Effect of foundation flexibility on the elastic earthquake response of asymmetric structures

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ABSTRACT: A parametric study is presented of soil-structure interaction effects on torsional coupling in asymmetric buildings subjected to earthquakes. The study is based on a simple five-degree-of-freedom elastic interaction model consisting of a single-storey structural idealisation with homogeneous half-space foundation. The earthquake input is represented by idealised design spectra, obtained from detailed studies of a large number of strong motion records from the United States. By comparing the lateral and torsional responses of identical building models founded on both rigid and flexible foundations, the influence of soil-structure interaction on torsional coupling is assessed for a range of building-foundation systems. Use is made of the effective eccentricity concept to define the coupled response, or edge displacement of the building. Comparisons are also made with the equivalent static design torque and shear provisions of major building codes. The results indicate that for certain ranges of the parameters which control structural response, typical of actual buildings, interacting systems exhibit a substantial increase in torsional response compared with equivalent rigidly based structures. In these cases increased allowance for torsional coupling is necessary for safe design.

1 INTRODUCTION

Widespread investigations of linear and non-linear interaction effects on the response of structures have been presented in the literature (Whitman 1970, Liu & Fagel 1971, Chopra & Gutierrez 1974, Luco 1985), however for buildings which have structural eccentricities little is known of the effects of interaction on the resultant torsional response arising during earthquake vibrations. The Mexico City earthquake of 1985 resulted in the partial or total collapse of several buildings due to torsion (Chandler 1986), and as a result it has been suggested that the Mexican earthquake code design recommendations for torsional effects in asymmetric buildings (National University of Mexico 1977) do not include a sufficient safety margin.

The idealised structure-soil model employed in this study comprises a single-storey building with structural eccentricity perpendicular to the direction of earthquake input, resting on the surface of an elastic homogeneous half space (Figure 1). To facilitate analysis of response in the time domain, interaction forces at the building-soil interface are represented by frequency independent spring-dashpot coefficients, making suitable adjustment to impedance values according to an approximate procedure which takes into account the influence of modal coupling and the values of the controlling system parameters. By varying parameters which represent dynamic soil properties, a wide range of foundation flexibility has been considered.

The building-foundation system in this form does not admit classical normal modes because of non-proportional damping in the foundation medium, and hence straightforward application of the response spectrum technique is not possible. The procedure proposed by Tsai (1974) and later developed for asymmetric buildings by Balendra, Tat & Lee (1982) is employed in this study to obtain approximate normal modes and the corresponding modal damping for the asymmetric shear building model. The technique matches rigorous and approximate normal mode solutions of the response of the structure for all five natural frequencies of the interaction system. The maximum structural response to earthquake input is then obtained by standard response spectrum techniques, employing the complete quadratic combination (CQC) method (Newmark & Rosenblueth 1971) to combine the modal maxima. A detailed parametric study has been

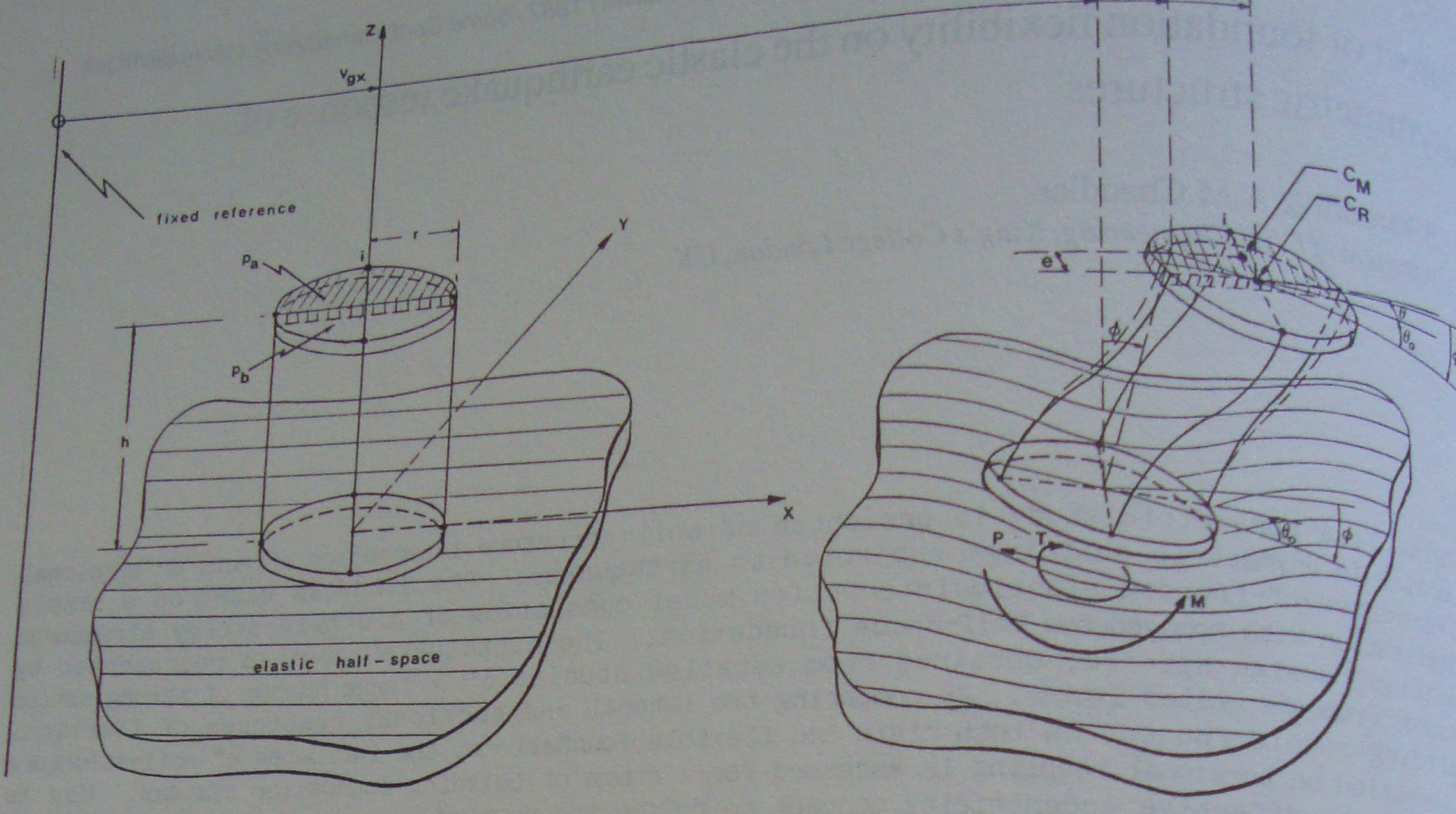

Fig.1. Response of a torsionally coupled single-storey building model on a flexible foundation

implemented for ranges of the key controlling parameters, typical of actual building-soil systems. First, analysis is made for a rigid foundation, comparing the dynamic floor lateral and torsional responses with the design provisions of major codes. The study is then extended to cover a range of flexible foundations representing typical soil properties, including suitable comparison with rigid foundation results in order to assess the importance of interaction on torsional response. Finally, comments are made on current design practice in the light of these results, and the need for revision in certain cases to allow for increased torsional response is indicated.

