Seismic Design of Multistorey Concrete Structures

Lecture 4
Dynamic Analysis of Buildings

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NBCC 2005

- Objective of NBCC:
  - Building structures should be able to resist major earthquakes without collapse.
- Must design and detail structure to control the location and extent of damage.
- Damage limits effective force acting on structure.
  - But damage increases displacements!

NBCC 2005 Requirements:

4.1.8.7. Methods of Analysis

1) Analysis for design earthquake actions shall be carried out in accordance with the **Dynamic Analysis Procedure** as per **Article 4.1.8.12**. (see Appendix A), except that the Equivalent Static Force Procedure as per Article 4.1.8.11. may be used for structures that meet any of the following criteria:

a) for cases where $I_E F_a S_a(0.2)$ is less than 0.35,

b) regular structures that are less than 60 m in height and have a fundamental lateral period, T_a, less than 2 seconds in each of two orthogonal directions as defined in Article 4.1.8.8., or

c) structures with structural irregularity, Types 1, 2, 3, 4, 5, 6 or 8 as defined in Table 4.1.8.6. that are less than 20 m in height and have a fundamental lateral period, T_a, less than 0.5 seconds in each of two orthogonal directions as defined in Article 4.1.8.8.

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4.1.8.12. Dynamic Analysis Procedures

1) The Dynamic Analysis Procedure shall be in accordance with one of the following methods:
   a) Linear Dynamic Analysis by either the **Modal Response Spectrum Method** or the **Numerical Integration Linear Time History Method** using a structural model that complies with the requirements of **Sentence 4.1.8.3.(8)** (see Appendix A); or
   b) **Nonlinear Dynamic Analysis Method**, in which case a special study shall be performed (see Appendix A).
2) The **spectral acceleration values** used in the Modal Response Spectrum Method shall be the design spectral acceleration values S(T) defined in Sentence 4.1.8.4.(6)
3) The **ground motion histories** used in the Numerical Integration Linear Time History Method **shall be compatible with a response spectrum** constructed from the design spectral acceleration values S(T) defined in Sentence 4.1.8.4.(6) (see Appendix A).
4) The effects of **accidental torsional moments** acting concurrently with and due to the lateral earthquake forces shall be accounted for by the following methods :
   a) the static effects of torsional moments due to at each level x, where F_x is determined from Sentence 4.1.8.11.(6) or from the dynamic analysis, shall be combined with the effects determined by dynamic analysis (see Appendix A), or
   b) if B as defined in Sentence 4.1.8.11.(9) is less than 1.7, it is permitted to use a 3-dimensional dynamic analysis with the centres of mass shifted by a distance - 0.05 and + 0.05.

4.1.8.12. Dynamic Analysis Procedures (continued)

5) The **elastic base shear, V_e** obtained from a Linear Dynamic Analysis shall be multiplied by the Importance factor I_E as defined in Article 4.1.8.5. and shall be divided by R_dR_o as defined in Article 4.1.8.9. to obtain the **design base shear V_d.**
6) Except as required in Sentence (7), if the base shear **V_d** obtained in Sentence (5) is **less than 80% of the lateral earthquake design force, V**, of Article 4.1.8.11., **V_d** shall be taken as **0.8V.**
7) For **irregular structures** requiring dynamic analysis in accordance with Article 4.1.8.7., V_d shall be taken as **the larger of the V_d** determined in Sentence (5) and 100% of V.
8) Except as required in Sentence (9), the values of elastic *storey* shears, *storey* forces, member forces, and deflections obtained from the Linear Dynamic Analysis shall be multiplied by **V_d/V_e** to determine their design values, where V_d is the base shear.
9) For the purpose of calculating deflections it is permitted to use V determined from T_a defined in Clause 4.1.8.11.(3)(e) to obtain V_d in Sentences (6) and (7).

4.1.8.7. Methods of Analysis

Equivalent Static Force Procedure used
- areas of low seismicity, or
- regular, H<60m and T<2s
- not torsionally irregular, H<20m,T<0.5s

Dynamic Analysis
- default method
- base shear tied back to statically determined

4.1.8.3.8 Structural Modelling

Structural modelling shall be representative of the magnitude and spatial distribution of the mass of the *building* and stiffness of all elements of the SFRS, which includes stiff elements that are not separated in accordance with Sentence 4.1.8.3.(6), and shall account for:

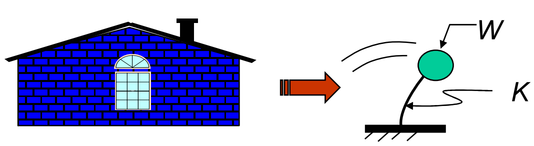
a) the effect of the finite size of members and joints.

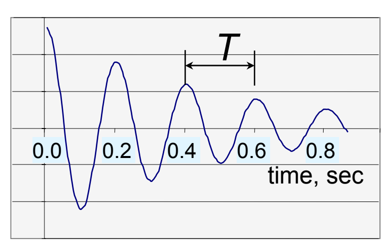
b) sway effects arising from the interaction of gravity loads with the displaced configuration of the structure, and

c) the effect of cracked sections in reinforced concrete and reinforced masonry elements.

d) other effects which influence the *buildings* lateral stiffness.

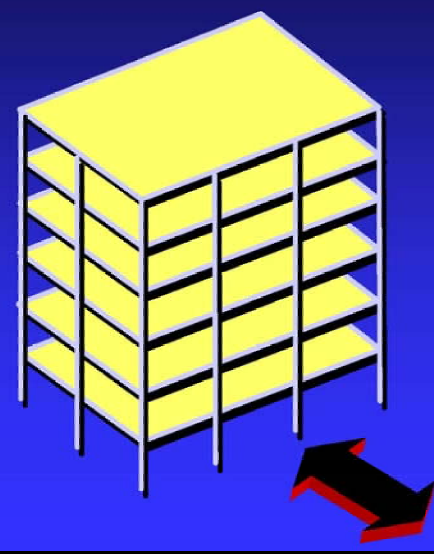
Linear Response of Structures





Vibration Period $T = 2\pi\sqrt{\frac{W/g}{K}}$

MDOF Systems Response to Base Excitation



Structural Analysis Procedures for Earthquake Resistant Design

Category	Analysis Procedure	Force–Deformation Relationship	Displacements	Earthquake Load	Analysis Method
Equilibrium	Plastic Analysis Procedure	Rigid-plastic	Small	Equivalent lateral load	Equilibrium analysis
Linear	Linear Static Procedure	Linear	Small	Equivalent lateral load	Linear static analysis
	Linear Dynamic Procedure I	Linear	Small	Response spectrum	Response spectrum analysis
	Linear Dynamic Procedure II	Linear	Small	Ground motion history	Linear response history analysis
Nonlinear	Nonlinear Static Procedure	Nonlinear	Small or large	Equivalent lateral load	Nonlinear static analysis
	Nonlinear Dynamic Procedure	Nonlinear	Small or large	Ground motion history	Nonlinear response history analysis

Dynamic Equilibrium Equations – discrete systems

$$M\,a + Cv + Ku = F(t)$$

a = Node accelerations
v = Node velocities
u = Node displacements
M = Mass matrix
C = Damping matrix
K = Stiffness matrix
F(t) = Time-dependent forces

Problem to be solved

$$M\,a + C\,v + K\,u = \sum f_i\, g(t)_i$$

For 3D Earthquake Loading:

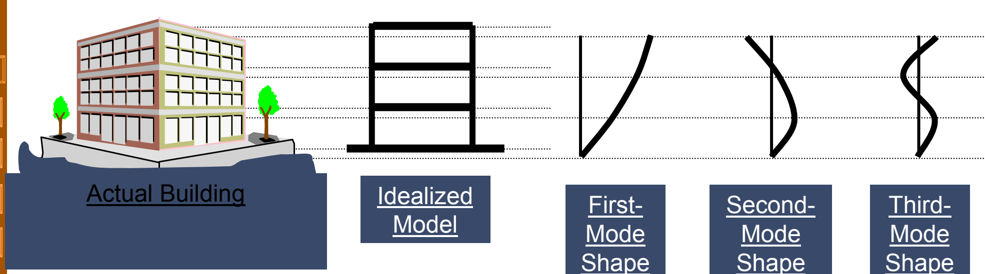
$$= -M_x\,a_x - M_y\,a_y - M_z\,a_z$$

Purpose of Analysis

- Predict, for a design earthquake, the force and deformation demands on the various components that compose the structure
- Permit evaluation of the acceptability of structural behavior (performance) through a series of $\frac{\text{Demand}}{\text{Capacity}}$ checks

Multi-Story Structures

- Multi-story buildings can be idealized and analyzed as multi-degree-of-freedom systems.
- Linear response can be viewed in terms of individual modal responses.

