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Code provisions for structures on deep soft sites

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ABSTRACT

It is generally recognized that soil deposits can amplify seismic ground motions as these propagate from the underlying rock strata to the surface. During the 1985 Mexican earthquake, immense amplifications in the Lake Zone of Mexico City resulted in unprecedented loss of life and destruction.

In an effort to realize the engineering significance of the amplification effects observed in Mexico City, the foundation factor (F) in NBCC was increased to 2.0 for very soft and soft fine-grained soils with depth greater than 15 m. Emergency changes were also introduced to the design code for structures in the Federal District of Mexico City, increasing design forces for structures on deep soft deposits by as much as 67%. In this paper, the dynamic response of bilinear sdof systems is used to investigate the adequacy of these protection measures and the significant differences between both codes in accounting for amplification effects of deep soft deposits.

INTRODUCTION

On 19 September 1985, a large earthquake (M_s = 8.1) occurred near the south coast of Mexico. Mexico City, situated at 300-400 km from the epicentre, sustained severe building damage and heavy casualties. Buildings that sustained moderate or higher levels of damage were restricted exclusively to the Lake Zone (EEFIT 1986). Dobry and Vucetic (1987) demonstrated that the immense amplifications recorded in Mexico City were primarily due to the almost linear behaviour of Mexico City clays within the range of shear strains (≤.3%) induced by the earthquake. Taking into consideration that at such long epicentral distances, a substantial portion of the seismic energy at bedrock is associated with low frequencies, the levels of site amplification recorded in Mexico City

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should not be surprising. In fact, similar damage patterns were observed during the 1957 earthquake ($M_s = 7.5$). During that event, 96% of the damage sustained in Mexico City was restricted to the Lake Zone (Mitchell et al. 1986). Since soil deposits tend to focus the energy of the ground motions at frequencies in the neighbourhood of the natural frequency of the soil deposit, higher damage potential is expected to structures of fundamental periods close to the site period. This argument is substantiated by the damage pattern observed during the 1985 earthquake for which the most affected structures were over 6 storeys in height (Mitchell et al. 1986). Soil deposits in the Lake Zone (Zone III in Mexico City code) are characterized by natural periods in excess of 1.0 sec and depths exceeding 20 m of compressible soft clay (Gomez et al. 1988).

Following the 1985 earthquake, a presidential decree was published in Mexico giving emergency code changes for construction in the Federal District. The emergency changes involved increasing design forces by as much as 67% for buildings in the Lake Zone (Mitchell et al. 1986). Based on observations during the 1985 event, a foundation factor (F) of 2.0 was specified in the NBCC (National Building Code of Canada) for very soft and soft fine-grained soils with depth greater than 15 m (NBCC 1990). This implies a 33% increase in design forces as compared to the previous value of F=1.5.

Heidebrecht et al. (1990) had shown that the amplification levels associated with deep soft clay sites and subjected to low intensity, very low a/v (ratio of peak ground acceleration to peak ground velocity) motions are underpredicted by the factor F=2.0 provided in NBCC 90. Their results were based on computations of the response of simple elastic systems. In the current study, evaluation of code provisions for site amplification is extended to computations of the nonlinear response of simple bilinear sdof (single degree of freedom) systems. Ground motions included in the study are exclusively restricted to those recorded in Mexico City during the 1985 main shock. The immense amplifications recorded in Mexico City are a valid test of NBCC 90 provisions because as stated by Mitchell et al. (1986) :

1. The potential for large earthquakes near Vancouver could give rise to amplification effects by the thick recent sediments in the Fraser River delta similar to those recorded in Mexico City.
2. Sensitive clays are present in the seismically active parts of the St. Lawrence River valley.

As the Mexico City code is based on past experience with substantial site amplification effects, it is instructive to compare it to the NBCC 90 in this study. Mexico City code, henceforth, shall refer to the code after the 1985 emergency changes (Gomez et al. 1988).

ANALYTICAL MODEL

Bilinear sdof systems are used to model the nonlinear response of structures having fundamental periods in the range 0.1 to 4.0 sec. For longer structural periods, sdof systems are not a reliable model for actual structures due to lack of representation of higher mode effects. Strain hardening in actual structures is accounted for by a post-yield stiffness taken as 3% of the initial elastic stiffness. A viscous damping of 5% is allowed for in the model. The computer program used for computations of the dynamic response was provided by Zhu (1985).

The program has been modified to specify the yield strength (P_y) of the sdof systems according to either NBCC 90 or Mexico City code rather than NBCC 85. In this paper, the measure of the dynamic response is the peak ductility (μ). Peak ductility is the ratio of the peak relative displacement of the sdof system to its yield displacement.

GROUND MOTION RECORDS

Of the strong motion records obtained during the 1985 main shock, three records from the Hill Zone and five records from the Lake Zone are used as ground motion excitations in this study. Information on these records is presented in Tables 1 and 2. Locations of the recording stations are shown on Fig. 1. The Hill Zone (Zone I in Mexico City code) is characterized by shallow (less than 3 m) deposits of very competent soils and is generally referred to as firm ground.

For the purpose of this study, the 1985 main shock is considered to be the design earthquake for Mexico City due to the following reasons :

1. Design spectra in Mexico City code are based on motions recorded since December 1959 (Rosenblueth 1979) and the 1985 main shock is the largest event recorded since then.
2. Peak ground accelerations recorded in the Hill Zone during the 1985 main shock (≈70 year return period) are more consistent with the design spectra in Mexico City code than the .05 g value (g is the acceleration due to gravity) based on the longer return period of 100 years.

Consequently, the zonal velocity ratio used to specify design base shear in NBCC 90 is based on peak ground velocities recorded in the Hill Zone during this event and which are .1 m/s on average. Moreover, the records obtained during the 1985 event classify Mexico City as a region of low a/v motions. This is of consequence in specifying the seismic response factor, S, in NBCC 90.

