Chapter 2
Existing Codes on Earthquake Design with and Without Seismic Devices and Tabulated Data

2.1 Existing Codes on Earthquake Design

Some well-known codes are discussed in brief and they are classified under items such as seismic actions, dynamic characteristics, seismic weights, forces, moments, storey drift (P – Δ effect), seismic factors, site characteristics and building categories. These are only briefs and for detailed codified design, a reference is made to individual codes where detailed applications are available from the references given at the end of this chapter.

2.1(a) Existing Codes–Comparative Study

They are briefly mentioned below as given by the different countries:

2.1.1 Algeria: RPA (1989)

Seismic actions/dynamic characteristics

Load combination

$$G + Q + E \quad \text{or} \quad 0.8G + E \quad \text{or} \quad G + Q + 1.2E \tag{2.1}$$

G = dead load
Q = live load
E = seismic load
N = number of storeys
h = height of the building
$T = 0.1\,N$ s (building frames with shear walls)
$T = \dfrac{0.09\,h}{\sqrt{L}}$ (other buildings)

M.Y.H. Bangash, *Earthquake Resistant Buildings*,
DOI 10.1007/978-3-540-93818-7_2, © M.Y.H. Bangash 2011

Table 2.1 Seismic factors

A/BC	Seismic coefficient		
	I	II	III
1	0.12	0.25	0.35
2	0.08	0.15	0.25
3	0.05	0.10	0.15

Seismic weights, forces and moments

$$F_{\mathrm{k}} = \text{equivalent lateral force} = \frac{(V - F_{\mathrm{t}})W_{\mathrm{k}}h_{\mathrm{k}}}{\sum_{i=1}^{N} W_i h_i} \tag{2.2}$$

$$V = ADB\bar{Q}W \tag{2.3}$$

Where

A = seismic coefficient = 0.05–0.25
W = seismic weight

$$\text{Quality factor } \bar{Q} = 1 + \sum_{1}^{6} Pq \qquad B = 0.20{-}0.5 \tag{2.4}$$

$F_{\mathrm{t}} = 0$

$$T \leq 0.7\,\mathrm{s} = 0.07TV \leq 0.25\,V \quad \text{for } T > 0.7\,\mathrm{s} \tag{2.5}$$

F_{t} = additional force at the building top

Storey drift/P–Δ effect

$$\Delta_i = X_i - X_{i-1} \qquad \text{with } X_0 = 0 \tag{2.6}$$
$$X_i = \text{lateral displacement at } b$$
$$\Delta \ngtr 0.0075 \times \text{storey height}$$

$$P_{\mathrm{F}} = \text{performance factor} = 0.2{-}0.67 \tag{2.7}$$

Building category (BC)
Category 1 (500 year return)
Category 2 (100 year return)
Category 3 (50 year return)

Table 2.2 Site characteristics/building categories

Zone	Seismicity zone
0	Negligible
I	Low
II	Average
III	High

2.1.2 Argentina : INPRES-CIRSOC 103 (1991)

Seismic actions/dynamic characteristics

Horizontal seismic spectra

$$\left.\begin{array}{ll} S_a = a_s + (b - a_s)\dfrac{T}{T_1} & \text{for } T \le T_1 \\ S_a = b & \text{for } T_1 \le T \le T_2 \\ S_a = b\left[\dfrac{T_2}{T_1}\right]^{2/3} & \text{for } T \ge T_2 \end{array}\right\} \tag{2.8}$$

$$\left.\begin{array}{ll} S_a = a_s + (f_{\mathrm{A}} b - a_s)\dfrac{T}{T_1} & \text{for } T \le T_1 \\ S_a = f_{\mathrm{A}} b & \text{for } T_1 \le T \le T_2 \\ S_a = \left[1 + (f_{\mathrm{A}} - 1)\dfrac{T_2}{T_1}\right]\left[b\left\{\dfrac{T_2}{T_1}\right\}^{2/3}\right] & \text{for } T \ge T_2 \end{array}\right\} \tag{2.9}$$

$\xi =$ damping 5%
$T =$ fundamental period
$f_{\mathrm{A}} =$ amplification factor due to ξ

$$= \sqrt{\frac{5}{\xi}} \text{ for } 0.5\% \le \xi \le 5\% \tag{2.10}$$

$\xi =$ relative damping = percentage of critical damping

$$T_0 = 2\pi\sqrt{\frac{\sum_{i=1}^{n} W_i u_i^2}{g\sum_{i=1}^{n} \bar{F}_i u_i}} \tag{2.11}$$

$W_i =$ gravity load at level i
$g =$ acceleration
$u_i =$ displacement at i
$\bar{F}_i =$ normal horizontal force

For regular building level height equals

$$T_0 = \frac{h_n}{100}\sqrt{\frac{W_n u_n}{g\bar{F}_n}} \tag{2.12}$$

Alternative empirical formula is

$$T_{0l} = \frac{h_n}{100}\sqrt{\frac{30}{L} + \frac{2}{1 + 300}} \tag{2.13}$$

T_{0l} = fundamental period
h_n = height of the building
L = length of the building
d = density of the wall

Torsional effects

$$M_{ti} = \text{torsional moment at level } i = (1.5e_1 + 0.10L)V_i \tag{2.14}$$

or

$$M_{ti} = (e_1 - 0.10L)V_i \tag{2.15}$$

e_1 = distance between CS at level i and the line of action of the shear force measured perpendicular to the analysed direction
L = maximum dimension in plan measured perpendicular to the direction of V_i

Seismic weights, forces and moments

$$\bar{F}_i = \frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i} \tag{2.16}$$

W_i = seismic weight at level $i = G_i + \eta L_i$
$\eta = 0{-}1.0$
h_i = height of the storey level i above the base level
n = number of levels in the building
G_i and ηL_i = dead and live loads respectively

Vertical seismic actions

$$S_{aV} = f_V S_a \tag{2.17}$$

f_V	Seismic zone
0.6	4
0.6	3
0.5	2
0.4	1
0.4	0

Load states

$1.3E_W \pm E_S$ E_W = actions due to gravitational loads

$0.85E_W \pm E_S$ E_S = seismic actions

Building separation due to hammering

$$Y_i = \text{ separation between adjacent structures} = \delta_i + f_{\mathrm{S}} h_i \tag{2.18}$$

$$Y_i \geq f_0 h_i + 1 \text{ cm} \tag{2.19}$$

$$Y_i \geq 2.5 \text{ cm} \tag{2.20}$$

f_{S} = factor depending on foundation soil = 0.001–0.0025
f_0 = different soils in seismic zones = 0.003–0.010

$$V_i = \text{storey shear force} = \sum_{k=1}^{n} F_k \tag{2.21}$$

$$M_i = \text{overturning moment} = \alpha \sum_{k=i+1}^{n} F_k (h_k^* - h_i^*) \tag{2.22}$$

where

$$\begin{aligned} h_k^*, h_i^* &= \text{heights at level } k \text{ and } i \text{ from the foundation level} \\ i &= 0, 1, 2, 3, \ldots, n-1 \end{aligned} \tag{2.23}$$

$$V_0 = \text{base shear force} = CW = C \sum_{i=1}^{n} W_i \tag{2.24}$$

$$C = \frac{S_a \gamma_d}{R} \tag{2.25}$$

$$F_i = \text{lateral force} = \frac{W_i h_i V_0}{\sum_{k=1}^{n} W_k h_k} \tag{2.26}$$

$$F_{\mathrm{V}} = \text{vertical seismic forces} = \pm C_V \gamma_d W \tag{2.27}$$

Storey drift/*P*–Δ effect

Lateral displacements δ and storey drift Δ

$$\Delta_i = \delta_i - \delta_{i-1} \quad \text{with } \delta_0 = 0 \tag{2.28}$$

Alternatively

$$\delta_i = \frac{g}{4\pi^2} \frac{T^2 F_i}{W_i} \tag{2.29}$$

$\delta =$ horizontal displacements at levels i

Limiting values for storey drift
Non-structural elements attached are damaged

Group A_0	Group A	Group B
0.01	0.011	0.014

Non-structural elements attached are not damaged

0.01	0.015	0.019

$P - \Delta$ effect

$$\beta_i = \frac{P_i \Delta_i}{V_i H_i} \geq 0.08 \tag{2.30}$$

$P_i =$ total seismic weight at level i
$V_i =$ shear force at storey i
$H_i =$ storey height i
$\psi =$ amplification factor for forces and displacements

$$= \frac{1}{1 - \beta_{\max}}$$

$\beta_{\max}$ is the $\beta_{i\max}$ value.

Reduction factor R
A factor for the dissipation of the energy by inelastic deformation:

$$R = 1 + (\mu - 1)\frac{T}{T_1} \quad \text{for } T \leq T_1 \tag{2.31}$$

$$R = \mu \quad \text{for } T \leq T_1 \tag{2.32}$$

μ varies from 6 to 1.

Table 2.3 Seismic factors γ_d defines the risk factor:

Group	γ_d
A_0	1.4
A	1.3
B	1.0

Table 2.4 Site characteristics/building categories
Seismic zones

Zone	Seismicity
0	Negligible
I	Low
II	Average
III	High

Table 2.5 Seismic zone

Zone	Risk
0	Very low
1	Low
2	Moderate
3	High
4	Very high

Table 2.6 Building classifications

Group	Classification
A_0	Important centres
A	Hotels, stadia, etc.
B	Private, commercial, industrial buildings
C	Containers, silos, sheds, stables

Zone	a_s	b	T_1	T_2
4	0.35	1.05	0.2–0.4	0.35–1.0
3	0.25	0.75	0.2–0.4	0.35–0.1
2	0.16–0.18	0.48–0.54	0.2–0.4	0.5–1.0
1	0.08–0.10	0.24–0.30	0.2–0.4	0.6–1.2
0	0.04	0.12	0.10	1.2–1.6

Table 2.7 Vertical seismic coefficient (C_V)

Zone	Balcony and cantilevers	Roof and large spans
4	1.20	0.65
3	0.86	0.47
2	0.52	0.28
1	0.24	0.13

Building category (BC)

Category 1 (500 year return)
Category 2 (100 year return)
Category 3 (50 year return)

2.1.3 Australia: AS11704 (1993)

Seismic actions/dynamic characteristics

Bearing walls and frames where k_d = deflection amplification factor

Bearing walls	1.25–4.0
Building frame	1.50–4.0
Moment-resisting frame	2.0–5.5
Dual system with a special moment-resisting frame	4.0–6.5
Dual system with intermediate moment frame (steel or concrete)	4.5–5.0

Torsional effects

$e_{d1} = A_1 e_S + 0.05b$
$e_{d2} = A_2 e_S - 0.05b$
$A_S =$ dynamic eccentricity factors
$e_S =$ eccentricity
$A_1 = 2.6 - \frac{3.6e_S}{b} \geq 1.4 = 2.6$
$A_2 = 0.5$
$b =$ maximum dimension at level i

Seismic weights, forces and moments

$$V = \text{total horizontal force (kN)} = ZIKCSW \tag{2.33}$$

$I =$ occupancy importance factor
$= 1.2$ essential facilities
$= 1.0$ other buildings
$W =$ total dead load $+$ 0.25 live load

$$V = \text{seismic base shear } = \frac{ICS}{R_f} Gg$$

$$V_i = \text{horizontal shear force} = \sum_{x=i}^{n} F_x$$

$$M_0 = \text{overturning moment} = \alpha \sum_{i=1}^{n} F_i h_i$$

$i =$ levels number
$\alpha = 0.75$ general
$\alpha = 1.0$ at base
$\alpha = 0.5$ at top

Base shear distribution

$$F_x = C_{Vx} \cdot V \tag{2.34}$$

$$C_{Vx} = \frac{G_{gs} h_x^k}{\sum_{i=1}^{n} G_{gi} \cdot h_i^k} \tag{2.35}$$

$x = i$ levels
$k = 1 \ T \leq 0.5$ s
Gg = gravity load $= G + \psi Q$
G = dead load (kN)
Q = live load (kN)

Storey drift/P–Δ effect

$$\begin{aligned} \delta_x &= \text{interstorey drift} = k_\mathrm{d}\delta_{xe} \\ \delta_{xe} &= \text{lateral displacement at levels } i \end{aligned} \tag{2.36}$$

$$= a_i\left(\frac{T}{2\pi}\right)^2 = \frac{gF_i}{G_{gi}}\left(\frac{T}{2\pi}\right)^2 \tag{2.37}$$

P–Δ effect
To allow for P–Δ effect, the storey drift is increased by

$$\frac{0.9}{(1-m)} \geq 1.0 \tag{2.38}$$

When $m < 0.1$ there is no effect

$$m = \text{stability coefficient} = \frac{P_x\Delta_x}{V_x h_{sx} k_\mathrm{d}} \tag{2.39}$$

$$P_x = \text{total vertical design load}$$

Seismic factors

$$C = \text{seismic response factor} = \frac{1}{15\sqrt{T}} \leq 0.12 \tag{2.40}$$

$$a = \text{acceleration coefficient} = 0.05\text{–}0.11$$

$$C = \frac{1.25a}{T^{2/3}} \tag{2.41}$$

$$T = \frac{h_0}{46} \text{ (main direction)} \tag{2.42}$$

$$T = \frac{h_0}{58} \text{ (orthogonal direction)} \tag{2.43}$$

$\psi = 0.4$–0.6
R_f = response modification factor = 1.5–8.0

k = horizontal force factor
= 0.75 (ductile)
= 3.2 (brittle)
Z = zone factor
= 0 for zone A ductile
= 0.09 for zone A non-ductile
= 0.18 for zone 1
= 0.36 for zone 2

Site characteristics/building categories

S = soil structure resonance factor = 1.5 if not calculated

Building classification

Type I Domestic and not more than two storeys
Type II Buildings with high occupancy (schools, theatres, etc.)
Type III Buildings for essential functions (power stations, tall structures, hospitals, etc.)

Table 2.8 Seismic design categories

a_S	III	II	I	Domestic
$a_S \geq 0.20$	E	D	C	H_3
$0.10 \leq a_S \leq 0.20$	D	C	B	H_2
$a_S < 0.10$	C	B	A	H_1

C = static analysis
D = static and dynamic analysis
E = static and dynamic analysis
S = site factor varies from 0.67 to 2.0

2.1.4 China: TJ 11-78 and GBJ 11-89

Seismic actions/dynamic characteristics

S = total effect of horizontal seismic action

$$= \sqrt{\sum_{j=1}^{N} S_j^2} \tag{2.44}$$

T_1 = fundamental period
S_j = modal effect caused by seismic forces of the jth mode
N = number of modes

$\delta_{\rm n}$ values

$$\left.\begin{array}{lll} Tg & T_1 > 1.47\,g & T_1 \leq 1.4Tg \\ \leq 0.25 & 0.08T_1 + 0.07 & \text{no need to consider} \\ 0.3 - 0.4 & 0.08T_1 + 0.01 & \\ \geq 0.55 & 0.08T_1 - 0.02 & \end{array}\right\}\ \text{concrete building} \tag{2.45}$$

Horizontal seismic action

$$F_{xji} = \alpha_j \gamma_{tj} W_i \qquad F_{yji} = \alpha_j \gamma_{tj} Y_{ji} W_i \tag{2.46}$$

$$F_{tji} = \alpha_j \gamma_{tj} r_i^2 \phi_{ji} W_i \tag{2.47}$$

ϕ_{ji} = angular rotation at ith floor jth mode
x, j, t = directions in x, y and angular direction
r_i = mass radius of gyration
α_j = seismic coefficient
$i = 1, 2, ..., n$
$j = 1, 2, ..., m$

Torsional effects

Modelling with degrees of freedom including two orthogonal horizontal displacements and one angular rotation for each level. The complete quadratic combination (CQC) can be used to obtain the response "S" (force, moment and displacement) given by

$$S = \sqrt{\sum_{j=1}^{n} \sum_{k=1}^{n} \rho_{jk} S_j S_k} \tag{2.48}$$

S_j, S_k = effects caused by seismic forces

$$\rho_{jk} = \frac{0.02(1 + \lambda_{\rm T})(\lambda_{\rm T})^{1.5}}{(1 - \lambda_{\rm T}^2)^2 + 0.01(1 + \lambda_{\rm T})^2 \lambda_{\rm T}} \tag{2.49}$$

where $\lambda_{\rm T}$ = ratio of the periods of the kth and jth modes.

Seismic weights, forces and moments

Base shear force:

$$F_{\rm EK} = \alpha W_{\rm eq} = \alpha \sum_{i=1}^{n} W_i \tag{2.50}$$

F_i = horizontal seismic forces at i level

$$= \frac{W_i H_i}{\sum_{j=1}^{n} W_j H_j} F_{\rm EK}(1 - \delta_n) \tag{2.51}$$

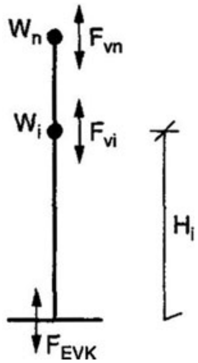


Fig. 2.1 Seismic actions at various levels

ΔF_n = additional seismic force applied to the top level of the building.

$$= \delta_n F_{\mathrm{EK}} \tag{2.52}$$

H = height from the base

$$M_i = \text{overturning moment} = \sum_{j=i+1}^{n} f_j(H_j - H_i) \tag{2.53}$$

$$F_{\mathrm{EVK}} = \text{vertical seismic action force} = \alpha_{\mathrm{V_{max}}} W_{\mathrm{eq}} \tag{2.54}$$

At ith level (see Fig. 2.1)

$$F_{\mathrm{V}i} = \frac{W_i H_i}{\sum_{j=1}^{n} W_j H_j} F_{EVK} \tag{2.55}$$

Storey drift/P–Δ effect

The elastic relative displacement is

$$\Delta U_{\mathrm{e}} \leq [\theta_{\mathrm{e}}] H \tag{2.56}$$

where θ_{e} = elastic drift limitation.

Elasto-plastic deformation

$$\Delta U_{\mathrm{p}} = \eta_{\mathrm{p}} \Delta U_{\mathrm{e}} \tag{2.57}$$

Table 2.9 Values of θ_{e}

Structure frame	θ_{e}
Brick infill walls	1/550
Others	1/450
Public buildings	1/800
Frame shear walls	1/650

Table 2.10 The value of θ_p

θ_p	Structure
1/30	Single-storey RC frame
1/50	Frame with infill
1/70	Frame in the first storey of a brick building

or

$$\Delta U_p = \mu \Delta U_y = \frac{\eta_p \Delta U_y}{\xi_y} \tag{2.58}$$

ΔU_y = storey yield displacement
ΔU_e = elastically calculated storey displacement
η_p = amplification coefficient

$$U_p \leq [\theta_p] H \tag{2.59}$$

Seismic factors

$$\alpha = \text{seismic coefficient}$$

$$\alpha = \alpha_{max}(5.5T_1 + 0.45) \quad \text{for } T_1 \leq 0.1\text{s} \tag{2.60}$$

$$\alpha = \alpha_{max} \quad \text{for } 0.1 < T_1 \leq 0.1Tg$$

$$\alpha = \left(\frac{Tg}{T_1}\right)^{0.9} \alpha_{max} \quad \text{for } Tg < T_1 < 3\text{s} \tag{2.61}$$

δ_n = additional seismic action coefficient
γ_{tj} = the mode participation factor of the jth mode
ρ_{jk} = coupling coefficient for jth and kth modes
μ = storey ductility coefficient

$$\xi_y = \text{storey yield strength coefficient} = \frac{F_y}{Q_e} \tag{2.62}$$

Table 2.11 α_{max} values for various intensities

Intensity	α_{max}		
VI	VII	VIII	IX
0.04	0.08	0.16	0.32

Table 2.12 Values of Epicentre at different sites

Epicentre	Site			
	I	II	III	IV
Near	0.2	0.3	0.4	0.65
Remote	0.25	0.40	0.55	0.85

Table 2.13 The values of ξ_y

Structure	ξ_y			
	0.5	0.4	0.3	0.2
2–4 storeys	1.3	1.4	1.6	2.1
5–7 storeys	1.5	1.65	1.8	2.4
8–12 storeys	1.8	2.0	2.2	2.8
Single storey	1.3	1.6	2.0	2.6

$F_y = 2Q_{y1} + (m-2) \times Q_{y2}$
m = total number of columns in a storey
$Q_{y1} Q_{y2}$ = average yield strength of exterior and interior columns, respectively

Site characteristics/building categories

Building classifications

Type A Structures not failing beyond repair. Important structures.
Type B Buildings and structures in the main city.
Type C Structures not included in A, B, D.
Type D Structures of less importance not likely to cause deaths, injuries or economic losses.

Tg = characteristic period of vibration

2.1.5 Europe: 1-1 (Oct 94); 1-2 (Oct 94); 1-3 (Feb 95); Part 2 (Dec 94); Part 5 (Oct 94); Eurocode 8

Note: From these parts minimum items are given. For details see the entire codes and parts.

Seismic actions/dynamic characteristics

Horizontal seismic action: two orthogonal components with the same response spectrum.

Vertical seismic action:

$T < 0.15$ s the vertical ordinates = $0.15 \times$ horizontal
$T > 0.15$ s the vertical ordinates = $0.5 \times$ horizontal

T between 0.15 and 0.5 s – linear interpolation.

$$S_e(T) = \text{elastic response spectrum} \quad 0 \le T \le T_B \tag{2.63}$$

$$S_e(T) = a_g \cdot S\left[1 + \frac{T}{T_B}(\xi B_0 - 1)\right] \; T_B \le T \le E \tag{2.64}$$

$$S_e(T) = a_g S \xi B_0 \; T_C \le T \le T_D \tag{2.65}$$

$$S_e(T) = a_g S \xi B_0 \left[\frac{T_C}{\mathrm{T}}\right]^{K_1} \; T_D = T \tag{2.66}$$

$$S_e(T) = a_g S \xi B_0 \left[\frac{T_C}{\mathrm{T_D}}\right]^{K_1} \left[\frac{T_D}{T}\right]^{K_2} \tag{2.67}$$

At A $= a_g \cdot S$
At B $= a_g \cdot S \cdot \xi B_0$

where

$S_e(T)$	ordinate of the elastic response spectrum,
T	vibration period of a linear single-degree-of-freedom system,
a_g	design ground acceleration for the reference return period,
T_B, T_C	limits of the constant spectral acceleration branch,
T_D	value defining the beginning of the constant displacement range of the spectrum,
S	soil parameter
η	damping correction factor with reference value $\eta = 1$ for 5% viscous damping,

B_0 = spectral acceleration amplification factor for 5% viscous damping,
K_1, K_2 = exponents that influence the shape of the spectrum for vibration at T_D and T_C (see Fig. 2.2).

For the three subsoil classes A, B and C the values of the parameters B_0, T_B, T_C, T_D, S are given in Table 2.14 reproduced from the code.

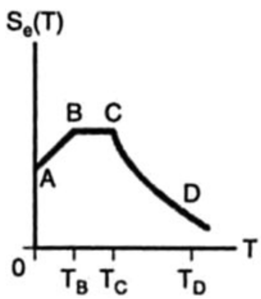


Fig. 2.2 Response spectrum

Table 2.14 Values of the parameters describing the elastic response spectrum

Sub-soil class	S	β_0	k_1	k_2	T_B[s]	T_C[s]	T_D[s]
A	[1,0]	[2,5]	[1,0]	[2,0]	[0,10]	[0,40]	[3,0]
B	[1,0]	[2,5]	[1,0]	[2,0]	[0,15]	[0,60]	[3,0]
C	[0,9]	[2,5]	[1,0]	[2,0]	[0,20]	[0,80]	[3,0]

These values are selected so that the ordinates of the elastic response spectrum have a uniform probability of exceedance over all periods (uniform risk spectrum) equal to 50%.

