# Lecture 6
# Design of Concrete structures in accordance with CSA A23.3-04

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## 2005 NBCC Seismic Design Loads

**4.1.8.11. Equivalent Static Force Procedure for Structures Satisfying the Conditions of Article 4.1.8.6.**

**1)** The static loading due to earthquake motion shall be determined according to the procedures given in this Article.

**2)** The minimum lateral earthquake force, V, shall be calculated using the following formula:

$$V = S(T_a)M_vI_EW/(R_dR_o)$$

except that V shall not be less than

$$S(2.0)M_vI_EW/(R_dR_o)$$

and for an SFRS with an $R_d$ equal to or greater than 1.5, V need not be greater than

$$\frac{2}{3}S(0.2)I_EW/(R_dR_o)$$

Where:

- S = design spectral response acceleration
- $T_a$ = fundamental period of vibration of the building
- $M_v$ = factor to account for higher mode effects
- $I_E$ = earthquake importance factor
- W = Weight of the building
- $R_d$ = ductility-related force modification factor
- $R_o$ = Overstrength-related force modification factor

# 2005 NBCC Seismic Design Loads

**V= S(Ta) Mv IE W / (RdRo)**

**6)** The design spectral acceleration values of S(T) shall be determined as follows, using linear interpolation for intermediate values of T:

$S(T) = F_aS_a(0.2)$ for $T \le 0.2$ s
$= F_vS_a(0.5)$ or $F_aS_a(0.2)$, whichever is smaller for $T = 0.5$ s
$= F_vS_a(1.0)$ for $T = 1.0$ s
$= F_vS_a(2.0)$ for $T = 2.0$ s
$= F_vS_a(2.0)/2$ for $T \ge 4.0$ s

**Where:**

**$F_a$ = Acceleration based site coefficient** **$F_v$ = Velocity based site coefficient**

**$S_a$ = 5% damped spectral response acceleration (ratio of gravity) varies with region and building period**





# 2005 NBCC Seismic Design Loads

**V= S(Ta) Mv IE W / (RdRo)**

**Table C-2 (Continued)**

| Province and Location | Elev., m | Design Temperature | | | | Degree-Days Below 18°C | 15 Min. Rain, mm | One Day Rain, 1/50, mm | Ann. Rain, mm | Moist. Index | Ann. Tot. Ppn., mm | Driving Rain Wind Pressures, Pa, 1/5 | Snow Load, kPa, 1/50 | | Hourly Wind Pressures, kPa | | Seismic Data[1] | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | January | | July 2.5% | | | | | | | | | | | | | | | | | |
| | | 2.5% °C | 1% °C | Dry °C | Wet °C | | | | | | | | $S_s$ | $S_r$ | 1/10 | 1/50 | $S_a(0.2)$ | $S_a(0.5)$ | $S_a(1.0)$ | $S_a(2.0)$ | PGA |
| **Alberta** | | | | | | | | | | | | | | | | | | | | | |
| Athabasca | 515 | -35 | -38 | 28 | 19 | 6000 | 18 | 86 | 370 | 0.58 | 480 | 80 | 1.5 | 0.1 | 0.30 | 0.40 | 0.12 | 0.056 | 0.023 | 0.006 | 0.059 |
| Banff | 1400 | -30 | -32 | 27 | 17 | 5500 | 18 | 65 | 300 | 0.58 | 500 | 120 | 3.6 | 0.1 | 0.39 | 0.48 | 0.24 | 0.14 | 0.066 | 0.037 | 0.12 |
| Barrhead | 645 | -34 | -37 | 28 | 19 | 6000 | 20 | 86 | 375 | 0.58 | 475 | 100 | 1.7 | 0.1 | 0.32 | 0.44 | 0.12 | 0.056 | 0.023 | 0.006 | 0.059 |
| Beaverlodge | 730 | -35 | -38 | 28 | 18 | 5900 | 25 | 92 | 315 | 0.49 | 470 | 100 | 2.4 | 0.1 | 0.27 | 0.36 | 0.13 | 0.078 | 0.039 | 0.022 | 0.070 |
| Brooks | 760 | -32 | -34 | 32 | 19 | 5200 | 18 | 86 | 260 | 0.26 | 340 | 220 | 1.2 | 0.1 | 0.39 | 0.52 | 0.12 | 0.056 | 0.023 | 0.012 | 0.059 |
| Calgary | 1045 | -31 | -33 | 29 | 17 | 5200 | 23 | 103 | 325 | 0.37 | 425 | 220 | 1.1 | 0.1 | 0.40 | 0.50 | 0.15 | 0.084 | 0.041 | 0.023 | 0.088 |
| Campsie | 660 | -34 | -37 | 28 | 19 | 6000 | 20 | 86 | 375 | 0.58 | 475 | 100 | 1.7 | 0.1 | 0.32 | 0.44 | 0.12 | 0.056 | 0.023 | 0.009 | 0.059 |
| Camrose | 740 | -33 | -35 | 29 | 19 | 5700 | 20 | 92 | 355 | 0.54 | 470 | 160 | 2.0 | 0.1 | 0.30 | 0.40 | 0.12 | 0.056 | 0.023 | 0.007 | 0.059 |
| Cardston | 1130 | -30 | -33 | 29 | 18 | 4750 | 20 | 108 | 340 | 0.38 | 550 | 140 | 1.5 | 0.1 | 0.74 | 1.02 | 0.18 | 0.11 | 0.054 | 0.031 | 0.095 |
| Claresholm | 1030 | -31 | -34 | 29 | 18 | 4800 | 15 | 103 | 310 | 0.35 | 440 | 200 | 1.3 | 0.1 | 0.66 | 0.87 | 0.15 | 0.092 | 0.046 | 0.027 | 0.092 |
| Cold Lake | 540 | -36 | -38 | 28 | 20 | 6100 | 15 | 81 | 320 | 0.53 | 430 | 140 | 1.7 | 0.1 | 0.31 | 0.40 | 0.12 | 0.056 | 0.023 | 0.006 | 0.059 |
| Coleman | 1320 | -31 | -34 | 28 | 18 | 5300 | 15 | 76 | 400 | 0.46 | 550 | 120 | 2.7 | 0.3 | 0.54 | 0.76 | 0.24 | 0.13 | 0.066 | 0.037 | 0.12 |
| Coronation | 790 | -31 | -33 | 30 | 19 | 5800 | 20 | 92 | 300 | 0.45 | 400 | 200 | 2.2 | 0.1 | 0.23 | 0.37 | 0.12 | 0.056 | 0.023 | 0.006 | 0.059 |
| Cowley | 1175 | -31 | -34 | 29 | 18 | 5100 | 15 | 81 | 310 | 0.36 | 525 | 140 | 1.6 | 0.1 | 0.73 | 1.00 | 0.20 | 0.12 | 0.057 | 0.033 | 0.10 |
| Drumheller | 685 | -31 | -33 | 29 | 18 | 5300 | 20 | 86 | 300 | 0.39 | 375 | 220 | 1.2 | 0.1 | 0.32 | 0.44 | 0.12 | 0.056 | 0.023 | 0.012 | 0.059 |
| B.C. | 3 | -6 | -8 | 27 | 19 | 3000 | 10 | 80 | 1000 | 1.14 | 1050 | 160 | 1.3 | 0.2 | 0.37 | 0.49 | 1.1 | 0.73 | 0.35 | 0.18 | 0.54 |
| | 15 | -8 | -10 | 29 | 20 | 3100 | 8 | 112 | 1450 | 1.53 | 1500 | 160 | 2.4 | 0.2 | 0.36 | 0.47 | 1.1 | 0.71 | 0.33 | 0.17 | 0.53 |
| New Westminster | 10 | -8 | -10 | 29 | 19 | 2950 | 10 | 134 | 1500 | 1.59 | 1575 | 160 | 2.3 | 0.2 | 0.36 | 0.47 | 0.99 | 0.66 | 0.33 | 0.17 | 0.49 |
| North Vancouver | 135 | -7 | -9 | 26 | 19 | 3000 | 10 | 150 | 2000 | 2.07 | 2100 | 160 | 3.0 | 0.3 | 0.36 | 0.48 | 0.88 | 0.61 | 0.33 | 0.17 | 0.44 |
| Richmond | 5 | -7 | -9 | 27 | 19 | 3000 | 8 | 86 | 1070 | 1.20 | 1100 | 160 | 1.5 | 0.2 | 0.36 | 0.48 | 1.0 | 0.68 | 0.34 | 0.18 | 0.50 |
| Surrey (88 Ave & 156 St.) | 90 | -8 | -10 | 29 | 20 | 3050 | 10 | 128 | 1500 | 1.58 | 1575 | 160 | 2.4 | 0.3 | 0.36 | 0.47 | 1.0 | 0.69 | 0.33 | 0.17 | 0.51 |
| Vancouver | 15 | -7 | -9 | 26 | 19 | 2925 | 10 | 112 | 1325 | 1.44 | 1400 | 160 | 1.8 | 0.2 | 0.36 | 0.48 | 0.94 | 0.64 | 0.33 | 0.17 | 0.46 |
| Vancouver (Granville & 41 Ave) | 120 | -6 | -8 | 28 | 20 | 2950 | 10 | 107 | 1325 | 1.44 | 1400 | 160 | 1.9 | 0.3 | 0.36 | 0.48 | 0.95 | 0.65 | 0.34 | 0.17 | 0.47 |
| West Vancouver | 45 | -8 | -10 | 28 | 19 | 3250 | 9 | 150 | 1600 | 1.69 | 1700 | 160 | 2.4 | 0.2 | 0.36 | 0.48 | 0.88 | 0.62 | 0.33 | 0.17 | 0.43 |
| Vernon | 405 | -20 | -23 | 33 | 20 | 3900 | 13 | 43 | 350 | 0.41 | 400 | 80 | 2.2 | 0.1 | 0.32 | 0.44 | 0.27 | 0.17 | 0.083 | 0.047 | 0.14 |
| Victoria Region | | | | | | | | | | | | | | | | | | | | | |

# 2005 NBCC Seismic Design Loads

**V= S(Ta) Mv IE W / (RdRo)**

**Table 4.1.8.4.A.**
**Site Classification for Seismic Site Response**
Forming Part of Sentences 4.1.8.4.(2) and (3)

| Site Class | Ground Profile Name | Average Properties in Top 30 m, as per Appendix A | | |
|---|---|---|---|---|
| | | Average Shear Wave Velocity, $\bar{V}_s$ (m/s) | Average Standard Penetration Resistance, $\bar{N}_{60}$ | *Soil* Undrained Shear Strength, $s_u$ |
| A | Hard *rock* | $\bar{V}_s > 1500$ | n/a | n/a |
| B | *Rock* | $760 < \bar{V}_s \leq 1500$ | n/a | n/a |
| C | Very dense *soil* and soft *rock* | $360 < \bar{V}_s < 760$ | $\bar{N}_{60} > 50$ | $s_u > 100$ kPa |
| D | Stiff *soil* | $180 < \bar{V}_s < 360$ | $15 \leq \bar{N}_{60} \leq 50$ | 50 kPa $< s_u \leq 100$ kPa |
| E | Soft *soil* | $\bar{V}_s < 180$ | $\bar{N}_{60} < 15$ | $s_u < 50$ kPa |
| | | Any profile with more than 3 m of *soil* with the following characteristics:<br>• plasticity index: PI > 20<br>• moisture content: $w \geq 40\%$, and<br>• undrained shear strength: $s_u < 25$ kPa | | |
| F | Other *soils*(1) | Site-specific evaluation required | | |

*Seismic Design of Multistorey Concrete Structures*





# 2005 NBCC Seismic Design Loads

**V= S(Ta) Mv IE W / (RdRo)**

**Vancouver**

Sa(0.2) = 0.94
Sa(0.5) = 0.64
Sa(1.0) = 0.33
Sa(2.0) = 0.17

**By Interpolating**
Site Class A Fa = 0.8
Site Class E Fa = 1.0
Site Class A Fv = 0.5
Site Class E Fv = 1.8

**Table 4.1.8.4.B.**
**Values of $F_a$ as a Function of Site Class and $S_a(0.2)$**
Forming Part of Sentence 4.1.8.4.(4)

| Site Class | Values of $F_a$ | | | | |
|---|---|---|---|---|---|
| | $S_a(0.2) \leq 0.25$ | $S_a(0.2) = 0.50$ | $S_a(0.2) = 0.75$ | $S_a(0.2) = 1.00$ | $S_a(0.2) \geq 1.25$ |
| 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 | (1) | (1) | (1) | (1) | (1) |