DESIGN BASE SHEAR

NBCC 90

The design base shear, V, is given by :

$$V = (V_e/R)U \tag{1}$$

Where :

$$V_e = vSIFW \tag{2}$$

= base shear for elastic response.

R = force modification factor.

U = calibration factor based on experience = 0.6

v = zonal velocity ratio = .1 for Mexico City.

S = seismic response factor (Z_a/Z_v less than 1.0 for Mexico City).

I = importance factor = 1 for structures of normal importance.

F = foundation factor = 1 for Hill Zone.
= 2 for Lake Zone (FS not to exceed 3.0).

W = dead load.

Eq. 1 may also be rewritten as :

$$V(1/U) = V_e/R \tag{3}$$

As discussed by Tso and Naumoski (1991), (1/U) in the left hand side of Eq. 3 is basically an overstrength factor reflecting the fact that actual buildings designed to the code base shear, V, will usually sustain larger loads prior to yielding. Since the analytical model in this study does not inherently model this overstrength effect, it should be explicitly incorporated in specification of the yield strength (P_y). Thus, for the purpose of achieving realistic estimates of the ductility demands in actual structures, P_y is specified as :

$$P_y = V(1/U) \tag{4}$$

Mexico City code

For buildings of normal importance, the design base shear, V, is given by:

$$V = C_s W \tag{5}$$

For a building having a fundamental period, T :

$$C_s = [(1+3T/T_a)c/4]/Q' \qquad T<T_a \tag{6}$$

$$Q' = [1+(Q-1)T/T_a] \tag{7}$$

$$C_s = c/Q \qquad T_a \le T \le T_b \tag{8}$$

$$C_s = [q(1-r(1-q))+1.5rq(1-q)]c/Q \qquad T>T_b \tag{9}$$

$$q = (T_b/T)^r \tag{10}$$

Q is a ductility factor that corresponds closely to the force modification factor, R, in NBCC 90. For the purpose of the current study, both factors are considered equivalent. Other factors in Eqs. 6 to 10 are given in Table 3. As stated earlier, overstrength should be allowed for in specifying P_y. For consistency and for comparisons to be made between both codes, the same overstrength factor (1/U) used for NBCC 90 shall be applied to the Mexico City code design base shear, V, to derive P_y as given by Eq. 4.

The design base shear, V, according to NBCC 90 and Mexico City code for the Hill Zone and Lake Zone is presented in Figs. 2,3 in terms of the base shear coefficient. This dimensionless coefficient is the ratio of the design base shear, V, to the dead load, W. It is obvious that V based on Mexico City code are higher than those based on NBCC 90 with larger discrepancies for the Lake Zone. An important feature of Mexico City code lacking in NBCC 90 is adoption of period dependent reduction factors for buildings having a fundamental period less than .2 sec (Hill Zone) and .6 sec (Lake Zone). This is in recognition of the fact that use of period independent reduction factors results in short period structures being subjected to excessive ductility demands. Tso and Naumoski (1991) propose adopting a period dependent reduction factor for structures of fundamental periods less than .5 sec. However, results based on the current study, and presented in this paper, tend to support the approach adopted in the Mexico City code, whereby the range of structural periods over which the reduction factor becomes period dependent is wider for structures on soil deposits than for structures situated directly on rock.

RESULTS

In this paper, results are presented for R(or Q)=2.0 and 4.0 to model structures with nominal ductility and ductile structures respectively.

Hill Zone (firm ground)

Mean plus one standard deviation (M+SD) peak ductility (μ) results are presented in Figs. 4 and 5 based on the six components recorded in the Hill Zone. On each figure is superimposed, as a solid line, the ductility level commensurate with the reduction factor used. This level signifies the design (tolerable) ductility limit associated with the specified reduction factor. Figs. 4 and 5 show that both codes provide an excellent level of protection for structures in the Hill Zone. Mexico City code appears to be more conservative by limiting μ to values less than 1.2 corresponding to a tolerable limit of 2.0 (Fig. 4) and to values less than 2.0 corresponding to a tolerable limit of 4.0 (Fig. 5). NBCC 90, on the other hand, maintains the ductility demands closer to, yet lower than, the specified tolerable limits. Since the design base shear based on NBCC 90 is lower than that based on Mexico City code as indicated by Fig. 2, it appears that NBCC 90 offers a more economical design for structures in the Hill Zone without violating the ductility potential for these structures. Based on the above, any violation of the ductility potential for structures in the Lake Zone, and designed to the base shear provided in NBCC 90, would imply deficiencies in the foundation factor, F, in accounting for the actual site amplification effects.

Lake Zone (compressible soil)

M+SD peak ductility results are presented in Figs. 6 and 7 based on the ten components recorded in the Lake Zone. For results based on NBCC 90, a foundation factor (F) of 2.0 is incorporated in the design base shear. Again, the design ductility limit is superimposed, as a solid line, on each figure. In Fig. 6, it is observed that for design based on NBCC 90, the tolerable ductility limit is exceeded with the exception of the structural period range of .2 to .8 sec. For structural periods close to the natural periods of the sites included in the study (1.9-3.9 sec), μ is highest at almost twice the tolerable ductility limit. For design based on Mexico City code, μ does not exceed 1.5, indicating adequate protection to structures of nominal ductility in the Lake Zone.

In Fig. 7, it is again observed that while peak ductilities up to 4.0 can be tolerated, design according to Mexico City code limits μ to values no higher than 2.2. As for design based on NBCC 90, ductility demands exceed the tolerable limits within the whole range of structural periods included in the study. The excessive ductility demand for structures of fundamental periods less than 1.0 sec is a direct consequence of using a period independent force modification factor. However, it can be observed that if a period dependent reduction factor is applied only to structural periods less than .5 sec, ductility demand would still be excessive in the .5 to 1.0 sec period range.