Design spectrum

Here, a_g is replaced by α and $S_e(T)$ by$S_d(T)$

$$\xi = \frac{1}{q} \tag{2.68}$$

K_1 and K_2 are replaced by K_{d1} and K_{d2}, respectively

$$T_C = T_D \geq 0.2\alpha \tag{2.69}$$

Combinations of seismic actions

$$\sum G_{Kj} + \sum \psi_{Ei} Q_{Ki} \tag{2.70}$$

$\sum \psi_{Ei} =$ combination coefficients for variable actions $i = 0.5-1.0$

G and Q are characteristic values of actions

$\psi_{Ei} = \phi\psi_{2i}\ \phi = 0.5-1.0$

Design seismic coefficient = 0.2

Seismic weights, forces and moments

$$M_{1it} = e_{1j} F_i \tag{2.71}$$

e_{1i} = accidental torsional eccentricity
L_i = floor dimensions perpendicular to seismic action

$$\text{seismic base shear} = F_b = V = S_e(T_1)W \tag{2.72}$$

$W = W_t$ = total weight
$T_1 \leq 4T_C$ (fundamental period)

$$F_i = F_{\mathrm{b}} \frac{S_i W_i}{\sum S_j \cdot W_j} \tag{2.73}$$

when horizontal displacement is increasing linearly.

$$F_i = F_{\mathrm{b}} \frac{Z_i W_i}{\sum Z_j \cdot W_j} \tag{2.74}$$

S_i, S_j = displacement of masses M_i and M_j in the fundamental mode shape

For sites with ground conditions not matching the three subsoil classes A, B, C special studies for the definition of the seismic action may be required.

The value of the damping correction factor η can be determined by the expression

$$\eta = \sqrt{7/(2+\xi)} \geq 0,7 \tag{2.75}$$

where ξ is the value of the viscous damping ratio of the structure, expressed in percent. If for special studies a viscous damping ratio different from 5% is to be used, this value will be given in the relevant parts of Eurocode 8.

Peak ground displacement

1. Unless special studies based on the available information indicate otherwise the value $U = \mathrm{d_g}$ of the peak ground displacement may be estimated by means of the following expression:

$$U = \mathrm{d_g} = [0,05] \cdot a_{\mathrm{g}} \cdot S \cdot T_{\mathrm{C}} \cdot T_{\mathrm{D}} \tag{2.76}$$

with the values of $a_{\mathrm{g}}, S, T_{\mathrm{C}}, T_{\mathrm{D}}$ defined as

W_i, W_j = corresponding weights
Z_i, Z_j = heights of masses M_i and M_j, respectively
d_{S} = displacement induced by design seismic action
= $q_{\mathrm{d}} d_{\mathrm{e}} \gamma_{\mathrm{I}}$
q_{d} = displacement behaviour factor
d_{e} = displacement from the linear analysis
F_{a} = horizontal force on non-structural element
W_{a} = weight of the element
q_{a} = $\bar{B}$ = behaviour factor
= 1.0–2.0 = q

Storey drift/*P*–Δ effect

P–Δ effect is not considered if the following is satisfied:

$$\phi = \frac{P_{\mathrm{tot}} \cdot d_r}{V_{\mathrm{tot}} \cdot h_i} \leq 0.10 \tag{2.77}$$

d_r $= \Delta$ in other literature
h_{i} $=$ interstorey height
P_{tot} $=$ total gravity load at and above the storey
V_{tot} $=$ total seismic storey shear

For buildings with non-structural elements

$$\frac{d_r}{\bar{R}_d} \leq 0.004\, h \tag{2.78}$$

When structural deformation is restricted

$$\frac{d_r}{\bar{R}_d} \leq 0.006\, h \tag{2.79}$$

where $0.1 < \theta \leq 0.2$ increases the seismic action effects by

$$\frac{1}{(1-\theta)} \ngtr 0.3 \tag{2.80}$$

Seismic factors

$I = \gamma_{\mathrm{I}} = 0.8{-}1.5$
$B_0 = 5\%$ viscous damping
$\xi =$ damping acceleration factor ≥ 0.7
$S =$ soil parameter $= 1.0$ for 5% damping
$q = \bar{B} =$ behaviour factor

$$S_a = \text{seismic coefficient} = \frac{\alpha \times 3\left(1 + \frac{z}{h}\right)}{\left[1 + \left(1 - \frac{T_a}{T_1}\right)^2\right]} \tag{2.81}$$

$$a = \frac{a_g}{a} \tag{2.82}$$

T_a for non-structural elements
T for structural elements
z $=$ height of the non-structural element
$\bar{R}_D = v$

Site characteristics/building categories

Subsoil Class A

$$\begin{aligned} V_S &= 100\ \mathrm{m/s} \quad 5\,\mathrm{m} \\ &= 400\ \mathrm{m/s} \quad 10\,\mathrm{m} \end{aligned}$$

Table 2.15 *K*-values

Class	K_{d1}	K_{d2}	K_1	K_2
A	2/3	5/3	1	2
B	2/3	5/3	1	2
C	2/3	5/3	1	2

Table 2.16 S_1, B_0, T_B, T_C, T_D and $\bar{R}_D$ values

S_1	B_0	T_B	T_C	T_D
1.0	2.5	0.10	0.40	3.0
1.0	2.5	0.15	0.60	3.0
0.9	2.5	0.20	0.80	3.0

Subsoil Class B

$$\begin{aligned} V_S &= 200\ \text{m/s} \quad 10\,\text{m} \\ &= 350\ \text{m/s} \quad 50\,\text{m} \end{aligned}$$

Subsoil Class C

$$V_S = 200\ \text{m/s}\ 20\,\text{m}$$

Ground acceleration $a_g \ngtr 0.10$ g

$$d_g = \text{peak ground displacement} = 0.5 a_g S \cdot T_C T_D \tag{2.83}$$

Building categories versus $\bar{R}_D$

	$\bar{R}_D$
I	2.5
II	2.5
III	2.0
IV	2.0

2.1.5.1 Symbols

In addition to the symbols listed in Part 1-1, the following symbols are used in Part 1-2 with the following meanings:

E_E effect of the seismic action;
E_{Edx}, design values of the action effects due to the horizontal components of the seismic;
E_{Edy} action;
E_{Edz} design value of the action effects due to the vertical component of the seismic action;
F horizontal seismic force;

F_a	horizontal seismic force acting on a non-structural element (appendage);
H	building height;
R_d	design resistance;
T_1	fundamental vibration period of a building;
T_a	fundamental vibration period of a non-structural element (appendage);
W	weight;
W_a	weight of a non-structural element (appendage);
d	displacement;
d_r	design interstorey drift;
e_1	accidental eccentricity of a storey mass from its nominal location;
h	interstorey height;
m	mass;
q_a	behaviour factor of a non-structural element;
q_d	displacement behaviour factor;
s	displacement of a mass m in the fundamental mode shape of a building;
z	height of the mass m above the level of application of the seismic action;
γ_a	important factor of a non-structural element;
θ	interstorey drift sensitivity coefficient.

2.1.5.2 Characteristics of Earthquake-Resistant Buildings

Basic Principles of Conceptual Design

(1) P The aspect of seismic hazard shall be taken into consideration in the early stages of the conceptual design of the building.

(2) The guiding principles governing this conceptual design against seismic hazard are

- structural simplicity,
- uniformity and symmetry,
- redundancy,
- bidirectional resistance and stiffness,
- torsional resistance and stiffness,
- diaphragmatic action at storey level,
- adequate foundation.

(3) Commentaries to these principles are given in Annex B.

Structural Regularity

General

(1) P For the purpose of seismic design, building structures are distinguished as regular and non-regular.

(2) This distinction has implications on the following aspects of the seismic design:
- method of analysis such as power spectrum, non-linear time history and frequency domain
- the value of the behaviour factor 'q'
- geometric non-linearity exceeding the limit by the Eurocode 8
- Non-regular distribution of overstrength in elevation exceeding the limit by Eurocode 8
- Criteria describing regularity in plan and in elevation

Safety Verifications

General

(1) P For the safety verifications the relevant limit states and specific measures (see Clause 2.2.4 of Part 1-1) shall be considered.

(2) For building of importance categories II–IV (see Table 3.3) the verifications prescribed in Sects. 4.2 and 4.3 may be considered satisfied if the following two conditions are met:

(a) The total base shear due to the seismic design combination (see Clause 4.4 of Part 1-1), calculated with a behaviour factor $q = [1,0]$, is less than that due to the other relevant action combinations for which the building is designed on the basis of a linear elastic analysis.

(b) The specific measures described in Clause 2.2.4 of Part 1-1 of the code are taken, with the exception that the provisions contained in Clause 2.2.4.1 (2)–(3) of Part 1-1 need not be demonstrated as having been met.

Ultimate Limit State

General

(1) P The safety against collapse (ultimate limit state) under the seismic design situation is considered to be ensured if the following conditions regarding resistance, ductility, equilibrium, foundation stability and seismic joints are met.

Resistance Condition

(1) P The following relation shall be satisfied for all structural elements – including connections – and the relevant non-structural elements:

$$E_d \leq R_d \tag{2.84}$$

where

$$E_d = E\left\{\sum G_{kj}, \gamma_I \cdot A_{ED}, P_K, \sum \psi_{2i} \cdot Q_{ki}\right\}$$

is the design value of the action effect due to the seismic design situation (see Clause 4.4 of Part 1-1), including – if necessary – second-order effects, and

$$R_d = R\{f_k/\gamma_M\} \tag{2.85}$$

is the corresponding design resistance of the element, calculated according to the rules specific to the pertinent material (characteristic value of property f_k and partial safety factor γ_M) and according to the mechanical models which relate to the specific type of structural system.

(2) Second-order effects ($P - \Delta$ effects) need not be considered when the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{\text{tot}} \cdot d_r}{V_{\text{tot}} \cdot h} \leq 0.10 \tag{2.86}$$

where

- θ interstorey drift sensitivity coefficient,
- P_{tot} total gravity load at and above the storey considered, in accordance with the assumptions made for the computation of the seismic action effects,
- d_{r} design interstorey drift, evaluated as the difference of the average lateral displacements at the top and bottom of the storey under consideration,
- V_{tot} total seismic storey shear,
- h interstorey height.

(3) In cases when $0.1 < \theta \leq 0.2$, the second-order effects can approximately be taken into account by increasing the relevant seismic action effects by a factor equal to $1/(1 - \theta)$.

(4) P The value of the coefficient θ shall not exceed 0.3.

Ductility Condition

(1) P It shall be verified that both the structural elements and the structure as a whole possess adequate ductility taking into account the expected exploration of ductility, which depends on the selected system and the behaviour factor.

(2) P Specific material-related requirements as defined in Part 1-3 shall be satisfied, including – when indicated – capacity design provisions in order to obtain the hierarchy of resistance of the various structural components necessary for ensuring the intended configuration of plastic hinges and for avoiding brittle failure modes.

(3) P Capacity design rules are presented in detail in Part 1-3.

Equilibrium Condition

(1) P The building structure shall be stable under the set of actions given by the combination rules of Clause 4.4 of Part 1-1. Herein are included such effects as overturning and sliding.
(2) P In special cases the equilibrium may be verified by means of energy balance methods or by geometrically non-linear methods with the seismic action defined as described in Clause 4.3.2 of Part 1-1 of the code.

Resistance of Horizontal Diaphragms

(1) P Diaphragms and bracings in horizontal planes shall be able to transmit with sufficient overstrength the effects of the design seismic action to the various lateral load resisting systems to which they are connected.
(2) Paragraph (1) is considered satisfied if for the relevant resistance verifications the forces obtained from the analysis are multiplied by a factor equal to 1.3.

Resistance of Foundations

(1) P The foundation system shall be verified according to Clause 5.4 of Part 5 and to Eurocode 7.
(2) P The action effects for the foundations shall be derived on the basis of capacity design considerations accounting for the development of possible overstrength, but they need not exceed the action effects corresponding to the response of the structure under the seismic design situation inherent to the assumption of an elastic behaviour ($q = 1.0$).
(3) If the action effects for the foundation have been determined using a behaviour factor q $\leq$ [1, 5], no capacity design considerations according to (2) P are required.

Seismic Joint Condition

(1) P Building shall be protected for collisions with adjacent structures induced be earthquakes.
(2) Paragraph (1) is deemed to be satisfied if the distance from the boundary line to the potential points of impact is not less than the maximum horizontal displacement.
(3) If the floor elevations of a building under design are the same as those of the adjacent building, the above referred distance may be reduced by a factor of [0,7].
(4) Alternatively, this separation distance is not required if appropriate shear walls are provided on the perimeter of the building to act as collision walls ("bumpers"). At least two such walls must be placed at each side subject to pounding and must extend over the total height of the building. They must be perpendicular to the side subject to collisions and they can end on the boundary line. Then the separation distance for the rest of the building can be reduced to [4,0] cm.

Serviceability Limit State

General

(1) P The requirement for limiting damage (serviceability limit state) is considered satisfied if – under a seismic action having a larger probability of occurrence than the design seismic action – the interstorey drifts are limited according to 4.3.2 of the code.
(2) Additional verifications for the serviceability limit state may be required in the case of buildings important for civil protection or containing sensitive equipment.

Limitation of Interstorey Drift

(1) P Unless otherwise specified in Part 1-3, the following limits shall be observed:

(a) for buildings having non-structural elements of brittle materials attached to the structure

$$d_r/v \leq [0,004] \cdot h \tag{2.87}$$

(b) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations

$$d_r/v \leq [0,006] \cdot h \tag{2.88}$$

where

d_{r} design interstorey drift as defined in 4.2.2.(2) of the code,
h storey height,
v reduction factor to take into account the lower return period of the seismic event associated with the serviceability limit state.

(2) The reduction factor can also depend on the importance category of the building. Values of v are given in Table 2.17.
(3) Different values of v may be required for the various seismic zones of a country. The code provides methodologies in detail for buildings and their elements made in concrete steel, timber and masonry. Design concepts, material properties, building systems, dissipative zones and structural types of behaviour factors are dealt with in greater depths in the code.

Table 2.17 Values of the reduction factor v

Importance category	I	II	III	IV
Reduction factor v	[2,5]	[2,5]	[2,0]	[2,0]

2.1.6 India and Pakistan: IS-1893 (1984) and PKS 395-Rev (1986)

Seismic actions/dynamic characteristics

T (moment-resisting frame, shear walls) $= 0.1n$

$$T\text{ (other buildings) } = 0.09H/\sqrt{d} \tag{2.89}$$

$n =$ number of storeys
$d =$ maximum base dimension
$H =$ height of the building

Response spectrum

$$S_a/g \text{ versus } T \text{ when } \xi = 5\%,\ \xi = 10\%,\ \xi = 20\% \tag{2.90}$$

$F_{ir} =$ seismic design lateral force at the ith floor level corresponding to the rth mode

$$= K\beta IF_0\phi_{ir}C_r\frac{S_{ar}}{g}W_i \tag{2.91}$$

$\phi_{ir} =$ mode shape coefficient

$$C_r = \sum_{j=i}^{n}\frac{W_j\phi_{jr}}{\sum_{j=1}^{n}W_j\left[\phi_{jr}\right]^2} \tag{2.92}$$

Seismic weights, forces and moments

$$V = \text{design base shear} = KC\beta I\alpha_0 W \quad \text{(India)} \tag{2.93}$$

$$V = \text{design base shear} = C_S\omega_t \quad \text{(Pakistan)} \tag{2.94}$$

$$C_S = ZISM\gamma_d Q \tag{2.95}$$

$$Z = ACF \tag{2.96}$$

Table 2.18 S_a/g versus T(s)

S_a/g	T(s)	T(s)
0	0.16	0.2
0.1	1.00	1.9
0.2	0.30	1.18
0.3	_	0.80
0.4	_	0.60
0.5	_	0.50
0.6	_	0.40
0.7	_	0.15

From the above, the average acceleration coefficient S_a/g is obtained.

$$F_i = V\frac{W_i h_i^2}{\sum_{j=1}^{n} \omega_j h_j^2} \tag{2.97}$$

W = total load = dead + appropriate live loads
F_i = lateral force at the ith floor = P_i
W_i = gravity load
h_i = from the base to the ith floor
n = number of storeys = N
W_j = individual floor load

India

$$F_i = \text{force in the } i\text{th frame to resist torsion} = \frac{M_t(K_i r_i)}{\sum K_i r_i^2} \tag{2.98}$$

where

K_i = stiffness of the ith frame
r_i = distance of the ith frame from the centre of the stiffness

$$M_t = \text{torsional moment} = 1.5\,eV \tag{2.99}$$

Pakistan

$$W_i = D_i + nL_i \tag{2.100}$$

$$\eta = 0.25\text{–}0.50$$

$$M_i = \text{overturning moment} \tag{2.101}$$

$$= \sum_{i+1}^{n} F_i(h_i - h_j) + F_i(h_n - h_j)$$

Torsional effects

$$e_a = 1.5e + 0.1b \quad \text{or} \quad e_a = e - 0.1b \tag{2.102}$$

b = the largest distance or dimension
e = eccentricity
e_a = eccentricity for torsional moment
L_i = live load at ith level

Storey drift/P–Δ effect

Δ_{max} between two floors $\ngtr 0.004 \times h_i$ for height >40 m (India)

$$\left.\begin{aligned} &\delta_i = \frac{g}{4\pi^2}\frac{T^2 F_i}{W_i} \\ &\Delta = \delta_i - \delta_{i-1} \qquad \delta_0 = 0 \\ &P - \Delta \text{ effect} \\ &\theta_\mathrm{i} = \frac{W_i \Delta_j}{V_i h_i} \ngtr 0.3 \end{aligned}\right\} \text{(Pakistan)} \tag{2.103}$$

Seismic factors

F_0 = seismic zone factor
α_0 = basic horizontal seismic coefficient
$C = 5\%$
$A = 0\text{–}0.08$
M = material factor = 0.8–1.2
Q = construction factor = 1.0
$S = 0.67\text{–}3.2$
$\alpha_0 = 0.01\text{–}0.08$
I = importance factor = 1.0–1.5
β = 0.01–0.08

For different soil foundations:

Site characteristics/building categories

Table 2.19 Height versus γ_d

γ_d	Height (m)
0.4	up to 20
0.6	40
0.8	60
1.0	90

Table 2.20 Values of α_0 and F_0 for various zones

Zone	α_0	F_0
V	0.08	0.40
IV	0.05	0.25
III	0.04	0.20
II	0.02	0.10
I	0.01	0.05

2.1.7 Iran: ICRD (1988)

Seismic actions/dynamic characteristics

Design methodologies: Analyses

(a) Equivalent static
(b) Pseudo-dynamic

(c) Dynamic analysis using acceleration data

$$T = 0.09\frac{h}{\sqrt{L}} \not> T \leq 0.06\,h^{3/4} \tag{2.104}$$

$$T = 0.08\,h^{3/4} \quad \text{(steel frame)} \tag{2.105}$$

$$T = 0.07\,h^{3/4} \quad \text{(RC frame)} \tag{2.106}$$

$$F_V = \text{vertical seismic action} = \frac{2AI}{R_V} W_P \tag{2.107}$$

R_V = reaction coefficient
= 2.4 for steel
= 2.0 for concrete

$$W_p = G_i + L_i + \text{total} \tag{2.108}$$

Seismic weights, forces and moments

$$V = \text{minimum base shear force} = CW_t \tag{2.109}$$

$$C = \frac{A\bar{R}I}{\bar{B}} \tag{2.110}$$

Lateral forces

$$F_i = (V - F_t)\frac{W_i h_i}{\sum_{j=1}^{n} W_j h_j} \tag{2.111}$$

F_t = additional lateral force at top level

$$= 0 \quad \text{if } T \leq 0.7\,\text{s} \tag{2.112}$$

$$= 0.07 \quad \text{if } T_V \leq 0.25\,V \tag{2.112a}$$

$$M_i = F_t(h_N - h_i) + \sum_{j=i+1}^{N} F_j(h_j - h_i) \quad i = 0 \text{ to } N-1 \tag{2.113}$$

$$M_{ti} = \sum_{j=1}^{n} e_{ij}F_j + M_{\text{ta}} \tag{2.114}$$

M_{ta} = accidental torsional moment

Storey drift/P–Δ effect

The lateral drift is $\not>$ 0.005h_i. Both lateral forces and torsional moment effects are coupled.

Seismic factors

$$W_t = G_i + \eta L_I \tag{2.115}$$

$$\eta = 20\text{–}40\% \tag{2.116}$$

$$A = \text{design base acceleration} = 0.35\text{–}0.20 \tag{2.117}$$

$$\bar{R} = 2.0\left(\frac{T_0}{T}\right)^{2/3} \quad 0.6 \leq \bar{R} \leq 2.0 \tag{2.118}$$

$$I = 0.8\text{–}1.2$$

$$\bar{B} = 5\text{–}8$$

Site characteristics/building categories

1. High-priority buildings
2. Medium-priority buildings
3. Low-priority buildings

Classification

(a) Regularity in plan
(b) Regularity in elevation

Soil Classification
I to IV where

$$\begin{aligned} T_0 &= \text{characteristic period on site} \\ &= 0.3\text{–}0.7 \end{aligned}$$

2.1.8 Israel: IC-413 (1994)

Seismic actions/dynamic characteristics

$$\begin{aligned} T &= 0.073\,h^{3/4} \quad \text{(concrete)} \\ &= 0.085\,h^{3/4} \quad \text{(steel)} \\ &= 0.049\,h^{3/4} \quad \text{(others)} \end{aligned} \tag{2.119}$$

Vertical seismic action

$$F_r = \pm\frac{2}{3}ZW \quad \text{cantilevers} \tag{2.120}$$

$$= W_{\min} - 1.52ISW \quad \text{for concrete beams} \tag{2.121}$$

Modal lateral force $= F_{im}$ at level i

$$F_{im} = \frac{V_m[W_i\phi_{im}]}{\sum_{i=1}^{n} W_i\phi_{im}} \tag{2.122}$$

Modal displacements

$$\delta_{im(max)} = \pm K \sum_{j=0}^{i} \Delta_{jm} \tag{2.123}$$

or

$$\delta_{im(max)} = \frac{g}{4\pi^2}\frac{T_m^2 F_{im}}{W_i} \tag{2.124}$$

where $T_m = m$th natural period.