**Calgary**

Sa(0.2) = 0.15
Sa(0.5) = 0.084
Sa(1.0) = 0.041
Sa(2.0) = 0.023

**By Interpolating**
Site Class A Fa = 0.7
Site Class E Fa = 2.1
Site Class A Fv = 0.5
Site Class E Fv = 2.1

**Table 4.1.8.4.C.**
**Values of $F_v$ as a Function of Site Class and $S_a(1.0)$**
Forming Part of Sentence 4.1.8.4.(4)

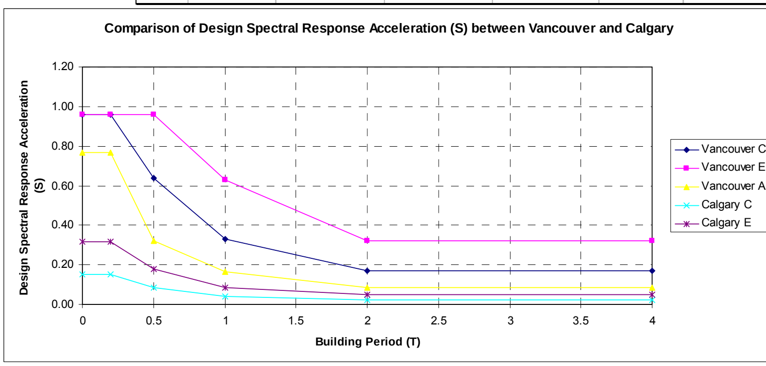
| Site Class | Values of $F_v$ | | | | |
|---|---|---|---|---|---|
| | $S_a(1.0) \leq 0.1$ | $S_a(1.0) = 0.2$ | $S_a(1.0) = 0.3$ | $S_a(1.0) = 0.4$ | $S_a(1.0) \geq 0.5$ |
| A | 0.5 | 0.5 | 0.5 | 0.6 | 0.6 |
| B | 0.6 | 0.7 | 0.7 | 0.8 | 0.8 |
| C | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| D | 1.4 | 1.3 | 1.2 | 1.1 | 1.1 |
| E | 2.1 | 2.0 | 1.9 | 1.7 | 1.7 |
| F | (1) | (1) | (1) | (1) | (1) |

*Seismic Design of Multistorey Concrete Structures*

# 2005 NBCC Seismic Design Loads

V= S(Ta) Mv IE W / (RdRo)

| Period (T) | Vancouver C | Vancouver E | Vancouver A | Calgary C | Calgary E |
|---|---|---|---|---|---|
| 0 | 0.96 | 0.96 | 0.77 | 0.15 | 0.32 |
| 0.2 | 0.96 | 0.96 | 0.77 | 0.15 | 0.32 |
| 0.5 | 0.64 | 0.96 | 0.32 | 0.08 | 0.18 |
| 1 | 0.33 | 0.63 | 0.17 | 0.04 | 0.09 |
| 2 | 0.17 | 0.32 | 0.09 | 0.02 | 0.05 |
| 4 | 0.17 | 0.32 | 0.09 | 0.02 | 0.05 |



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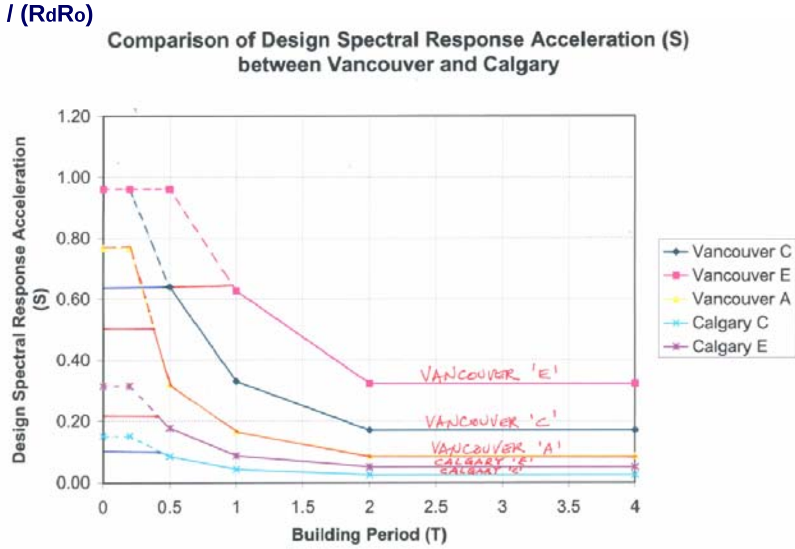
# 2005 NBCC Seismic Design Loads

V= S(Ta) Mv IE W / (RdRo)



# 2005 NBCC Seismic Design Loads

V= S(Ta) Mv IE W / (RdRo)

**Table 4.1.8.9.**
**SFRS Ductility-Related Force Modification Factors, $R_d$, Overstrength-Related Force Modification Factors, $R_o$, and General Restrictions[1]**
Forming Part of Sentence 4.1.8.9.(1)

| Type of SFRS | $R_d$ | $R_o$ | Restrictions[2] Cases Where $I_EF_aS_a(0.2)$ < 0.2 | ≥ 0.2 to < 0.35 | ≥ 0.35 to ≤ 0.75 | > 0.75 | Cases Where $I_EF_vS_a(1.0)$ > 0.3 |
|---|---|---|---|---|---|---|---|
| Concrete Structures Designed and Detailed According to CSA A23.3 | | | | | | | |
| Ductile moment-resisting frames | 4.0 | 1.7 | NL | NL | NL | NL | NL |
| Moderately ductile moment-resisting frames | 2.5 | 1.4 | NL | NL | 60 | 40 | 40 |
| Ductile coupled walls | 4.0 | 1.7 | NL | NL | NL | NL | NL |
| Ductile partially coupled walls | 3.5 | 1.7 | NL | NL | NL | NL | NL |
| Ductile shear walls | 3.5 | 1.6 | NL | NL | NL | NL | NL |
| Moderately ductile shear walls | 2.0 | 1.4 | NL | NL | NL | 60 | 60 |
| Conventional construction | | | | | | | |
| Moment-resisting frames | 1.5 | 1.3 | NL | NL | 15 | NP | NP |
| Shear walls | 1.5 | 1.3 | NL | NL | 40 | 30 | 30 |
| Other concrete SFRS(s) not listed above | 1.0 | 1.0 | 15 | 15 | NP | NP | NP |



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# 2005 NBCC Seismic Design Loads

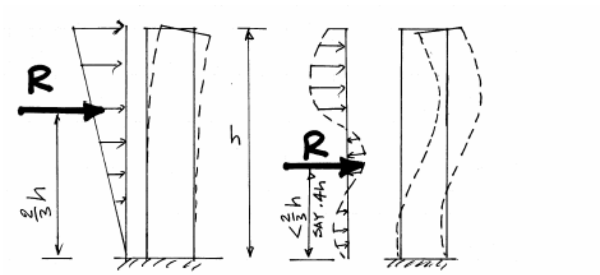
V= S(Ta) Mv IE W / (RdRo)

**Table 4.1.8.11.**
**Higher Mode Factor, $M_v$, and Base Overturning Reduction Factor, J[1][2]**
Forming Part of Sentence 4.1.8.11.(5)

| $S_a(0.2)/S_a(2.0)$ | Type of Lateral Resisting Systems | $M_v$ For $T_a \leq 1.0$ | $M_v$ For $T_a \geq 2.0$ | J For $T_a \leq 0.5$ | J For $T_a \geq 2.0$ |
|---|---|---|---|---|---|
| < 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 |
| | 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 |
| | Walls, wall-frame systems, other systems[4] | 1.0 | 2.5 | 1.0 | 0.4 |

# 2005 NBCC Seismic Design Loads

**V= S(Ta) Mv IE W / (RdRo)**




$OTM = V \cdot \frac{2}{3}h$

$V_{demand} = \frac{M_{yield}}{\frac{2}{3}h}$

$V_{demand} = 1.5 \frac{M_{yield}}{h}$

$OTM = V \cdot .4h$

$V_{demand} = \frac{M_{yield}}{.4h}$

$V_{demand} = 2.5 \frac{M_{yield}}{h}$

**Therefore, not accounting for higher mode effects is unconservative**





# 2005 NBCC Seismic Design Loads

**6)** The total lateral seismic force, V, shall be distributed such that a portion, $F_t$, shall be assumed to be concentrated at the top of the *building*, where $F_t$ is equal to 0.07 $T_a$V but need not exceed 0.25 V and may be considered as zero where the fundamental lateral period, $T_a$, does not exceed 0.7 s; the remainder, V - $F_t$, shall be distributed along the height of the *building*, including the top level, in accordance with the following formula:

$$F_x = (V - F_t)\, W_x h_x / \left( \sum_{i=1}^{n} W_i h_i \right)$$

**7)** The structure shall be designed to resist overturning effects caused by the earthquake forces determined in Sentence (6) and the overturning moment at level x, $M_x$, shall be determined using the following equation:

$$M_x = J_x \sum_{i=1}^{n} F_i (h_i - h_x)$$

where

$J_x$ = 1.0 for $h_x \geq 0.6h_n$, and
$J_x$ = J + (1 - J)($h_x$ / 0.6$h_n$) for $h_x < 0.6h_n$

where

J = base overturning moment reduction factor conforming to Table 4.1.8.11.

# 2005 NBCC Seismic Design Loads

**8)** Torsional effects that are concurrent with the effects of the forces mentioned in Sentence (6) and are caused by the following torsional moments shall be considered in the design of the structure according to Sentence (10):

a) torsional moments introduced by eccentricity between the centres of mass and resistance and their dynamic amplification, or

b) torsional moments due to accidental eccentricities.

**9)** Torsional sensitivity shall be determined by calculating the ratio $B_x$ for each level x according to the following equation for each orthogonal direction determined independently:

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

where

B = maximum of all values of $B_x$ in both orthogonal directions, except that the $B_x$ for one-*storey* penthouses with a weight less than 10% of the level below need not be considered,

$\delta_{max}$ = maximum *storey* displacement at the extreme points of the structure, at level x in the direction of the earthquake induced by the equivalent static forces acting at distances $\pm$ 0.10 $D_{nx}$ from the centres of mass at each floor, and

$\delta_{ave}$ = average of the displacements at the extreme points of the structure at level x produced by the above-mentioned forces.

**10)** Torsional effects shall be accounted for as follows:

a) for a *building* with B $\le$ 1.7, by applying torsional moments about a vertical axis at each level throughout the *building*, derived for each of the following load cases considered separately:

i) $T_x = F_x(e_x + 0.10\ D_{nx})$, and

ii) $T_x = F_x(e_x - 0.10\ D_{nx})$

where $F_x$ is the lateral force at each level determined according to Sentence (6) and where each element of the *building* is designed for the most severe effect of the above load cases, or

b) for a *building* with B > 1.7, in cases where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35, by a Dynamic Analysis Procedure as specified in Article 4.1.8.12.





# 2005 NBCC Seismic Design Loads

**4.1.8.12. Dynamic Analysis Procedure**

**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, 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 the lateral earthquake forces that cause them shall be accounted for by the following methods:

a) the static effects of torsional moments due to ($\pm$ 0.10 $D_{nx}$)$F_x$ 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 three-dimensional dynamic analysis with the centres of mass shifted by a distance of - 0.05 $D_{nx}$ and + 0.05 $D_{nx}$.

# 2005 NBCC Seismic Design Loads

**5)** The elastic base shear, $V_e$, obtained from a Linear Dynamic Analysis shall be multiplied by the importance factor, $I_E$, as determined in Article 4.1.8.5., and shall be divided by $R_dR_o$, as determined in Article 4.1.8.9., to obtain the base shear, $V_d$.

**6)** Except as required by 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 0.8 V.

**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 by 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 a value for V based on the value for $T_a$ determined in Clause 4.1.8.11.(3)(d) to obtain $V_d$ in Sentences (6) and (7).





# A23.3-04 Chapter 21 Provisions

**N21.2.1 Capacity Design**

The nonlinear analysis needed to predict the response of typical building structures under earthquake motions is usually too complex for design. **Capacity design is a simplified design approach where the designer chooses the inelastic mechanism and then provides the appropriate strengths of the elements of the concrete structure to ensure that the selected mechanism will form when the structure is deformed beyond the linear range**.

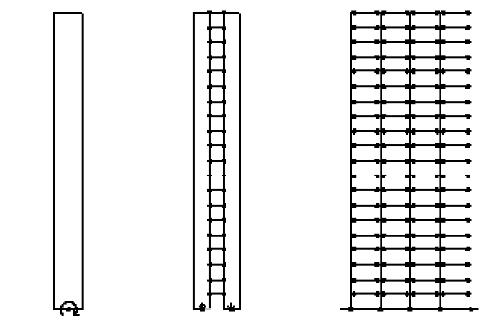
Special detailing for ductility is only required in those elements that have been selected to become inelastic. **A simple analogy that has been used to explain capacity design is a chain where all but a few links are made stronger, and when the elastic capacity of the chain is exceeded, only the weak links in the chain will become inelastic.** As long as the weak links of the chain have sufficient ductility, the complete chain will have sufficient ductility.