FOUNDATION FACTOR

Results for the Lake Zone indicate that the foundation factor of 2.0 provided in NBCC 90 does not adequately account for the site amplification effects recorded in Mexico City. To investigate this point further, the values of F required to limit M+SD ductility demands, for structures in the Lake Zone, and designed to the NBCC 90 base shear, to the tolerable ductility limits shown in Figs. 6,7 are computed through a process of iterations. In Fig. 8, results are directly compared to the F values provided in NBCC 90. It is observed that for structural periods close to the natural periods of the sites included in the study, actual requirements for F range from 3.0 to 3.5 compared to 2.0 in NBCC 90. This observation holds for both R=2.0 and 4.0. For the case of R=4.0, the demand for design forces higher than those provided in NBCC 90 for structural periods less than 1.0 sec is merely a consequence of using period independent reduction factors. This argument is substantiated by the fact that for the case of R=2.0, the foundation factor provided in the code is quite adequate for that range of structural periods.

Heidebrecht et al. (1990) had studied a deep soft clay site in Arnprior, Ontario and another in U.K. Their results indicate that for structural periods in the range of the site period, actual requirements for the foundation factor reach 3.0, higher than the value of 2.0 provided in NBCC 90.

CONCLUSIONS

Following are the main conclusions based on the current study :

1. F=2.0 provided in NBCC 90 for deep soft clay deposits does not adequately account for site amplification effects similar to those recorded in Mexico City during the 1985 earthquake.
2. There is a need for adoption of period dependent force modification factors in NBCC. The range of structural periods for which the force modification factor is period dependent should be a function of the geological conditions at the site being considered.
3. Mexico City code provides adequate protection to structures of nominal ductility and ductile structures in both the Hill Zone and Lake Zone.
4. Mexico City code is an example of how microzonation offers the better alternative for future aseismic codes.

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Table 1. Ground motion records from the Hill Zone, Mexico City.

Station	Comp	PGA*	PGV**
CU01	S00E N90W	.0236 .0341	.1020 .0938
CUIP	N00E N90W	.0323 .0353	.1025 .0937
CUMV	S00E N90W	.0381 .0396	.0919 .1101

* Peak ground acceleration in g.
** Peak ground velocity in m/s.

Table 2. Ground motion records from the Lake Zone, Mexico City.

Station	Comp	PGA	PGV
CAF	S00E N90W	.0821 .0965	.2485 .3757
CAO	N00E N90E	.0705 .0820	.3498 .4186
SCT	S00E N90W	.0999 .1712	.3874 .6050
TLB	N00E N90W	.1385 .1087	.6410 .4461
TLD	N00E N90W	.1199 .1137	.3490 .3606

Table 3. Factors used to derive design base shear according to Mexico City code.

