Determination of Seismic Design Forces

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The original Powerpoint presentation was prepared by

Jag Humar of Carleton University

which is greatly appreciated.

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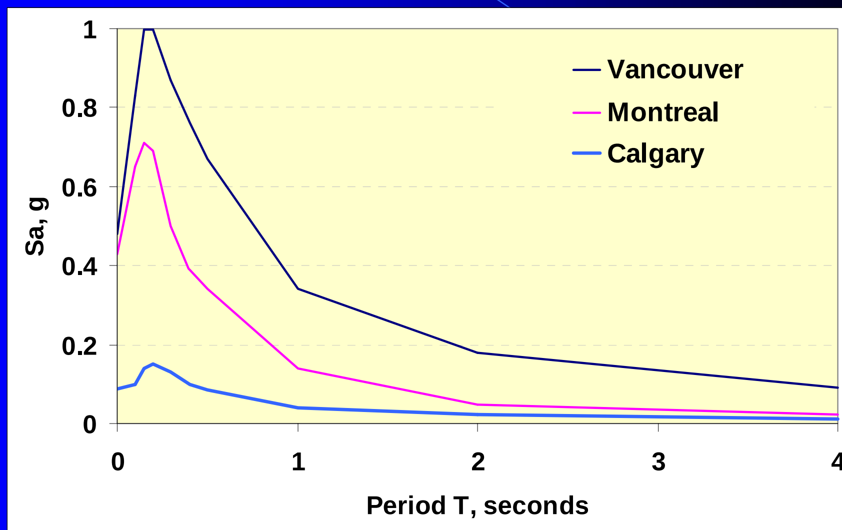
Overview

- UHS spectrum
- Foundation factors
- Design spectrum
- Static load provisions
- Periods
- M_v and J factors
- Torsion
- Dynamic analysis

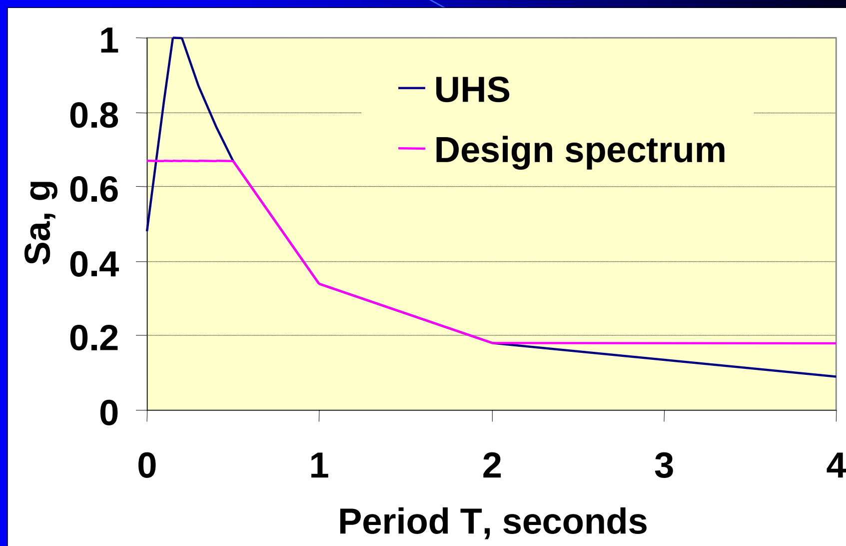
2%/50 year UHS, firm ground (soil class C), 5% damping



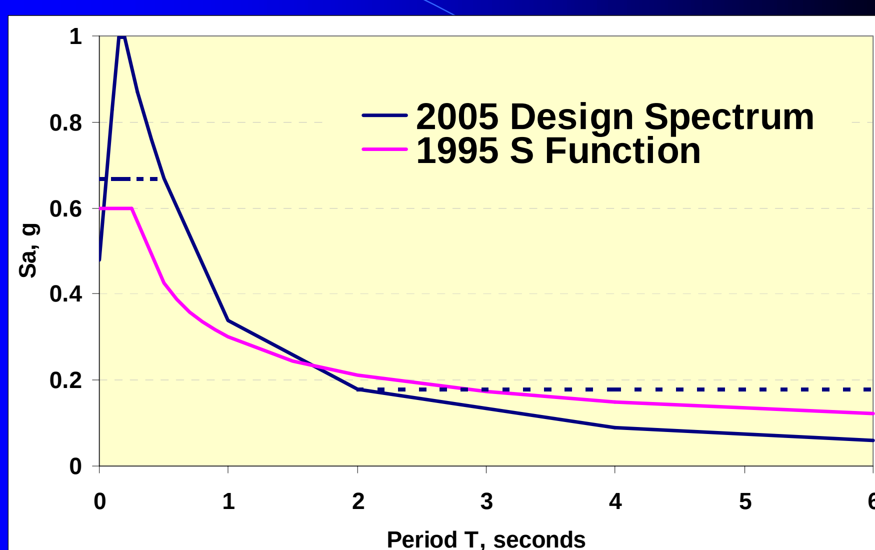
Representation Of Seismic Hazard Uniform Hazard Spectrum

- Provides maximum acceleration for a 5% damped SDOF system at selected periods, $S_a(T)$
- Spectral values derived for a uniform probability of exceedance, 2% in 50 years
- Does not include the subduction earthquake

Vancouver, 2%50 year hazard



Vancouver NBCC 2005 vs 1995



Foundation Factor

- Foundation soil generally amplifies wave motion as it propagates up through the layer
- Amplification depends on stiffness and depth of soil, as well as on ground motion intensity
- Foundation factor *F* in NBCC 1995 will be replaced by factor F_a for short periods and F_v for long periods

Foundation Site Classification

Site class	*Soil* profile name	Average properties in top 30 m		
		Soil shear wave average velocity V_s, m/s	Standard penetration resistance, N_{60}	*Soil* undrained shear strength, s_u, kPa
A	Hard *Rock*	> 1500	NA	NA
B	*Rock*	760 to 1500	NA	NA
C	Very dense *Soil* and soft *Rock*	360 to 760	> 50	> 100
D	Stiff *Soil*	180 to 360	15 to 50	50 to 100
E	Soft *Soil*	< 180	< 15	< 50
E		Any profile with more than 3m of soil with the following characteristics • Plastic index PI ≥ 20 • Moisture content w ≥ 40%, and • Undrained shear strength s_u < 25 kPa		
F	[(1)]Others			

[(1)] Other soils include:

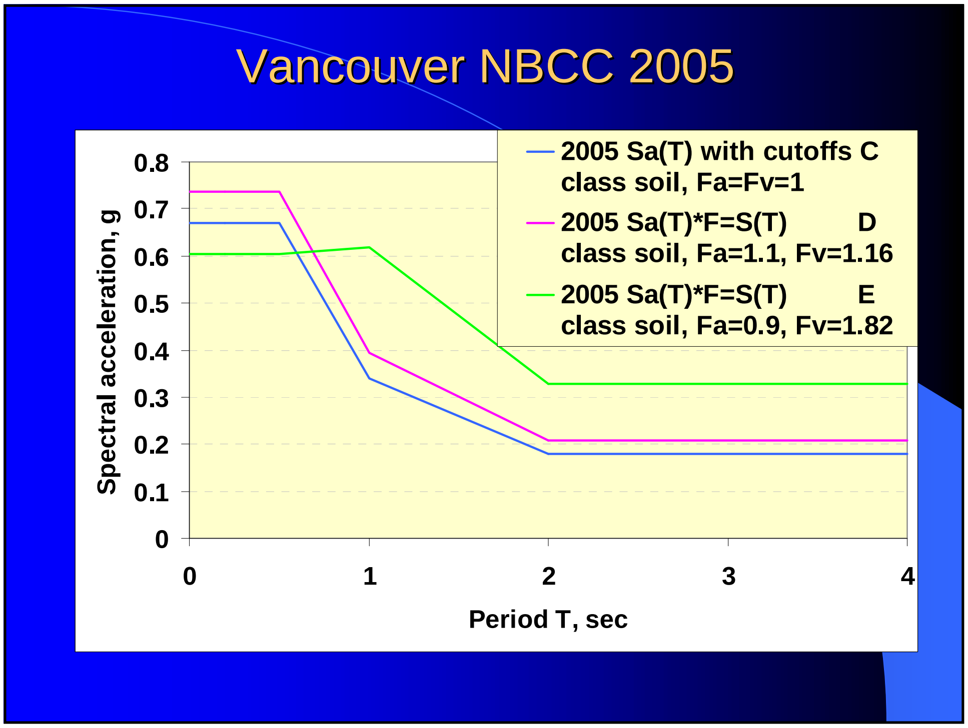
- Liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils, and other soils susceptible to failure or collapse under seismic loading.
- Peat and/or highly organic clays greater than 3m in thickness
- Highly plastic clays (PI > 75) with thickness greater than 8m
- Soft to medium stiff clays with thickness greater than 30m

