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Seismic vulnerability evaluation of a 32-story reinforced concrete building

A.M. Memari†

Department of Civil and Environmental Engineering, The Pennsylvania State University, 212 Sackett Building, University Park, PA 16802, U.S.A.

A.R. Yazdani Motlagh‡ and M. Akhtari‡

International Institute for Earthquake Engineering and Seismology, Tehran, Iran

A. Scanlon‡†

The Pennsylvania State University, University Park, PA 16802, U.S.A.

M. Ghafory Ashtiany‡‡

International Institute for Earthquake Engineering and Seismology, Tehran, Iran

Abstract. Seismic evaluation of a 32-story reinforced concrete framed tube building is performed by checking damageability, safety, and toughness limit states. The evaluation is based on Standard 2800 (Iranian seismic code) which recommends equivalent lateral static force, modal superposition, or time history dynamic analysis methods to be applied. A three dimensional linearly elastic model checked by ambient vibration test results is used for the evaluation. Accelerograms of three earthquakes as well as linearly elastic design response spectra are used for dynamic analysis. Damageability is checked by considering story drift ratios. Safety is evaluated by comparing demands and capacities at the story and element force levels. Finally, toughness is studied in terms of curvature ductility of members. The paper explains the methodology selected and various aspects in detail.

Key words: seismic vulnerability evaluation; framed tube building; response spectra; time history; ambient vibration; ductility; drift ratio; 3-D mathematical modeling.

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1. Introduction

After the devastating Manjil-Rudbar earthquake of June 1990 in northern Iran measuring 7.3 on the Richter scale (IIEES 1991), seismic vulnerability of even modern existing office buildings became a concern. The structure subject of this study is a 20 year old 32-story framed tube

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† Visiting Assistant Professor
‡ Former Research Assistant, P.O. Box 14475-155, Tehran, Iran

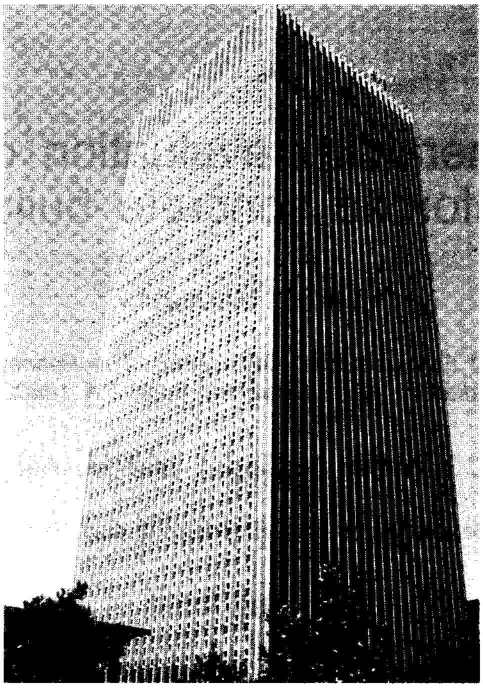

Fig. 1 A photograph of Sepehr Tower

reinforced concrete office building (Sepehr Tower, Fig. 1), the headquarters to Bank Saderat in downtown Tehran. Lack of sufficient evidence (original design calculations), that would indicate adequate seismic provisions have been considered, prompted the building owners to seek a seismic assessment of the structure solely based on the available design drawings. This paper presents the results of the seismic vulnerability evaluation of the structure primarily considering the Iranian seismic code, Standard 2800 (BHRC 1988), requirements, which are actually meant to apply to new designs. These requirements, that follow closely the provisions of ATC 3-06 (ATC 1978), were used since there were no other Iranian seismic provisions specially applicable to existing buildings.

Framed tube buildings (Khan and Amin 1973) show a complex type of behavior as their lateral force resisting mechanism is composed of frame action and cantilever tube action. There is a scarcity of reported seismic vulnerability study of this type of construction. Seismic vulnerability methodologies in general vary greatly and each study is unique in its own right. From various vulnerability investigations, several components can be identified: local material identification, global system identification, laboratory testing of elements to obtain hysteretic response, linear elastic analysis, nonlinear static push-over analysis, nonlinear dynamic analysis, soil-structure interaction, and cumulative damage analysis. Needless to say, it is neither feasible, nor justified to apply all these steps in every building evaluation. The selected approach should, however, address the critical issues. In practical seismic analysis, lack of seismicity data and sometimes unavailability of nonlinear analysis capability pose constraints on the evaluation approach. The approach used here, while taking into account these limitations, involves checking damageability, safety, and toughness limit states through ambient vibration testing, three dimensional linear elastic modeling, response spectrum analysis, time history dynamic analysis, and capacity analysis.

2. Seismic evaluation methodology

The overall objectives of the seismic evaluation of this building were to check damageability, safety, and toughness limit states. Specifically, this required evaluation of story drift ratios, story and member demand capacity ratios, and structure and member ductility factors. The analysis methods and seismic input had to satisfy Standard 2800 requirements. Moreover, the phase of the study reported here was limited to the use of three dimensional linear elastic mathematical modeling for computer analysis to illustrate the evaluation approach without the use of nonlinear analysis.

According to Standard 2800, seismic demand forces can be obtained using equivalent static lateral force, modal superposition, or time history methods of analysis. The Standard is very specific about the seismic input; for the first two approaches, it provides a design response spectrum, and for the third approach, it specifies two severe earthquakes to be used as design earthquakes. These are the accelerograms of Tabas N 16 W September 16, 1978 (PGA=0.93 g) and Naghan Longitudinal April 6, 1977 (PGA=0.72 g). These two earthquakes are meant to be considered as maximum credible earthquakes regardless of the location of the site.