Seismic weights, forces and moments

$$V = C_d \sum_{i=1}^{N} W_i \tag{2.125}$$

$$\left.\begin{array}{l} C_d = \dfrac{R_a IZ}{\bar{R}} \\ \text{or } \geq \dfrac{SIZ}{\sqrt{3\bar{R}}} \end{array}\right\} \ngtr \begin{array}{ll} 0.3I & \text{low} \\ 0.2I & \text{medium} \\ 0.1I & \text{high ductility} \end{array} \tag{2.126}$$

Lateral forces

$$F_t = 0.07TV \leq 0.25V \tag{2.127}$$

$$F_i = \frac{(V - F_t)W_i h_i}{\sum_{i=1}^{N} W_i h_i} \quad \text{at the top level } F_N + F_t \tag{2.128}$$

$$\begin{aligned} e &= \text{torsional accidental eccentricity} \\ &= \pm 0.05L \end{aligned} \tag{2.129}$$

$$3.0 \geq A_T = 2.75\left(\frac{\delta_{max}}{\delta_{max} + \delta_{min}}\right)^2 \geq 1.0 \tag{2.130}$$

$$\begin{aligned} \xi &= \text{multiplying factor applied to lateral load at each stiffness} \\ &\quad \text{element to account for torsional effect} \\ &= 1.0 + 0.6\tfrac{y}{L} \end{aligned} \tag{2.131}$$

Modal overturning moment

$$M_{i\mathrm{m}} = \sum_{j=i+1}^{N} F_{\mathrm{pm}}\left(h_j - h_i\right) \tag{2.132}$$

Modal torsional moment

$$= M_{\mathrm{t}i\mathrm{m}} = (d_i \pm e_i)V_{i\mathrm{m}} \tag{2.133}$$

$d_i =$ eccentricity
$e_i =$ accidental eccentricity

Modal weight

$$W_{\mathrm{m}} = \frac{\left(\sum_i W_i \phi_{i\,\mathrm{m}}\right)^2}{\sum_i W_i \phi_{i\,\mathrm{m}}^2} \tag{2.134}$$

$\phi_{i\mathrm{m}} =$ amplitude at ith level of mth mode

Storey drift/$P{-}\Delta$ effect

$P{-}\Delta$ effect

$$\theta_i > 1.0 \tag{2.135}$$

$$\theta_i = W\Delta_{\mathrm{el},i}\frac{K}{V_i h_i} \tag{2.136}$$

$$V_i = \sum_{j=i}^{N} F_j \quad \text{(storey shear force)} \tag{2.137}$$

$$\Delta_{\mathrm{el},i} = \left[\sum_{m=1}^{N}\left(\Delta_{\mathrm{el},\,i\mathrm{m}}\right)^2\right]^{1/2} \tag{2.138}$$

$$\begin{aligned} \Delta_{\mathrm{el},\,i\mathrm{m}} &= \text{elastic modal drift at the level of } i \\ V_i &= \text{modal shear force at } i \\ &= \left[\sum_{m=1}^{N}\left(V_{i\mathrm{m}}\right)^2\right]^{1/2} \end{aligned} \tag{2.139}$$

Drift limitations

$$\text{for } T \leq 0.7\,\mathrm{s} \quad \Delta_{i,\mathrm{lim}} = \min\left(\frac{h_i}{40K}; \quad \frac{h_i}{200}\right) \tag{2.140}$$

$$\text{for } T > 0.7\,\text{s} \quad \Delta_{i,\text{lim}} = \min\left(\frac{0.75h_i}{40K};\quad \frac{h_i}{250}\right) \tag{2.141}$$

$$\text{maximum displacement } \delta_{t_{\max}} = \pm K \sum_{i=0}^{i} \Delta_i \tag{2.142}$$

$\Delta_i =$ computed interstorey displacement

Storey drift

$$\Delta_{i\text{m}} = \delta_{i\text{m}} - \delta_{(i-j)\text{m}} \tag{2.143}$$

Seismic factors

$$\begin{aligned}
\bar{R} &= \text{steel } 4\text{–}8, \text{ concrete } 3.5\text{–}7 \\
W_i &= G_i + K_g(Q_i + A_i \cdot q_i) \qquad (2.144) \\
Q_i &= \text{concrete load} \\
q_i &= \text{UDL} \\
A_i &= \text{area} \\
K_g &= \text{live load factor} \\
&= 0.2 \quad (\text{dwellings}) \\
&= 0.5 \quad (\text{stores, etc.}) \\
&= 1.0 \quad (\text{storage}) \\
I &= 1.0\text{–}1.4 \\
C_\text{d} &= \text{seismic coefficient} \\
R_\text{a} &= \text{spectral amplification factor}
\end{aligned}$$

$$\begin{aligned}
R_\text{a}(T) &= \frac{1.255}{T^{2/3}} \\
2.5 \geq R_\text{a}(T) &\geq 0.2\,K
\end{aligned} \tag{2.145}$$

Table 2.21 Z values for various zones

Seismic zones	Z
I	0.075
II	0.075
III	0.10
IV	0.15
V	0.25
VI	0.30

$S = 1.0\text{–}2.0$

Site characteristics/building categories

Regular structures
Category B < 80 m high
Category C < 80 m high

with normalized seismic zones $Z \leq 0.075$.

2.1.9 *Italy: CNR-GNDT (1986) and Eurocode EC8 is Implemented*

Seismic actions/dynamic characteristics

Structure	Maximum height (m)		
Frame	$S = 6$	$S = 9$	$S = 12$
Frame	No limitation		
Masonry	16.0	11.0	7.5
Walls	32.0	25.0	15.0
Timber	10.0	7.0	7.0

Seismic index $S = 6$, 9 and 12
Fundamental period T_0

$$T_0 > 0.8\,\mathrm{s} \quad \bar{R} = 0.862/T_0^{2/3} \tag{2.146}$$

$$T_0 \leq 0.8\,\mathrm{s} \quad \bar{R} = 1.0 \tag{2.147}$$

$$T_0 = 0.1\frac{h}{\sqrt{L}} \quad \text{for a framed structure} \tag{2.148}$$

Lateral and vertical effects

$$\alpha = \left[\alpha_h^2 + \alpha_v^2\right]^{1/2} \tag{2.149}$$

$$\bar{\eta} = \left[\eta_h^2 + \eta_v^2\right]^{1/2} \tag{2.150}$$

$\alpha =$ single force component
$h, v =$ subscripts for horizontal and vertical
$\eta =$ single displacement component

Combination of modal effects

$$\alpha = \sqrt{\sum_{\alpha} \alpha_i^2} \tag{2.151}$$

$$\eta = \sqrt{\sum_n n_i^2} \tag{2.152}$$

α_{tot} = combined total force = $\alpha \pm \alpha_{\text{p}}$
α_{p} = action due to non-seismic loads such as permanent loads and live load fraction
$\alpha_{\text{tot}} = \alpha \pm \alpha_{\text{p1}}$ (non-seismic loads)
$= \alpha \pm \alpha_{\text{p2}}$ (fraction of live load)
η_{r} = actual displacement for design purpose in elastic situation
$= \eta_{\text{p}} \pm \phi\eta$
η_{p} = elastic displacement due to non-seismic loads
$\phi = 6$ if displacements obtained from static analysis
$= 4$ if displacements obtained from dynamic analysis

2.1.10 Japan: BLEO (1981)

Seismic actions/dynamic characteristics

R_{t} = spectral coefficient = 0.4

$$= 1 - 0.2\left(\frac{T}{x} - 1\right)^2 \tag{2.153}$$

$x = 0.4$ (hard soil)
$= 0.6$ (medium soil)
$= 0.8$ (soft soil)
$T = h(0.02 + 0.18)$ s
h = full height of the building

$$Q_{bi} = (1 + 0.7\beta_i)Q_i \tag{2.154}$$

$$Q_{bi} \leq 1.5Q_i \tag{2.154a}$$

Q_{bi} = lateral shear strength

$$\beta_i = \frac{\text{lateral shear in bracings}}{\text{total storey shear}} \tag{2.155}$$

$$Q_i = C_i\bar{W}_i \tag{2.156}$$

$$Q_{\text{u}i} = D_{\text{S}}F_{\text{es}}Q_i \quad \text{[ultimate shear strength (lateral)]} \tag{2.157}$$

D_{S} = structural coefficient

$$F_{\mathrm{es}} = F_{\mathrm{E}} \cdot F_{\mathrm{S}} \tag{2.158}$$

Seismic intensity

Elastic response	0.15–0.25g
Elasto-plastic response	0.30–0.5g

Seismic weights, forces and moments

Lateral seismic shear force

$$Q_i = C_i \bar{W}_i \tag{2.159}$$

$$\bar{W}_i = \text{portion of the total seismic weight at the level } i$$

$$C_i = Z R_{\mathrm{t}} A_i C_0 \tag{2.160}$$

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \frac{2T}{1 + 3T} \tag{2.161}$$

$$\alpha_i = \frac{\bar{W}_i}{W_0} = 0{-}1.0 \tag{2.162}$$

W_0 = weight above ground floor
$C_0 = 0.2$ (moderate earthquake motion)
$= 1.0$ (severe earthquake motion)

$$q = KW \tag{2.163}$$

$$M_i = \sum_{j=i+1}^{n} F_j (h_j - h_i) \tag{2.164}$$

or

$$= \sum_{j=i+1}^{n} Q_j h_j \tag{2.165}$$

h_j = interstorey height

$$Q_{\mathrm{B}} = \text{horizontal seismic shear at the basement} = Q_{\mathrm{p}} + K W_{\mathrm{B}} \tag{2.166}$$

W_{B} = weight of the basement
q = horizontal seismic shear force
$= KW$ (for appendages such as penthouse, chimneys)

Torsional stiffness

$$r_{ex} = \left[\frac{J_{xy}}{\sum_{i=1}^{N_x} K_{xi}}\right]^{1/2} \tag{2.167}$$

$$r_{ey} = \sqrt{\frac{J_{xy}}{\sum_{i=1}^{N_y} K_{yi}}} \tag{2.168}$$

Eccentricity ratio

$$R_{ex} = \frac{e_x}{\gamma_{ex}} \leq 0.15 \tag{2.169}$$

$$R_{ey} = \frac{e_y}{\gamma_{ey}} \leq 0.15 \tag{2.170}$$

For the x and y directions

$$J_{xy} = \sum_{i=1}^{N_x} K_{xi} y_i^2 + \sum_{i=1}^{N_y} K_{yi} x_i^2 \tag{2.171}$$

where N_x, N_y = number of resisting elements.

Connections

$$M_u = \alpha M_p \tag{2.172}$$

$\alpha = 1.2{-}1.3$
M_p = full plastic moment of the column or beam
M_u = maximum bending strength of the connection

Storey drift/P–Δ effect

$$\text{Seismic lateral forces} = F_i = Q_i - Q_{i+1} \tag{2.173}$$

$\Delta \not> \frac{1}{200} h_i$ or equal to $\frac{1}{120} h_i$ for non-structural elements for buildings not exceeding 60 m height, i.e. $h \not> 60$ m.

For steel buildings $\not> 31$ m

$$\Delta \not> \frac{1}{200} h_i \tag{2.174}$$

$$R_e \leq 0.15 \quad R_s \geq 0.6$$

Q_i is increased by Q_{bi}

Seismic factors

R_t = spectral coefficient
A_i = distribution factor
Z = seismic coefficient = 0.7–1.0

Seismic coefficient K of the basement

$$\geq 0.1\left(1-\frac{H}{40}\right)\times Z \tag{2.175}$$

H = depth of the basement in metres ($\ngtr 20\,\mathrm{m}$).

Site characteristics/building categories

$$\text{Type 1} \quad \text{Hard soil} \quad \frac{0.64}{T} \tag{2.176}$$

$$\text{Type 2} \quad \text{Medium soil} \quad \frac{0.96}{T} \tag{2.177}$$

$$\text{Type 3} \quad \text{Soft soil} \quad \frac{1.28}{T} \tag{2.178}$$

T = 0–2.0 s
T_C for Type 1 soil = 0.4
T_C for Type 2 soil = 0.6
T_C for Type 3 soil = 0.8

D_S = structural coefficients
= 0.25–0.5 (for steel)
= 0.30–0.55 (for RC)

F_e = function of eccentricity
= 1.0 for $R_e < 0.15$
= 1.5 for $R_e > 0.3$

$$\begin{aligned} F_S &= \text{function of stiffness ratio } R_x \\ &= 1.0 \quad R_S > 0.6 \\ &= 1.5 \quad R_S < 0.3 \end{aligned} \tag{2.179}$$

Buildings
Those of one to two-storeys: wooden $\ngtr 500\,\mathrm{m}^2$
Special building $\ngtr 100\,\mathrm{m}^2$

RC buildings

$$25\sum A_{\mathrm{W}} + 7\sum A_{\mathrm{C}} \geq Z\bar{W}_i A_i \quad \text{for} \quad h \not> 20 \text{ m} \tag{2.180}$$

$$25\sum A_{\mathrm{W}} + 7\sum A_{\mathrm{C}} + 10\sum A'_{\mathrm{C}} \geq 0.75 Z A_i \bar{W}_i \quad \text{for} \quad h \not> 31 \text{ m} \tag{2.181}$$

Steel buildings

(1) Not exceeding three storeys
$h \leq 13$ m span ≤ 6 m
total floor area ≤ 500 m^2
$C_0 = 0.3$
(2) Not exceeding 60 m

$$Q_{\mathrm{u}i} \leq Q_{\mathrm{u}} \tag{2.182}$$

$\sum A_{\mathrm{C}}$ = summation of total column areas
$\sum A_{\mathrm{W}}$ = summation of total horizontal cross-sectional areas of walls

2.1.11 Mexico: UNAM (1983) M III (1988)

Seismic actions/dynamic characteristics

$$T_0 = 4\sum_i \frac{t_i}{\beta_{1i}} \tag{2.183}$$

Design spectra
Referring to Fig. 2.3:

$$a = \left(1 + \frac{3T}{T\alpha}\right)\frac{C}{4} \quad \text{for } T \leq T_{\mathrm{a}} \text{ s} \tag{2.184}$$

$$a = C \qquad \text{for } T_{\mathrm{a}} < T \leq T_{\mathrm{b}} \text{ s} \tag{2.185}$$

$$a = \left(\frac{T_{\mathrm{b}}}{T}\right)^r c \qquad \text{for } T > T_{\mathrm{b}} \text{ s} \tag{2.186}$$

$$T \text{ (reduced lateral forces)} = 0.63\left(\sum_{i=1}^{N} \frac{W_i x_i^2}{g\sum_{i=1}^{N} F_i x_i}\right)^{1/2} \tag{2.187}$$

x = displacements
F_i = force at ith level
a = spectral acceleration

Dynamic analysis
Modal analysis accepted by the code:

$$V_0 \geq \frac{0.8aW_0}{Q'} \tag{2.188}$$

$$\begin{aligned} \bar{R} &= \text{total response} = S \\ &= \sqrt{\sum S_i^2} \\ S_i &= \text{modal responses} \end{aligned} \tag{2.189}$$

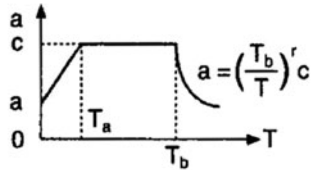


Fig. 2.3 Design spectra: acceleration versus time

Table 2.22

Zone	T_a	T_b	r	Seismic zone A	B	C	D
				c values			
I	0.2	0.6	$\frac{1}{2}$	0.08	0.16	0.24	0.48
II	0.3	1.5	$\frac{2}{3}$	0.12	0.20	0.30	0.56
III	0.6	3.9	1.0	0.16	0.24	0.36	0.64

Seismic weights, forces and moments

Seismic loads
Dead loads as prescribed (DL)
Live loads $= 90-180\,\text{kg/m}^2 (LL)$
Load combinations

$$\begin{aligned} \text{UIF} &= \text{ultimate internal forces} \\ &= 1.1(\text{DL} \pm \text{LL} \pm \text{ELL} \pm 0.3\text{ELT}) \end{aligned} \tag{2.190}$$

or

$$= 1.1(\text{DL} \pm \text{LL} \pm 0.3\text{ELL} \pm \text{ELT}) \tag{2.191}$$

where

ELL = earthquake loads (longitudinal)
ELT = earthquake loads (transverse)

Static method
Lateral forces

$$F_i = \frac{W_i h_i}{\sum_{j=1} W_j h_j} V_0 \tag{2.192}$$

$$V_0 = \frac{c}{Q'} W_0 \tag{2.193}$$

$W_0 = W_t =$ total weight

If $T < T_b \leftarrow$ delimiting period, the response spectrum F_i (reduced) will be obtained by changing

$$V_0 = \frac{a}{Q'} W_0 \tag{2.194}$$

If $T > T_b$

$$F_i(\text{reduced}) = W_i\left(K_i h_i + K_2 h_i^2\right) \frac{a}{Q'} \tag{2.195}$$

$$K_1 = \left(\frac{T_b}{T}\right)^r \left\{1 - r\left[1 - \left(\frac{T_b}{T}\right)^r\right]\right\} \frac{W_0}{\sum_{j=1}^{N} w_j h_j} \tag{2.196}$$

$$K_2 = 1.5r\left(\frac{T_b}{T}\right)\left\{1 - \left(\frac{T_b}{T}\right)^r\right\} \times \frac{W_0}{\sum_{j=1}^{N} w_j h_j^2} \tag{2.197}$$

Torsional effects

$$e = 1.5e_s + 0.1L \tag{2.198}$$

$$e = e_s - 0.1L \tag{2.199}$$

$e_s =$ static eccentricity
$e =$ torsional eccentricity
$L =$ plan dimensions of a storey

The factor of 1.5 is reduced to take into account dynamic modifications of torsional motion.

Overturning moment

$$M_i = \sum_{j=i+1}^{N} F_j\left(h_j - h_i\right) \quad i = 0 \text{ to } N-1 \tag{2.200}$$

$$M_i \text{ is multiplied by } R_\mathrm{m} = 0.8 + 0.2z \text{ where} \tag{2.201}$$

$$z = \frac{\text{height above ground}}{h}$$

Storey drift/P–Δ effect

$$\Delta \ngtr 0.006h_i \quad \text{(main structural elements)} \tag{2.202}$$

$$\Delta \ngtr 0.012 \qquad \text{(for partition)} \tag{2.203}$$

P–Δ effect

When $\Delta > 0.08V/W$, this effect should be considered (2.204)

V = calculated shear force
W = total weight of the part of the structure above that storey

Seismic factors

$$R_\mathrm{d} = \bar{Q} \tag{2.205}$$

$$Q' = Q \qquad T \geq T_\mathrm{a} \tag{2.206}$$

$$Q' = 1 + \left(\frac{T}{T_\mathrm{a}}\right) \times (Q - 1) \quad T < T_\mathrm{a} \tag{2.207}$$

$$Q = 4 \text{ to } 1 \quad \text{for } Q \geq 3 \tag{2.208}$$

The point of application of the shear force

$$e_\mathrm{r} \geq e_\mathrm{s} - 0.2L \quad \text{if } Q = 3 \tag{2.209}$$

$$e_\mathrm{r} \geq e_\mathrm{s} - 0.1b \quad \text{if } Q > 3 \tag{2.210}$$

Site characteristics/building categories

Building classification
Group A Important buildings
Group B Ordinary buildings

Table 2.23 Seismic zones: soil types

Seismic zones	Soil types
I	Hill zone, stiff rock, soft clays
II	Transition zone, sandy, silty sands < 20 m
III	Lake bed zone, clays, silty clay

sands > 50 m

$$\beta_i = \frac{\sum_i t_i}{\sum_i \frac{t_i}{\beta_{1i}}} \tag{2.211}$$

t_i = thickness of soil layers i
G_i = shear modulus tons/m^2
γ_i = unit weight tons/m^3

$$\beta_1 = \sqrt{\frac{gG_i}{\gamma_i}} \quad \text{for Class II} \tag{2.212}$$

$$\beta_1 < 700 - 550T_0 \tag{2.213}$$

For outside Mexico

$$a = a_0 + (c - a_0)\frac{T}{T_\mathrm{a}} \quad \text{for } T \leq T_\mathrm{a} \tag{2.214}$$

2.1.12 New Zealand: NZS 4203 (1992) and NZNSEE (1988)

Seismic actions/dynamic characteristics

Methods

(a) Equivalent static method
(b) Modal response spectrum
(c) Numerical integration time – history analysis

$$T = 2\pi \left[\frac{\sum_{i=1}^{N} W_i u_i^2}{\sum_{i=1}^{N} W_i u_i}\right]^{1/2} \tag{2.215}$$

$$T = 0.042 \tag{2.216}$$

$u_i = 1.0$ for one storey
$= 0.85$ for six or more storeys
= for fewer than six storeys interpolate between 1.0 and 0.85
u_i = lateral displacement at level i
δ_t = lateral displacement in millimetres at the top of the building

Modal response spectrum

$$C_1(T_\mathrm{m}) = S_\mathrm{m} C_\mathrm{b}(T_\mathrm{m}, 1)\bar{R}Zf_\mathrm{L} \tag{2.217}$$

S_m = response spectrum scaling factor
T_m = modal period

Seismic weights, forces and moments

Basic shear force

$$V = CW \tag{2.218}$$

W = total weight

$$C = C_\mathrm{b}(T, \mu)\bar{R}Zf_\mathrm{L} > 0.025 \quad T > 0.4\,\mathrm{s} \tag{2.219}$$

$$C = C_\mathrm{b}(0.4, \mu)RZf_\mathrm{L} > 0.025 \quad T \leq 0.4\,\mathrm{s} \tag{2.220}$$

μ = 6 (structural steel)
= 5 [concrete (RC or prestressed)]
= 4 (masonry)
= 3–4 (timber)

Equivalent static force F_i at level i

$$F_i = 0.92V \frac{W_i h_i}{\sum_{j=1}^{N} W_j h_j} \tag{2.221}$$

Seismic weight with additional top force

$$F_t = 0.08V \tag{2.222}$$

Storey drift/P–Δ effect

$$\Delta = \text{storey drift} \tag{2.223}$$

$$\frac{\Delta}{h} \quad 0, 0.2, 0.4, 0.6 \quad (\%) \tag{2.224}$$

$\Delta_\mathrm{max} = 600$

P–Δ effects
Ultimately it should satisfy

(a) $T < 0.4\,\mathrm{s}$ (2.225)
(b) $h \ngtr 15\,\mathrm{m}$ for $T < 0.8\,\mathrm{s}$ (2.226)
(c) $\mu < 1.5$
(d)

$$\frac{u_i - u_{i-1}}{h_i - h_{i-1}} \leq \frac{V_i}{7.5\sum_{j=1}^{N} W_i} \tag{2.227}$$

V_i = storey shear

$$= F_t + \sum_{i=1}^{N} F_j \tag{2.228}$$

Seismic factors

Risk factor $\bar{R}$ is given by

$$\begin{aligned} \mathrm{I} &= 1.3 \\ \mathrm{II} &= 1.2 \\ \mathrm{III} &= 1.1 \\ \mathrm{IV} &= 1.0 \\ \mathrm{V} &= 0.6 \\ C &= \text{lateral force coefficient} \\ f_{\mathrm{L}} &= \text{limit state} \\ &= 1.0 \quad \text{(ultimate)} \\ &= 1/6 \quad \text{(serviceability)} \\ Z &= \text{zone factor} \end{aligned}$$

$$\begin{aligned} C_{\mathrm{b}}(T,\mu) &= \text{basic seismic acceleration factor} \\ &= 0-1.0 \quad \text{for } T = 0-4\,\mathrm{s} \text{ and } \mu = 1.0-0.6 \end{aligned} \tag{2.229}$$

$$S_{\mathrm{m}_1} = \frac{C_{\mathrm{b}}(T,\mu)}{C_{\mathrm{b}}(T,1)} \text{ for } T > 0.4\,\mathrm{s} \tag{2.230}$$

$$S_{\mathrm{m}_1} = \frac{C_{\mathrm{b}}(0.4,\mu)}{C_{\mathrm{b}}(0.4,1)} \text{ for } T \leq 0.4\,\mathrm{s} \tag{2.231}$$

$$S_{\mathrm{m}_1} = 1.0 \text{ for the limit state of serviceability}$$

$$\mathrm{S}_{\mathrm{m}_2} = \frac{K_{\mathrm{m}} C W}{V_1} \tag{2.232}$$

$$K_{\mathrm{m}} = 0.8 \quad \text{for } \mu = 1.0$$

Site characteristics/building categories

Building categories

- I Important buildings
- II Holding crowds
- III Building with highly valued contents
- IV Buildings not included in any category
- V Secondary nature

Soil category

(1) Rock or stiff soils
(2) Normal soil sites
(3) Flexible deep soil sites

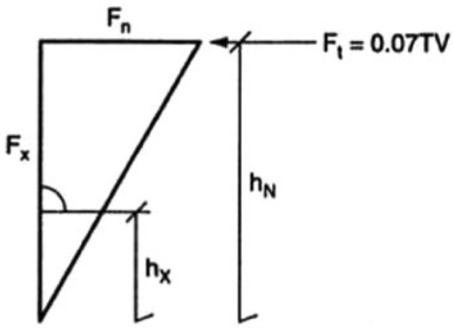


Fig. 2.4 Shear force distribution

2.1.13 USA: UBC-91 (1991) and SEAOC (1990)

Seismic actions/dynamic characteristics

Referring to Fig. 2.4

$$T = \text{fundamental period} = C_t[h_N]^{3/4} \tag{2.233}$$

$h_N = h =$ total height

$C_t = 0.035$ (steel moment-resisting frame)
$= 0.030$ (RC moment-resisting frame)
$= 0.020$ (for Rayleigh's formula)

Alternatively

$$T = 2\pi\left(\frac{\sum_{i=1}^{N} W_i\delta_i^2}{g\sum_{i=1}^{N} f_i\delta_i}\right)^{1/2} \tag{2.234}$$

$f_i =$ lateral force distribution
$V_x =$ storey shear force at x

$$= F_t + \sum_{i=x}^{N} F_i \tag{2.235}$$

F_x is evaluated under seismic weights.

Dynamic method
Using modal shapes

$$[K] - \omega^2[M]\{\phi\} = \{0\} \tag{2.236}$$

work out ω, f and T and finally modal shape $[\phi]$.

