ON THE CHARACTERISTICS OF SOFT SOIL INELASTIC DESIGN RESPONSE SPECTRAS FOR THE CITY OF GUAYAQUIL, ECUADOR

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SUMMARY

Five accelerograph records obtained in the city of Guayaquil, Ecuador were normalized to a 0.28 g maximum acceleration expected in the area. Two of the records are from rock sites and the other 3 from medium soft, soft and very soft soil sites. The predominant period varies from 0.12 seconds for the rock sites to 2 seconds period for the very soft site, and the incremental velocities vary from 130 mm/sec at rock sites to 2200 mm/sec for very soft soil sites. The inelastic Design Response Spectras (IDRS) [Moncayo, 1999] for all the normalized records clearly show the advantages of inelasticity for short period waves records, with the only restriction of avoiding large displacements. On the other hand, for records containing long duration continuous pulses, the advantages of inelasticity diminish greatly showing that even for large non advisable ductility factors the seismic coefficient at yielding is a large fraction of the elastic one.

1 INTRODUCTION

It is very well known that earthquake induced vibration of structures is the main concern of structural engineers. The codes, for buildings and bridges which importance allows inelastic behavior, clearly specify reduction factors in order to achieve economical, serviceable and safe design. The city of Guayaquil, Ecuador, is mainly on a soft deep clay deposit, [Lara, 1996], which is continuously excited by earthquakes occurring on the border of the Nazca and South American plates, source of the most severe strong motions that have shocked the ecuadorian coast, or on the numerous active faults that cross the country.

2. OBJECTIVE

To study the main characteristics of soft soil inelastic design response spectras in terms of the characteristics of earthquakes recorded at the city of Guayaquil, Ecuador.

3. EARTHQUAKE RECORDS

The Catholic University accelerograph network registered some small earthquakes in different parts of the city and allowed our University to use them for this research. These records, were normalized to an acceleration of 0.28g which is expected in the city, from the subduction zone, more than 200 km away, every 475 years, [Lara, 1985]. Figures 1 to 5 and 11 to 15, show one of the horizontal components of each accelerograph record as well as the corresponding velocity record. Figures 6 to 10 show the Fourier spectras of such normalized records.

As it can be seen in figures 1, 11 and 2, 12 as well as in figures 6 and 7, the CICG and UC records have a very large content of small period waves of about 0.12 seconds period with small incremental velocities in the order of 130 mm/sec. For the AE record (Fig 3) the predominant period is about 0.5 seconds as it is showed in the Fourier transform (Fig 8). The incremental velocity (Fig 13) reaches 280 mm/sec.

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The BC record (Fig 4) shows a similar trend as that of the AE record but with longer periods. The peak corresponding to the soil in the Fourier spectra (Fig 9) is located at a frequency of about 1.0 Hz that is a period of about 1.0 seconds. The incremental velocity (fig 14) is in the order of 350 mm/sec.

Finally, the TM record (Fig 5) shows the largest periods of all the records along the whole time of recording. The Fourier spectra (Fig 10) shows an important peak at about 0.6 Hz, that is, in the order of 2.0 seconds period. The incremental velocity (Fig 15) reaches 2200 mm/sec.

The CICG and UC accelerograms were obtained on rock. The first one on a rock outcrop at sea level and the second on rock but at the top of a 30000 mm hill.

The AE record was obtained on a medium soft clay about 15000 mm depth, Whereas the BC record was taken on a soft clay about 25000 mm depth.

Finally, the TM record corresponds to a very soft clay about 60000 mm depth which contains a loose to medium dense sand layer of about 10000 mm located between the 15000 and 25000 mm levels. All of the soft clay deposits have very low compressive resistant, about 0.05 N/mm^2.

4. NONLINEAR ANALYSIS

The differential equation that controls the response of a linear-elastic single degree of freedom system (SDOFS) subjected to an earthquake motion is

$$m\ddot{u}(t) + c\dot{u}(t) + ku = -m\ddot{u}_g(t) \qquad (1)$$

m is the mass of the SDOFS, $\ddot{u}(t)$, $\dot{u}(t)$ and $u(t)$ are the acceleration, velocity and displacement of the SDOFS at any time t; c is the damping; k is the stiffness and $\ddot{u}_g(t)$ is the acceleration of the earthquake motion.