To supplement the code specified design earthquakes with an earthquake in moderate PGA range, the El Centro N-S May 18, 1940 earthquake (PGA=0.32 g) was chosen here as the probable earthquake and also as a basis for comparison to this work. It is to be noted that 2/3 times the intensity (Housner 1959) of this record has been used as the intensity of the probable design earthquake in the past (Fintel and Ghosh 1991). Moreover, the earthquake has also served as a

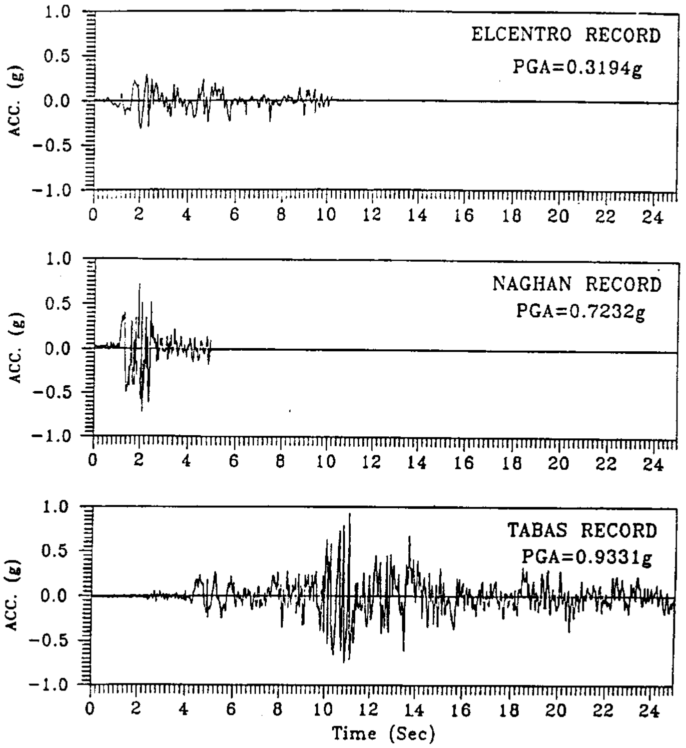

Fig. 2 Accelerograms for time-history analysis

reference record for response spectrum and time history analyses of buildings in Japan (Yagi *et al.* 1990, Adachi and Nagata 1990) and in Armenia (Anderson and Agbabian 1994) as examples of use in other countries. Fig. 2 shows accelerograms of these three earthquakes. To further supplement the time history dynamic analysis, three levels of peak ground accelerations, PGA=0.3 g, 0.4 g, and 0.5 g were chosen to define alternate design earthquakes. The selected PGAs are used here to develop linear elastic design response spectra based on Newmark's method (Newmark and Hall 1973).

Damageability limit state is checked by comparing story drift ratios with established criteria and earthquake damage observations. Saftety limit state is evaluated here by studying demand to capacity ratios of overall base (first story) shear and representative member end forces. Furthermore, beam and column capacities are compared to evaluate possible failure modes. Toughness limit state is usually defined in terms of ductility demand at the member level, which is usually expressed in the form of rotational or curvature ductilities. The global displacement ductility and story ductility can be used to assess the overall damageability conditions. In order to evaluate the performance at the member level, member curvature ductility capacities can be studied.

Linear elastic design response spectra (LEDRS) for peak ground accelerations of 0.3 g, 0.4 g, and 0.5 g are determined by scaling the "standard earthquake" (Newmark and Hall 1973) defined by maximum ground acceleration=1.0 g, maximum ground velocity=48 in./sec, and maximum ground displacement=36 in. as the ground motion at the site. With a damping ratio of 5%, the

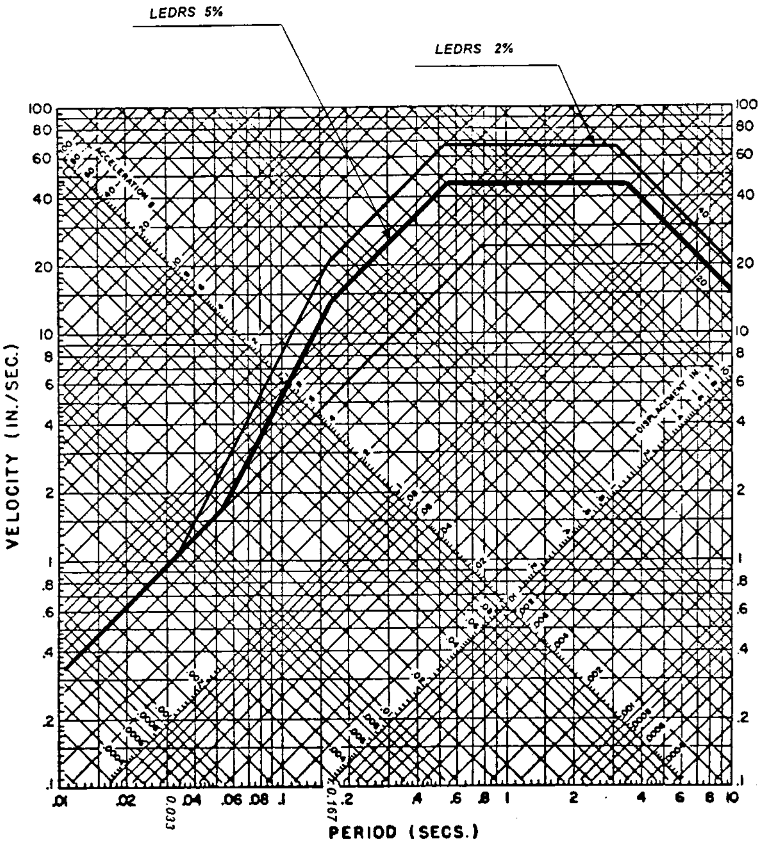

Fig. 3 An example of constructed spectrum

amplification factors for displacement, velocity, and acceleration are, respectively, 1.4, 1.9, and 2.6 (Newmark and Hall 1973). An example of the LEDRS obtained based on Newmark's method is plotted in Fig. 3.

According to Standard 2800, seismic induced forces in the two orthogonal directions should be considered independently but in two opposite senses. It does not prescribe any combination of the forces in the two orthogonal directions for "regular" buildings. As for combination with other types of applicable loads, it recommends the load combination specified in the structural material building design code to be followed. In this study, design is evaluated based on ACI code (1989), which prescribes the following two load combinations: 0.75(1.4D+1.7L+1.87E) and (0.9D+1.43E). In this study, the first scheme is used, as in most regular designs it governs. The ACI code does not differentiate between methods of analysis in specifying the load combinations. Some provisions do not prescribe any load factors in the combination (FEMA 1994). They prescribe, however, consideration of simultaneous orthogonal seismic input by using a combination such as 100% of the response in one direction and 30% of the response in the other direction. In order to follow the Standard, such orthogonal effect combination is not considered here; however, load factors specified by ACI are applied. The net effect is more conservative than the alternative approach.

