compression index values (C_C) of 0.70 to 0.97 for effective vertical stresses (σ'_V) between 2 and 6 MPa. These C_C values are much higher than quartz sand C_C values (0.07 to 0.20) for very loose sand, but similar to reported values by Mesri and Vardhanabhuti (2009) from carbonate sands (0.5 to 1).

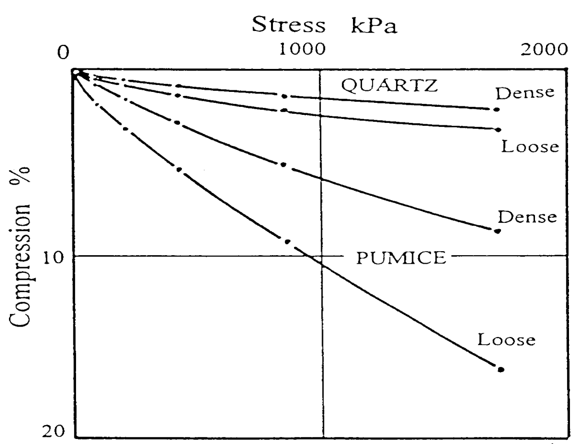

Fig. 2 Compressibility measured in an oedometer test (from Wesley et al. 1999).

Mesri and Vardhanabhuti (2009) proposed a classification for compression behavior of granular materials. It considers three different types (A, B and C). Type A is typical for clean well-rounded strong medium to coarse sands, while type C is associated to angular weak particles such as carbonate and pumice sands. In the type C behavior there is damage in the soil particles that can go from abrasion or grinding of particle surface asperities (level I) to breaking or crushing of particle surface protrusions and sharp particle corners and edges (level II), and even fracturing, splitting, or shattering of particles (level III) could occur. There is a continuous net locking effect throughout the effective stress increment range, and tangent constrained modulus (M) increases as effective vertical stress (σ'_v) increases. Figure 3 shows two typical consolidation curves, one of quartz sand (type A) and the other of carbonate and pumice sand (type C).

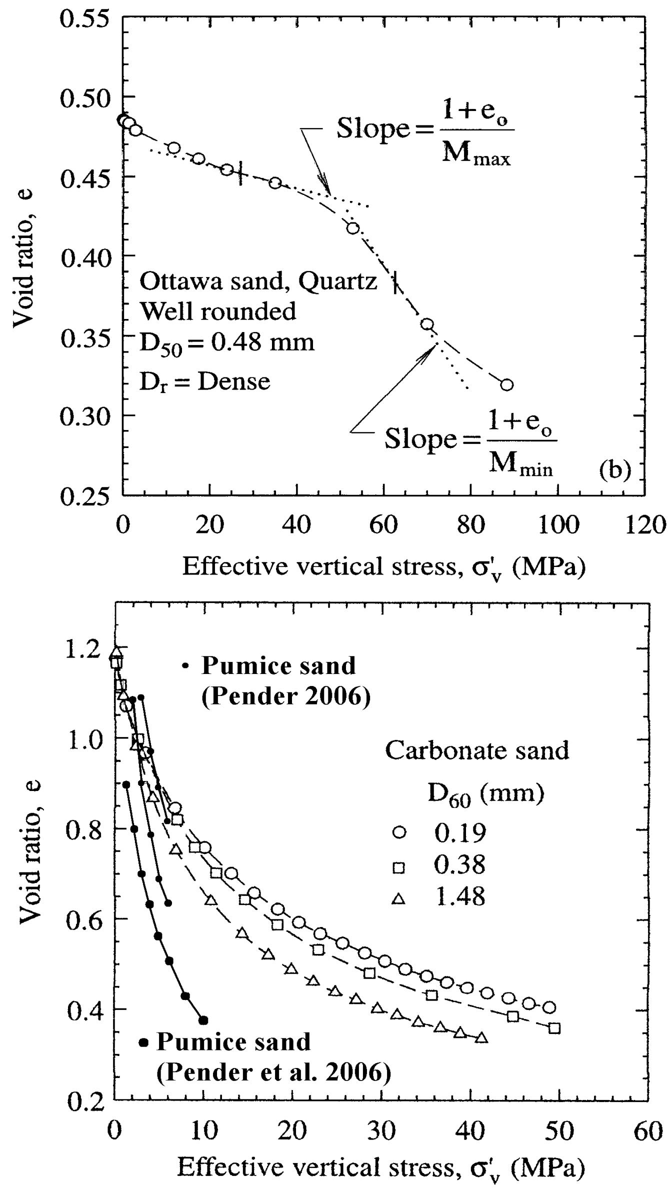

Fig. 3 Type A (upper graph) and type C (lower graph) compression behavior of dense quartz sand and carbonate sand, respectively (from Mesri and Vardhanabhuti 2009), and additional information of pumice sand (Pender 2006; Pender et al. 2006).

