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Seismic hazard and structural safety in Mexico City

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ABSTRACT: A bayessian seismic hazard model is applied to estimate the seismic hazard in Mexico's City Lake zone, and reliability concepts, as well as Monte Carlo simulation techniques are used to evaluate its impact on the safety of single frame R/C systems with and without masonry fill. The systems were designed according with the 1976 and 1987 Mexico City Seismic Code (MCSC). The conclusions of the work are: 1) the seismic hazard estimated here agrees with the one reported by Chavez and de Leon in the 1984 WCEE, 2) the probability of failure (Pf) of the bare frames designed with the 1976 MCSC are three time larger than the ones designed with the 1987 MCSC, both are of the order of 10^{-3}, 3) the Pf of the infilled frames are one order larger than the ones for bare frames, this is probably due to the crude modeling of the contribution of the infill to the seismic behaviour of this type of structures.

1 INTRODUCTION

The occurrence of recent large earthquakes such as the 1985 in Mexico, the 1985 in Chile, the 1988 in Armenia, and the 1989 in California, and their destructive effects on constructions designed accordingly with the seismic codes of those seismic regions, show the importance of performing seismic hazard studies that reflect not only the uncertainties about the ocurrence, location, and magnitude of future earthquakes, but also the particular characteristics of the ground motions, (i.e. amplitudes, frequency content, duration) expected at specific sites on a given seismic region. Also, the study of the damage observed on constructions located in the regions mentioned above, has clearly shown that in order to estimate the seismic safety of structural systems, it is necessary to incorporate the nonlinear behaviour of the systems under seismic loading, as well as the uncertainties on the parameters used for their seismic design.

After the 19/09/1985 Mexico's earthquake, about 100 instruments have been installed in Mexico City, and a number of accelerograms have been recorded. Also, new statistical data has been published in relation with the mechanical and geometrical parameters of R/C elements designed with the 1976 and 1987 versions of the mentioned Code.

In this paper we study the impact of this new information on the estimation of the seismic hazard in Mexico's City Lake zone, and on the probability of failure, **Pf**, of simple R/C systems with and without infill, designed accordingly with those Codes. The methodology we applied is basically the same as the one proposed by Chavez and de Leon (1984), however, in the present research the DRAIN code was utilized to compute the seismic response of the structural systems under consideration.

2 SEISMIC HAZARD IN MEXICO CITY LAKE ZONE

As in the previous works (Chavez and de Leon, 1984, Chavez, 1987) here the strategy for the estimation of the seismic hazard in Mexico's City Lake zone, rely upon the actual recordings of the ground motion parameters observed for medium to large size interplate and intraplate seisms, in diferent sites of the zone. In Table 1 we show the hypocentral data of the earthquakes used in the study. The time span of the data is about 30 years, it includes 20 seisms, 15 of them are superficial depth events, and the other 5 have an intermediate depth. Their superficial wave magnitudes, Ms, vary from 5.6 to 8.1. The epicentral distances of the earthquakes fluctuates between 35 to 466Km. In the same Table we present 50 values of the ground motion parameters of interest in this work (the maximum acceleration, A_{max}, and the maximum ordinate of the seudovelocity spectra for a 10% of critical damping, $S_v(10\%)$), which were recorded during the 20 earthquakes, in 23 different sites distributed in Mexico's City Lake zone. The observed values of A_{max}, and $S_v(10\%)$ varied from 6 to 167cm/s^2, and 2 to 213cm/s, respectively. The larger values correspond to the maximum values recorded in Mexico City during the 19/09/1985 earthquake.

The parameter we choose to estimate the sesmic hazard of Mexico's City Lake zone is Sv(10%), which shows a reduced variability, for a wide band of frequencies of interest. In Figure 1 we show the annual rate of exceedance of the Sv data, with squares the data up to 1980 (used in Chavez and de Leon, 1984, Chavez, 1987) and with crosses the corresponding to the data set of Table 1. Notice that the two data sets overlap up to a Sv(10%) value of 50cm/s.