3. Building description

Sepehr Tower is a 32-story reinforced concrete building with a height of 100.5 m above grade

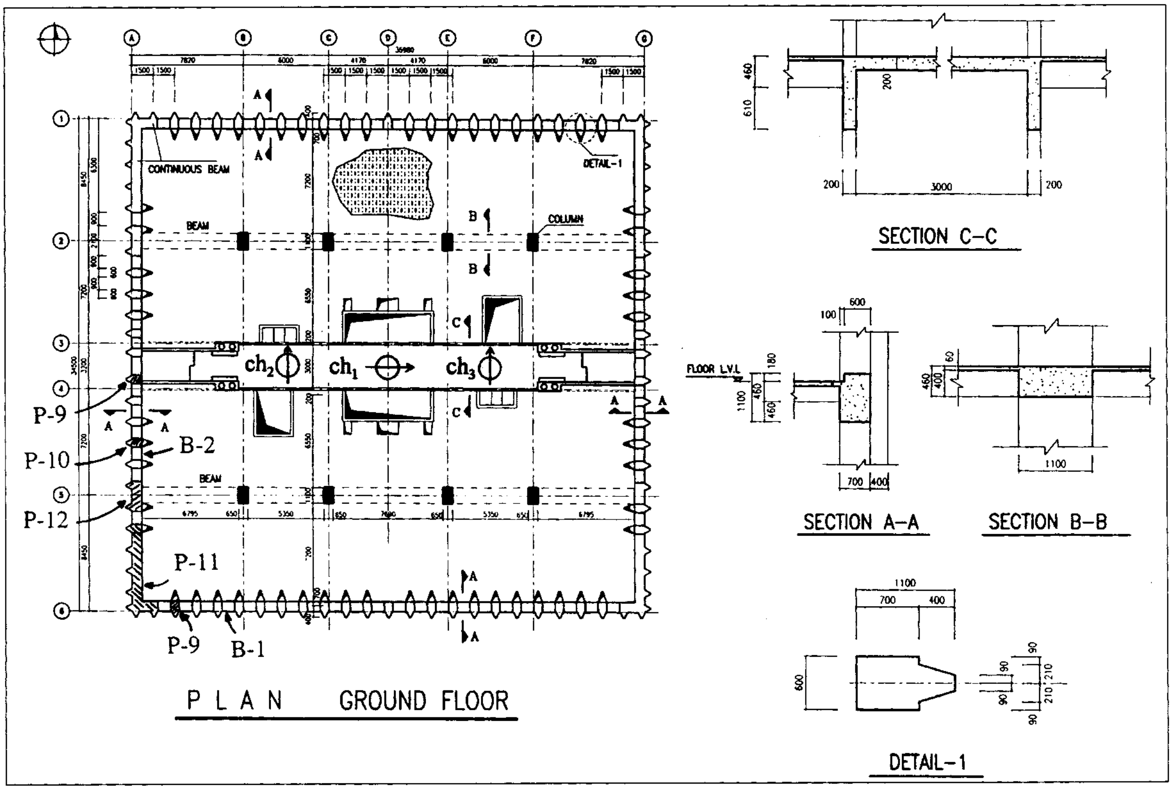

Fig. 4 Building plan and some details

and plan dimensions of 34.5 m in the N-S and 36.0 m in the E-W directions. The first story above grade is the entrance and lobby. The lowest three stories are below grade and serve as parking levels. Therefore, story 29 is considered the most top story here. The structural system is identified as framed tube with closely spaced perimeter columns framed by relatively deep spandrel beams (Khan and Amin 1973). The building then in effect is a perforated tube stiffened by rigid diaphragms. This presumably makes the structure very efficient in resisting lateral forces since it combines tube action (cantilever wall action) and normal frame action. This notion, however, may not be strictly valid unless the strong column-weak beam design philosophy is satisfied as well.

The floor system shown in Fig. 4 consists of a 6 cm reinforced concrete slab supported by reinforced concrete 11 cm wide by 40 cm deep joists at 50 cm on centers. The slab has increased thickness of 20 cm between column lines 3 and 4 along the corridor adjacent to elevator and stair shafts. The floor system for the top two stories has added 7 cm thick lower slab under the joists, forming a sandwich type system. The joists are supported by two E-W direction shallow beams (40 cm deep, 110 cm wide) along column lines 2 and 5, interior constant 20 cm thick walls along column lines 3 and 4, and exterior framing system.

The framing system consists of perimeter columns spaced at 1.5 m on centers and beams with a depth of 1.1 m. The column cross section (Fig. 4) consists of a rectangular main and a trapezoidal nose portion. The main rectangular portion has a constant width of 60 cm and a variable depth of

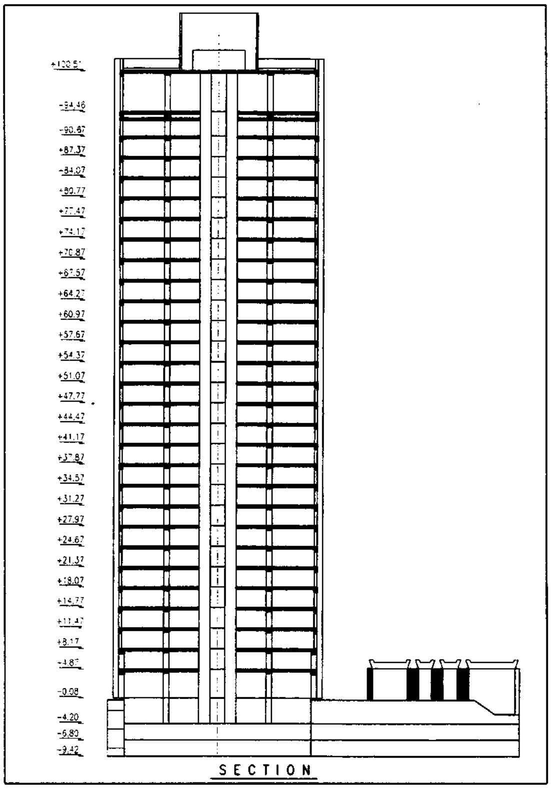

Fig. 5 Building elevation

70, 60, 50, 40, and 30 cm, respectively, in the 1st to 3rd, 4th to 9th, 10th to 12th, 13th to 19th, and 20th to 29th floor. The area of 2.6 cm or 3.2 cm diameter reinforcement in the two primary perimeter column types varies from 38 cm^2 (steel ratio=1.27%) to 222 cm^2 (steel ratio=4.11%) at different stories with four to six legs of 1 cm or 1.2 cm diameter ties at spacings varying from 12 cm to 30 cm. Perimeter beams mostly have equal amount of tension and compression 2.3 cm, 2.6 cm, or 3.2 cm diameter reinforcement with areas varying from 32 cm^2 (steel ratio=0.5%) to 112 cm^2 (steel ratio=1.55%) with 1.2 cm diameter stirrups at 15 cm spacings. The interior frames consist of two rows with columns connected by E-W beams along column lines 2 and 5. This interior system is designed to carry gravity loads. The only exterior above grade walls are four 6.3 m shear walls along column lines A and G in the N-S direction (two between column lines 1-2 and two between column lines 5-6).