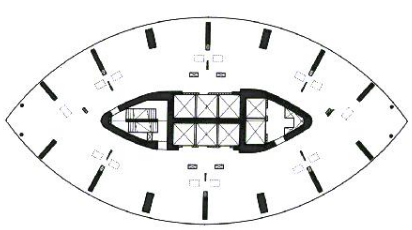


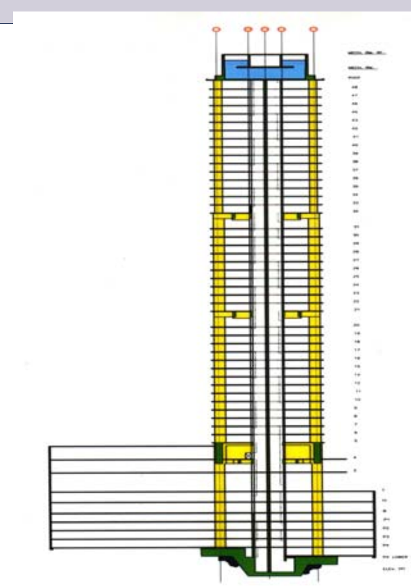
Example of a Building Model

- 48 stories (137 m)
- 6 underground parking levels
- Oval shaped floor plan (48.8m by 23.4m)
- Typical floor height of 2.615 m
- 7:1 height-to-width ratio

Structural Details

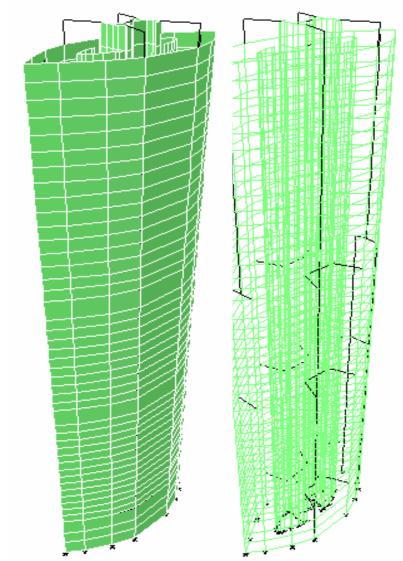
- Central reinforced concrete core
  - Walls are up to 900 mm thick
- Outrigger beams
  - Level 5: 6.4 m deep
  - Level 21 & 31: 2.1 m deep
- Tuned liquid column dampers
  - Two water tanks (183 m^3 each)





FEM of the Building

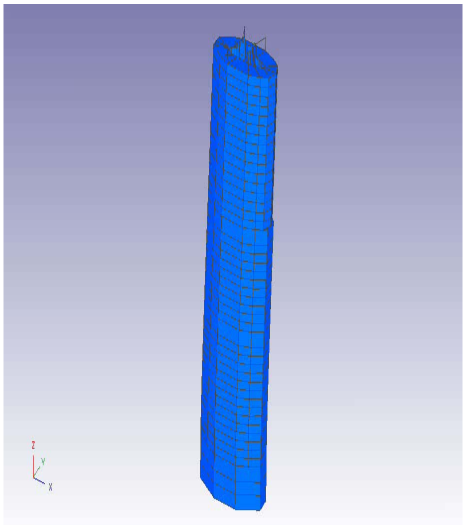
- 616 3D beam-column elements
- 2,916 4-node plate elements
- 66 3-node plate elements
- 2,862 nodes
- 4 material properties
- 17,172 DOFs



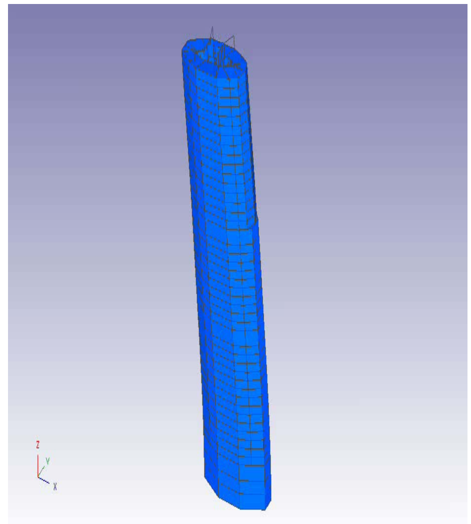
Calibrated FEM with Ambient Vibration Tests

Mode No.	Test Period (s)	Analytical	
		Period (s)	MAC (%)
1	3.57	3.57	99
2	2.07	2.07	87
3	1.46	1.46	99
4	0.81	0.81	99
5	0.52	0.52	86
6	0.49	0.49	87

1st mode



2nd mode



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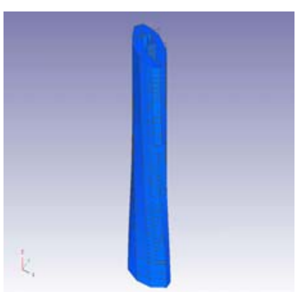
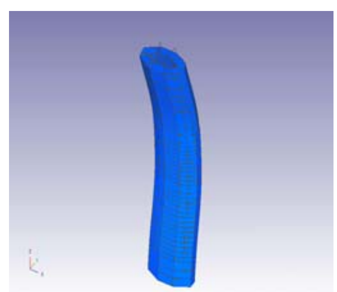
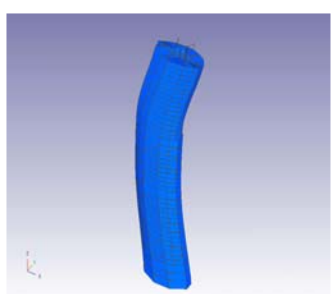


3rd mode



4th mode



5th mode

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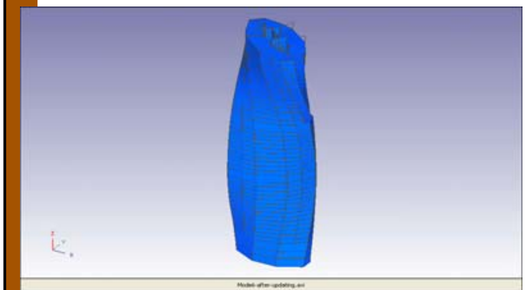
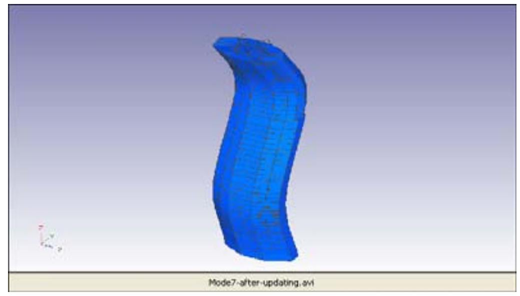
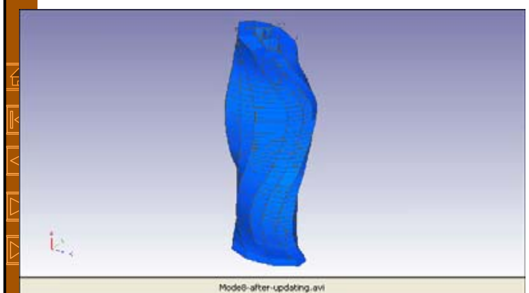
6th mode

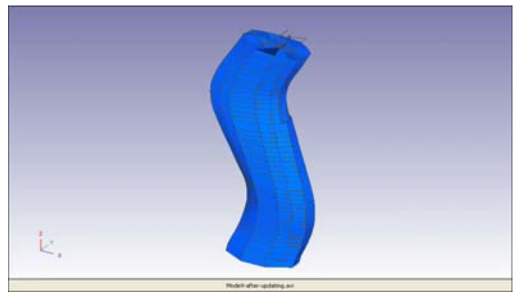


7th mode





8th mode



9th mode

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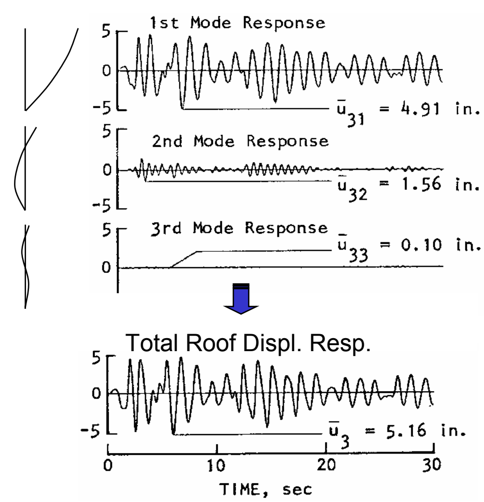
Multi-Story Structures

- Individual modal responses can be analyzed separately.
- Total response is a combination of individual modes.
- For typical low-rise and moderate-rise construction, first-mode dominates displacement response.