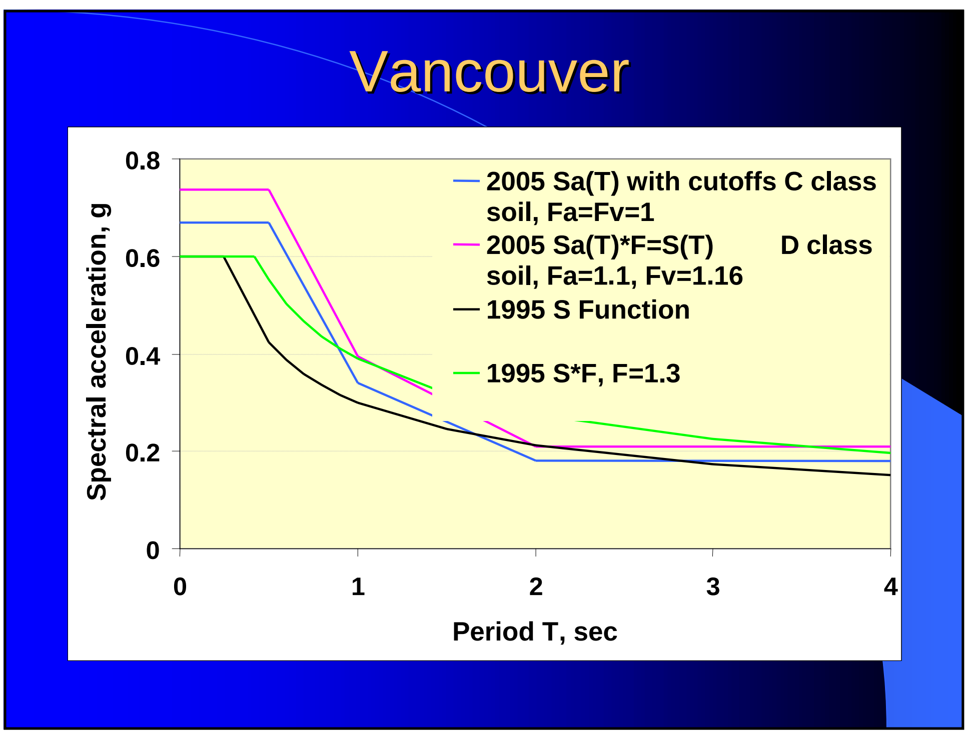
F_a					
Site Class	Sa(0.2)				
	≤ 0.25	0.50	0.75	1.00	1.25
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	Site specific geotechnical investigation and dynamic site response required				

Fv					
Site Class	$S_a(1.0)$				
	≤ 0.10	0.20	0.30	0.40	≥ 0.50
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	Site specific geotechnical investigation and dynamic site response required				

Design Spectral Acceleration

$$
\begin{aligned}
S(T) &= F_a S_a(0.2) \quad \text{for } T \le 0.2\,\text{s} \\
&= F_v S_a(0.5) \text{ or } F_a S_a(0.2) \\
&\quad \text{whichever is smaller,} \\
&\quad \text{for } T = 0.5\,\text{s} \\
&= F_v S_a(1.0) \quad \text{for } T = 1.0\,\text{s} \\
&= F_v S_a(2.0) \quad \text{for } T = 2.0\,\text{s} \\
&= F_v S_a(2.0)/2 \quad \text{for } T \ge 4.0\,\text{s}
\end{aligned}
$$


Vancouver NBCC 2005
2005 Sa(T) with cutoffs C class soil, Fa=Fv=1
2005 Sa(T)*F=S(T) D class soil, Fa=1.1, Fv=1.16
2005 Sa(T)*F=S(T) E class soil, Fa=0.9, Fv=1.82
Spectral acceleration, g
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
Period T, sec


Vancouver
2005 Sa(T) with cutoffs C class soil, Fa=Fv=1
2005 Sa(T)*F=S(T) D class soil, Fa=1.1, Fv=1.16
1995 S Function
1995 S*F, F=1.3
Spectral acceleration, g
0.8
0.6
0.4
0.2
0
0
1
2
3
4
Period T, sec

Obtaining Design Forces from UHS

- Dynamic analysis procedure (default method)
- Equivalent static load procedure (allowed for some structures)

Conditions under which Static Load Procedure may be used

- Structures located in zones of low seismicity, that is, $I F_a S_a(0.2) < 0.35$, or
- Regular structures that are less than 60 m in height and have $T_a < 2$ s, where T_a is the fundamental period, or
- Irregular structures that are less than 20 m in height, have $T_a < 0.5$ s and are not torsionally sensitive

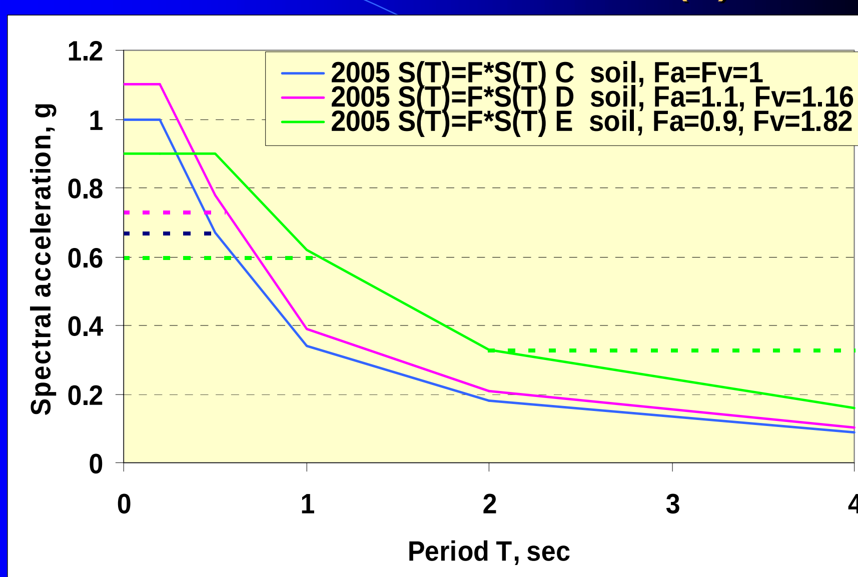
Equivalent Static Load Procedure
Elastic Base Shear

- Elastic base shear is derived from

  $V_e = S(T_a)M_V W,$ where

- T_a is the fundamental period,
- W is the weight of the structure contributing to inertia forces, and
- M_V is a factor to account for higher mode shears

Vancouver NBCC 2005 S (T)



Equivalent Static Load procedure Design Base Shear

$$V = V_e I / R_d R_o$$

V = factored design base shear

I = importance factor, 1.0, 1.3, or 1.5

R_d = ductility related force modification factor

R_o = overstrength related force modification factor

Equivalent Static Load Procedure Design Base Shear

- Because of uncertainty associated with the S_a values for periods greater than 2.0 s, $S(T_a)$ is taken as $S(2.0)$ for $T_a > 2.0$
- For ductile structures where R_d is 1.5 or more, the following upper limit is specified on the design shear (affects short periods)

$$V = \frac{2}{3}\frac{S(0.2)IW}{R_d R_o}$$

Importance Factor

I = 1.3	Buildings used as post disaster shelters, such as, schools and community centres, and manufacturing facilities containing toxic, explosive or hazardous substances
I = 1.5	Buildings used for post-disaster recovery, such as, hospitals, telephone exchanges, generating stations, fire and police stations, water and sewage treatment facilities
I = 1.0	All other buildings

Fundamental Period T_a

Calculation of period by analytical methods allowed, but the value should be limited to a multiple of the empirical value; 1.5 for moment frames, 2.0 for braced frames and shear walls. Reasons are:

- Possible inaccuracies in modelling
- Difference between design and as-built condition
- Uncertainties regarding participation of non-structural elements

Fundamental Period Empirical Expressions

For concrete frames

$T = 0.075\,(h_n)^{3/4}$

For steel frames

$T = 0.085\,(h_n)^{3/4}$

For other moment frames

$T = 0.1N$

where h_n in the height above base in m and N is the total number of storeys above grade

Fundamental Period Empirical Expressions

Braced frames $T = 0.025\,h_n$

Shear wall buildings $T = 0.05(h_n)^{3/4}$

The corresponding NBCC 1995 formula was

$$T = \frac{0.09\,h_n}{\sqrt{D_S}}$$

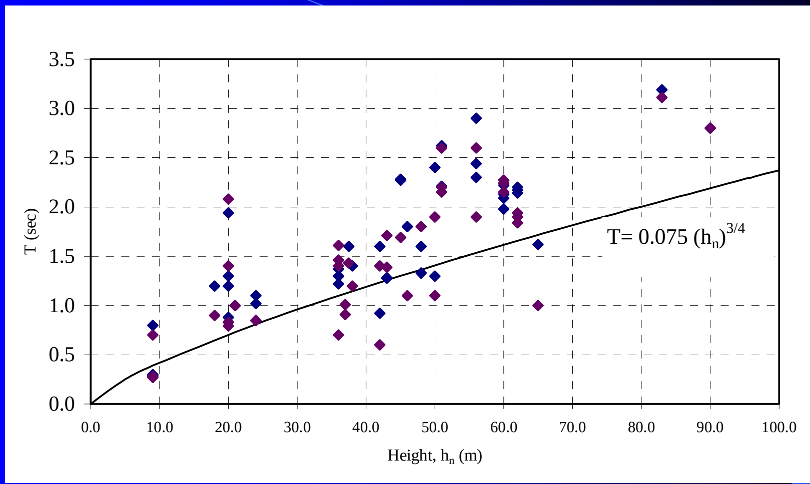


Fig. 2: Comparison of empirical and measured periods for concrete moment frame buildings