Spectral accelerations

Scaled down by peak one of $0.3g/R_\mathrm{w} = 12$:

$$S_\mathrm{a1} = 0.034g \quad S_\mathrm{a2} = 0.063g \tag{2.237}$$

$$S_\mathrm{a3} = 0.061g \quad S_\mathrm{a4} = 0.05g \tag{2.238}$$

Seismic weights, forces and moments

$$V = \text{basic shear force} = \frac{ZICW}{R_\mathrm{w}} \tag{2.239}$$

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \tag{2.240}$$

R_w = structural factor = 4–12

$$F_x = \frac{(V - F_\mathrm{t})W_x h_x}{\sum_{i=1}^{N} W_i h_i} \quad \text{at level } x \text{ from the base} \tag{2.241}$$

$$F_\mathrm{t} = 0.07TV \leq 0.25V \quad \text{for } T > 0.7\,\mathrm{s} \tag{2.242}$$

$$F_\mathrm{t} = 0 \quad \text{for } T \leq 0.7\,\mathrm{s}$$

$$V = F_\mathrm{t} + \sum_{i=1}^{N} F_i \tag{2.243}$$

Storey shear force

$$V_x = F_\mathrm{t} + \sum_{i=x}^{N} F_i \tag{2.244}$$

M_x = overturning moment, thus

$$M_x = F_\mathrm{t}(h_N - h_x) + \sum_{i=x+1}^{N} F_i(h_i - h_x) \quad \text{where } x = 0, 1, 2, \ldots, N-1 \tag{2.245}$$

Torsional moment

Torsional (based on UBC-91) irregularities in buildings are considered by increasing the accidental torsion by A_x (amplification factor)

$$A_x = \left(\frac{\delta_\mathrm{max} \text{ at } x}{1.20\delta_\mathrm{avg}}\right)^2 \leq 3.0 \tag{2.246}$$

δ_avg = the average displacement at extreme points at level x.

The floor and the roof diaphragms shall resist the forces determined by the following formula:

$$F_{px} = \frac{F_t + \sum_{i=x}^{N} F_i}{\sum_{i=x}^{N} W_i} W_{px} \tag{2.247}$$

where

W_{px} = weight of the diaphragm and attached parts of the building.

$$\text{Correspondingly } F_{px} \ngtr 0.75ZIW_{px} \text{ and } \nless 0.35ZIW_{px} \tag{2.248}$$

Storey drift/*P*–Δ effect

$$\text{Drift calculated} \ngtr \frac{0.04}{R_w} \times h_i \tag{2.249}$$

$$\text{or} \quad \ngtr 0.005h_i \quad \text{for } T < 0.7\,\text{s}$$

$$\text{or} \quad \ngtr 0.004h_i \quad \text{for } T \geq 0.7\,\text{s}$$

$P - \Delta$ effect

Secondary moment formula drift

Primary moment due to lateral forces

$$\frac{M_{xs}}{M_{sp}} \ngtr 0.10 \quad \text{not considered for greater values} \tag{2.250}$$

$$\theta_x = \frac{M_{xs}}{M_{sp}} = \frac{P_x \Delta_x}{V_x H_x} \tag{2.251}$$

Subscript x means to the level of storey x.

Elastic storey drift

$$\Delta_{ie} = \frac{V_i}{K_i} \tag{2.252}$$

$$\Delta_i = \text{inelastic storey drift}$$

$$= \frac{\Delta_{ie}}{K} \quad (K < 1.0) \tag{2.253}$$

or

$$\Delta_i = R\Delta_{ie} \tag{2.254}$$

$$\delta_i = \text{lateral displacement} = \sum_{i=1}^{N} \Delta_i \tag{2.255}$$

Total displacement

$$\delta_{px} = \delta_{ix} - \theta Y_p \tag{2.256}$$

$$\delta_{py} = \delta_{iy} + \theta Y_p \tag{2.257}$$

$$\begin{aligned} \delta_{ix}, \delta_{iy} &= \text{lateral displacement in } x \text{ and } y \text{ directions} \\ \theta &= \text{storey rotation} \\ \delta_{px}, \delta_{py} &= \text{lateral displacement in } x \text{ and } y \text{ directions} \\ \delta_{p} &= \text{total displacement at a selected point P} \\ &= \left(\delta_{px}^2 + \delta_{py}^2\right)^{1/2} \end{aligned}$$

Seismic factors

$$\begin{aligned} Z &= \text{seismic zone factor} \\ Z &= 0.075 \quad \text{Zone I} \\ &= 0.15 \quad \text{Zone IIA} \\ &= 0.20 \quad \text{Zone IIB} \\ &= 0.30 \quad \text{Zone III} \\ &= 0.40 \quad \text{Zone IV} \\ I &= \text{occupancy or importance factor} = 1.0{-}1.25 \end{aligned}$$

Site characteristics/building categories

S_1 type

$$S_a = 1 + 10T \quad 0 < T \le 0.15\,\text{s} \tag{2.258}$$

$$S_a = 2.5T \qquad 0.15 < T \le 0.39\,\text{s} \tag{2.259}$$

$$S_a = \frac{0.975}{T} \qquad T > 0.39\,\text{s} \tag{2.260}$$

S_2 type

$$\begin{aligned} S_a &= 1 + 10T \quad 0 < T \le 0.15\,\text{s}(2.261) \\ S_a &= 2.5T \quad 0.15 < T \le 0.585\,\text{s}(2.262) \end{aligned} \tag{2.261}$$

Table 2.24 Soils in seismic zones

Seismic zones		Soil S*
I	Rock like	1.0 (S_1)
IIA	Dense stiff	1.2 (S_2)
IIB		
II	Soft-medium clay	1.5 (S_3)
IV	Soft clay	2.0 (S_4)

S* = site coefficient

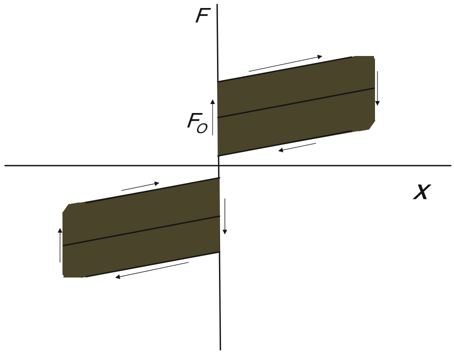


Fig. 2.5 F versus X

$$S_a = \frac{1.463}{T} \qquad T > 0.585\,\mathrm{s} \tag{2.262}$$

S_3 *type*

$$S_a = 1 + 75T \qquad 0 < T \leq 0.2\,\mathrm{s} \tag{2.263}$$

$$S_a = 2.5T \qquad 0.2 < T \leq 0.915\,\mathrm{s} \tag{2.264}$$

$$S_a = \frac{2.288}{T}T \qquad T > 0.915\,\mathrm{s} \tag{2.265}$$

K = spring parameter of the damper
C = viscous constant of the damper
α = parameter of the damper, α usually varies between 0.1 and 0.4

Parameters should be experimentally calibrated by the supplier.

Additional deliberations on viscoelastic dampers

A convenient way to improve the dynamic performance of a structure subjected to wind or earthquake loading vibration is to incorporate mechanical dampers to augment the structural damping. This damping increase yields a reduction in the expected structural damage through a significant reduction of the interstory drifts of the structure during the dynamic event.

Although the developments in research and analysis techniques, paralleled by significant improvements and refinements of device hardware, make the mechanical dampers totally suitable for consideration in new or retrofit design, there are still relatively few applications to buildings (Mahmoodi 1969; Aiken and Kelly 1990; Tsai and Lee 1993; Inaudi et al. 1993; Inaudi and Kelly 1995; Lai et al. 1995; Shen and Soong 1995; Makri et al. 1995).

Among a number of viable types of energy dissipation devices proposed, the viscoelastic dampers have found several successful applications for wind-induced vibration reduction of the tall buildings. Remarkable examples are the 110-story twin towers of the World Trade Center, in New York City, the

73-story Columbia SeaFirst Building and the 60-story Number Two Union Square Building, both in Seattle.

The implementation of viscoelastic dampers (VEDs) for seismic protection has been investigated only in the last few years (Zhang et al. 1989; Zhang and Soong 1992; Bergman and Hanson 1993; Hanson 1993; Tsai 1993; Chang et al. 1993; Kasai et al. 1993; Abbas and Kelly 1993; Chang et al. 1995; Munshi and Kasai 1995).

An accurate model for the mechanical behaviour of VEDs subjected to seismic loading must incorporate the variability of the material's physical properties with the deformation amplitude, the excitation frequency and the temperature conditions during dynamic service.

2.1.14 Codes Involving Seismic Devices and Isolation Techniques

2.1.14.1 General Introduction

Although the excessive vibrations can be based on active, semi-active or passive control techniques, the use of them depends on a number of factors. The passive control device is presently the most common, reliable and economic technical solution. Among many passive control devices are tuned mass dampers (TMDs), tuned liquid mass dampers (TLMDs) and fluid viscous dampers (FVDs). The general principles for the design of a damping system are

a. It should be accessible
b. It should have a low maintenance
c. Its design must take into account corrosion
d. Where high amplitude oscillations exist, buffers should be associated
e. It should allow a later adjustment
f. Its design must be accompanied with experimental tests

Different types of possible damping systems are discussed in this section.

2.1.14.2 Different types of Damper Devices

a. *Pendulum damper* (horizontal tuned mass damper)

For movement in the horizontal plane, a pendulum is clearly a simple system with one degree of freedom, the spring constant and a natural frequency, which depends on the length of its hanger l (g – acceleration due to gravity)

$$K_d = \frac{m_d g}{l}; \quad f_d = \frac{1}{2\pi}\sqrt{\frac{g}{l_d}}; \quad l_d = \frac{g}{4\pi^2 f_d^2} \tag{2.266}$$

where f_d = pendulum frequency
l_d = pendulum length
g = acceleration due to gravity

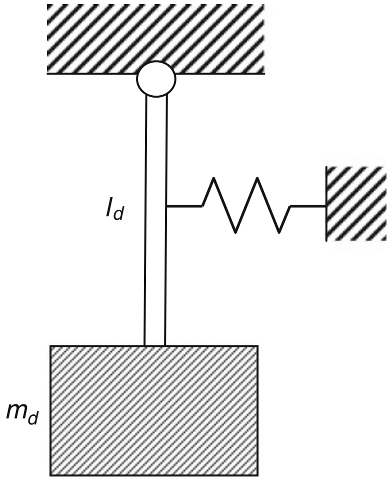


Fig. 2.6 Damper idealization

If the problem exists in construction and damping system, the pendulum length can be calculated as

$$l_d = \frac{g}{(2 \cdot \pi \cdot f)^2 - \frac{c}{m}} \tag{2.267}$$

For a given pendulum length, the spring constant c of the damping system can be derived as

$$c = \left[(2 \cdot \pi \cdot f)^2 - \frac{g}{l_d} \right] \cdot m \tag{2.268}$$

The necessary damping ξ_D can be provided by hydraulic dampers, friction-type dampers and the maximum swing period of the damper.

Absorber T_{abs} should be (Anagnostopoulos 2002)

$$T_D = 2(T_{\text{str}} + \tau) \tag{2.269}$$

where T_{str} is the period of the vibration of the structure; τ is the duration of the impact, introduced as 0.01–0.1 s.

b. *Ball damper*

The idea of the dynamic vibration absorber arranged as a rolling ball is not quite new and in the Czech Republic was already designed for the first time at the beginning of the 1990s. In the application transversal vibrations of a suspended pre-stressed concrete footbridge spanning 252 m were shown, the natural frequency of which was 0.15 Hz. Pimer M and Urus hadze (Eurodyn 2005) mill press Rotterdam (2005) suggested, the relationship between damper mass and modal mass of structures

$$\mu = \frac{m_d}{m_{\text{str}}} \tag{2.270}$$

ranging from 0 to 1 for logarithmic decrement ϑ of 01004 where m_d is the mass of the ball and m_{str} is the mass of the structure. Initial amplitudes are 100, 200 and 300 mm.

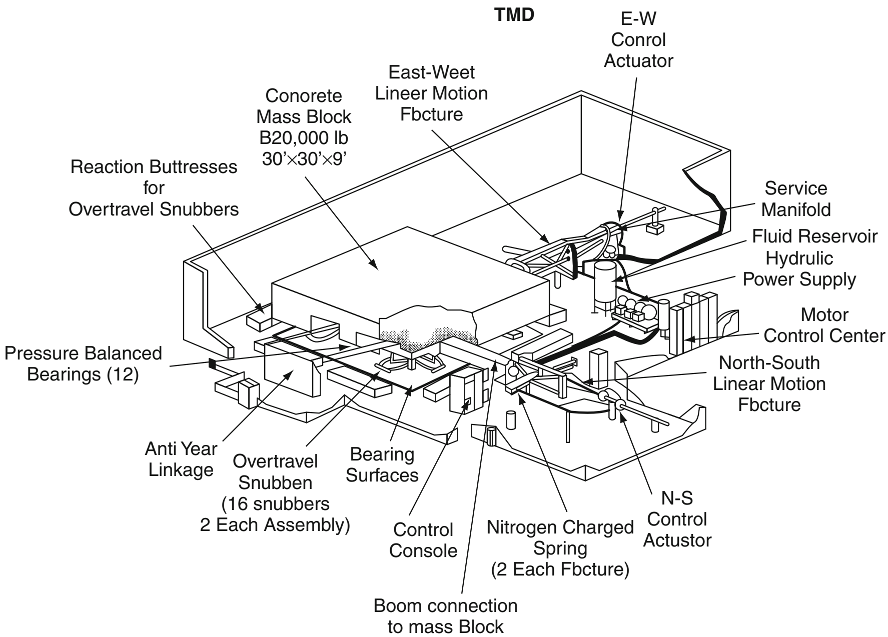


Fig. 2.7 TMD in Citicorp Center (Petersen, 1980, 1981)

The natural frequency of the rolling of the sphere is

$$f_{ds} = \frac{1}{2\pi}\sqrt{\frac{g}{(R-r)(1+(I_{\mathrm{sph}}/m_{\mathrm{sph}}r^2))}} \tag{2.271}$$

where $I_{\mathrm{sph}} =$ moment of inertia of the rolling sphere and $f_{ds} =$ natural frequencies of the rolling sphere.

The horizontal (H) and tangential forces (T) have been computed by Petersen et al as

$$H = -m_{\mathrm{sph}}g\sin\varphi\cos\varphi + \frac{1}{1+\dfrac{I_{\mathrm{sph}}}{m_{\mathrm{sph}}r^2}} \tag{2.272}$$

$$T = mg\sin\varphi - \frac{mg\sin\varphi}{m+\dfrac{I_{\mathrm{sph}}}{mr^2}} \tag{2.273}$$

Table 2.25 Tuned mass dampers in John Hancock Tower, Boston, and Citicorp Center, New York

		John Hancock Boston, MA	Citicorp Center New York, NY
Typical floor size	(ft)	343 × 105	160 × 160
Floor area	(sq ft)	36,015	25,600
Building height	(ft)	800	920
Building model weight	(tons)	47,000	20,000
Building period 1st mode	(s)	7.00	6.25
Design wind storm	(years)	100	30
Mass block weight	(tons)	2×300	373
Mass block size	(ft)	18×18×3	30×30×8
Mass block material	(type)	lead/steel	concrete
TMD/AMD stroke	(ft)	±6.75*	±4.50*
Max spring force	(kips)	135	170
Max actuator force	(kips)	50	50
Max hydraulic supply	(gms)	145	190
Max operating pressure	(psi)	900	900
Operating trigger – acceleration	(g)	0.002	0.003
Max power	(HP)	120	160
Equivalent damping	(%)	4.0%	4.0%

* Including overtravel.

Note: Collected data and then tabulated. Data provided by Boston and New York Borough councils. They have been checked from literature.

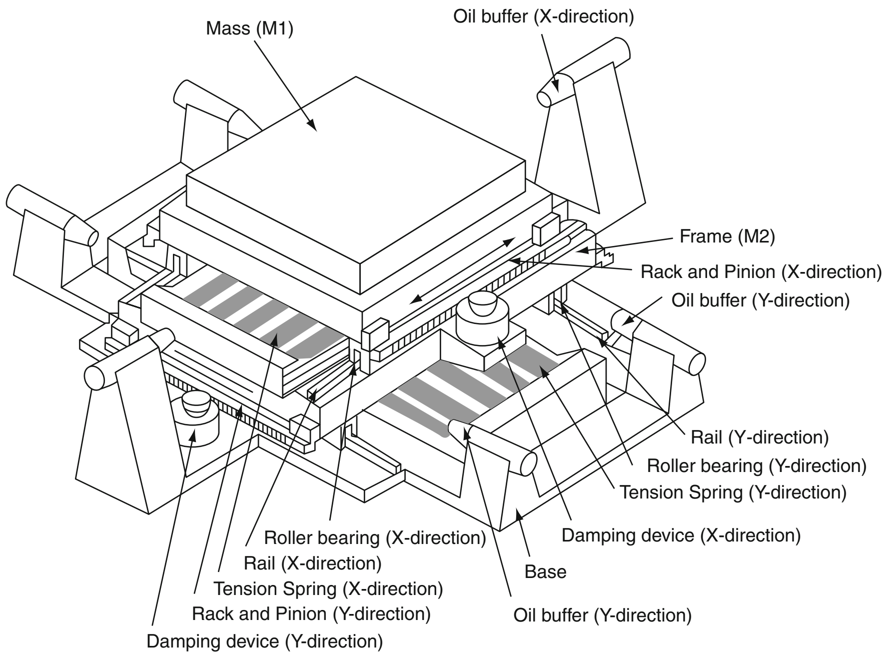


Fig. 2.8 TMD in Chiba Port Tower (Ohtake et al. 1992)

c. *Tuned mass damper*

Petersen [1995] provides a method design for tuned mass dampers using equation (2.279)

Calculation of damper mass:

$$m_d = m_h \cdot \mu \tag{2.274}$$

Calculation of optimum turning K_{opt} and of optimum damping ratio ξ_{opt}:

$$K_{\text{opt}} = 1/1 + \mu \tag{2.275}$$

$$\xi_{\text{opt}} = \sqrt{3} \cdot \mu/8 \cdot (1 + \mu)^3 \tag{2.276}$$

Calculation of the optimum damping frequency, f_d:

$$f_d = K_{\text{opt}} \cdot f_h \tag{2.277}$$

Calculation of the spring constant of the damping element, k_d:

$$k_d = (2 \cdot \pi \cdot f_d)^2 \cdot m_d \tag{2.278}$$

Calculation of the damping coefficient of the damping element, ${}^{\mathrm{d}}\mathrm{d}_{\mathrm{c}}$:

$${}^{\mathrm{d}}\mathrm{d}_{\mathrm{c}} = 2 \cdot m_d \cdot (2 \cdot \pi \cdot f_d) \cdot \xi_{\text{opt}} \tag{2.279}$$

d. *Viscous dampers*

Viscous elastic dampers or dry friction dampers use the action of solids to dissipate the oscillatory energy of a structure. It is also possible to use a fluid for obtaining the same goal.

The immediate device is the one derived from the "dashpot". In such a device, the dissipation is obtained by the conversion of the mechanical energy into heat with the help of a piston that deforms and displaces a very viscous substance such as silicon. Another family of dampers is based on the flow of a fluid in a closed container. The piston not only deforms the viscous substance but also forces the passage of the fluid through calibrated orifices. As in the preceding case, the dissipation of the energy results in development of the heat.

The main difference between these two techniques is the following. In the "dashpot" damper, the dissipative force is a function of the viscosity of the fluid, while in the other one that force is principally due to the volumic mass of the fluid. This means that the dampers with orifices are more stable against temperature variations in comparison with the "dashpot" ones.

Viscous dampers must be placed between two points of the structure with differential displacement between them. They can be either on an element linking a pier and the deck or on the horizontal wind bracing of the deck.

Several recent studies have shown that supplement fluid viscous damping effectively reduces the seismic responses of asymmetric plan systems. However, these investigations examined the behaviour of asymmetric plan systems with linear fluid viscous dampers. Non-linear fluid viscous dampers (velocity exponent less than one) have the apparent advantage of limiting the peak damper force at large velocities while still providing sufficient supplemental damping for linear dampers, the damper force increasing linearly with damper velocity.

A recent investigation examined the seismic response of asymmetric systems with non-linear viscous and viscoelastic dampers. It was found that structural response is weakly affected by damper non-linearity, and non-linear dampers achieve essentially the same reduction in response but with much smaller damper force compared to linear dampers; reductions up to 20% were observed for edge deformations and plan rotations of short-period systems. Furthermore, it was shown that the earthquake response of the asymmetric systems with non-linear dampers can be estimated with sufficient degree of accuracy by analysing the same asymmetric systems with equivalent linear dampers. A simplified analysis procedure for asymmetric plan systems with non-linear dampers has also been developed.

The investigation by Lin and Chopra examined the effects of damper non-linearity on edge deformations and damper forces. For asymmetric plan systems, however, other important response quantities of interest for design purposes include base shear, base torque and base torque generated by asymmetric distribution of dampers. Therefore, it is useful to investigate the effects of damper non-linearity on these responses.

e. *Viscous elastic dampers*

The use of viscous elastic materials for the control of vibrations goes back to the 1950s. Their application in structural engineering dates back to the 1960s.

The viscous elastic materials are principally polymers dissipating the energy by shear. The characteristics of viscous elastic dampers depend not only on frequency but also on temperature. The damping coefficient is expressed by

$$C = \frac{W_d}{\pi \cdot \Omega \cdot x^2} \tag{2.280}$$

where W_{d} = energy dissipation in the structure per cycle
Ω = circular frequency of excitation
x = (piston) displacement

To calculate the effect of dampers in the structure a time history dynamic analysis could be carried out. Viscoelastic damper's behaviour could be represented with the following simplified model:

$$F = F_0 + Kx + Cv^{\alpha} \tag{2.281}$$

where F = force transmitted by damper
F_0 = preloading force
x = (piston) displacement
v = velocity

f. *Tuned liquid dampers*

Tuned liquid dampers are fluid-filled containers and provide an interesting possibility for footbridge damping systems. Accelerations of the container cause inertial and damping forces that can be used as system damping. The damping forces are dependent on the viscosity of the fluid and the texture of the container walls. Figure 2.9 a – d is referred to as tuned liquid column dampers while Fig. 2.9e and f is referred to as tuned sloshing dampers.

The natural frequency of the liquid dampers as in Fig. 2.9a and b can be expressed as

$$f = \frac{1}{2\pi}\sqrt{\frac{2g}{L_f}} \tag{2.282}$$

where L_f = total length of fluid column and g = acceleration due to gravity.

The natural frequency of the liquid damper as in Fig. 2.9e and f can be expressed as

$$f = \frac{1}{2\pi}\sqrt{\alpha \cdot \frac{g}{L} \cdot \tanh\left(\alpha \cdot \frac{H}{L}\right)} \tag{2.283}$$

where $\alpha = \pi/2$ for a rectangular container and 1.84 for a cylindrical container.

H and L can be determined from Fig. 2.9. The radius of the cylinder can be replaced by L in (2.288) for cylindrical containers.

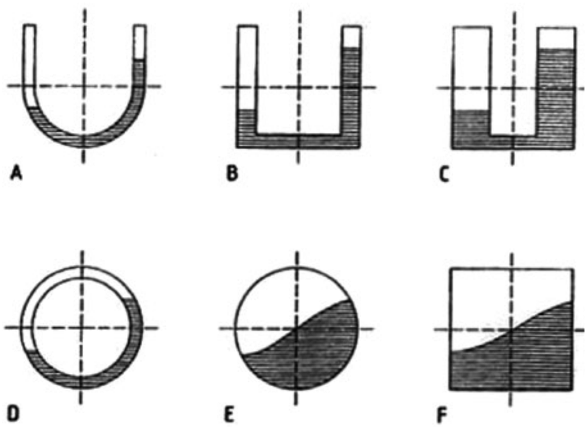


Fig. 2.9 Various types of tuned liquid dampers

The effectiveness of a fluid damper depends on the ratio between modal mass of the damper and the structure and the detuning. The dimensions of the container, height of fluid and the viscosity of the fluid also play an important role. For larger accelerations, the behaviour of the fluid damper is non-linear. Petersen recommends an experimental tuning of the fluid damper. Contrary to a number of high-rise buildings, it seems that no footbridge has been equipped with this type of damper yet.