Key concept to understand:

In capacity design, the fusible links are flexural yielding of bending members, which are ductile. Shear failure is considered not ductile and hence should be avoided under seismic loading. The idea is to increase the shear capacity of the system such that, upon actual flexural yielding, the actual shear demand is less than the shear capacity of the system

# A23.3-04 Chapter 21 Provisions

| Type of SFRS | $R_d$ | $R_o$ |
| --- | --- | --- |
| Concrete Structures Designed and | | |
| Ductile moment-resisting frames | 4.0 | 1.7 |
| Moderately ductile moment-resisting frames | 2.5 | 1.4 |
| Ductile coupled walls | 4.0 | 1.7 |
| Ductile partially coupled walls | 3.5 | 1.7 |
| Ductile shear walls | 3.5 | 1.6 |
| Moderately ductile shear walls | 2.0 | 1.4 |
| Conventional construction | | |
| Moment-resisting frames | 1.5 | 1.3 |
| Shear walls | 1.5 | 1.3 |
| Other concrete SFRS(s) not listed above | 1.0 | 1.0 |








# Ductile Moment Resisting Frames

**21.2.5 Analysis and proportioning of structural members**

**Table 21.1**
**Section properties for analysis**

| Element type | Effective property |
| --- | --- |
| Beam | $I_e = 0.4I_g$ |
| Column | $I_e = \alpha_c I_g$ |
| Coupling beam (Clause 21.6.8.6) | $A_{ve} = 0.15A_g$ ; $Ie = 0.4I_g$ |
| Coupling beam (Clause 21.6.8.7) | $A_{ve} = 0.45A_g$ ; $Ie = 0.25I_g$ |
| Slab frame element | $I_e = 0.2I_g$ |
| Wall | $A_{xe} = \alpha_w A_g$ ; $I_e = \alpha_w I_g$ |

# Ductile Moment Resisting Frames

**21.3 Ductile moment-resisting frame members subjected to predominant flexure ($R_d$ = 4.0)**

(a) the axial compressive force in the member due to factored load effects shall not exceed $A_g f'_c/10$;

(b) the clear span of the member shall be not less than four times its effective depth;

(c) the width-to-depth ratio of the cross-section shall be not less than 0.3; and

(d) the width shall be not less than 250 mm and not more than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member not exceeding three-quarters of the depth of the flexural member.





# Ductile Moment Resisting Frames

**21.3.2 Longitudinal reinforcement**

At any section of a flexural member , the areas of top reinforcement and bottom reinforcement shall each be not less than 1.4 $b_w d/f_y$ , and the reinforcement ratio, $\rho$ , shall not exceed 0.025. At least two effectively continuous bars shall be provided at both top and bottom.

The positive moment resistance at the face of a joint shall be not less than one-half of the negative moment resistance provided at that face of the joint. Neither the negative nor the positive moment resistance at any section along the member length shall be less than one-quarter of the maximum moment resistance provided at the face of either end joint.

Lap splices of flexural reinforcement shall be permitted only if hoop reinforcement is provided over the lap length. The maximum spacing of the transverse reinforcement enclosing the lapped bars shall not exceed $d$/4 or 100 mm. Lap splices shall not be used

(a) within the joints;
(b) within a distance of 2$d$ from the face of the joint; and
(c) within a distance $d$ from any plastic hinge caused by inelastic lateral displacements.

# Ductile Moment Resisting Frames

**21.3.3 Transverse reinforcement**

Hoops shall be provided in the following regions of frame members:
(a) over a length equal to 2*d*, measured from the face of the joint; and
(b) over regions where plastic hinges can occur and for a distance *d* on either side of these hinge regions.

The first hoop shall be located not more than 50 mm from the face of a supporting member. The maximum spacing of the hoops shall not exceed
(a) *d*/4;
(b) eight times the diameter of the smallest longitudinal bars;
(c) 24 times the diameter of the hoop bars; or
(d) 300 mm.

Where hoops are not required, stirrups with seismic hooks at each end shall be spaced not more than *d*/2 throughout the length of the member.



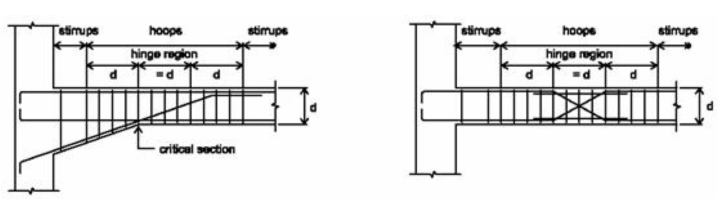


# Ductile Moment Resisting Frames

**N21.3.3 Transverse Reinforcement**

**N21.3.3.1**
These hoops are intended to prevent buckling of the longitudinal bars in the compression zone in plastic hinge regions where both the top and bottom reinforcement can be subjected to yielding in tension and compression due to reversed cyclic flexure. Bars that buckle in compression and are subsequently stressed to yield in tension usually rupture.

**N21.3.3.1(b)**
When a plastic hinge region is deliberately relocated away form the column then hoop reinforcement must be provided within and adjacent to the plastic hinge region.




(b) Hinge Due to Haunch
(a) Hinge Due to Special Reinforcement Details

# Ductile Moment Resisting Frames

1.3.4 Shear strength requirements

1.3.4.1 Design forces

he factored shear resistance of frame members shall be at least equal to the shear determined by assuming that moments equal to the probable moment resistance act at the faces of the joint so as to produce maximum shear in the member, and that the member is then loaded with the tributary transverse load along the span. The moments corresponding to probable strength shall be calculated using the properties of the member at the faces of the joint. The factored shear need not exceed that determined from factored load combinations, with load effects calculated using $R_d R_o$ equal to 1.0.

1.3.4.2 Shear reinforcement

hear reinforcement shall be designed to the requirements of Clause 11, with the following exceptions:

a) the values of $\Theta$ = 45° and $\beta$ = 0 shall be used in the regions specified in Clause 21.3.3.1; and





# Ductile Moment Resisting Frames

N21.4 Ductile Moment Resisting Frame Members Subjected to Flexural and Axial Load

N21.4.2 Minimum Flexural Resistance of Columns

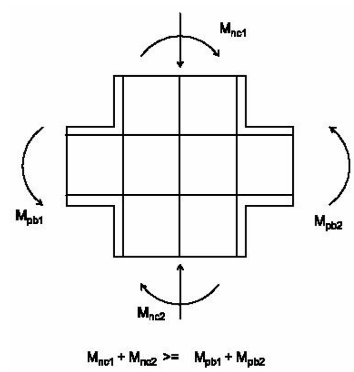
The energy dissipation necessary for a multi-storey frame to survive a severe earthquake should in general occur by the formation of ductile plastic hinges in beams (see Fig. N21.4.2(b)). Plastic hinges in beams are capable of tolerating larger rotations than hinges in columns. Further, as can be seen from Fig. N21.4.2(b), mechanisms involving beam hinges cause energy to be dissipated at many locations throughout the frame. An additional consideration is that extensive hinging in columns may critically reduce the gravity load carrying capacity of the structure.

(a) Column - Sidesway Mechanism
Undesirable

(b) Beam - Hinging Mechanism
Desirable

# Ductile Moment Resisting Frames

**N21.4.2.2**
To achieve the desired beam hinging mechanism, the Standard specifies a “strong column-weak beam” design approach. Eq. (21-3) requires that the total nominal resistance of the columns must be greater than the total probable resistance, based on $\phi_s$ = 1.25, of the beams framing into the joint.








# Ductile Moment Resisting Frames

**1.4.2 Minimum flexural resistance of columns**

$$\Sigma M_{nc} \geq \Sigma M_{pb}$$

he flexural resistances of the columns and the beams shall satisfy

where

$_{nc}$ = the sum of moments, at the centre of the joint, corresponding to the nominal resistance of the columns framing into the joint. The nominal resistance of the columns shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, that results in the lowest flexural resistance

$_{pb}$ = the sum of moments, at the centre of the joint, corresponding to the probable resistance of the beams and girders framing into that joint. In T-beam construction where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width specified in Clause 10.3 shall be assumed to contribute to flexural strength if the slab reinforcement is developed at the critical section for flexure

**1.4.3 Longitudinal reinforcement**

**1.4.3.1** The area of longitudinal reinforcement shall be not less than 0.01 or more than 0.06 times the gross area, $A_g$ , of the section

(21-6)

# Ductile Moment Resisting Frames

**1.4.4 Transverse reinforcement**

ransverse reinforcement, specified as follows, shall be provided unless a larger amount is required by Clause 21.4.4.3 or 21.4.5:

a) the volumetric ratio of circula $\rho_s = 0.4 k_p \frac{f'_c}{f_{yh}}$ nent, $\rho_s$ , shall be not less than that given by

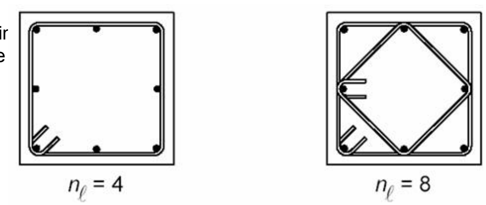
here $k_p = P_f / P_o$ and $f_{yh}$ shall not be taken as greater than 500 MPa. However, $\rho_s$ shall not be less than that required by Equation (10-7);

$$A_{sh} = 0.2 k_n k_p \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yh}} s h_c$$

ve area in each of the prir shall t $k_n = n_\ell / (n_\ell - 2)$ the

where $k_p = P_f / P_o$

$$A_{sh} = 0.09 \frac{f'_c}{f_{yh}} s h_c$$




tangular





# Ductile Moment Resisting Frames

c) transverse reinforcement may be provided by single or overlapping hoops. Crossties of the same bar size and spacing as the hoops may be used. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar; and

d) if the thickness of the concrete outside the confining transverse reinforcement exceeds 100 mm, additional transverse reinforcement shall be provided within the cover at a spacing not exceeding 300 mm.

**1.4.4.3**

ransverse reinforce $s_x = 100 + \left( \frac{350 - h_x}{3} \right)$ d at distances not exceeding the smallest of the following:

a) one-quarter of the minimum member dimension;

b) six times the diameter of the smallest longitudinal bar; or

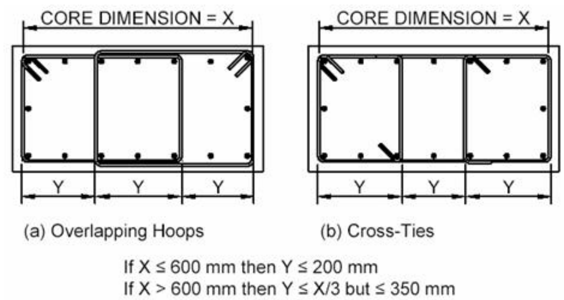
The spacing limits are intended to provide a minimum degree of confinement of the core and also to provide lateral support for the longitudinal reinforcing bars.

c) $s_x$ , as follows:

# Ductile Moment Resisting Frames

**21.4.4.4**

On each face of a column, the distance $h_x$ shall not exceed the greater of 200 mm or one-third of the core dimension in that direction, and shall not be more than 350 mm.








# Ductile Moment Resisting Frames

**1.4.4.5**

ransverse reinforcement in the amount specified in Clauses 21.4.4.1 to 21.4.4.3 shall be provided over a length, $\ell_o$, from the face of each joint and on both sides of any section where flexural yielding can occur as a result of inelastic l $P_f > 0.5\phi_c f'_c A_g$ cement of the frame. The length, $\ell_o$, shall be determined as follows:

a) where , $\ell_o$ shall be not less than 1.5 $P_f \le 0.5\phi_c f'_c A_g$ gest member cross-section dimension or one-sixth of the clear span of the member; and

b) where , $\ell_o$ shall be not less than twice the largest member cross-section dimension or one-sixth of the clear span of the member.

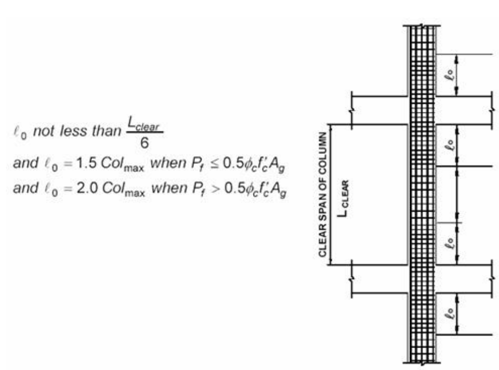



Fig. N21.4.4.5

# Ductile Moment Resisting Frames

**21.4.4.6**

Columns that can develop plastic hinges because of their connection to rigid members such as foundations or discontinued walls or because of their position at the base of the structure shall be provided with transverse reinforcement as specified in Clauses 21.4.4.1 to 21.4.4.3 over their clear height. This transverse reinforcement shall continue into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column. If the column terminates on a footing or mat, this transverse reinforcement shall extend into the footing or mat as required by Clause 21.11.