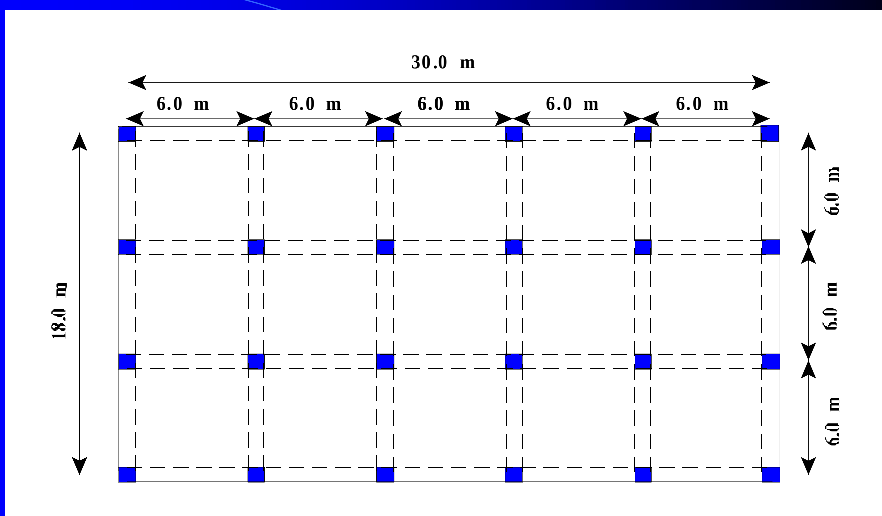


Fig. 3a: Concrete frame buildings, plan

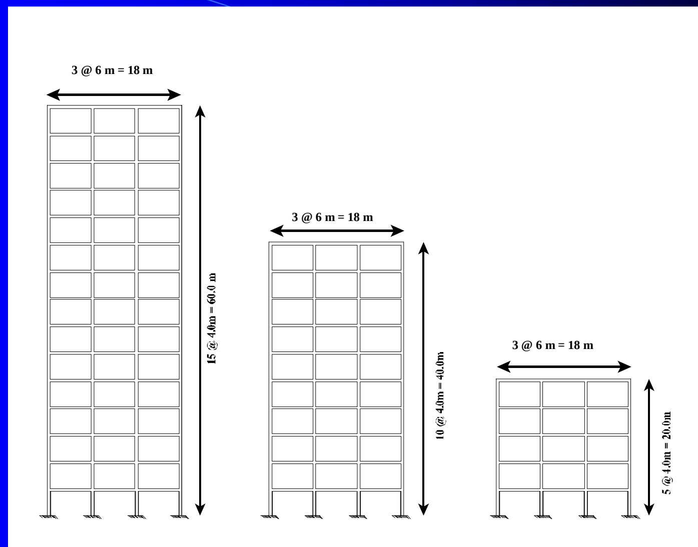


Fig. 3b: Concrete frame buildings, Elevations

Empirical Versus Analytical Periods

Building height	NBCC period s	Analytical period s
5	0.7	1.7
10	1.2	3.7
15	1.6	5.7

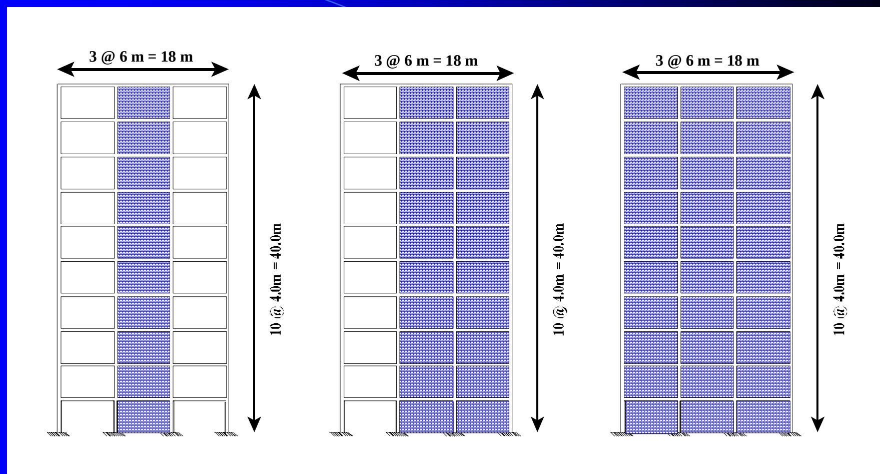


Fig. 4: masonry infill walls in exterior frames

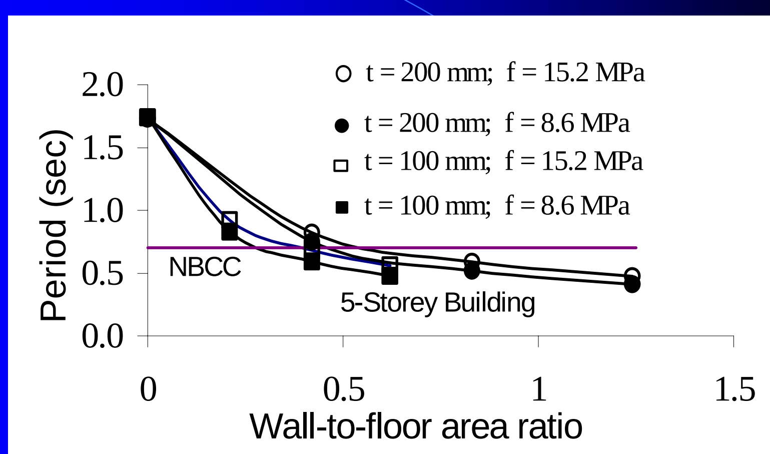


Fig. 5: Effect of masonry infills, 5-storey building

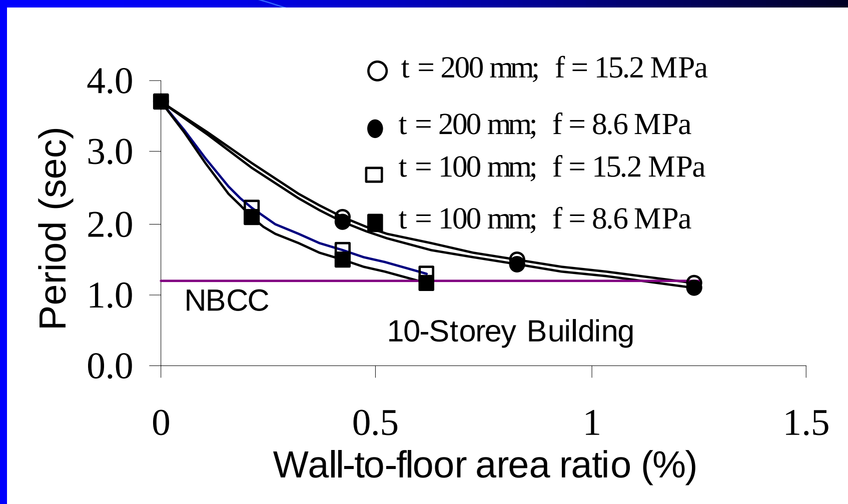


Fig. 6: Effect of masonry infills, 10-storey building

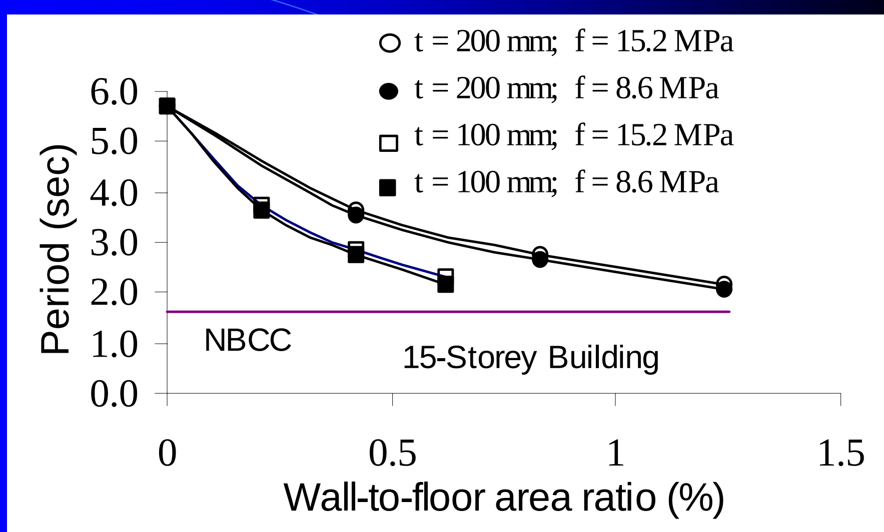


Fig. 7: Effect of masonry infills, 15-storey building

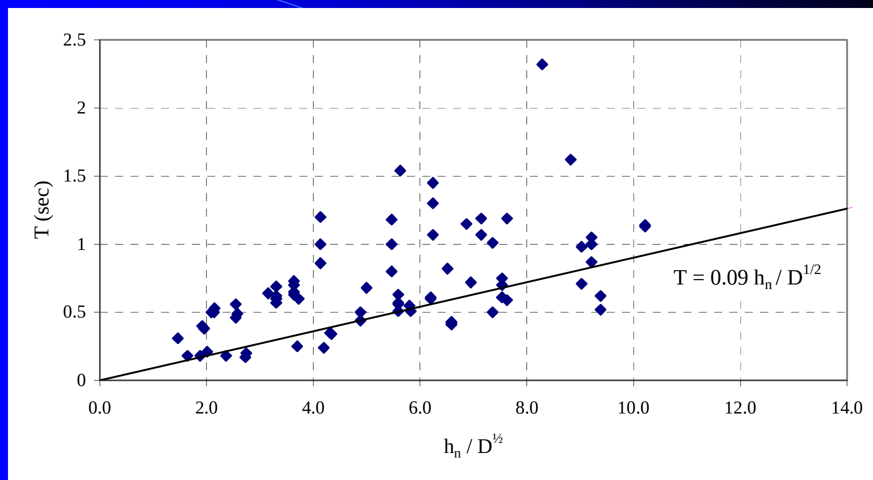


Fig. 8: Comparison of empirical and measured periods, concrete shear wall buildings, 1995 code period formula.