If we assume that the occurrence of the two types of earthquakes (intraplate and interplate) follows a Poisson process, such that the intensities and the detailed ground motion associated to any two different events are statistically independent and identically distributed, the probability P, that a particular Sv is exceeded can be expressed as

$$P(Sv) = \exp(-\nu(Sv)To) \qquad (1)$$

where To is the lapse of interest and the rate of occurrence $\nu(Sv)$ can be calculated with the expression

$$\nu(Sv) = k(Sv^{-q} - Sv_1^{-q}) \qquad (2)$$

here k and q depend on the seismicity of the region where the site is located, and Sv_1 is the maximum Sv which may occur in the site. The estimation of these parameters for the 30 years data of Figure 1, was performed by applying bayessian statistics , the resulting expected posterior values of k and q were 25 and 2 respectively. The expected Sv_1 value for this case is assumed to be 400cm/s, which is an estimate of this parameter. This estimate takes into account that an Sv_1(10%) equal to 214cm/s was the maximum recorded in 1985 Mexico's earthquake, Table 1.

In Fig 1 we present the $\nu(Sv)$ curve corresponding to the mentioned k, q, and Sv_1 values. This curve is very similar to the one proposed by Chavez and de Leon (1984). For the present study the value of To equal to 50 years is of interest, therefore the resulting P(Sv) accordingly with equation 2 is shown in Fig 2. An important feature of this curve is that it is bounded (in this case at Sv_1 = 400cm/s). This is in agreement with recent experimental results that show that the dynamic shear resistance of Mexico's City Lake zone soils is limited, under a cyclic type of loading which simulates Mexico City 1985 recordings, Diaz-Rodriguez (1989). This means that the amplitudes of the ground motions expected at the top of those soils has also to be bounded, and therefore the value of Sv(10%).

The second objective of this work requires to have samples of accelerograms which reflect the seismic hazard of Mexico's City Lake zone (which is synthesized in Figures 1 and 2), as well as the characteristics of the ground motions

Table 1. Hypocentral data of earthquakes used in the study, including the maximum acceleration (A_{max}) recorded at specific sites in Mexico's City Lake zone and the corresponding maximum ordinate of the seudovelocity spectra for a 10% of critical damping (SV(10%)).