Zone	c	T_a(sec)	T_b(sec)	r
Hill Zone	.16	0.2	0.6	0.5
Lake Zone	.40	0.6	3.9	1.0

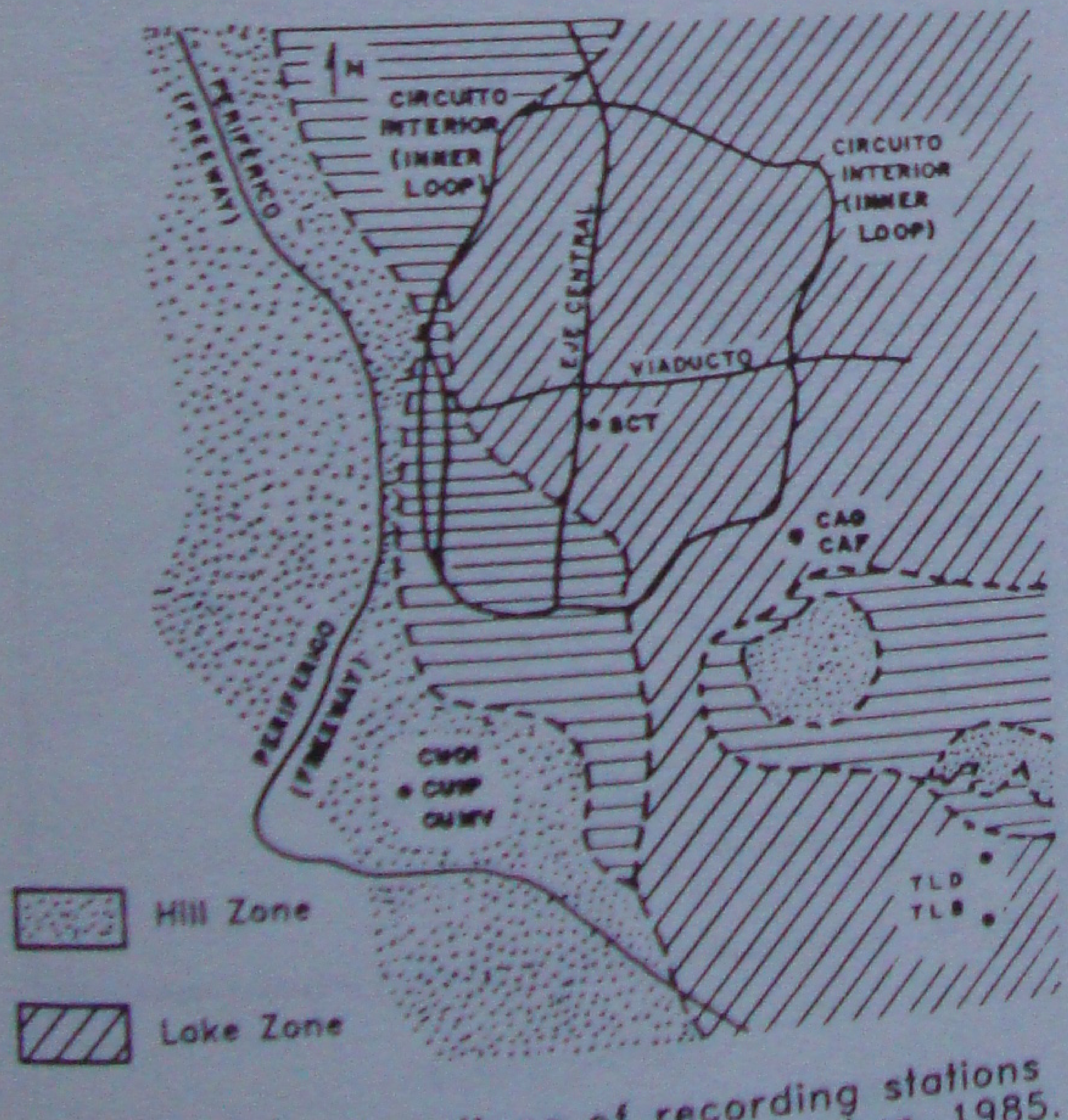

Figure 1. Locations of recording stations for the earthquake of 19 September, 1985.

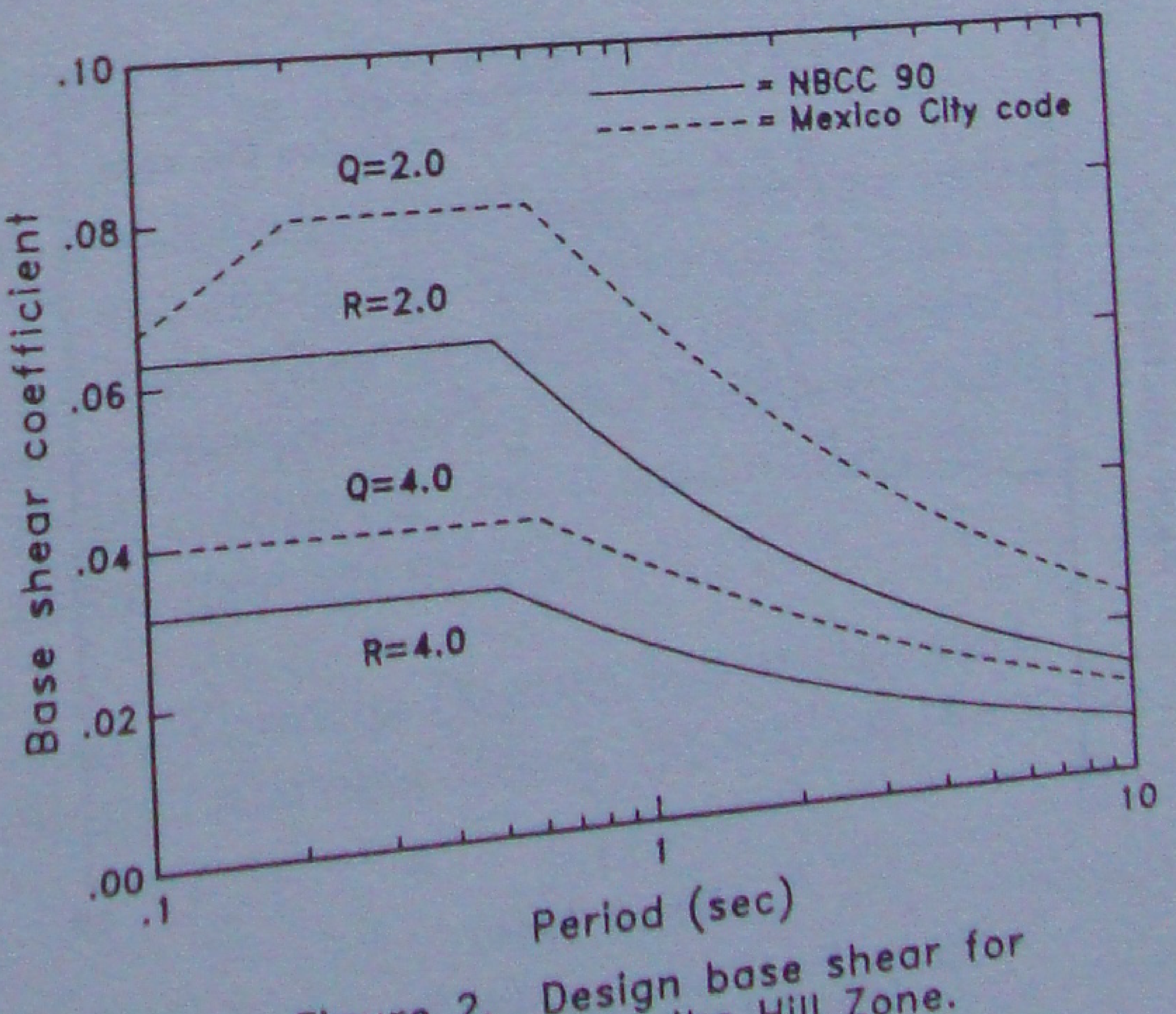

Figure 2. Design base shear for structures in the Hill Zone.

