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Structure-foundation interaction in the earthquake response of torsionally asymmetric buildings

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A detailed parametric study is made of torsional coupling in earthquake-excited asymmetric buildings including the effects of soil-structure interaction. The objectives are to assess the changes in the lateral-torsional earthquake response of such structures in relation to rigidly based buildings, and to make appropriate comparisons with current codified seismic provisions for coupling effects by empirical design procedures based on rigid base analysis of asymmetric buildings. The response of an idealised elastic structure-foundation model has been evaluated when subjected to free-field earthquake input motions represented by design spectra. These spectra account for localised site effects over the range of soil foundation properties for which significant interaction effects are expected. It has been found that for short and long period structures, torsional provisions based on analyses of rigidly based buildings also suffice for those supported on flexible foundations. However, for intermediate height buildings with moderate or large eccentricity, increased torsional loadings must be specified to account conservatively for the accentuation of the combined lateral-torsional response for such structures when supported on moderately flexible and very flexible foundations. Some recommendations for implementing such provisions in building codes have been outlined.

Key words: torsional coupling, soil-structure interaction, earthquake building codes.

NOMENCLATURE

$A(t)$, A	Earthquake acceleration time-history and peak value (g)
a_p, a_s, b	Building code coefficient used in defining e_{d1}, e_{d2}, e_a
c	Coefficient of viscous damping for the structure
CM	Centre of mass of floor diaphragm
CR	Centre of resistance for loading in the direction of applied earthquake
D	Peak earthquake ground displacement (m)
DAF	Dynamic Amplification Factor for spectral acceleration
e	Structural (static) eccentricity
e_1	Supplementary eccentricity (Eurocode 8)
e_a	Accidental eccentricity stipulated by building codes
e_{bi}	Design eccentricity at floor i
e_{bp}, e_{bs}	Primary and secondary design eccentricities
e_{d1}, e_{d2}	Dynamic eccentriticies stipulated by building codes
e_e	Effective eccentricity for design of members at the flexible edge of a building
F_o	design base shear
F_i	Lateral force at floor i
h	Effective height to centre of mass of multi-storey building
h_i	Height of ith floor above the base
h_s, m_s	Storey height and mass in multi-storey building
H	Total building height ($=Nh_s$)

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K_v, K_θ	Lateral and torsional stiffnesses of the structure, resepectively
m, m_o	Total building mass ($=Nm_s$) and foundation mass, respectively
N	Number of building storeys
r, r_o	Radius of floor and foundation (base) discs, respectively
RMSA	Root Mean Square Acceleration (g)
S, S_{def}	Recorded earthquake duration and strong-motion duration (sec)
T_{bi}	Design torque acting at floor i
T_v	Fundamental lateral period of uncoupled building ($=2\pi/\omega_v$)
v	Floor translation due to structural deformation
v_o	Foundation translation due to soil flexibility
V	Peak earthquake ground velocity (m/s)
V_s	Soil shear wave velocity (m/s or ft/s)
W, w_i	Building weight and storey weight at floor i, respectively
W_o	Weight of structural foundation
X, Y	Plan dimensions of building, parallel and perpendicular to earthquake direction, respectively
α	Structure-foundation stiffness ratio
δ_h	Building height ratio (h/r)
ζ_n	Critical damping ratio in mode n
ϕ_o	Foundation rocking due to soil flexibility
v	Poisson's ratio of soil
ρ	Soil mass density
ω_v, ω_θ	Uncoupled lateral and torsional frequencies, respectively
Ω	Uncoupled torsional to translational frequency ratio ($=\omega_\theta/\omega_v$)

INTERACTION EFFECTS IN EARTHQUAKE ANALYSIS AND DESIGN

Because of deformations within the soil immediately beneath a structure, the motion of the base of a building may differ from the motion of the ground at some distance away. Such a difference is indicative of soil-structure interaction. Soil-structure interaction primarily affects the horizontal and vertical motions of the base of the structure, but also may give rise to significant rocking.[1] Furthermore, if the input motion is non-uniform it may cause torsional movements of the base.[2]

Soil structure interaction effects are different from so-called site effects.[3,4] The site effects refer to the fact that the characteristics of the free-field ground motion induced by a dynamic event at a given site depend on the properties and geological features of the site. Interaction effects on the other hand refer to the fact that the dynamic response of a structure built on that site depends not only on the characteristics of the free-field ground motion but also on the inter-relationship of the dynamic structural properties and those of the underlying soil deposits.

The importance of soil-structure interaction has long been recognised, and a number of techniques have been formulated for analysing its effects on earthquake building response. In a commonly employed idealisation,[5–10] the effects of the foundation medium on the structural response are simulated by a series of springs and dashpots representing a theoretical half-space surrounding the base of the structure. Alternatively, the soil-structure system can be represented by a two or three-dimensional finite-element model.[11,12]

Balendra *et al.*[8] considered the seismic response of asymmetric multi-storey buildings on linear elastic foundations. Modelling the impedences at the soil-structure interface by means of frequency-independent coefficients, a method was developed which enabled the use of standard response spectrum techniques to evaluate the coupled lateral-torsional structural responses. This involved the determination of approximate normal modes and modal damping for torsionally coupled buildings on elastic foundations, a method first introduced by Tsai[10] who analysed similar but symmetric (uncoupled) flexibly based buildings. In the present paper, the interaction impedences are represented by frequency independent spring-dashpot coefficients, employing similar techniques to Balendra *et al.*[8] for determining the approximate normal modes and modal damping of parametrically defined torsionally coupled foundation-structure systems.

In many studies observing the parametric behaviour of building-foundation interaction, the structural system has been modelled as a simple single-storey building frame (Fig. 1). More generally, the structural model represents a multi-storey building that responds approximately as a single degree of freedom system in its fixed base condition.[7,13] The foundation degrees of freedom have then been included in analysing the overall dynamic response of the building-foundation system. This method of structural idealisation is particularly advantageous in modal analysis, provided the earthquake response is dominated by the fundamental mode of vibration.[5]

The ATC3[4] provisions for soil-structure interaction in earthquake-resistant building design are based on an idealised single-storey building-foundation system. A typical configuration is shown in Fig. 1, where the linear structure of mass m, with lateral stiffness K_v and coefficient of viscous damping c, is assumed to be supported by a foundation of mass m_o at the surface of a homogeneous, elastic half-space. The rigid foundation mat is idealised as a circular plate of radius r_o and negligible thickness, bonded to the supporting medium. The system exhibits three degrees of freedom; these are the displacement v_o and rocking ϕ_o of the foundation, and the floor displacement v relative to the base of the structure.

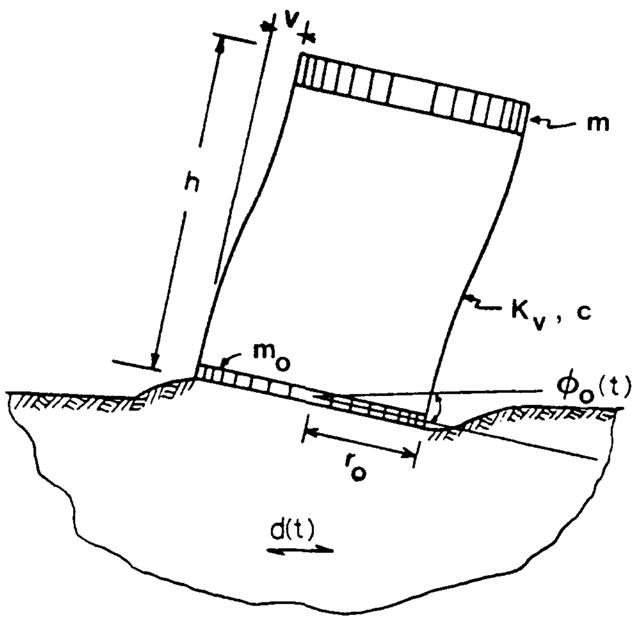

Fig. 1. Simple building-foundation system considered by ATC3 recommendations.

Evaluating the foundation stiffness and damping coefficients from frequency independent expressions leads to empirical expressions for the effective fundamental period, stiffness and damping. Provided that soil-structure interaction may be considered to affect significantly the contribution of the fundamental mode only, and that this effect can be expressed by changes in the fundamental natural period and the associated damping/stiffness of the system, the base shear of the interacting building is then determined by either an equivalent lateral force procedure or a simplified modal analysis.

The effects of interaction included in the design of buildings in accordance with the ATC3 guidelines result in a reduced value of base shear compared with the rigid base system, due to the increased damping and the lengthening of the fundamental structural period. ATC3 recommends furthermore that the reduction of shear should be restricted to a maximum of 30%, to ensure conservative estimates of earthquake forces for structural design.

INFLUENCE OF TORSIONAL COUPLING ON SEISMIC BUILDING PERFORMANCE

Significant advance has been made in the recent development and use of analytical models for determining soil-structure interaction effects in the earthquake response of torsionally coupled (asymmetric) buildings.[8,9,14–18] Torsional coupling is a dynamic interaction of lateral and rotational vibration modes which in some cases causes severe amplification of earthquake forces in peripheral structural members. This effect is generally underestimated by the torsional provisions of major building codes.[19,20]

The Mexican earthquake of September 1985 resulted in the partial or total collapse of a large number of buildings in Mexico City as a result of torsional effects. Most of these buildings were founded on soft to firm lake bed clays,[21] and as a result it has been suggested[22] that the 1977 Mexican earthquake code[23] design recommendations for torsional effects in asymmetric buildings do not include a sufficient safety margin.

The torsional provisions recommended by current major seismic building codes are based on analysis of the response of buildings founded on rigid (non-interacting) foundations. Whilst the inadequacies of the provisions have been identified and studied in detail by previous research,[19,20] the extent to which torsional responses are affected by soil-structure interaction has received relatively minor attention. Recently, Sivakumaran[17] studied the response of a 5-storey mono-symmetric building founded on a flexible foundation, to the El Centro 1940 and Mexico City 1985 earthquakes. The results showed that very soft soil conditions simulated in the analytical model (typical of Mexico City lake bed clay deposits) affect significantly the response of the building, with reductions of up to a third in the storey shears and up to half in the storey torques, compared with the same model supported on a rigid foundation. However, the limitation of this study to a single structure-foundation model and only two earthquake motions make the results unsuitable for drawing general conclusions on interaction effects in asymmetric buildings.