No.	Date	Lat.	Long.	Depth (Km)	.Ms	Epicentral Dist. (Km)	Recording Site	A max. (cm/s2)	SV (10%) (cm/s)
1	611210	19.70	99.10	33	5.6	35.91	Alameda Central	17.00	06.50
2	620511	17.25	99.58	33	7.2	308.00	Alameda Central	46.00	38.00
3	620519	17.12	99.57	33	6.9	232.97	Alameda Central	30.00	33.00
4	621130	17.30	99.43	--	5.8	203.00	Alameda Central	06.20	02.40
5	640706	18.03	100.77	55	7.2	225.36	Edif. M. Glez.	30.00	25.00
6	640706	18.03	100.77	55	7.2	225.36	Edif. Hgo. P.	42.50	47.00
7	640706	18.03	100.77	55	7.2	225.36	Edif. Hgo. P.	47.50	52.00
8	640706	18.03	100.77	55	7.2	225.36	Nonoalco Atiz. S.	24.00	21.00
9	650823	16.30	95.80	33	7.8	561.24	Nonoalco Atiz. S.	20.00	20.00
10	651209	17.30	100.00	57	6.5	329.81	Nonoalco Atiz. S.	09.50	09.00
11	680701	17.64	100.27	41	6.7	243.39	Nonoalco Atiz. S.	15.00	15.00
12	680802	16.60	97.70	33	7.4	370.79	Nonoalco Atiz. S.	40.00	50.00
13	680802	16.60	97.70	33	7.4	379.79	Nonoalco Atiz. P.	30.00	32.00
14	730130	18.40	101.80	33	7.5	322.00	Pal. de los Depor.	16.67	41.75
15	730828	18.27	96.60	84	6.8	290.00	Pal. de los Depor.	19.52	36.19
16	781129	16.00	96.69	19	7.8	466.00	Nonoalco Atiz. P.	24.57	15.79
17	790314	17.81	101.28	15	7.6	308.00	Texcoco Sosa	54.88	43.01
18	790314	17.81	101.28	15	7.6	304.00	Texcoco Centro L.	48.19	35.90
19	790314	17.81	101.28	15	7.6	304.00	Texcoco Chimal.	31.86	17.76
20	790314	17.81	101.28	15	7.6	285.00	Sismex SHAOP	33.37	32.55
21	790314	17.46	101.46	15	7.6	283.77	Nonoalco Atiz. S.	41.57	52.00
22	801024	17.90	98.15	65	7.0	187.64	Lot. Nacional S.	24.86	17.82
23	801024	17.90	98.15	65	7.0	187.64	Texcoco Chimal.	30.88	14.93
24	801024	17.90	98.15	65	7.0	187.64	Sismex SHAOP	33.65	20.52
25	801024	17.90	98.15	65	7.0	187.64	Texcoco Centro L.	47.19	29.27
26	801024	17.90	98.15	65	7.0	187.64	Texcoco Viv. C.	46.62	08.98
27	801024	17.90	98.15	65	7.0	187.64	Nonoalco Atiz. S.	33.01	19.47
28	801024	17.90	98.15	65	7.0	187.64	Texcoco Sosa	42.47	20.23
29	811025	17.75	102.25	20	7.3	370.00	Texcoco Centro L.	29.69	21.27
30	811025	17.75	102.25	20	7.3	373.00	Texcoco Sosa	28.32	22.22
31	820607	16.54	98.42	21	7.0	324.00	Texcoco Chimal.	22.27	15.29
32	820607	16.54	98.42	21	7.0	328.00	Lot. Nacional S.	15.02	10.85
33	820607	16.54	98.42	21	7.0	342.00	Texcoco Sosa	32.27	22.08
34	820607	16.61	98.36	34	6.9	333.00	Nonoalco Atiz. S.	21.58	19.66
35	820607	16.61	98.36	34	6.9	331.00	Nonoalco Atiz. S.	11.64	09.38
36	850919	18.40	102.71	16	8.1	394.20	Tlahuac Deport.	117.67	99.29
37	850919	18.40	102.71	16	8.1	394.20	Texcoco Sosa	103.04	90.72
38	850919	18.40	102.71	16	8.1	394.20	C. Abast. Frig.	94.62	137.11
39	850919	18.40	102.71	16	8.1	394.20	C. Abast. Ofic.	80.40	126.21
40	850919	18.40	102.71	16	8.1	394.20	S. C. T.	167.92	213.86
41	850921	17.62	101.82	22	7.6	366.50	Tlahuac Deport.	51.58	50.32
42	850921	17.62	101.82	22	7.6	366.50	C. Abast. Ofic.	48.66	54.43
43	850921	17.62	101.82	22	7.6	366.50	C. Abast. Frig.	42.43	39.23
44	860430	18.40	102.97	21	7.0	411.80	C. Abast. Frig.	13.42	15.77
45	860430	18.40	102.97	21	7.0	411.80	C. Abast. Ofic.	32.27	68.34
46	890425	16.53	99.55	33	6.9	290.00	Col. Roma	55.04	34.40
47	890425	16.53	99.55	33	6.9	289.00	S. C. T.	37.89	38.66
48	890425	16.53	99.55	33	6.9	286.00	C. Abast. Ofic.	34.17	49.59
49	890425	16.53	99.55	33	6.9	264.00	Tlahuac Bombas	48.13	61.44
50	890425	16.53	99.55	33	6.9	264.00	D.F. La Viga	33.77	37.25