It is important to appreciate that during a severe earthquake some column hinging (e.g. at the base of the column in Fig 21.4.2(b)) and some yielding of columns will occur even if the "strong column-weak beam" philosophy has been followed. For this reason columns, need to be detailed for ductility in accordance with the requirement of Clause 21.4.4.6.





# Ductile Moment Resisting Frames

**N21.5 Joints of Ductile Moment Resisting Frames**

**N21.5.1.2**

Fig. N21.5.1.2 illustrates the procedure for determining the factored shear in joints of ductile frames. The 1.25 factor is intended to account for the likely stress in the bars when plastic hinges form in the beams.

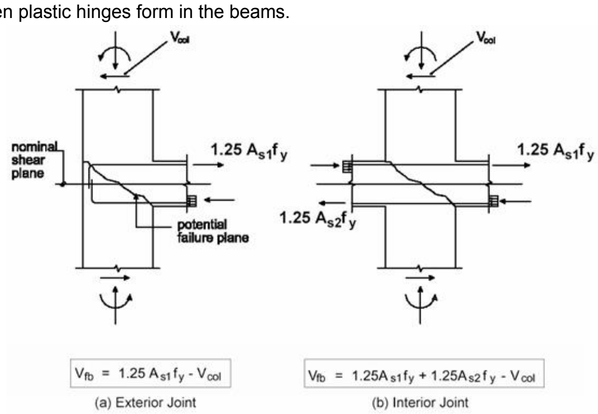



**Fig. N21.5.1.2 Determining Factored Shear Force in Joints**

# Ductile Moment Resisting Frames

**N21.5.2 Transverse Reinforcement in Joints**

**N21.5.2.1 and N21.5.2.2**
Regardless of the magnitude of the calculated shear force in a joint, confining reinforcement must be provided through the joint around the column reinforcement. This confining reinforcement may be reduced if horizontal members framing into all four sides of the joint provide sufficient external confinement.

**N21.5.3 Longitudinal Column Reinforcement**
Confinement of the joint core is provided by the cage formed from the longitudinal steel and the corresponding hoops and ties. Depending on the size of the column the maximum spacing between adjacent longitudinal bars is between 200 mm (Clause 21.5.3) and 350 mm (Clause 21.4.4.4).

**N21.5.4 Shear Resistance of Joints**
The shear force is transferred through a joint by diagonal compressive struts in the concrete acting together with tensile forces in the vertical reinforcing bars. Rather than calculating the required amount of joint shear reinforcement, the approach taken is that a joint containing transverse and longitudinal reinforcement satisfying Clauses 21.5.2 and 21.5.3 will have the factored shear resistance given in this clause.





# Ductile Moment Resisting Frames

**N21.5.5 Development Length for Tension Reinforcement in Joints**

**N21.5.5.2**
35M bars and smaller can be anchored with standard 90 deg. hooks. The development lengths for bars with $f_y$ = 400 MPa are:

| Bar Size | X (mm) | $\ell_{dh}$ (mm) $f'_c$ MPa | | | | |
|---|---|---|---|---|---|---|
| | | 20 | 25 | 30 | 35 | 40 |
| 10M | 44 | 179 | 160 | 150 | 150 | 150 |
| 15M | 64 | 268 | 240 | 219 | 203 | 190 |
| 20M | 80 | 358 | 320 | 292 | 270 | 253 |
| 25M | 100 | 447 | 400 | 365 | 338 | 316 |
| 30M | 150 | 537 | 480 | 438 | 406 | 379 |
| 35M | 216 | 626 | 560 | 511 | 473 | 443 |

These lengths were determined by considering the beneficial effects of confinement which must be present. See Fig. N21.5.5.2 for illustration of the case.

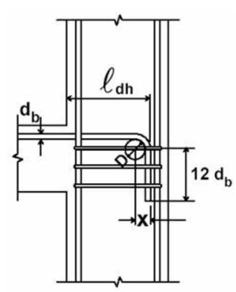



Fig. N21.5.5.2 Values of Development Lengths for Hooked Bars with $f_y$ = 400 MPa Anchored in Confined Column Cores

# Ductile Moment Resisting Frames





# Ductile Walls

**N21.6 Ductile Walls**

**N21.6.1 Application**

**N21.6.1.1**
Walls with $h_w / l_w$ of 2.0 or less are squat shear walls that are more likely to develop an inelastic shear mechanism rather than an inelastic flexural mechanism. These walls must be designed to the new requirements for squat walls contained in Clause 21.7.4.

**N21.6.1.2**
The question sometimes arises as to when a wall with openings is a solid shearwall and when it is a coupled wall. The analysis used to determine whether the elements connecting wall segments have sufficient stiffness must account for the reduced section properties given in Clause 21.2.5.2. The effective properties of the elements connecting the vertical wall segments shall be taken as those specified for coupling beams.
The elements connecting the wall segments would have adequate stiffness for the wall to act as a solid shearwall if the vertical strains in all wall segments follow essentially a single linear variation. That is, the horizontal plane section across all wall segments remains plane.

# Ductile Walls

**N21.6.2 General Requirements**

**N21.6.2.1**
It is very important that the detailing required for the plastic hinge regions of walls be provided wherever yielding may occur in walls.

**N21.6.2.2**
The length of the plastic hinge in a wall is expected to be about equal to the length of the wall. Thus the length over which special detailing must be provided is 1.5 times the length of the wall.

Due to diagonal cracking of the wall, the demand in the vertical reinforcement at the critical section will spread over a height approximately equal to the wall length. Thus the vertical reinforcement required at the critical section must be provided over at least this height. Shear failures occur along a diagonal crack that extends over a height approximately equal to the length of the wall. For these two reasons, and to prevent premature yielding above the critical section, both the vertical and horizontal reinforcement calculated for the critical section shall extend over the plastic hinge region.





# Ductile Walls

Once increased strength has been provided over the height of the plastic hinge, the factored bending moment envelope must be modified in order to prevent premature yielding above the plastic hinge region. In the previous commentary it was suggested that a linear variation in factored bending moment be used from the top of the plastic hinge region to the top of the wall. The new Standard permits a shape based on an amplified factored bending moment envelope to be used.

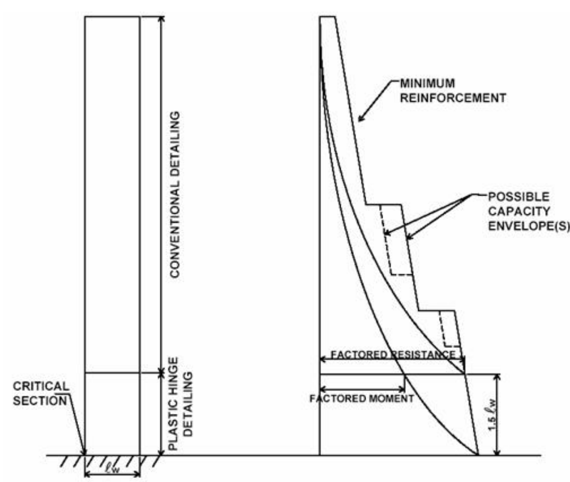



Fig. N21.6.2.2 illustrates a situation for a single wall where hinging will only form at the base of the wall. Note that a portion of the wall bending capacity is derived from the axial compression due to building dead load which reduces approximately linearly over the building height. Thus the bending capacity reduces linearly where the vertical reinforcement is uniform.

Instructor: John Pao

# Ductile Walls

[Fig. N21.2.2](#) shows that for coupled systems ductile detailing and consideration of capacity design principles are required over the height of the building. For the uniform coupled wall shown in Fig. N21.2.2 the location of the plastic hinges in the walls is limited to the base, and principles similar to those illustrated in Fig. N21.6.2.2 may be applied but must be supplemented with the additional requirements of Clauses 21.6.8.

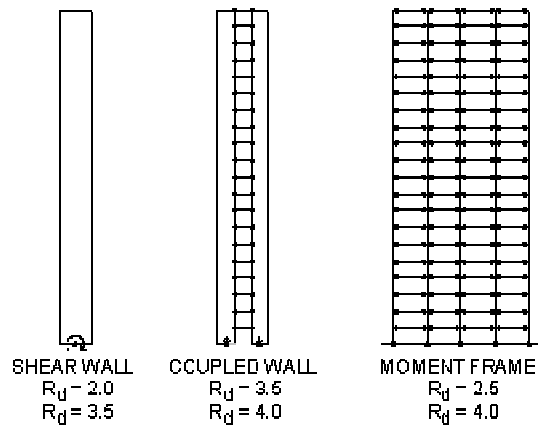



Fig. N21.2.2



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# Ductile Walls

**N21.6.3 Dimensional Limitations**

Care must be taken to prevent possible instability in potential plastic hinge zones. Regions of the walls, where yielding of the reinforcement and concrete compressive strains in excess of 0.0015 are expected, need to be checked for stability.

Fig. N21.6.3 (a) illustrates how thickened boundary elements can be used to provide stability in the highly stressed regions of the walls.

For simple rectangular walls with low axial compressions, the required depth of compression may be small enough to enable the remainder of the wall to provide sufficient restraint to the small highly compressed regions (see Fig. N21.6.3 (b)).

Walls that provide continuous lateral support of adjacent components within a distance of 3b' from a line of support is exempted from the slenderness limitation. The shaded part of the flange in Fig. N21.6.3(c) is considered to be too remote to be effectively restrained by the web portion of the wall and hence it needs to comply with the slenderness requirement.

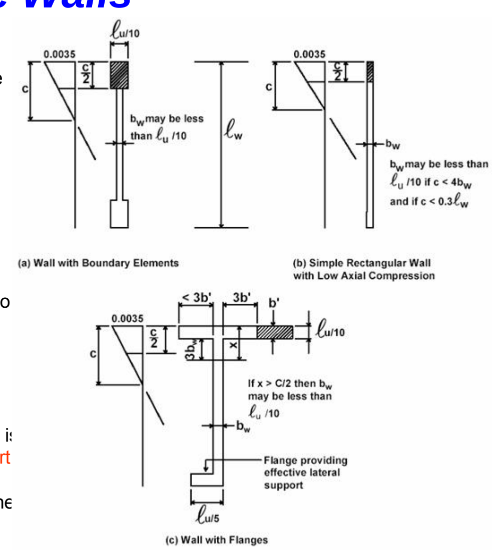



Fig. N21.6.3 Minimum Wall thickness in Plastic Hinge Regions

# Ductile Walls

**21.6.4 Reinforcement**

All lap splices shall have a minimum length of 1.5 $\ell_d$ .

The reinforcement ratio within any region of concentrated reinforcement, including regions containing lap splices, shall be not more than 0.06.

This clause is intended to avoid excessive congestion of reinforcement. The reinforcement ratio is calculated using the area of concrete surrounding the concentrated reinforcement.

Several such areas are shown shaded in Fig. N21.6.4.3. Note that this limit applies at lap splice locations.

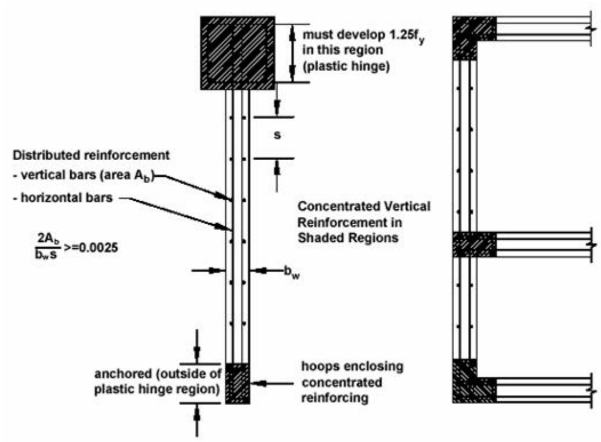



Fig. N21.6.4.3 Distributed Reinforcement and Concentrated Reinforcement





# Ductile Walls

**21.6.5 Distributed Reinforcement**

This clause introduces tie requirements for vertical distributed reinforcement similar to those introduced in Clause 14, except that in Clause 21, it applies to all vertical reinforcement.