On the other hand by noting that $k = m\omega^2$, $c = 2m\omega\xi$; where, ω is the radial frequency of the SDOFS and ξ is the damping ratio, and dividing by m and by u_y, where u_y is the yielding displacement, which is a constant value with respect to time, equation 2 is obtained:

$$\frac{\ddot{u}}{u_y} + 2\omega\xi\frac{\dot{u}}{u_y} + \frac{k}{m}\frac{u}{u_y} = -\frac{\ddot{u}_g(t)}{u_y} \qquad (2)$$

Calling: $\ddot{\mu} = \ddot{u}/u_y$; $\dot{\mu} = \dot{u}/u_y$ and $\mu = u/u_y$, then:

$$\ddot{\mu} + 2\omega\xi\dot{\mu} + \omega^2\frac{R}{k}\frac{k}{R_y} = -\frac{\ddot{u}_g(t)}{R_y}k \qquad (3)$$

R is the elastic base shear required by the motion. $R=ku$ and $R_y = ku_y$. R_y is the base shear at yielding.

Defining $\eta = \dfrac{R_y}{m\ddot{u}_g max}$ and $\rho = R/R_y$; then:

$$\ddot{\mu} + 2\omega\xi\dot{\mu} + \omega^2\rho = -\frac{\omega^2\ddot{u}_g(t)}{\eta\ddot{u}_g max} \qquad (4)$$

Besides, $\eta = \dfrac{Ry}{W/g\,\ddot{u}_g max} = \dfrac{C_y}{\ddot{u}_g\, max/g}$ (5)

Where $C_y = Ry/W$: yielding coefficient or seismic resistance coefficient.

Equation 5 is another way of looking at the parameter η, that is, as a fraction of the maximum ground acceleration. Equation 4 allows to determine a non dimensional inelastic response of a non-linear SDOFS, characterized by μ and ρ, to a non dimensional earthquake ground motion characterized by $\ddot{u}_g(t)/\ddot{u}_g max$. This response is evaluated in terms of the parameter η. The resistance function of the non-linear SDOFS is assumed to be elastic perfectly plastic. Figs 16 to 20 show the inelastic design response spectras (IDRS) for the 5 records above mentioned. The yielding resistance is known during the analysis by defining the parameter η. The damping coefficient is 5% for all cases.

5. ANALYSIS OF THE IDRS

Looking at figures 16 to 20, it is important to recognize that the ductility factors are variable depending on the period of the SDOFS and its resistance which is given by the parameter η. It is very well known that the larger the ductility developed by buildings or bridges during an earthquake response, the larger will be the damage. Therefore it is necessary to limit the values of ductility used in design according to the importance of the structure.

The IDRS for the rock sites records (figures 16 and 17) show that it is possible to take advantage of inelasticity even for low period structures and, as the period becomes longer, the resistant factor given by η decreases rapidly.

For example, in the IDRS for CICG record (Fig 16), a 0.5 second period SDOFS could be designed for η= 0.15 if $\mu = 4$. If $\ddot{u}_g max/g = 0.28$, the base shear at yielding, R_y, could be as low as 0.042 times the weight of the structure.

For the AE record the IDRS (Fig 18) shows that for periods lower than 0.4 seconds the value of the base shear at yielding, R_y, should be equal or larger than the weight of the structure as long as the ductility factor is kept lower than 4. For $\ddot{u}_g max/g = 0.28$ and a 0.3 seconds period structure, η is 1.1 and R_y should be 0.31 times the weight of the structure. On the other hand, if it is desired to use a more economical design, a ductility factor of 6 could be used along with $\eta = 0.8$. Therefore, R_y reaches 0.224 times of weight of the structure. Clearly, in this range of periods, in order to take advantage of inelasticity, larger and therefore not recommended ductility factors should be used for design. However, as the period becomes larger it is possible to design for lower values of R_y keeping $\mu = 4$.

A somehow similar situation is observed in the IDRS for the BC record. However both, the AE and BC IDRS's present a quite different situation with respect to the rock sites IDRS's. While for the CICG IDRS a 0.5 seconds structure could be designed for 0.042 W, being $\mu = 4$ and $\ddot{u}_g max/g = 0.28$, the same structure for the medium soft and soft soil sites, AE and BC records, must be designed for 0.168 W using the same value of μ and $\ddot{u}_g max/g$.

Finally for a very soft soil, as is the case of the TM record (Fig 20), the η value should be kept larger or equal to 1 for structures up to 1.6 seconds period. This means that the seismic resistance coefficient should be larger than the maximum recorded acceleration if it is desired to use a ductility factor as small as 2 within that range of period. In this case, for the 0.5 second SDOFS, if $\mu = 4$ and $\ddot{u}_g max/g = 0.28$, the value of η is 1.2 and the base shear at yielding, R_y, reaches 0.336 W. On the order hand, a 2.0 seconds period SDOFS requires an η = 0.6 and $R_y = 0.168$ if the ductility factor is kept equal to 4.

These perhaps surprising results have an explanation by analysis of the results presented in figures 3-5, 8-10 and 13-15. The AE, BC and TM records show large periods waves with predominant periods ranging from 0.5, 1.0 and 2.0 seconds respectively. These long periods are associated to medium soft, soft and very soft clays.