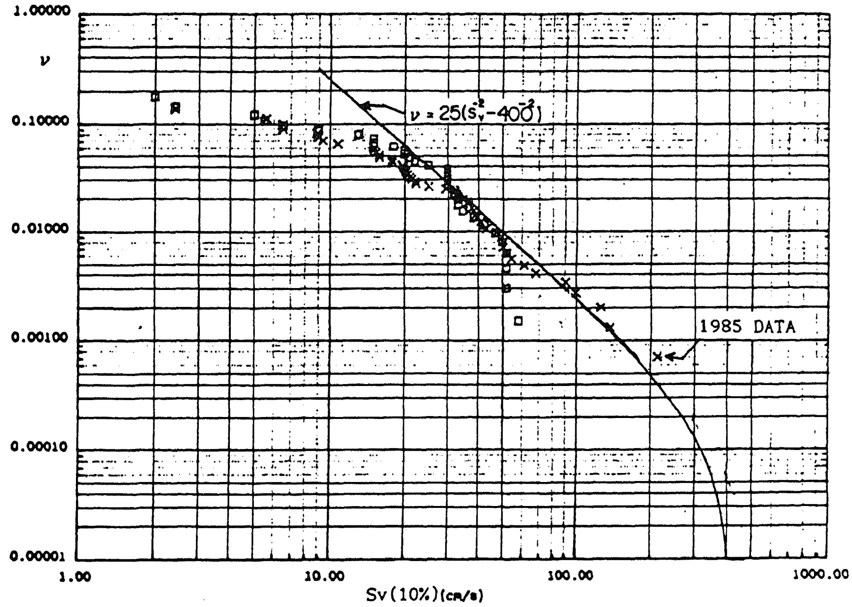

Figure 1. Annual rate of exceedance, ν, of the maximum values of the seudovelocity spectra, Sv(10%), observed in Mexico City Lake zone for a 30 years sample.

expected in the future in this region. Therefore, the sample of 50 accelerograms associated to the earthquakes included in the study (Table 1) were selected as representative of the future ground motions at the site. These motions were scaled by aplying Monte Carlo simulation techniques in order to generate families of accelerograms which comply with the seismic hazard, and the details of the ground motions expected in the mentioned zone. The peak ground accelerations of the data set varied from 6.2 to 167cm/seg^2, and their duration from 50 to 180s, Table 1.

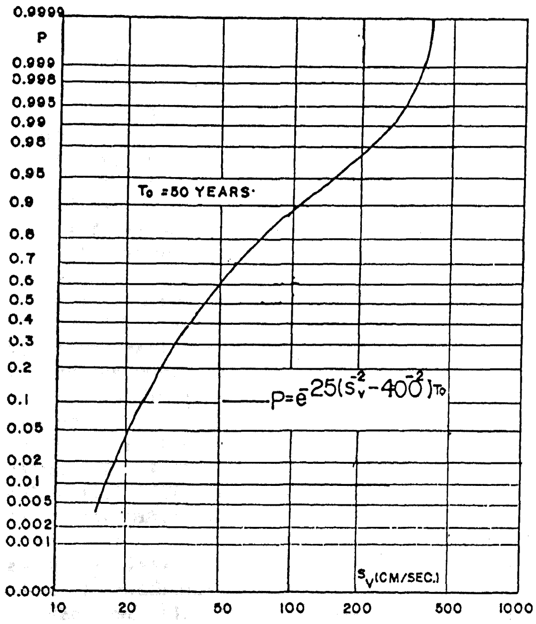

Figure 2. Accumulated probability distribution, P, of the maximum values of the seudovelocity spectra, Sv(10%), for a lapse time period of 50 years, for Mexico City Lake zone.

The scaling of the accelerograms was performed as follows: sets of Sv(10%) values were randomly simulated from its distribution (Figure 2), and for each simulated Sv(10%) one of the 50 accelerograms was randomly chosen and scaled so as to produce the corresponding value of the mentioned Sv(10%). Tests were carried out in order to compare the response spectra of the simulated accelerograms with the response spectra of the recorded ones, the comparison was satisfactory (Chavez and Gonzalez, 1992).

3 SEISMIC SAFETY OF SIMPLE R/C FRAMES WITH AND WITHOUT MASONRY INFILL

3.1 Structural systems considered

In this work we are interested in estimating the safety of the structural systems shown in Figures 3 and 4. They are simple ductile R/C frames without (Figure 3) and with a masonry infill (Figure 4). In both systems it is assumed that the beam is infinitely rigid, and that the behaviour of the columns , or the columns plus the masonry, under flexural cyclic loading is of the elastoplastic hysteretic type, Figure 5. This behaviour is characterized by the initial lateral stiffness, K, and the yield force, Vy. The collapse mechanisms of the systems under horizontal ground motion, $\ddot{u}_g$, are also shown in Figures 3 and 4. The failure modes are assumed to be unique and due to the simultaneous formation of plastic hinges at the ends of the columns. In recent works these type of collapse mechanisms were found for the systems under consideration (Stylianidis, 1985; Liauw and Kwan, 1985) .