2 THEORETICAL DEVELOPMENT

2.1 Idealised single-storey interaction model

The structure-foundation model employed in the analysis is shown in Figure 1. A single-storey building of height h and radius r is supported on an elastic homogeneous half-space of Poisson's ratio ν, mass density ρ and shear wave velocity V_s. The floor diaphragm and foundation mat are idealised rigid circular discs of negligible thickness and have masses m and m_o respectively. The static eccentricity e of the building, measured between the centre of mass C_M and the centre of resistance C_R (the latter coincident with the centre of the floor disc), arises due to different mass densities ρ_a and ρ_b (ρ_a > ρ_b) in the two halves of the disc, and is within the range $0 \leq e \leq 0.424r$ (Chandler 1985). For a uni-directional horizontal component of free-field ground acceleration $\ddot{v}_{gx}$, assumed to be uniform over the base of the building, the system has five degrees of freedom, viz: i) translation v_M of the centre of mass C_M in the x-direction due to structural deformation, ii) rotation θ of the floor mass about the z-axis (through C_R) due to structural deformation; and three degrees of freedom due to interaction at the building-soil interface, namely iii) translation v_o of the foundation mass in the x-direction, iv) rotation θ_o of the foundation mass about the z-axis, and v) rocking ϕ of the whole building about the y-axis. Note that $v_M = v_R - e\theta$ (see Figure 1). The equations of motion for translation in the x-direction and rotation about C_M are expressed as

$$\underset{[M_S]}{\begin{bmatrix} m & 0 \\ 0 & ml^2 \end{bmatrix}} \begin{Bmatrix} \ddot{v}_M^t \\ \ddot{\theta}^t \end{Bmatrix} + \underset{[C_S]}{\begin{bmatrix} c_v & ec_v \\ ec_v & c_{\theta M} \end{bmatrix}} \begin{Bmatrix} \dot{v}_M \\ \dot{\theta} \end{Bmatrix} + \underset{[K_S]}{\begin{bmatrix} K_v & eK_v \\ eK_v & K_{\theta M} \end{bmatrix}} \begin{Bmatrix} v_M \\ \theta \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \quad (1)$$

where v_M^t is the total displacement of C_M in the x-direction ($= v_{gx} + v_o + h\phi + v_M$), θ^t is the total twist of the floor about the z-axis ($= \theta_o + \theta$), l is the radius of gyration about C_M and the corresponding mass moment of inertia $J_{\theta M} = ml^2$; K_v and $K_{\theta M}$ are the lateral and torsional stiffnesses defined at C_R and C_M, respectively, c_v and $c_{\theta M}$ are the corresponding damping for the superstructure, assumed to be proportional to the stiffness as defined by:

$$c_v = \beta K_v \text{ , } c_{\theta M} = \beta K_{\theta M} \tag{2}$$

The constant of proportionality, β, is evaluated on the basis of five per cent of critical damping in the fundamental mode of the superstructure (Balendra et al 1982). The torsional stiffness and corresponding damping defined at C_R are given by:

$$K_{\theta R} = K_{\theta M} - e^2 K_v \text{ , } c_{\theta R} = c_{\theta M} - e^2 c_v \tag{3}$$

The equations of motion for the whole system are expressed as:

$$m_o(\ddot{v}_{gx} + \ddot{v}_o) + m\ddot{v}_M{}^t + P(t) = 0 \tag{4a}$$
$$l^2 m_o \ddot{\theta}_o + l^2 m\ddot{\theta}^t + T(t) = 0 \tag{4b}$$
$$(I_{y_0} + I_{y_1})\ddot{\phi} + hm\ddot{v}_M{}^t + M(t) = 0 \tag{4c}$$

where I_{yj} (j=0,1) are moments of inertia of the base and floor masses respectively about the y-axis, P(t), T(t) and M(t) are the interaction forces for translation, rotation and rocking respectively (Figure 1). These interaction forces are approximated by frequency independent spring-dashpot coefficients (Richart, Hall & Woods 1970) in the form:

$$P(t) = C_T\dot{v}_o + K_T v_o \tag{5a}$$
$$T(t) = C_Z\dot{\theta}_o + K_Z\theta_o \tag{5b}$$
$$M(t) = C_R\dot{\phi} + K_R\phi \tag{5c}$$

where the coefficients C_T, C_Z, C_R, K_T, K_Z, K_R have been developed according to the method of Ghaffar-Zadeh and Chapel (1983).

2.2 Approximate normal mode method

Putting $v_M{}^t = v_M{}' + v_{gx}$ (where $v_M{}' = v_o + h\phi + v_M$), equations (1) and (4) can be written as

$$[M']\{\ddot{Y}\} + [C']\{\dot{Y}\} + [K']\{Y\} = -\{P'\} \tag{6a}$$

Where

$$\{Y\} = \begin{Bmatrix} v_M{}' \\ \theta t \\ v_o \\ \theta_o \\ \phi \end{Bmatrix} \qquad \{P'\} = \begin{Bmatrix} m\ddot{v}_{gx} \\ 0 \\ m_o\ddot{v}_{gx} \\ 0 \\ 0 \end{Bmatrix} \tag{6b,6c}$$

$$[M'] = \begin{bmatrix} [M_S] & & & \\ & m_o & & \\ & & l^2 m_o & \\ & & & I_{y_0} + I_{y_1} \end{bmatrix} \tag{6d}$$

$$[C'] = \begin{bmatrix} [C_S] & \begin{matrix} -c_v & -c_v e & - c_v h \\ -c_v e & -c_{\theta M} & -ec_v h \end{matrix} \\ & \begin{matrix} C_T + c_v & ec_v & c_v h \\ & C_Z + c_{\theta M} & ec_v h \\ & & C_R + h^2 c_v \end{matrix} \end{bmatrix} \tag{6e}$$

and matrix [K'] is obtained by replacing $[C_S]$ with $[K_S]$; c_v and $c_{\theta M}$ with K_v and $K_{\theta M}$; C_T, C_Z and C_R with K_T, K_Z and K_R in equation (6e).

Define the following transformation:

$$\{Y\} = [A_1]\{r\} \tag{7a}$$

in which

$$[A_1] = \left[\begin{array}{c|ccc} [\Phi] & & & \\ \hline & 1/(m_o)^{1/2} & & \\ & & 1/l(m_o)^{1/2} & \\ & & & 1/(I_{y_0}+I_{y_1})^{1/2} \end{array}\right] \tag{7b}$$

where

$$[\Phi] = \begin{bmatrix} \phi_v{}^1 & \phi_v{}^2 \\ \phi_\theta{}^1 & \phi_\theta{}^2 \end{bmatrix} \tag{7c}$$

is the [2x2] normal mode matrix for the superstructure, satisfying:

$$[\Phi]^T [M_S][\Phi] = [I] \tag{8a}$$
$$[\Phi]^T [K_S][\Phi] = [\omega_k{}^2] \tag{8b}$$
$$[\Phi]^T [C_S][\Phi] = [2\zeta_k\omega_k] \tag{8c}$$

in which ω_k (k=1,2) are the lateral and torsional circular frequencies and ζ_k (k=1,2) are the modal fractions of critical damping for the superstructure ($\zeta_1=0.05$, $\zeta_2=\beta\omega_2/2$), and [I] is a unit matrix. Premultiplying equation (6a) by $[A_1]^T$ and applying equation (7a) gives:

$$\{\ddot{r}\} + [\hat{C}]\{\dot{r}\} + [\hat{K}]\{r\} = -\{\hat{Y}\} \tag{9a}$$

where

$$[\hat{K}] = \left[\begin{array}{c|c} [K_U] & [K_{UL}]^T \\ [K_{UL}] & [K_L] \end{array}\right]_{5x5} \tag{9b}$$

$$[K_U] = [\omega_k{}^2]_{2x2} \text{ , } k=1,2 \tag{9c}$$

and the submatrices $[K_{UL}]$ and $[K_L]$ are given by Balendra et al (1982). The matrix $[\hat{C}]$ is obtained by replacing $[\omega_k{}^2]$ by $[2\zeta_k\omega_k]$ and K_T, K_Z and K_R by C_T, C_Z and C_R in $[\hat{K}]$. Furthermore

$$\{\hat{Y}\}^T = \{m\phi_v{}^1 \;\; m\phi_v{}^2 \;\; (m_o)^{1/2}\ddot{v}_{gx} \;\; 0 \;\; 0\} \tag{9d}$$