Another recent study by Bazan & Bielak[18] has emphasized the importance of interaction in determining design forces for asymmetric structures, and proposes a modified static method for its incorporation into the design process. The proposal accounts for the influence of interaction on the prescribed target ductility for inelastic shear response of a structure, prior to inclusion of torsional effects using a supplementary static approach as in current code procedures.[23] This study was again limited to a single building-foundation model, and employed only the El Centro earthquake record in the calculations. The results presented in the present paper are intended to clarify the influence of interaction effects in asymmetric structures by studying the earthquake response of a range of simple asymmetric building-foundation systems. Firstly, however, a brief review is made of the significant results from parametric studies of rigidly-based buildings, which give a basis for quantifying changes brought about by foundation interaction.

Figure 2 shows the plan of a typical floor of a torsionally coupled building. The centre of mass *CM* and the centre of resistance *CR* are separated by the structural stiffness eccentricity *e*, assumed perpendicular to the incident earthquake direction. Only mono-symmetric structures are considered by building codes in accounting for torsional coupling effects due to uni-directional earth-

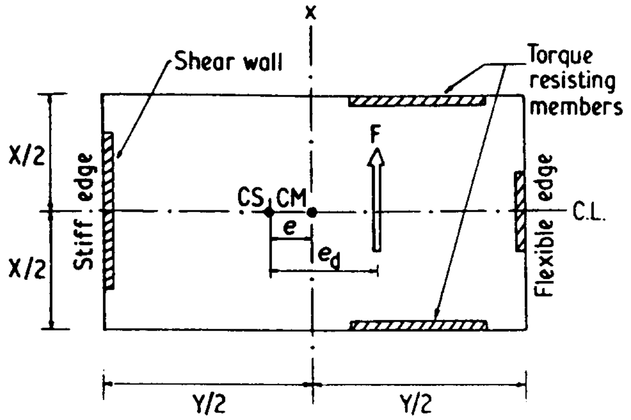

Fig. 2. Floor plan of a typical mono-symmetric building as considered by building codes.

quake ground motion, since it has been shown[24] that the lateral response of systems with bi-directional eccentricity is generally smaller. The allowance for torsion is provided at each floor level by means of a static torsional moment which acts simultaneously with the lateral shear force F_i:

$$F_i = F_o\left[\frac{w_i h_i}{\sum_{i=1}^{N} w_i h_i}\right] \tag{1a}$$

$$T_{bi} = \left[F_o - \sum_{n=1}^{i} F_n\right] e_{bi} \tag{1b}$$

= interstorey shear at floor i multiplied by design eccentricity at floor i

The total lateral force F_o (the base shear) is evaluated as a proportion of the total effective building weight, W. The coefficient employed takes into account the dynamic properties of the structure, the soil conditions at the site, the seismicity of the region and the importance of the building. The base shear is distributed according to eqn (1a) over the height of the structure and the design eccentricity e_{bi} in eqn (1b) is taken as either of the expressions given in eqns (2a) and (2b); the expression adopted is that which provides the more unfavourable stresses on the lateral load resisting member being considered:

$$e_{bp} = e_{d1} + e_a = a_p e + bY \tag{2a}$$

$$e_{bs} = e_{d2} - e_a = a_s e - bY \tag{2b}$$

where e_{dj} (j = 1,2) and e_a are termed respectively the dynamic and accidental eccentricity, stipulated by the various building codes in terms of empirical coefficients a_p, a_s and b, the static eccentricity e and the building plan dimension Y perpendicular to the earthquake direction. Equations (2a), (2b) are referred to as the primary and secondary design eccentricity provisions and in general apply to the design of vertical members situated at the flexible and stiff edges of the building, respectively (Fig. 2). Conditions for determining the least favourable of the eccentricity provisions for these two cases have been presented in Ref. 25; the current paper concentrates on the primary design eccentricity requirements, affecting members situated furthest from CR (Fig. 2).

Table 1. Codified provisions for primary design eccentricity

Building Codes	a_p	b
ATC3	1·0	0·05
Canada	1·5	0·10
Mexico	1·5	0·10
New Zealand	1·0	0·10
Eurocode EC8	$1{\cdot}0 + e_1/e$	0·05

There are significant differences in the values assigned by building codes to the dynamic and accidental eccentricity coefficients. The primary torsional design provisions of the five major seismic codes considered in this paper are listed in Table 1 and compared in Fig. 3. The supplementary eccentricity e_1 specified by Eurocode 8 is related to the building plan dimensions X, Y, and the ratio K_θ/K_v of torsional and translational stiffnesses of the structure.[39] Hence Eurocode 8 includes by the latter parameter the effect of uncoupled frequency ratio Ω on the coupled dynamic response. The Eurocode 8 provisions shown in Fig. 3 apply to the case Ω = 1, for which torsional coupling has its greatest effect.[19,24]

In Ref. 26, Dempsey and Tso suggested that since the evaluation of torsional coupling effects depends on the combined influence of the dynamic lateral shear and torque acting on a building, the dynamic eccentricity approach presently advocated by all the major building codes should be replaced by an effective eccentricity concept. In this case, instead of matching the design torsional moment with results from dynamic analysis (independent of lateral shear forces), an effective eccentricity is defined as the distance from CR that the static design shear should act in order to give rise to the same peak edge (or member) displacements as those obtained by two- or three-dimensional dynamic analysis including

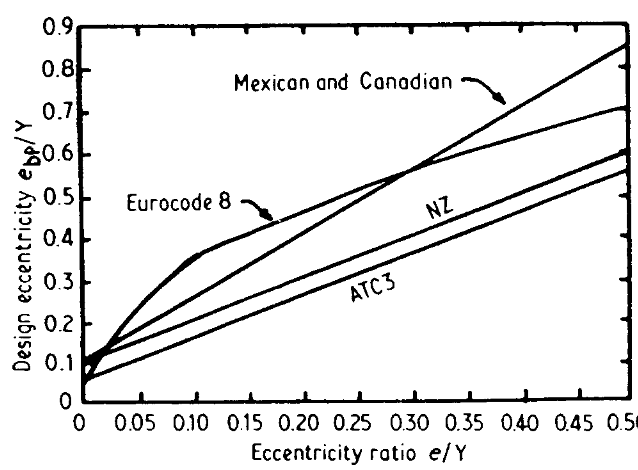

Fig. 3. Comparison of primary design eccentricity for different seismic codes.

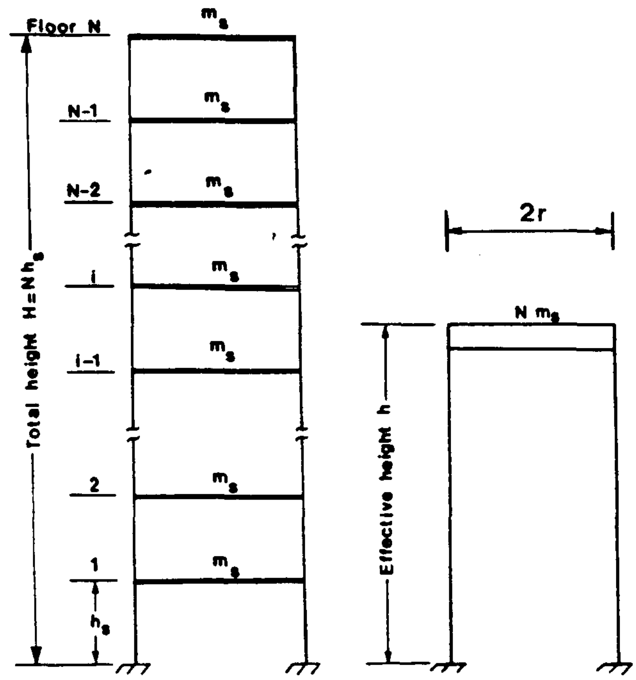

Fig. 4. Idealisations of multi-storey building model and its single storey equivalent.

torsional coupling effects. Changes in the torsional provisions of the National Building Codes of Canada (1985)[27] were implemented following further studies of the effective eccentricity design concept[28] using idealised response spectra to represent the earthquake input. A further detailed study has recently been carried out,[29] based on a statistical analysis of time-history responses of torsionally coupled structures to a series of strong-motion European earthquake records.

Envelopes of effective eccentricity for use in design have been derived in a related study[30] by parametric analysis of asymmetric structures in response to design spectra generated from a statistical survey of 20 earthquake time histories recorded on rock or stiff soil sites in the USA and Europe. The fundamental translational building period, T_v, was varied in the range of 0·1–2·0 sec by altering the effective height h of the idealised single-storey structure (Fig. 4). This represented the effective location of the total structure mass m, assuming the building vibrates in its fundamental sway mode.[10] The radius r represents an equivalent floor and foundation dimension for a circular building plan, a geometry which facilitates extension of the study to include soil-structure interaction effects.

The results shown in Fig. 5 indicate that torsional coupling is particularly significant for very short-period buildings ($T_v \leqslant 0{\cdot}2$ sec) and for buildings with small eccentricity ($e/r < 0.2$). The variation of effective eccentricity with the static or structural eccentricity is non-linear, especially for buildings with low eccentricity, which is in contrast to the majority of code provisions which therefore underestimate torsional coupling effects

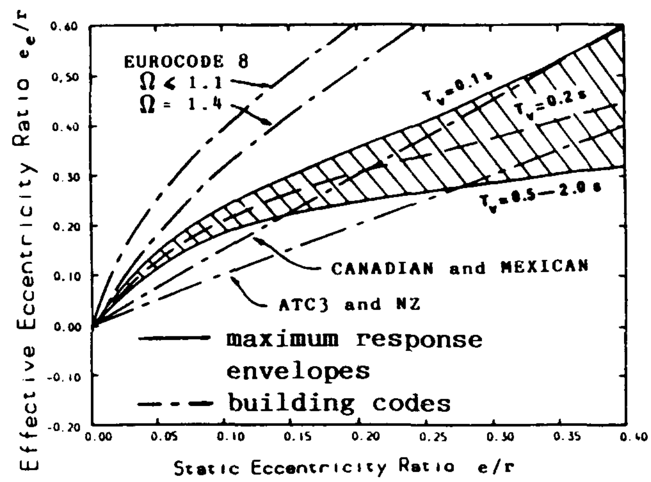

Fig. 5. Variation of envelopes of peak effective eccentricity with static eccentricity, and comparison with building codes.

in this range. The Eurocode 8 provisions are shown to be over-conservative throughout, even when based on buildings which are relatively stiff in torsion ($\Omega = 1{\cdot}4$). The analysis was carried out for buildings of various realistic uncoupled frequency ratios Ω in the range 0·6–2·0 and the results shown in Fig. 5 represent the worst-case response, which was found[30] in general to occur for $\Omega = 1{\cdot}1$ for small static eccentricity ratios e/r, and at $\Omega = 1{\cdot}2$ for $e/r > 0{\cdot}2$.