Pender (2006) reported that particle crushing in this material depends not only on the magnitude of stresses, but also on the rate of deformation. The longer the load period, the more crushing it happens. These results are in accordance to what Mesri and Vardhanabhuti (2009) reported: when compression index (C_C) increases, as it happens with carbonate and pumice sands, the secondary compression index (C_α) also increases with increment of time. This is based on the C_α/C_C law of compressibility proposed by Mesri (1987). For granular materials, C_α/C_C varies from 0.01 to 0.03 (Mesri *et al.* 1990).

Field testing

Sands and gravels are soils with such a natural structure that for practical purpose it is impossible to get undisturbed samples for laboratory testing. So, frictional soils, such as pumice sand and gravel, are commonly characterized by *in situ* or field tests. Dynamic probing was one of the first *in situ* tests used for geotechnical purposes, followed by standard penetration test (SPT) and later by pseudo-static cone penetration test (CPT). In recent years other field tests such as presurometer and flat dilatometer have extended their use, but there is no reported experience in pumice soils (Broms and Floding 1988; Lazcano 2007).

Particularly when dealing with pumice soils, it is obvious the importance of taking into account their crushability. So SPT, due to the high dynamic energy applied and shape of sampler, is not an appropriated test. It crushes pumice in such a way that it is far from the condition a foundation, or some geotechnical structure, would work with that soil.

CPT could be considered a better field test for pumice. Nevertheless, Wesley *et al.* (1999) did a study of pumice sand in a calibration chamber, and concluded that tip cone resistant values reflect no difference in measurements in dense and loose pumice sand.

Considering the particularities of pumice soils (crushability, high angle of friction, light-weight, etc.) and the questionable use of mechanical intrusive tests such as SPT or CPT for characterizing them, a possibility is to use seismic geophysical methods for measuring shear wave velocity (V_S).

Shear wave velocity (V_S) by itself is a useful parameter for seismic classification of soils. A widely used seismic soil profile criteria considers the average V_S in the upper 30 m ($V_{S\,30}$) (IBC 2006). Regarding with this aspect, some experiences with pumice from Guadalajara will be presented below.

Stokoe *et al.* (2004) have worked in the field of liquefaction potential analysis based on V_S, and it seems to be a convenient tool to work in conjunction to some other field and lab testing.

By knowing shear wave velocity (V_S), mass density (ρ) and Poisson ration (μ) of a soil, it is possible to calculate shear modulus ($G_O = \rho\ V_S^2$) and elastic modulus (E_O) at very low shear strain. G_O and E_O are useful parameters for studying soil behavior not only under dynamic conditions, but also under static ones (Jamiolkowski and Robertson 1988; Burland 1989; Stokoe *et al.* 2004).

SURFACE WAVE ANALYSIS

The first seismic geophysical methods used in geotechnical exploration were seismic refraction, downhole and crosshole. In the seismic refraction method it is possible to measure the compression or primary wave velocity (V_P) of gradually stiffer subsoil strata. In the downhole and crosshole methods both body waves V_P and V_S (shear wave velocity) can be obtained.

Shear wave velocity (V_S) has the advantages of reflecting the stiffness of soils, independently of groundwater level, and it is not the same with V_P. That is why V_S has become an important parameter in subsoil exploration.

Downhole and crosshole are intrusive tests because they need boreholes to lower sensors and vibration source (for crosshole) down to the depth of interest, and boreholes take time and money. On the other hand, for seismic refraction sensors and vibration source are located on the surface, so no borehole is needed. This type of test is named non-intrusive.

In the 80's there was an important advance in non-intrusive seismic geophysical methods, with the development of the first modern method of surface wave analysis and it was named SASW (Spectral Analysis of Surface Waves) (Nazarian and Stokoe 1984). Almost at the same time another similar test was invented, the continuous surface-wave (CSW) (Abbiss 1981; Mattheus *et al.* 1996). Former development of the same principle is the MASW (Multichannel Analysis of Surface Waves) (Park *et al.* 1999), ReMi (Refraction Microtremor) (Louie 2001), and other variants. This probes the increasing acceptance of surface wave analysis methods in the engineering geotechnical field (Lazcano 2007).