The expressions to compute K and Vy for the two systems are the following:

$$K = 24\ E\ I/\ L^3 \qquad (3)$$

$$Vy = 4\ My/\ L \qquad (4)$$

for the frame without infill, and

$$1/K = B^3/\ (3\ E\ I_c) + B\ /\ (G\ A) \qquad (5)$$

$$Vy = 2\ (My\ f'\ t)^{1/2} \qquad (6)$$

for the frame with infill. Expresions 5 and 6 were suggested in the Mexico City Construction Code (1976, 1987), and by Liauw and Kwan (1985), respectively. In equations 4 and 6, My

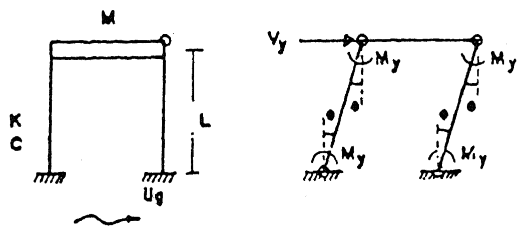

Figure 3. Single frame system and its colapse mechanism.

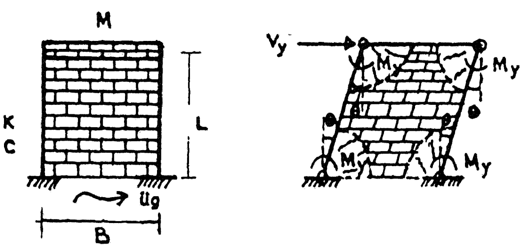

Figure 4. Single infilled frame system and its colapse mechanism.

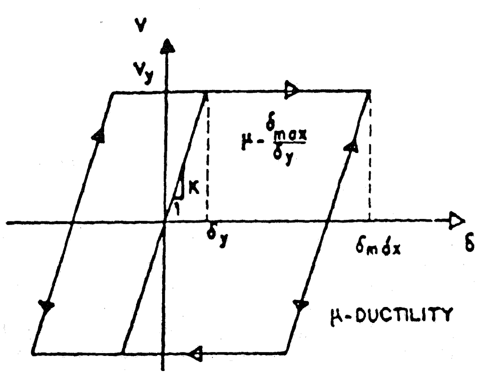

Figure 5. Idealized elastoplastic hysteretic behaviour of the single frame and single infilled frame system.

represents the yielding moment of the columns, this can be computed as follows (Mexico City Construction Code, 1976, 1987):

$$My = \phi\ As\ fy\ d\ (\ 1 - As\ fy/\ 2\ b\ d\ fc\ ) \qquad (7)$$

The parameters of equations 3 to 6 are: E is the elasticity modulus of the concrete, I is the inertia moment of the cross section of the columns, L the length of the columns, B is the span of the beam, I_c is the inertia moment of the cross section of the columns with respect to B/2, G is the shear modulus of the infill, A is the shear area of the cross section of the columns plus the shear area of the infill, f' is the crushing stress of the infill, t is the thickness of the infill. Finally the parameters involved in equation 7 are the following: ϕ is a formulae error parameter, As is the cross section of the reiforcing steel, fy is the yielding stress of the reinforcing steel, d is the effective depth of the column cross section, b is the width of the column cross section, and fc is the concrete strenght.

3.2 Seismic safety of simple frame systems

If R and S represent the random resistance and the random action on the frames of interest (Figures 3 and 4) respectively, and if they are considered to be statistically independent, the probability of failure, **Pf**, of the structural system can be computed as follows:

$$Pf = P(S>R) = \int_0^\infty \bar{F}s(\alpha)\ f_R(\alpha)\ d\alpha \qquad (8)$$

in expression (8) P(S>R) means the probability that S will be larger than R at a certain time during the lifetime of the frame. $\bar{F}s$ is the complementary accumulated probability distribution of S, and f_R is the probability density function of R.

In this work the resistance, R, of the frames is associated to the variable δ, where $\delta = \mu-1$, and μ is a random variable representing the ductility available in the frame. Here μ is defined as the ratio of the maximum deformation to the yield deformation. The variable δ is assumed to possess a lognormal distribution, as suggested by Meli (1976). In Fig 6 we present the probability density functions of δ ,that in what follows it will be called f_R, for several values of its coefficient of variation, $V\mu$, and a characteristic value of μ of 4. The latter value is suggested in Mexico City Construction Code (1976, 1987) for the structural system of Figure 3.