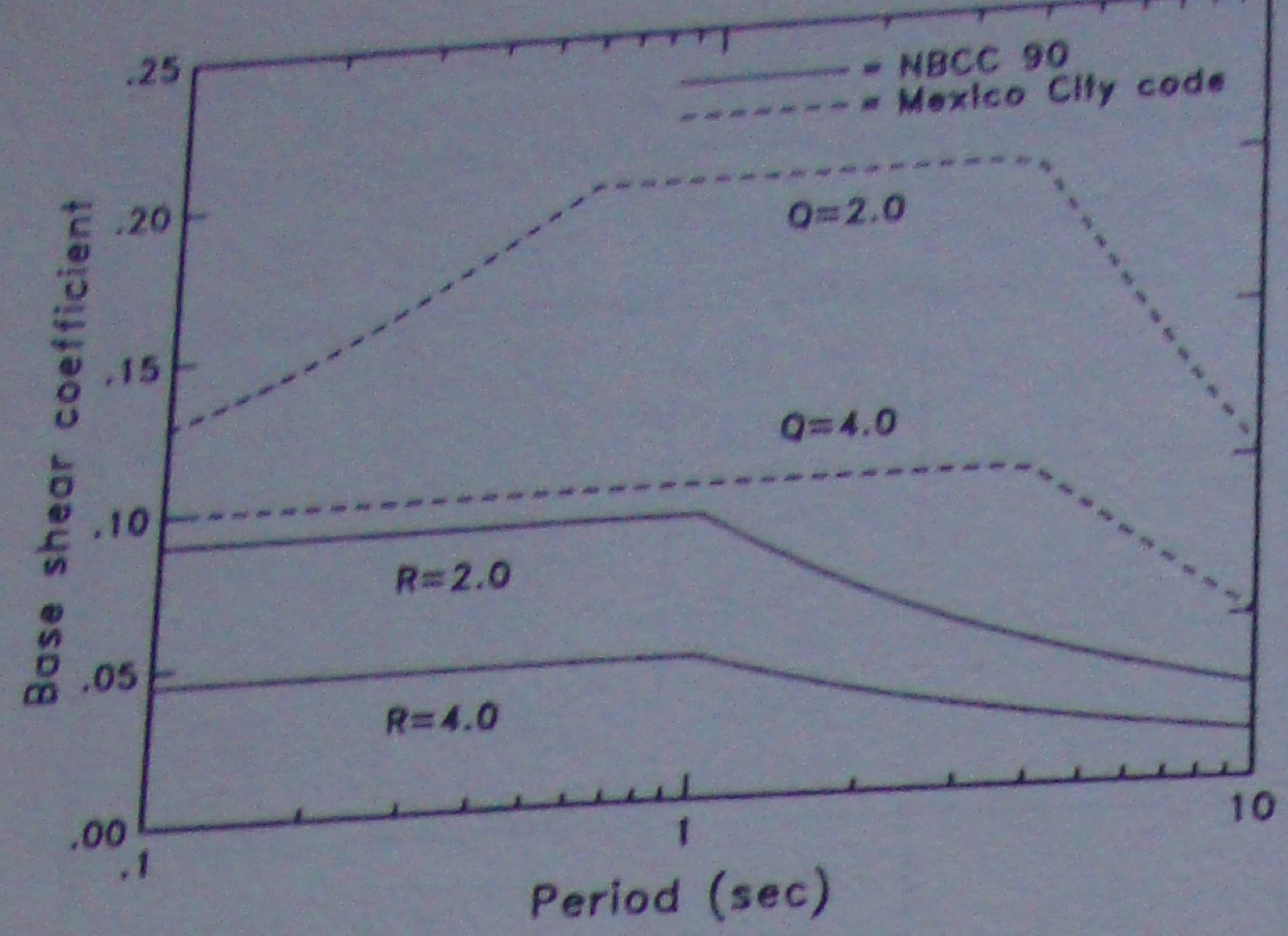

Figure 3. Design base shear for structures in the Lake Zone.