When an infinite elastic media is subjected to vibration, body waves travel within the media, and there are two types of waves: compression (V_P) and shear (V_S) waves, already mentioned (Ritchard *et al.* 1970). On the average, in geotechnical materials, V_P travels 1.7 to 2.5 times faster than V_S.

If the elastic media is not infinite but semi-infinite (as a soil deposit could be idealized) in the boundary of the media (on the surface of a soil), vibration generates the so called "surface waves", and there are two types: Rayleigh (V_R) and Love (V_L) waves (Ritchard *et al.* 1970). What the surface wave analysis methods do is to register and analyze Rayleigh wave velocity (V_R), which is slightly slower (around 8%) than V_S, and for practical purposes they are considered equivalent.

When the surface of a soil deposit is hit by a hammer, 66% of the generated waves are surface waves (mainly Rayleigh type), 27% are shear waves and only 7% compression waves (Woods 1968).

In an ideally homogeneous soil deposit, Rayleigh waves travel at a speed V_R which is independent of their wave length.

However, if there are strata with different stiffness, density or Poisson ratio, then V_R depends on its wave length. When velocity and wave length (or frequency) depends on each other, it is said that the wave is dispersive. This is the behavior of Rayleigh waves in non uniform media, such as in a stratified soil deposit, and it is the fundament of the surface wave analysis methods (Matthews *et al.* 1996).

Most of the energy of surface waves is contained within a zone with approximately a wave length depth. In this way, long wave lengths (or short frequencies) help in characterizing deep strata, while short wave lengths (or long frequencies) near surface strata.

<u>Refraction microtremor (ReMi) method</u>

Refraction microtremor (ReMi) method was developed by Louie (2001). It uses a seismograph and a line with 12 or more equally spaced geophones, to register surface waves at frequencies as low as 2 Hz. Noise record is analyzed and Rayleigh waves can be separated to finally obtain a V_S-depth profile.

In comparison to "normal" seismic refraction, with ReMi, as well as with the different surface analysis methods, shear wave velocity (V_S) is measured, instead of compression wave velocity (V_P), and soft layers below hard ones can be detected. This latter aspect is a critical limitation for "normal" seismic refraction from the geotechnical point of view.

The four steps for a ReMi survey are:

1) Record of vibration with a seismograph and a straight line of at least 12 equally spaced, low frequency (4.5 Hz), vertical-component geophones. Typically, for a 120 m array, several 20-second records are registered. Ambient noise with or without induced vibration is used as a source, and it is possible to run the test even in noisy urban areas.

2) Perform of a p-f (slowness-frequency) transformation of the vibration to create "velocity spectrum" (see Figure 4).

3) From the p-f (slowness-frequency) image pick the dispersion curve (Rayleigh wave phase velocity versus period) (see Figure 5).

4) From the dispersion curve derive a one-dimensional shear wave velocity profile of the subsurface (see Figure 6).

ReMi has proved to be a reliable tool in different projects, for determining V_S-depth profiles down to approximately one third to half the length of the array and up to around 100 m (Pullammanappallil *et al.* 2003a; Pullammanappallil *et al.* 2003b; Rucker 2003; Pullammanappallil *et al.* 2004; Veronese and Garbari 2004; Stephenson *et al.* 2005; Lambert *et al.* 2006; Pancha et al. 2007). ReMi technique has been used also in several locations in the pumice soil deposits in Guadalajara, Mexico, for determining V_S-depth profile and $V_{S\ 30}$ determinations (Lazcano, 2007).

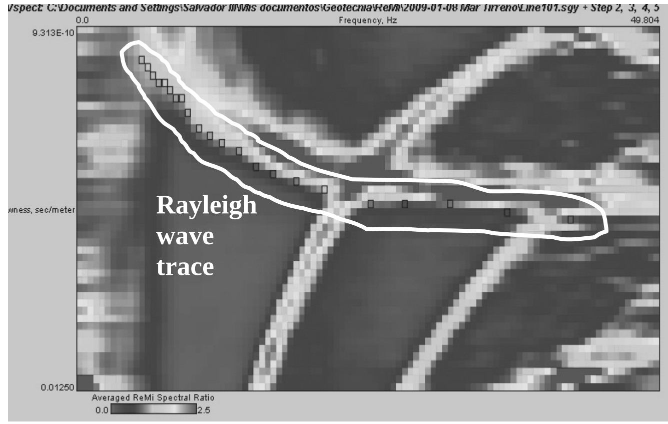

Fig. 4 Velocity spectrum (p-f) derived from vibration.