In contrast, the CICG and UC records show a large content of very short period waves which correspond to rock sites with periods ranging from 0.1 to 0.15 seconds. All of the above periods coincide very well with the results of an ambiance vibration test research carried on the soil surface of the city, [Lara 1996].

6. ENERGY ANALYSIS

The above mentioned observations correspond to an energy interchange analysis. When short period waves conform the main frequency content of a record, the kinetic energy entering the structural system is transformed into strain energy:

Assume a linear elastic - perfectly plastic force - deformation relationship (Fig 21). Then:

$$\frac{1}{2}m\dot{u}^2 = \frac{1}{2}R_y u_y(2\mu - 1) \tag{6}$$

where R_y is the yielding resistance, u_y is the yielding displacement, u_m is the maximum displacement and μ is the structure displacement ductility factor.

It is well known that an earthquake record can be seen as a series of pulses which in the case of the CICG and UC records such pulses are of very short duration, therefore from the differential equation of motion:

$$\dot{u} = \frac{I}{m} \tag{7}$$

Where $I^2 = \int_o^{td} p(t)dt$. t_d is the duration of the pulse; *p(t)* is the pulse and *dt* is the differential of time.

Substituting in (6):

$$I^2 = \frac{R_y{}^2(2\mu - 1)}{\omega^2} \quad \text{and} \quad R_y = \frac{I\omega}{\sqrt{2\mu - 1}} \tag{8}$$

If the behavior is elastic: $\mu = 1$ and $\mathrm{Re} = \frac{2\pi I}{T}$. For inelastic behavior, assume $\mu = 5$, then: $R_y = \frac{2\pi I}{3T}$

Therefore: $\frac{R_y}{\mathrm{Re}} = \frac{1}{3}$. Where Re = Elastic resistance. However the structural displacement will be 5 times the yielding displacement.

On the other hand, the pulses in the cases of AE, BC and TM records, mainly the last one, are long, that is the period of the waves are large. If it is assumed that the inertia force $F_1 = m\ddot{u}_g$ is constant during the long duration acceleration pulse, the work done by this force up to the time of maximum response u_m is $F_1 u_m$. Then, the total energy imparted to the SDOFS must be equal to the total energy absorved by the system, i.e., $R_y\lfloor u_m - u_y/2\rfloor$. (Fig 21). Thus:

$$F_1 u_m = Ry\left[u_m - \frac{u_y}{2}\right] = R_y u_m\left[1 - \frac{1}{2\mu}\right] \tag{9}$$

Where F_1 is the maximum amplitude of the pulse.

$$R_y = F_1 \Big/ \left[1 - \frac{1}{2\mu}\right] \quad (10)$$

If the behavior is elastic: $\mu = 1$ and $\text{Re} = 2F_1$

For inelastic behavior, assume $\mu = 5$, then: $R_y = \frac{10}{9} F_1$. Looking at equation 10, for any μ, R_y must be always larger than the maximum amplitude of the pulse. This means that no matter how large the ductility factor used for design can be, the yielding resistance will always be larger than the maximum recorded acceleration.

From the above values, $\frac{R_y}{\text{Re}} = \frac{5}{9}$ that is, for long duration waves the inelasticity that can be used for design is lower than the one for short duration waves for the same ductility factor.

7. CONCLUSIONS

[Bertero et al., 1978], have shown that, for the Derived Pacoima Dam record of the 1972 San Fernando earthquake, the presence of one long duration pulse affects the inelastic response of SDOFS inducing to design for a seismic coefficient larger than the maximum ground acceleration even for low ductility factors. The record for very soft soil can be seen as a series of long duration pulses which do not allow that the yielding resistance be lower than the maximum recorded acceleration. Looking at the IDRS of the TM record for example, even for a small μ, the values of C_y must be larger than the maximum ground acceleration. Therefore, the presence of one long duration pulse or a series of long duration pulses in a earthquake record changes the characteristics of the IDRS compared to short period waves IDRS typical of rock or stiff soil sites.

The required, R_y for long pulse waves will always be a larger fraction of Re compared to that of short period waves. Besides, Re for long period waves will always be twice the maximum ground acceleration. The design for short period waves earthquakes can take larger advantage of plastic behavior than long period waves earthquakes.

The above mentioned conclusions can be observed in Fig 22 which shows the seismic resistance coefficient, spectra, C_y, for $\mu = 4$ and $\ddot{u}_g max / g = 0.28$, obtained from the IDRS of the five earthquake records analyzed. Clearly, the advantage of inelasticity become less and less important as the soil becomes softer, being the extreme the case of very soft soils where up to 1.6 seconds period structures must be designed within a band between 25 and 40% of *g*. These values are almost equal or larger than the maximum recorded acceleration of 0.28 *g*.

8. ACKNOWLEDGE

Thanks are given to Professor V.V. Bertero who read the paper and made very valuable suggestions.