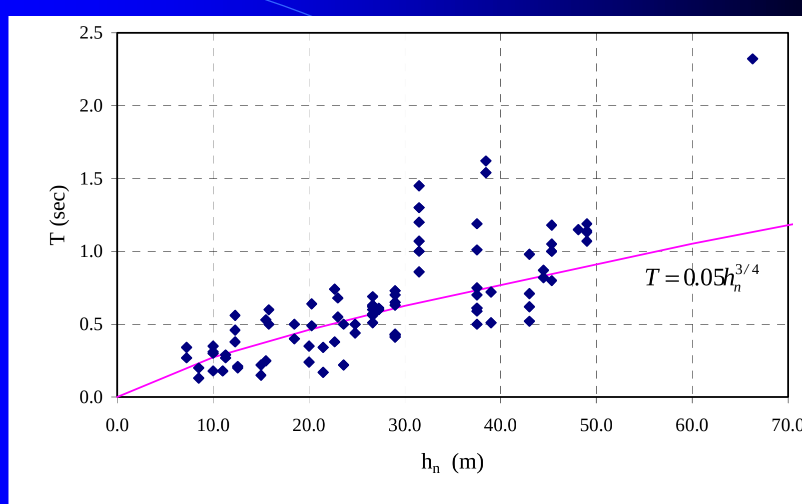


Fig. 9: Comparison of empirical and measured periods, concrete shear wall buildings, 2005 code period formula.

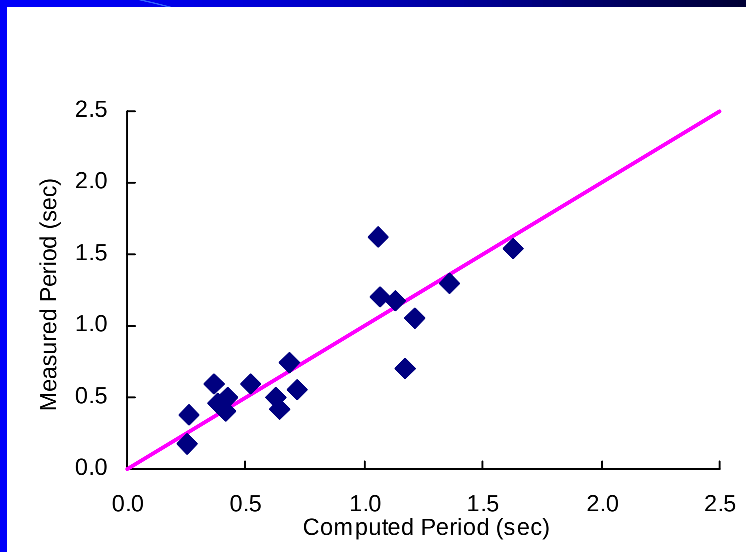


Fig. 10: Comparison of analytical and measured periods, concrete shear wall buildings

Equivalent Static Load Procedure Elastic Base Shear

- Elastic base shear is derived from

$$V_e = S(T_a)M_V\,W, \quad \text{where}$$

T_a is the fundamental period,

W is the weight of the structure contributing to inertia forces, and

- M_V is a factor to account for higher mode shears

Effect of Higher Modes on Base Shear, M_v factor

Relative contribution of higher modes depends on

1. Spectral shape
2. Relative modal periods
3. Mass participation factor

Table 3: Proposed base shear and overturning moment adjustment factors, M_v and J for different structural systems [1,2]

$\frac{S_a(0.2)}{S_a(2.0)}$	Type of lateral force resisting system	M_v for T ≤ 1.0	M_v for T ≥ 2.0	J for T ≤ 0.5	J for T ≥ 2.0
< 8.0 WEST	Moment-resisting frames or "coupled walls" [3]	1.0	1.0	1.0	1.0
	Braced frames	1.0	1.0	1.0	0.8
	Walls, wall-frame systems, other systems [4]	1.0	1.2	1.0	0.7
> 8.0 EAST	Moment-resisting frames or "coupled walls" [3]	1.0	1.2	1.0	0.7
	Braced frames	1.0	1.5	1.0	0.5
	Walls, wall-frame systems, other systems [4]	1.0	2.5	1.0	0.4

Notes:

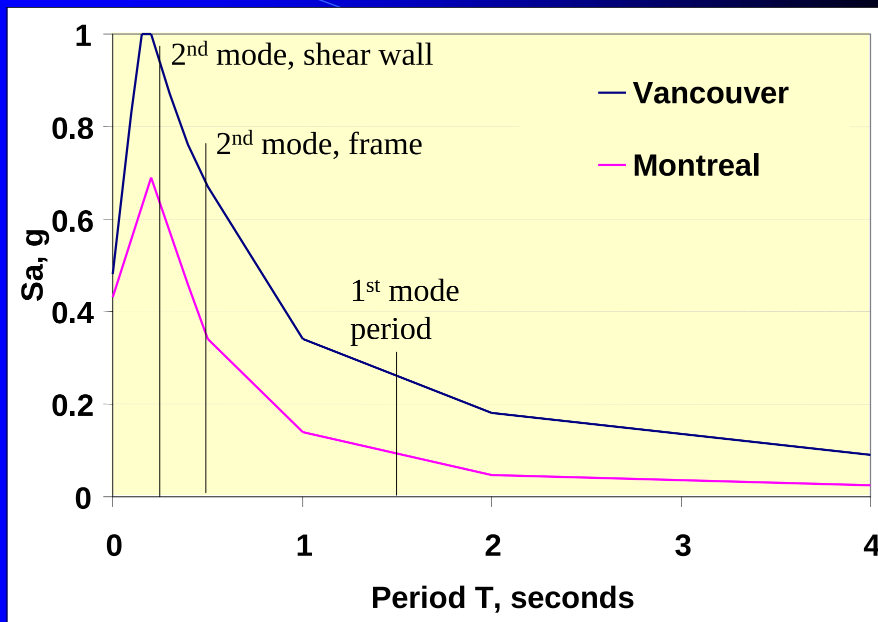
1. Values of M_v between periods of 1.0 and 2.0 s are to be obtained by linear interpolation.
2. Values of J between periods of 0.5 and 2.0 s are to be obtained by linear interpolation.
3. Coupled wall is a wall system with coupling beams where at least 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces resulting from shear in the coupling beams
4. For hybrid systems, use values corresponding to walls or carry out a dynamic analysis

Relative modal periods and modal weights for flexural and shear cantilevers

Mode	Uniform shear wall		Uniform frame, stiff beams	
No.	Period	Modal weight	Period	Modal weight
1	1.000	0.616	1.000	0.811
2	0.167	0.188	0.333	0.090
3	0.057	0.065	0.200	0.032
4	0.030	0.032	0.143	0.017
5	0.018	0.020	0.111	0.010

Effect of Higher Modes on Base Shear - example

- Consider a building structure with first mode period of 1.5 s
- Assume that contribution from only the first two modes are significant
- Use the UHS for Vancouver and Montreal and the data in previous table



Design shears in a building of two different structural types located in Vancouver and Montreal

Structure type	1st mode period	2nd mode period	Modal weight in 1st mode	Modal weight in 2nd mode	Spectral acceleration (g) in 1st mode	Spectral acceleration (g) in 2nd mode	Base shear in 1st mode	Base shear in 2nd mode	SRSS shear	Base shear assuming entire weight at 1st mode period	M_v
Vancouver											
Frame	1.50	0.50	0.811W	0.090W	0.256	0.630	**0.208W**	**0.057W**	**0.216W**	**0.256W**	**0.84**
Shear wall	1.50	0.25	0.616W	0.188W	0.256	0.900	**0.158W**	**0.169W**	**0.231W**	**0.256W**	**0.90**
Montreal											
Frame	1.50	0.50	0.811W	0.090W	0.073	0.340	**0.059W**	**0.031W**	**0.067W**	**0.073W**	**0.91**
Shear wall	1.50	0.25	0.616W	0.188W	0.073	0.600	**0.045W**	**0.113W**	**0.122W**	**0.073W**	**1.67**

Methodology for Estimating Shear Adjustment Factor

$$M_v = \frac{\sqrt{\sum \{S_a(T_i)W_i\}^2}}{S_a(T_1)W} = \frac{V_b}{S_a(T_1)W}$$

$S_a(T_i) =$ spectral acceleration in the i th mode

$T_i =$ i th mode period

$W_i =$ modal weight in the i th mode

Shear Adjustment Factor *M_v*

Structural types studied

1. Moment-resisting frame
2. Concentrically braced frame
3. Flexural wall
4. Coupled flexural walls
5. Hybrid frame-wall system