The elevator and stair shafts are made of 20 cm thick reinforced concrete walls of constant thickness throughout the height of the building. These thin walls have reinforcement and floor connection details such that they can be considered primarily as gravity load carrying systems and do not qualify as shear walls forming dual system with the tube. The reinforced concrete basement walls on the perimeter of the building in the three below grade stories are 70 cm thick and the foundation is a solid 1.7 m thick mat. The building has varying story heights; namely, 4.9 m for the 1st story, 3.3 m for the 2nd through the 27th, 3.8 m for the 28th, and 6.0 m for the 29th. An elevation view of a typical N-S section of the building is shown in Fig. 5.

4. Ambient vibration test

Results of ambient vibration testing (Crawford and Ward 1964, Ward and Crawford 1966) of the building are discussed in this section. The general objective of the test was to determine the dynamic properties of the building to fine tune the analytical modeling. The properties of interest from the test are frequencies, mode shapes, and damping ratios for the first few modes. The instruments used consist of six Kinemetrics SS-1 ranger seismometers, two Kinemetrics SSR-1 portable event recorders, and a laptop computer. The seismometer measures the velocity which is proportional to an output constant voltage. Due to the limited number of instruments,

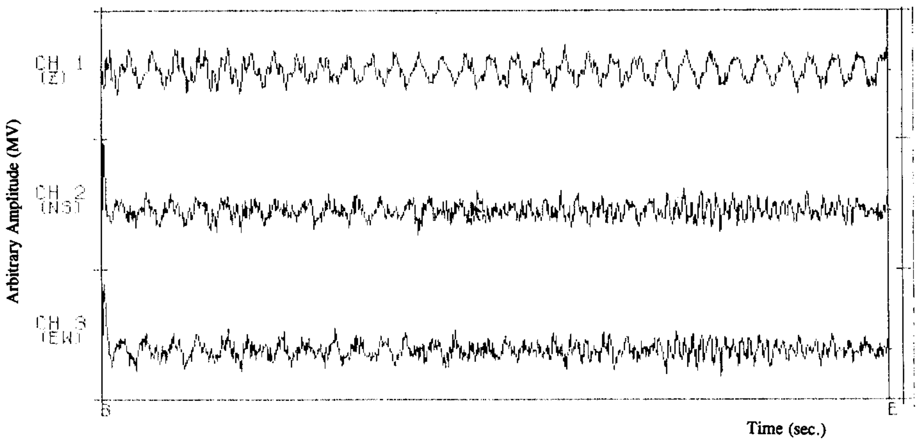

Fig. 6 An example of recorded signal in channels 1, 2, and 3

measurements were taken simultaneously at only two floors at a time. The properties in the two principal lateral directions and the torsional response were of interest. The procedure used and described below in carrying out the test essentially follows that presented by Trifunac (1970).

The 28th floor was used as the reference floor. One of the SSR-1 recorders and three SS-1 seismometers were placed on this floor, and a similar set was successively moved to floors 26, 24, 22, 20, 17, 14, 11, 8, 5, 3, and 1 for each round of measurements. Fig. 4 shows the arrangement of the instruments (channel 1: E-W, channels 2 & 3: N-S) on a typical floor. The three seismometers on each floor were placed on the east-west centerline of the floor and connected to a common recorder. Each of the two N-S direction instruments was located 7 m from the center E-W direction instrument. Their parallel orientation was necessary to determine the torsional mode.

A typical example of the recorded data for about 60 sec. simultaneously for channels 1, 2, and 3 is shown in Fig. 6. High frequency vibrations (noise) were later filtered. To differentiate between