Frictional damping systems
Frictional damping systems use friction between surfaces to achieve a damping effect. Frictional damping systems have been used in the footbridges in Germany. A total of eight dampers are generally installed near the bearings, providing frictional damping in the vertical and longitudinal directions.

Isolators

(a) *Natural rubber bearings*
Figure 2.10 shows the devices known as bearings made in natural rubber. They are either round or square in shape. The main construction is that such a bearing is composed of laminated rubber bearings with inner steel plates and flange plate. Sometimes it is encased by a layer of surface rubber.

Dynamic Characteristics

The vertical stiffness Kv of the natural rubber is given by

$$Kv = \alpha \cdot \frac{A_r}{t} \cdot \frac{E_r(1 + 2kS_1^2)G_b}{E_r(1 + 2kS_1^2) + G_b} \tag{2.284}$$

where A_r : cross-sectional area of laminated rubber
t : total rubber thickness
S_1 : primary shape factor
α_e: correction modulus of longitudinal elasticity
E_r : longitudinal elastic modulus of rubber

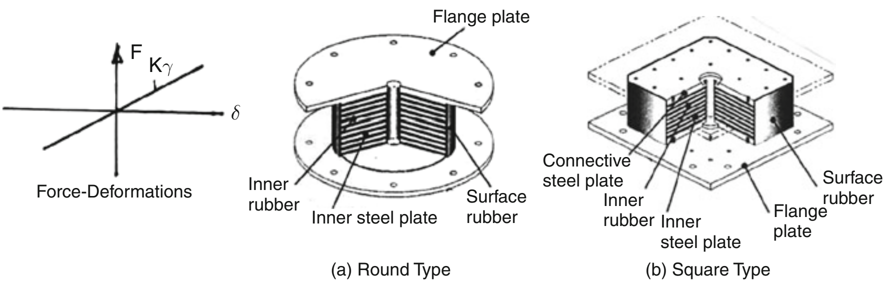
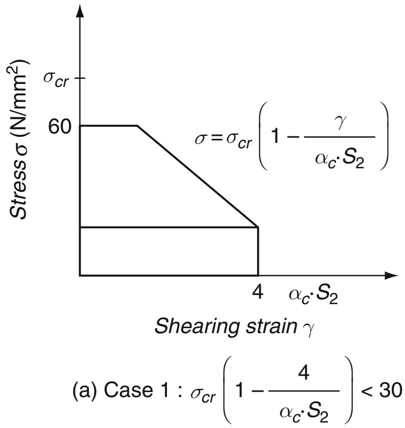


Fig. 2.10 Natural rubber bearings – isolators

Fig. 2.11 Stress versus shearing strain for case 1



γ : Shear strain $= \epsilon_s$
G_b: bulk modulus of rubber
K : correction modulus of rubber hardness

The maximum compressive strength at critical level is 60 N/mm^2. Maximum shearing strain $T_{r(\max)}$ is 400%.

σ_{cr} : compressive critical strength for shearing strain $= 0$

$$\sigma_{cr} = \xi \cdot G_r \cdot S_1 \cdot S_2 \tag{2.285}$$

$$\text{where } \xi = \begin{cases} 0.85 & (S_1 \geq 30) \quad \text{damping} \\ 0.90 & (S_1 < 30) \quad \text{factor} \end{cases} \tag{2.286}$$

G_r: shear modulus of rubber

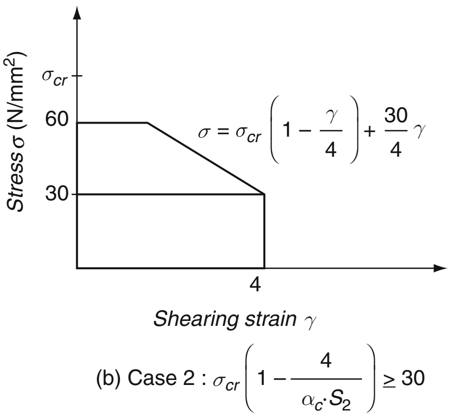


Fig. 2.12 Stress versus shearing strain – for case 2

$$\alpha_c = \begin{cases} 1 & (S_2 < 4) \\ 0.1(S_2 - 3) + 1 & (S_2 \geq 4) \end{cases} \tag{2.287}$$

S_2 : secondary shape factor

$$S_2 = \begin{cases} S_2 & (S_2 \leq 6) \\ 6 & (S_2 > 6) \end{cases} \tag{2.288}$$

When $\gamma \neq 0$, then the maximum stresses are determined.

$\sigma_{cr}\left(1 - \frac{4}{\alpha_c \cdot S_2}\right) < 30$ (Case 1):

$\sigma = \sigma_{cr}\left(1 - \frac{\gamma}{\alpha_c \cdot S_2}\right)$ (The maximum value of σ is 60 N/mm^2):

$\sigma_{cr}\left(1 - \frac{4}{\alpha_c \cdot S_2}\right) \geq 30$ (Case 2):

$\sigma = \sigma_{cr}\left(1 - \frac{\gamma}{4}\right) + \frac{30}{4}\gamma$ (The maximum value of σ is 60 N/mm^2):

The lateral stiffness K_r at 15° is given for the hysteresis loop model as

$$K_r = G_r \cdot \frac{A_r}{t} \tag{2.289}$$

where G_r = shear modulus of the rubber.

Selecting dimension, the F-8 relation can be achieved using the analysis and a typical experiment.

(b) *High damping rubber bearing*
A procedure similar to (a) stated for natural rubber bearing on page 105 shall be adopted and the hysteresis shear stress versus shear strain can be drawn.

(c) *Lead-rubber bearing*
Fundamental dynamic characteristics

$$K_d = C_{Kd}\left(K_r + K_p\right) \quad (\text{at } 15°) \tag{2.290}$$

where K_r : lateral stiffness

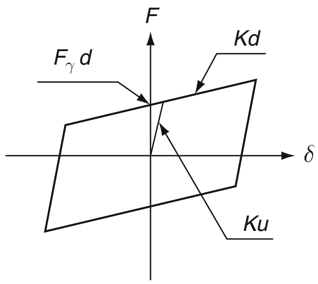


Fig. 2.13 Hysteresis loop model of lead-rubber bearing

$$K_{\mathrm{r}} = G_{\mathrm{r}} \cdot \frac{A_{\mathrm{r}}}{t} \tag{2.291}$$

where A_{r} : cross-section of laminated rubber
K_{p} : additional stiffness by lead plug

$$K_{\mathrm{p}} = \alpha \cdot \frac{Ap}{H} \tag{2.292}$$

F_{yd} = yield force
K_{u} = initial stiffness
K_{d} = secondary stiffness
$C_{k\mathrm{d}}$ = modification modulus k_{d}

where α: shear modulus of lead
A_{p}: cross-section area of plug
C_{Kd} : modification modulus on K_{d} by strain dependency

$$C_{\mathrm{Kd}} = \begin{cases} 0.779\gamma \ [\gamma < 0.25] & (2.293) \\ \gamma \ [0.25 \leq \gamma < 1.0] & (2.294) \\ \gamma \ [1.0 \leq \gamma < 2.5] & (2.295) \end{cases}$$

The yield force F_{yd} is determined as

$$F_{\mathrm{yd}} = C_{\mathrm{yd}} \sigma_{\mathrm{pb}} A_{\mathrm{p}} \text{ at } 15^{\circ} \tag{2.296}$$

2.1.14.3 Seismic Isolation Codes and Techniques

General Introduction

After the 1994 Northridge earthquake in the USA, the 1995 Hyogo-Ken Nanbu earthquake in Japan and the 1999 Chi-Chi earthquake in Taiwan, the number of seismically isolated buildings has increased rapidly. Over the same period, building codes have been revised and updated to include requirements for design of seismically isolated buildings. In the USA, seismic isolation provisions have been included in building codes since first appearing in the 1991 Uniform Building Code. The current US provisions are contained in the International Building Code, IBC 2003, which makes reference to the requirements of ASCE 7-02. In Japan, the most recent building code provisions took effect in 2000 and in China and Taiwan in 2002.

In this section, a test study on a seismically isolated building is presented in order to understand and illustrate the difference in the isolation provisions of the building codes of Japan, China, the USA, Italy and Taiwan. The concept of the design spectrum in each code is summarized first. To consider the seismic region coefficients, the target construction sites are assumed to be in Tokyo, Beijing, Los Angeles, Potenza and Taipei, respectively. A fixed soil profile is assumed in all cases where the average shear wave velocity within the top 30 m is about 209 m/s.

If the control device needs external energy to modify the vibration properties of a structure, a closed-loop control system can be used, which does not affect many properties of external vibrations. In the case of an open loop control system external vibrations are sensed as soon as they reach the foundation and before they are incident on the building. The control is exercised in such a way that the building does not vibrate in resonance with the severe changes in seismic motion. The usefulness of the open-loop system depends on the proper functioning of the brain unit which recognizes the information from various sensors and generates signals to counter the earthquake.

Various devices are being considered for the control mechanism. The main mechanisms being discussed are (a) variable stiffness mechanism where stiffness of the structure is varied according to the seismic motion striking the foundation of the building so that the building does not attain resonance condition and (b) a mechanism requiring external energy or a mechanism with additional control power which actively and effectively absorbs the energy incident on the building according to the response and which can restore the deformation caused in the building due to the action of the seismic force.

Design Spectrum

In general, seismic load is expressed by a 5% damped design spectrum as follows:

$$S(T) = IS_{\mathrm{a}}(T) \tag{2.297}$$

where

I: occupancy importance factor, which is taken as 1.0 in this study
T: fundamental period of the structure
$S_{\mathrm{a}}(T)$: the design spectrum on site.

The design spectrum generally consists of two parts, namely, a uniform acceleration portion in the short-period range and a uniform velocity portion in the longer period range.

In the Chinese code, spectrum in the constant velocity portion is additionally increased to ensure the safety of structures having long natural periods, such as high-rise buildings or seismically isolated buildings.

A two-stage design philosophy is introduced in the Japanese, Chinese and Italian codes. The two stages are usually defined as damage limitation (level 1) and life safety (level 2). In this chapter, response analyses in the life safety stage will be discussed. In addition, an extremely large earthquake with 2% probability of exceedance in 50 years is defined to check the maximum design displacement of the isolation system in the US and Italian codes.

China Codified Method

Plate 2.1 gives an overall specification of the design spectrum. The response spectrum for isolated buildings requires design spectrum.

Time history analysis is suggested to calculate the response.

- The first branch for periods less than 0.1 s.
- The constant design spectrum branch, with amplitude listed in Plate 2.1 between 0.1 s and the characteristic period of ground motion T_g.
- The third branch, which decreases over the period range T_g to five times T_g.
- The fourth decreasing branch for periods greater than T_g and defined up to 6 s.

Plate 2.1

Country	Design spectrum	Spectral acceleration versus period

China

There are four site classes which are classified by characteristic period Tg shown in Table

$$\alpha(g) = \begin{cases} \left(0.45 + \frac{\eta_2 - 0.45}{0.1} T\right)\alpha_{\max} & T \le 0.1 \\ \eta_2 \alpha_{\max} & 0.1 < T \le T_g \\ \left(\frac{T_g}{T}\right)^{\gamma} \eta_2 \alpha_{\max} & T_g < T \le 5T_g \\ \left[\eta_2 0.2^{\gamma} - \eta_1 \left(T - 5T_g\right)\right]\alpha_{\max} & 5T_g < T \le 6.0 \end{cases} \quad (2.298)$$

where

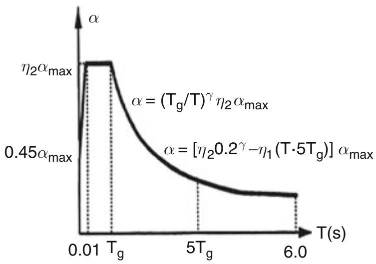
$\alpha_{\max}$: zone factor

η_1, γ : shape coefficients;

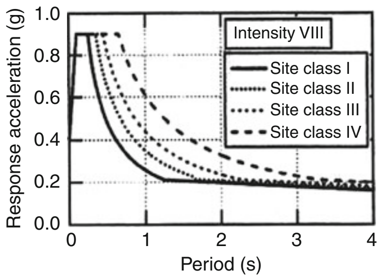
η_2 : response reduction factor defined in Equation

Return period Level 1 50 years
Level 2 1600-2500 years

Storey drift, 1/550



Design response spectrum (China)



Site spectra for the four kind site classes (China)

Zone factor $\alpha_{\max}$ *(g)* based on Seismie Intensity (China)

	Intensity			
Level	VI	VII	VIII	IX
Level 1	0.04	0.08 (0.12)	0.16 (0.24)	0.32
Level 2		0.50 (0.72)	0.90 (1.20)	1.40

( ): regions where the amplitude of design basic acceleration is 0.15 g or 0.30 g.

Characteristic period T_g related to site class (China)

	Site			
Zone	I	II	III	IV
1	0.25	0.35	0.45	0.65
2	0.30	0.40	0.55	0.75
3	0.35	0.45	0.65	0.90

T_g: characteristic period related to the site soil profile: ζ: effective damping.

$$\gamma = 0.9 + \frac{0.05 - \zeta}{0.5 + 5\zeta}$$

$$\eta_1 = 0.02 + (0.05 - \zeta)/8, \eta_1 \ge 0$$

$$\eta_2 = 1 + \frac{0.05 - \zeta}{0.06 + 1.7\zeta}, \eta_2 \ge 0.55 \quad (2.299)$$

Fortification intensity		6	7	8	9
α_{max}	Frequently occurred earthquake	0.04	0.08(0.12)	0.16(0.24)	0.32
	Seldom occurred earthquake	0.28	0.50(0.72)	0.90(1.20)	1.40
	Design basis earthquake	0.05	0.10(0.15)	0.20(0.30)	0.40

Note: In the items of α_{max} the values in brackets are used for the regions which Design Basis Earthquake acceleration values are 0.15 g or 0.30 g.

Table 2.26 The peak value of acceleration based on time history analysis

Seismic intensity/peak acc. (gal)	6	7	8	9
Frequently occurred earthquake	18	35 (55)	70 (110)	140
Seldom occurred earthquake		220 (310)	400 (510)	620

Japan

Introduction

Japan is situated at the complex intersection of the Eurasian, North American, Pacific and Philippine tectonic plate boundaries, a region that is considered as having one of the highest risks of severe seismic activity of any area in the world. Nearly 60% of Japan's population is concentrated in the three largest cities of the Kanto, Chubu and Kansai regions. The Kanto region includes Japan's two largest cities, Tokyo and Yokohama, the Chubu region includes Nagoya and the Kansai area includes Kyoto, Osaka and Kobe. In an east – west area these three regions are situated along the subduction zone of the Philippine and Pacific plates and have experienced many large earthquakes such as the 1854 Ansei-Tokai earthquake (M8.4), the 1923 Kanto earthquake (M7.9). All of these cities have suffered destructive damage in the past earthquakes. The northwestern coast of Japan lies on the boundaries of the Eurasian and North American plates. The 1964 Niigata Prefecture earthquake (M6.8) had occurred almost all over Japan.

The severe seismic threat faced by the entire country has led to the extensive development of the field of earthquake engineering and resulted in widespread innovation and application of innovative seismic structural technologies in Japan.

Recent applications of seismic isolation have extended beyond implementing the plane of isolation at the base of a building to mid-story isolation and also to applying isolation to high-rise buildings with heights greater than 60 m. Moreover, seismic isolation has been utilized as a means to realize architectural design aesthetics, a realm that hitherto was much restricted in traditional Japanese earthquake-resistant design.

The Japan Society of Seismic Isolation (JSSI) published "the Guideline for Design of Seismically Isolated Buildings" in 2005 summarizing the basic concepts and approach for performing time history analysis of seismically isolated buildings.

In the 1995 Hyogo-Ken Nanbu earthquake, a large number of condominium buildings suffered severe damage, but mostly they did not collapse. Subsequently, many complex issues arose between the engineer and residence owners in deciding whether or not to demolish or to repair the damaged buildings. These difficulties called developer's attention to the importance of maintaining a building's function or limiting damage to a low and repairable level, even after a severe earthquake.

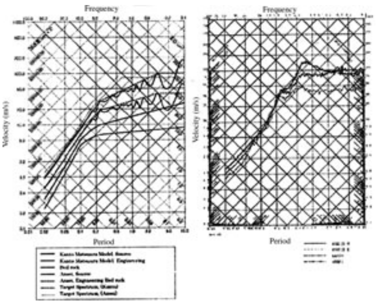
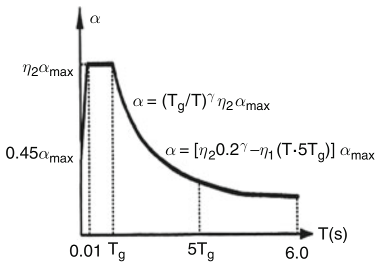
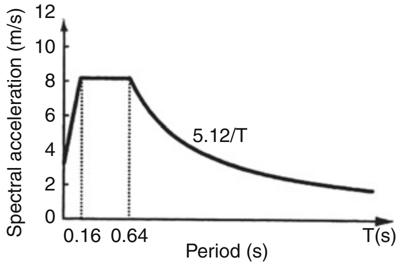
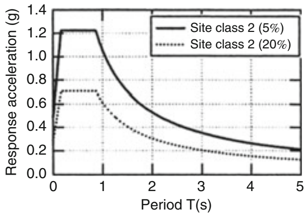
Japanese Codified Method

The basic parameters are indicated briefly in Plate 2.2 and give the response factor at ground face for three features of devices. Plate 2.2 gives also the relatives for ground characteristics together with the isolation devices.

The verification response values. The response acceleration, ${}_{\mathrm{e}}S_{\mathrm{a}}$, is determined as the value of the vertical axis at the corresponding natural period calculated as shown. The response displacement ${}_{\mathrm{e}}\delta$ at gravity centre is determined as follows:

$$ {}_{\mathrm{e}}\delta = {}_{\mathrm{e}}Q/K_{\mathrm{eq}} = \frac{M_{\mathrm{e}}S_{\mathrm{a}}}{K_{\mathrm{eq}}} \tag{2.300} $$

Plate 2.2 Design spectrum adopted in Japan

<table>
<tr><th>Country</th><th>Design spectrum</th><th>Spectral acceleration versus period</th></tr>
<tr><td>Japan</td><td>In general, the five percent-damped spectral acceleration, $s_a(T)$ is:
$S_a(T) = ZG_1(T)S_0(T)$
where Z: the seismic hazard zone factor.
$Gst(T)$: a soil amplification factor dependent on the soil profile.
$S_0(T)$: the design spectral acceleration at engineering bedrock defined in Equation (3.3) which is shown in Figure for Level 2
$S_a(m/s^2) = \begin{cases} 3.2 + 30T & T \leq 0.16 \\ 8.0 & 0.16 < T \leq 0.64 \\ 5.12/T & 0.64 < T \end{cases}$
Storey drift, 1/200
Return year period 50 – 500

</td><td>
Design spectral acceleration at the engineering bedrock (V_S>400m/s) (Japan)
</td></tr>
</table>

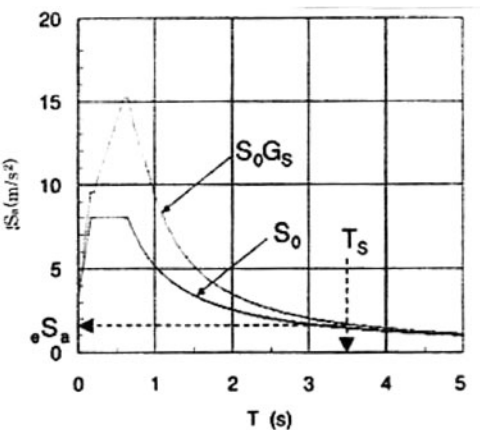


Fig. 2.14 Response spectrum at ground surface

Considering the layout of isolation devices, which cause eccentricities between the gravity centre and stiffness centre, the overall response displacement of the isolation interface ${}_{\mathrm{e}}\delta_{\mathrm{r}}$ is obtained as follows:

$${}_{\mathrm{e}}\delta = 1.1\,{}_{\mathrm{e}}\delta_{\mathrm{r}} < (\delta_{\mathrm{s}})$$

$${}_{\mathrm{e}}\delta_{\mathrm{r}} = \alpha_{\mathrm{e}}\delta$$

Table 2.27 Features of devices

	NRB700	LRB800	LRB850
Diameter (mm)	700	800	850
Inner diameter (mm)	15	160	170
Rubber sheet (mm) *Layer	4.7×30	5.1×33	5.25×32
Area (cm^2)	3,847	4,825	5,448
Steel plate (mm)	3.1×29	4.4×32	4.4×31
Height of rubber (mm)	141	168	168
1 st shape factor	36.4	39.2	40.5
2 nd shape factor	5	4.8	5.1
Diameter of lead core (mm)	–	160	170
Diameter of flange (mm)	1,000	1,150	1,200
Flange thickness (mm)	28 – 22	32 – 24	32 – 24
Height (mm)	286.9	373.1	368.4
Weight (kN)	6.4	11.5	12.7
Total number	2	4	6

Table 2.28 Eccentricities of isolation interface

	Gravity		Rigidity		Eccentricity		Eccentricity	
Story	g_x(m)	g_y(m)	I_x(m)	I_y(m)	e_x(m)	e_y(m)	R_{ex}	R_{ey}
$\gamma = 1.0$	1148.3	708.2	1121.4	707.4	26.8	0.8	0.001	0.029
$\gamma = 1.5$	1148.3	708.2	1121.3	689.6	27.0	18.6	0.020	0.029

Table 2.29 Relation for ground characteristics

	Formulae	Minimum values
$T \leq 0.8T_2$	$G_s = G_{s2} \frac{T}{0.8T_2}$	1.2
$0.8T_2 < T \leq 0.8T_1$	$G_s = \frac{G_{s1} - G_{s2}}{0.8(T_1 - T_2)} T + G_{s2} - 0.8 \frac{G_{s1} - G_{s2}}{0.8(T_1 - T_2)} T_2$	1.2
$0.8T_1 < T \leq 1.2T_1$	$G_s = G_{s1}$	1.2
$1.2T_1 < T$	$G_s = \frac{G_{s1} - 1}{\frac{1}{1.2T_1} - 0.1} \cdot \frac{1}{T} + G_{s1} - \frac{G_{s1} - 1}{\frac{1}{1.2T_1} - 0.1} \cdot \frac{1}{1.2T_1}$	1.0

Note: Data provided by the Japanese manufacturers for devices.

Table 2.30 Dimensions of dampers

		Steel bar damper	Lead damper
Rod	Rod diameter (mm)	$\phi 90$	$\phi 180$
	Number of rods	4	1
	Loop diameter	$\phi 760$	–
	Material (Standard No.)	SCM415 (JIS G 4105)	Lead (JIS H 2105)
	Number of dampers	16	6

Table 2.31 Characteristics of isolation devices

		Rubber bearings		Steel rod damper	Lead damper
	Item	ϕ 800	ϕ 800A		
Horizontal stiffness (kN/m)	Initial stiffness K_1	1,060	860	7,110	12,000
	Secondary stiffness K_2	–	–	0	0
	Yield load (kN)	–	–	290	90
	Yield displacement (m)	–	–	0.0408	0.0075

where α is the safety factor for temperature-dependent stiffness changes and property dispersions in manufacturing of devices and superstructures, which must be smaller than their strength and allowable stress, respectively.

The United States of America

Introduction

This section presents an overview of seismic isolation and passive energy dissipation technologies in the USA. A historical survey of seismic isolation and energy dissipation applications is presented with descriptions of selected notable projects. The types of devices that are most commonly used in the USA are described along with a brief overview of research on the technologies and the evolution of code regulations governing their use. The section concludes with comments on the future direction of the technologies.

Overview of Seismic Isolation Applications in the USA

Construction of the first seismically isolated building in the USA was completed in 1985 and by mid-2005 there were approximately 80 seismically isolated buildings in the USA. Some of the most significant early projects are discussed below, along with examples of several more recent projects.