Reference: A. K. Chopra, Dynamics of Structures: A Primer, Earthquake Engineering Research Institute



What is the Response Spectrum Method, RSM?

The Response Spectrum is an estimation of maximum responses (acceleration, velocity and displacement) of a family of SDOF systems subjected to a prescribed ground motion.

The RSM utilizes the response spectra to give the structural designer a set of possible forces and deformations a real structure would experience under earthquake loads.

-For SDF systems, RSM gives quick and accurate peak response without the need for a time-history analysis.

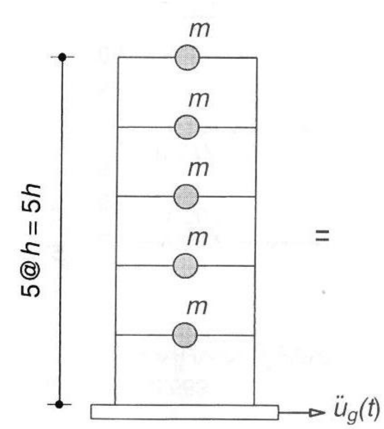
-For MDF systems, a true structural system, RSM gives a *reasonably* accurate peak response, again without the need for a full time-history analysis.

RSM – a sample calculations of a 5-storey structure.

Solution steps:

- Determine mass matrix, m
- Determine stiffness matrix, k
- Find the natural frequencies ω_n (or periods, $T_n=2\pi/\omega_n$) and mode shapes φ_n of the system
- Compute peak response for the n^{th} mode, and repeat for all modes.
- Combine individual modal responses for quantities of interest (displacements, shears, moments, stresses, etc).

RSM – a sample calculations of a 5-storey shear-beam type building.



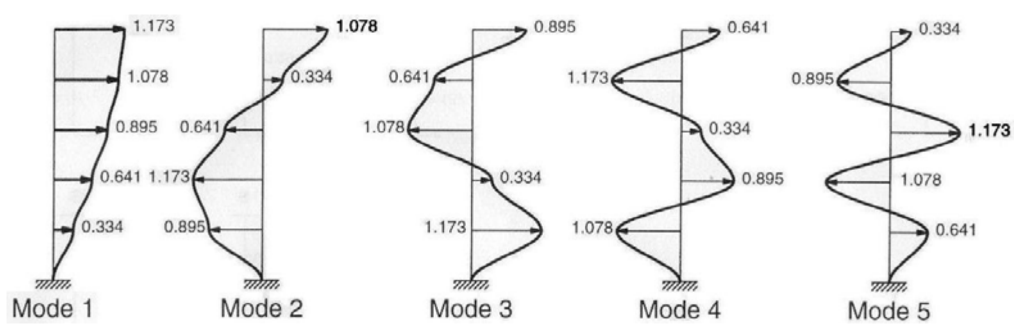
Typical storey height is h=12 ft.

$$\mathbf{m} = \begin{vmatrix} 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 1 \end{vmatrix} \times \mathbf{100\ kips/g}$$

$$\mathbf{k} = \begin{vmatrix} 2 & -1 & 0 & 0 & 0 \\ -1 & 2 & -1 & 0 & 0 \\ 0 & -1 & 2 & -1 & 0 \\ 0 & 0 & -1 & 2 & -1 \\ 0 & 0 & 0 & -1 & 1 \end{vmatrix} \times \mathbf{31.54kip/in.}$$

Natural vibration modes of a 5-storey shear building.

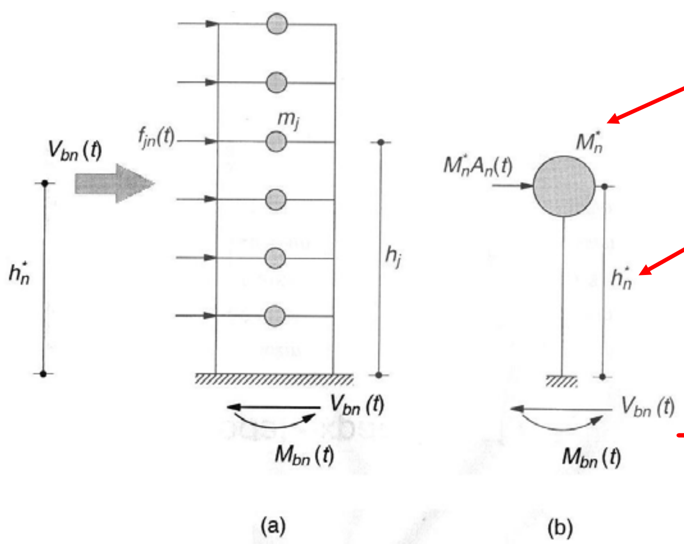
Mode shapes $\boldsymbol{\varphi}_n$ of the system:



T_1 = 2.01s T_2 = 0.68s T_3 = 0.42s T_4 = 0.34s T_5 = 0.29s

Assume a damping ratio of 5% for all modes

Effective modal masses and modal heights



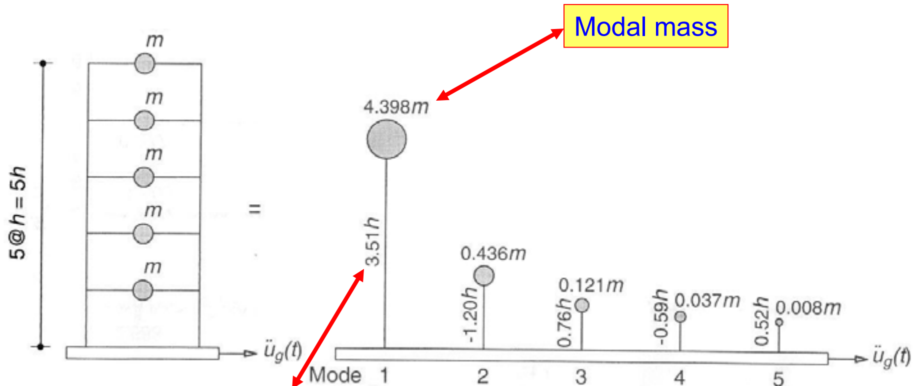
Modal mass

Modal height

We use this information to compute modal base shears and modal base overturning moments

Effective modal masses and modal heights

Modal mass



Modal height

We use this information to compute modal base shears and modal base overturning moments

. . . Solution steps (cont'd)

a. Corresponding to each natural period T_n and damping ratio ζ_n, read SD_n and SA_n from design spectrum or response spectrum

b. Compute floor displacements and storey drifts by

$$u_{jn} = \Gamma_n \varphi_n D_n$$

where $\Gamma_n = \varphi_n^T \mathbf{m} / (\varphi_n^T \mathbf{m} \varphi_n)$ is the modal participation factor

c. Compute the equivalent static force by

$$f_{jn} = \Gamma_n m_j \varphi_{jn} SA_n$$

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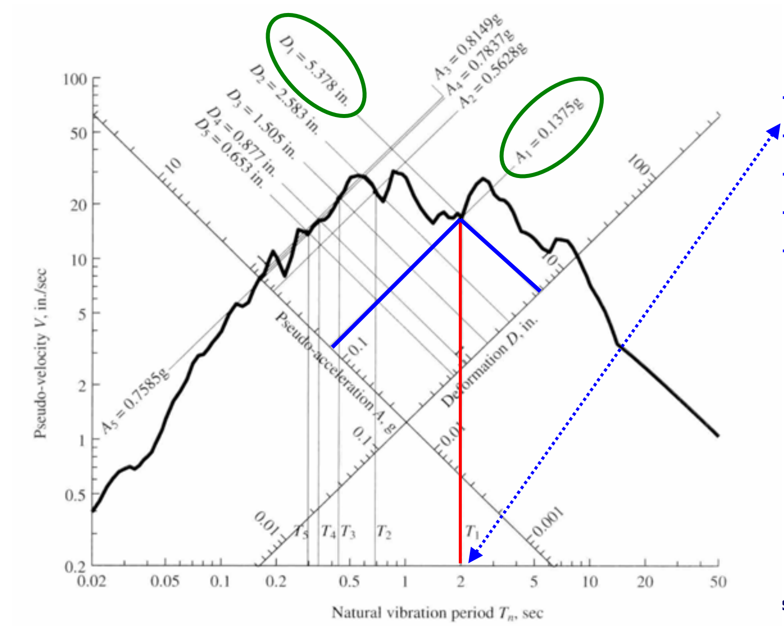
. . . solution steps (cont'd.)

d. Compute the story forces – shears and overturning moment – and element forces by static analysis of the structure subjected to lateral forces $\mathbf{f}_n$

- Determine the peak value r of any response quantity by combining the peak modal values r_n according to the SRSS or CQC modal combination rule.