Buckling prevention ties for vertical distributed reinforcement are required in plastic hinge regions in anticipation of reverse cyclic yielding where 20M and larger bars are used.

| | Plastic Hinge | Other Region |
|---|---|---|
| Distributed reinforcement | | |
| Amount | $\rho \geq 0.0025$ | $\rho \geq 0.0025$ |
| Spacing | $\leq$ 300 mm | $\leq$ 450 mm |
| Tying | Buckling prevention ties, Clause 21.6.6.9 | Column ties, Clause 7.6.5 |
| Horizontal reinforcement anchorage | Develop 1.25 $f_y$ within region of concentrated reinforcement | extend into region of concentrated reinforcement |
| Concentrated reinforcement | | |
| Where required | at ends of walls and coupling beams, corners, and junctions | at ends of walls and coupling beams |
| Amount* (at least 4 bars) | $A_s \geq 0.0015\ b_w \ell_w$ $A_s \leq 0.06$ x area of concentrated reinforcement region | $A_s \geq 0.001\ b_w \ell_w$ $A_s \leq 0.06$ x area of concentrated reinforcement region |
| Hoop requirements | must satisfy Clauses 7.6 and 21.6.6.9 | hoop spacing according to Clause 7.6 |
| Splice requirements | 1.5 $\ell_d$ and not more than 50% at the same location. Unless lap length less than ¼ storey height lap alternate floors | 1.5 $\ell_d$ and 100% at the same location. |

# Ductile Walls

**N21.6.6.2**
The wall moments should be resisted primarily by concentrated reinforcement. Walls designed with only distributed steel often fail by rupture of the edge tension reinforcement prior to developing significant ductility. Nevertheless when calculating the wall resistance the distributed reinforcement is to be taken into account.

**21.6.6.4**
The minimum area of concentrated reinforcement in regions of plastic hinging shall be at least 0.0015 $b_w \ell_w$ at each end of the wall

This minimum reinforcement requirement is intended to ensure that the wall possesses post-cracking capacity.

**21.6.6.7**
In regions of plastic hinging, not more than 50% of the reinforcement at each end of the walls shall be spliced at the same location. In such walls, a total of at least one-half of the height of each storey shall be completely clear of lap splices in the concentrated reinforcement.

The requirement to keep at least half the storey height free of lap splices is intended to provide a section of wall with a capacity no greater than that anticipated in the design.

**21.6.6.8**
The concentrated reinforcement shall be at least tied as a column as specified in Clause 7.6, and the ties shall be detailed as hoops. In regions of plastic hinging, the concentrated reinforcement shall be tied with buckling prevention ties as specified in Clause 21.6.6.9.

The closer spacing of ties in the plastic hinge region is intended to prevent buckling of bars under compression.





# Ductile Walls

**N21.6.7 Ductility of Ductile Shear Walls**

The concept of a "limit state" for inelastic deformations is introduced. The inelastic rotational capacity of a wall plastic hinge must be greater than the inelastic rotational demand.

The inelastic rotation demand given by Eq. (21-10) is simply the inelastic displacement (total displacement minus elastic portion) at the top of the wall divided by the distance from the centre of the plastic hinge in the wall to the top of the wall. The plastic hinge length is assumed to be equal to the length of the wall $l_w$ in Eq. (21-10). The minimum inelastic rotation demand of 0.004 is to ensure a minimum level of ductility in buildings where the predicted inelastic drift is small.

$$\theta_{id} = \frac{\left(\Delta_f R_o R_d - \Delta_f \gamma_w\right)}{\left(h_w - \frac{\ell_w}{2}\right)} \geq 0.004$$

*Design Displacement* $= \Delta_f R_o R_d$

*Elastic Displacement* $= \Delta_f \gamma_w$

*Inelastc Displacement* $= \Delta_f R_o R_d - \Delta_f \gamma_w$

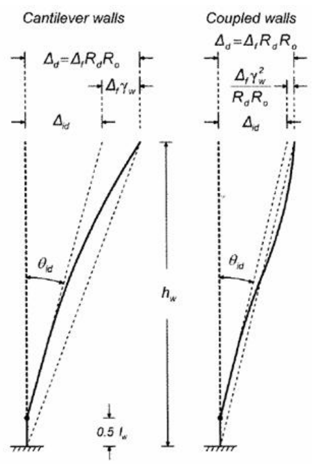



Fig. N21.6.7.2 & N21.6.8.2

# Ductile Walls

**N21.6.7.3**

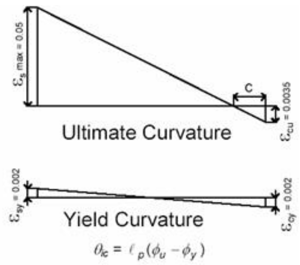
The inelastic rotation capacity given by Eq. (21-11) is equal to the total curvature capacity of the wall $\varepsilon_{cu}$ / c minus the assumed yield curvature of 0.004 / $l_w$ times the assumed plastic hinge length of $l_w$ / 2 . As the plastic hinge length varies between $l_w$ and $l_w$ / 2; to be safe the larger value was used to estimate demand and the smaller value was used to estimate inelastic rotation capacity.

$$\theta_{ic} = \left(\frac{\varepsilon_{cu}\ell_w}{2c} - 0.002\right) \le 0.025$$

$$Plastic\ \ Hinge\ Length = \frac{\ell_w}{2}$$

$$\theta_{ic} = \frac{\ell_w}{2}\left(\frac{\varepsilon_{cu}}{c} - \frac{\varepsilon_{sy} + \varepsilon_{cy}}{\ell_w}\right)$$

$$\theta_{ic\ max} = \frac{\ell_w}{2}\left(\frac{\varepsilon_{smax}}{\ell_w}\right)$$








# Ductile Walls

**N21.6.8 – Additional Requirements for Ductile Coupled and Partially Coupled Shear Walls**

**N21.6.8.1**

Ductile coupled walls and ductile partially coupled walls dissipate energy by the formation of plastic hinges in all coupling beams and near the base of the walls as shown in Figure N21.2.2.

**N21.6.8.2**

The bending moments from coupling beams cause reverse bending at the top of coupled walls. As a result, the elastic portion of the total displacement is generally much smaller in coupled walls than in shear walls without coupling beams. For simplicity, the inelastic rotation (inelastic displacement divided by height above plastic hinge) is assumed to be equal to the global drift (total displacement divided by total height of wall). That is, replacing the height of the wall above the plastic hinge with the total height of the wall compensates for the assumption that the elastic displacement is zero (See Fig N21.6.8.2). While the inelastic displacement is a larger portion of the total displacement in coupled walls, the total displacement demand is greatly reduced by the coupling beams.

# Ductile Walls

**1.6.8.4**

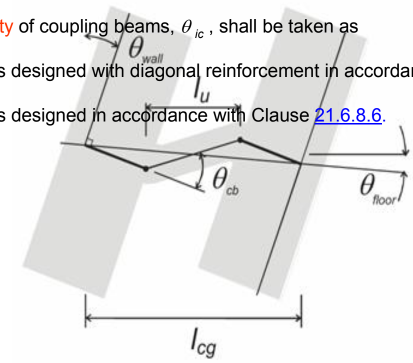
he inelastic rotational demand on coupling beams shall be taken as

$$\theta_{id} = \left(\frac{\Delta_f R_o R_d}{h_w}\right)\frac{\ell_{cg}}{\ell_u} \quad \textbf{(21-14)}$$

he inelastic rotational capacity of coupling beams, $\theta_{ic}$ , shall be taken as

a) 0.04 for coupling beams designed with diagonal reinforcement in accordance with Clause 21.6.8.7; and

b) 0.02 for coupling beams designed in accordance with Clause 21.6.8.6.






Instructor: John Pao

# Ductile Walls

**N21.6.8.5**
Both ductile coupled walls and ductile partially coupled walls require ductile coupling beams. Diagonal reinforcing must be provided in coupling beams to avoid sliding shear failure at the ends of the beams as transverse reinforcement is not effective in preventing such failures. The maximum shear stress that can be applied without causing a sliding shear failure is proportional to the slenderness of the coupling beams. Conventional reinforcement can be used if the shear stress is low and the rotational demands meet the limits in 21.6.8.4.

**N21.6.8.6(a)**
The longitudinal reinforcement in coupling beams must be capable of yielding in tension at one end and yielding in compression at the other end. Thus the beam must be long enough or the bar diameter small enough to permit the development of $2f_y$ along the length of the beam.

**N21.6.8.6(c&d)**
It is preferred that coupling beams be the same width as the wall and be centered on the wall. When this is not the case, additional design considerations are required. It is important to remember that coupling beams are expected to develop plastic hinges at the wall faces and therefore the design factored resistance of continuing elements of the coupling beams should be based on the probable resistance of the beam at the wall face when considering shear, torsion and out-of-plane bending to provide a hierarchy of capacity.

# Ductile Walls

**N21.6.8.7**

The shear and moment in diagonally reinforced coupling beams are resisted entirely by the reinforcement so the limitations on design capacity are usually dictated by the difficulty of placing the diagonal reinforcing through the wall-zone steel at the ends of the wall. Fig. N21.6.8.7 illustrates the design and detailing requirements for diagonal reinforcing. In the case where more than four bars are used in each diagonal, buckling prevention ties in addition to the outside hoop will be required. Ties on the diagonal reinforcement are not required where they pass through the inside of buckling prevention ties provided for the concentrated steel adjacent to the opening

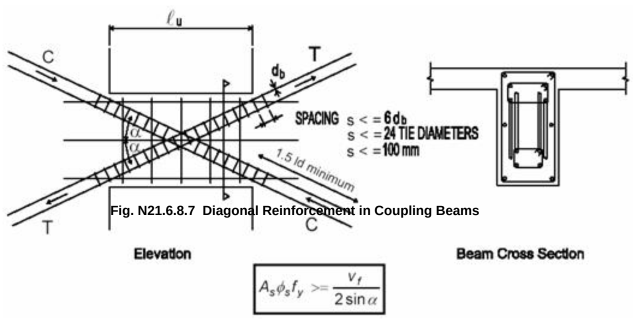



Fig. N21.6.8.7 Diagonal Reinforcement in Coupling Beams



Instructor: John Pao

# Ductile Walls

To simplify the design of coupling beams while at the same time avoiding significant over-strength, the shear forces applied to coupling beams may be redistributed from the linear-elastic distribution as illustrated in Fig. N21.6.8.7(b). The shear in any individual beam should not be reduced by more than 20% from the linear-elastic distribution nor should it be reduced below that required for other load cases such as wind. The sum of the resistance over the height of the building must be greater than or equal to the sum of the elastically determined values. For the case where forces are determined using equivalent static loading, the design coupling beam shears should not be reduced to correspond to the overturning moment calculated using the reduction factor J.

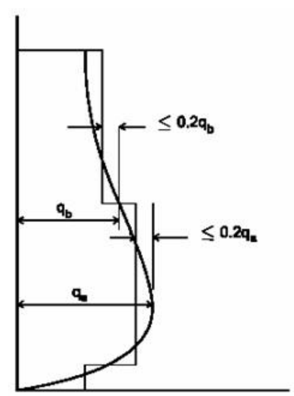



Fig. N21.6.8.7(b) Design of Coupling Beams Using Redistribution

# Ductile Walls

**N21.6.8.8**
In order to ensure that the plastic hinges form in the coupling beams and not in the walls, the wall at each end of the coupling beam must be stronger than the coupling beams framing into it. This is similar to the requirement for strong columns weak -- beams in section 21.4.2.2 for ductile frames.

**N21.6.8.9**
There are cases where the configuration of a building is such that the requirements of Clause 21.6.8.8 cannot be achieved at one end of a coupling beam. In that case, the inelastic mechanism is expected to consist of plastic hinges in the wall above and below the beam, and the wall segments must be designed as ductile moment resisting-frame elements.

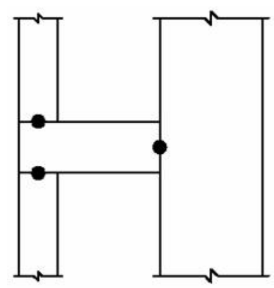


Fig. N21.6.8.9





# Ductile Walls

**21.6.8.8**

Except as permitted by Clause 21.6.8.9, walls at each end of a coupling beam shall be designed so that the factored moment resistance of the wall about its centroid, calculated using axial loads $P_s$ and $P_n$ , exceeds the moment at its centroid resulting from the nominal resistance of the coupling beams framing into the wall and the factored moment in the wall.

In order for the assumed energy dissipating mechanism to form in coupled and partially coupled walls, the coupling beams must yield. Clause 21.6.8.8 requires sufficient local bending capacity of the walls to ensure coupling beams will yield prior to the walls. In addition, the axial capacity of the walls at any height must be sufficient to resist the sum of the coupling beam shear forces required to yield the coupling beams above that height. In the previous edition of the Standard, the requirement was expressed in terms of a first-mode push-over mechanism. To account for higher mode effects, the requirement has been expressed in the current provisions as a multiplier on the factored axial forces. For a tall building, the factored axial forces would presumably be determined by a linear dynamic analysis and would account for higher mode effects.