Consider a second coordinate transformation

$$\{r\} = [A_2]\{q\} \tag{10a}$$

in which

$$[A_2]^T[A_2] = [I] \tag{10b}$$
$$[A_2]^T[\hat{K}][A_2] = [\bar{\omega}_k{}^2] \text{ , } k=1 \text{ to } 5 \tag{10c}$$

where $\bar{\omega}_k$, k=1 to 5, are the natural circular frequencies of the building-foundation system.

Substituting equation (10a) into (9a), and premultiplying by $[A_2]^T$ gives:

$$\{\ddot{q}\} + [\bar{C}]\{\dot{q}\} + [\bar{\omega}_k{}^2]\{q\} = -\{\bar{Y}\} \tag{11a}$$

(11b)

(11c)

$$\text{where} \quad [\overline{C}] = [A_2]^T[\hat{C}][A_2]$$
$$\{\overline{Y}\} = [A_2]^T\{\hat{Y}\}$$

In equation (11a), the 5 equations are coupled by the off-diagonal terms in $[\overline{C}]$; hence in applying the normal mode method it is assumed that equation (11a) can be approximated by the following uncoupled equations:

$$\{\ddot{q}\} + [2\overline{\zeta}_k\overline{\omega}_k]\{\dot{q}\} + [\overline{\omega}_k{}^2]\{q\} = -\{\overline{Y}\} \tag{12}$$

The response vector in equation (6a) can be approximated to:

$$\{\hat{Y}\} = [\Delta]\{q\}, \text{ where } [\Delta] = [A_1][A_2] \tag{13}$$

2.3 Estimation of modal damping

By assuming harmonic ground motion $v_{gx}=e^{i\omega t}$, where ω is the circular frequency of the ground excitation, the exact solution for response may be expressed as:

$$\{Y(\omega)\}=\omega^2[A_1]\left[[\hat{K}]-\omega^2[I]+i\omega[\hat{C}]\right]^{-1}\{\hat{Y}\} \tag{14}$$

and the approximate normal mode solution is:

$$\{\hat{Y}(\omega)\}=\omega^2[\Delta]\left[[\overline{\omega}_k{}^2]-\omega^2[I]+i\omega[2\overline{\zeta}_k\overline{\omega}_k]\right]^{-1}\{\overline{Y}\} \tag{15}$$

The modal damping $\overline{\zeta}_k$ is then determined by matching the exact and approximate response from equations (14) and (15), at frequencies $\omega=\overline{\omega}_k$ (k=1 to 5), and for a dominant approximate modal response component in each mode considered. For this study, these have been established as response components 1,2,4 for modes 1,2,4 and the maximum approximate response component for modes 3 and 5. Hence

$$|Y_j(\overline{\omega}_k)| = |\hat{Y}_j(\overline{\omega}_k)| \text{ , k=1 to 5} \tag{16}$$

and j is the degree of freedom as given above. A first estimate of $\overline{\zeta}_k$ is obtained by neglecting the off-diagonal terms in $[\overline{C}]$ (equation (11a)), hence

$$\overline{\zeta}_k = \frac{\overline{C}_{kk}}{2\overline{\omega}_k} \text{ , k=1 to 5} \tag{17}$$

By varying the damping ratio corresponding to mode 1 ($\overline{\zeta}_1$) whilst keeping the others unaltered, the right-hand side of equation (16) is iterated until the responses are equal. The second damping ratio $\overline{\zeta}_2$ is then evaluated using the established value for $\overline{\zeta}_1$ and values given by equation (17) for the remainder. Similarly, $\overline{\zeta}_3$, $\overline{\zeta}_4$ and $\overline{\zeta}_5$ are established.

2.4 Earthquake spectra

The earthquake input used in this study is represented by design spectra as proposed by Hall, Mohraz & Newmark (1976), obtained from a statistical study of 85 accelerogram records from western United States spanning a 39-year period beginning 1933. Based on a given maximum ground motion and a specified cumulative probability of exceedance, spectral amplification factors for displacement, velocity and acceleration are specified for different critical damping ratios and across a frequency range. Hence, the results could be presented in a tripartite logarithmic form. The ground motion set used in this study is normalised to a maximum ground acceleration, a, of 0.3g, a (v/a) ratio (v = maximum ground velocity) of 48 in/sec/g (corresponding to an intermediate soil stiffness), and an ad/v^2 ratio (d = maximum ground displacement) of 6.0 to ensure that the spectrum represents an adequate band (frequency) width to accomodate a range of earthquakes. Also a cumulative probability of exceedance of 15.8% (median plus one standard deviation) is chosen, which is a value commonly used in design spectra to incorporate an adequate degree of conservatism.

Figure 2 shows acceleration spectra normalised to a maximum ground acceleration of 0.3g, vs natural period for damping of 2, 5, 10, 20, 30 and 40 per cent of critical. Also shown on the figure are normalised 5% acceleration spectra for the El Centro earthquake of 1940 and Romanian earthquake of 1977. The design spectrum for 5% damping provides a reasonable representation of the corresponding El Centro spectrum throughout the period range shown; in contrast the same design spectrum gives a poor match with the Romanian curve, particularly for periods longer than 1 sec where there is a significant underestimation of the calculated response. This highlights the differences in spectral shape observed in earthquakes recorded under different localised conditions, which will be discussed with reference to torsional response effects in section 3.3.