STRUCTURE-FOUNDATION MODEL AND ANALYTICAL PROCEDURES

Model definition and characteristic parameters

A five degrees of freedom structure-foundation model has been employed to represent an idealised torsionally coupled building resting on a compliant soil foundation. The dynamic response of the system to uni-directional free-field ground motion is represented by the translation and rotation of the floor about its centre of resistance, together with the translation, rotation and rocking at the structure-foundation interface.[16] Hence the model is similar to that defined in the ATC3 recommendations (Fig. 1), with the addition of the torsional degrees of freedom at base and floor levels.

To facilitate the analysis of such models in the time domain, previous studies[8,31] have derived simplified foundation impedences which replace the frequency dependent impedences by approximate frequency independent coefficients. These have been derived on the basis that they are functions of foundation and soil properties only, and hence do not include the influence of the superimposed structure. Furthermore, they are limited to a specific range of excitation frequency. In the present study the improved approximation of frequency independent impedences introduced by Ghaffar-Zadeh & Chapel[13] has been adopted, incorporating parameters defining the first-mode response of the structure, as well as the foundation half-space.

The response spectrum technique has been employed by adopting an approximate method similar to that proposed by Tsai,[10] and further developed by Balendra, Tat & Lee.[8] In this method, a set of approximate normal modes for the structure-foundation system are obtained. These diagonalise the inertia and stiffness matrices in the governing equations of motion, but the transformed damping matrix is non-diagonal. Hence, an equivalent diagonal matrix is constructed by an iterative process based on matching the approximate normal mode solutions to exact steady-state solutions, at each modal frequency. This procedure ensures that the orthogonality conditions are satisfied for the inertia, stiffness and damping matrices of the system. The maximum structural responses to horizontal ground motion represented by appropriate elastic response or design spectra are then obtained using the derived approximate normal modes and damping values. Full details of these analytical procedures on which the results of the present study are based have been given in Refs 16 and 30.

In carrying out a parametric study of torsional coupling in interacting building systems, the following parameters are required in addition to those conventionally employed to define rigidly based asymmetric building models (namely the uncoupled frequency ratio Ω, eccentricity ratio e/r, modal damping ζ_n and uncoupled natural period T_v). Firstly, the foundation/structure mass ratio m_o/m and the height ratio $\delta_h = h/r$ (Fig. 4) are required to extend the set of structural parameters. Secondly, the Poisson's ratio ν, the structural/soil density ratio $m/\pi r^2 h\rho$ (ρ = soil mass density) and the shear wave velocity ratio $\alpha = V_s/H(\omega_v/2\pi)$ are employed as parameters defining the soil and soil-structure interaction properties.[16]

In this study, the mass ratio $m_o/m = 0{\cdot}3$, the Poisson's ratio $\nu = 1/3$ and the structure/soil density ratio $= 0{\cdot}14$. These values are representative of actual building/foundation systems and moreover their variation has not been found to produce significant changes to the response trends.[9,15] The structural damping in the fundamental mode of vibration (neglecting interaction effects) has been taken as 5 per cent of critical throughout. Hence the key parameters controlling the dynamic structural response including the effects of foundation interaction are e/r, Ω, δ_h and α. An empirical (approximately linear) relationship has been assumed between the height ratio δ_h and the period T_v of the equivalent uncoupled rigidly based building. Hence in this study, T_v has been taken as 0·2, 0·5 and 2·0 sec (relating approximately to 2, 5 and 20 storey buildings, respectively) and the corresponding height ratios δ_h are, approximately, 0·5, 1·0 and 4·0.

The parameter α which is regarded as a measure of soil-structure interaction[1] has been assigned the values of 3, 6, 10 and infinity. The normalisation of the shear velocity V_s with respect to the product Hf_v (f_v is the uncoupled frequency, $1/T_v$) is used since for many practical types of building structure, Hf_v is approximately constant. For a representative storey height of 3 m, the product Hf_v is approximately equal to 30 m/s (100 ft/s). Shear wave velocities in the order of 300–500 ft/s correspond to very soft soils whereas the range 3000–5000 ft/s corresponds to very stiff soils or rock.[1] These velocities represent at the lower end the range of V_s for which soil-structure interaction is significant. In this study, the values $\alpha = 3$, 6, 10 correspond respectively to loose, coarse-grained granular materials and soft clay/silt soils ($V_s \simeq 300$ ft/s), medium sands and medium stiff clays ($V_s \simeq 600$ ft/s), and stiff sands or very stiff overconsolidated clays ($V_s \simeq 1000$ ft/s). The value $\alpha = \infty$ represents a rock foundation ($V_s > 5000$ ft/s) for which negligible interaction is expected. Systems with smaller values of α therefore correspond to greater foundation interaction. Although some differences in results have been observed between $\alpha = 10$ and $\alpha = \infty$, these are not large; consequently values of α between 10 and infinity have not been considered.

DERIVATION OF IDEALISED DESIGN SPECTRA

A detailed study has been made[30] of a selection of 31 strong-motion earthquake records generated by 16 major earthquakes, leading to the derivation of design spectra corresponding to a range of soil foundation media. These spectra are appropriate for analysing the earthquake response of systems which exhibit interaction effects. The approach is similar to that of Newmark & Hall,[32] who proposed a set of design spectral shapes following a detailed statistical study of strong motion earthquakes in the Western United States. In the present study, earthquake records have been taken from the Western USA (18 records), Eastern and Southern Europe (7), North Africa (2) and the Middle East (4). By selecting accelerograms from several seismically active regions generated by earthquakes from different geological settings and source mechanisms, a wide variety of data has been analysed in the statistical development of response spectrum shapes. A summary of the key data relating to the earthquakes employed is given in Table 2.

The recorded duration S of the 31 accelerograms ranges between 3·0 and 78·6 seconds. Research by Trifunac & Brady[33] has indicated that for the same earthquake intensity the average duration of strong-motion acceleration on a 'soft' site is about twice that at a 'hard' site. The so-called strong motion duration (S_{def}) is defined as the time in which a predetermined percentage (here taken as 90%) of the total energy is input to a structure. The ratio S_{def}/S (Table 2) varies in the range 9–83%, indicative of the wide variety of earthquake motions included in the ensemble.

The root mean square acceleration (RMSA) is less

Table 2. Earthquake ground motion parameters

Number	Group	Earthquake	Comp.	Year	Recording Station	Duration S (sec)	Ratio S_{def}/S (%)	Max Ground Accel· A (g)	Ratio RMSA/A (%)	V/A m/s/g	V/A (in/s/g)	AD/V^2
1	A	Valfabrica, USA	N45E	1971	Palombina	3·05	9·0	0·0052	13·5	0·168	(6·61)	6·30
2	A	Patras, Greece	Trans	1974	Patras	3·94	75·9	0·2956	12·9	0·211	(8·31)	10·53
3	A	Rocca, Italy	T15NS	1972	Rocca	14·28	18·4	0·6100	10·5	0·218	(8·58)	22·33
4	A	Valfabrica, USA	N45E	1971	Valfabrica	3·11	20·2	0·0089	17·9	0·288	(11·34)	6·46
5	A	El Asnam, Algeria	Trans	1980	El Safsaf	4·54	77·2	0·0106	28·3	0·346	(13·62)	8·60
6	A	El Asnam, Algeria	Long	1980	El Safsaf	4·18	66·7	0·0063	30·2	0·538	(21·18)	5·88
7	A	Friuli, Italy	NS	1976	Tolemezzo	10·10	15·4	0·0996	14·8	0·399	(15·71)	10·99
8	A	Friuli, Italy	EW	1976	Tolemezzo	10·17	15·2	0·1591	13·7	0·503	(19·90)	11·10
9	A	Parkfield, USA	N05W	1966	Shandon 5	44·01	17·0	0·4038	8·5	0·553	(21·77)	9·95
10	A	Parkfield, USA	N85E	1966	Shandon 5	44·03	14·8	0·4667	12·3	0·550	(21·65)	6·38
11	A	Parkfield, USA	N54W	1966	San Luis	29·93	43·8	0·0160	17·5	0·496	(19·53)	18·99
12	A	Parkfield, USA	N36E	1966	San Luis	29·99	59·6	0·0163	22·1	0·724	(28·50)	20·01
13	A	San Fernando, USA	S16E	1971	Pacoima Dam	12·64	66·5	0·0256	16·3	0·500	(19·69)	15·23
14	A	(aftershock)	S74W	1971	Pacoima Dam	12·64	56·0	0·0315	15·2	0·785	(30·91)	13·64
15	A	Ferndale, USA	S44E	1951	Ferndale	55·87	27·0	0·1234	10·0	0·573	(22·56)	39·81
16	A	Ferndale, USA	S46W	1951	Ferndale	55·87	28·6	0·1196	11·1	0·727	(28·62)	26·12
17	A	San Jose, USA	N59E	1955	Bank of America	51·75	13·1	0·1279	10·0	0·362	(14·25)	125·20
18	A	San Jose, USA	N31W	1955	Bank of America	49·36	18·5	0·1194	11·4	0·967	(38·07)	7·24
19	A	San Fernando, USA	S74W	1971	Pacoima Dam	41·72	17·3	1·2506	9·6	0·465	(18·31)	6·28
20	A	San Fernando, USA	S16E	1971	Pacoima Dam	41·82	16·9	1·2410	9·7	0·886	(34·88)	6·55
21	B1	Gazli, USSR	EW	1976	Gazli	13·03	50·5	0·7313	21·5	0·789	(31·06)	11·04
22	B1	Gazli, USSR	NS	1976	Gazli	13·45	45·5	0·6217	24·8	1·082	(42·60)	3·72
23	B1	Leukas, Greece	Long	1973	Leukas	23·28	22·2	0·4172	11·6	1·022	(40·24)	2·42
24	B2	El Centro, USA	NS	1940	El Centro	29·16	82·5	0·3124	27·6	1·039	(40·90)	6·07
25	B2	Thessaloniki, Greece	A	1978	Thessaloniki	20·01	34·6	0·3160	19·2	1·084	(42·68)	3·70
26	B2	Tabas, Iran	Long	1978	Tabas	49·98	12·4	0·8836	9·2	1·192	(46·93)	7·89
27	B2	Tabas, Iran	Trans	1978	Tabas	49·98	33·2	0·9290	13·4	1·526	(60·47)	6·73
28	B3	Parkfield, USA	N65E	1966	Shandon 2	43·78	17·0	0·5080	10·4	1·514	(59·61)	3·16
29	B3	Kern County, USA	NS	1952	Hollywood	78·61	41·2	0·0626	16·1	1·213	(47·76)	14·78
30	B3	Kern County, USA	EW	1952	Hollywood	78·61	46·3	0·0442	22·6	2·320	(91·69)	11·78
31	B3	Romanian	NS	1977	–	22·01	49·9	0·2032	23·7	3·975	(156·50)	0·89