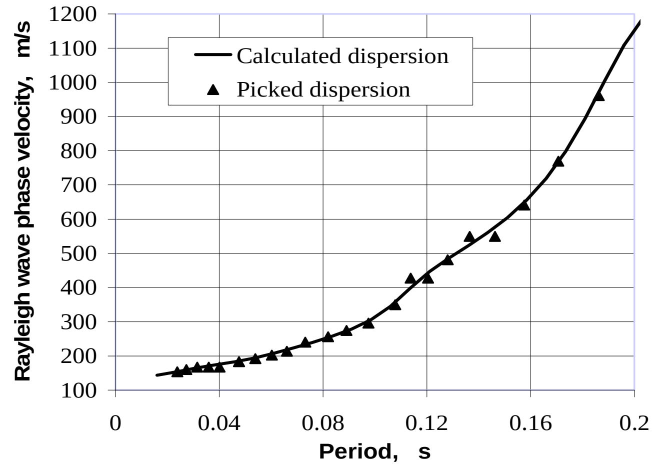

Fig. 5 Picks chosen in figure 4 are interactively modeled to derive a V_S-depth profile.

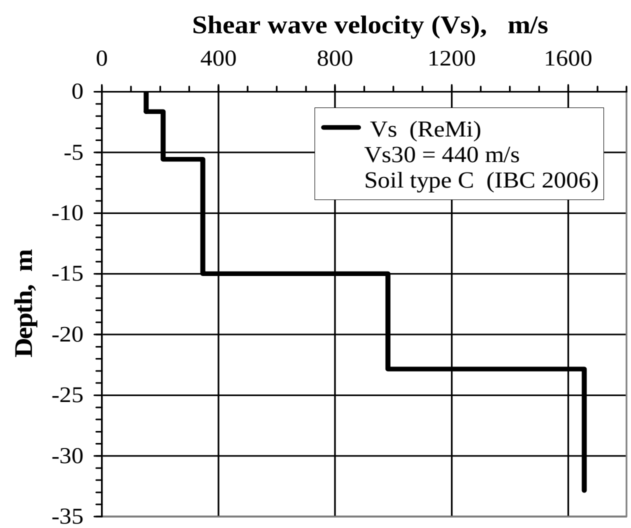

Fig. 6 One-dimension V_S-depth profile determined interactively modeling from Figure 5.

GUADALAJARA SUBSOIL

On the average, subsoil in the west part of Guadalajara is formed by up to 15 m of loose to medium dense pumice silty sand, sand and gravel (locally named “jal”), followed by up to 80 m of stiff pumice silty sand with some pumice gravel.

Probably the upper 15 m or so are transported and re-deposited pumice and the lower 80 m or so is a stiff pumice deposit named “Tala” tuff, which can be consider either as a very dense soil or as a soft rock. This was originated around 95 thousand years ago, when there was an important explosive eruption of approximately 20 km^3 of ashes that covers some 700 km^2, including the west part of Guadalajara (Mahood 1980).

Underlying “Tala” tuff there is basalt or ignimbrite. Occasionally, over basalt there might be a small layer (< 2 m) of a residual profile of clay and basaltic gravel, boulders and blocks. Below basalt there are different volcanic rock strata, such as tuff, ignimbrite and rhyolite.

<u>V_S from ReMi method</u>

ReMi technique has been used in several locations in the pumice soil deposits in Guadalajara, Mexico, for determining shear wave velocity (V_S) versus depth profile and average shear wave velocity in the upper 30 m ($V_{S\,30}$) (Lazcano, 2007). Figures 4 to 6 are from a location where bedrock is 15 m below surface, $V_{S\,30}$ is 440 m/s, consequently soil type C, according to IBC (2006).

Concerning V_S-depth profiles, results of depth to bedrock (V_S > 800 m/s) obtained by ReMi have been consistent down to 85 m, with an error of only 10 to 15%, when compared to data from boreholes in ten of the fifteen presented sites. It is also possible to detect with some confidence the upper part of the “Tala” tuff, because its V_S tends to be above 400 m/s. Similar values of V_S have been reported in pumice tuff in San Salvador, El Salvador (Faccioli *et al.* 1989).