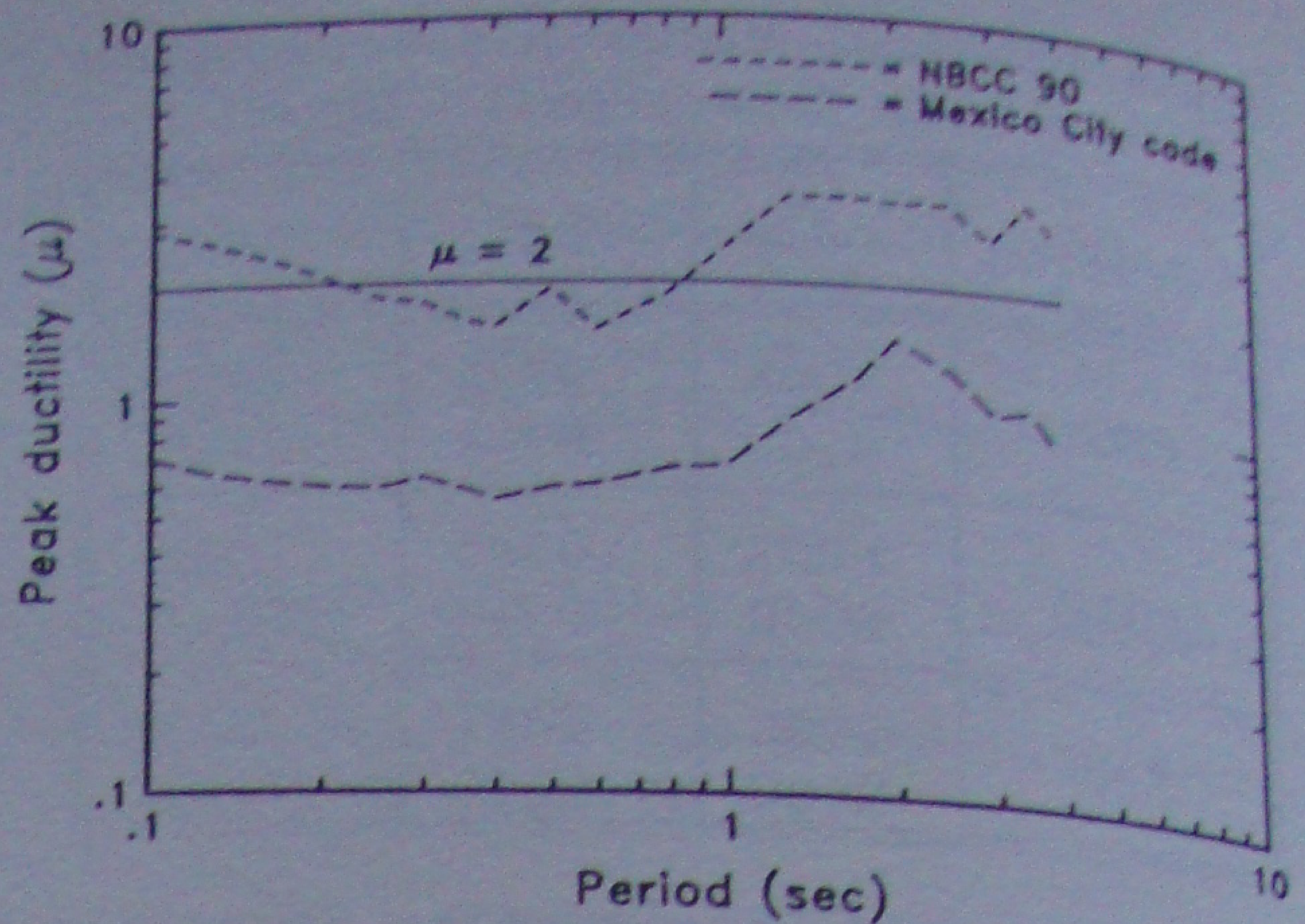

Figure 6. Peak ductility for structures of nominal ductility in the Lake Zone.

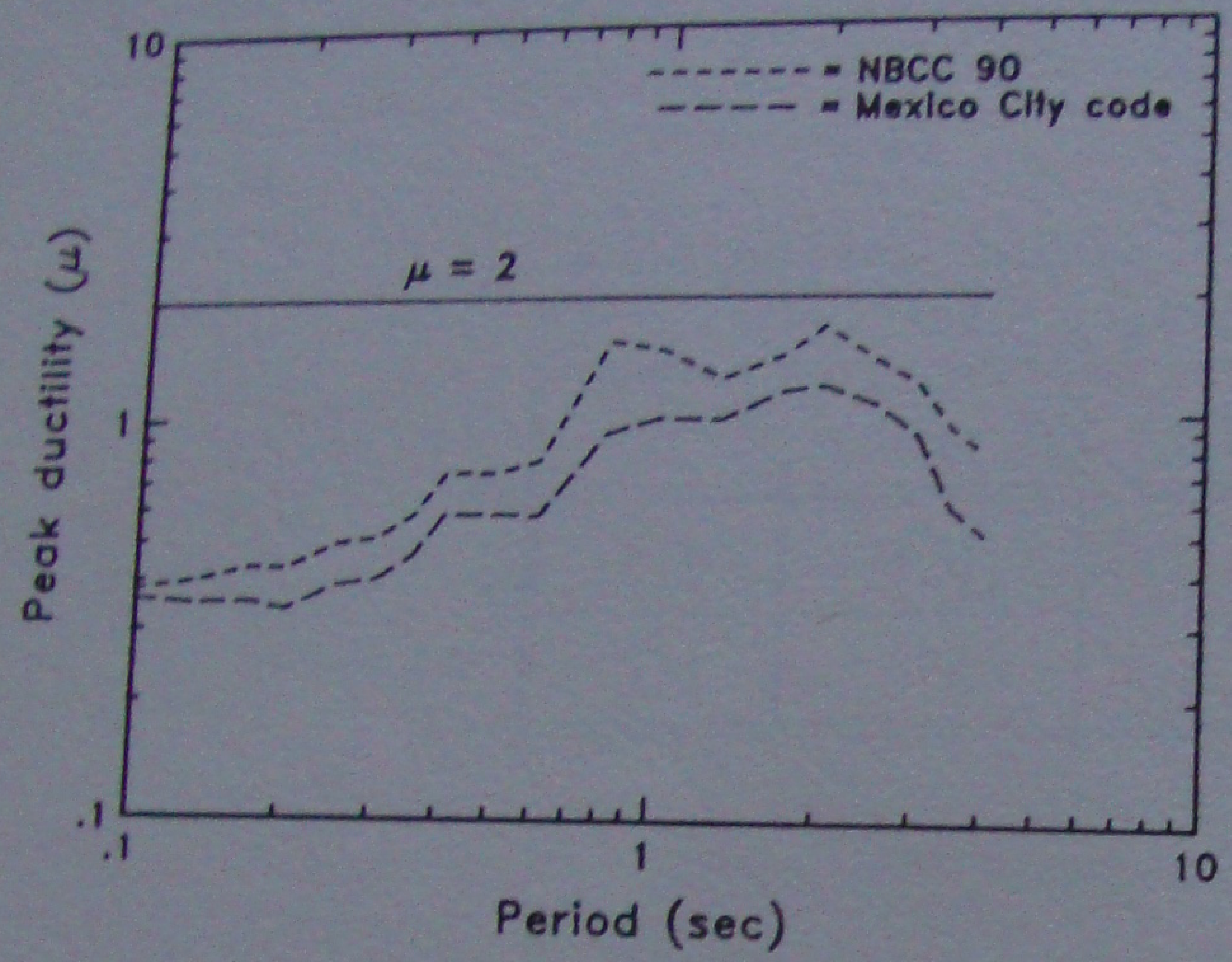

Figure 4. Peak ductility for structures of nominal ductility in the Hill Zone.