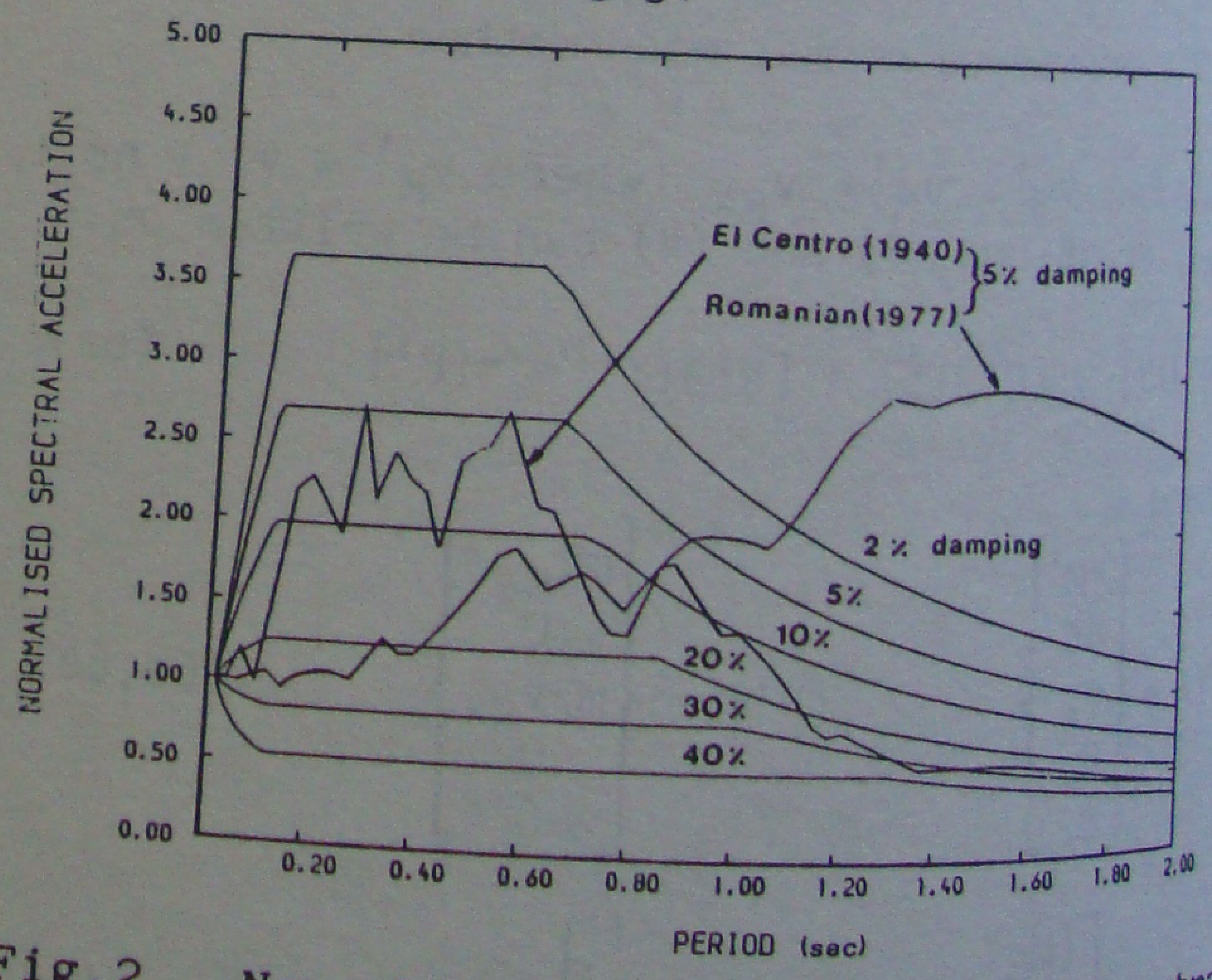

Fig.2. Normalised design acceleration spectra for various damping values (after Hall, Mohraz and Newmark 1976), together with response spectra for 5 per cent damping for the El Centro (1940) and Romanian (1977) earthquakes

3 PARAMETRIC RESPONSE OF RIGIDLY BASED STRUCTURE

3.1 Choice of parameters

In order to carry out a detailed study of the soil-structure system, the parameters influencing the response are developed in a non-dimensional form and implemented in the equations of motion (1) and (4). The parameters employed are listed in Table 1, and consist of the non-dimensional terms e_r, λ_T, α and δ_h; the following values have been assigned to parameters whose influence on the trend of response has been found to be negligible: $\nu=1/3$, $\delta_m=0.3$, $\delta_p=0.14$. The natural frequencies of the system are calculated on the basis of the uncoupled translational natural period T_v, relating to the rigidly based building. Parameter α in Table 1 is a measure of the structure-foundation stiffness and is a function of the total height H of the building (see below), the shear wave velocity V_s of the foundation medium and the fundamental coupled frequency ω_1 of the rigidly based structure. Values of α considered in this paper are α=3, 6, 10 and ∞, the first three values corresponding approximately to shear wave velocities of 100, 200 and 330 m/s, respectively, and the latter value corresponding to a rigid foundation. Although some differences in results are observed between α=10 and $\alpha=\infty$, these are not large; consequently values of α intermediate between 10 and infinity are not presented. The parameters e_r and λ_T are of special significance in studies of torsional coupling in rigidly based buildings (Chandler & Hutchinson 1987); e_r is a measure of the static eccentricity e/r of the structure and λ_T is the ratio of uncoupled torsional and translational natural frequencies ω_θ/ω_v, where

$$\omega_v=2\pi/T_v=(K_v/m)^{1/2}\ ;\ \omega_\theta=(K_{\theta R}/J_{\theta R})^{1/2} \quad (18)$$

The terms K_v, $K_{\theta R}$ and $J_{\theta R}$ (= $J_{\theta M}$−me^2) have been defined in section 2.1.

By assuming an empirical relationship T_v=0.1N (Applied Technology Council 1978) between the natural period T_v and N, the number of building storeys, the following relationships are derived:

$$m = 10T_v m_s \quad (19a)$$
$$h = (H+h_s)/2 \quad (H=Nh_s) \quad (19b)$$

where m_s is the floor mass of each storey (assuming a floor intensity of 5.2 kN/m² and r=10m), h_s is the inter-storey height (taken as 3.75m), and H is the total height of the building. The building mass m, together with height h measured to the overall centre of mass of the N-storey building, are employed in this study with the single-storey idealisation shown in Figure 1.

3.2 Torsional code provisions

The design storey torque given by building codes comprises two parts:

$$T_b = T_{eb} + T_a \quad (20)$$

where T_{eb} and T_a are equivalent static torsional moments accounting for static and accidental eccentricity respectively. The latter term accounts, approximately, for a number of effects which cannot be quantitatively determined (Chandler 1985). The components T_{eb} and T_a are written in general form

$$T_{eb} = aeQ_{vu} \quad (21a)$$
$$T_a = bYQ_{vu} \quad (21b)$$

where Y is the plan dimension of the building perpendicular to the earthquake direction, Q_{vu} is the uncoupled (design) shear corresponding to e=0, and a,b are coefficients whose values

Table 1. Dimensionless parameters for single-storey torsionally coupled interacting systems.

Type	Description	Definition	Symbol
Structural parameters	Static eccentricity ratio	e/r	e_r
	Torsional to translational freq. ratio	ω_θ/ω_v	λ_T
	Mass ratio	m_o/m	δ_m
	Height ratio	h/r	δ_h
Soil parameters	Poisson's ratio	-	ν
Interaction parameters	Density ratio	$m/\pi r^2 hp$	δ_p
	Shear wave velocity ratio	$V_s/H(\omega_1/2\pi)$	α

for the codes considered are specified in Table 2. The design eccentricity is given by $e_b = T_b/Q_{vu} = ae + bY$, and the dynamic eccentricity ratio is defined as:

$$e_{dr} = e_d/r = T_{eb}/rQ_{vu} = ae_r \qquad (22)$$

where the coefficient, a, represents the code allowance for torsional coupling, in terms of the amplification of static eccentricity required to induce the design torsional moment T_{eb}.

Table 2. Code coefficients for design eccentricity.