# Ductile Walls

**1.6.8.12**

ssemblies of coupled and partially coupled shear walls connected together by coupling beams that function as a closed tube or tubes shall be designed with

a) OTM due to lateral loads resisted by axial forces in the walls, increased by the ratio of the sum of the nominal capacities to the sum of the factored forces in the coupling beams above the level under consideration; and

b) an additional increase in overturning moment resisted by axial forces in the walls at each level corresponding to the increase in the sum of the nominal capacities of the coupling beams required to resist the accidental torsion above the level under consideration.

**N21.6.8.12**

Punched tubes are often used in highrises as they provide good drift control and seismic force resistance. Tubes resist torsional loads by shear flow around the tube and through the coupling beams, as a result, there are no net induced overturning moments due to torsion in tubes, as there are in separated walls. However, that there will be local shears, moments and axial forces in the linked sections of tubes to equilibrate the shears and moments applied to them by the coupling beams for all load cases, including those that have a torsional component.

Punched tubes have shears in the coupling beams from two sources, the lateral forces and the accidental torsion. The required wall design axial load increase needed to provide this overturning moment capacity is divided into two cases, For case (a) the ratio is determined from a design with the nominal beam capacities and the factored beam forces determined for lateral load without consideration of accidental torsion. For case (b) the design for lateral and accidental torsion results in an increase in nominal beam capacities above the design for case (a) and this increase is applied as an increase in overturning moment resisted by wall axial forces. It is necessary to account for this to encourage the desirable coupling beam yield mechanism.





# Ductile Walls

**21.6.8.13**

In lieu of a more detailed assessment, wall segments that act as tension flanges in the flexural mode shall be assumed to have no shear resistance over the height of the plastic hinge. For wall assemblies carrying torsion as a tube, the shear forces in the tension flange shall be redistributed.

Shear cannot be transferred through large cracks in the tension flanges of shearwalls in the plastic hinge zone.

The tube system on the left is carrying shears due to lateral loading $V_L$ and due to torsion $V_T$. The torsional shear on the tension flange in the plastic hinge region cannot be carried across the large tension cracks. The portion of the torsion carried by the couple, $V_T$ times the distance between the tension and compression flanges must be redistributed to the perpendicular walls as illustrated in Figure N21.6.8.13(b) over the height of the plastic hinge region. Note: this redistribution changes the shears and moments in the coupling beams as well as the affected wall sections.

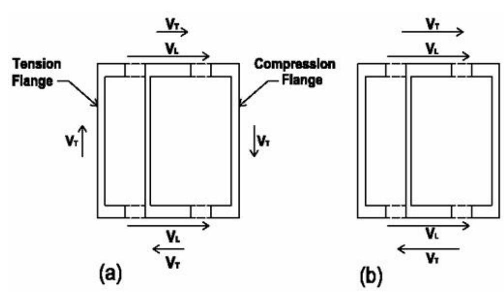



Fig. N21.6.8.13

# Ductile Walls

**21.6.9 Shear Strength of Ductile Walls**

**21.6.9.1**

Walls shall have a factored shear resistance greater than the shear due to the effects of factored loads. The shear due to the effects of factored loads shall account for the magnification of the shear due to the inelastic effects of higher modes. In addition, the factored shear resistance shall not be less than the smaller of

(a) the shear corresponding to the development of the probable moment capacity of the wall system at its plastic hinge locations; or

(b) the shear resulting from design load combinations that include earthquake, with load effects calculated using $R_d R_o$ equal to 1.0.

A capacity design approach is applied to ensure that flexural yielding of the wall will occur prior to a shear failure. For ductile walls, the shear forces determined in a linear analysis are magnified by the ratio of probable moment capacity of the wall to factored moment applied to the wall. In cases where the flexural resistance of the concrete walls exceeds the linear demand, the capacity design approach is not needed since the structure will remain linear. This is the reason for the upper limit given in (b).



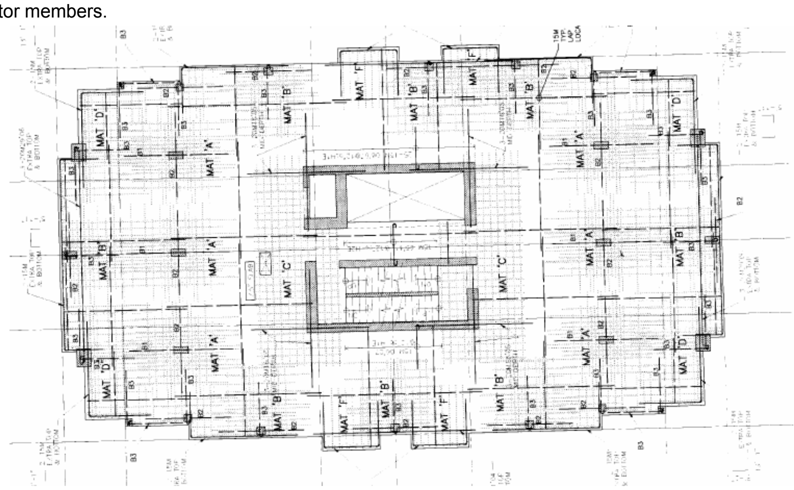
Instructor: John Pao

# Diaphragms

**21.10 Structural Diaphragms**

This is a new clause in this edition of A23.3 It is to reflect the new diaphragm requirements contained in the 2005NBCC.

Legible load paths are fundamental to the design of diaphragms. Particular attention should be paid to the provision of adequate collector members.



# Foundations

**21.11 Foundations**

**21.11.1.2**

The factored resistance of the foundation system and the supports of frames or walls shall be sufficient to develop the nominal moment capacity of the frames or walls and the corresponding shears. Where the factored moment resistance of any wall or frame exceeds the required factored moment, the following shall apply:

(a) the factored resistance of unanchored footings supporting those walls or frames need not exceed the maximum factored load effects determined with loads calculated using $R_d R_o$ equal to 2.0; and

(b) where frames or walls are supported by anchored footings or elements other than foundations, the factored resistance of those elements need not exceed the maximum factored load effects determined with loads calculated using $R_d R_o$ equal to 1.0.

The intent of this clause is to provide capacity design of the foundation system. The foundation system shall be taken as all portions of the lateral load resisting system below the lowest design plastic hinge. The upper limit on foundation capacity has been introduced to cover situations where the system capacity approaches that of an elastically responding system. The difference between the anchored and un-anchored case has been introduced to recognize that there is some energy absorption in "stamping" or "rocking" footings.





# Foundations

**21.11.2.3** Concentrated wall reinforcement shall extend to the bottom of the footing, mat, or pile cap and terminate with a 90° hook.

This clause was added over concerns that designers may not be providing a complete load path from shear wall zones through the footing into the soil. Fig. N21.11.2.3 illustrates how the strut-and-tie model given in Clause 11.4 may be used to visualize the force flow within a footing. The critical area for anchorage of the vertical wall reinforcement is indicated by a circle.

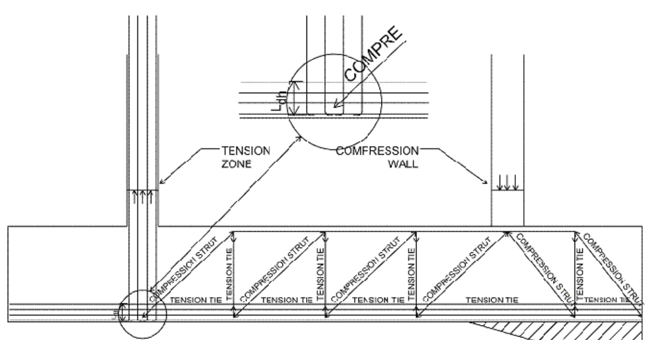
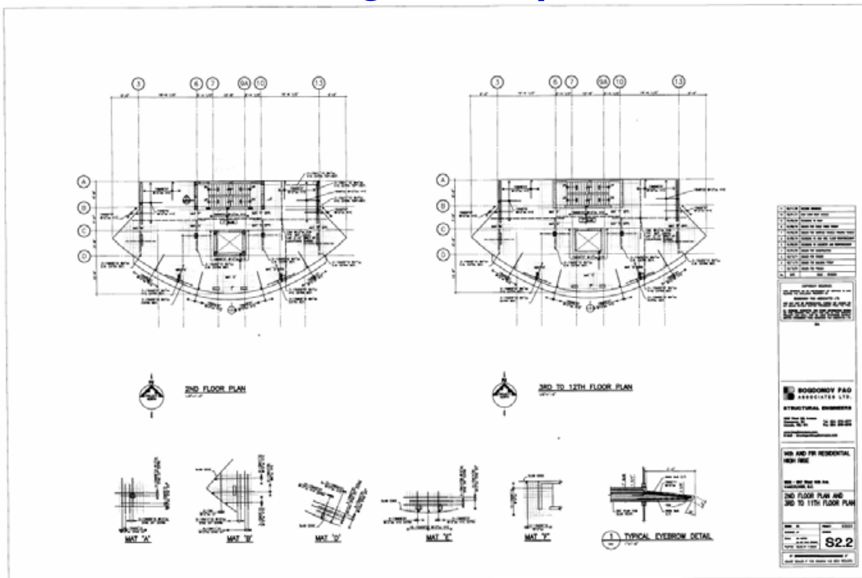



Fig. N21.11.2.3

# Design Example







# Design Example

**BOGDONOV PAO ASSOCIATES LTD.**
STRUCTURAL ENGINEERS

| | | | |
|---|---|---|---|
| Project: | 14th & FIR RESIDENTIAL HIGH RISE | Job No. | 03223 |
| | | By: | LD |
| | | Date: | April 15, 2003 |

## 1995 N.B.C. Seismic Analysis

**Seismic Data:**

v = 0.2 Za = 4 Zv = 4 F = 1 I = 1

**Building Design Values:**

X - Direction
$R_x$ = 3.5
$Ds_x$ = 19.6 ft
5.968 m
Frame Type = 3
*other*

Y - Direction
$R_y$ = 3.5
$Ds_y$ = 13.3 ft
4.063 m
Frame Type = 3
*other*

**Seismic Force Calculations:**

$h_n$ = 118.0 ft
35.976 m

Wt = 4438 kips
19738 kN

X - Direction
$T_x$ = 1.325
$S_x$ = 1.303
$FS_x$ = 1.303
$v_x$ = 0.045 Wt
$V_x$ = 198 kips
$Ft_x$ = 18 kips

Y - Direction
$T_y$ = 1.606
$S_y$ = 1.184
$FS_y$ = 1.184
$v_y$ = 0.041 Wt
$V_y$ = 180 kips
$Ft_y$ = 20 kips

**Building Forces:**

| | | | | | X - Direction | | | Y - Direction | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Level | Height (ft) | $h_i$ (ft) | $W_i$ (kips) | $h_i x W_i$ (kip-ft) | $F_x$ (kips) | $V_x$ (kips) | $M_x$ (kip-ft) | $F_y$ (kips) | $V_y$ (kips) | $M_x$ (kip-ft) |
| 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 13th (roo | 13 | 118.03 | 551 | 65005 | 58 | 58 | 759 | 56 | 56 | 725 |
| 12th | 9.333 | 105.03 | 371 | 38925 | 24 | 82 | 1528 | 21 | 77 | 1445 |
| 11th | 9.333 | 95.697 | 350 | 33524 | 21 | 103 | 2489 | 18 | 95 | 2336 |
| 10th | 9.333 | 86.364 | 350 | 30254 | 19 | 122 | 3624 | 17 | 112 | 3381 |
| 9th | 9.333 | 77.031 | 350 | 26985 | 17 | 138 | 4914 | 15 | 127 | 4563 |
| 8th | 9.333 | 67.698 | 350 | 23715 | 15 | 153 | 6340 | 13 | 140 | 5867 |
| 7th | 9.333 | 58.365 | 350 | 20446 | 13 | 165 | 7884 | 11 | 151 | 7276 |
| 6th | 9.333 | 49.032 | 350 | 17176 | 11 | 176 | 9527 | 9 | 160 | 8772 |
| 5th | 9.333 | 39.699 | 350 | 13907 | 9 | 185 | 11249 | 8 | 168 | 10339 |
| 4th | 9.333 | 30.366 | 350 | 10637 | 7 | 191 | 13032 | 6 | 174 | 11960 |
| 3rd | 9.333 | 21.033 | 350 | 7368 | 5 | 196 | 14858 | 4 | 178 | 13619 |
| 2nd | 11.7 | 11.7 | 363 | 4252 | 3 | 198 | 17177 | 2 | 180 | 15725 |
| main | | | | 292194 | | | | | | |