Based on ReMi testing, in Table 1 there is information about average shear wave velocity in the upper 30 m ($V_{S\,30}$), soil type classification according to IBC (2006), depth to bedrock (V_S > 800 m/s) and the fundamental period of soils above bedrock ($T = 4\,H / V_S$ average, where H: thickness of soil deposit).

In Figure 7 there is a map of the west part of Guadalajara with different sites reported in Table 1, where ReMi surveys were done. In the next section there will be presented some results of ground response seismic analysis in four places: site 1, which is next to the Cathedral (XVII century church); site 3 (Country) in a zone with several 20-story buildings; site 11, where there is a project of a 336 m tall communication tower named Torrena; finally site 13, where a 42 story Riu Hotel (the tallest in the city) is under construction.

Table 1. Sites with soil seismic classification

Site	**1 Cathedral**	**2**	**3 Country**	**4**	**5**
$V_{S\,30}$ (m/s)	262	260	434	477	425
Soil type (IBC)	D	F	C	C	C
Depth to rock (m)	31	25	18	22	17
T (s)	0.47	0.46	0.22	0.22	0.26
Site	**6**	**7**	**8**	**9**	**10**
$V_{S\,30}$ (m/s)	311	345	400	375	321
Soil type (IBC)	D	D	C	C	D
Depth to rock (m)	48	71	45	38	62
T (s)	0.57	0.62	0.44	0.39	0.63
Site	**11 Torrena**	**12**	**13 Riu**	**14**	**15**
$V_{S\,30}$ (m/s)	339	304	307	357	371
Soil type (IBC)	D	D	D	D	C
Depth to rock (m)	85	65	54	57	36
T (s)	0.80	0.65	0.61	0.52	0.37

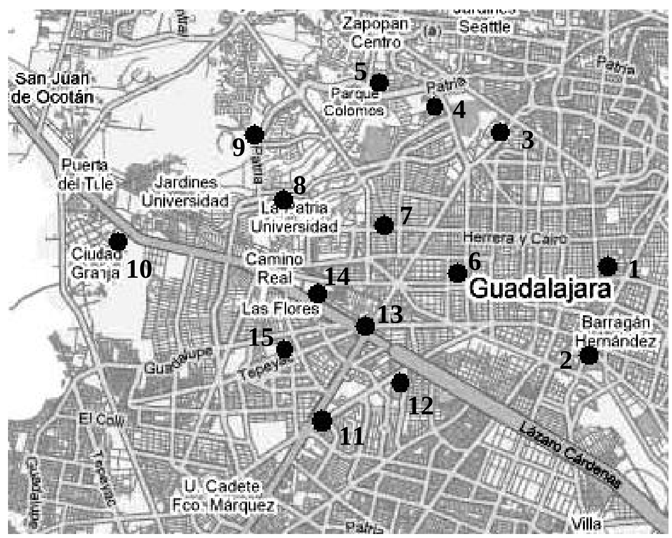

Fig. 7 West part of Guadalajara with the location of different ReMi surveys (from Google Maps). Site 1 is next to the Cathedral, and from 1 to 10 there are 10 km.

Eight out of the fifteen sites are soil type D, according to IBC, six are C, and one F. Site 2 (see Figure 7) is the F soil type, and it is located in a low level area within Guadalajara. At the beginning of the XX century there was a small lake in that zone and subsoil is formed by lacustrian and aluvial sediments with sand, silt and silty sand layers in the upper profile. Water table is around 2 m below ground surface and stiffness in the upper 9 m is very soft to soft, with V_S values from 120 to 180 m/s. Consequently, it is probable that submerged soils, from 2 to 9 m depth, will liquefy during an intense earthquake.

Ground response seismic analysis

For many years, Guadalajara had a horizontal development, and 50 years ago there were very few buildings taller than 4 floors. So, most of constructions were short fundamental period ones, smaller than around 0.4 seconds. Specially in the last 30 years there has been an important increment in construction of building 4 to 10-floor tall (periods from 0.4 to 1 second). Only in the last 10 years buildings of 20 to 30 floors have been built. Nowadays, the tallest building in Guadalajara is a 40-story apartment tower (site 9 in Figure 7), and a 42-story hotel tower under construction (site 13, Riu Hotel).

On the other hand, seismic activity in Guadalajara during the last fifty years, has been relatively quiet in comparison to the historic reports (Lazcano 2001). The recent important earthquakes were 1973, 1985, 1995 and 2003, but peak ground acceleration in rock (amax rock) was at the most 0.015 g.