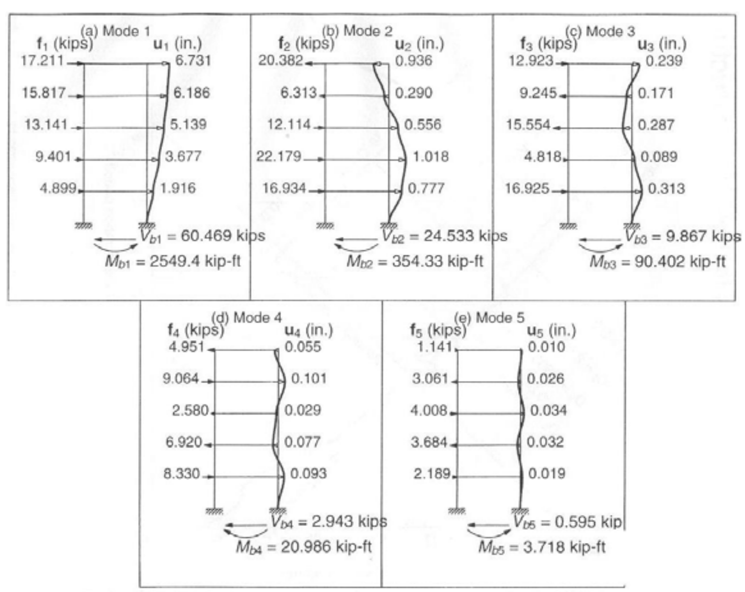
Obtain values from Response Spectrum:



T_1 = 2.01s
T_2 = 0.68s
T_3 = 0.42s
T_4 = 0.34s
T_5 = 0.29s

s

. . . Results:



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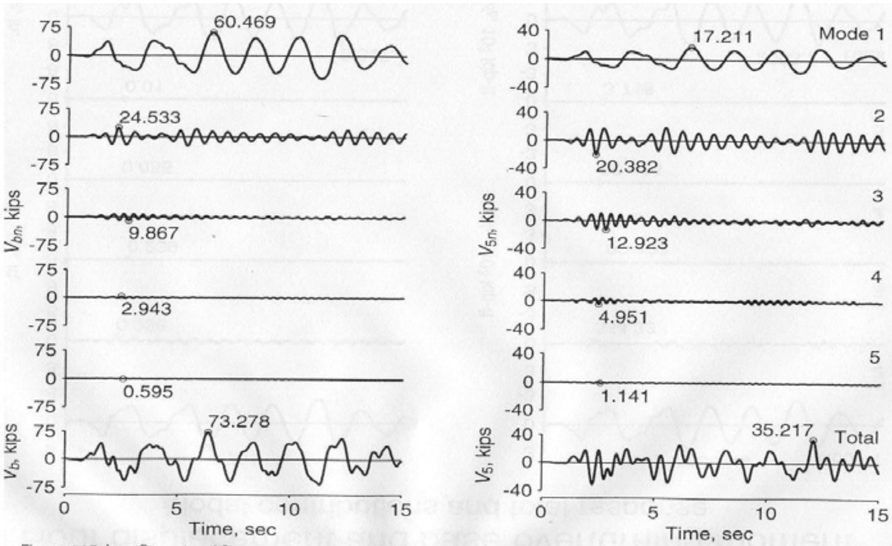
Comparison with time-history analysis results:



Seismic Design of Multistorey Concrete Structures

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. . . mathematically speaking.

Maximum modal displacement $X_{n.Max} := \frac{L_n}{M_n} S_d(T_n, \xi_n) \cdot \phi_n$

Modal forces $F_{i.n.Max} := \frac{L_n}{M_n} S_a(T_n, \xi_n) \cdot \phi_n$

Modal base shears $V_{b.n.Max} := \frac{L_n^{\,2}}{M_n} S_a(T_n, \xi_n)\ \phi_n$

L_n, M_n, and φ_n are system parameters determined from the Modal Analysis Method.

$\mathbf{L_n} := \phi_n^{\,T} \mathbf{m} \cdot 1$ $\quad$ $\mathbf{M_n} := \phi_n^{\,T} \mathbf{m} \cdot \phi_n$

$S_a(T_n, \xi_n)$ System response from spectrum graph.

A number of methods to estimate of the total system response from these idealized SDF systems can summarized as follow;

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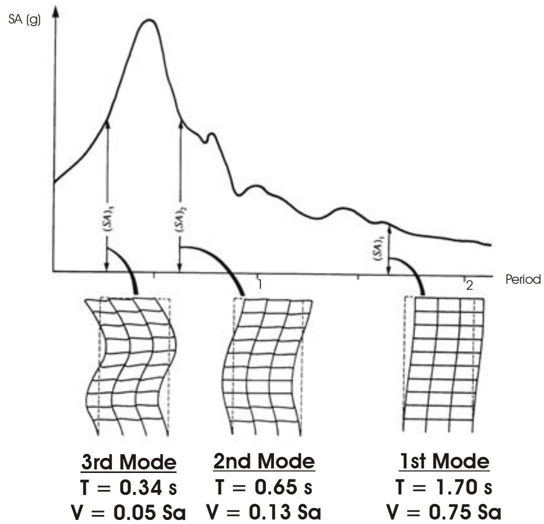
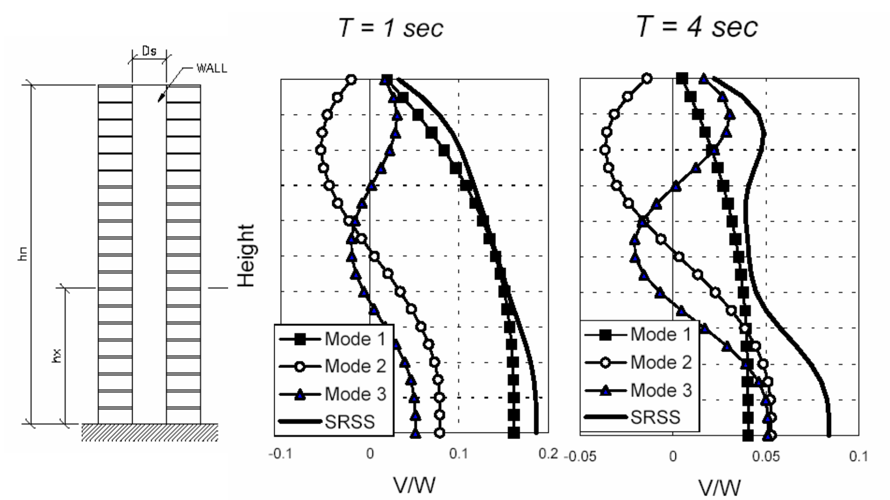
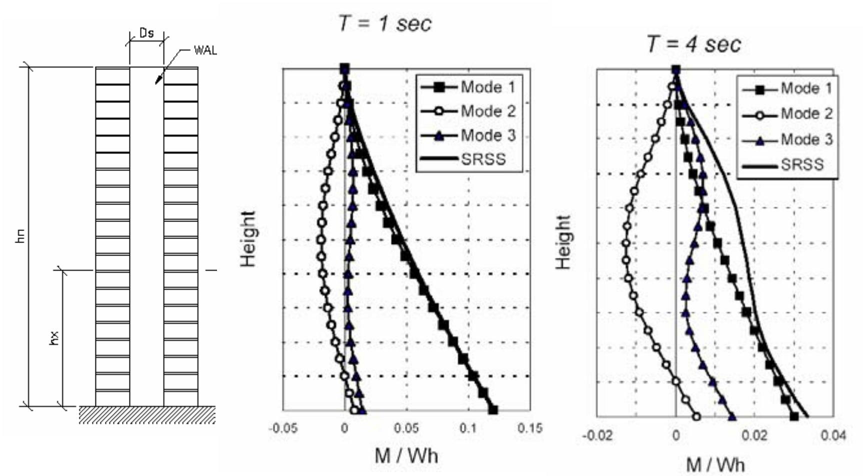
Modal Responses of a ten-storey frame building



Modal contributions to shear forces in a building



Modal contributions to overturning moments in a building



Modal Combinations

- Modal maxima do not occur at the same time, in general.
- Any combination of modal maxima may lead to results that may be either conservative or unconservative.
- Accuracy of results depends on what modal combination technique is being used and on the dynamic properties of the system being analysed.
- Three of the most commonly used modal combination methods are:

Modal Combinations….

a) Sum of the absolute values:

- leads to very conservative results
- assumes that maximum modal values occur at the same time
- response of any given degree of freedom of the system is estimated as

$$x_{i_{max}} \approx \sum_{n=1}^{L} \left| x_{i,n_{max}} \right|$$

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Modal Combinations…..

b) Square root of the sum of the squares (SRSS or RMS):

- Assumes that the individual modal maxima are statistically independent.
- SRSS method generally leads to values that are closer to the "exact" ones than those obtained using the sum of the absolute values.
- Results can be conservative or unconservative.
- Results from an SRSS analysis can be significantly unconservative if modal periods are closely spaced.
- The response is estimated as:

$$x_{i_{max}} \approx \sqrt{\sum_{n=1}^{L} (x_{i,n_{max}})^2}$$

Modal Combinations….

c) Complete quadratic combination (CQC):

- The method is based on random vibration theory
- It has been incorporated in several commercial analysis programs
- A double summation is used to estimate maximum responses,

$$x_{i_{max}} \approx \sqrt{\sum_{n=1}^{L}\sum_{m=1}^{L} x_{i,n_{max}} \rho_{n,m} x_{i,m_{max}}}$$

In which, ρ is a cross-modal coefficient (always positive), which for constant damping is evaluated by

$$\rho_{n,m} = \frac{8z^2(1+r)r^{1.5}}{(1-r^2)+4z^2 r(1+r)^2}$$

where $r = \rho_n / \rho_m$ and must be equal to or less than 1.0.