Building codes	a	b
ATC3 (California)	1.0	0.05
Canada	1.5	0.10
Mexico	1.5	0.10
New Zealand	$1.7-0.5e_r$	0.10
Eurocode No.8	$1+e_{1r}/e_r$	0.05

The supplementary eccentricity ratio e_{1r} specified by Eurocode 8 (1984) is assigned the smaller of the two values computed from the following expressions:

$$e_{1r}=0.894(e_r)^{1/2}, \leq 0.4 \qquad (23a)$$

$$\text{or } e_{1r}=(0.5/e_r)\{0.667-e_r^2-0.5\lambda_T^2+[(0.667+e_r^2-0.5\lambda_T^2)^2+2e_r^2\lambda_T^2]^{1/2}\} \qquad (23b)$$

Furthermore, if $\lambda_T^2 > 10(0.667+e_r^2)$, then e_{1r} is taken to be zero.

3.3 Effective eccentricity approach

The effective eccentricity $e_e^{(i)}$ (Dempsey & Tso 1982) represents the primary design requirement (that is, for the displacement of edge element i in Figure 1), and is found by matching the peak displacement v_i^{max} of element i as obtained by dynamic analysis with that corresponding to an effective storey torque given by $T_{eb} = e_e^{(i)}Q_{vu} = e_{er}^{(i)}(rQ_{vu})$. Q_{vu} represents the design storey shear as specified by all the building codes in section 3.2, determined by analysis of the response of the uncoupled structure to specified earthquake loading. Hence $Q_{vu} = K_v v_u(\omega_v,\zeta)$ where v_u is the peak (spectral) displacement. The expression for primary design effective eccentricity ratio is then given by (Chandler 1985):

$$e_{er}^{(i)} = \lambda_T^2(R_i-1)/2 \qquad (24)$$

where R_i is the coupled to uncoupled edge displacement ratio v_i^{max}/v_u.

The variation of $e_{er}^{(i)}$ with e_r for rigidly based buildings is shown in Figures 3(a)-(c). Figure 3(a) shows the maximum effective eccentricity envelopes obtained from dynamic response to the design spectra illustrated in Figure 2, for T_v of 0.1, 0.2, 0.5 and 2.0 sec, and assuming 5% of critical damping in the fundamental mode. For each value of T_v (corresponding to buildings of the proportions shown in Figure 3(a)), the envelopes have been drawn from results obtained over a range of λ_T between 0.6 and 2.0, corresponding to most actual buildings (Hart, DiJulio & Lew 1975). Shown on the same figure are the average response curves for a range of T_v

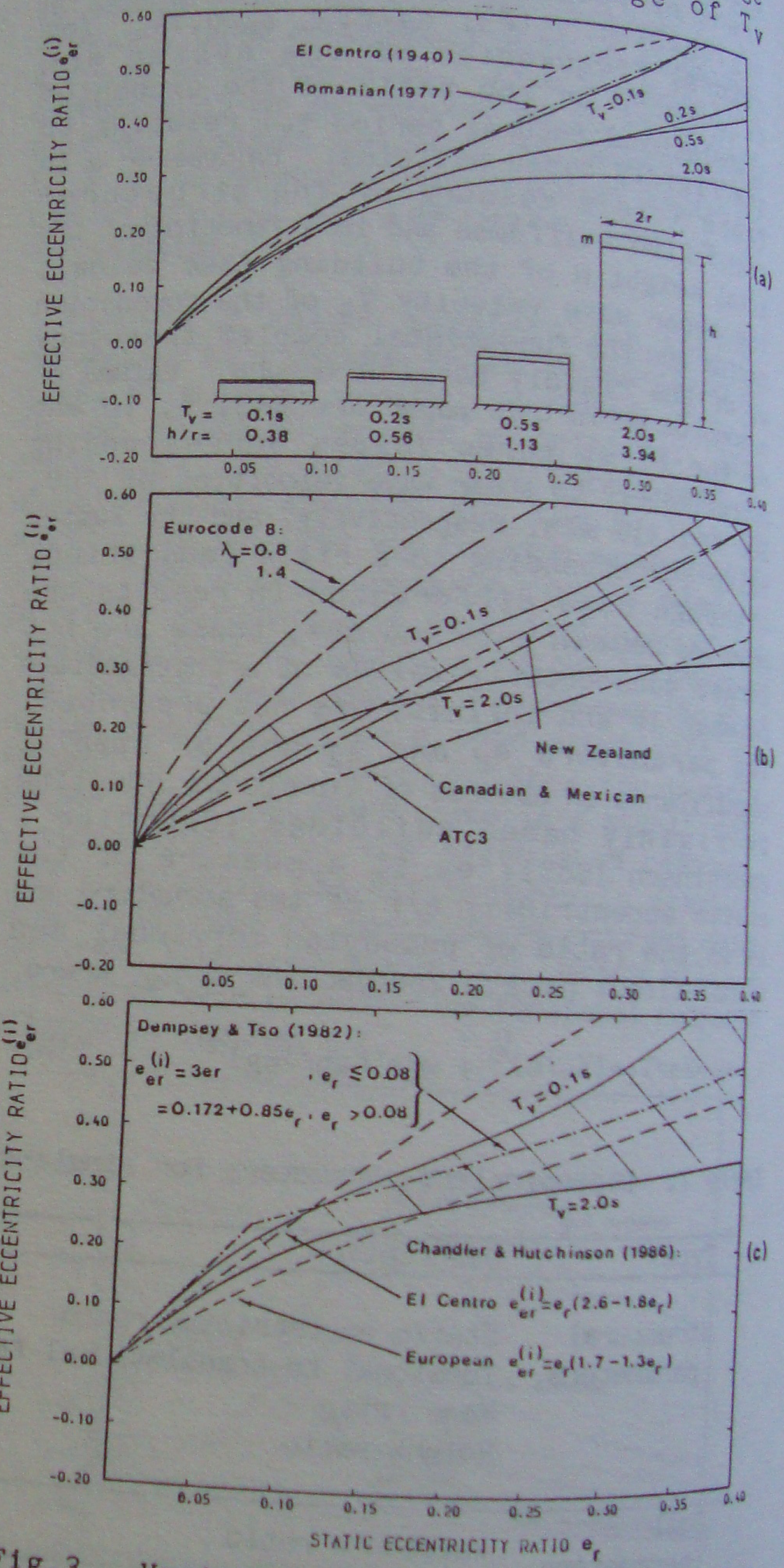

Fig.3. Variation of envelopes of effective eccentricity ratio with e_r for rigidly based buildings, and comparison with response to the El Centro and Romanian earthquakes (a), with the dynamic eccentricity provisions of major building codes (b), and with various alternative design proposals (c)

corresponding to the El Centro and Romanian earthquakes, obtained by time history analysis assuming 5% damping in both vibration modes. The results obtained using design spectra indicate that torsional coupling is especially significant for short-period buildings, as expected in view of the spectral shape. For $e_r < 0.15$, the time history curves match the results obtained using design spectra ; for higher values of e_r, time history analysis yields the more conservative effective eccentricity requirements for the records chosen. Despite the variation in spectral shape between the El Centro and Romanian earthquakes (Figure 2), the effective eccentricity envelope averaged over a range of T_v (between 0.1 and 2.0 sec) is relatively consistent (Figure 3(a)).