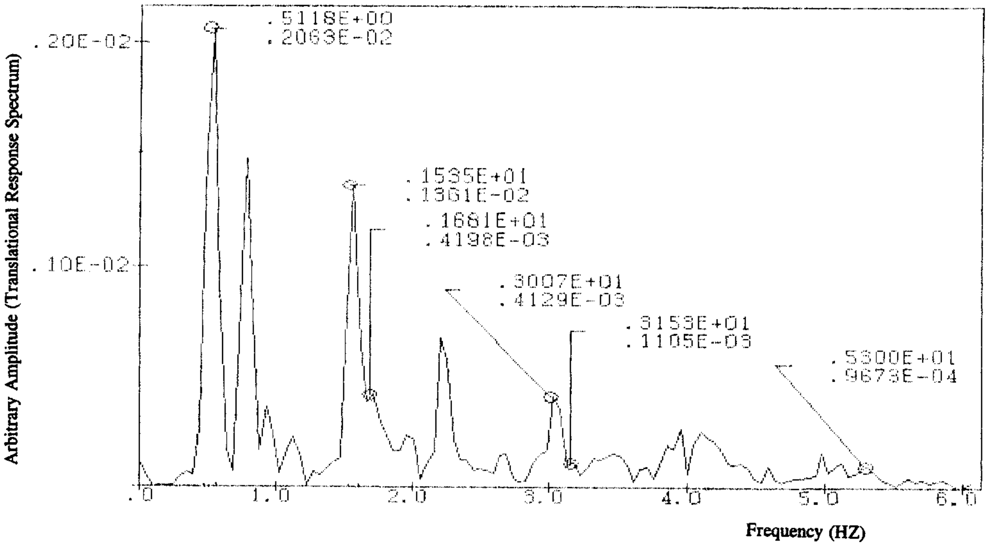

Fig. 7 An example of the fourier spectrum for the reocrd at 26th floor

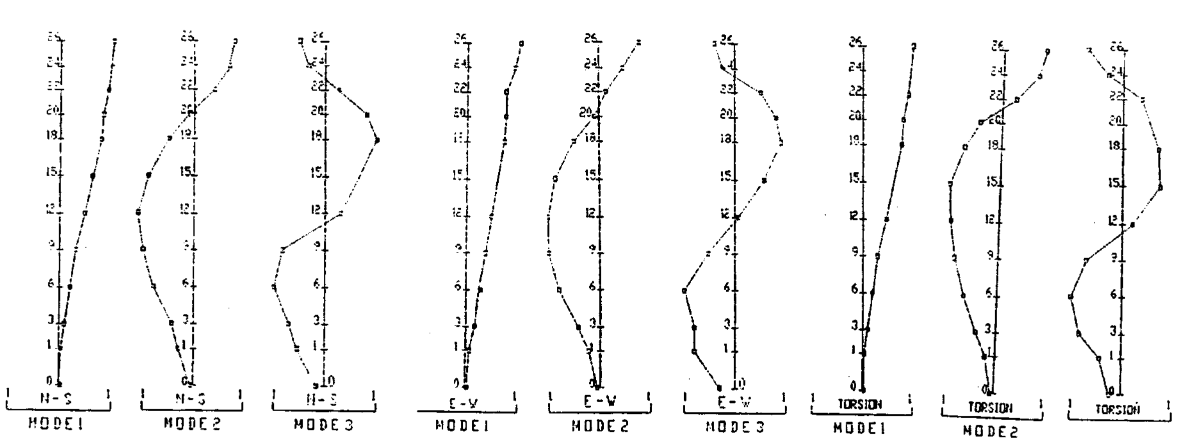

Fig. 8 Mode shapes resulting from ambient vibration test

Table 1 Frequencies (Hz) and dampling ratios resulting from ambient vibration test

Mode No.	N-S Frequency	E-W Frequency	Torsion Frequency	N-S Damping	E-W Damping	Torsion Damping
1	0.49	0.49	0.78	0.060	0.077	0.091
2	1.65	1.66	2.14	0.021	0.025	0.044
3	3.07	3.17	3.95	0.010	0.014	0.030

Table 2 Typical measured natural vibration frequencies in midrise to tall RC buildings

No.	No. of stories	F1. Area (m^2)	N-S f_1 (Hz)	N-S f_2 (Hz)	E-W f_1 (Hz)	E-W f_2 (Hz)	Reference
1	21	800	0.67	2.38	0.85	2.86	Jeary & Ellis 1981
2	27	870	0.56	2.04	0.66	2.33	Hosahalli *et al.* 1992
3	30	580	0.58	1.69	0.58	1.69	Stephen *et al.* 1995
4	32	550	0.55	2.33	0.65	0.34	Schuster *et al.* 1994
5	46	NA	0.44	1.69	0.44	1.67	Jeary & Ellis *et al.* 1981

translational and torsional modes of response, parallel channels can be once added and once subtracted. If the response amplitude increases when the two parallel records are added or decreases when the two records are subtracted, the resulting frequencies correspond to translation. If, however, by adding the two records the response amplitude decreases or by subtracting the records the amplitude increases, the frequencies correspond to torsional response. Fourier amplitude spectra were obtained for the record of channels 1 and 2 and the subtracted record of channels 2 and 3 for each pair of floors where simultaneous measurements were taken. Fig. 7 shows typical Fourier amplitude spectra.

The characteristic frequencies for each direction were determined by averaging the respective frequencies corresponding to peak amplitudes of the Fourier spectra for all the tests at the reference floor. The first three mode characteristic frequencies are summarized in Table 1. Mode shapes for each direction were obtained by dividing the peak amplitude corresponding to each characteristic frequency of the spectrum at any floor by the respective spectral values at the reference floor. Fig. 8 shows the normalized mode shapes for the first three modes. Finally, average modal damping ratios were obtained using the half power method. These values are also shown in Table 1. Although the natural frequencies for a steel building with the same height would be smaller than those in Table 1 (e.g., Trifunac 1970, Taoka 1981), tall reinforced concrete buildings (20 to 40+ stories), due to inherent larger stiffness, generally have fundamental frequencies larger than steel counterpart buildings. Examples of measured frequencies for the first two modes in the principal directions for a few reinforced concrete buildings with various plan configurations are given in Table 2.

5. Analytical modeling and vibration analysis

A discretized finite element model of the building based on the modeling options of the ETABS computer program (Habibullah 1986) is used to perform the analytical studies. The building is modeled with 29 floor levels above grade, considering the three parking levels below

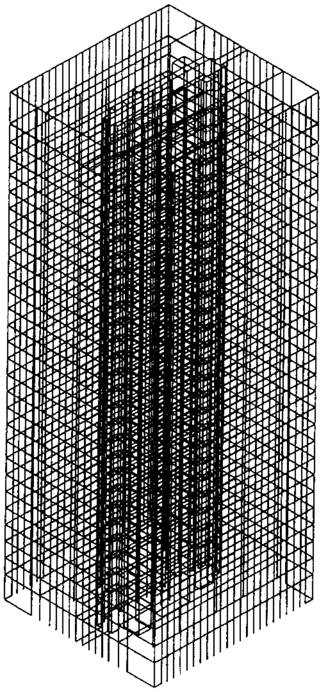

Fig. 9 ETABS three-dimensional mathematical model

grade secured by R/C perimeter walls as part of the rigid foundation system. The structure is thus assumed to be fixed at the ground level. The structural members in the building consist of walls, including L-shaped corner walls and the walls making up the elevator shafts and stair cases, columns, including perimeter tube and interior columns and some interior wall piers, and beams, including perimeter tube and the main E-W floor beams and coupling members at openings of interior walls. Column, beam, and panel elements are used to model respectively, column, beam, and wall members. The perimeter tube, consisting of the four exterior frames interconnected by rigid floor diaphragms, is thus assembled from beams, columns and corner L-shaped walls (panel elements). All beam to column and beam to wall connections are assumed to be rigid. However, the finite joint dimension option, which assigns 50% of the dimension of an element in a joint as rigid zone, is also invoked. The resulting model is shown in Fig. 9.

The model mass consists of contributions from distributed floor loads (including 20% of the applicable live load) and various member weights. Each floor is subdivided into convenient areas for purposes of mass calculation, and the program locates the mass centroids and applies the total floor mass at that point. Each floor level is defined at the centerline elevation of each floor (consisting of slab and beams), where the program assumes a rigid diaphragm. This feature of the program internally links all nodes on a floor such that any relative displacement between nodes is eliminated, thus creating a floor that is rigid in its own plane. This way, the prescribed flexural stiffnesses of the beams will remain effective. The program then considers the effect of eccentricity by calculating locations of the centers of mass and rigidity.

After initial trial of uncracked section properties for all elements, section properties of beam and column elements were chosen, respectively, as 50% and 100% of the gross so that the dynamic properties of the model would be acceptable compared with the ambient vibration test results.