9. REFERENCES

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FIGURES

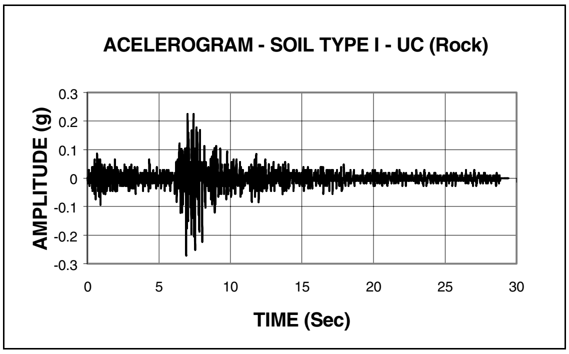

Figure 1: Acelerogram – CICG (Rock)

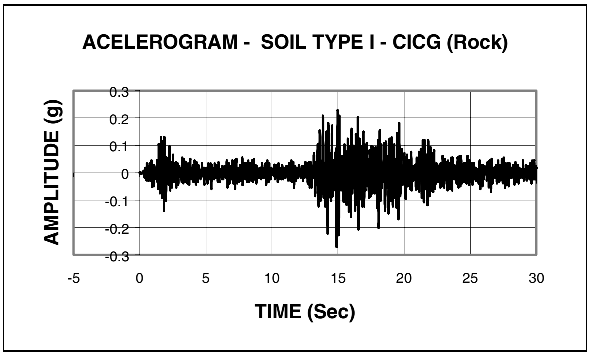

Figure 2: Acelerogram – UC (Rock)

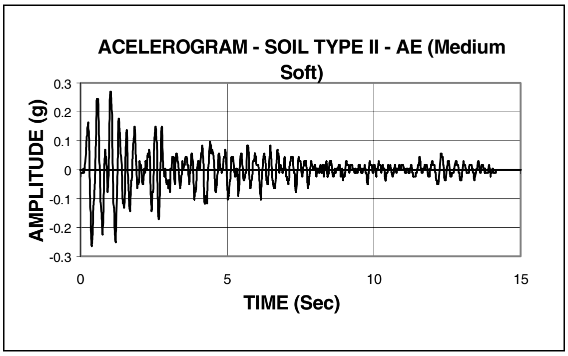

Figure 3: Acelerogram – AE (Medium Soft)

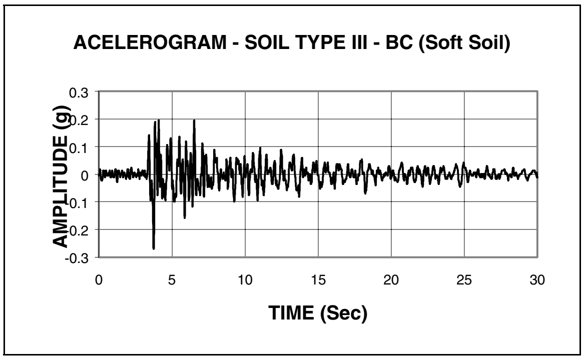

Figure 4: Acelerogram – BC (Soft Soil)

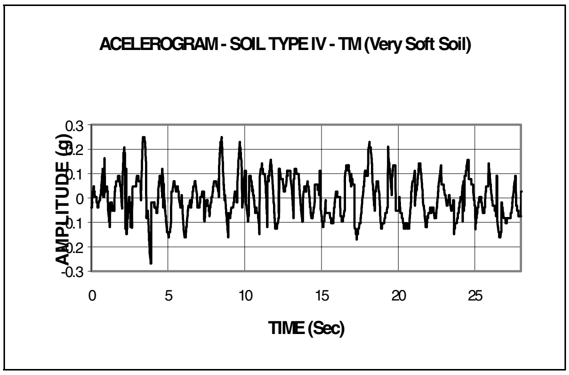

Figure 5: Acelerogram - Soil Type IV - TM (Very Soft Soil)

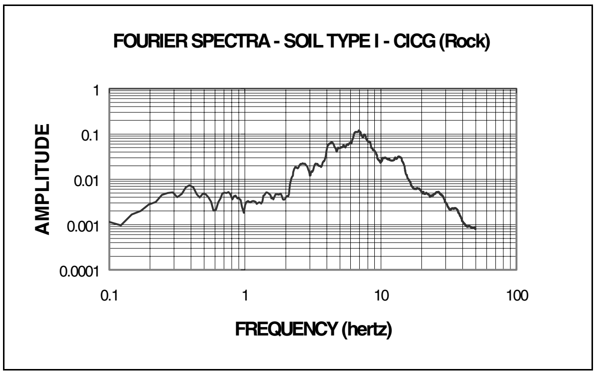

Figure 6: Fourier Spectra - Soil Type I - CICG (Rock)