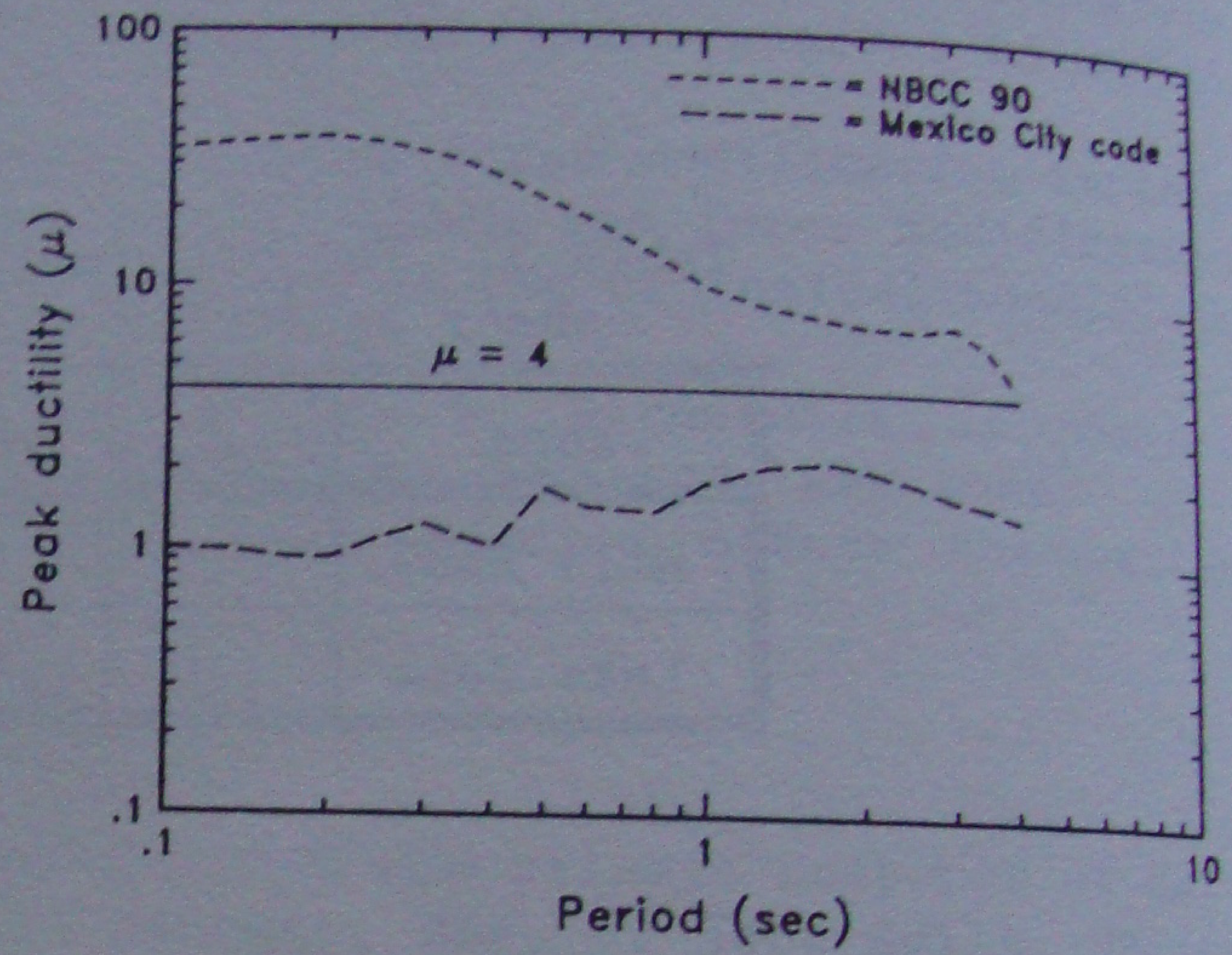

Figure 7. Peak ductility for ductile structures in the Lake Zone.