Corner columns in ETABS are treated as three dimensional elements with axial deformation modeling, which is appropriately suited for the framed tube building under study. Moreover, equivalent effective shear areas were also defined for all elements. As for material properties, concrete strength of 270 kg/cm^2 and modulus of elasticity of 248000 kg/cm^2 were used for the model.

Using the described model, a vibration analysis was performed to determine natural periods of vibration. Based on the final assumption of section properties, the following periods (in seconds) were obtained for the first three modes: N-S: 1.92, 0.55, and 0.27; E-W: 1.63, 0.51, and 0.26; and Torsional: 1.14, 0.37, and 0.19. Comparison of these results with those obtained from the ambient vibration test indicates differences ranging from 6% for the first N-S translational mode to 24% for the third torsional mode, with all the analytical values being smaller than the corresponding test results. The general indication is that the finite element model is somewhat stiffer than the actual building for low levels of vibration, more so in the E-W direction due to the elevator shaft and stair case walls. This is also to be expected since the model was asumed to be fixed at the ground level, whereas in reality there is some flexibility there. If the effective height in the model is taken to include the three below grade stories, the differences would be much smaller. Moreover, by reducing column and wall section stiffnesses to slightly less than 100% of the gross, we will still increase the analytical periods. However, in light of the fact that the objective was seismic evaluation with many uncertainties with respect to seismic input and safety factors, the slightly conservative model was kept for the rest of the study.

6. Capacity evaluation

Due to the variety of members and details, it was necessary to develop a computer program to calculate axial force, bending moment, and shear force capacities of beams, columns, and walls and also the points for moment-curvature diagram. The force capacities are calculated according to ACI (ACI 1989). For columns and walls, in addition to pure bending and axial load capacities, the program determines the necessary values for interaction diagrams. For any desired level of axial force, it will then give the moment capacity. Shear capacity equations appropriate for beam, column, and wall members according to ACI are used in the program.

The relation between section moment M and curvature φ at a given axial load level is

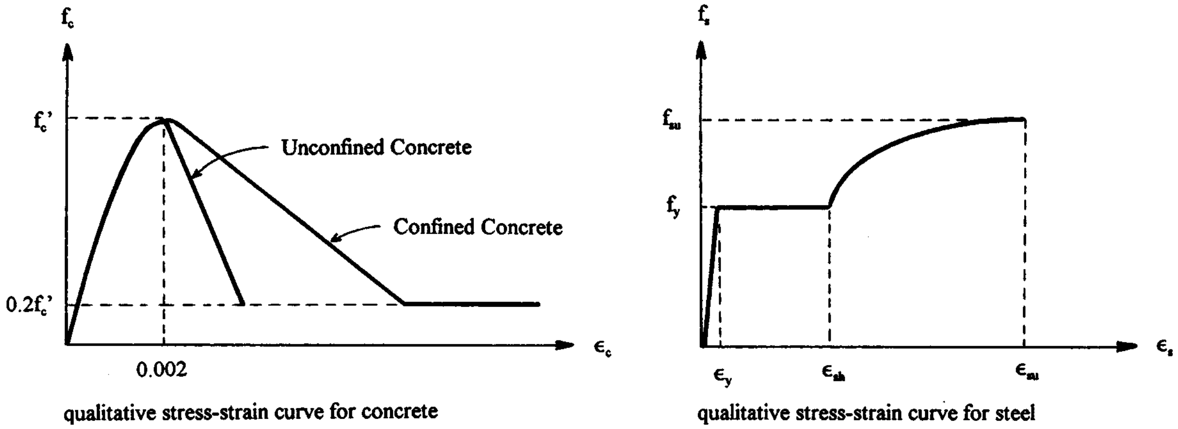

Fig. 10 Sress-strain models for concrete and steel.

established by finding discrete values of moment and curvature corresponding to different values of concrete extreme compression fiber strain ε_{cm}. The procedure, which involves considering the section to be made up of several laminae parallel to the neutral axis and distributing the steel areas in equivalent strips, is well suited to computer programming (Park and Paulay 1975). In summary, the neutral axis depth, kd, is first determined so that it satisfies the section axial force equilibrium. Stress-strain relation (Fig. 10) for concrete is based on Kent and Park model (Kent and Park 1971) which considers the effect of confinement, and the relation for steel is based on Burns and Siess model (Burns and Siess 1962) as suggested by Park and Paulay (Park and Paulay 1975). For each value of extreme fiber strain, ε_{cm}, the average strain in each lamina is determined. The strain in the lamina is then used to obtain steel and concrete stresses from the appropriate stress-strain models. Moment capacity is then determined by considering moment equilibrium at the section, and the corresponding curvature is given by the ratio ε_{cm}/kd. By increasing the extreme fiber strain, corresponding values for moment capacity and curvature are determined.

As axial load influences the curvature and there is no unique moment-curvature relation, we need to consider the pair of axial load and moment that cause failure. The curvature corresponding to this state with ultimate concrete strains is then the ultimate curvature capacity ε_u. Similarly, we can obtain yield curvature φ_y corresponding to the state of first yield in steel. However, when the axial load is greater than the balanced axial load, φ_y is taken equal to the curvature corresponding to the strain at the crushing of concrete cover, assumed to occur at 50% drop in unconfined concrete strength. The ratio φ_u/φ_y then provides the curvature ductility capacity corresponding to the axial load level of interest.

7. Damageability and safety evaluation

In this section, damageability and safety criteria as applied to a story and to typical elements

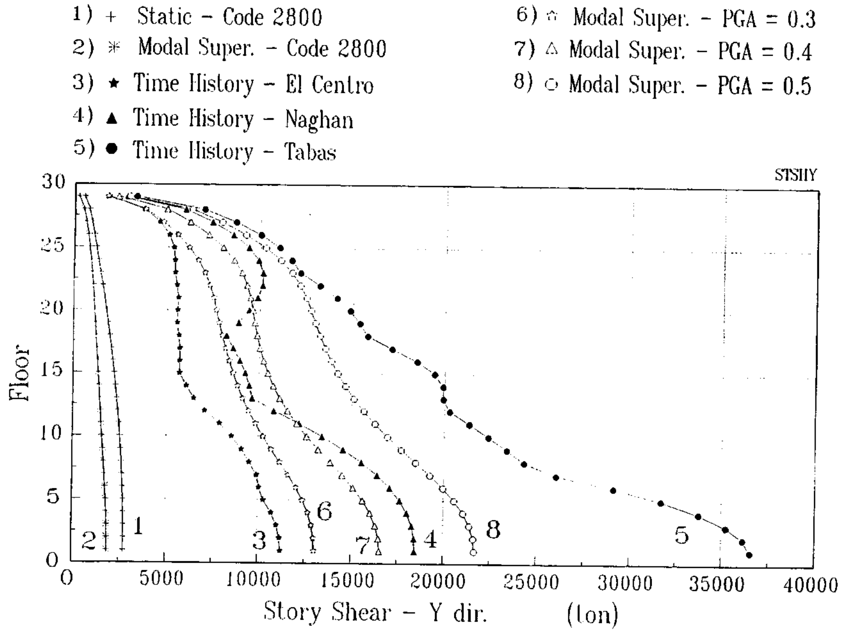

Fig. 11 Story shear distributions for the N-S direction analysis

Table 3 Base shear demand, shear demand/capacity ratios and mean drift ratios (MDR)