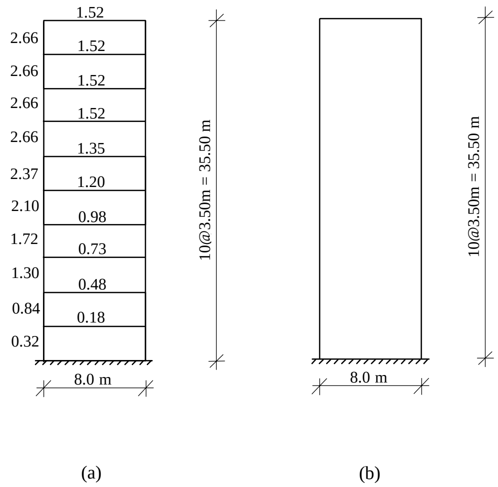


Fig. 12: Elevations (a) Moment frame, (b) Shear wall

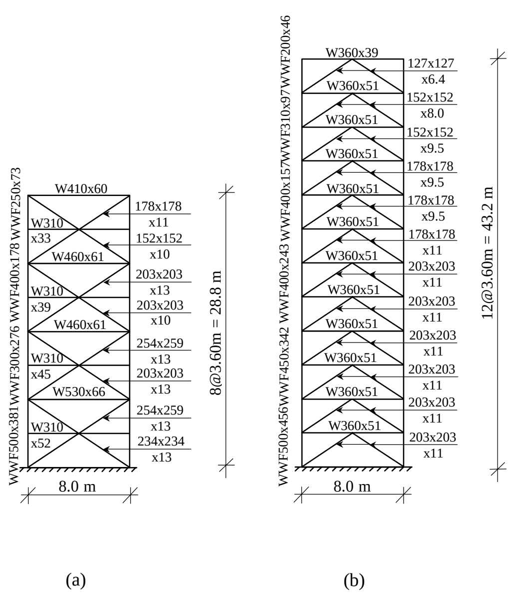


Fig. 13: Elevations, braced frames

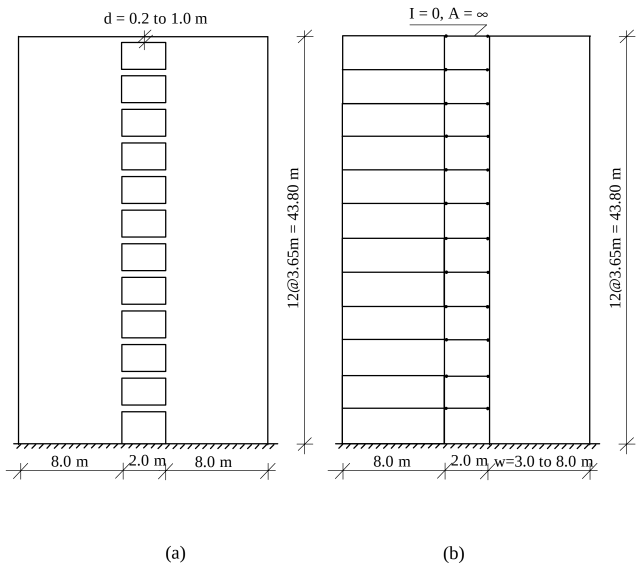


Fig. 14: Elevations (a) coupled flexural walls, (b) hybrid frame-wall system

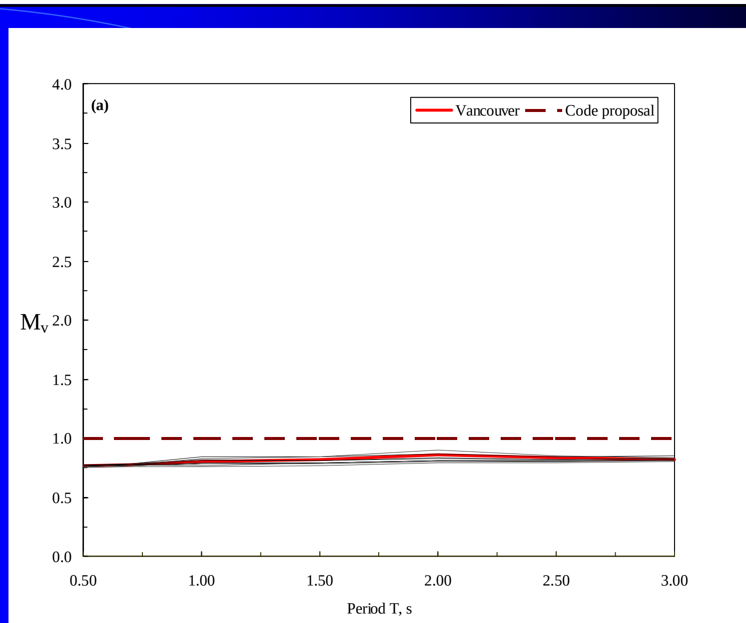


Fig. 15a: M_v factor, moment frames, cities in the west

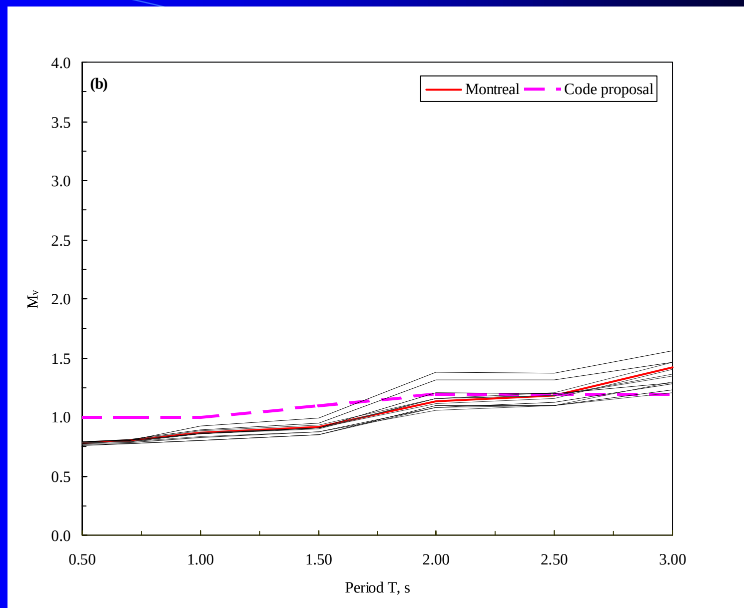


Fig. 15b: *Mv* factor, moment frames, cities in the east

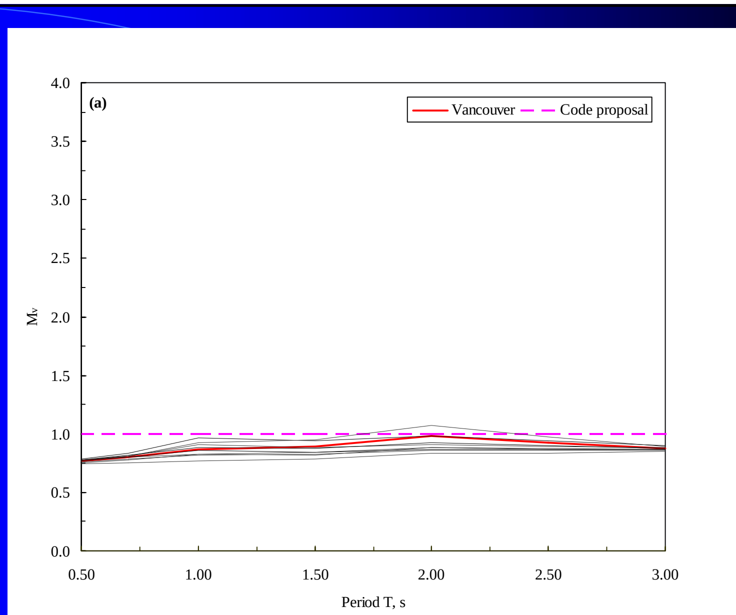


Fig. 16a: M_v factor, braced frames, cities in the west

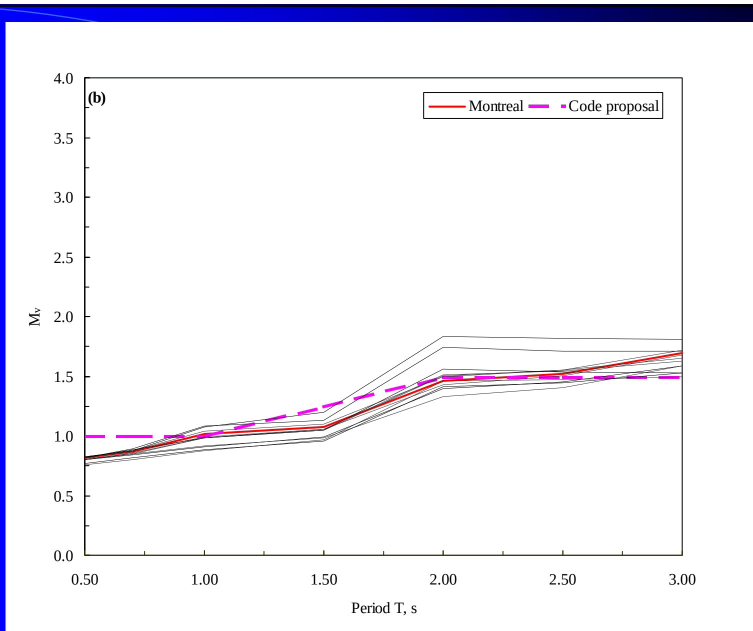


Fig. 16b: M_v factor, braced frames, cities in the east

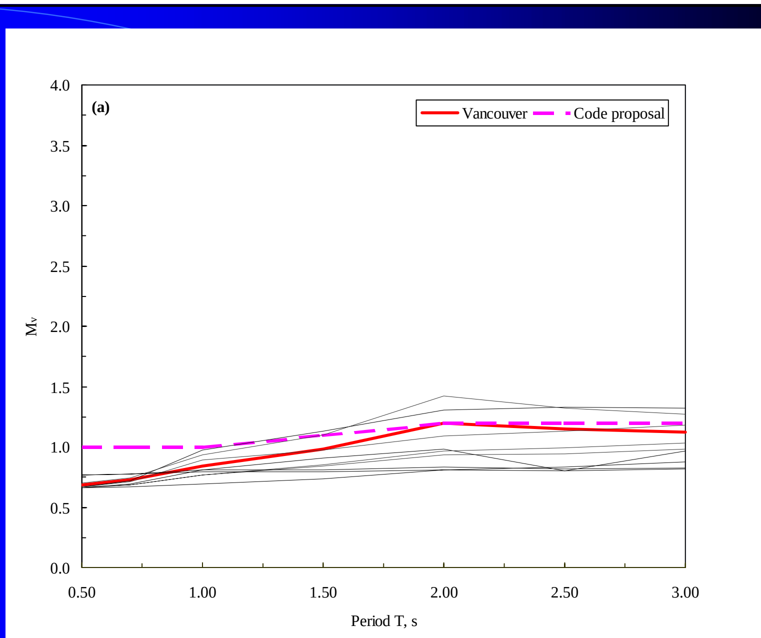


Fig. 17a: *Mv* factor, shear walls, cities in the west

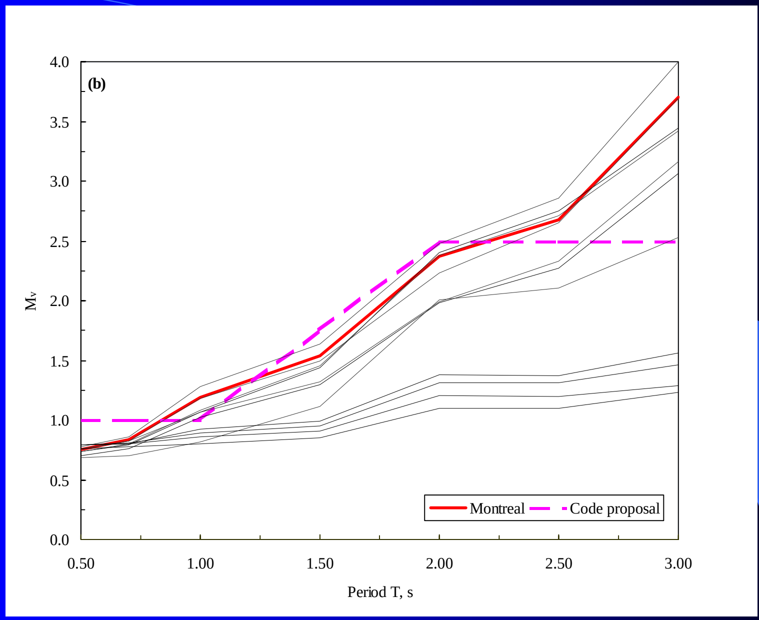


Fig. 17b: M_v factor, shear walls, cities in the east

Distribution of Shear

Similar expressions are used in NBCC 1995 and NBCC 2005

$$F_i = \frac{W_i h_i}{\sum W_i h_i}(V - F_t)$$

$$\begin{aligned} F_t &= 0 && \text{for} && T_a \le 0.7 \\ F_t &= 0.07 T_a V && \text{for} && 0.7 < T_a < 3.6 \\ F_t &= 0.25V && \text{for} && T_a \ge 3.6 \end{aligned}$$

Overturning Moments

- Estimates of overturning moment depend on the manner in which the base shear is distributed up the height
- First mode distribution gives the highest overturning moments
- Higher mode effects are important near the top of the structure

Moment Adjustment Factor *J*

- NBCC distribution is based predominantly on first mode, except for F_t applied at top
- An adjustment factor J is applied to the base overturning moment to account for higher mode effects (and F_t)
- Adjustment factor J_x is applied to the calculated moment at level x

Determination of J, J_x factor

1. Determine from the code static shear forces the overturning moment at the base, M_{bc}, and at each level x, M_{xc}.
2. Obtain more precise estimates of the base moment, M_{bd}, and at each level, M_{xd}, by a dynamic response spectrum analysis
3. Then, $J = M_{bd}/M_{bc}$, $J_x = M_{xd}/M_{xc}$