Figure 3(b) compares the results of this study for T_V=0.1 and 2.0 sec (the upper and lower bounds to the results shown in Figure 3(a)) with the corresponding building code provisions for dynamic eccentricity ratio (equation (22), see also Table 2). The results of earlier studies (Tsicnias & Hutchinson 1981, Chandler & Hutchinson 1987) are confirmed; namely, the ATC3 (Applied Technology Council 1978), Canadian (National Building Code of Canada 1985), Mexican (National University of Mexico 1977) and New Zealand (Standards Association of New Zealand 1976) codes are non-conservative to varying degrees for buildings with small eccentricity ($e_r < 0.15$). For higher eccentricity ratios, the Canadian, Mexican and New Zealand codes give reasonable estimates of response, whilst the ATC3 code remains non-conservative to a decreasing extent. The Eurocode 8 (1984) provisions, shown for λ_T=0.8 and 1.4, are conservative throughout and especially for large eccentricities.

A further assessment of dynamic results is made in Figure 3(c), in comparison with the bi-linear and parabolic approximations of effective eccentricity suggested by Dempsey & Tso (1982) and Chandler & Hutchinson (1986), respectively. In the latter case two curves are shown corresponding to the computed responses to the El Centro (1940) earthquake, and the average response to a series of 7 recent strong-motion European earthquakes including the Romanian earthquake of 1977. The Dempsey and Tso envelope is reasonable for $e_r < 0.10$, but is non-conservative compared with the higher of the dynamic response envelopes (T_V = 0.1 sec) for $e_r > 0.15$. The average results for European earthquakes indicate less torsional coupling for structures with small eccentricities (e_r < 0.2) than obtained using the design spectra of Figure 2.

4 EFFECT OF INTERACTION ON LATERAL-TORSIONAL COUPLING

4.1 Dynamic amplification of shear and torque

To understand torsional coupling effects in elastically founded buildings by a study of parametric response trends (enabling comparisons with building code provisions), it is useful to consider these effects in relation to the key structural parameters e_r, λ_T and the foundation stiffness parameter α, by averaging selected response quantities over a wide range of T_V. The trends observed are then representative of buildings with a range of natural periods and their general applicability is enhanced. These characteristics are examined using two response ratios:

(i) shear amplification ratio (Q_V/Q_{VU}), where Q_V is the dynamic storey shear calculated for the coupled system. Both Q_V and Q_{VU} are calculated for the interaction model, the uncoupled structure having three degrees of freedom.

(ii) amplification of eccentricity (e_{dr}/e_r), where e_{dr} is given in equation (22), T_{eb} being the dynamic storey torque calculated for the coupled system.

Figure 4(a) illustrates the variation of the shear amplification ratio Q_V/Q_{VU} with λ_T, for models with α=3, 6, 10 and ∞ (a range from very soft to infinitely rigid soils). The structure-soil model is subjected to free-field earthquake spectra as shown in Figure 2, taking damping in the fundamental mode to be ζ_1=0.05 and eccentricity ratio e_r=0.3. Furthermore the results presented are based on the average response of systems with T_V in the range 0.1-2.0 sec. The recommendation of building codes, Q_V/Q_{VU} = 1 (i.e. no allowance for shear reduction) is also shown. Comparison of the curves for various α show that the reduction of shear for a soft soil condition (α=3), when $\lambda_T > 1$, is less than for stiffer soils ($\alpha \geq 6$), for which interaction has negligible effect. For $\lambda_T < 1$ the reverse situation applies, with sharp reduction of shear for α=3. In all cases the coupled shear is less than the corresponding uncoupled value.

Figure 4(b) compares the dynamic amplification of eccentricity e_{dr}/e_r (arising from the effects of torsional coupling) with the corresponding recommendations of major building codes (Table 2), taking ζ_1=0.05 and e_r=0.05. The results of dynamic analysis show that greatest amplification occurs at, or around λ_T=1 for rigidly based buildings (see also Tsicnias & Hutchinson 1981, Chandler & Hutchinson 1987), whilst for buildings

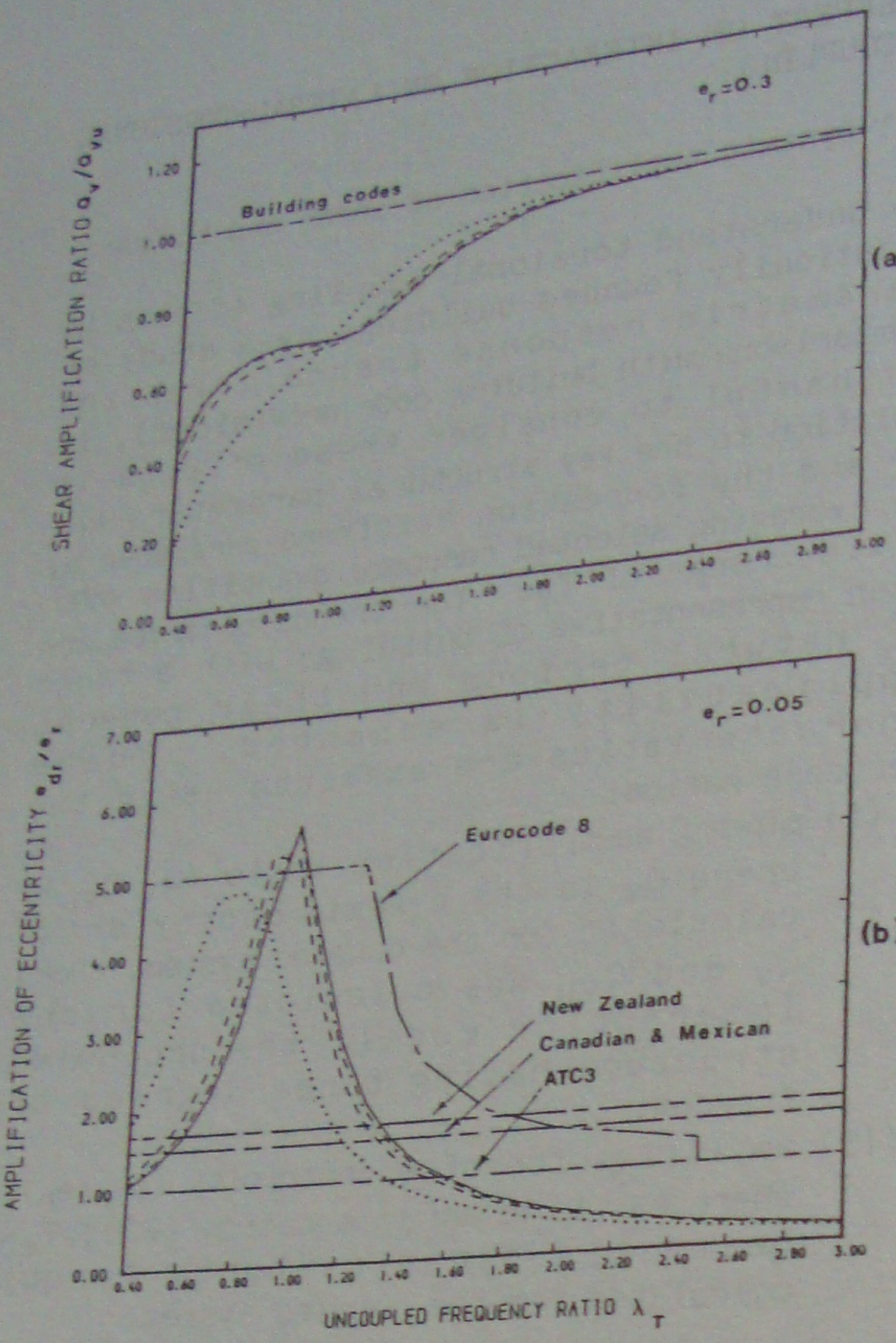

Fig.4. Variation of the dynamic amplification of shear (a) and eccentricity (b) with λ_T for the flexibly based building model with ζ_1=0.05, and comparison with the corresponding values given by building codes (—— - ——); α=3 (........), α=6 (— — — —), α=10 (—·—·—), α=∞ (———)

supported on flexible foundations the maximum amplification occurs when λ_T is somewhat less than unity (when α=3 for example, the peak occurs at λ_T=0.8). For λ_T < 0.85, systems with flexible foundations exhibit much higher amplification ratios than for corresponding rigidly based buildings. The significance of this result is discussed in section 4.2.