susceptible to the large fluctuations which often occur in other parameters such as the peak ground acceleration, A, defining the severity of earthquake ground motion. The ratio RMSA/A varies between 8 and 30% for the earthquakes employed in this study (Table 2). Nevertheless, the more readily available peak horizontal ground acceleration, A, remains the standard parameter used by engineers to characterise seismic hazard. The main factors affecting the spatial variation of A, together with the corresponding peak ground velocity V, and displacement, D, are local soil conditions, epicentral distance and earthquake magnitude.[34]

For scaling purposes, it is desirable to work with a unit horizontal ground acceleration, and the ratio of the peak ground velocity to peak ground acceleration, V/A, for estimating the ground velocity. In a similar manner, the non-dimensional ratio AD/V^2 containing all three peak ground motion parameters (which is at least in part a function of the focal distance of the earthquake[35]) is then employed to compute the peak ground displacement, D. Table 2 lists the non-dimensional ratio AD/V^2 for the 31 accelerogram records, the values ranging between 0·9 and 125·2. The range of the majority of results is in good agreement with those obtained by Newmark,[36] $1{\cdot}8 \leqslant AD/V^2 \leqslant 17{\cdot}6$, for the horizontal strong motion components of 14 Western US earthquakes. This ratio can be employed as an index for monitoring the shape, especially the breadth of the acceleration response spectrum.[30]

Research by Seed & Idriss[34] has shown that it is possible to determine values for the ratio V/A which are representative of different geological site conditions. Table 2 shows that the peak ground motion ratio V/A ranges from 0·17 to 3·98 m/s/g (6·6–156·5 in/s/g). The accelerogram records have been divided according to the ratio V/A into two groups, A and B. Group A records correspond to hard rock or very stiff soil sites, whilst group B records relate to intermediate and soft soil sites (i.e. soils that may exhibit significant interaction effects with superimposed structures). This results in the grouping shown in Table 2. The criteria adopted for grouping the accelerogram records were chosen to be compatible with previous research work[32] in which V/A ratios appropriate to different soil types have been defined. The 20 records in group A have V/A ratios in the range 6–27 in/s/g. Where appropriate, the mean value for two components of the same earthquake is taken as being representative of an individual recording site. For the 11 records in group B, $V/A > 36$ in/s/g.

Previous statistical analyses of recorded earthquake ground motions[32,37] have employed spectral data approximating to both normal and log-normal response distributions to develop suitable design spectra. The method of moments has been used[30] as an estimator for evaluating the sample mean (or first moment), median, standard deviation or square root of the variance (second moment), and finally skewness (third moment). From the findings regarding the third moments, together with an

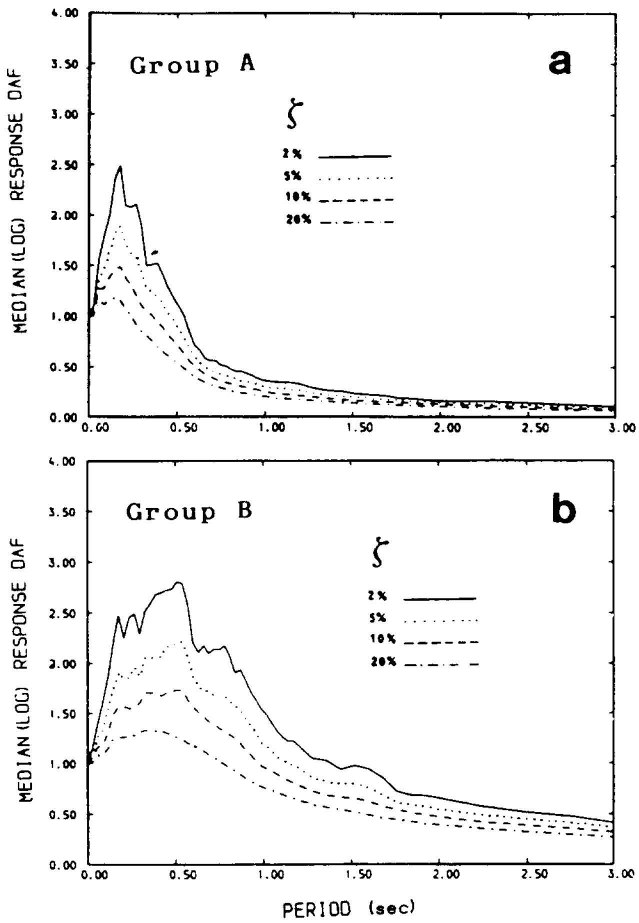

Fig. 6. Median response dynamic amplification factor (DAF) for various damping levels, assuming log-normal distribution amongst Group A records (a); and Group B records (b).

assessment of the differences between the computed values of sample mean and median, a log-normal distribution is considered to be the most appropriate distribution and has therefore been adopted in this study for evaluation of probabilistic acceleration dynamic spectral amplification (DAF) values. The method of moments can also be used to estimate spectral DAF values corresponding to a certain probability of exceedance.[38]

Figures 6(a), (b) show the median DAF spectral values for earthquake groups A and B, based on an assumed log-normal distribution of these data. For earthquake design purposes, spectra corresponding to 50% and 15.9% probability of exceedance (median and median plus one standard deviation responses, respectively) are the most commonly employed.[32] However for very critical structures such as nuclear power generation plants a probability of exceedence of 2.3% (median plus two standard deviation response) may be considered appropriate. Figures 7(a), (b) show the group A DAF spectrum shapes corresponding to probability of exceedance levels of 50% and 15·9%, respectively. Figures 8(a), (b) show the equivalent results for group B earthquakes.

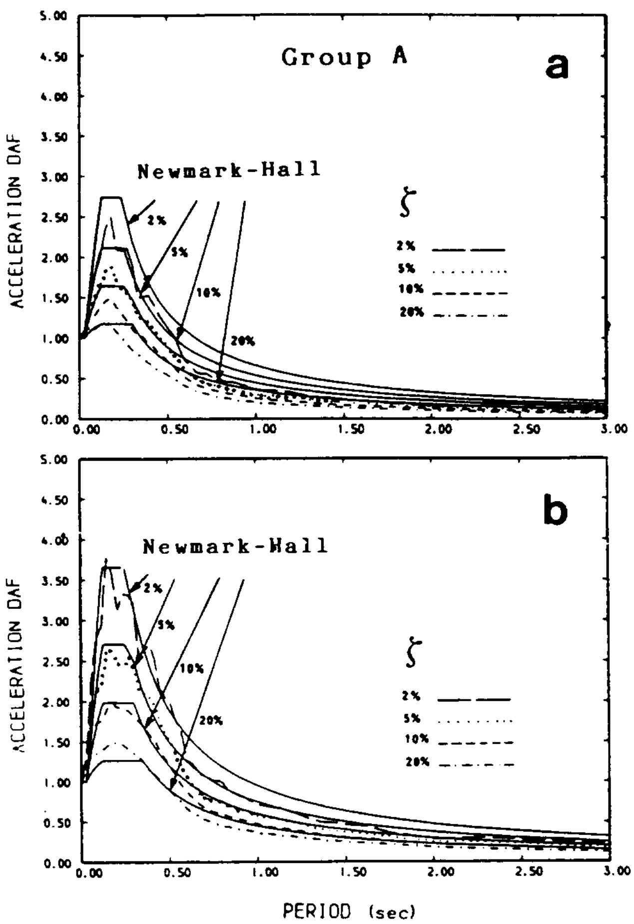

Fig. 7. Comparison of spectral dynamic amplification factor (DAF) for Group A records with the recommendations of Newmark and Hall corresponding to $A = 0{\cdot}3\,g$, $V/A = 20{\cdot}2$ in/sec/g and $AD/V^2 = 13{\cdot}3$: (a) 50% cumulative probability; (b) 84% cumulative probability.