Buildings

The first building in the USA to be seismically isolated, the Foothill Communities Law and Justice Center in Rancho, California, was completed in 1985, a four-storey plus basement building. The realization of the project was the culmination of the efforts of numerous parties. The use of high-damping rubber bearings was the first application in the world of this type of isolation system.

The US Court of Appeals building, in San Francisco, is another example of a large historic building retrofit of numerous other monumental building structures, including City Hall in Oakland and State Capitols in South Carolina and Utah.

Seismic isolation has been used throughout the US buildings up and down the country; many reports exist on the testing facilities of passive energy dissipation systems. However, code provisions for seismic isolation and passive energy systems are briefly dealt with below.

Current Status and Future Development

Seismic isolation. Given the 20-year application history of seismic isolation in the USA, the approximately 80 projects completed is a modest total. While many notable projects, particularly the retrofit of a number of landmark historic buildings, have been undertaken, fewer projects of this type are expected in the future. Seismic isolation has not moved into the mainstream as a widely accepted and used seismic-resistant technology.

Somewhat unfairly, seismic isolation has suffered under the conventional wisdom that it is an expensive technology. Many of the most prominent early isolation projects were large and costly retrofits of historic buildings, projects that would have been expensive regardless of whether or not isolation was used. Nonetheless, the general belief has evolved that seismic isolation is expensive and that it is not economically feasible to consider for typical buildings.

Codified Method

According to IBC 2003, the general design response spectrum curve with various equations are indicated in Plate 2.3.

Apart from these parameters, the values of the site coefficient F_a as a function of the site class and mapped spectra response acceleration at short period (S) would be needed and they are given below:

Plate 2.3 Design spectrum and Data (U.S.A)

Country	Design spectrum	Spectral acceleration versus period
USA	According to the IBC 2003, the general design response spectrum curve is as shown in Figure and is defined by Equation	

$$S_a = \begin{cases} 0.6\frac{S_{DS}}{T_0} + 0.4S_{DS} & T \le T_0 \\ S_{DS} & T_0 \le T \le T_S \\ \frac{S_{D1}}{T} & T_S < T \end{cases}$$

where:
S_{DS}, S_{D1}: the design spectral response acceleration at short periods and one second period, respectively, as determined by Equation (3.8).

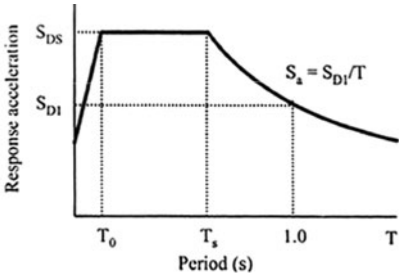
$$T_0 = 0.2S_{D1}/S_{DS};\ T_S = S_{D1}/S_{DS}$$

$$S_{DS} = \tfrac{2}{3}S_{MS} = \tfrac{2}{3}F_a S_S$$

$$S_{D1} = \tfrac{2}{3}S_{M1} = \tfrac{2}{3}F_v S_1$$

$$S_S \sim 1.55\,g;\ S_1 = 0.623\,g$$

Where: F_a, F_v: site coefficients defined in Tables and, respectively. S_s, S_1: the mapped spectral accelerations for short periods and one second period.

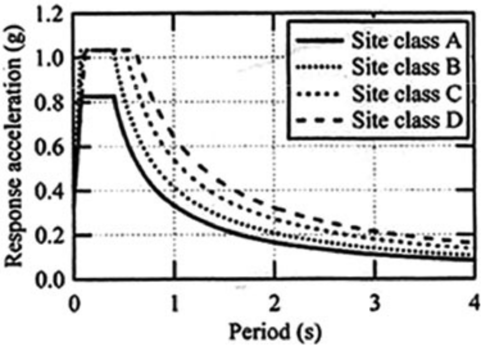


Design response spectrum, IBC 2003 (USA)

Table Values of the site coefficient F_a as a functions of the class and mapped spectral response acceleration at short period (S_s)[a]

Mapped spectral accelerations at short periods					
Site Class	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \le 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

a Use straight line interpolation for intermediate values of mapped spectral acceleration at short period
b. Site specific geotechnical investigation and dynamic site response analyses shall be performed.



Site spectra at the four kinds of site classes (USA)

Table Values of site coefficient Fa as a function of site class and mapped spectral response acceleration at short period (S_t)[a]

Mapped spectral accelerations at one second period					
Site Class	$S_t \le 0.1$	$S_t = 0.2$	$S_t = 0.3$	$S_t = 0.4$	$S_t \le 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.74	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

Values of site coefficient F_a as a function of site class and mapped spectral response are given in tables provided by the code.

Based on ISO, the following information is required if one goes on using the code:

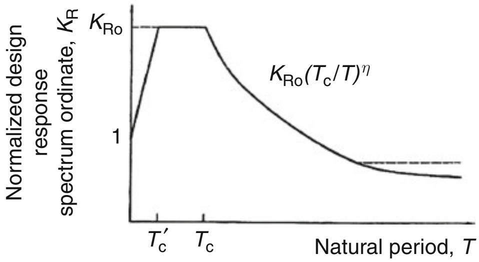
Acceleration at short period (Ss)[a]
Mapped spectral accelerations at short periods

Site Class
A reference is made to the code

ISO 3010: 2001E. Plate 2.4 indicate the relationships for normalised response spectra.

Sheet No. S 2.4 to 2.5.3 give brief information on this subject based on the ISO requirements. These sheets are available.

Plate 2.4 Design spectrum based on ISO

Country	Design spectrum	Spectral acceleration versus period
ISO 3010: 2001E	 Normalized design response spectrum	T_c, T'_c

Eurocode-8

Introduction

The seismic protection of conventional structures is based on the favourable changes of their dynamic characteristics, induced by yielding and damage occurring in structural and non-structural elements under intense seismic action. Such changes can be essentially described as an increase of flexibility and of damping. Due to the usual spectral characteristics of earthquakes and/or to the energy dissipation occurring in the structure, these changes give rise to a considerable reduction in the structural mass accelerations and, then, of the inertia forces. This makes it possible for a ductile structure to survive a "destructive" earthquake without collapsing.

In the last two to three decades, new strategies have been developed which still rely upon deformation and energy dissipation capabilities. These properties, however, are concentrated in special devices, in the form of rubber or sliding bearings, of energy dissipating and/or re-centring viscous or hysteretic devices. Such devices are incorporated in the structure so as to store and dissipate most of the input energy. The inertia forces acting on the structure during a strong earthquake are considerably reduced, so that no damage to structural and non-structural elements is in principle required to further reduce them, and hence higher levels of seismic protection are obtained.

The two most frequently used "passive control" strategies for buildings are

– energy dissipation
– seismic isolation

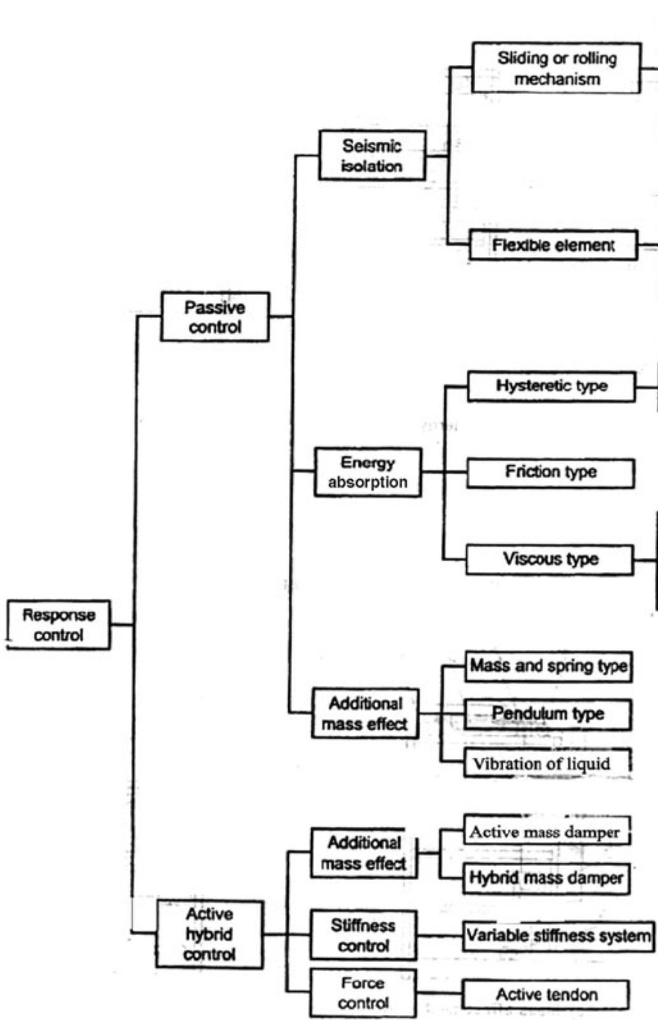
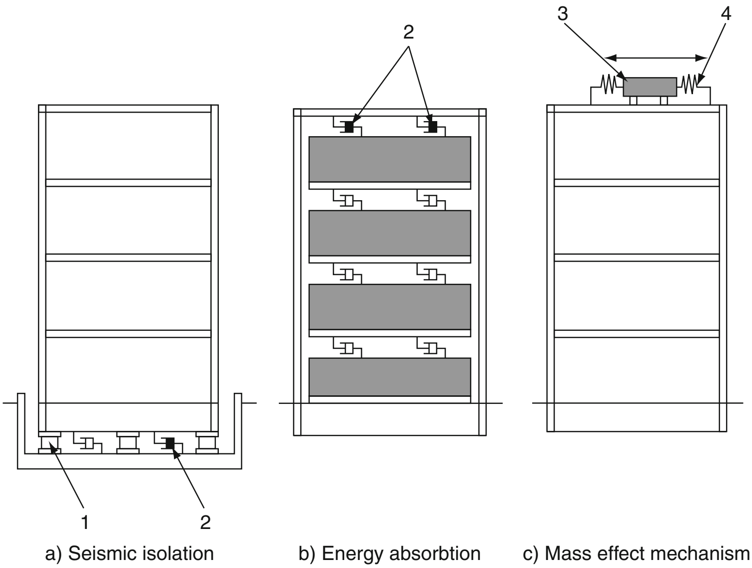


Fig. 2.15 Classification of response-control systems

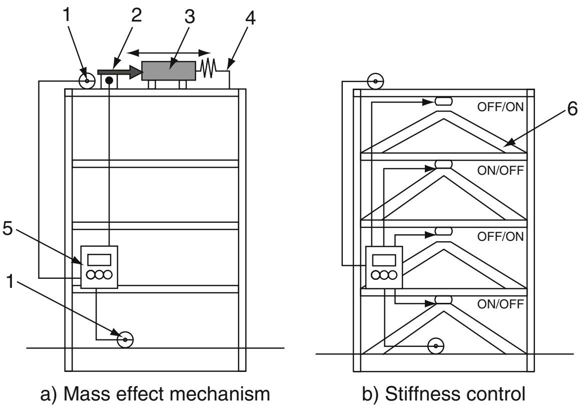
The energy dissipation strategy consists of the introduction within the structural system of elements specifically designed to dissipate energy in the dynamic deformation of the structure. These elements may take the form of dissipative steel bracings separate for the structure and working in parallel with it or they can be obtained by the use of friction devices, viscous dampers or elasto-plastic steel components.

Seismic isolation essentially uncouples the structural movement from the ground motion by introducing a strong discontinuity in the lateral stiffness distribution along the height of the structure (usually at their base in buildings). The structure is thus subdivided into two parts: the substructure, rigidly



Key
1 Isolator
2 Damper
3 Mass
4 Spring

Fig. 2.16 Example of passive control system



Key
1 Sensor
2 Actuator
3 Mass
4 Spring
5 Computer
6 Brace

Fig. 2.17 Example of active control system

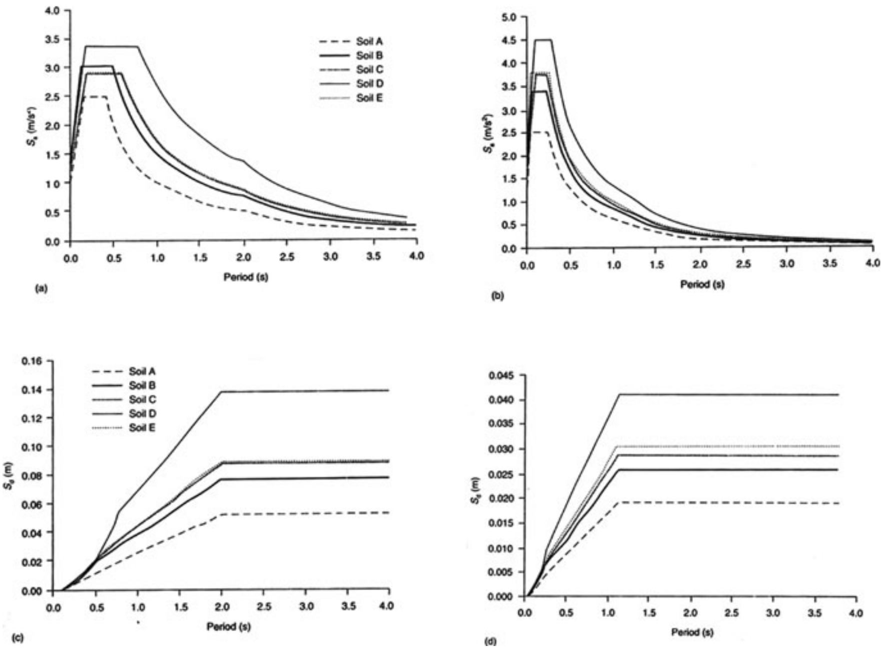


Plate 2.5 Eurocode 8 elastic response spectra for 5% damping: (a, b) pseudo-acceleration spectra; (c, d) displacement spectra; (a, c) recommended spectra of type 1; (b, d) recommended spectra of type 2.
Note: these diagrams are taken from Eurocode 8 with indebtedness and compliment

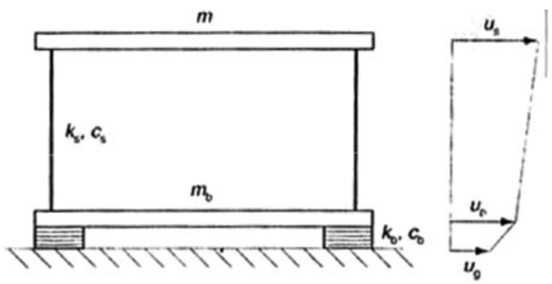
connected to the ground, and the superstructure. They are separated by the isolation interface, which includes the isolation system.

After a careful consideration of the Eurocode-8, four diagrams covering elastic response spectra, pseudo-acceleration spectra and displacement spectra are given in Plate 2.5

Classification and Characteristics of Response-Controlled Structures

In general there is a great non-uniformity in the classification of response-controlled structure. Various methodologies have been delivered. Some are listed in Plate 2.5.

Fig. 2.18 Simplified two-degrees-of-freedom model of a base-isolated structure



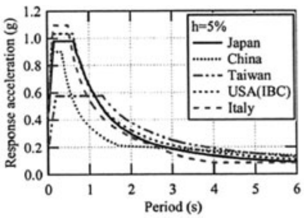
A Comparative Study of Seismic Codes

Introduction

Plate 2.6 gives a brief comparison of response acceleration versus period based on a fixed soil profile using isolation techniques adopted in Japan, China, the USA, Italy and Eurocode 8. The Far East countries data were compared with 5 and 20% damping. Where the effective damping rate is greater than 15%, the reduction factor in the Japanese is comparatively smaller. The dynamic characteristics of the soils such as the relationship between G (shear stiffness), γ (shear strain) and effective damping have been

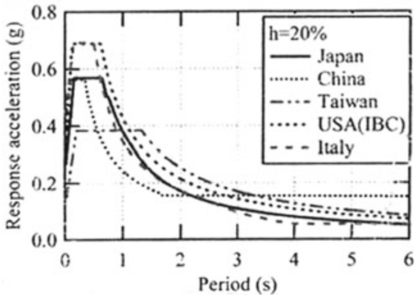
Plate 2.6 Design spectra–a comparative study

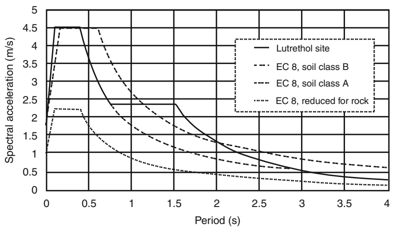
Country	Design spectrum	Spectral acceleration versus period
Comparison Japan China Taiwan The USA Italy Euro 8	(see figures below)	In order to evaluate the differences in the spectral accelerations, a comparison study is conducted. For this study, the building sites are assumed to be in Tokyo, Beijing, Los Angeles, Potenza and Taipei. A fixed soil profile is assumed, where Vs, average = 209 m/s. Typically, seismically isolated buildings should be located on relatively stiff ground, such as that defined. In the Japanese code, an iterative procedure is used to calculate the site amplification coefficient, rather than using the amplification coefficients defined in the code. The detailed procedure is shown in Section 5.3.2. The dynamic characteristics of the soils such as the relationship between shear stiffness G and shear strain and the relationship between effective damping and shear strain were obtained from the site investigation. Ground surface 5% damped acceleration response spectra given by the five different codes. In the short-period range, less than about 0.5 s, Sa, Italy, is the largest. For periods longer than about 0.6 s, Sa, USA and Sa, Japan, have approximately the same value. Beyond about 1.2 s, Sa, Taiwan, has the largest value due to the Taipei basin geology. Soong et al. ( ) used a fixed soil profile for comparison. It is seen that for structures having natural periods longer than 3 s, the spectral acceleration level is about the same for all five codes, with the exception of the Italian code, which gives slightly lower values



Five percent-damped acceleration response spectra for Tokyo, Beijing, Los Angeles, Potenza and Taipei

Five percentamped acceleration response spectra for Tokyo, Beijing. Los Angles, Potenza and Taipei





Ref. seismic conceptual Design of Buildings. Prof Hugo Backman DFA and DETEC, Switzerland BBL publications, BWG 2003.

Soil Profile used for study, where $V_{s\ average} = 209\,\text{m/s}$

Layer	Depth (m)	Vs (m/s)	γ (t/m^3)
1	0.00	120	1.85
2	2.85	120	1.50
3	5.90	120	1.80
4	8.95	310	1.90
5	14.35	220	1.85
6	18.55	380	2.00
7	23.50	320	1.75
8	28.50	400	1.95

Table 2.32 Examples of response-controlled structures in Japan and other countries

No.	Name of building	Location	No. of floors	Built-up area, m^2	Structure	Application	Year of construction	Remarks (details of response control)
1	2	3	4	5	6	7	8	9
1	M.I.E. building					Computer room floor		Ball bearing support
2	Dynamic floor					Computer room floor		Teflon sheets
3	Fudochokin Bank (now Kyowa Bank)	Himeji	+3, –1	791	RCC	Bank branch	1934	Sway-type hinge column
		Shimonoseki Tokyo	+3	641	RCC	Bank branch	1934	
4	Tokyo Science University	Auckland, New Zealand	+17, –1	14,436	Steel	School	1981	Double columns
5	Union House	Skopje, Yugoslavia	+12, –1		RCC	Office	1984	Double columns
6	Pestaloci Elementary School	California, USA	+3		RCC	School	1969	Rubber
7	Foothill Law and Justice Center	Wellington, New Zealand	+4, –1		Steel	Court	1986	Laminated rubber
8	W. Clayton Building	France	+4		RCC	Office	1983	Laminated rubber
9	Cruas Atomic Power Plant	South Africa			RCC	Atomic furnace	1984	Laminated rubber
10	Koeberg Atomic Power Plant	Chiba, Japan			RCC	Atomic furnace	1983	Laminated rubber
11	Hachiyodai Apartments	Tochigi, Japan	+2	114	RCC	Housing	1983	Laminated rubber
12	Okumura group, Okumura Research Center, Japan office wing	Miyagi, Japan	+4	1,330	RCC	Research centre	1986	Laminated rubber
13	Tohoku University, Shimizu Construction Laboratory		+3	200	RCC	Observatory	1986	Laminated rubber, viscous response control
14	Obayashi group, Technical Research Center, 61st Laboratory	Tokyo	+5, –1	1,624	RCC	Laboratory	1986	Laminated rubber
15	Fujizawa Industries, Fujizawa Plant, TC wing	Kanagawa, Japan	+5	4,765	RCC	Laboratory, office	1986	Laminated rubber
16	Funabashi Taketomo Dormitory	Chiba, Japan	+3	1,530	RCC	Dormitory	1987	Laminated rubber
17	Kashima Constructions Research Laboratory, West Chofu, Acoustic Laboratory	Tokyo	+2	655	RCC	Research laboratory	1986	Laminated rubber
18	Christian Data Bank	Kanagawa, Japan	+2	293	RCC	Data center	1988	Laminated rubber
19	Chiba Port Tower	Chiba, Japan	125 m	2,308	Steel	Tower	1986	Dynamic response control

Note: Data are collected from the earthquake reports, newspapers and T.V. records. They are put in the current format.