# Design Example

Ductile Wall Design Note — Wall Identification: score1

Mf = 16796 K'
Vf = 203 K
Pd = 1306 K
Pd + Pl/2 = 1415 K

Conc f'c = 35 Mpa

Wall: b = 10 in; lw = 281 in; cover = 1 in; height = 1416 in

Quantity: 4 per zone per face
Zone Steel: 25 M @ 2 in
Field Steel: @ 12 in

Zone Steel First Location = 1.49 in
Zone Steel Second Location = 273.51 in
Field Steel Quantity = 22
Field Steel Location = 14.50 in

Run ConcCol with the appropriate data from above, then fill in the yellow input boxes below

@ Pd, Mr = 33750 K'
@ Pd + Pl/2, Mrp = 46250 K'
@ Pd + Pl/2, Cc = 13 in

**If Cc greater than either of these, see 21.5.7**

overstrength, gammaw = 2.75 0.14 gammaw lw = 108.3 0.55 lw = 154.6

Epsilon v = 0.00380
gammaw vf / phic f'c = 0.06710

FOUR WAY ITERPOLATION TABLE

CSA A23.3-94 Table 21-1
Values of BETA for walls subjected to seismic shear

Input Epsilon v in the appropiate horizontal yellow highlight box
Input gammaw vf / phic f'c in the appropiate vertical yellow highlight box
Read BETA from the blue box intersecting the two yellow input boxes

| gammaw vf/phic f'c | Average vertical strain, Epsilon v | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.00050 | 0.00075 | 0.00100 | 0.00128 | 0.00150 | 0.00160 | 0.00200 | 0.00250 | 0.00300 | 0.00380 | 0.00500 | 0.00532 | 0.01000 | 0.01250 | 0.01500 |
| 0.02500 | 0.22800 | 0.21150 | 0.19500 | 0.18212 | 0.17200 | 0.16880 | 0.15600 | 0.14450 | 0.13300 | 0.12180 | 0.10500 | 0.10282 | 0.07100 | 0.06250 | 0.05400 |
| 0.03750 | 0.22500 | 0.20900 | 0.19300 | 0.18068 | 0.17100 | 0.16780 | 0.15500 | 0.14350 | 0.13200 | 0.12060 | 0.10400 | 0.10182 | 0.07000 | 0.06175 | 0.05350 |
| 0.05000 | 0.22200 | 0.20650 | 0.19100 | 0.17924 | 0.17000 | 0.16680 | 0.15400 | 0.14250 | 0.13100 | 0.11960 | 0.10300 | 0.10082 | 0.06900 | 0.06100 | 0.05300 |
| 0.06710 | 0.21858 | 0.20376 | 0.18895 | 0.17719 | 0.16795 | 0.16488 | 0.15263 | 0.14113 | 0.12963 | 0.11843 | 0.10163 | 0.09946 | 0.06763 | 0.05963 | 0.05163 |
| 0.07500 | 0.21700 | 0.20250 | 0.18800 | 0.17624 | 0.16700 | 0.16400 | 0.15200 | 0.14050 | 0.12900 | 0.11780 | 0.10100 | 0.09882 | 0.06700 | 0.05900 | 0.05100 |
| 0.08160 | 0.21568 | 0.20131 | 0.18694 | 0.17533 | 0.16621 | 0.16321 | 0.15121 | 0.13984 | 0.12847 | 0.11727 | 0.10047 | 0.09830 | 0.06647 | 0.05821 | 0.04994 |
| 0.10000 | 0.21200 | 0.19800 | 0.18400 | 0.17280 | 0.16400 | 0.16100 | 0.14900 | 0.13800 | 0.12700 | 0.11580 | 0.09900 | 0.09682 | 0.06500 | 0.05600 | 0.04700 |
| 0.12317 | 0.20737 | 0.19429 | 0.18122 | 0.14926 | 0.12415 | 0.12875 | 0.14715 | 0.13615 | 0.12515 | 0.11395 | 0.09715 | 0.09497 | 0.06315 | 0.03329 | 0.00344 |
| 0.12500 | 0.20700 | 0.19400 | 0.18100 | 0.14740 | 0.12100 | 0.12620 | 0.14700 | 0.13600 | 0.12500 | 0.11380 | 0.09700 | 0.09482 | 0.06300 | 0.03150 | 0.00000 |
| 0.13750 | 0.20450 | 0.19175 | 0.17900 | 0.15688 | 0.13950 | 0.14070 | 0.14550 | 0.13450 | 0.12350 | 0.11250 | 0.09600 | 0.09187 | 0.03150 | 0.01575 | 0.00000 |
| 0.15000 | 0.20200 | 0.18950 | 0.17700 | 0.16636 | 0.15800 | 0.15520 | 0.14400 | 0.13300 | 0.12200 | 0.11120 | 0.09500 | 0.08892 | 0.00000 | 0.00000 | 0.00000 |

BETA from table above = 0.11843

Assuming 1-15M e.f. horizontal reinf, spacing = 24.4 in



Instructor: John Pao

19/05/2006

# Design Example

**DESIGN OF DUCTILE FLEXURAL WALL- SCORE1 (section at base)**

$MPa := 10^6 \frac{newton}{m^2}$

$Kips := 1000lbf$

$kN := 1000newton$

***Material Properties:***

- concrete strength: $fc := 35MPa$ $\phi c := 0.6$
- reinforcement: $fy := 400MPa$ $\phi s := 0.85$

***Preliminary Choose of Vertical Reinforcement***

*- wall in X direction* $lw := 281in$

$bw := 10in$

*minimum area of concentrated reinforcement (Clause 21.5.6.4)*

at plastic hinge $Asmin := 0.002 \cdot bw \cdot lw$ $Asmin = 6\,in^2$

for 25M $As1 := 500mm^2$

number of bars required: $nb := \frac{Asmin}{As1}$ $nb = 7.3$ use $nb := 8$

$As := nb \cdot As1$ $As = 4000\,mm^2$

*maximum concentrated reinforcement (Clause 21.5.4.3)*

area of concentrated reinforcement region: $lconc := 16in$

$bconc := 12in$

$Asmax := 0.06 \cdot lconc \cdot bconc$ $Asmax = 7432\,mm^2$

$check := if(As \le Asmax, "OK", "NG")$ **check = "OK"**

*- wall in Y direction:* $lw := 114in$

$bw := 12in$

*minimum area of concentrated reinforcement (Clause 21.5.6.4)*

at plastic hinge $Asmin := 0.002 \cdot bw \cdot lw$ $Asmin = 1765\,mm^2$

for 15M $As1 := 200mm^2$

number of bars required: $nb := \frac{Asmin}{As1}$ $nb = 8.8$ use $nb := 10$

$As := nb \cdot As1$ $As = 2000\,mm^2$

ductil_shear_wall_design_01.mcd

Instructor: John Pao

# Design Example

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*maximum concentrated reinforcement (Clause 21.5.4.3)*

area of concentrated reinforcement region: $lconc := 12in$

$bconc := 12in$

$Asmax := 0.06 \cdot lconc \cdot bconc$ $\quad Asmax = 5574\,mm^2$

$check := if(As \le Asmax, "OK", "NG")$ $\quad$ **check = "OK"**

*maximum bar diameter (Clause 21.5.4.4)*

- wall in X direction $\quad bw := 10in$

$dbmax := \frac{bw}{10}$ $\quad$ **dbmax = 25.4 mm** $\quad$ use 25M

- wall in Y direction $\quad bw := 12in$

$dbmax := \frac{bw}{10}$ $\quad$ **dbmax = 30.5 mm**

*distributed reinforcement (Clause 21.5.5.1)*

- in plastic hinge region $\quad smax := 300mm$
- minimum distributed reinforcement in each direction $\quad \rho min := 0.0025$

- wall in X direction: $\quad bw := 10in$

assume two curtains of 15M@12in $\quad As := 2 \cdot 200mm^2$

$s := 12in$

$\rho := \frac{As}{bw \cdot s}$ $\quad \rho = 0.00517$

$check := if(\rho \ge \rho min, "OK", "NG")$

**check = "OK"** $\quad$ use 10M@12in

- wall in Y direction: $\quad bw := 12in$

assume two curtains of 15M@12in $\quad As := 2 \cdot 200mm^2$

$s := 12in$

$\rho := \frac{As}{bw \cdot s}$ $\quad \rho = 0.00431$

$check := if(\rho \ge \rho min, "OK", "NG")$

**check = "OK"** $\quad$ use 15M@12in

ductil_shear_wall_design_01.mcd



Instructor: John Pao

# Design Example

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*check if two curtains of reinforcement are required*

- wall in X direction: $\quad lw := 229.5in$ $\quad bw := 10in$

wall gross area: $\quad Acv := lw \cdot bw$

$Vxmin := 0.2 \cdot \phi c \cdot \sqrt{fc \cdot MPa} \cdot Acv$

**Vxmin = 236.3 Kips**

- wall in Y direction: $\quad lw := 114in$ $\quad bw := 12in$

wall gross area: $\quad Acv := lw \cdot bw$

$Vymin := 0.2 \cdot \phi c \cdot \sqrt{fc \cdot MPa} \cdot Acv$

**Vymin = 140.9 Kips**

**<u>SEISMIC FORCE IN X DIRECTION</u>**

- seismic forces: $\quad Mf := 229447 Kips \cdot in$ $\quad Mf = 19121\,Kips \cdot ft$

$Vf := 203Kips$

-moments at base (using ConcCol): $\quad Mr := 35625 Kips \cdot ft$

$Mn := 41250 Kips \cdot ft$

$Mp := 46721 Kips \cdot ft$

- distance to neutral axis (using ConcCol): $\quad c1 := 11in$ $\quad c2 := 17.5in$

- over strength factor used for shear design $\quad \gamma w := \frac{Mn}{Mf}$ $\quad \gamma w = 2.157$

- ductility checks:

**check if c<0.55*lw**

$c := if(c1 \le c2, c2, c1)$ $\quad c = 17.5in$

$lw := 281in$

$check := if(c \le 0.55 \cdot lw, "OK", "NG")$

**check = "OK"**

ductil_shear_wall_design_01.mcd

ıstructor: John Pao

# Design Example

19/05/2006

**check if c<0.14*γw*lw**

$check := if(c \le 0.14 \cdot \gamma w \cdot lw, \text{"OK"}, \text{"special wall confinement required"})$

**$check = \text{"OK"}$**

- check wall stability:

  wall thickness : $bw := 10in$

  floor to floor elevation: $hf := 9.33ft$

  slab thickness: $ts := 7in$

  unsupported wall length at base: $hu := hf - ts$ $\quad hu = 105\ in$

  $bw\_min := \frac{hu}{10}$ $\quad$ **$bw\_min = 10.5\ in$**

- confinement of concentrated reinforcement:

  in plastic hinge regions, the hoop spacing shall not exceed:

  for 25M vertical bars $\quad db := 25mm \quad s1 := 6 \cdot db \quad s1 := 5.9in$

  for 10M ties $\quad dt := 10mm \quad s2 := 24 \cdot dt \quad s2 = 9.4\ in$

  for walls thickness of $\quad bw := 10in \quad s3 := \frac{bw}{2} \quad s3 = 5\ in$

  use 10M @ $\quad s\_hoop := 5in$

<u>DESIGN FOR SHEAR AT BASE OF WALL</u>

- design base shear @ $\quad Vx := \frac{Mp}{Mf} \cdot Vf \quad$ **$Vx = 496\ Kips$**

  $Vxmin = 236.3\ Kips$ (see previous calculations)

  $check := if(Vx \le Vxmin, \text{"use one curtain"}, \text{"use two curtains"})$

  **$check = \text{"use two curtains"}$**

- effective shear wall depth: $dv := 261in$

- total wall height: $hw := 118ft$

- average vertical strain: $\quad \varepsilon v := \frac{0.001}{\gamma w} \cdot \left(0.5 - \frac{c}{lw}\right)\left(18 + \frac{hw}{lw}\right) \quad$ **$\varepsilon v = 0.004675$**

- factored shear stress: $\quad vf := \frac{Vx}{bw \cdot dv} \quad vf = 190\ psi$

ductil_shear_wall_design_01.mcd



Instructor: John Pao

# Design Example

19/05/2006

**$\frac{vf}{\phi c \cdot fc} = 0.0624$**

- from the table 21-1 $\quad \beta := 0.10634$

- shear capacity taken by concrete: $\quad Vcg := 1.3 \cdot \phi c \cdot \beta \cdot \sqrt{fc \cdot MPa} \cdot bw \cdot dv \quad Vcg = 185.8\ Kips$

- for assumed minimum horizontal shear reinforcement 15M@ each face, $\quad Av := 2 \cdot 200mm^2$

- shear capacity taken by steel $\quad s\_max := \frac{\phi s \cdot Av \cdot fy}{\frac{Vx}{dv} - 1.3 \cdot \phi c \cdot \beta \cdot \sqrt{fc \cdot MPa} \cdot bw} \quad$ **$s\_max = 25.7\ in$**