Analysis type	Demand V_{ux} (ton)	Demand V_{uy} (ton)	$(V_u/V_n)_x$	$(V_u/V_n)_y$	MDR N-S	MDR E-W
Equiv. static, standard 2800	2786	2744	0.400	0.333	1/1271	1/1784
Modal super., standard 2800	2099	1684	0.301	0.204	1/2164	1/2676
Modal super., PGA=0.3 g	14513	13038	2.082	1.582	1/350	1/424
Modal super., PGA=0.4 g	18966	16556	2.720	2.009	1/290	1/381
Modal super., PGA=0.5 g	24501	21685	3.514	2.631	1/212	1/245
Time history, E1 Centro	12939	11235	1.856	1.363	1/484	1/535
Time history, Naghan	16943	18462	2.430	2.240	1/308	1/391
Time history, Tabas	35561	36543	5.101	4.434	1/161	1/147

are checked using the response based on equivalent static lateral force, modal superposition, time history, and the linear elastic design response spectra (Fig. 3) approaches. The seismic base shear coefficient according to Standard 2800 is C=ABI/R. For the site and the structure under consideration, the following assumptions are made for the equivalent static lateral force method: the design base acceleration coefficient A=0.35, the importance factor I=1.2, the system behavior coefficient R=8, the spectral response coefficient $B=2.0(T_0/T)^{2/3}$, where the soil coefficient T_0=0.4 and the fundamental period $T=0.09H/D^{1/2}<0.06H^{3/4}$, H and D being, respectively, the height and width in the direction under consideration. It should be noted that although Standard 2800 prescribes an importance factor of 1.0 for regular office buildings in the category of average importance, a value of 1.2 corresponding to the high importance category was chosen because of the special importance of this building to the Bank. The base shear is then obtained as V=CW, where the building weight is W=45,306.6 tonne. The resulting base shear in the E-W and the N-S direction is, respectively, 1975 tonne and 1948 tonne. The base shear in each direction is then to be distributed to floor levels according to $F_i=(V-F_t)\ w_i\ h_i/\Sigma w_j\ h_j$, where F_i=lateral force level applied at level i, w_i=weight of level i, h_i=elevation of level i, and F_t=0.07TV <0.25V is an additional concentrated force applied at the roof level (F_t=0 for T<0.7 sec.). For the modal superposition (MS) approach according to the Standard, peak modal responses are determined and combined using SRSS combination scheme. In this study, the first nine modes (three modes in each lateral direction and three modes in torsional response) were considered. The cumulative effective mass participation in each direction were between 91 and 92% of the total mass. If the resultant base shear is smaller than that of the equivalent static lateral force (ESLF) method, all the responses should be increased by the ratio of the latter to the former base shear.

To check damageability and safety criteria according to the code, story displacements and shears are first studied. Fig. 11 shows plots of story shear distributions according to all the seismic inputs and analysis methods considered for the N-S direction. The mean drift ratios are listed in the last two columns of Table 3. According to Standard 2800, which does not specify any deflection amplification factors, story drift should be less than 0.005 times the story height, which means a maximum story drift ratio of 1/200. Considering the drift ratios in Table 3, it is obvious that the code requirement is satisfied. According to Wood *et al.* (1991), past experience has shown that at mean drift ratios greater than about 1/130, moderate structural damage can occur. Based on analysis of a 41-story reinforced concrete tube building, Yagi *et al.* (1991) have shown that at drift ratios between 1/200 to 1/100 plastic hinges form in beams. The columns of that building, however, were designed with higher moment capacity than the beams. Considering the drift ratios in Table 3, it can be expected that under an earthquake like Tabas, some structural damage will be incurred, while under an earthquake with PGA=0.5 g, some nonstructural damage can be expected.

As a first safety check, demand/capacity ratios are obtained for the base shear of the entire building. Base shear capacity has been conservatively determined assuming only the tube columns and walls parallel to the direction of applied seismic loads are effective, resulting in 6972 tonne and 8241 tonne, respectively, in the E-W and N-S directions. The demand-capacity ratios are listed in Table 3. The ratios for the equivalent static lateral force method are 0.4 and 0.33, respectively, in the E-W and N-S directions, indicating that the building satisfies code level design force requirement. On the other hand, the ratio for Tabas earthquake is 5.10, suggesting that the overall level of global displacement ductility requirement according to base shear is of the order of 5 if we assume the equal displacement concept (Newmark and Hall 1973).

At the element level, columns marked P-9, P-10, and P-12 and the beams marked B-1 and B-2 on the plan view of Fig. 4 are chosen as representative elements. For safety considerations, demand/capacity ratios based on the minimum code requirements (modal superposition results increased by the ratio V_{base}(ESLF)/V_{base}(MS)) for the representative elements were evaluated. Such ratios for shear and moment at various stories were found to be less than about 0.5, indicating that from the code level strength demand (shear and moment) point of view, the members have sufficient capacity. Moreover, the 1.2 cm diameter ties with 30 cm spacing in perimeter columns of the first story generally satisfy the ACI code requirement of 48 times the tie diameter or 16 times the bar diameter. However, the 30 cm spacing does not strictly satisfy the criterion when half of these spacings are considered based on ACI seismic provisions.

The pure moment capacity of P-9 and P-10 columns at ground floor level are, respectively, 108 t-m and 181 t-m. Moment capacity of beams framing into these columns are, respectively, 348 t-m and 486 t-m. It is noted that on the average, beams are three times stronger than columns, and this is the typical ratio for other floors as well. Obviously weak beam - strong column design criterion has not been considered in the design of this building, which means under the extreme condition, columns probably yield before beams. The corner columns are part of corner shear walls marked as P-11 on Fig. 4. The ground floor axial load capacity of this column is 16,860 tonne. As examples of axial force demand, those due to the equivalent lateral force method and response spectrum analysis with PGA=0.5 g are, respectively, 4,560 tonne and 14,260 tonne. It is therefore concluded that the corner columns, which are usually of concern in tube structures, are not critical with respect to pure axial force. However, with tie spacing of 30 cm in the first story, the P-11 columns do not satisfy the ACI seismic provisions on tie spacing.