Similar equations can be applied for the computation of member forces, interstorey deformations, base shears and overturning moments.

Modal combination methods, strength and shortfalls

ABSSUM: Summation of absolute values of individual modal responses

$V_b \leq \Sigma |V_{bn}|$ = 98.41 kips → grossly over-estimated

SRSS: Square root of sum of squares

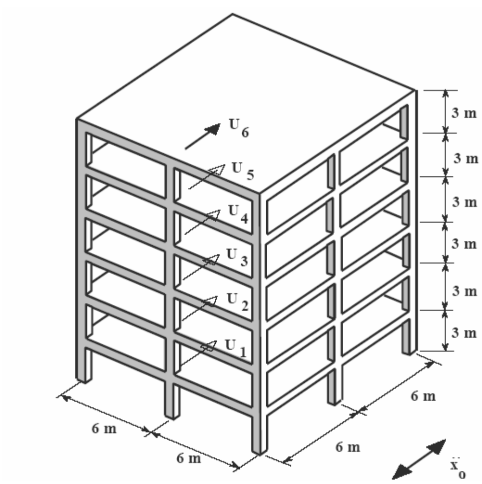
$V_b = (\Sigma V_{bn}^2)^{1/2}$ = 66.07 kips
→good estimate if frequencies are spread out

CQC: $V_b = (\Sigma\Sigma V_{bi}\, \rho_{in} V_{bn})^{1/2}$ = 66.51 kips
→ good estimate if frequencies are closely spaced

Correct base shear is 73.278 kips (from time-history analysis)

Example:

(See notes from Garcia and Sozen for details of the building presented here)



We want to study the response of the building to the N-S component of the recorded accelerations at El Centro, California, in May 18 of 1940.

We are interested in the response in the direction shown in the figure.

Damping for the system is assumed to be ξ **= 5%**

All girders of the structure have width **b = 0.40 m** and depth **h = 0.50 m**.

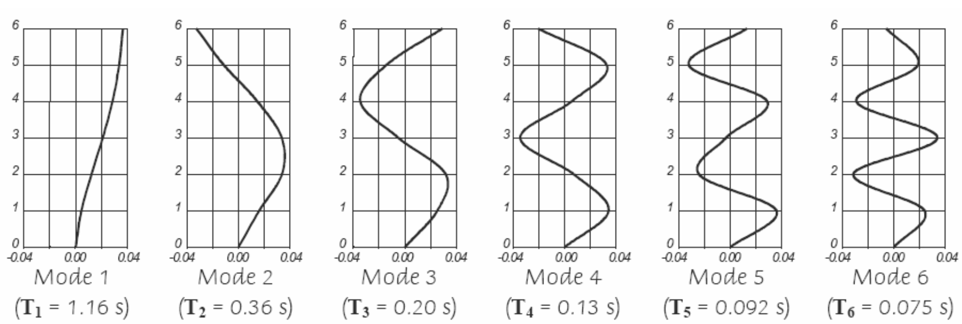
All columns have square section with a cross section dimension **h = 0.50 m**.

The material of the structure has a modulus of elasticity **E = 25 GPa**.

The self weight of structure plus additional dead load is **780 kg/m2** and the industrial machinery, which is firmly connected to the building slabs, increases the mass per unit area by **1000 kg/m2**, for a total mass per unit area of **1780 kg/m2**.

Example – Modal Properties of building

Mode	ω^2 (rad/s)²	ω (rad/s)	f (Hertz)	T (s)
1	29.108	5.39	0.859	1.16
2	301.81	17.4	2.76	0.36
3	973.78	31.2	4.97	0.20
4	2494.3	49.9	7.95	0.13
5	4686.5	68.5	10.9	0.092
6	7113.8	84.3	13.4	0.075



Comparison of Results for given example

Table 9 - Example 7 - Comparison of the results from Examples 5, 6, and 7

Parameter	Example 5 Step-by-step Analysis	Example 6 Modal spectral Absolute value	Example 7 Modal spectral SRSS
Roof lateral displacement	0.149 m	0.160 m	0.149 m
Base shear	4 360 kN	6 170 kN	4 330 kN
Overturning moment	54 400 kN · m	56 700 kN · m	53 900 kN · m

Design Example

CONCRETE EXAMPLE:

HIGH – RISE CONCRETE TOWER

The following example was kindly provided by Mr. Jim Mutrie. P.Eng., a partner of Jones-Kwong-Kishi in Vancouver, BC

DESIGN PROBLEM – ASSUMED TOWER

25 Floors

Floor Area 650 m^2

Ceiling Height 2600 mm

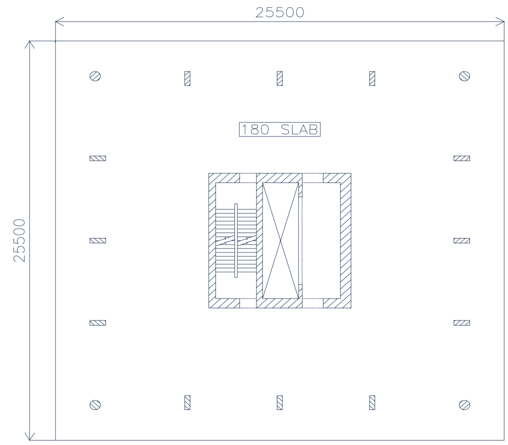
FTFH 2780 mm

Door 2180 mm

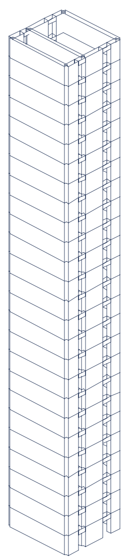
Header Depths 600 mm

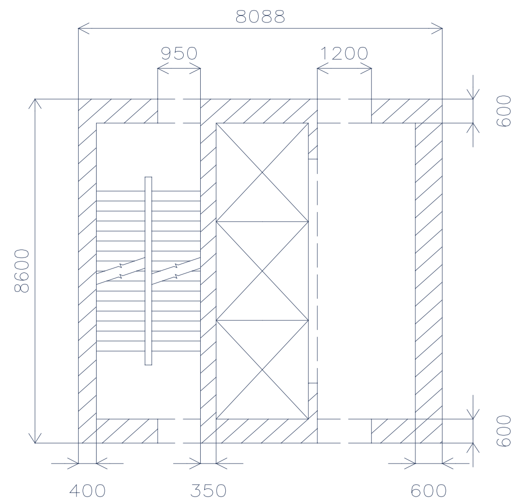
Vancouver

Site Class “C”



COUPLED SHEAR WALL CORE MODEL





CLADDING CONCRETE

N21.12.1.1

The building envelope failures experienced in the West Coast have encouraged the use of additional concrete elements on buildings as part of the envelope system that are not part of either the gravity or the seismic force resisting systems. These elements have the potential to compromise the gravity and/or the seismic force resisting systems when the building is deformed to the design displacement. This clause provides steps that need to be taken so a solution to one problem does not jeopardize the buildings seismic safety.