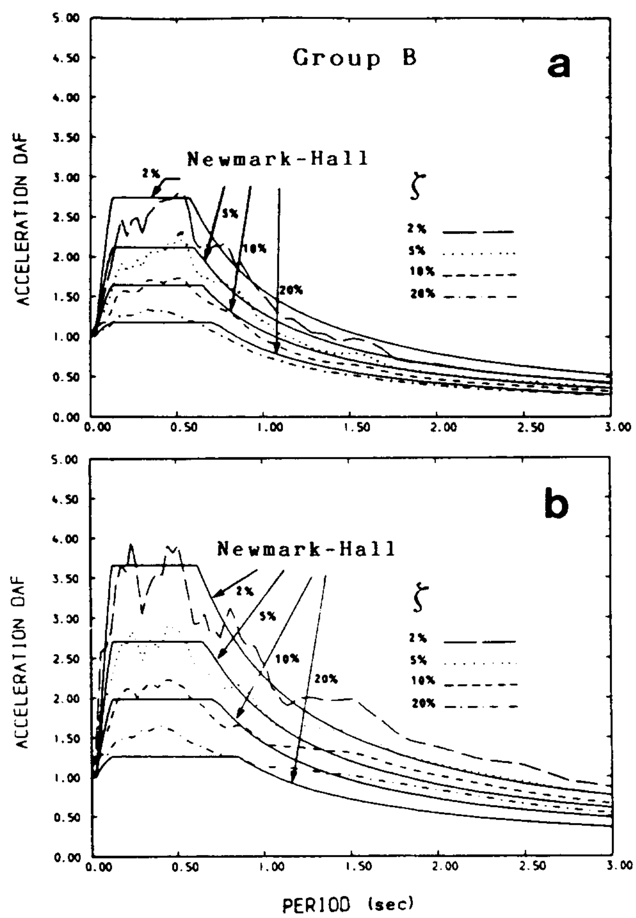

Fig. 8. Comparison of spectral dynamic amplification factor (DAF) for Group B records with the recommendations of Newmark and Hall corresponding to $A = 0{\cdot}3\,g$, $V/A = 50{\cdot}3$ in/sec/g and $AD/V^2 = 6{\cdot}5$: (a) 50% cumulative probability; (b) 84% cumulative probability.

The design spectra recommended by Newmark & Hall[32] have been summarised in Table 3. The equations represent the acceleration, velocity and displacement DAF's as a function of the damping ratio ζ, for two probabilites representing the median plus one standard deviation and median response levels. They further recommended the use of a V/A ratio of 48 in/s/g for a site with a 'competent' soil type, for example loose coarse-grained granular materials to medium sands and medium-stiff clays, and a V/A ratio of 36 in/s/g for a rock site. Further, they have suggested that an AD/V^2 ratio of 6 be used to ensure that an adequate range of earthquakes is taken into consideration.

From the results listed in Table 2, the mean V/A ratios for groups A and B are 0·51 m/s/g (20·2 in/s/g) and 1·28 m/s/g (50·3 in/s/g), respectively. In the latter case the Romanian record has been excluded because of its unusually high ratio which would produce an unrepresentative bias in the results. The corresponding mean AD/V^2 values are 13·3 for group A and 6·5 for group B records. Using the values for group A, appropriate spectral design curves have been obtained for rock/stiff soil sites (Figs 7(a), (b)). In both cases there is good comparison with the spectra derived in this study. The solid spectral design curves shown in Fig. 7 have therefore been employed in the subsequent analysis to represent the earthquake input motion for structures founded on 'rigid' foundations ($\alpha > 60$). Further comparison is made in Figs 8(a), (b) taking for group B the mean V/A and AD/V^2 ratios as 50·3 in/s/g and 6·5, respectively. The results suggest that Newmark and Hall's recommended amplification factors

Table 3. Spectrum dynamic amplification factors as a function of damping ratio, ζ (after Newmark & Hall[32])

Spectral Quantity	Percentile Probability, %	Equation
Acceleration	84.1	$4{\cdot}38 - 1{\cdot}04\ln\zeta$
Velocity		$3{\cdot}38 - 0{\cdot}67\ln\zeta$
Displacement		$2{\cdot}73 - 0{\cdot}45\ln\zeta$
Acceleration	50·0	$3{\cdot}21 - 0{\cdot}68\ln\zeta$
Velocity		$2{\cdot}31 - 0{\cdot}41\ln\zeta$
Displacement		$1{\cdot}82 - 0{\cdot}27\ln\zeta$

Table 4. Site soil conditions influencing the shape of response spectra

Type	Geological condition	Seed & Idriss[34] Average V/A ratio		Present study Average V/A ratio		Average AD/V^2
		m/s/g	(in/s/g)	m/s/g	(in/s/g)	
A	Rock site	0·55	(21·7)	0·51	(20·2)	13·3
B1	Stiff soil sites (less than 200ft deep)	1·10	(43·3)	0·98	(36·7)	5·7
B2	Deep cohesions soil sites (greater than 200ft deep)	1·35	(53·3)	1·27	(50·0)	6·1
B3	Site underlain by soft to medium stiff clay deposits	> 1·35	(> 53·3)	1·68	(66·3)	7·6

may again be used as a means of representing the earthquake input motion, in this case for soft to intermediate types of soil.

In detailed studies of earthquake motions carried out by Seed *et al.*[6,34] four different soil conditions were identified which influence the shape of the response spectrum. These site geological conditions, together with the corresponding average V/A ratios are shown in Table 4.

The normalised spectral curves recommended by Seed and Idriss[34] for use in building codes are shown in Fig. 9(a). These spectra account for the different acceleration levels for rock and other site conditions (Fig. 9(b)), where the spectral curves recommended for use in building codes, Fig. 9(a), have been appropriately normalised to the maximum ground acceleration on rock.

A similar approach has been adopted for the analysis of the accelerogram data used in this study. This has enabled relative spectral curves to be produced, similar to the ones shown in Fig. 9(a). For this purpose, the group B data has been further sub-divided into groups B1, B2 and B3, as in Tables 2 and 4. The mean V/A ratios for groups A, B1–B3 match closely those of Seed & Idriss.[34] By taking the V/A and AD/V^2 values from Table 4, together with the dynamic amplification for 84·1% probability level (Table 3) and 5% of critical damping, the acceleration design spectra shown in Fig. 10 are obtained. These curves assume a maximum ground acceleration of 0·3 g for a rock site. The curves presented in Fig. 10, and similar ones obtained for other damping values constitute the design spectra representing the input motion as employed in this study.

EARTHQUAKE RESPONSE OF TORSIONALLY COUPLED BUILDINGS WITH FOUNDATION INTERACTION

A parametric study of soil-structure interaction effects exhibited by a single-storey 5DOF torsionally coupled building model has been reported in Ref. 16. The design spectra corresponding to earthquake groups A and B derived in the previous section (see Figs 7 and 8) were employed in this study to represent the input motion for foundations based on rock (A) or soft to stiff soils (B). The effects of structural eccentricity on systems exhibiting foundation interaction were assessed by evaluating

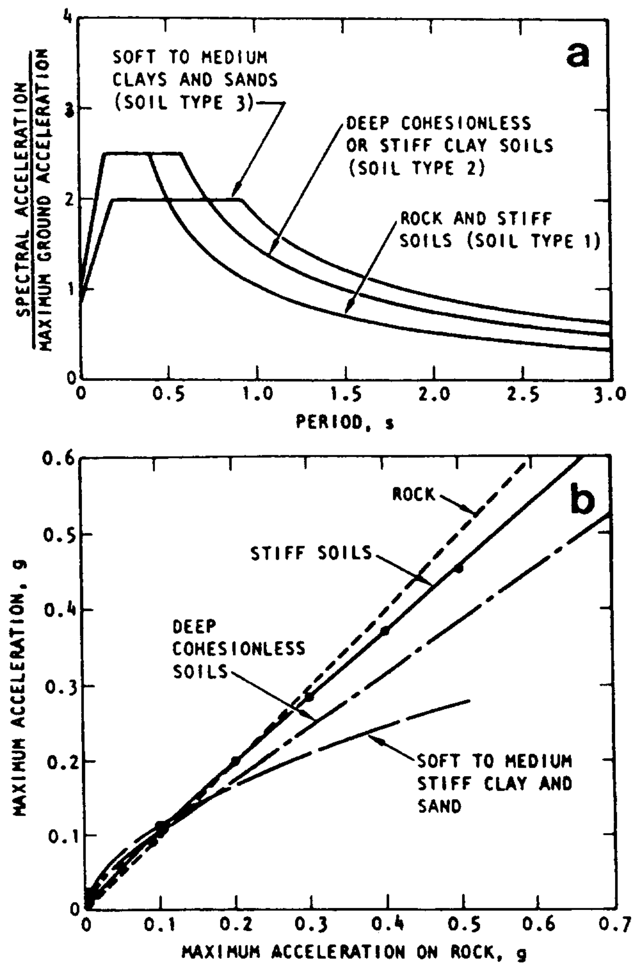

Fig. 9. (a) ATC3 recommended spectral design curves normalised to peak effective acceleration; (b) approximate relationships between maximum accelerations on rock and other local site conditions; after Seed and Idriss.

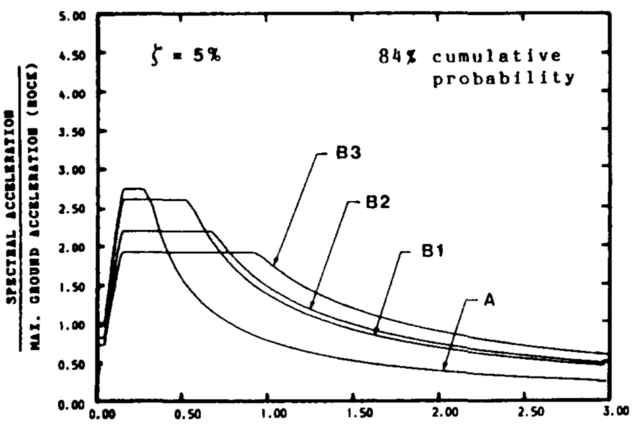

Fig. 10. Spectral design curves normalised to maximum ground acceleration on rock, derived for various site conditions (see Table 4).