Table 2.32 (continued)

No.	Name of building	Location	No. of floors	Built-up area, m^2	Structure	Application	Year of construction	Remarks (details of response control)
1	2	3	4	5	6	7	8	9
20	Sydney Tower	Australia	325 m		Steel	Tower	1975	Dynamic response control
21	Citicorp Center	New York, USA	+ 59		Steel	Office	1977	Tuned mass response control
22	Hitachi Headquarters	Tokyo	+ 20, –3	57,487	Steel	Office	1983	Steel response control
23	World Trade Center	New York, USA	+ 110		Steel	Office	1976	VEM damper (viscous elastic mass)
24	Columbia Center	Seattle, USA	+ 76		Steel	Office	1985	VEM damper (viscous elastic mass)
25	Radar Construction	Chiba, Japan			Steel	Instrument Platform	1980	Roller bearing
26	Christchurch Chimney	Christchurch, New Zealand	35 m		RCC	Chimney		Steel response control
27	Commerce, Cultural Center	Saitama, Japan	+ 30		Steel	Office	1987	Friction response control
28	Fujita Industries Technical Research Laboratory, (6th Laboratory)	Kanagawa, Japan	+ 3	3,952	RCC	Research Center	1987	Laminated rubber
29	Shibuya Shimizu No. 1 Building	Tokyo	+ 5, –1	3,385	RCC	Office	1988	Laminated rubber
30	Fukumiya Apartments	Tokyo	+ 4	682	RCC	Cooperative housing	1988	Laminated rubber
31	Lambeso C.E.S.	France	+ 3	4,590	RC prefab	School	1978	Laminated rubber
32	Lambeso C.E.S.	France	+ 3	4,590	RC prefab	School	1978	Laminated rubber
33	Pestaloci Elementary School	Skopje, Yugoslavia	+ 3		RCC	School	1969	Laminated rubber
34	Cruas Atomic Power Plant	France			RCC	Atomic furnace		Laminated rubber
35	Kousberg Atomic Power Plant	South Africa			RCC	Atomic furnace		Laminated rubber
36	Pestaloci Elementary School	Skopje, Yugoslavia	+ 3		RCC	School	1969	Laminated rubber
37	Foothill Law and Justice Center	California, USA	+ 4, –1		Steel	Court	1983	Laminated rubber
38	W. Clayton Building	Wellington, New Zealand	+ 4		RCC	Office	1984	Laminated rubber
39	Hachiyodai Apartments	Chiba, Japan	+ 2	114	RCC	Housing	1983	Laminated rubber
40	Okumura group, Namba Research Center, Office Wing	Tochigi, Japan	+ 4	1,330	RCC	Research center	1986	Laminated rubber

Table 2.32 (continued)

No.	Name of building	Location	No. of floors	Built-up area, m^2	Structure	Application	Year of construction	Remarks (details of response control)
1	2	3	4	5	6	7	8	9
41	Obayashi group, Technical Research Center, 61 Laboratory	Tokyo	+ 5, –1	1,024	RCC	Laboratory	1986	Laminated rubber
42	Oires Industries, Fujizawa Plant, TC wing	Kanagawa, Japan	+ 5	4,765	RCC	Laboratory, Office	1986	Laminated rubber
43	Funabashi Taketomo Dormitory	Chiba, Japan	+ 3	1,530	RCC	Dormitory	1987	Laminated rubber
44	Kashima Constructions Research Laboratory, West Chofu, Acoustic Laboratory	Tokyo, Japan	+ 2	655	RCC	Research Laboratory	1986	Laminated rubber
45	Christian Data Bank	Kanagawa, Japan	+ 2	293	RCC	Data center	1988	Laminated rubber
46	Tohoku University, Shimizu Construction Laboratory	Miyagi, Japan	+ 3	200	RCC	Observatory	1986	Laminated rubber
47	Fujita Industries Technical Research Laboratory, 6th Laboratory	Kanagawa, Japan	+ 3	3,952	RCC	Research center	1987	Laminated rubber
48	Shibuya Shimizu No. 1 Building	Tokyo, Japan	+ 5, –1	3,385	RCC	Office	1988	Laminated rubber
49	Fukumiya Apartments	Tokyo, Japan	+ 4	685	RCC	Housing	1988	Laminated rubber
50	Shimizu Constructions Tsuchiura Branch	Ibaraki, Japan	+ 4	637	RCC	Office	1988	Laminated rubber
51	Toranomon 3-chome building	Tokyo, Japan	+ 8	3,373	RCC	Office	1989	Laminated rubber
52	National Institute for Research in inorganic materials, vibration free special laboratory	Ibaraki, Japan	+ 1	616	RCC	Laboratory	1988	Laminated rubber
53	Fudochokin Bank (now Kyowa Bank)	Himeji, Japan	+ 3, –1	791	RCC	Bank branch	1933	Sway-type hinge column
54	Fudochokin Bank (now Kyowa Bank)	Shimonoscki, Japan		641				Sway-type hinge column
55	Tokyo Science University	Tokyo, Japan	+ 17, –1	14,436	Steel	School	1981	Double columns
56	Union House	Auckland, New Zealand	+ 12, –1		RCC	Office	1983	Double columns
57	A Certain Radar	Chiba, Japan	+ 9 Atop the building	711	Steel platform	Instrument	1980	Roller bearing
58	Taisei Construction, Technical Research Center, J Wing	Kanagawa, Japan	+ 4	1,029	RCC	Office	1988	Sliding support

Table 2.32 (continued)

No.	Name of building	Location	No. of floors	Built-up area, m^2	Structure	Application	Year of construction	Remarks (details of response control)
1	2	3	4	5	6	7	8	9
59	World Trade Center	New York, USA	+110		Steel	Office	1976	VEM damper (viscoelastic material)
60	Columbia Center	Seattle, USA	+76		Steel	Office	1985	VEM damper
61	Commerce, Cultural Center	Saitama, Japan	+31	105,060	Steel	Office	1988	Friction damper
62	1-Chome Complex, Office Wing	Tokyo, Japan	+22	34,650	Steel	Office	1990 [sic]	Friction damper
63	Sydney Tower	Australia	325 m		Steel	Tower	1975	Dynamic damper
64	Citicorp Center	New York, USA	+59		Steel	Office	1977	Tuned mass damper
65	Chiba Port Tower	Chiba, Japan	125 m	2,204	Steel	Tower	1986	Dynamic damper
66	Toyama Park Observation Tower	Aichi, Japan	134 m	2,929	Steel	Tower	1988	Dynamic damper
67	Gold Tower		136 m	1,193	Steel	Tower	1988	Aqua damper
68	Yokohama Marine Tower	Kanagawa, Japan	103 m	3,325	Steel	Tower	1988	Super sloshing-damper

Note: + indicates floors above ground and – indicates floors below ground.
Data collected from the reports, newspapers and television records. They have been put in the current order.
Individual information collected from reports, newspapers and television records. They are placed and put in this order.
The data are taken from newspapers such as Time Index and placed in the given order.

Table 2.33 Seismically isolated buildings in the USA

Building	Height/ storeys	Floor area (m^2)	Isolation system	Date
Foothill Communities Law and Justice Center	4	17,000	10% damped elastomeric bearings	1985/86
Salt Lake City and County Building (Retrofit)	5	16,000	Rubber and lead-rubber bearings	1987/88
Salt Lake City Manufacturing Facility (Evans and Sutherland Building)	4	9,300	Lead-rubber bearings	1987/88
USC University Hospital	8	33,000	Rubber and lead-rubber bearings	1989
Fire Command and Control Facility	2	3,000	10% damped elastomeric bearings	1989
Rockwell Building (Retrofit)	8	28,000	Lead-rubber bearings	1989
Kaiser Computer Center	2	10,900	Lead-rubber bearings	1991
Mackay School of Mines (Retrofit)	3	4,700	10% damped elastomeric bearings plus PTFE	1991
Hawley Apartments (Retrofit)	4	1,900	Friction pendulum/slider	1991
Channing House Retirement Home (Retrofit)	11	19,600	Lead-rubber bearings	1991
Long Beach VA Hospital (Retrofit)	12	33,000	Lead-rubber bearings	1991

Note: Data is obtained from various Newyork Times.

Table 2.34 Bridges seismically isolated in Italy

No. of bridges	Name/location	Range of lengths (m)	Total length (m)	Superstructure type	Isolating system	Date completed
1	Somplago, Udine-Tarvisio		1,240	Precast segments	EL (neoprene disc)	1974
5	Tiberina E45		1,700		OL	1974
16	Udine-Tarvisio	240–900	7,900	Box girders	Long: elastom. Sleeves Transv: elastom. discs	1981–1986
3	Udine-Tarvisio	400–830	1,600	Box girders	Long: EP dampers Transv: elastom. discs	1983
1	Cellino, Road SS251		580	Concrete beams	EL (neoprene)	1983
3	Udine-Tarvisio	480–900	2,100	Steel girders	OL	1983–1986
1	Sesia, Trafori Highway		2,100		OL	1984
1	Bruscata, Greco		70	Steel truss	EL	1984
1	Pontebba, Udine-Tarvisio		960	Box girders	EL (elastomer)	1984
2	Milano-Napoli	350–780	1,100	Box girders	EP (steel)	1985
12	*Napoli-Bari	70–720	5,700	PCB boxed, piers or framed RC columns	Long: EP devices on abutments or on each span. Transv: EP on pier	1985–1988
1	Slizza 3, Udine-Tarvisio		160	Steel girders	EL	1985
1	Vallone, railway		240	Steel girders	EL	1985
1	Rivoli Bianchi, Udine-Tarvisio		1,000	Concrete beams	Pneumatic dampers	1985
2	Salerno-Reggio	600	1,400	Concrete beams	OL	1988
3	Fiano-San Cesareo	300–1,200	1850	Concrete beams	RB + metal shock	1986–1987
5	Fiano-San Cesareo	120–700	1,400	Box girders	RB + metal shock	1986–1987
6	Fiano-San Cesareo	100–650	1,600	Box girders/concrete beams	EL (rubber discs)	1986–1987
2	Fiano-San Cesareo	300–700	1,000	PCB	Viscoelastic shock absorber	1986–1987
3	*Napoli-Bari	130–200	500	PCB	LRB (long and transv)	1986
2	Milano-Napoli		170	Concrete beams	LRB	1986
2	Salerno-Reggio	350–900	1,200	PCB	OL	1987
1	Sizzine, Trafori Highway		1,800	PCB	OL	1987
1	Aqua Marcia, Milano-Napoli		325	Box girders	Long: EP Transv: EL dampers	1987
4	Monte Vesuvio		6,000	PCB	EL dampers with mechanical dissipators	1987–1990
12	Roma-Firenze railway	200–2,700	12,400	Box girders	OL	1987–1989
1	Lontrano, Salerno-Reggio		550	Box girders	OL	1988
1	Tagliamento, Pontebbana		1,000	PCB	Viscoelastic	1988

Note: Data collected from the Italian Embassy in London by the author.

Table 2.34 (continued)

No. of bridges	Name/location	Range of lengths (m)	Total length (m)	Superstructure type	Isolating system	Date completed
6	Roma-L'Aquila-Teramo	128–450	1,800	Box girders	EL (rubber + metal shock)	1988
1	Calore, Caserta (railway)		100	PCB	EL dampers + mechanical dissipators	1988
1	Granola, railway overpass		120	Concrete slab	Bearings + EL buffers	1988
2	Viaducts, San Mango	600–640	1,200	Steel girders	OL	1988–1990
1	Morignano, A14 highway		450	PCB	EP dampers	1989
1	*Lenze-Pezze, Napoli-Bari		300	PCB	EP dampers	1989
2	Vittorio Veneto – Pian di Vedoia	210–2,100	2,300	PCB	Long: Viscoelastic. Trans: EP	1989
1	Pont Suaz, Aosta		240	PCB	EP shock absorber	1989
1	Flumicello, Bologna-Firenze		300	PCB	OL	1989
1	Temperino, Roma-L'Aquila		830	PCB	EP dampers	1989
1	S.Onofrio, Salerno-Reggio		450	PCB	OL	1989
3	Roma-L'Aquila	230–1,300	1,800	Box girders	OL + RB	1989
1	*D'Antico, Napoli-Bari		250	Composite deck	EP	1989
1	Viadotto, Targia-Siracusa		23	Concrete beams	EP	1989
3	Napoli-Bari (retrofitted)	160–390	720	PCB	EP	1989–1990
1	*3rd Line, Roma-Napoli		580	Concrete beams	LRB	1990
7	*Milano-Napoli	100–200	1,000	PCB	EP	1990–1991
1	Santa Barbara, railway overpass		120	Concrete slab	EP	1990
1	Tora, Firenze-Pisa-Livorno		5,000	Steel girders	EP multidirectional	1990
3	Roma-L'Aquila	230–500	1,200	Box girders	Pneudynamic + RB	1990–1991
2	Salerno-Reggio	190, 390	600	Concrete beams	OL	1990
1	Railway Rocca Avellino		400	Concrete beams	OL	1990
1	SS 206, Firenze-Pisa-Livorno		2,500	Steel girders	EP	1990
1	Tiasca, Trafori highway		1,610	PCB	Elastic buffers	1990
1	Vesuvio, SS 269		1,860	PCB	Elastic buffers	1990
3	Messina-Palermo	900	900	Prestressed concrete box girder	EP (long)	1990
1	Mortaiolo, Livorno-Civitavecchio		9,600	Prestressed concrete slabs	EP with shock absorbers	1990–1992
1	S Antonio, Bretella		700	Prestressed concrete	EP with shock absorbers	1991
2	Salerno-Reggio	350. 500	850	PCB	EP	1991
2	PN-Conigliano	500, 800	1,300	Prestressed concrete	EP	1991
1	Minuto, Fondo Valle Sele		1,000	PCB	OL	1991
3	Roma-L'Aquila-Teramo	200–300	700	Box girders	OL	1991–1992

Table 2.34 (continued)

No. of bridges	Name/location	Range of lengths (m)	Total length (m)	Superstructure type	Isolating system	Date completed
1	Poggio Ibema, Livorno-Civitavecchia		2,500	PCB	OL	1991–1992
3	Livorno-Cecina	600–1800	2,800	PCB	EP, EP + RB	1991–1992
1	*Rumeano, Via Salaria		340	PCB	EP	Retrofit designed
1	Viadotto No 2, Tangenziale Potenza		240	PCB	EP	1990
1	Angusta, Siracusa		450	Boxed RC beams	EL	1990
7	*Salerno-R Calabria	100–500	1,800	PCB with connecting slabs	EP	Retrofit designed
1	Fragneto		870	Steel box girder with RC slabs	EP devices on piers, with ST long. Highest piers connected	Designed
1	Ponte Nelle Alpi, Via Veneto-Pian di Vedoia		310	Steel box girder with RC slabs	Long: EP with ST Transv: EP on all piers	Designed

Key:
EP = Elastic – plastic behaviour
EL = Elastic
OL = Oleodynamic system (EP equivalent)
SL = Sliding support
ST = Shock transmitter system associated with SL
RB = Rubber bearings
LRB = Lead-rubber bearings
RC = Reinforced concrete
PCB = Prestressed concrete beams

Notes:
Where bridges are two-way, they have been regarded as a single bridge in estimating the length.
The total length of isolated bridges is thus greater than 100 km.
Of the more recent bridges (1985–1992), typical design values of the parameters are
Yield/weight ratio: 5–28%, with a representative value of 10%.

- Maximum seismic displacement: ±30 to ±150 mm, with a representative value of ±60 mm.
- Peak ground acceleration: 0.15–0.40 g, with a representative value of 0.25 g.

Known retrofits are indicated with an asterisk (*)
Information and data collected from the Italian Embassy in London by the author.
Information collected from Research Library, Westminster, London by the author.
Research data from the Research Library, Westminster, London by the author.
Note: Research Data from the Times documents in Westminster Research Library, London by the author.

Table 2.35 Various applications and possibilities of using response-controlled structures in buildings

Effect/technical theme	Building application: Housing a	General office building b	Public high-rise building c	Disaster preventive building d	Art gallery/museums facility e	Atomic power facility f	Hospital facility g	Modern industrial facility h
1. Ensure safety of building structure	Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe
2. Freedom in design of cross-section of members	Economy, design freedom	Economy, design freedom						
3. Prevent vibrations, sliding, rolling of contents	Safe	Satisfactory performance	Satisfactory performance	Satisfactory perfor-mance	Protect the exhibits	Protect contents. Satisfactory performance. Prevent hazardous discharge	Protect contents. Satisfactory performance. Prevent hazardous discharge	Satisfactory performance. Prevent hazardous discharge
4. Prevent loss of secondary materials	Safe economy, design freedom	Safe economy, design freedom	Satisfactory performance	Satisfactory performance	Satisfactory performance	Satisfactory performance	Satisfactory performance	Satisfactory performance
5. Sensitivity control when the vibrations are not comfortable	Satisfactory living conditions	Satisfactory living conditions	Satisfactory living conditions					
6. Maintain proper functioning of machinery, equipment, etc.	Satisfactory performance	Satisfactory performance	Satisfactory performance					

Note: They are collected and then placed in the correct order from numerous literature on earthquakes and buildings related to earthquakes. They are based on constructed facilities.

Table 2.36

Item	(0) Hachiya Apartments	(1) Christian Data Center	(2) Okumura group	(3) Obayashi Group Technical Research Center	(4) Dieres Industries Fujizawa site TC wing	(5) Funabashi Taketomo Dormitory	(6) Okumura Group
Designed by	Tokyo Building Research Center	Tokyo Building Research Center, Unitika	Tokyo Building Research Center, Okumura group	Obayashi group	Oires Industries, Symiono Constructions, Yasui Building	Takenaka Komyten	–
Design requirements		Antiseismic. Prevent any damage to goods stored	Antiseismic. Protect computer and stored data for technical	Antiseismic. Protection of computer and other laboratory equipment	Designers	Safety and fire resistance during earthquake	–
No. of floors	+2	+2, –1	+4	No. of floors	+5	+5	+3
Built-up area, m^2	60.18	226.21	348.18	Built-up area, m^2	351.92	1136.5	719.28
Application	Housing (residential	Data house	Research center	Application	Laboratory	Research laboratory and office	Dormitory
Structure	RCC frame (shear wall)	RCC frame (shear wall)	RCC frame	Structure	RC	RC	RC
Foundation	Raft foundation with cast in situ piles	Deep foundation	RCC cast in situ raft	Foundation	PHC tie (cement grout method)	Concrete in situ raft (each-drilling method)	Concrete in situ raft (earth-drilling method)
Isolator: Dimensions, mm	82 × 300 dia	Rubber 5 thick × 300 dia (12 layers)	Rubber 7 thick × 500 dia (14 layers)	Isolator: Dimensions, mm	Rubber 4.4 thick × 740 dia (61 layers)	Rubber 10 thick × 24 dia (H = 363), OD = 650, 700, 750, 800)	Rubber 7 thick × 670 dia (19 layers) (H = 187)
							Rubber 8 thick × 700 dia (18 layers) (H = 195)
Numbers	6	32	25	Numbers	14	35	14
Supporting force	σ = 45 kg/cm^2, 0.5 t/cm (32 t)	σ = 60 kg/cm^2, 0.5 t/cm (42.5 t)	σ = 60 kg/cm^2, 0.86 t/cm (120 t)	Supporting force	200 t		200 t → 6 Nos.
							150 t → 8 Nos.
Response-control device	Friction force between PC plates	Uses plastic deformation of steel bars bent to make a loop (8 Nos.)	Uses plastic deformation of steel bars bent to make a loop (12 Nos.)	Response-control device	Uses elasto-plastic recovery of steel bars (96 Nos.)	Lead plug inserted in laminated rubber	Viscous damper (8 Nos.)
Shear force coefficient used in design	0.2	0.15	0.15	Shear force coefficient used in design	0.15	0.2	0.15
Fundamental period Small deformation	1.83 s	14 s	1.4 s	Fundamental period Small deformation	X = 1.33 s	X = 0.895 s	X = 2.09 s
					Y = 1.24 s	Y = 0.908 s (at 50% deflection)	Y = 2.10 s
Large deformation		1.9 s	2.1 s	Large deformation	X = 3.12 s	X = 2.143 s	

Note: Data collected from the British Library in London by the author and placed in the given order.

Table 2.36 (continued)

Item	(0) Hachiya Apartments	(1) Christian Data Center	(2) Okumura group	(3) Obayashi Group Technical Research Center	(4) Dieres Industries Fujizawa site TC wing	(5) Funabashi Taketomo Dormitory	(6) Okumura Group
					$Y = 3.08$ s	$Y = 2.148$ s (at 100 % deflection)	
Incident seismic wave	El Centro 1940 (NS)	El Centro 1940 (NS)	El Centro 1940 (NS)	Incident seismic wave	El Centro 1940 (NS)	El Centro 1940 (NS)	El Centro 1940 (NS)
	Hachinohe 1968 (NS)	Hachinohe 1968 (NS)	Taft 1952 (EW)		Hachinohe 1968 (NS)	Hachinohe 1968 (NS)	Taft 1952 (EW)
	Hachinohe 1968 (EW)	Taft 1952 (EW)	Hachinohe 1968 (NS)		Hachinohe 1968 (EW)	Hachinohe 1968 (EW)	Tokyo 101 1956 (NS)
	Taft 1952 (EW)				Taft 1952 (EW)	Taft 1952 (EW)	Hachinohe 1968 (NS)
					Man-made earthquake two waves	KT 008 1980 (NS)	Man-made earthquake four waves
						Man-made earthquake two waves	
Input level	300 gal	300 gal, 450 gal	300 gal, 450 gal	Input level	25 cm/s, 50 cm/s	25 cm/s, 50 cm/s	25 cm/s, 50 cm/s
Item	(6) Kashima Constructions Research Center, Nishi Chofu Acciystue	(7) Christian Data Center (reapplied)	(8) Fukumiya Apartments	(9) Shibuya Shimizu Building No. 1	(10) Fujita Industries, 6th Laboratory		(11) Inorganic Materials Research Center, Vibration-Free Wing
1	2	3	4	5	6		7
Designed by	Kashima Constructions	Tokyo Building Research Center	Okumura group	Obayashi group	Fujita Kogyo		Secretariat of the Ministry of construction, Planning Bureau
Design requirements	Reduce seismic input and insulate (isolate) from Earth's vibrations	Anti-seismic. Prevent any damage to stored goods	Safety of building. Added value in business	Protect the main building and OA equipment installed therein	Protect the main building and the equipment stored therein such as laboratory equipment and computers		Protect the main building and research equipment stored therein
No. of floors	+2	+2	+4	+5, –2	+3		+1
Built-up area, m^2	379.10	149.43	225.40	560.30	102.21		8,341 (old – 7,725; new – 616)
Application	Research laboratory	Data house	Cooperative housing	Office	Research laboratory		Research center
Structure	RC	RC	RC	RCC	RCC		RCC

Table 2.36 (continued)

Item	(0) Hachiya Apartments	(1) Christian Data Center	(2) Okumura group	(3) Obayashi Group Technical Research Center	(4) Dieres Industries Fujizawa site TC wing	(5) Funabashi Taketomo Dormitory	(6) Okumura Group
Foundation	Concrete in situ raft (deep foundation)	Deep foundation	Concrete in situ raft (miniature earth-drilling method)	Concrete in situ raft (earth-drilling method)	PHC pile (type A, B)	cement grout method	PHC raft (type A) blast method
Isolator: Dimensions, mm	320 × 1340 (48 thick × 5 dia); 308 × 1080 (38 thick × 6 dia)	4 thick × 435 dia (25 layers)		5.0 thick × 620 dia (50 layers); 6.0 thick × 740 dia (45 layers)	4.0 thick × 450 dia (44 layers)		3.2 thick × 420 dia (62 layers)
Numbers	18	12		20	4		32
Supporting force	165 t : 1340 dia	σ = 60 kg/cm^2; 0.55 t/cm (90 t)		100-150 t : 620 dia			65 t (max 80 t)
	100 t: 1080 dia			200-250 t : 740 dia			
Response-control device	Elasto-plastic damper using mild steel bars (14 Nos.)	Uses plastic deformation of steel bars bent to make a loop (6 Nos.)	Uses plastic deformation of steel bars bent to make a loop (7 Nos.)	Elasto-plastic damper using mild steel bars (48 Nos.)			Elasto-plastic damper using mild steel bars (108 nos.)
Shear force coefficient used in design	0.2	0.15	0.2	0.15: Basement, 1st floor; 0.183: 3rd floor ; 0.205: 5th floor	0.15: 1st floor, 0.17: 2nd floor; 0.20: 3rd floor		0.15
Primary period Small deformation	X = 0.828 s	1.3 s	1.4 s	X = 1.30 s	1.35 s		X = 1.17 s
	Y = 0.809 s			Y = 1.24 s			Y = 1.17 s
Large deformation	1.80 s	1.9 s	2.2 s	X = 2.99 s			X = 2.26 s
				Y = 2.97 s			Y = 2.26 s
Incident seismic wave	El Centro 1940 (NS)	El Centro 1940 (NS)	El Centro 1940 (NS)	El Centro 1940 (NS)	El Centro 1940 (NS)		El Centro 1940 (NS)
	Taft 1952 (EW)	Taft 1952 (EW)	Taft 1952 (EW)	Taft 1952 (EW)	Taft 1952 (EW)		Taft 1952 (EW)
	Tokyo 101 1956 (NS)	Hachinohe 1968 (NS)	Tokyo 101 1956 (NS)	Hachinohe 1968 (NS)	Hachinohe 1968 (NS)		Hachinohe 1968 (NS)
	Sendai THO38-1 1978 (EW)		Hachinohe 1968 (NS)	Hachinohe 1968 (EW)	Hachinohe 1968 (EW)		Hachinohe 1968 (EW)
				Sdkanrig, Sdkantig, Sdansrig } Man - made seismic waves	Artm 79L00 (seismic wave)	Tsukuba 85 NS, Tsukuba 85 EW, Tsukuba 86 NS, Tsukuba 86 EW }	Observed seismic waves
Input level	25 cm/s, 50 cm/s	300 gal, 450 gal	25 cm/s, 50 cm/s	25 cm/s, 50 cm/s	25 cm/s, 50 cm/s		25 cm/s, 50 cm/s

Notes:
Designers data collected by the author from stated companies in Japan.
Data collected from U.S.A and Japanese reports, British Library, Kingscross, London by the author. They are placed under topics.
Data collected from the companies in the reports available with British Library, London. They are placed under topics.

evaluated. The spectral acceleration level is about the same in all codes mentioned here. The velocity versus coverage (average) was taken by Soong et al. to be 210 m/s.

Data on Constructed Facilities

Some constructed building and other structural facilities have been examined with respect to the usage of seismic devices and are categorized on the basis of the types of structures and seismic devices installed to control and minimize various parameters inclusive of disastrous vibrations included by the earthquakes. Tables (2.34) to (2.36) give the details of the constructed facilities where various devices have been installed.

Bibliography

Abe, M., and Fujino, Y. Dynamic characteristics of multiple tuned mass dampers and some design formulas. *Earthquake Eng. Struct. Dyn.* 1994; 23(8), 813–836.