The random variable S in this work represents the ductility demand, and to calculate $\bar{F}s$, it is required to have samples of the seismic responses of the frames (Figures 3 and 4), under the expected ground motions at the Lake zone of Mexico City. As some of the parameters utilized to express K and Vy, in equations 3 to 6 are

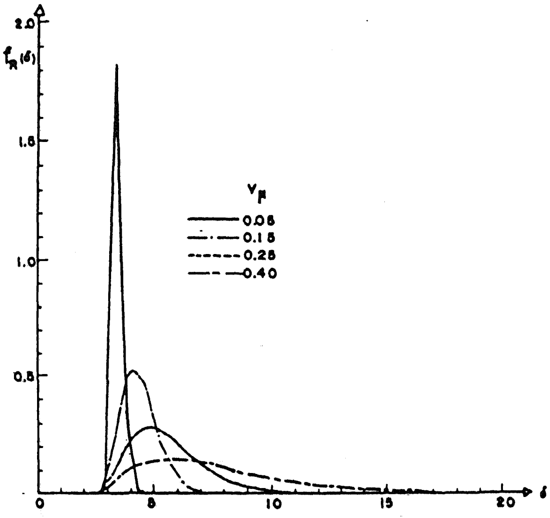

Figure 6. Probability density function, fR, of the nominal ductility, μ = 4, assumed for the single frame and the single infilled frame; the fR, is shown for several coefficients of variation, $V\mu$, and $\delta = \mu-1$.

random, second moment probability descriptions of K and Vy can be obtained in terms of the second moments of their parameters, and if it is assumed that K and Vy are statistically independent and lognormally distributed, by Monte Carlo simulation it is posible to simulate pairs of Ki and Vyi values, with i = 1..m, which represent m structural systems of the type shown in Figures 3 and 4.

When the samples of structural systems and of expected accelerograms (generated as suggested in Chapter 2) have been simulated, they are randomly paired, and by step by step dynamic analysis, the maximum responses of the structural systems can be computed, in this case the ductility demands, therefore $\bar{F}s$ can be calculated. Once $\bar{F}s$ and fR have been computed, equation 8 can be applied and the **Pf** of the systems under consideration estimated.

4 SEISMIC SAFETY OF SIMPLE R/C FRAMES WITH AND WITHOUT INFILL DESIGNED WITH THE 1976 AND 1987 MEXICO CITY CONSTRUCTION CODES

Here the methodology described above is applied in order to evaluate the **Pf** of the structural systems shown in Figures 3 and 4, both systems are designed according with the 1976 and 1987 Mexico City Construction Codes. The systems analyzed have an initial natural period of 0.5, 1.5 and 2.5s, and a percentage of critical damping of 0.05 (as recommended in those codes for the R/C constructions). The values of the parameters utilized to simulate the families of the systems of interest, are presented elsewhere, Chavez and Gonzalez, (1992). The sample sizes used were 40 and 80.

The best fitting of the ductility demands, S, is obtained with an extreme type distribution (Chavez and Gonzalez, 1992), as it was found in the previous works (Chavez and de Leon, 1984, and Chavez, 1988), and for other researchers Bolotin (1988).

Once the $\bar{F}s$ distributions are computed for the systems of interest, expression 8 is applied and results of the type shown in Figures 7 and 8 are obtained. In Figure 7 we plotted the **Pf** of the simple R/C systems with initial natural periods of 0.5, 1.5 and 2.5s; a nominal ductility factor of 4 (as recommended in the code) is assumed and coefficients of variation $V\mu$ equal to 0.05, 0.25 and 1.00 were used to represent fR (Figure 6). In Figure 7 it can be observed that for the bare frame systems with an initial period of 0.5 (stiff frames) the **Pf** is of the order of 10^{-8}, however, for the flexible ones (initial periods of 1.5 and 2.5s) their **Pf** are of the order of $10^{-3}$and 10^{-2}, respectively. Also, in Figure 7 the same type of results are presented for the infilled frame systems. Here the **Pf** for the stiff frame are of the order of 10^{-4}, and for flexible ones the **Pf** values are of the order of 10^{-2} and 10^{-1}, respectively.