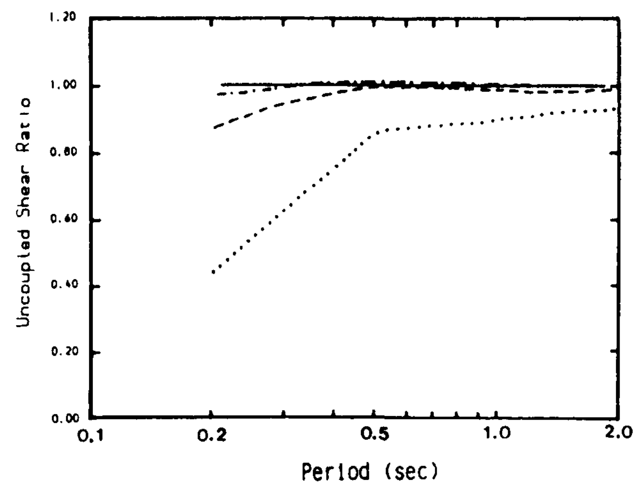

Fig. 11. Reduction of shear for symmetric systems due to interaction effect; $\alpha = 3$ (····), $\alpha = 6$ (– – –), $\alpha = 10$ (–·–·–·–), $\alpha = \infty$ (——).

the key response parameters in relation to the design shear F_o associated with an equivalent interacting uncoupled model with 3DOF, that is neglecting the torsional degrees of freedom of the base and floor diaphragms (Fig. 1). Thus the results presented in Ref. 16 demonstrate the influence of torsional coupling, by comparison of the response of asymmetric interacting systems with that of corresponding symmetric interacting systems. The analyses throughout included both soil-structure interaction and site effects. The latter were accounted for by appropriate selection of design spectra.

It was concluded[16] that the deficiencies of building code torsional provisions identified for rigidly based structures[19,20] are not qualitatively changed when an interacting model is analysed. Apart from ATC3[4] (see earlier), all the major codes neglect the direct effects of interaction in specifying equivalent static design forces, although a site factor is usually included to allow for the amplification effect of soft soils.[21,22] Neglecting interaction effects has been shown to be a conservative assumption with regard to the lateral forces, but results have shown that in some critical cases the torsional forces induced by torsional coupling in asymmetric structures supported on flexible elastic foundations are significantly greater than for equivalent rigidly based buildings. In particular, the amplification of torsional response for $0{\cdot}75 < \Omega < 1{\cdot}0$, a range of uncoupled frequency ratio associated with buildings which are weak in torsion, is substantially increased by interaction effects. This leads to increased effective eccentricity requirements to account for the contributions from both lateral shear and torsional moment acting simultaneously. Hence, in these cases it has been found that designs for torsional effects based on a rigid base assumption are significantly non-conservative. In contrast, for $\Omega > 1{\cdot}4$ interaction has negligible effect on the effective eccentricity requirements, and for practical design purposes therefore a rigid base analysis is justified in this case. The present study examines the alternative case in which the design is based on an asymmetric structure on a flexible foundation, but which neglects soil-structure interaction in determining the lateral design shear. Nevertheless the design incorporates site effects, in accordance with the current engineering practice recommended by seismic codes for design of symmetric (uncoupled) buildings. In this approach, shear reduction due to interaction is neglected since it has been established (Fig. 11) that the rigidly based design shear is somewhat conservative in comparison with the same building based on a flexible foundation and exhibiting soil-structure interaction.

The design shear V_o (representing the building codes' provision) has therefore been calculated on the basis of a single degree-of-freedom (SDOF) rigidly based uncoupled building model, incorporating only the site effects represented by the spectra shown in Fig. 10. The dynamic storey shear and torque arising from lateral-torsional coupling in asymmetric buildings have then been evaluated by considering a single storey 5DOF building model exhibiting various levels of foundation interaction (characterised by the parameter α) and subjected to the appropriate earthquake design spectra. Hence the results obtained demonstrate the effects of soil-structure interaction on torsional coupling, and reveal certain important implications for earthquake-resistant design practice.

The interaction parameter α defined earlier has values 3, 6, 10 and ∞ corresponding to soft and medium clays, deep cohesionless soils, stiff soils and rock, respectively (Fig. 10). The effects of interaction on the effective eccentricity for 'primary' torsional effects (i.e. for members situated on the flexible edge of the building, Fig. 2) are shown in Fig. 12. The torsional code provisions neglecting accidental effects (Table 1) are also shown for comparison. The results correspond to the mean response

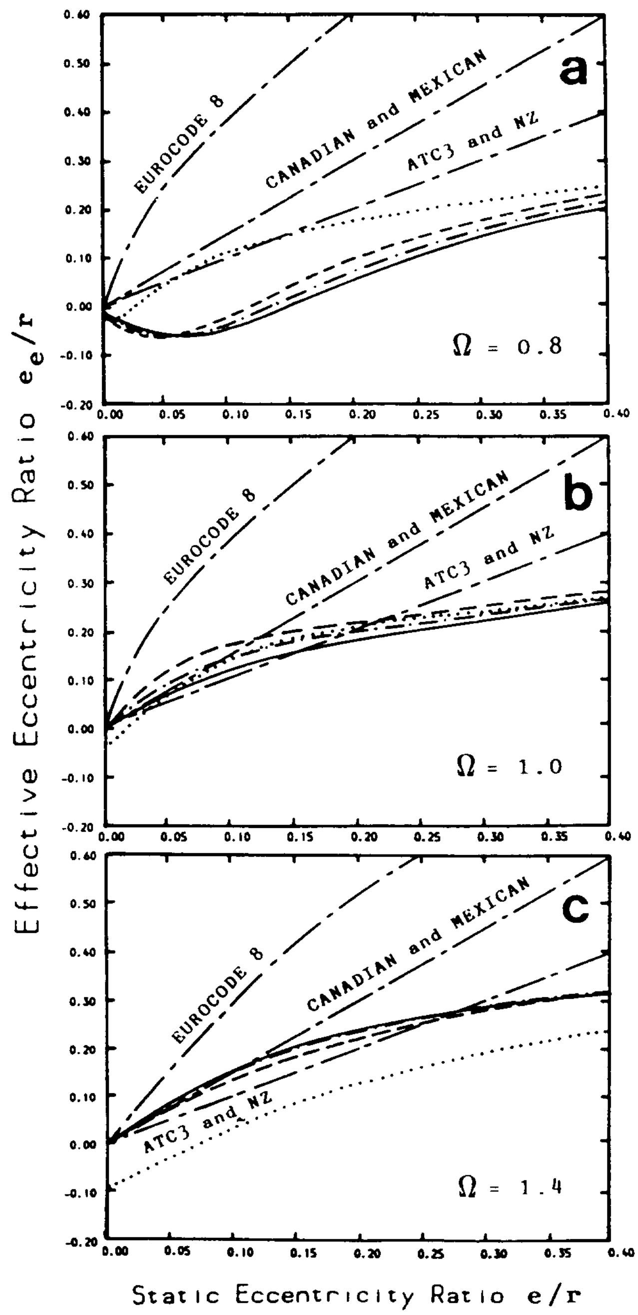

Fig. 12. Variation of mean peak effective eccentricity with static eccentricity and comparison with building codes, neglecting interaction in the design shear; $\alpha = 3$ (····), $\alpha = 6$ (- - -), $\alpha = 10$ (-·-·-·-), $\alpha = \infty$ (——).

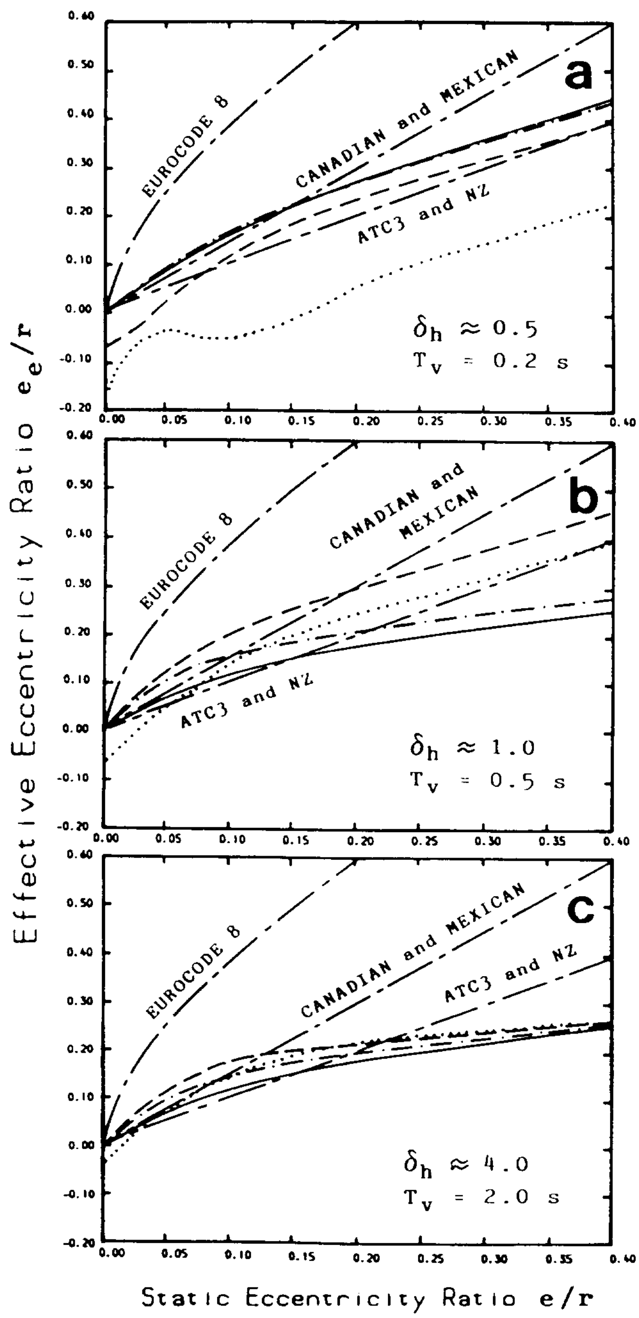

Fig. 13. Variation of effective eccentricity with static eccentricity and comparison with building codes; $\Omega = 1{\cdot}0$, $\alpha = 3$ (····), $\alpha = 6$ (- - -), $\alpha = 10$ (-·-·-·-), $\alpha = \infty$ (——).

for buildings with natural periods T_v in the range 0·2–2·0 sec. Whilst the results for $\Omega = 0{\cdot}8$ (Fig. 12(a)) show higher effective eccentricity for systems with higher levels of interaction, the code provisions are adequate throughout the full range of structural eccentricity e/r. For $\Omega = 1{\cdot}0$ (Fig. 12(b)), however, the influence of interaction and site effects together with the amplification of eccentricity lead to the greatest response in systems with $\alpha = 6$, that is with moderate interaction. In this case all the codes except Eurocode 8 are slightly non-conservative for $e/r < 0{\cdot}12$. For $\Omega = 1{\cdot}4$ (Fig. 12(c)) the torsional stiffness is high and the resulting peak torsional response is associated with rigidly based systems ($\alpha = \infty$). For small and moderate static eccentricities, the ATC3 and New Zealand code provisions underestimate the response for rigid to moderately flexible foundations.