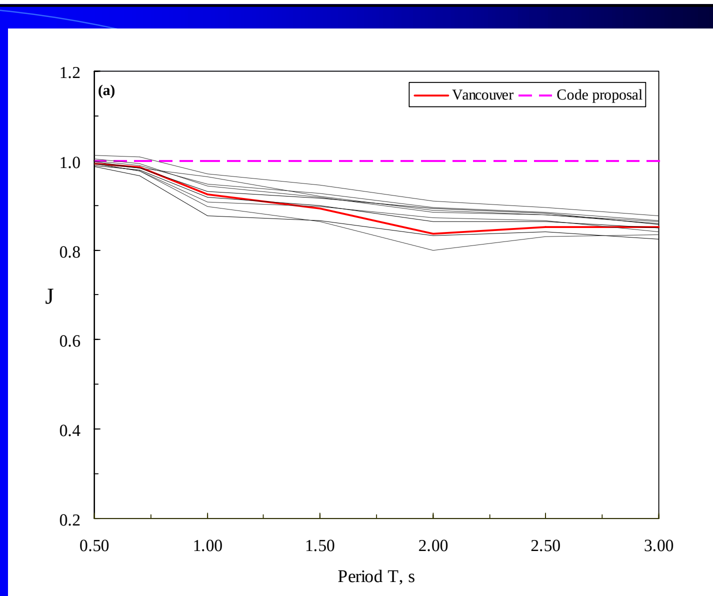


Fig. 18a: *J* factor, moment frames, cities in the west

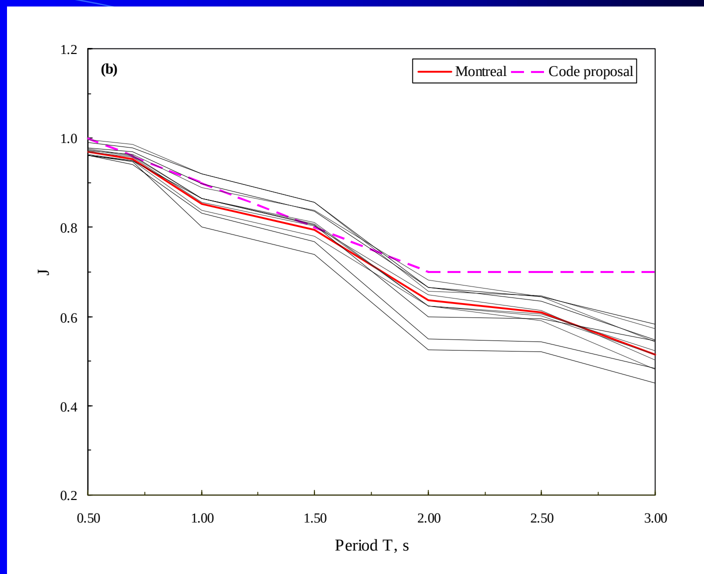


Fig. 18b: *J* factor, moment frames, cities in the east

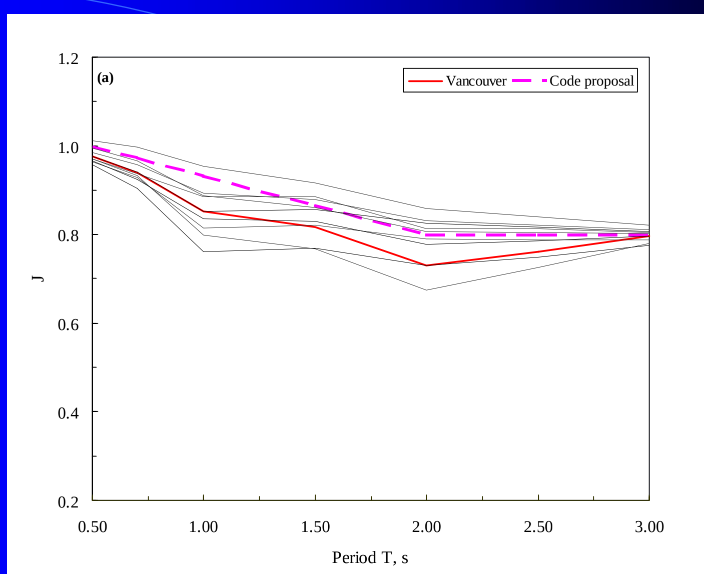


Fig. 19a: *J* factor, braced frames, cities in the west

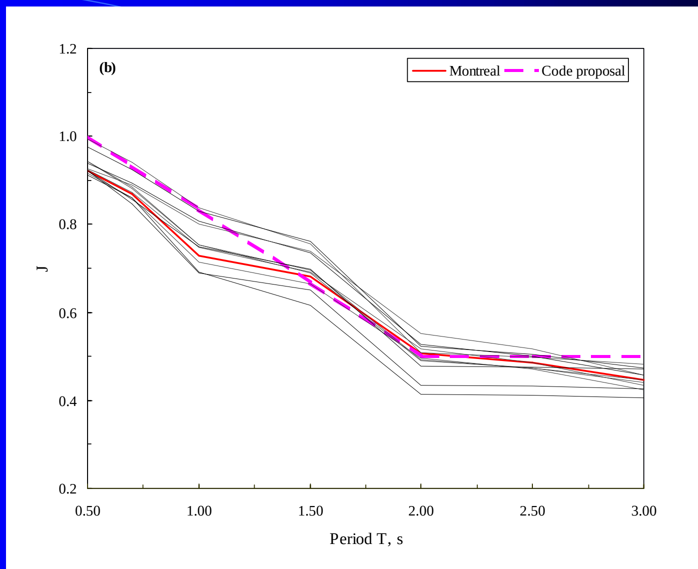


Fig. 19b: *J* factor, braced frames, cities in the east

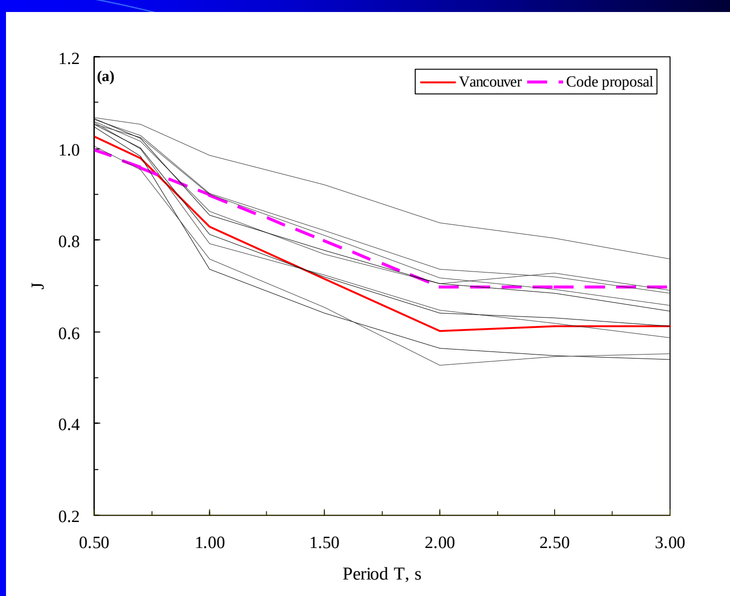


Fig. 20a: *J* factor, shear walls, cities in the west

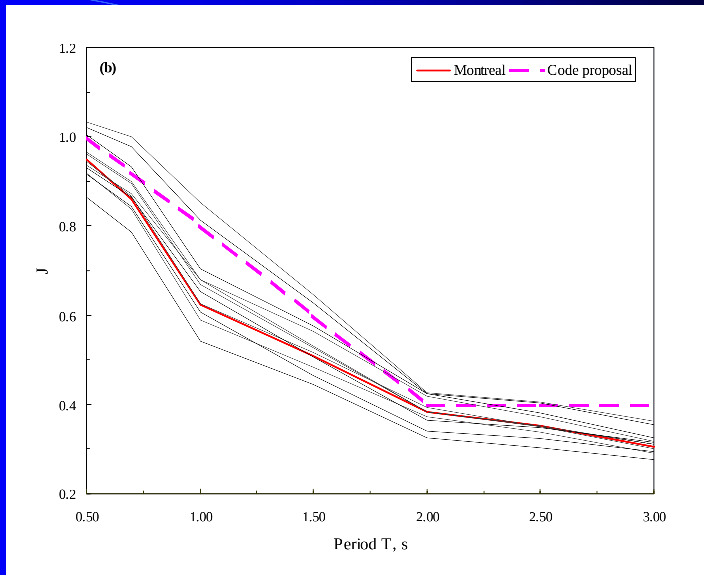


Fig. 20b: *J* factor, shear walls, cities in the east

Proposed base shear and overturning moment adjustment factors, M_v and ***J*** for different structural systems [1,2]

$\frac{S_a(0.2)}{S_a(2.0)}$	Type of lateral force resisting system	**M_v** T ≤ 1.0	**M_v** T ≥ 2.0	**J** **T ≤ 0.5**	**J** T ≥ 2.0
< 8.0	Moment-resisting frames or "coupled walls" [3]	1.0	1.0	1.0	1.0
	Braced frames	1.0	1.0	1.0	0.8
WEST	Walls, wall-frame systems, other systems [4]	1.0	1.2	1.0	0.7
>8.0	Moment-resisting frames or "coupled walls" [3]	1.0	1.2	1.0	0.7
	Braced frames	1.0	1.5	1.0	0.5
EAST	Walls, wall-frame systems, other systems [4]	1.0	2.5	1.0	0.4

Notes:

1. Values of M_v between periods of 1.0 and 2.0 s are to be obtained by linear interpolation.
2. Values of *J* between periods of 0.5 and 2.0 s are to be obtained by linear interpolation.
3. Coupled wall is a wall system with coupling beams where at least 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces resulting from shear in the coupling beams
4. For hybrid systems, use values corresponding to walls or carry out a dynamic analysis

Overturning Moment Reduction Factor, J_x

$$J_x = 1.0 \quad \text{for} \quad h_x \geq 0.6h_n$$