CLADDING CONCRETE

21.12.1.2

Elements not required to resist either gravity or lateral loading shall be considered non-structural elements. These elements need not be detailed to the requirements of this clause provided:

a) the effects of their stiffness and strength on forces and deformations in all structural elements at the Design Displacement are calculated, and
b) the factored capacity of all structural elements includes for these forces and deformations, and
c) the non-structural elements are anchored to the building in accordance with section 4.1.8.17 of the *National Building Code of Canada.*

1. DYNAMIC PROPERTIES

NBCC 2005 Empirical Periods

$$T = 0.05 \times (h_n)^{3/4} \text{ Clause } 4.1.8.11.3)c)$$

$$T = 0.05 \times (69.5)^{3/4} = 1.20 \text{ sec.}$$

$$T_{max} = 2.0 \times T \quad \text{Clause } 4.1.8.11.3)d)ii$$

$$T_{max} = 2.0 \times 1.20 = 2.41 \text{ sec}$$

$$V_{min} \quad \text{Clause } 4.1.8.11.2 \text{ uses } S(2.0)$$

2. NBCC 2005 ANALYSIS METHOD

NBCC 4.1.8.7.1

a) $I_E F_a S_a(0.2) < 0.35$
   $I_E = 1.0$, assumed ,
   $F_a = 1.0$ from Table 4.1.8.4.B and
   $S_a(0.2) = 0.94$ from Appendix C for Vancouver.
   $I_E F_a S_a(0.2) = 0.94 > 0.35$

b) Regular building, height < 60m and period < 2.0 sec.

c) Irregular building, height < 20m and period < 0.5 sec.

This building does not comply with any of the cases that allow Equivalent Static Procedures.

3. BUILDING MASS

Floor

Slab 0.18 * 23.5 =	4.23 kN/m^2
Partition Allowance =	0.60 kN/m^2
Floor Finish =	0.50 kN/m^2
Columns	
$(0.3 * 0.9 * 12 + \pi * 0.3^2 * 4) * 2.6 * 23.5/(25.5 * 25.5) =$	0.41 kN/m^2
Sum	5.74 kN/m^2

Curtain Wall

0.7 kPa * 2.6 m = 1.82 kN/m

$\text{Mass} = 5.74 * 25.5^2 + 1.82 * 4 * 25.5 = 3732 + 186 =$ **3918 kN**

3. BUILDING MASS

$\text{MMI} = 3732 * 25.5^2 /6 + 186.6 * 25.5^2 /3 = 404{,}456 + 40{,}316 =$ 444,772 kNm^2

Core

Area = 17.115 m2 * 2.6 * 23.5 =	1046 kN/Floor
Headers = (0.95 + 1.2) * 2 * 0.6 * 0.42 * 23.5 =	25 kN/Floor
MMI = 21,401 kNm^2 (program calculation)	

Total Mass/Floor = 3918 + 1046 + 25 = 4989 kN/Floor

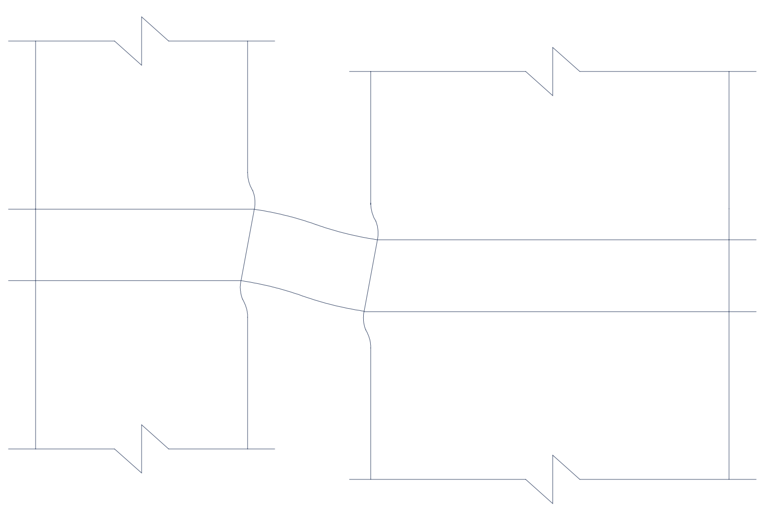
Total MMI/Floor = 444,772 + 21,401 = 466,173 kNm^2

4. STIFFNESS ASSUMPTIONS FOR PROGRAM

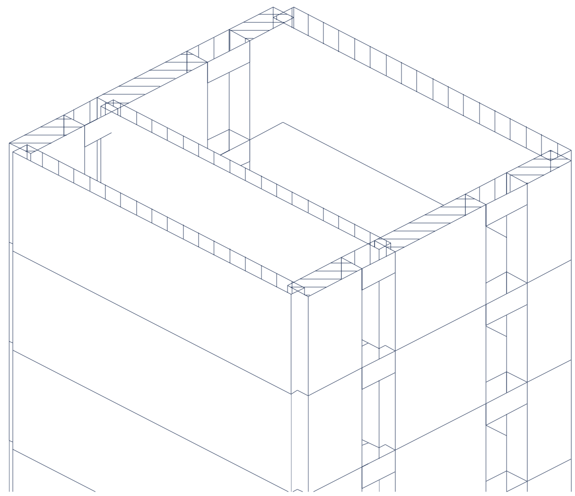
Element	CPCA	A23.3-04	
		Area	Inertia
Coupling Beam (Diagonally Reinforced)	0.4	$0.45\ A_g$	$0.25\ I_g$
Coupling Beam (Conventionally Reinforced)	0.2	$0.15\ A_g$	$0.4\ I_g$
Walls	0.7	$\alpha_w\ A_g$	$\alpha_w\ I_g$

$$\alpha_w = 0.6 + \frac{P_s}{f'_c A_g} \leq 1.0 = 0.6 + \frac{55{,}711}{35 \times 17.115 \times 1000} = 0.693$$

COUPLING BEAM – EFFECTIVE LENGTH



COUPLED SHEAR WALL CORE MODEL



CODE STATIC BASE SHEARS - VANCOUVER

NBCC 1995

$$V_e = 0.2 \times \frac{1.5}{\sqrt{2.219}} \times 1.0 \times 1.0 \times 124,159 = 25,005 \text{ kN}$$

$$V = \frac{25,005}{3.5} \times 0.6 = 4286 \text{ kN Uncoupled Direction}$$

$$V_e = 0.2 \times \frac{1.5}{\sqrt{2.548}} \times 1.0 \times 1.0 \times 124,159 = 23,335 \text{ kN}$$

$$V = \frac{23,335}{4.0} \times 0.6 = 3500 \text{ kN Coupled Direction}$$

NBCC 2005

$$V = S(T_a) M_v I_E W / (R_d R_o)$$

$$S(T_a) = S(2.0) = 0.17 \ (4.1.8.11.2 - \text{Minimum } V)(S(2.0) \text{Appendix } C)$$

M_v from Table 4.1.8.11

$$V = 0.17 \times 1.2 \times 1.0 \times 124,159 / (3.5 \times 1.6) = 4523 \text{ kN Uncoupled}$$

$$V = 0.17 \times 1.0 \times 1.0 \times 124,159 / (4.0 \times 1.7) = 3104 \text{ kN Coupled}$$

CODE STATIC BASE SHEARS - VICTORIA

NBCC 1995

$$V_e = 0.3 \times \frac{1.5}{\sqrt{2.219}} \times 1.0 \times 1.0 \times 124,159 = 37,507 \text{ kN}$$

$$V = \frac{37,507}{3.5} \times 0.6 = 6,430 \text{ kN Uncoupled Direction}$$

$$V_e = 0.3 \times \frac{1.5}{\sqrt{2.548}} \times 1.0 \times 1.0 \times 124,159 = 35,002 \text{ kN}$$

$$V = \frac{35,002}{4.0} \times 0.6 = 5,250 \text{ kN Coupled Direction}$$

NBCC 2005

$$V = S(T_a) M_v I_E W / (R_d R_o)$$

$$S(T_a) = S(2.0) = 0.18 \ (4.1.8.11.2 - \text{Minimum } V)(S(2.0) \text{Appendix } C)$$

M_v from Table 4.1.8.11

$$V = 0.18 \times 1.2 \times 1.0 \times 124,159 / (3.5 \times 1.6) = 4,789 \text{ kN Uncoupled}$$

$$V = 0.18 \times 1.0 \times 1.0 \times 124,159 / (4.0 \times 1.7) = 3,287 \text{ kN Coupled}$$