Seismic instrumentation of Guadalajara is poor. From 1992 to 1998 there was an accelerometric network and it registered the October 1995 earthquake (M = 7.6) with epicenter distance 240 km from Guadalajara. The registered peak ground acceleration in rock in Guadalajara was around 0.006 g (Chavez Gonzales 1995).

The one-dimensional ground response seismic analysis of site 1, close to the Cathedral, was done with ProShake and using V_S values from ReMi (see Table 1). In Figure 8 there is the response spectrum obtained with ProShake and the one measured in the accelerometric station Rotonda, next to the Cathedral (Chavez Gonzales 1995). Both spectra are in agreement for the 1995 earthquake, so, additional ground response analysis was done for the same site Cathedral considering amax rock values of 0.015 and 0.09 g (see Figure 9). The first value might be the highest acceleration in Guadalajara in the last 50 years, and 0.09 g is the value for this city that has a 10-percent probability of being exceeded in 50 years, according to seismic hazard analysis (PSM 1996).

The same one-dimensional ground response seismic analysis with ProShake for amax rock values of 0.015 and 0.09 g was done for three additional sites: Country, Torrena and Riu (sites 3, 11 and 13, respectively). Results are shown in figures 10, 11 and 12. The lower spectrum is for the 0.015 g and the upper for the 0.09 g peak rock acceleration.

It is important to mention that for site Riu (13), due to the relatively close distance to station Los Arcos (1.3 km) of the accelerometric network, it was done similar analysis than for Cathedral site. A response spectrum obtained with the V_S values from ReMi and ProShake for the 1995 earthquake scenario, is similar to the spectrum registered in Los Arcos station (Chavez Gonzales 1995).

Besides response spectra, several other parameters were calculated with ProShake for the four different sites, and the information is in Table 2. There are the peak acceleration in rock and at free-surface, the amplification factor between both, ante the predominant (T predom) and mean (T mean) periods. Predominant period is the period corresponding to the maximum value of the Fourier spectrum.

Table 2. Some response analysis parameters from ProShake.

Site	**1**	**3**	**11**	**13**
Rock depth (m)	31	18	85	54
$a_{max\ rock}$ (g)	0.015	0.015	0.015	0.015
$a_{max\ surface}$ (g)	0.043	0.054	0.039	0.048
Amplification	2.9	3.6	2.6	3.2
T predom (s)	0.40	0.17	0.79	0.52
T mean (s)	0.38	0.20	0.59	0.44
Site	**1**	**3**	**11**	**13**
$a_{max\ rock}$ (g)	0.09	0.09	0.09	0.09
$a_{max\ surface}$ (g)	0.175	0.246	0.165	0.179
Amplification	1.9	2.7	1.8	2
T predom (s)	0.44	0.18	0.86	0.55
T mean (s)	0.44	0.24	0.66	0.48

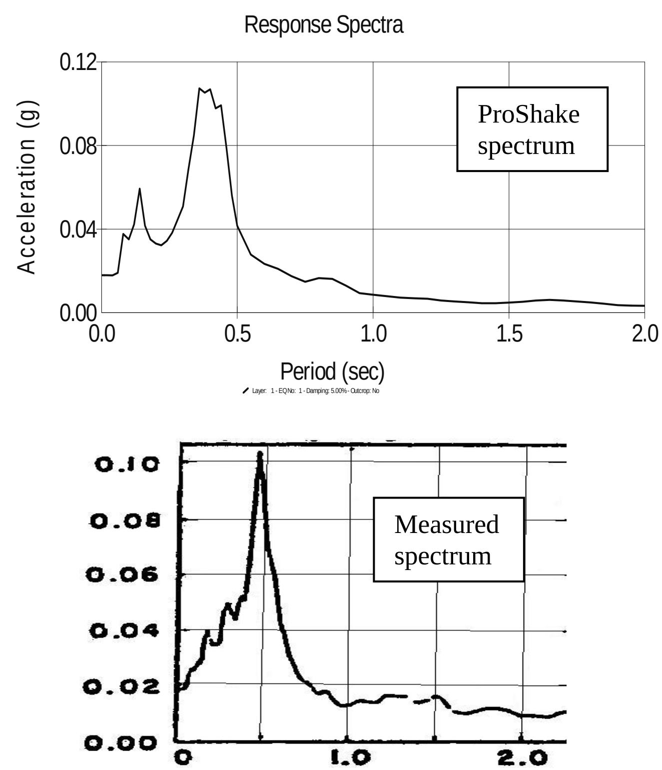

Fig 8 Calculated and measured (Gonzalez Chavez 1995) response spectra from 1995 earthquake in site 1 (Cathedral).