The inadequacy of most building codes in accounting for the amplification of dynamic torque due to coupling effects between lateral and torsional responses is highlighted in Figure 4(b). Within the range 0.6 < λ_T < 1.4, representative of most actual buildings (Hart, DiJulio & Lew 1975), all codes except Eurocode 8 severely underestimate the response. Eurocode 8 gives an accurate provision for the peak response but is over-conservative for λ_T < 0.9 and λ_T > 1.1. For λ_T > 1.6, all codes are conservative in their estimation of torsional response.

4.2 Influence of interaction on effective eccentricity

Figure 5 shows the variation of effective eccentricity ratio $e_{er}^{(i)}$ (equation (24)) with λ_T, for various α and setting e_r=0.15, the results being averaged over 0.1 < T_y < 2.0 sec.

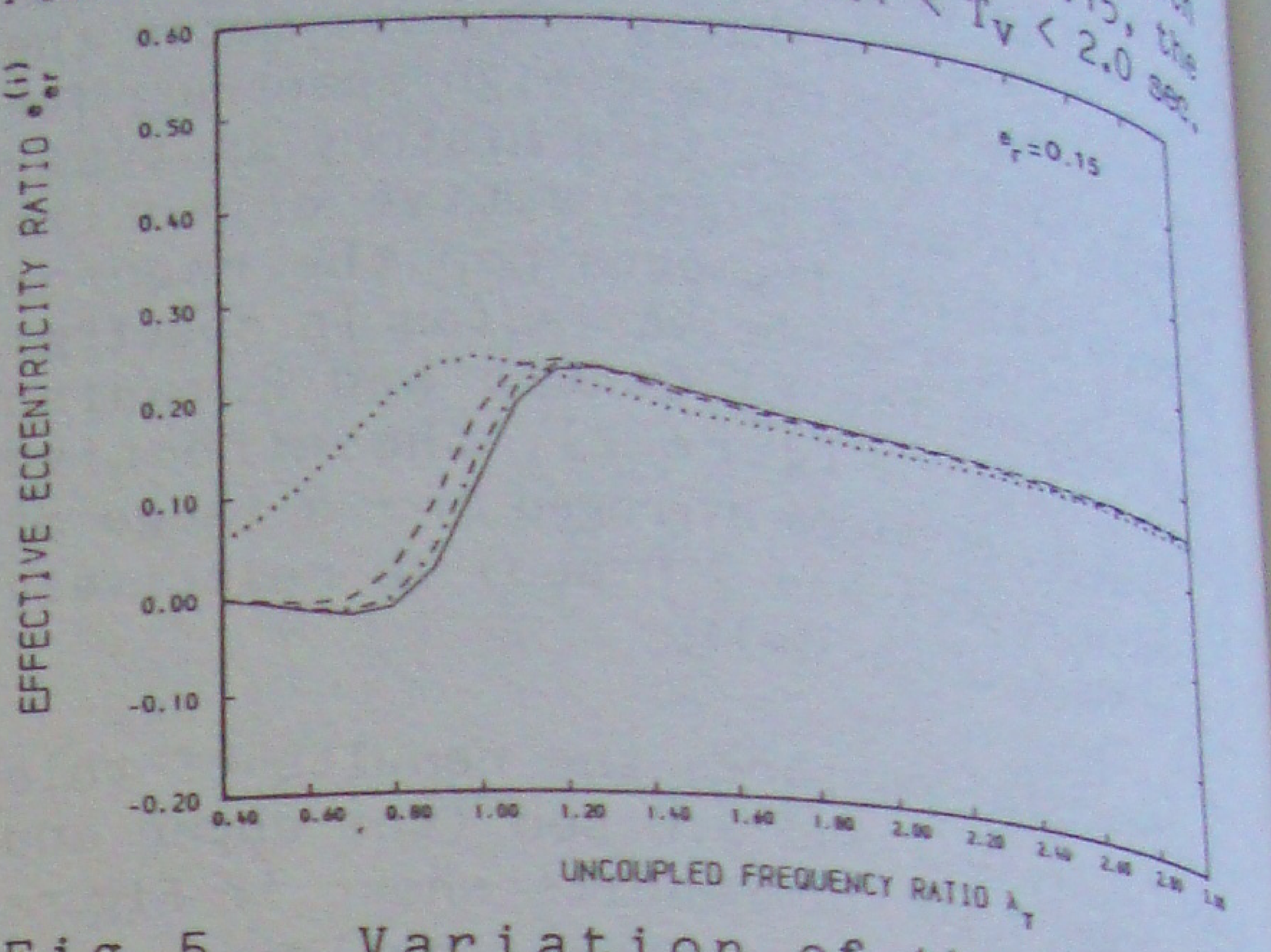

Fig.5. Variation of the effective eccentricity ratio with λ_T for the flexibly based building model (e_r=0.15); α=3 (......), α=6 (— — — —), α=10 (—·—·—), α=∞ (———)

It is shown that for λ_T < 1.2, interacting systems exhibit a much greater coupled response than the corresponding rigidly based structures, the effect being most pronounced for α=3. In contrast, for λ_T > 1.2 interaction has little effect on response. In Figures 6(a) and 6(b) the variation of $e_{er}^{(i)}$ with e_r is shown, setting λ_T=0.8 and 1.4 respectively, and comparison is made with major code provisions. In Figure 6(a), interaction induces a significant increase in effective eccentricity over the full range of e_r, although the building codes adequately account for this effect with the exception of buildings founded on very soft soils (α=3) when e_r is small or moderate. Interaction has little effect when λ_T=1.4 (Figure 6(b)) and hence it is sufficient in this case to analyse for a rigid foundation in accounting for torsional effects. The Canadian, Mexican and New Zealand codes give a good estimate of response for e_r < 0.15, whilst in the same range the ATC3 and Eurocode 8 provisions are non-conservative and over-conservative, respectively. For higher values of e_r, all codes except ATC3 are conservative.