COMPARISON OF THE TWO CODES

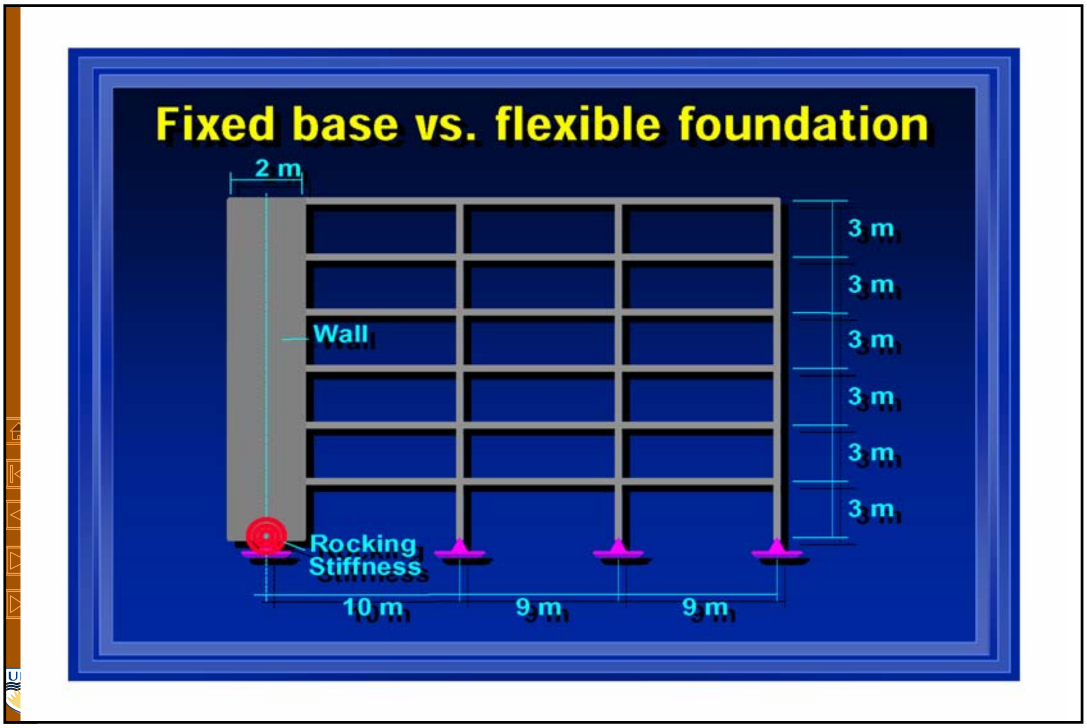
Vancouver	Shear		Moment	
Uncoupled	4265/4286	99.5%	122,417/118,510	103.3%
Coupled	2715/3500	77.6%	88,748/109,002	81.4%

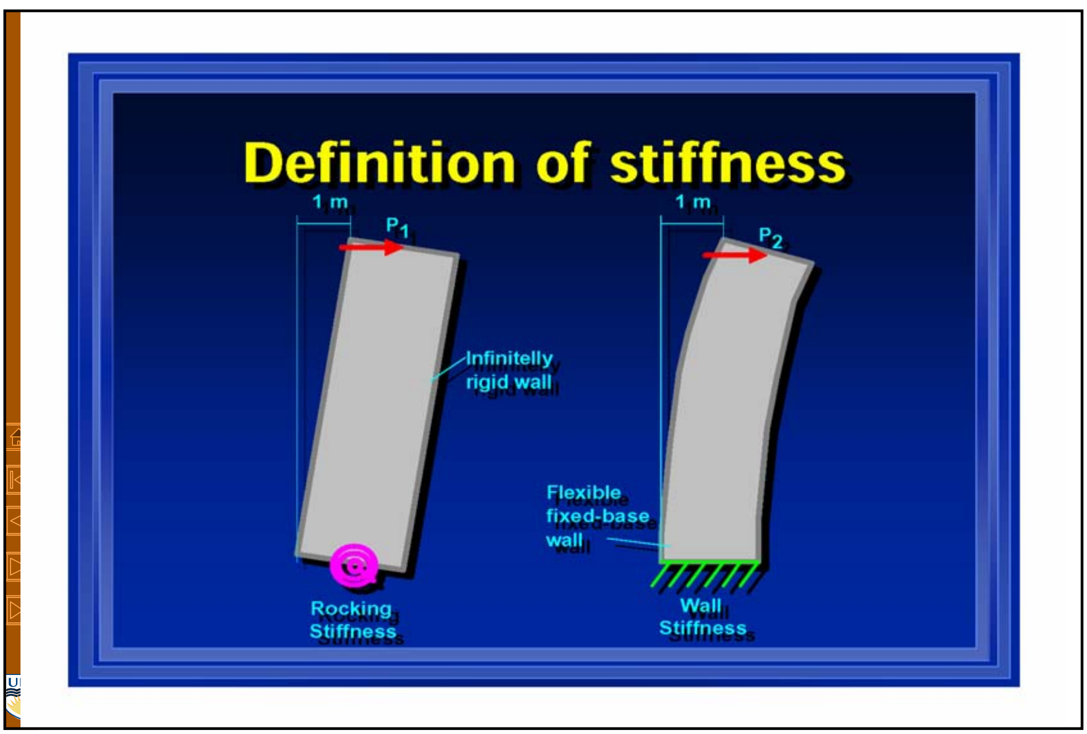
Victoria	Shear		Moment	
Uncoupled	5262/6430	81.8%	137,117/180,386	76.0%
Coupled	3343/5250	63.7%	98,239/163,714	60.0%

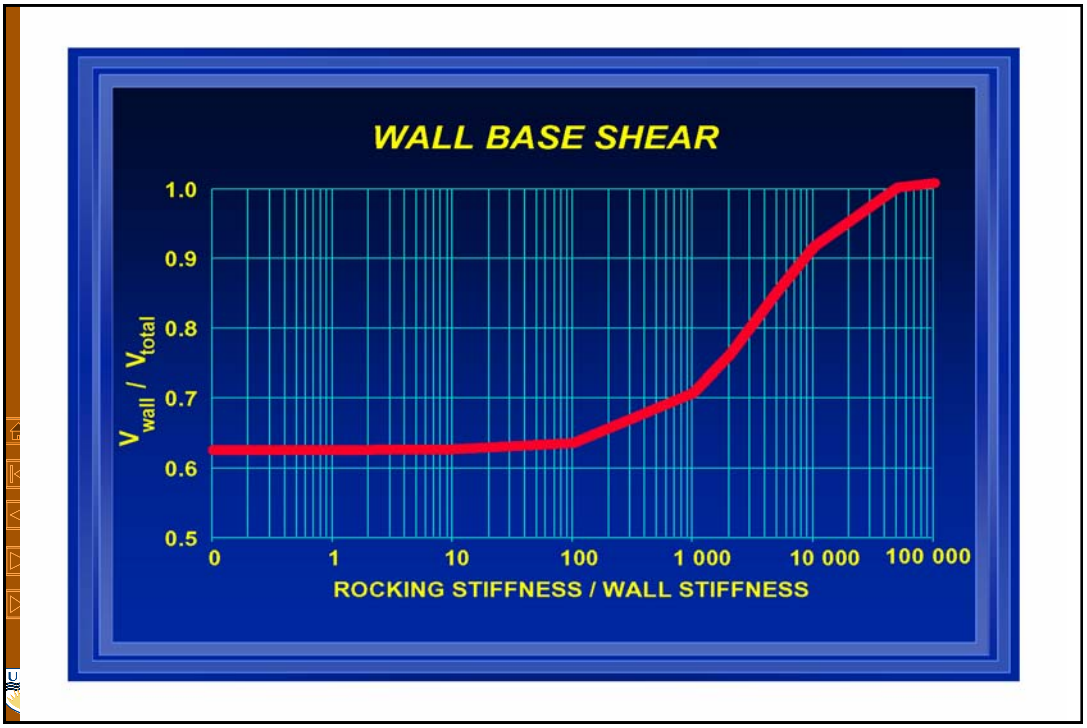
FACTORED NBCC 1995 WIND LOADS vs 2005 EQ VANCOUVER