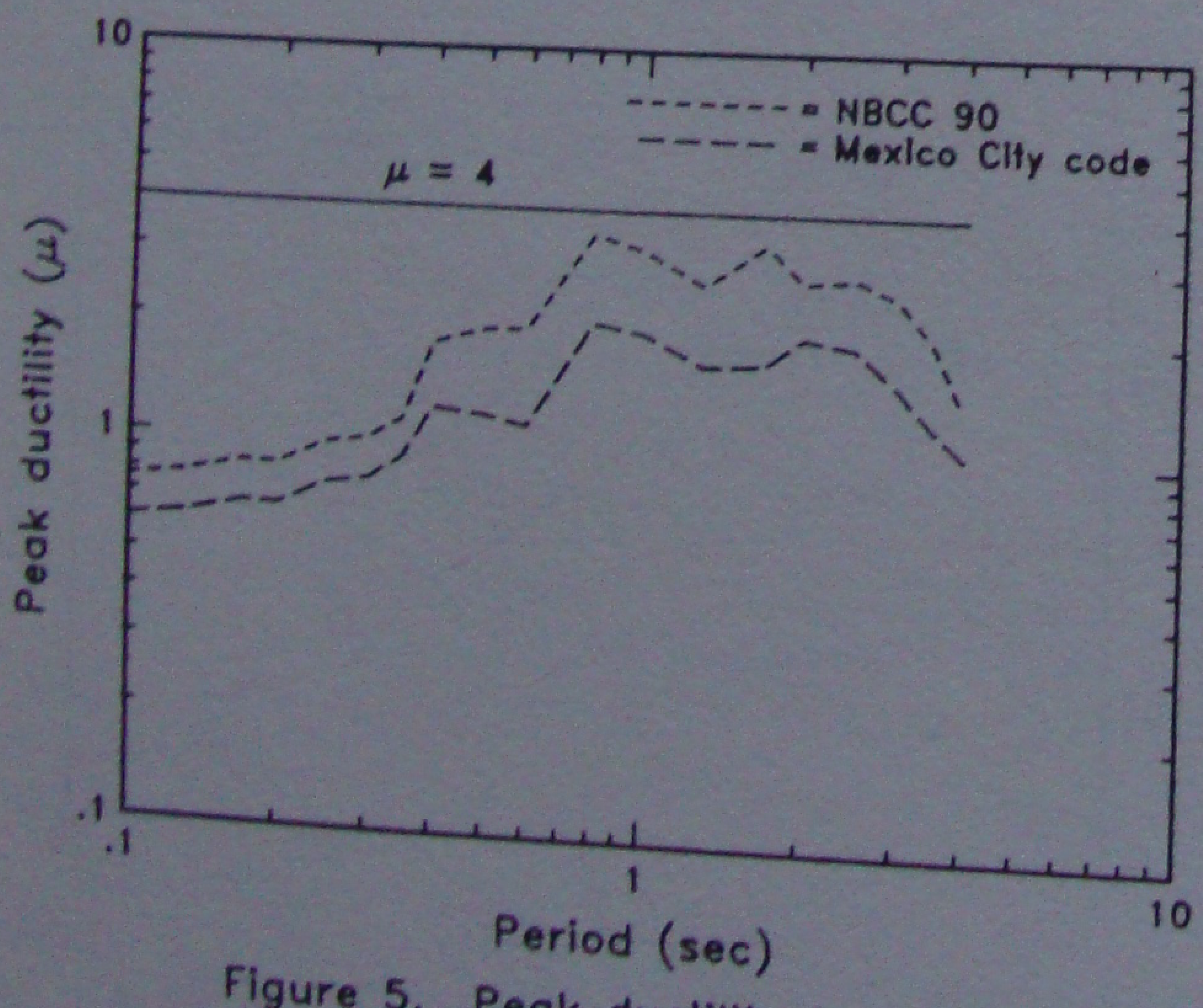

Figure 5. Peak ductility for ductile structures in the Hill Zone.

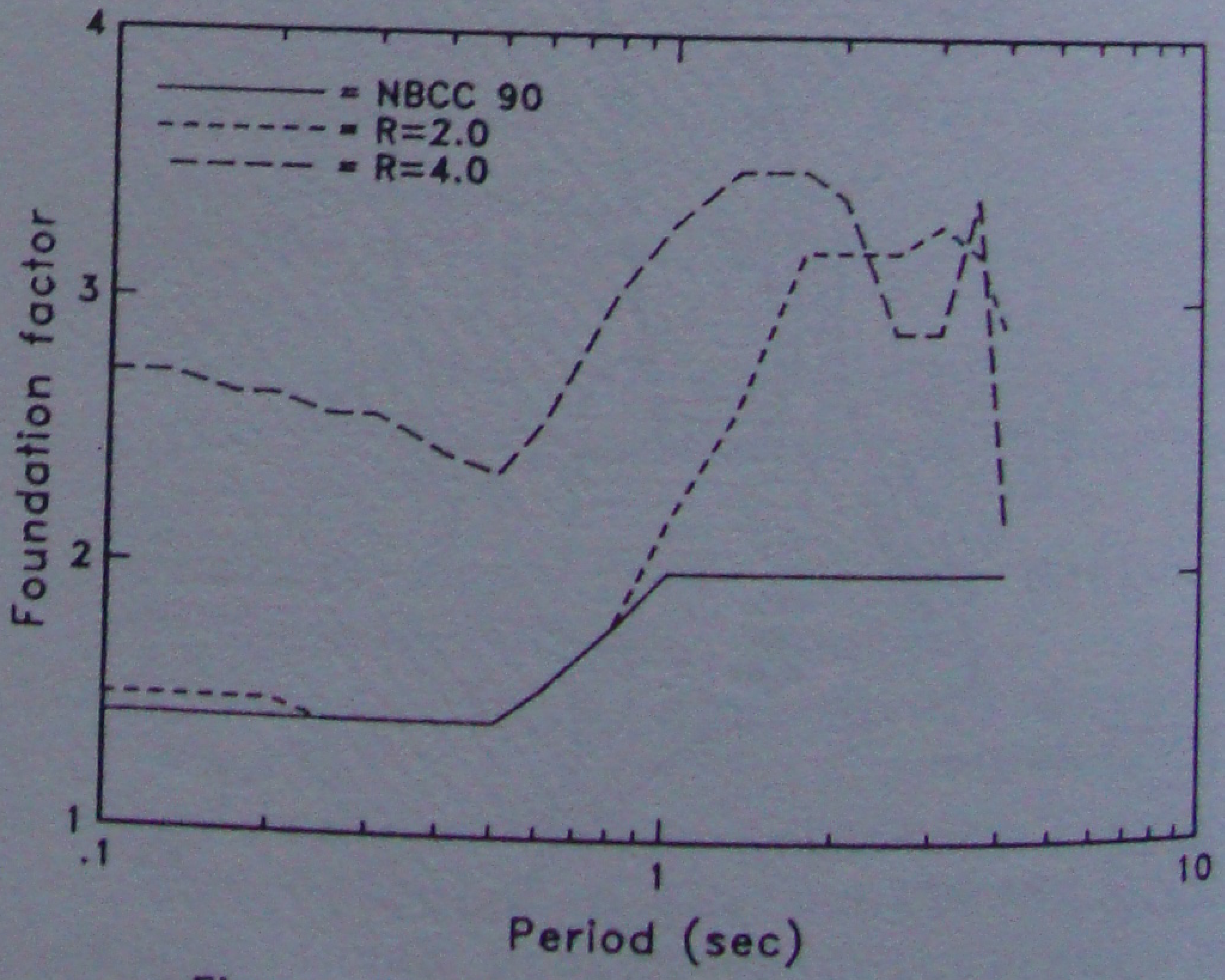

Figure 8. Required foundation factor for structures in the Lake Zone.