$$J_x = J + (1 - J)\frac{h_x}{h_n} \quad \text{for} \quad h_x < 0.6h_n$$

i.e., J_x = 1 for top 40% of the building,
then reduces linearly to J_x = J
at the base of the building.

Provisions for Torsion Design

NBCC 1995 design eccentricities

$e_{d1} = 1.5e + 0.1b$

$e_{d2} = 0.5e – 0.1b$

NBCC 2005 design eccentricities

$e_{d1} = e + 0.1b$

$e_{d2} = e – 0.1b$

- 0.05b is considered adequate to account for accidental torsion
- The remaining 0.05*b* should take care of the amplification of the natural torsion
- The 0.1*b* accidental torsion can be applied dynamically by shifting the mass by ±0.05b if the structure is 'torsionally stiff' (B≤1.7), else it must be applied statically

Forces Induced by Seismic Torsion

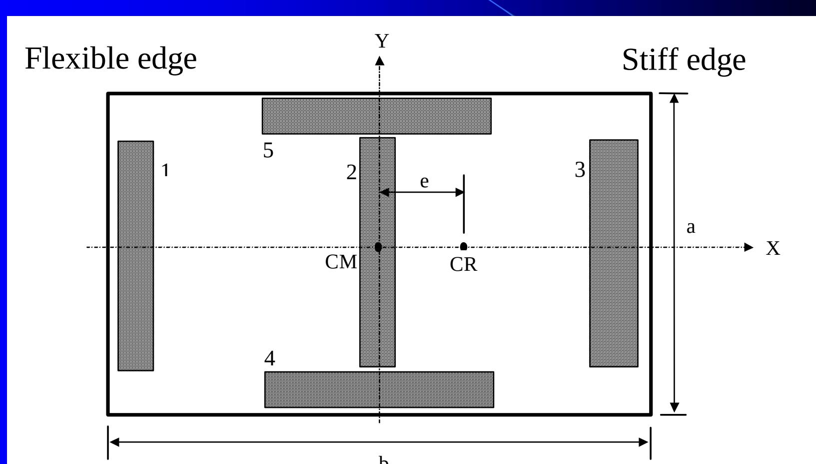


Fig. 22: Torsionally unbalanced building

Torsion Related Parameters

e = eccentricity between CR and CM

K_y = lateral stiffness in y direction

$K_{\theta R}$ = torsional stiffness about CR

$$\omega_y = \sqrt{\frac{K_y}{m}} = \text{uncoupled lateral frequency}$$

$$\omega_\theta = \sqrt{\frac{K_{\theta R}}{mr^2}} = \text{uncoupled torsional frequency}$$

$$\Omega_R = \frac{\omega_\theta}{\omega_y} = \text{frequency ratio}$$

Large Ω_R = torsionally stiff building

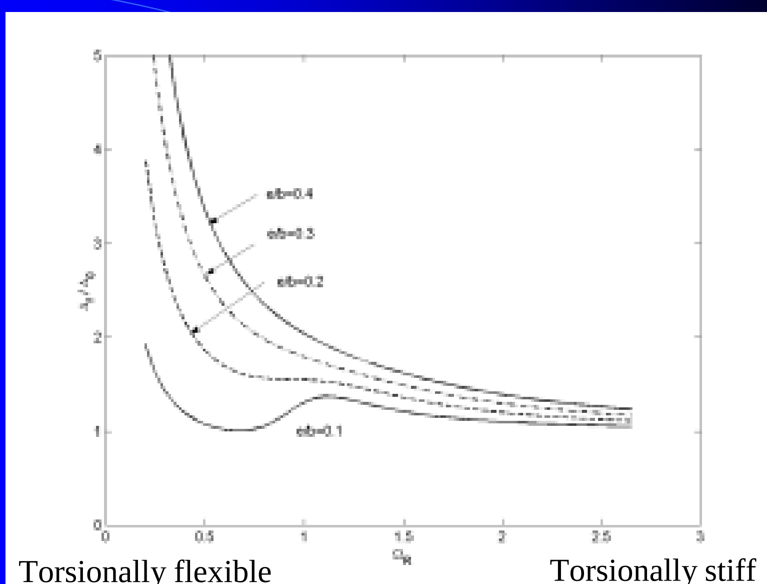


Fig. 23: Flexible edge displacement, hyperbolic spectrum, aspect ratio = 1

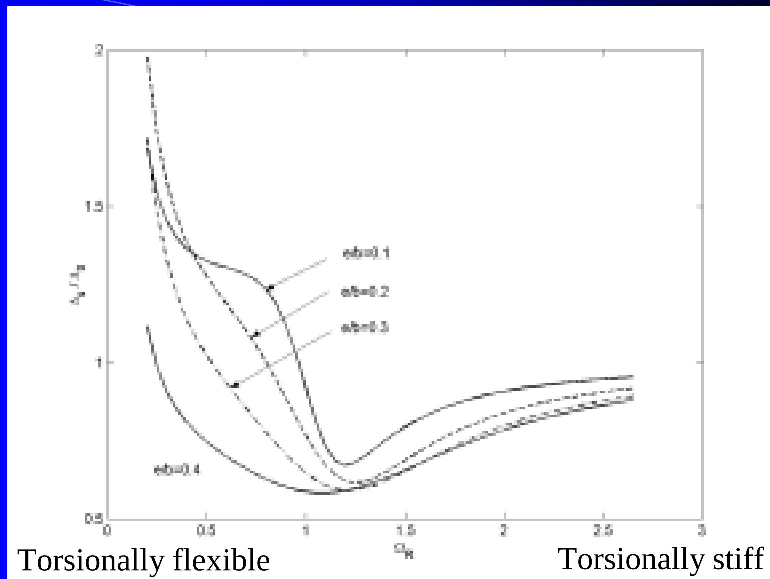


Fig. 24: Stiff edge displacement, hyperbolic spectrum, aspect ratio = 1

NBCC 2005 Provisions for Torsion

Use dynamic analysis for $B > 1.7$, where

B is the largest value of B_x, where

$B_x = \delta_{max} / \delta_{ave}$

δ_{max} = maximum displacement at extreme points of storey x, produced by lateral forces applied at an eccentricity of +/- $0.1b_{nx}$