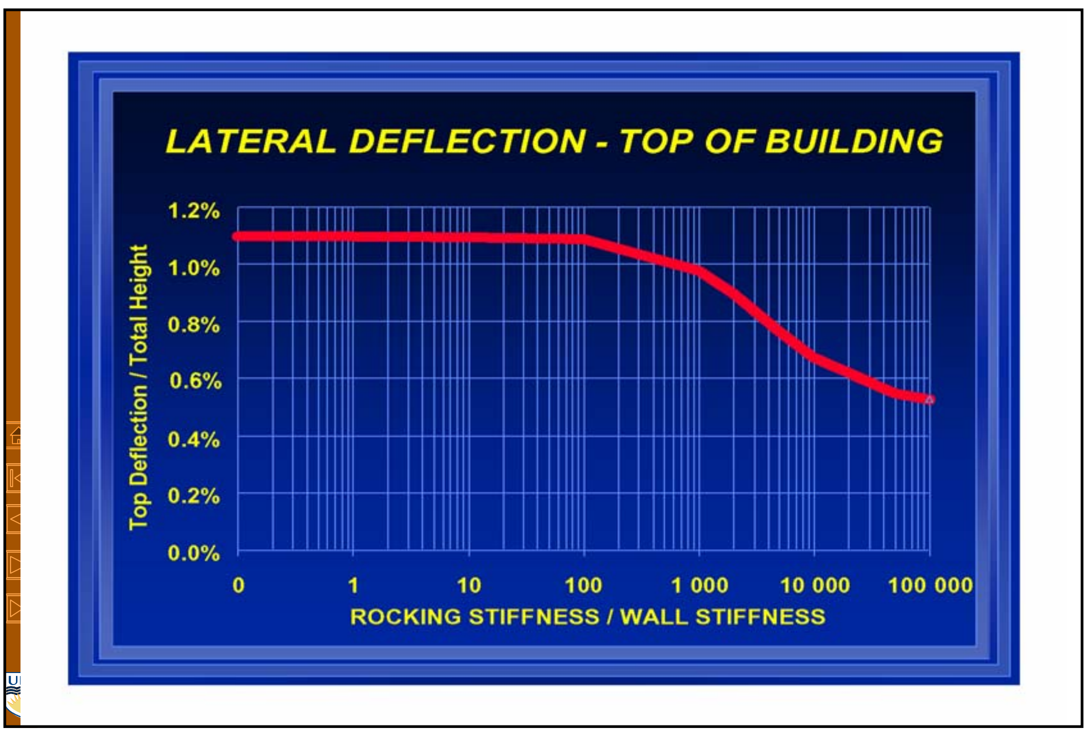
	Shear		Moment	
	EQ	Wind	EQ	Wind
Uncoupled	4,265	2,543	122,417	95,776
Coupled	2,715	2,624	88,748	99,123

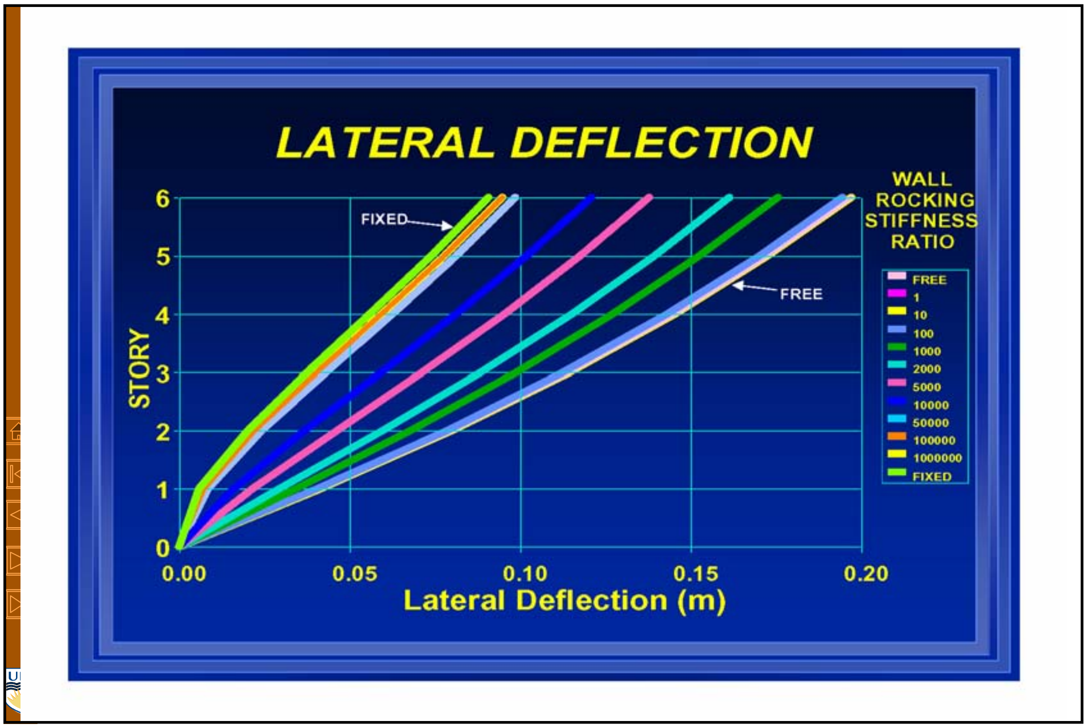
Effect of Foundation Flexibility on Earthquake Response - a brief discussion -

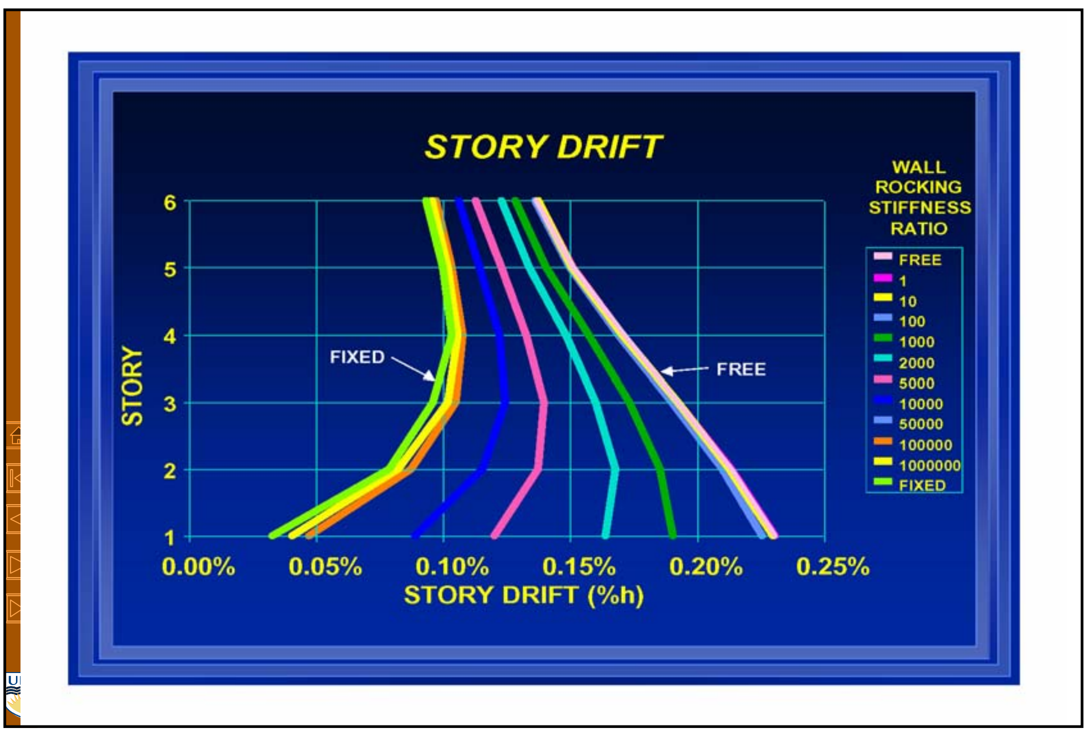

Fixed base vs. flexible foundation
2 m
Wall
3 m
3 m
3 m
3 m
3 m
3 m
Rocking
Stiffness
10 m
9 m
9 m


Definition of stiffness
1 m
P1
Infinitelly
rigid wall
Rocking
Stiffness
1 m
P2
Flexible
fixed-base
wall
Wall
Stiffness


WALL BASE SHEAR
V_{wall} / V_{total}
1.0
0.9
0.8
0.7
0.6
0.5
0
1
10
100
1 000
10 000
100 000
ROCKING STIFFNESS / WALL STIFFNESS


LATERAL DEFLECTION - TOP OF BUILDING
Top Deflection / Total Height
1.2%
1.0%
0.8%
0.6%
0.4%
0.2%
0.0%
0
1
10
100
1 000
10 000
100 000
ROCKING STIFFNESS / WALL STIFFNESS


LATERAL DEFLECTION
WALL ROCKING STIFFNESS RATIO
FREE
1
10
100
1000
2000
5000
10000
50000
100000
1000000
FIXED
FIXED
FREE
STORY
0
1
2
3
4
5
6
0.00
0.05
0.10
0.15
0.20
Lateral Deflection (m)


STORY DRIFT
WALL ROCKING STIFFNESS RATIO
FREE
1
10
100
1000
2000
5000
10000
50000
100000
1000000
FIXED
FIXED
FREE
STORY
1
2
3
4
5
6
0.00%
0.05%
0.10%
0.15%
0.20%
0.25%
STORY DRIFT (%h)

References & Notes

Anil Chopra, "Dynamics of Structures,"2nd Ed. Prentice Hall, 2001.
L.E. Garcia and M. Sozen, "Multiple Degrees of Freedom Structural Dynamics," class notes for CE571-Earthquake Engineering course at Purdue University, Spring 2003.

Notes:

Some of the slides included here were kindly provided by Dr. Mete Zosen and Dr. Luis Garcia of Purdue University and by Dr. Robert Schubak of Hooley & Schubak Ltd.

Notice

The end