Figure 13 shows the variation of effective eccentricity for interacting buildings with various height ratios δ_h, taking $\Omega = 1{\cdot}0$. The general trends are similar to the average response behaviour shown in Fig. 12(b), except that in Fig 13(a) the effective eccentricity for $\alpha = 3, 6$ is consistently smaller than that corresponding to $\alpha \geqslant 10$.

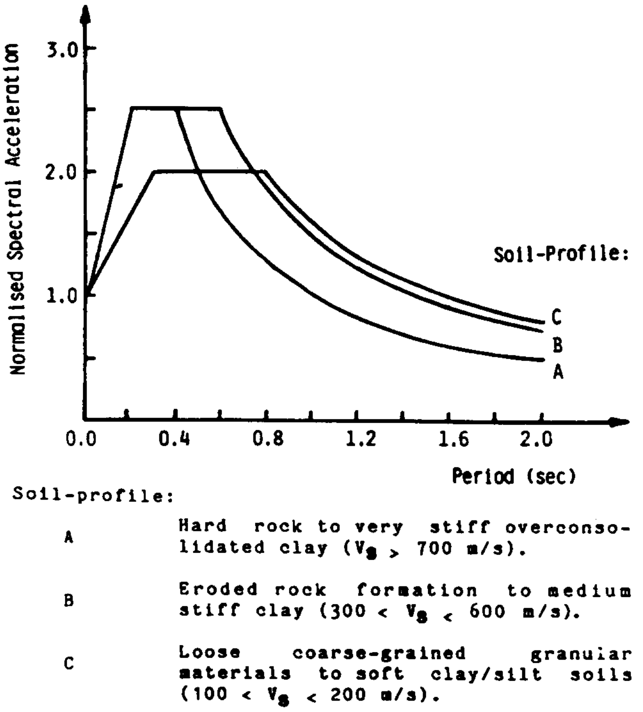

Fig. 14. Normalised spectral acceleration for different soil conditions, as recommended in the draft Eurocode EC8.

This is due to the small spectral accelerations for periods less than 0·3 sec, especially those associated with soft to medium clay and deep cohesionless soil sites (Fig. 10). However, as the structural height (and hence the period) increases, there is a reversal of the spectral ordinates corresponding to different site conditions, resulting in an amplification of the combined lateral-torsional response which is greatest for systems with moderate interaction ($\alpha = 6$, see Figs 13(b), (c)). These findings correlate well with the results of a previous parametric study[15] based on time-history analysis of the earthquake response of torsionally coupled interacting systems.

IMPLICATIONS FOR PRACTICAL EARTHQUAKE-RESISTANT DESIGN

Current design practice for determining lateral shear in accordance with simplified code procedures is based on the equivalent lateral force method (or a modal dynamic analysis), generally neglecting interaction effects. Allowance is made for the influence of soil conditions on the ground motion (that is, site effects); for example, Eurocode 8[39] accounts for the local site conditions by modifying the amplitude and shape of the design response spectrum, corresponding to three soil profiles (A: rock to very stiff over-consolidated clays with $V_s > 700$ m/s, B: medium stiff clays with $300 < V_s < 600$ m/s, and C: loose coarse-grained granular materials or soft clay/silt with $100 < V_s < 200$ m/s, as shown in Fig. 14.

Results from this and previous studies[20,24] have shown that dynamic lateral-torsional coupling reduces the lateral shear compared with the equivalent uncoupled value. This suggests that for asymmetric buildings, the evaluation of design torque from an uncoupled design shear (eqn (1b)) leads to a conservative solution provided torsional coupling is adequately accounted for in the design eccentricity expressions (eqn (2)).

In designing torsionally coupled buildings based on flexible foundations (that is, exhibiting soil-structure interaction), a consistent approach would be to calculate the uncoupled design shear by analysing appropriate systems using a flexible base assumption as in Ref. 16. As an optional secondary consideration, the maximum reduction of design shear due to interaction (Fig. 11) could be restricted to 30% in accordance with the ATC3[4] recommendations. Hence using these results, recommendations for design torque can be derived based on an empirical relationship involving the uncoupled flexibly based design shear and an appropriate expression for design eccentricity.

Alternatively, if the uncoupled design shear is based on rigid base analysis incorporating only the site effects, as in Figs 12 and 13, the complexities associated with analysis and design for torsional coupling in asymmetric buildings are reduced. This approach also neglects the secondary effects associated with foundation movements in particular the rocking effect. For highly interacting systems ($\alpha = 3$) and tall buildings ($\delta_h = 4{\cdot}0$), it has been shown[30] that the roof rocking displacement can be of the same magnitude as the corresponding translational displacement. Nevertheless the building can be analysed for design shear assuming it is rigidly based, provided adequate safety is incorporated in the allowance for torsion to account for both torsional coupling effects and the effects of soil-structure interaction on lateral and torsional forces.

The derivation of empirical relationships for design eccentricity in asymmetric buildings supported on flexible foundations is directly affected by the choice associated with the determination of the design shear. Including soil-structure interaction effects in this calculation results in a more severe torsional design requirement (based most appropriately on the effective eccentricity) compared with that obtained from a rigid base analysis. This trend has been summarised in Figs 15–17, which compare the design effective eccentricity requirements obtained from a fully rigid base analysis with those yielded by the above two approaches for interacting systems. Figure 15 compares the enveloped effective eccentricity ratio for rigidly based buildings over the range of $0{\cdot}6 < \Omega < 2{\cdot}0$ with results for interacting systems having $\Omega = 1{\cdot}0$. The period $T_v = 0{\cdot}2$ sec corresponds to low-rise buildings ($\delta_h \approx 0{\cdot}5$). Figures 16 and 17 give results for intermediate height ($T_v = 0{\cdot}5$ sec, $\delta_h \approx 1{\cdot}0$), and tall buildings ($T_v = 2{\cdot}0$ sec, $\delta_h \approx 4{\cdot}0$), respectively.

It is concluded from Figs 15(c)–17(c) that an adequately

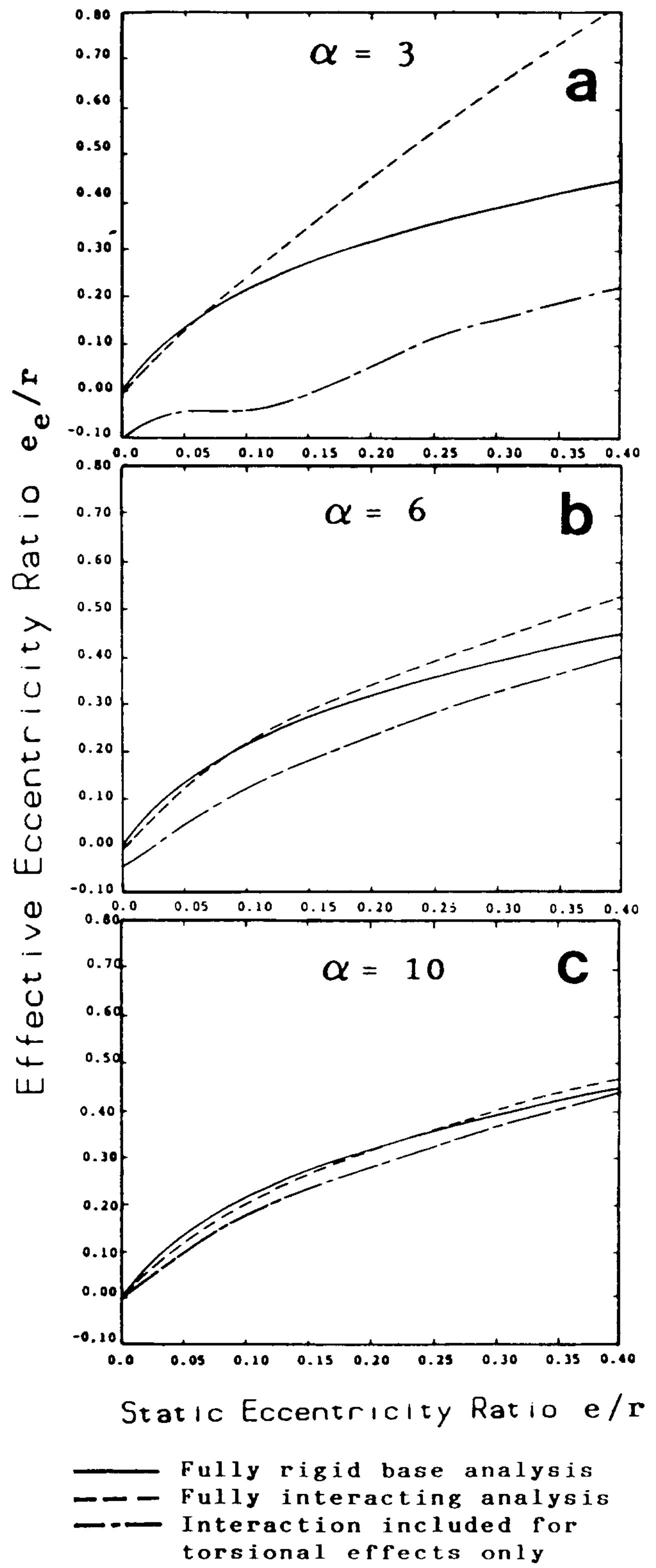

Fig. 15. Influence of soil-structure interaction on design effective eccentricity for short buildings, $\delta_h \approx 0{\cdot}5$, $T_v = 0{\cdot}2$ sec.