Table 4 Displacement and ductilities

Seismic input	Δ_{max} (cm)	μ_Δ
Equivalent static	7.87	1.0
PGA=0.3 g	29.02	3.69
PGA=0.4 g	35.11	4.5
PGA=0.5 g	47.92	6.0
E1 Centro	21.1	2.68
Naghan	33.5	4.25
Tabas	62.8	8.0

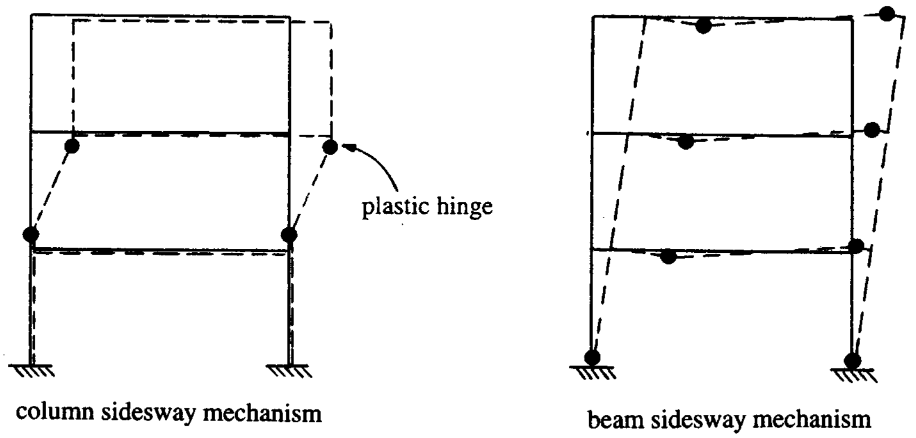

Fig. 12 Beam and column mechanisms

8. Toughness criterion

The structure displacement ductility demand μ_Δ can be approximated as the ratio of maximum displacement resulting from response spectrum or time history analysis to that of the equivalent static lateral force analysis. This approximate but practical approach is based on the assumption that all elements reach yield simultaneously, i.e., elastic perfectly plastic behavior, under code design forces, although we know elements reach yield at different times and make the structure load-displacement relation actually nonlinear. The resulting structure displacement ductility demands, varying between 2.68 and 8.0, are listed in Table 4. In general, global displacement duetility demand grater than 4.0 to 5.0 is considered high, since it requires large element ductility capacities (Park and Paulay 1975).

In order to evaluate the toughness criterion at the element level, it is necessary to determine the element curvature or rotation ductility demand when the building is subjected to the selected seismic input. In the absence of nonlinear dynamic or static push-over analysis software, it is possible to perform an approximate static collapse analysis by establishing a relation between the global displacement ductility demand and the corresponding local curvature ductility demand. Such a relation, however, is not unique and depends on the assumed collapse mechanism. In general, the column sidesway and the beam sidesway mechanisms, shown in Fig. 12, are the conventional mechanisms. If plastic hinges develop in columns before beams, and as the worst case all columns of one story develop plastic hinges at top and bottom, the mechanism is referred to as the column sidesway mechanism. If, however, beams develop plastic hinges before columns at all stories and only the bottom of the first story columns develop plastic hinges, the mechanism is known as the beam sidesway mechanism. As mentioned in the previous section, the beams are on the average three times stronger than columns, which makes the occurrence of a beam sidesway mechanism unlikely. On the other hand, occurrence of a column sidesway mechanism amounts to the requirement of curvature ductility capacity of members several times the corresponding ones in a beam sidesway mechanism. The curvature ductility demands at element level can be approximately determined using the collapse mechanism approach (Park and Paulay 1975). However, considering the already mentioned inadequate tie spacings, it is sufficient here to just determine ductility capacity of typical beam and columns for a more quantitative evaluation.

Based on the method described in Sec. 6, the curvature ductility capacity for perimeter beams varies between 22 and 40, with a mean value of 31. To determine ductility capacity for columns, we need to assume a compressive axial force level corresponding to which ductility capacity is determined. For example, for column type P-9, for two code level forces of 494 tonne and 165 tonne corresponding, respectively, to the ground and midheight stories, we get curvature ductility capacity of 2.7 and 5.6. Similarly, for column type P-10, at axial force levels of 590 ton and 197 tonne, the result is 2.1 and 4.5. In an average sense then, the beams have curvature ductility capacity of roughly 10 times those of the columns (for the elements considered). This shows that the perimeter columns are more critical than the beams and under an extreme loading condition will probably fail before beams, with the possibility of an undesireable presirable collapse mechanism mode. Obviously, for a more refined assessment, a static collapse analysis, nonlinear dynamic analysis, or cumulative damage analysis can be incorporated in the approach.

9. Conclusions

The paper has discussed some of the issues facing engineers in seismic evaluation of existing buildings from a practical point of view. The methodology employed deals with problems of lack of seismicity data for design earthquake determination and lack of nonlinear analysis capability for toughness limit state evaluation in a practical way suitable for design office application.

The study has provided some information on actual low amplitude natural vibration of a tall reinforced concrete framed tube building. It has discussed how ambient vibration test results can provide a reliable source to check the three-dimensional mathematical model and the effect of a framed tube modeling parameters on dynamic properties. The methodology discussed can be used to study the ultimate behavior of the structure in an approximate sense. The goal of such an evaluation is to attain a degree of confidence in the capacity of the structure to withstand the maximum credible earthquake without collapse or to have enough justification to further study the problem using the more sophisticated nonlinear analysis capability.

Since this building is a typical reinforced concrete framed tube building with design features generally practiced in most countries, the safety and toughness criteria check results can provide useful information for similar existing buildings. The study has shown that although the design generally satisfies a typical seismic code safety requirements with respect to member shear and moment demands, the weak beam-strong column criteria, that is now favored, has probably not been a consideration in the design philosophy of older framed tube buildings. Although at this time, there is still not a clear understanding of the true behavior of this particular type of construction in severe earthquakes, the study points out a possible problem with respect to ductility capacity of the columns of such construction in older buildings.

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