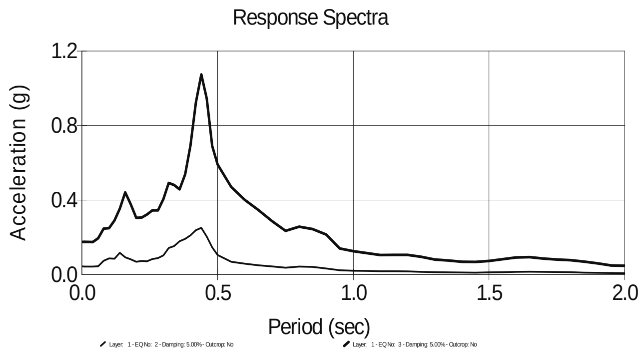

Fig 9 Calculated response spectra for amax rock of 0.015 and 0.09 g in site 1 (Cathedral).

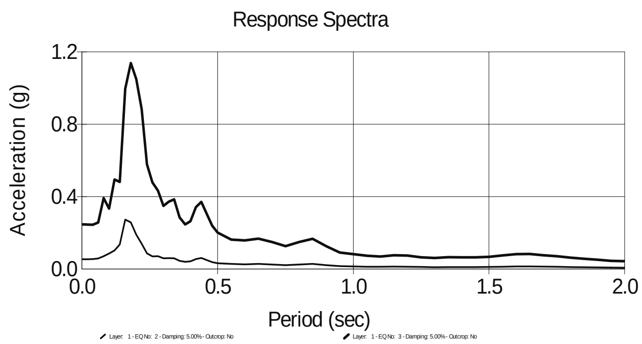

Fig 10 Calculated response spectra for amax rock of 0.015 and 0.09 g in site 3 (Country).

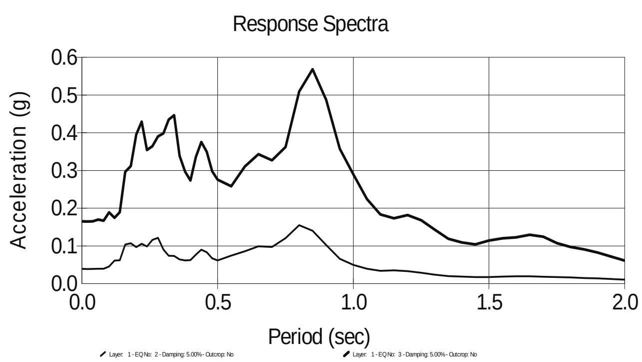

Fig 11 Calculated response spectra for amax rock of 0.015 and 0.09 g in site 11 (Torrena).

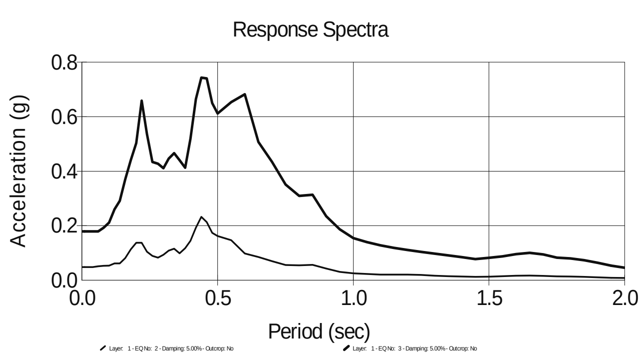

Fig 12 Calculated response spectra for amax rock of 0.015 and 0.09 g in site 13 (Riu Hotel).

From Table 2 we have that amplification factors for a peak rock acceleration of 0.015 g goes from 2.6 to 3.6, and from 1.8 to 2.7 for a peak rock acceleration of 0.09 g. So, the smaller the peak rock acceleration, the larger amplification factors, due to soil non-linear behavior.

With respect to periods from Table 2, there is increment in their values as peak rock acceleration goes from 0.015 to 0.09 g. In the case of predominant period it increases 6 to 10% and mean period 9 to 20%. This behavior is also related to soil non-linearity.