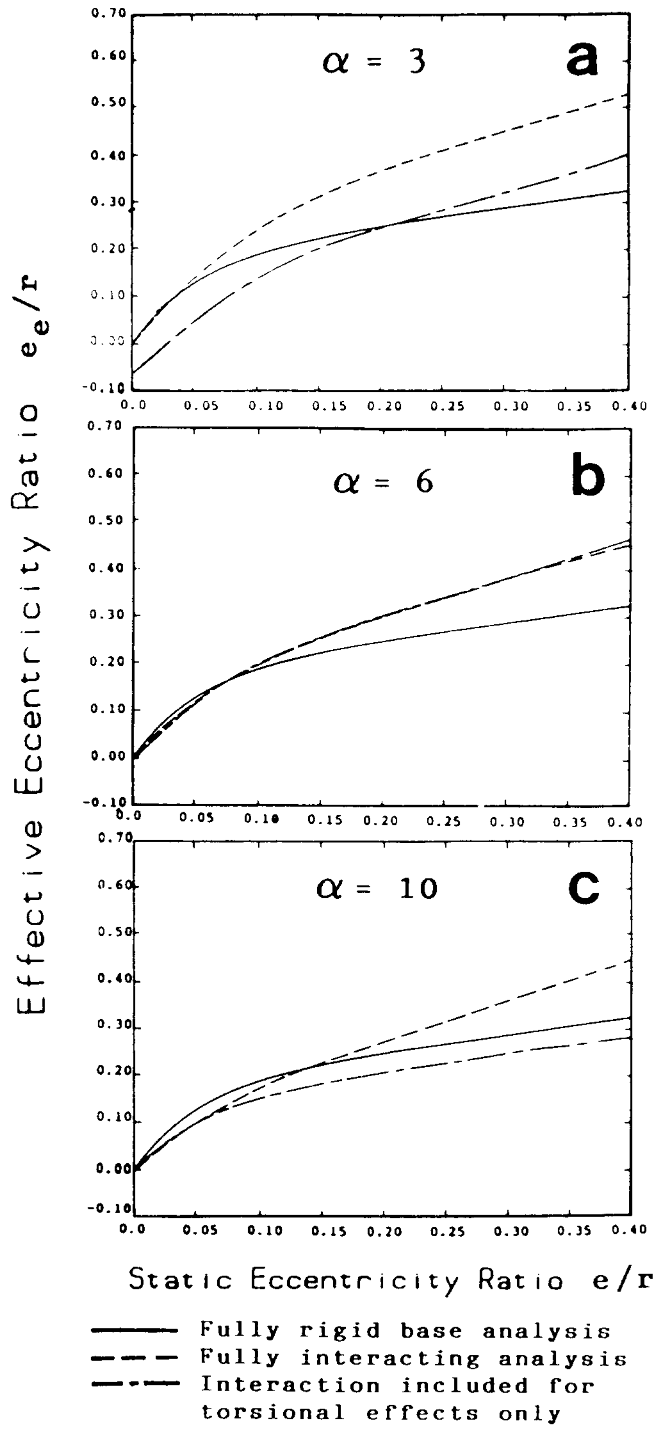

Fig. 16. Influence of soil-structure interaction of design effective eccentricity for intermediate height buildings, $\delta_h \approx 1{\cdot}0$, $T_v = 0{\cdot}5$ sec.

formulated torsional allowance for rigidly based buildings is also sufficient for buildings founded on stiff soils ($\alpha = 10$) exhibiting only minor interaction effects, provided the corresponding rigid base design shear is used. Furthermore, Figs 15 and 17 show that the effective eccentricity curves for the completely rigid based model give an adequate torsional allowance for short and tall buildings covering all practical levels of foundation flexibility (α = 3, 6, 10). Figure 15(a) is an extreme case which shows an exceptionally high effective eccentricity requirement when incorporating interaction in the

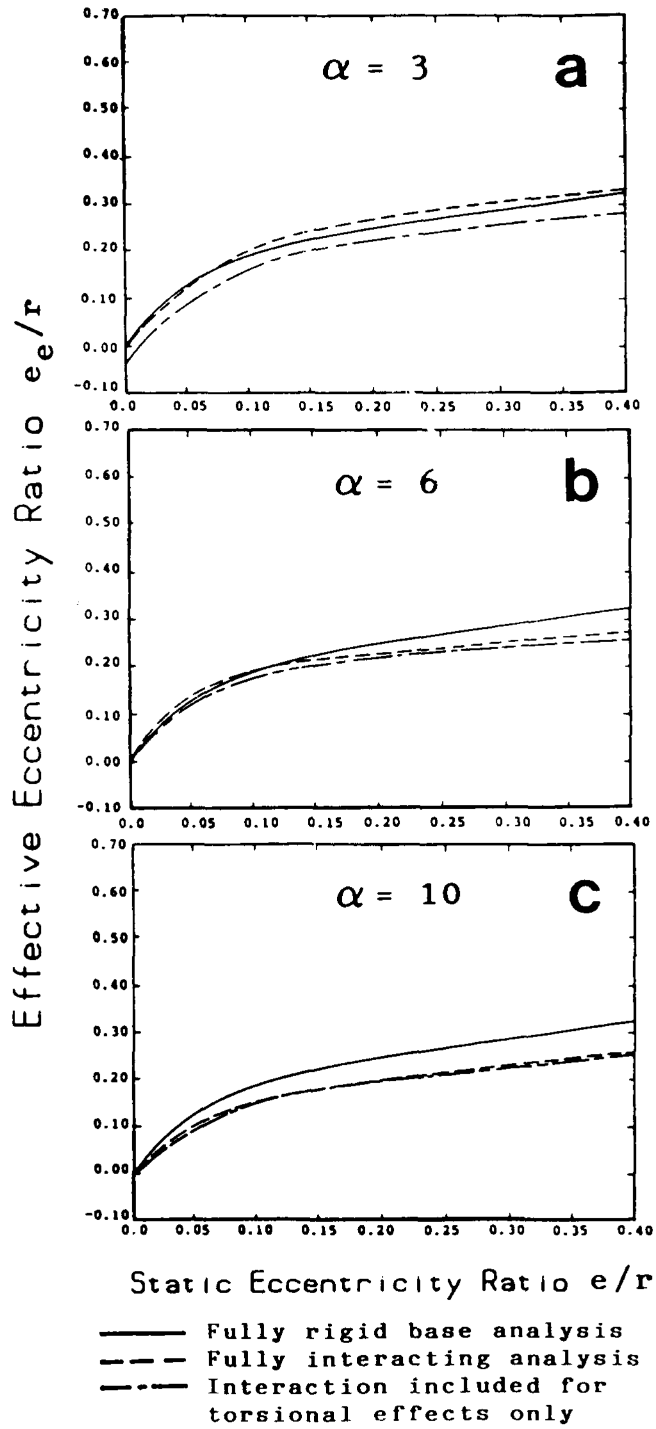

Fig. 17. Influence of soil-structure interaction on design effective eccentricity for tall buildings, $\delta_h \approx 4{\cdot}0$, $T_v = 2{\cdot}0$ sec.

calculation of the design shear. This case relates to short, stiff buildings founded on very flexible soils, for which the severe reduction in uncoupled shear due to interaction (Fig. 11) must be compensated by a corresponding increase in effective eccentricity.

For intermediate height buildings ($\delta_h \approx 1{\cdot}0$) with low and moderate soil foundation stiffness ($\alpha = 3, 6$) an important result arises (Figs. 16(a), (b)). For these cases, the interacting systems have higher effective eccentricity than the equivalent rigidly based buildings. For certain ranges of the structural eccentricity ratio e/r, this observation is valid whether or not the uncoupled shear is calculated by incorporating interaction effects. Figures 16(a), (b) show that the higher effective eccentricity based on results for non-interacting systems in calculating the design shear applies for $e/r > 0{\cdot}2$ when $\alpha = 3$ and $e/r > 0{\cdot}1$ for $\alpha = 6$. For $e/r = 0{\cdot}4$ for example, the effective eccentricity obtained when including interaction in the determination of torsional coupling effects exceeds that obtained by fully rigid base analysis by 25% when $\alpha = 3$ and by 41% when $\alpha = 6$ (Figs 16(a), (b)). Hence it has been concluded that significantly increased torsional coupling effects, as measured by the increase in total lateral edge response due to shear and torsion, are exhibited for intermediate height structures supported on soil foundations with low or moderate stiffness.

CONCLUSIONS

The design of eccentric buildings corresponding to a range of foundation flexibility can be approached in accordance with present aseismic building regulations, either by adopting an equivalent lateral force procedure or a modal dynamic analysis to determine the design shear forces. This calculation can be formulated in such a way that the building-foundation interface is modelled to exhibit movements (interaction), but more commonly the base of the building is assumed to be rigidly fixed. The mathematics and analysis of the former approach is somewhat complex and intricate, such that incorporation of interaction effects has not yet been developed in many major codes, which only provide simplified procedures for the design engineer. As a result, of the five major seismic codes considered in this study, only ATC3 accounts for soil-structure interaction effects, and then only on an empirical basis.

Parametric studies carried out in this paper have investigated the seismic lateral-torsional behaviour of asymmetric building models corresponding to a range of foundation flexibility, and have established the broad criterion that designing according to the maximum response arising from lateral-torsional coupling in rigidly based buildings suffices for most cases of structure-foundation systems described parametrically in terms of key controlling parameters. However, intermediate height buildings ($\delta_h = 1{\cdot}0$, $T_v = 0{\cdot}5$ s), with frequency ratios Ω close to unity and based on very flexible and moderately flexible foundations ($\alpha = 3$ and 6, respectively) require a torsional provision increased by up to 40% compared with rigidly based design, to account for amplification of torsional response due to lateral-torsional coupling.

The exisiting torsional provisions of major seismic codes, derived for rigidly based buildings, can be improved by continuing the use of an empirical approach for determining the design torque but based on an effective eccentricity rather than a dynamic eccentricity approach. The amplification of combined lateral-torsional response brought about by soil-structure interaction in certain limited ranges of the key parameters may also be incorporated in the design without increasing the complexity of the empirical approach.

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