5 INCORPORATION OF INTERACTION EFFECTS IN CODE DESIGN RECOMMENDATIONS

Comparisons made in this study with current building code provisions (see Figures 4,6) indicate that inadequacies identified in several earlier studies based on rigid foundations (Tsicnias & Hutchinson 1981, Tso &

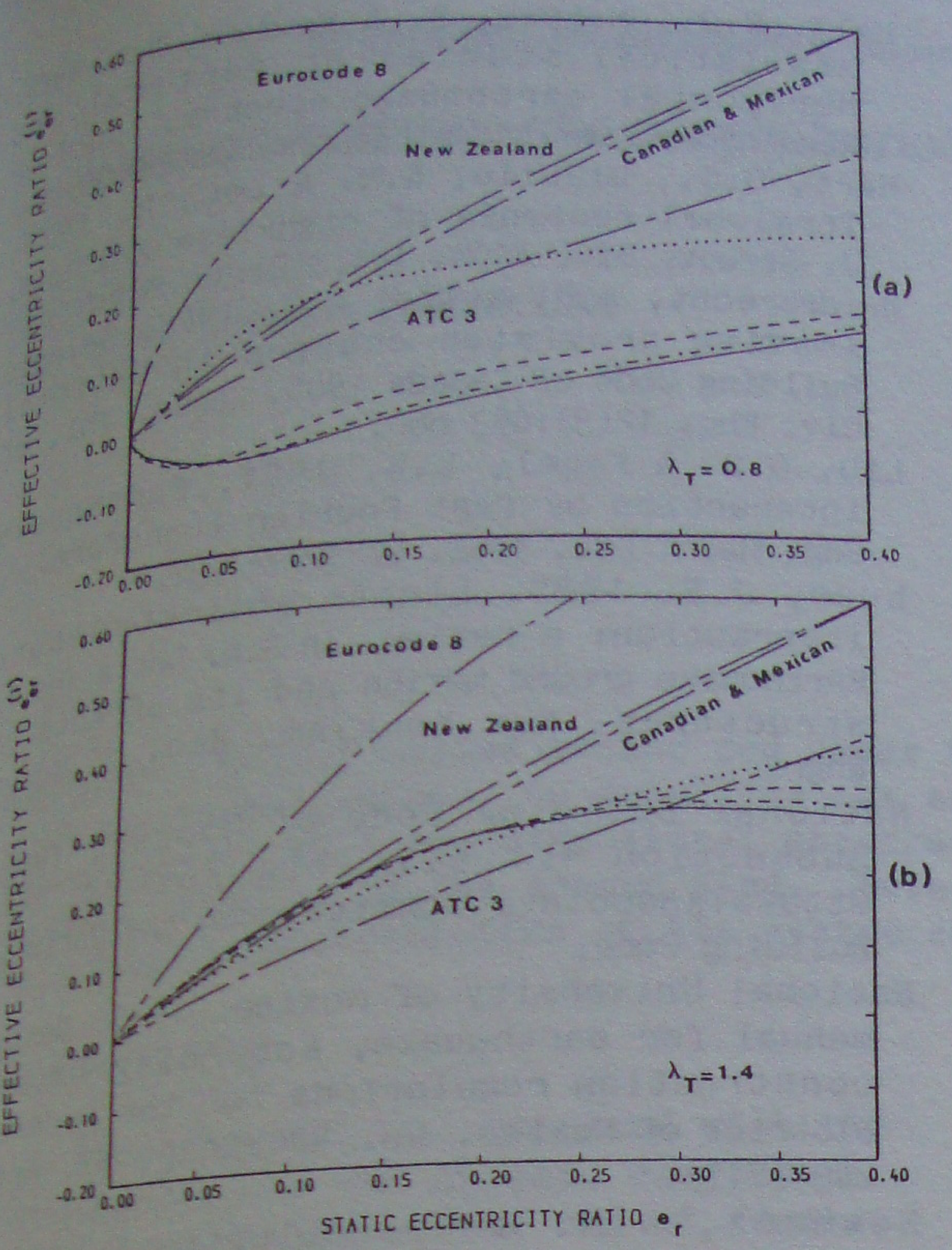

Fig.6. Variation of the effective eccentricity ratio with e_r for the flexibly based building model, λ_T=0.8 (a), λ_T=1.4 (b), and comparison with the dynamic eccentricity provisions of major building codes (—— — —); α=3 (......), α=6 (— — — —), α=10 (—·—·—), α=∞ (————)

Dempsey 1982, Chandler & Hutchinson 1987) are qualitatively unchanged when the interaction model is analysed, though detailed differences have been identified. Most codes neglect the effect of interaction in specifying equivalent lateral design forces, although the ATC3 code makes recommendations for their inclusion by means of empirical factors for reducing the calculated value of base shear. No code allowance is made for changes to the design torque provisions (Table 2) to account for soil-structure interaction.

As indicated in section 4, the results presented in Figures 4,5 and 6 show clearly that asymmetric structures supported on flexible foundations may exhibit greater coupling of lateral and torsional floor responses than the equivalent rigidly based buildings. In particular, the amplification of torsional response for λ_T < 0.85 (that is, for buildings weak in torsion) is substantially increased by interaction effects (Figure 4(b)), and leads to increased effective eccentricity requirements in this range (Figures 5,6(a)). Hence, in these cases it is non-conservative to implement a design for torsional coupling effects on the basis of a rigid base assumption. For λ_T > 1.2 interaction has negligible effect on the effective eccentricity requirement (Figures 5,6(b)) and for practical design purposes, therefore, a rigid base analysis is in this case justified.

It is apparent from these results that within certain ranges of the controlling parameters e_r, λ_T and α, an increased allowance for torsional coupling should be made due to the amplification of the combined lateral-torsional edge response resulting from soil-structure interaction. Further research is recommended in order to assess these effects in more detail, leading to specific recommendations for incorporation into earthquake building codes.

6 CONCLUSIONS

Current torsional design recommendations in building codes are based largely on the results of simplified studies using idealised response spectra. Previous parametric studies (Tsicnias & Hutchinson 1981, Tso & Dempsey 1982, Chandler & Hutchinson 1987) have highlighted the inadequacies of these provisions in comparison with dynamic analysis, particularly in their allowance for increased lateral forces in the peripheral structural members of the building. In these studies, the ranges of the key parameters e_r and λ_T for which greatest deficiency exists in the current recommendations have been clearly identified, and various alternative provisions have been proposed (Tso & Dempsey 1982, Chandler & Hutchinson 1986). As a result of this debate, Tso (1983) made a series of recommendations to improve some of the shortcomings of the seismic torsional provisions in the National Building Code of Canada 1980. These have subsequently been adopted, together with other changes to the seismic loading provisions, in the revised National Building Code of Canada 1985 (see Heidebrecht & Tso 1985). The present study has extended previous results to incorporate an evaluation of the effects of soil-structure interaction on torsional coupling, with the following conclusions:

(1) Torsional coupling effects, as exhibited by the amplification or attenuation of the individual torsional and translational response components of the structure, are not qualitatively affected by changes in the flexibility of the foundation medium.

(2) Interaction has negligible effect on lateral-torsional coupling in structures with uncoupled frequency ratios (λ_T) greater than 1.2, and hence conservative

design of these buildings can be achieved with the assumption of a rigid foundation.

(3) For soft and medium stiff soils, significant increases in coupled lateral-torsional response have been observed for structures having uncoupled frequency ratios (λ_T) less than 1.2, that is buildings which are relatively weak in torsion.

(4) As a result of these increases, building code provisions which have been based on the response of structures supported on rigid foundations need further examination to assess the need for revision, in specific circumstances, to account for increased torsional effects resulting from soil-structure interaction.

(5) The authors recommend that further research be carried out to assess these effects in more detail, resulting in specific design proposals in a form suitable for adoption by building codes.

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