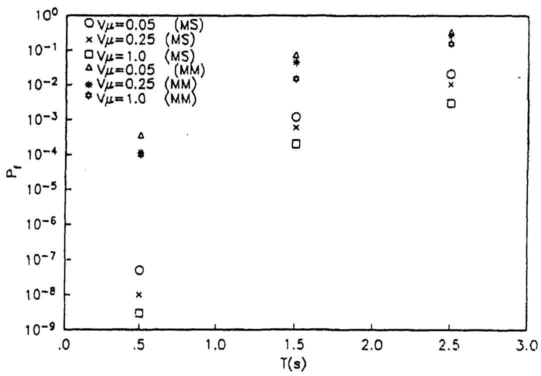

Figure 7. Probability of failure , Pf, for single frame (MS) and single infilled frame, (MM), systems designed according with Mexico's City 1987 Construction Code, with initial natural periods of 0.5, 1.5, and 2.5s. A nominal ductility factor μ = 4 is assumed, for coefficients of variation $V\mu$ = 0.05, 0.25, and 1. The systems are assumed to be located on Mexico's City Lake zone.

Finally, in Figure 8 we present the **Pf** values versus $V\mu$, for both type of frames, with initial natural periods of 2.5s. Results are shown for different size of samples (40 and 80). The larger $V\mu$ values aim to represent systems with the possibility of developing very large nominal ductility. From this Figure it can be observed that the **Pf** for the bare frames, designed according with the 1976 Code are about three times larger than their corresponding frames, but in this case designed with the 1987 version of the Code. Also notice that their **Pf** varies from 10^{-3} to 10^{-2}, these values are very similar to the ones reported by Chavez and de Leon

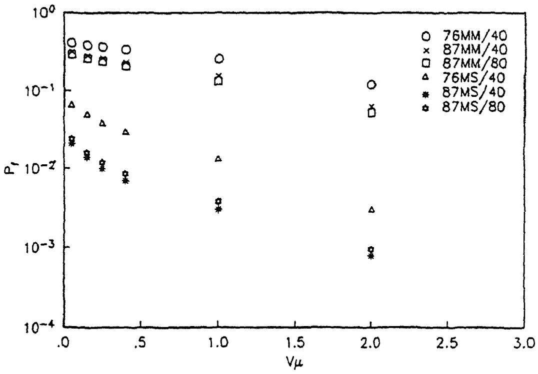

Figure 8 . Variation of the probability of failure, Pf, with Vμ, for single frame, (MS), and single infilled frame, (MM), structural systems, designed according with Mexico's City 1976, (76), and 1987, (87), Construction Codes. An initial natural period of 2.5s is assumed for both systems. Families of 40 and 80 structural systems were used in the Monte Carlo simulation. See Figure 7 for complementary information.

(1984). The results for a sample size of 40 are almost the same that with a size of 80. The smaller **Pf** correspond to systems with large nominal ductility .

In the same Figure 8 we show the **Pf** obtained for the infilled frame systems. The general trend of these results is similar to the ones described for the bare frames. However, notice that they are about one order larger than the ones for the bare frames. This contradicts the observations of the 1985 Mexico earthquake. We think this is due to the very crude representation used to incorporate the contribution of the infill to the seismic behaviour of this type of structural systems.

5 CONCLUSIONS

The main conclusions of the study are the following:

1) It is strongly recomended to give an important weight to the observational data, when estimating the seismic hazard at sites like in Mexico City Lake zone, in which the local soil conditions play a definite role in the characteristics of the ground motions expected in the future.

2) The probability distributions of the ductility demands for the considered structural systems are of the extreme type.

3) The probabilities of failure of bare R/C frames designed accordingly with the 1976 Mexico City Code are about three times larger than the ones designed with the 1987 version of the same code. However both are of the order of 10^{-3} for flexible systems, and of 10^{-8} for rigid frames.

4) The probability of failure for simple R/C infilled frame systems is of the order of 10^{-1}, in contrast with observations of the 1985 Mexico earthquake, this is probably due to a deficient idealization of these systems.

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