- for a wall length that is reduced at base $\quad dv\_reduced := 184in$

- maximum space for horizontal reinforcement: $\quad S\_max := \frac{dv\_reduced}{dv} \cdot s\_max \quad$ **$S\_max = 18.1\ in$**

**<u>SEISMIC FORCE IN Y DIRECTION ( for two walls )</u>**

- seismic forces: $\quad Mfx := 41907Kips \cdot in \quad Mfx = 3492.2\ Kips \cdot ft$

  $Mfy := 64956Kips \cdot in$

  $Vfx := 44.9Kips \quad Mfy = 5413\ Kips \cdot ft$

  $Vfy := 83Kips$

  assume that the maximum uniaxial bending moment is

  $Mf := \sqrt{Mfx^2 + Mfy^2} \quad Mf = 77301.2\ Kips \cdot in$

  $Vf := \sqrt{Vfx^2 + Vfy^2} \quad Vf = 94.4\ Kips$

-moments at base (using ConcCol): $\quad Mr := 7750Kips \cdot ft$

  $Mn := 19630Kips \cdot ft$

  $Mp := 21500Kips \cdot ft$

- distance to neutral axis (using ConcCol): $\quad c1 := 5.6in \quad c2 := 50in$
  (for service dead & live load)

ductil_shear_wall_design_01.mcd

# Design Example

- over strength factor used for shear design $\gamma_{wx} := \frac{Mn}{Mf}$ $\gamma_{wx} = 3.047$

***Note:*** since we have case with biaxial bending, based on A23.3-94,N21.5.7, it is conservative to assume very low $\gamma_w$ value equivalent to $1/\phi_s = 1.18$

use $\gamma_w := 1.18$

- ductility checks (Clause 21.5.7):

**check if c<0.55*lw**

$c := \mathrm{if}(c1 \le c2, c2, c1)$ $c = 50\,in$

$lw := 114in$

check := if(c ≤ 0.55·lw,"OK","NG")

**check = "OK"**

**check if c<0.14*yw*lw**

check := if(c ≤ 0.14·yw·lw,"OK","special wall confinement required")

**check = "special wall confinement required"**

- special wall confinement as for columns (Clause 21.4.4) is required over the minimum length of:

$L\_conf := c\cdot\left(0.25 + \frac{c}{lw}\right)$ $L\_conf = 34.4\,in$

- check wall stability:

wall thickness : $bw := 12in$

floor to floor elevation: $hf := 9.33ft$

slab thickness: $ts := 7in$

unsupported wall length at base: $hu := hf - ts$ $hu = 105\,in$

$bw\_min := \frac{hu}{10}$ $\mathbf{bw\_min = 10.5\,in}$

- confinement of concentrated reinforcement:

in plastic hinge regions, the hoop spacing shall not exceed:

for 15M vertical bars $db := 15mm$ $s1 := 6\cdot db$ $s1 := 5.9in$

for 10M ties $dt := 10mm$ $s2 := 24\cdot dt$ $s2 = 9.4\,in$

for walls thickness of $bw := 12in$ $s3 := \frac{bw}{2}$ $s3 = 6\,in$

use 10M @ $s\_hoop := 5in$

ductil_shear_wall_design_01.mcd





# Design Example

***Clause 21.4.4 Transverse Reinforcement in Ductile Frames***

confined gross area $Ag := 12in\cdot 34in$

core dimensions $hcx := 32in$ $hcy := 10in$

confined core area $Ach := hcx\cdot hcy$

-maximum transverse reinforcement spacing (Clause 21.4.4.3)

-minimum column dimension $bw := 12in$ $s1 := \frac{1}{4}\cdot bw$ $s1 = 3\,in$

-at least $s2 := 100mm$

-smallest longitudinal bar diameter for 15M $db := 15mm$

$s3 := 6\cdot db$ $s3 = 3.5\,in$

-as per requirements of Clause7.6

minimum s value $s := 3in$

- maximum distance between cross ties or legs of the overlapping hoops in the direction perpendicular to the longitudinal axis of the member

$s1\_horizontal := 200mm$ $s1\_horizontal = 7.9\,in$

**or**

$s2\_horizontal := \frac{hcx}{3}$ $s2\_horizontal = 270.9\,mm$ $s2\_horizontal = 10.7\,in$

$s2\_max := 300mm$ $s2\_max = 11.8\,in$

total cross sectional area of rectangular hoop reinforcement ( not less than i) or ii) ):

confined gross area $Ag := 12in\cdot 34in$

core dimensions $hcx := 32in$ $hcy := 10in$

confined core area $Ach := hcx\cdot hcy$

assume the same tie spacing as for vertical bars $s := 4in$

*in x direction*

i ) $Ash1 := 0.3\cdot s\cdot hcx\cdot\frac{fc}{fy}\cdot\left(\frac{Ag}{Ach} - 1\right)$ $Ash1 = 596.1\,mm^2$

ii ) $Ash2 := 0.09\cdot s\cdot hcx\cdot\frac{fc}{fy}$ $Ash2 = 650.3\,mm^2$

ductil_shear_wall_design_01.mcd

# Design Example

19/05/2006

$Ash := \text{if}(Ash1 \le Ash2, Ash2, Ash1)$ $\quad$ $Ash = 650.3\,\text{mm}^2$

*in y direction*

i ) $\quad Ash1 := 0.3 \cdot s \cdot hcy \cdot \frac{fc}{fy} \cdot \left(\frac{Ag}{Ach} - 1\right)$ $\quad$ $Ash1 = 186.3\,\text{mm}^2$

ii ) $\quad Ash2 := 0.09 \cdot s \cdot hcy \cdot \frac{fc}{fy}$ $\quad$ $Ash2 = 203.2\,\text{mm}^2$

$Ash := \text{if}(Ash1 \le Ash2, Ash2, Ash1)$ $\quad$ $Ash = 203.2\,\text{mm}^2$

- for columns that can develop plastic hinge over thier full height, this reinforcement shall be provided over the entire column length (Clause 21.4.4.7.)





# Design Example

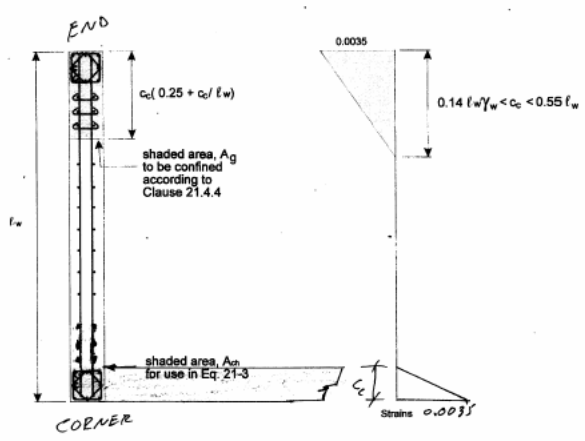
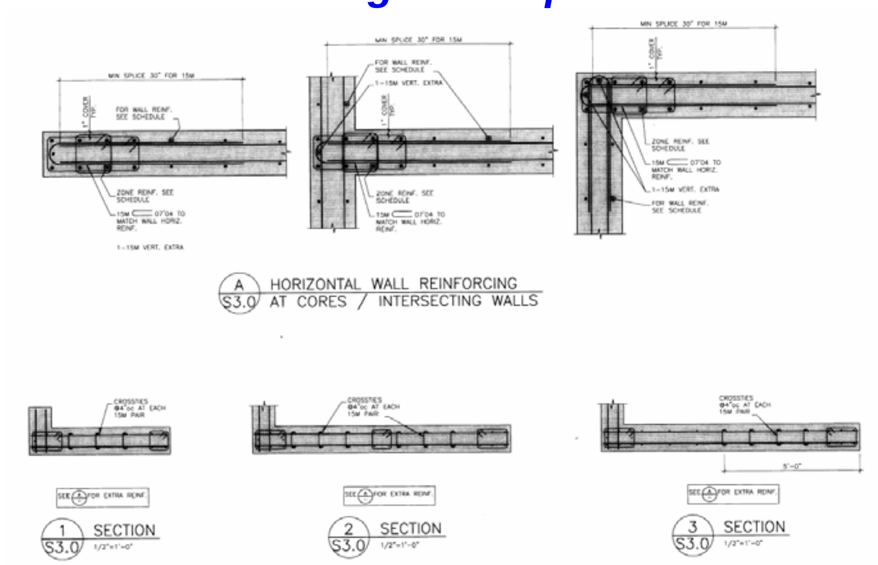



**Fig. N21.5.7**
**Confinement Requirements for Walls with Intermediate Values of $C_c$**

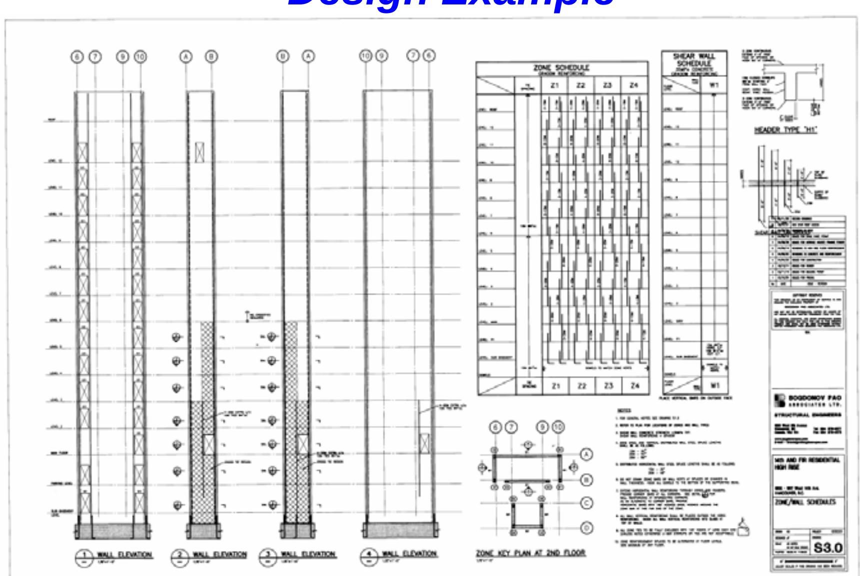
# Design Example








# Design Example



# Practical Design Guidelines

- Take the time to understand the seismic design principles and apply these principles appropriately
- Give your building a chance to survive the major earthquake by developing a reasonable concept. Focus you design on reasonable concepts rather than too many significant digits
- Remember, we don't know anything about the next major earthquake that the building is expected to resist
- We are designing for maximum acceleration without really accounting for duration
- A long duration earthquake that has lower maximum acceleration could cause more damage to the building than a short duration large acceleration EQ. Make sure the building has reasonable ductility properties.
- Don't forget that the contractor has to build the building, don't make it too complex or it would never be built to your design.
- Don't get bogged down by the numbers, start with a good concept and let the solution drive the problem, not the problem driving the solution
- Always have a perspective on your numbers, allow for the ability to do mental math
- Try and stick to maximum two significant digits, i.e. 330kN, or 5800kN, or 45,000kNm etc.
- From a seismic design perspective, the above numbers can be rounded to 300kN, or 6000kN, or 40 to 50,000kNm





# Practical Design Guidelines

- The more we understand through research, the more we need to apply the KISS principle. Keep the big picture in mind, keep in mind symmetry and repetition, a couple of extra pieces of zone steel here and there isn't going to make a difference between survival and collapse if the concept is reasonable
- Remember that the earthquake doesn't remember how you have analyzed the building, it will find any weaknesses the building has and exploit it.
- Don't convince yourself that just because you used the most sophisticated analysis and design tools that you can necessarily make a convoluted concept work. The more sophisticated tools that we have, the more common sense engineering is required to use those tools.
- Where a Structural Engineer earns his money is to work with the design team and all the stake holders to develop a logical and simple solution that will allow the building to behave well in a major earthquake and consequently save lives.
- Make sure that there is enough concrete between rebars. At least 75% of a good seismic design is proper detailing of joints and zones to ensure good ductility characteristics. The building code numbers only gets you into the range of capacity required.
- Once you determined how much system capacity you need, i.e. 1000kN or 2000kN, then you must focus on make sure that yielding will only take place in the right regions.
- Given the capacity design principles where it is important to maintain shear resistant capacity upon actual yielding, it is a good idea to round down the required moment capacity and round up the required shear capacity.
- Most importantly, in seismic design, more is not necessarily better. Optimum balance between strength and ductility is better.

## *Acknolwedgements*

Thanks to Andy Vizer of CAC for providing reference material

Reference material:

NBCC 2005

Concrete Design Handbook, Third Edition

Projects by Bogdonov Pao Associates Ltd.





# Thank You