Algeria RPA (1999) Algerian Earthquake Resistant Regulation-Document Technique, Reglementaire (DTR) 1982

Argentina INPRESS-CIROSC 103 (1991) -Australian standard AS1170-4-1993 Minimum Design Roads on structures Pa A4, Earthquake loads 1996

Australia AS I 1704 (1993) -Australian standard AS1170-4-1993 Minimum Design Roads on structures Pa A4, Earthquake loads 1996

Beliayev, V.S., Vinogradov, V.V., and Guskov, V.D. Most recent development of the studies for seismic isolation of nuclear structures in Russia, International Post-Smirt Conference Seminar on Seismic Isolation, Passive Energy Dissipation and Control of Vibrations of Structures, Santiago, Chile, 1995.

Bishop, R.E.D. and Welbourn, D.B. The problem of the dynamic vibration absorber. *Engineering, London*, 1952, 174, 769.

Brock, J.E. A Note on the Damped Vibration Absorber. *J. Appl. Mech.* 13(4), A-284.

Carter, W.J., and Lin, F.C. Steady-state behavior of nonlinear dynamic absorber. *J. Appl. Mech.* 1961; 60-WA-14, 67–70.

Chang, J.C.H., and Soong, T.T. Structural control using active tuned mass dampers. *J. Eng. Mech. Div. ASCE* 1980; 106(EM6): 1091–1098.

China T J 11-78 and GB J11-89

Chowdhury, A.H., Iwuchukwu, M.D., and Garske, J.J. *The Past and Future of Seismic Effectiveness of Tuned Mass Dampers, Structural Control* (Leipholz, H.H.E., ed.), Martinus Nijhoff Publishers, Dordrecht, 1987; 105–127.

Clarke, A.J. Multiple passive tuned mass damper for reducing earthquake induced building motion, Proc. 9th Wld. Conf. Earthquake Eng., 5, Japan, 1988; 779–784.

Den Hartog, J.P. *Mechanical Vibrations*, 4th edition, McGraw-Hill, New York, 1956.

Ding, J.H., and Ou, J.P. Theoretical study and performance experiment for cylinder with holes viscous damper. *World Inf. Earthquake Eng.* 2001; 17(1): 30–35 (in Chinese).

Englekirk, R., *Seismic Design of Reinforcement and Precast Concrete Buildings*. Wiley, New York, 2003.

ENR. Hancock Tower Now to Get Dampers, Engineering News-Record, Oct. 30, 1975; 11.

ENR. Tower Cables Handle Wind, Water Tank Dampens It, Engineering News-Record, Dec. 9, 1971; 23.

Europ 1-1 OCT(94), Eurocode 8-Design provisions for Earthquake Resistance of structures ENV 1994–1998

Fajfar, P. Equivalent ductility factors taking into account low-cycle fatigue. *Earthquake Eng. Struct. Dyn.* 1992, 21, 837–848.

Falcon, K.C., Stone, B.J., Simcock, W.D., and Andrew, C. Optimization of vibration absorbers: A graphical method for use on idealized systems with restricting damping. *J. Mech. Eng. Sci.* 1967, 9, 374–381.

Frahm, H. Device for Damped Vibrations of Bodies, U.S. Patent No. 989958, Oct. 30, 1909.

Fu, Y., and Kasai, K. Comparative study of frames using viscoelastic and viscous dampers. *J. Struct. Eng., Am. Soc. Civil Eng.* 1998; 122(10), 513–522.

Fujita, T. Studies about vibration isolating floor using pre-stretched or pre-compressed springs (Part 2. Vibration characteristics and vibration isolator properties II.). *Seisan Kenkyu*, December, 1980; 32(10), 28–31.

Fujita, T., Hattori, S., and Ishida, J. Studies about vibration isolating floor using pre-stretched or pre-compressed springs (Part 1. Vibration characteristics and vibration isolator properties I.). *Seisan Kenkyu*, August, 1980; 32(8), 48–51.

Fujita, T., Hattori, S., and Ishida, J. Studies about vibration isolating floor using pre-stretched or pre-compressed springs (Part 3. Vibration characteristics and vibration isolator properties III.). *Seisan Kenkyu*, December, 1980; 32(12), 22–25.

Furukawa, Y., Kawaguchi, S., Sukagawa, M., Masaki, N., Sera, S., Kato, N., Washiyama, Y., and Mitsusaka, Y. Performance and quality control of viscous dampers. Proc. Structural Engineers World Congress (SEWC), Yokohama, JAPAN, CD-Rom, T3-3-3, 2002.

Garcia de Jalon, J., and Bayo, E. *Kinematic and Dynamic Simulation of Multibody Systems: The Real Time Challenge*. Springer, Berlin, 1994.

GB50011-2001. *Code for Seismic Design of Buildings*. China Building Industry Press, Beijing, China (in Chinese), 2001.

Housner, G.W. et al. Structural control: Past, present and future. *J. Struct. Eng. ASCE* 1997; 123(9): 897–971.

India Pakistan Criteria for Earthquake Resistant designing structures fifth revision Part I IS 1893 (Part 1) 2002

Iran ICSRD (1988) Iranian code for seismic design of Buildings 1988. Building Housing regional centre, Tehran.

Ishikawa, K., Okuma, K., Shimada, A., Nakamura, H., and Masaki, N. JSSI Manual for Building Passive Control Technology: Part-5 Performance and Quality Control of Viscoelastic Dampers (Companion Paper, Presented at 13WCEE).

ISO3010: 1988 Base for design of structures_Seismic Actions on Structures. Notification of No.1446 of Ministry of Construction, Japan/May 31, 2000. Notification of No.2009 of Ministry of Construction, Japan/May 31, 2000

Israel IC-413 (1994) SI 413 Design provision for Earthquake Resistant of structures. Standard Institute of Israel 1988.

Italy CNR-GNDT (1986) & Eurocode Technical Rulers for Constructions in seismic zones Public works ministry 2000.

Iwan, W.D., and Gates, N.C. Estimating earthquake response of simple hysteretic structures. *J. Eng. Mech. Div. ASCE* 1979; 105(EM3): 391–405.

Izumi, M. Mitsuhashi, H., Sasaki, T., Katsukura, H., and Aizawa, F. Studies about damping of vibrations in buildings. Basic studies about energy dissipation-type structure. *Nippon Kecnchiku Gakkai, Tohoku-shibu*, February, 1980; 85–88.

Jangid, R., and Londhe, Y. Effectiveness of elliptical rolling rods for base isolation. *J. Struct. Eng. (ASCE)* 1988; 124: 469–472.

Jangid, R.S. Response of pure-friction sliding structures to bi-directional harmonic ground motion. *Eng. Struct.* 1997; 19: 97–104.

Jangid, R.S. Seismic response of sliding structures to bi-directional earthquake excitation. *Earthquake Eng. Struct. Dyn.* 1996; 25: 1301–1306.

Japan BLEO (1981) Part 1, Earthquake Resistant Design of Structures Part 2, Earthquake Resistant Design of Building (Tokyo Japan, 2000)

JSSI Manual. Design and Construction Manual for Passively Controlled Buildings, Japan Society of Seismic Isolation (JSSI), First Edition, Tokyo, JAPAN, October (in Japanese, 405 pages), 2003.

Kanada, M., Kasai, K., and Okuma, K. Innovative passive control scheme: a Japanese 12-story building with stepping columns and viscoelastic dampers. Proc. Structural Engineers World Congress (SEWC), Yokohama, JAPAN, CD-ROM, T2-2-a-5, 2002.

Kasai, K., and Kibayashi, M. JSSI Manual for Building Passive Control Technology, Part-1 Manual Contents and Design Analysis Methods", 13WCEE, No.2989, 2004.

Kasai, K., and Kibayashi, M. JSSI Manual for Building Passive Control Technology: Part-1 Control Device and System (Companion Paper, Presented at 13WCEE).

Kasai, K., and Nishimura, T. Equivalent linearization of passive control system having oil damper bi-linearly dependent on velocity. *J. Struct. Constr. Eng.* (Transaction of AIJ), (in Review), (in Japanese), 2004.

Kasai, K., and Okuma, K. Accuracy enhancement of Kelvin-type modeling for linear viscoelastic dampers. A refined model including effect of input frequency on, 2002

Kasai, K., and Okuma, K. Kelvin-type formulation and its accuracy for practical modeling of linear viscoelastic dampers (Part1: One-Mass System Having Damper and Elastic / Inelastic Frame). *J. Struct. Constr. Eng.* (Transactions of AIJ), No. 550, 2001; 71–78, Dec. (in Japanese).

Kasai, K., Ito, H., and Watanabe, A. Peak response prediction rule for a SDOF elasto-plastic system based on equivalent linearization technique. J. Struct. Constr. Eng. (Transactions of AIJ), No. 571, 2003; 53–62, Sep. (in Japanese).

Kasai, K., Kibayashi, M., Takeuchi, T., Kimura, Y., Saito, Y., Nagashima, I., Mori, H., Uchikoshi, M., Takahashi, O., and Oohara, K. Principles and current status of manual for design and construction of passively-controlled buildings: Part-1: Background scope, and design concept. Proc. Structural Engineers World Congress (SEWC), Yokohama, JAPAN, CD-ROM, T2-2-a-1, 2002.

Kasai, K., Suzuki, A., and Oohara, K. Equivalent linearization of a passive control system having viscous dampers dependent on fractional power of velocity. J. Struct. Constr. Eng. (Transactions of AIJ), No. 574, 2003; 77–84, Dec. (in Japanese).

Kibayashi, M., Kasai, K., Tsuji, Y., Kato, S., Kikuchi, M., Kimura, Y., and Kobayashi, T. Principles and current status of manual for design and construction of passively-controlled buildings: Part-2: JSSI Criteria for Implementation of Energy Dissipation Devices. Proc. Structural Engineers World Congress (SEWC), Yokohama, JAPAN, CD-ROM, T3-3-1, 2002.

Kibayashi, M., Kasai, K., Ysuji, J., Kukuchi, M., Kimura, Y., Kobayashi, T., Nakamura, J., and Matsubsa, Y. JSSI Manual for Building Passive Control Technology, Part-2 Criteria for Implementation of Energy Dissipation Devices, 13WCEE, No. 2990, 2004.

Kobori, T, and Minai, R., Analytical study on active seismic response control: Seismic-response-controlled structure 1. *Trans. Architectural Inst. Japan* 1960; 66: 257–260 (in Japanese).

Kobori, T, and Minai, R., Condition for active seismic response control: seismic-response-controlled structure 2. *Trans. Architectural Inst. Japan* 1960; 66: 253–256 (in Japanese).

Kobori, T. Quake resistant and nonlinear problems of the structural vibrations to violent earthquake. *J. Kyoto University Disaster Prevention Laboratory*, 5th Anniversary Edition 1956; 116–124 (in Japanese).

Kobori, T. The direction of earthquake resistant engineering. *J. Architecture Building Sci.* 1958; 73: 5–9 (in Japanese).

Kobori, T., Koshika, N. Yamada, K., and Ikeda, Y. Seismic-response-controlled structure with active mass driver system. Part 1: Design. *Earthquake Eng. Struct. Dyn.* 1991; 20: 133–149.

Kobori, T., Koshika, N. Yamada, K., and Ikeda, Y. Seismic-response-controlled structure with active mass driver system. Part 2: Verification, Design. *Earthquake Eng. Struct. Dyn.* 1991; 20: 151–166.

Krieg, R.D., and Krieg, D.N. Accuracy of numerical solution methods for the elastic, perfectly plastic model. *Pres. Ves. Technol.* 1997; 99: 510–515.

Kumaya, K., Arano, T., Kurihara, M., and Sasai, H. Structure incorporating sliding elements and rubber in the foundations of buildings (Part 2. Rocking vibrations developed in a vibration isolator structure developed at the time of sliding). *Nippon Kikai Gakkai Koen Ronbun-shu*, No. 800–3, 1980; 89–91.

Kumaya, K., Arano, T., Kurihara, M., Sasai, H. Structure incorporating sliding elements and rubber in the foundations of buildings (Part 2. Methods of response analysis in a multi-freedom systems and comparison with experimental findings). *Nippon Kikai Gakkai Koen Ronbun-shu*, No. 800–3, 1980; 92–94.

Li, Z., Rossow, E.C., and Shah, S.P. Sinusoidal forced vibration of sliding masonry system. *Struct. Eng. ASCE* 1989; 115: 1741–1755.

Liaw, T.C., Tian, Q.L., and Cheung, Y.K., Structures on sliding base subjected to horizontal and vertical motions. *Struct. Eng. ASCE* 1988; 114: 2119–2129.

Lin, B.C., and Tadjbakhsh, I.G. Effect of vertical motion on friction driven systems. *Earthquake Eng. Struct. Dyn.* 1986; 14: 609–622.

Lin, T.W., Chern, C.C., and Hone, C.C. Experimental study of base isolation by free rolling rods. *Earthquake Eng. Struct. Dyn.* 1995; 24: 1645–1650.

Lu, L.Y., and Yang, Y.B. Dynamic response of equipment in structures with sliding support. *Earthquake Eng. Struct. Dyn.* 1997; 26: 61–77.

Makris, N., Dargush, G.F., and Constantinou, M.C. Dynamic analysis of viscoelastic fluid dampers. *J. Eng. Mech. ASCE*, 1995; 121(10), 1114–1121.

Masri, S.F., Bekey, G.A., and Caughey, T.K. On-line control of nonlinear flexible structures. *J. Appl. Mech. ASME* 1982; 49: 877–884.

Mathworks, Inc., MATLAB, Reference Guide. Natick, MA, 1999.

Matsushita, S., Nomura, M., and Sasaki, T. Recovery properties of steel pipe type response-control device. *Nippon Kenchiku Gakkai, Tohoku-shibu*, March, 1980; 129–132.

Mexico UNAM (1983) M111 (1988)

Miyazaki, N., and Mitsusaka, Y. Design of a building with 20% or greater damping, Tenth World Conf. Earthquake Eng., Madrid, 1992; 4143–4148.

Mokha, A., Constantinou, M.C., and Reinhorn, A.M. Verification of friction model of Teflon bearings under triaxial load. *Struct. Eng. ASCE* 1993; 119: 240–261.

Mori, M., Arano, T., Kataoka, T., Ochiai, K., and Sasai, H. Structure incorporating sliding elements and rubber in the foundations of buildings (Part 1). *Nippon Kikai Gakkai Koen Ronbun-shu*, No. 800–3, 1980; 88–91.

Mostaghel, N., and Khodaverdian, M. Dynamics of resilient-friction base isolator (R-FBI). *Earthquake Eng. Struct. Dyn.* 1987; 15: 379–390.

Mostaghel, N., and Tanbakuchi, J. Response of sliding structures to earthquake support motion. *Earthquake Eng. Struct. Dyn.* 1983; 11: 729–748.

Mostaghel, N., Hejazi, M., and Tanbakuchi, J. Response of sliding structure to harmonic support motion. *Earthquake Eng. Struct. Dyn.* 1983; 11: 355–366.

Nakagawa, K., Watanabe, S., and Shimaguchi. S. Experimental studies about dynamic floor system (Part 2) – Vibration experiment with a computer using vibration table. *Dairin-gumi gijutsu Kenkyusho-ho*, No. 17, 1978; 17–21.

Nakata, Y. Performance and quality control of steel hysteretic damper. Proc. Structural Engineers World Congress (SEWC), Yokohama, Japan, CD-ROM, T3-3-5, 2002.

Nakata, Y., Hirota, M., Shimizu, T., and Iida, T. JSSI manual for building passive control technology: Part-6 performance and quality control of steel dampers (Companion Paper, Presented at 13WCEE).

New Zealand NZS 4203 (1992) and NZNSEE (1988) general structures designs, design loading for buildings. Standards association of New Zealand 2004.

Nishitani, A. Application of active structural control in Japan. *Prog. Struct. Eng. Mater.* 1998; 1(3): 301–307.

Niwa, N., Kobori, T., Takahashi, M., Hatada, T., Kurino, H., and Tagami, J. Passive seismic response controlled high-rise building with high damping device. *Earthquake Eng. Struct. Dyn.* 1995, 24, 655–671.

Okuma, K., Ishikawa, K., Oku, T., Sone, Y., Nakamura, H., and Masaki, N. Performance and quality control of viscoelastic dampers. *Proc. Structural Engineers World Congress (SEWC)*, Yokohama, Japan, CD-ROM, T3-3-4, 2002.

Okuma, K., Kasai, K., and Tokoro, K. JSSI manual for building passive control technology: Part-10 time-history analysis model for viscoelastic dampers (Companion Paper, Presented at 13WCEE), 2002.

Oldham, K.B., and Spanier, J. *The Fractional Calculus*. Academic Press, New York, 1974.

Ooki, Y., Kasai, K., and Amemiya, K. JSSI manual for building passive control technology: Part-11 Time-history Analysis Model for Viscoelastic Dampers Combining Iso-Butylene and Stylene Polymers (Companion Paper, Presented at 13WCEE).

Ou, J.P., and Zou, X. Experimental study on properties of viscoelastic damper. *J. Vibration Shock* 1999; 18(3): 12–19 (in Chinese).

Ou, J.P., He, Z., Wu, B., and Long, X. Seismic damage performance design for RC structures. *J. Building* (in Chinese), 2001; 21:1, 63–73.

Ou, J.P., Niu, D., and Wang, G. Seismic damage estimation and optimal design of nonlinear RC structures. *J. Civil Eng.* (in Chinese), 1993; 26:5, 14–21.

Ou, J.P., Wu, B., and Long, X. Parameter analysis of passive energy dissipation systems. *Earthquake Eng. Vibration* 1998; 18(1): 60–70 (in Chinese).

Park, Y.J., and Ang, A. H.-S. Mechanistic seismic damage model for reinforced concrete. *J. Struct. Eng.* 1985; 111(4): 722–739.

Pekcan, G., Mander, J.B., and Chen, S.S. The seismic response of a 1:3 scale model R.C. structure with elastomeric spring dampers. *Earthquake Spectra*, 1995; 11(2), 249–267.

Press, W.H., Teukolsky, S.A., Vetterling, W.T., and Flannery, B.P. Numerical recipes in FORTRAN, Cambridge University Press, Cambridge, UK, 1992.

Petersen et al. Dynamic analysis with tuned mass dampers private communication (Two reports 1995).

Qumaroddin, M., Rasheedazzafar, Arya, A.S., and Chadra, B. Seismic response of masonry buildings with sliding substructure. *Struct. Eng. ASCE* 1986; 112: 2001–2011.

Reinhorn, A.M., and Li, C. Experimental and analytical investigation of seismic retrofit of structures with supplemental damping, Part III: Viscous Damping Walls, Technical Report NCEER-95-0013, National Centre for Earthquake Engineering Research, Buffalo, NY, 1995.

Reinhorn, A.M., Li, C., and Constantinou, M.C. Experimental and analytical investigation of seismic retrofit of structures with supplemental damping, Part I: Fluid Viscous Damping Devices, Technical Report NCEER-95-0001, National Centre for Earthquake Engineering Research, Buffalo, NY, 1995.

Schwahn, K.J., and Delinic, K. Verification of the reduction of structural vibrations by means of viscous dampers, Seismic Engineering, ASME, Pressure Vessel and Piping Conf., Pittsburgh, PA, 1988, 144, 87–95.

Sekiguchi, Y., Kasai, K., and Ooba, K. JSSI manual for building passive control technology: Part-13 Time-history Analysis Model for Viscous Dampers (Companion Paper, Presented at 13WCEE).

Sekiguchi, Y., Kasai, K., and Takahashi, O. JSSI manual for building passive control technology: Part-12 time-history analysis model for nonlinear oil dampers. (Companion Paper, Presented at 13WCEE).

Shames, I.H., and Dym, C.L. Energy and *Finite Element Methods in Structural Mechanics*. Hemisphere Publishing, New York, 1985.

Shimazu, T., and Hidco, A. Basic experimental studies on vibration isolator properties of layered walls. *Nippn Kenchiku Gakkai Taikai*, September, 1980; 755–757.

Soong, T.T., and Dargush, G.F. *Passive Energy Dissipation Systems in Structural Engineering*. Wiley, New York, 1997.

Spencer Jr. B.F., and Sain, M.K. Controlling buildings: A new frontier in feedback. IEEE Control Systems Magazine: Special Issue on Emerging Technologies (Samad, T. Guest ed.), 1997; 17(6): 19–35.

Suresh, S. *Fatigue of Materials*. Cambridge University Press, Cambridge, 1991.

Tanaka, K., Kawagushi, S., Sukagawa, M., Masaki, N., Sera, S., Washiyama, Y., and Mitsusaka, Y. JSSI manual for building passive control technology: Part-4 Performance and Quality Control of Viscous Dampers (Companion Paper, Presented at 13WCEE).

Taylor, D.P., and Constantinou, M.C. Fluid dampers for applications of seismic energy dissipation and seismic isolation. Proceedings of the Eleventh World Conference on Earthquake Engineering, Acapulco, Mexico, 1996.

Tezcan, S., and Civi, A. Reduction in Earthquake Response of Structures by Means of Vibration Isolators, Proc. Second U.S. National Conf. on Earthquake Engrg., Earthquake Engrg. Research Institute, Stanford Univ., California, 1979; 433–442.

Tsopelas, P., and Constantinou, M.C. NCEER-Taisei Corporation Research Program on Sliding Seismic Isolation Systems for Bridges: Experimental and Analytical Study of a System Consisting of Sliding Bearings and Fluid Restoring Force/Damping Devices, Technical Report NCEER-94-0014, National Centre for Earthquake Engineering Research, Buffalo, NY, 1994.

Tsuyuki, Y., Kamei, T., Gofuku, Y., Iiyama, F., and Kotake, Y. Performance and quality control of oil-damper. *Proc. Structural Engineers World Congress (SEWC)*, Yokohama, Japan, CD-ROM, T3-3-2, 2002.

Tsuyuki, Y., Kamei, T., Gofuku, Y., Kotake, Y., and Iiyama, F. JSSI manual for building passive control technology: Part-3 Performance and Quality Control of Oil Dampers (Companion Paper, Presented at 13WCEE).

USA UBC. 95 (1995) 2 SEAOC (1990) ASCE Minimum Design Loads for Buildings and other structures, ASCE, USA.

Vidic, T., Fajfar, P., and Fischinger, M. Consistent inelastic design spectra: Strength and displacement. Earthquake Eng. Struct. Dyn. 1992; 23: 507–521.

Warburton, G.B., and Soni, S.R. Errors in response calculations for non-classically damped structures. Earthquake Eng. Struct. Dyn. 1977, 5, 365–376.

Westermo, B., and Udwadia, F., Periodic response of a sliding oscillator system to harmonic excitation. *Earthquake. Eng. Struct. Dyn.* 1983; 11: 135–146.

Yang, J.N. Application of optimal control theory to civil engineering structures. *J. Eng. Mech. Div. ASCE* 1975; 101(6): 819–838.

Yang, Y.B., Lee, T.Y., and Tsai, I.C. Response of multi-degree-of-freedom structures with sliding supports. *Earthquake Eng. Struct. Dyn.* 19, 739–752, 1990

Yao, J.T.P. Concept of structural control. *J. Struct. Div. ASCE* 1972; 98(ST7): 1567–1574.

Yasui, H., and Iniwa, C. Experimental studies about dynamic floor system (Part1) – Sinusoidal forced damped vibration experiment on a large-scale model. Dairin-gumi Gijutsu Kenkyusho-ho, No. 16, 1978; pp. 46–50.

Yasunishi, K., and Tada, H. Discussions about aseismic isolator (steel, rubber lamination pad). *Nippon Kenchiku Gakkai Taikai*, September, 1980; 751–753.

Younis, C.J., and Tadjbakhsh, I.G. Response of sliding structure to base excitation. *Eng. Mech. ASCE* 1984; 110: 417–432.

http://www.springer.com/978-3-540-93817-0

Earthquake Resistant Buildings
Dynamic Analyses, Numerical Computations, Codified
Methods, Case Studies and Examples
Bangash, M.Y.H.
2011, XXXII, 706 p., Hardcover
ISBN: 978-3-540-93817-0