δ_{ave} = average of the displacements at extreme points of storey x produced by the lateral forces

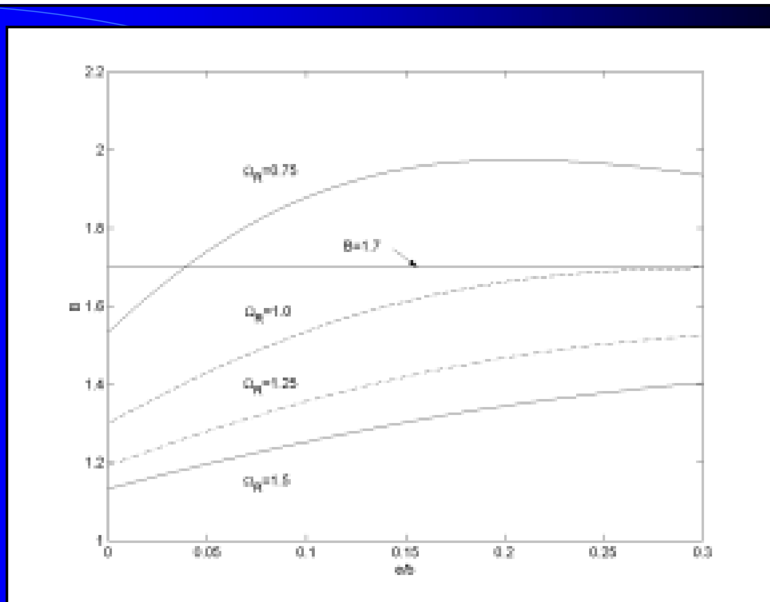


Fig. 27: Variation of torsional sensitivity parameter *B*

Deflections and Drift Limits

- Deflections calculated from an elastic analysis for the design forces shall be multiplied by R_dR_o/I to give the design deflections
- For any storey deflections should be limited to 0.01 h_s for post-disaster buildings, 0.02 h_s for schools, and 0.025 h_s for all other buildings, where h_s is the storey height

Dynamic Analysis

Dynamic analysis required for the following class of buildings

- Regular structures that have $h \geq 60$ m, or have $T_a \geq 2$ s, and are located in areas in which $IF_aS_a(0.2) \geq 0.35$
- Irregular buildings that have h≥20 m, or have $T_a \geq 0.5$ s, and are located in areas in which $IF_aS_a(0.2) \geq 0.35$
- All buildings that have rigid diaphragms and are torsionally sensitive, i.e. B ≥ 1.7

Scaling of Dynamic Analysis

- Obtain the dynamic elastic base shear V_e
- Obtain dynamic design base shear $V_d = V_e I / R_d R_o$
- If $V_d < 0.8 V_{static}$, take V_d to be no less than $0.8 V_{static}$
- For irregular structures requiring dynamic analysis V_d should be taken no less than V_{static}

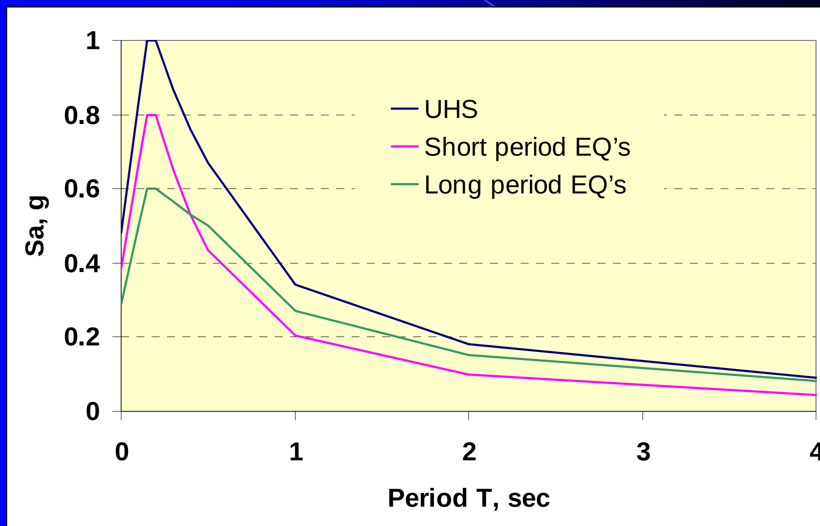
Dynamic Analysis Methods

- Modal response spectrum method – linear, expected to be most common dynamic method used
- Numerical integration linear time history method
- Numerical integration nonlinear time history method

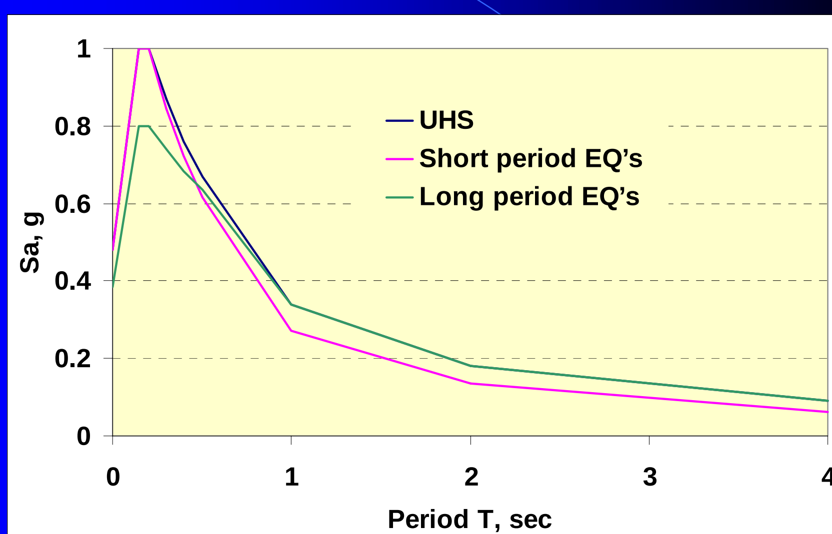
Response spectrum

- A response spectrum provides the maximum response of a SDOF system, for a given damping ratio and a range of periods, for a specific earthquake
- A design response spectrum is a smoothed spectrum used to calculate the expected seismic response of a structure
- A Uniform Hazard Spectrum (UHS) is a spectrum, for a given damping ratio, that has equal probability of occurring at all periods

Schematic of contribution to UHS of different type earthquakes



Two Spectrum Analysis Representation of UHS



Use of UHS in Modal Analysis

- Use of UHS in modal analysis will provide conservative results
- A number of calculations show the overestimation of response to be not more than 10%

Dynamic Analysis Methods

- Modal response spectrum method - linear
- Numerical integration linear time history method
- Numerical integration nonlinear time history method

Time history analysis requires ground motion records

Ground Motion Record Selection

- Ground motion records:
  - Should be of appropriate magnitude and distance
  - Some codes recommend at least 3 records with the maximum response being used, or the use of at least 7 records if the average response is used.
  - Should be scaled to be compatible with the design spectrum

Ground motion compatibility

Ground motion records should be compatible with the design spectrum

Two methods of making ground motion records compatible with a spectrum

- Scaling of records until the spectrum of the record is close to the design spectrum in the period range of interest – generally the 1^{st} and 2^{nd} mode periods
- Modifying the records so that the spectrum of the modified record matches the design spectrum

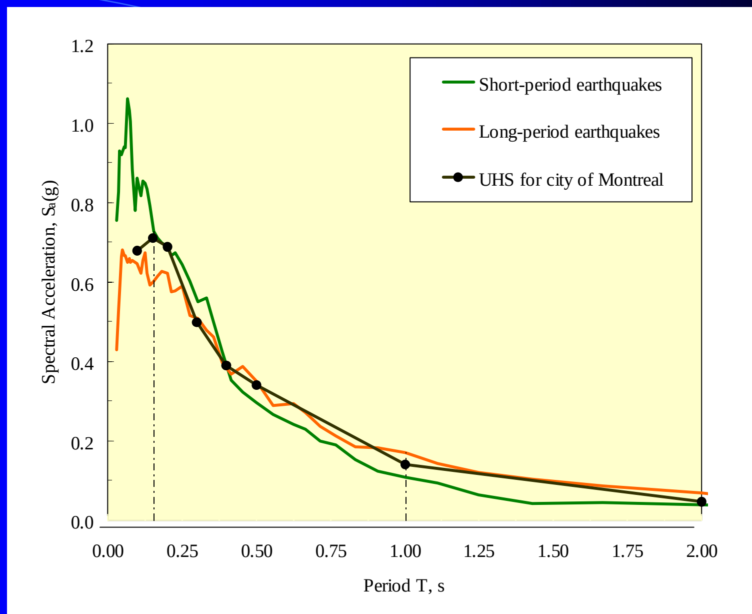


Fig. 11: Spectra for two sets of UHS-compatible records for Montreal

Ground motion compatibility

Ground motion records should be compatible with the design spectrum

Two methods of making ground motion records compatible with a spectrum

- Scaling of records until the spectrum of the record is close to the design spectrum in the period range of interest – generally the 1st and 2nd mode periods
- Modifying the records so that the spectrum of the modified record matches the design spectrum

- Records modified to fit the design spectrum
  - There are programs that will modify an acceleration record so that the response spectrum matches the target spectrum
  - There is controversy over the use of modified records but they have two great advantages:
    1. there is not as much scatter in the results, and
    2. not as much care must be taken in selecting the original records

The End