CONCLUSIONS

- Pumice soils have substantial differences when compared to quartz granular soils. Pumice soils are light-weight, rough surface and easily crushable particles.
- Due to the rough surface of pumice, friction angles (ϕ) are higher than "normal" sands, and values of 40° to 48° are reported in pumice soils from different countries.
- According to their compression behavior, pumice sands are more crushable than "normal" sands. In a classification system proposed by Mesri and Vardhanabhuti (2009), pumice are type C, with a behavior similar to carbonate sands.
- Neither SPT nor CPT intrusive field testing are reliable methods for pumice characterization.
- Direct shear wave velocity (V_S) determination by some geophysical method (downhole, crosshole or surface wave analysis) is an attractive alternative for pumice soil characterization.
- Surface wave analysis methods are non-intrusive, and there are different alternatives, such as SASW, MASW and ReMi. They test a large volume of soil, and that make them particularly useful for ground response seismic analysis.
- ReMi was done in several locations in pumice soil profiles in Guadalajara. In Table 1 there are the results from fifteen different ReMi surveys. Results include average shear wave velocity in the upper 30 m ($V_{S\ 30}$), IBC (2006) soil type, depth to bedrock, and fundamental period. Depth to bedrock was checked with boreholes in ten sites, and error of only 10 to 15% was founded.
- Eight (53%) out of the fifteen sites presented in Table 1 are D soil type, according to IBC, six (40%) type C and one (7%) type F. Actually, based on experience and some other studied locations, D soil type should be a larger proportional part, probably around 2/3, and the rest C type. Respect to F soil type from site 2, it was classified so due to the potential liquefaction, but there are very few zones in Guadalajara that could liquefy or be consider F type.

- In four, out of the fifteen sites from Table 1, one-dimensional ground response seismic analysis was done with ProShake. In two cases (Cathedral and Riu) calculated response spectrum was compared versus measured spectrum in the 1995 earthquake, and there was a good agreement.

- With V_S information from ReMi in four different sites in pumice soils from Guadalajara, ground response seismic analysis was done with ProShake, and results are presented in Table 2 and Figures 8 to 12.

- From Table 2 we have that amplification factors of acceleration from bedrock to the surface goes from 2.6 to 3.6 (3.1 on the average) for a peak rock acceleration of 0.015 g, and from 1.8 to 2.7 (2.1 on the average) for a peak rock acceleration of 0.09 g. These results are due to soil non-linear behavior.

- From Tables 1 and 2 we have that, with the exception of sites 3 to 5, periods of soil deposits varies from 0.37 to 0.86 seconds. Consequently, structures with similar periods, that are 4 to 9-floor building, are more susceptible to damages during earthquakes.

- Based on the fifteen studied sites, it was founded that the period of a soil profile in the west part of Guadalajara is close to the depth to bedrock (in meters) divided by 100. This is due to the fact that in most of the area there is the "Tala" tuff (very hard soil) from the bedrock up to 10 to 15 m below ground surface, and shear wave velocity (V_S) of this formation is around 400 m/s.

- In the last 50 years there has been a large growth of the city, and only in the last 30 years or so there has been a tendency for 4 to 40-floor building construction. On the other hand, seismic activity in the zone for the same 50-year period, has not been as intense as in the past. Particularly, seismic hazard analysis (PSM 1996) indicates a peak rock acceleration of 0.09 g that has a 10-percent probability of being exceeded in 50 years, but in the last 50 year there has been hardly peak rock acceleration values of 0.015 g (one sixth, at the most). So, building taller than 4 floors, the majority built in the last 50 years, have partially being tested under dynamic conditions.

- Figures 9 to 12 show response spectra for peak rock acceleration values of 0.015 and 0.09 g. Figure 10 shows the shortest period site and Figure 11 the longest one.

- Figure 9 could be considered a representative condition for old downtown area, where there are several buildings (particularly churches) from XVII to XIX century. Fundamental vibration period of soil in this zone goes from 0.4 to 0.5 seconds, which is approximately the period of 4 to 5-floor building and also of churches. There are reports from partial failures in several churches, and the most well documented one is the Cathedral. In 1818 an earthquake severely damaged the former towers; it stayed with no towers until 1854, where the actual towers were built (Lazcano 2001).

- Most of modern tall buildings have been built particularly in the zone of sites 3 to 9. Soils are somewhat different from old downtown area, but buildings are much more different. Additional studies in that zone are important, as well as an accelerometric network and seismically instrumented buildings. All these will help to learn more about pumice soils under seismic loads and its interaction with constructions build on them.

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